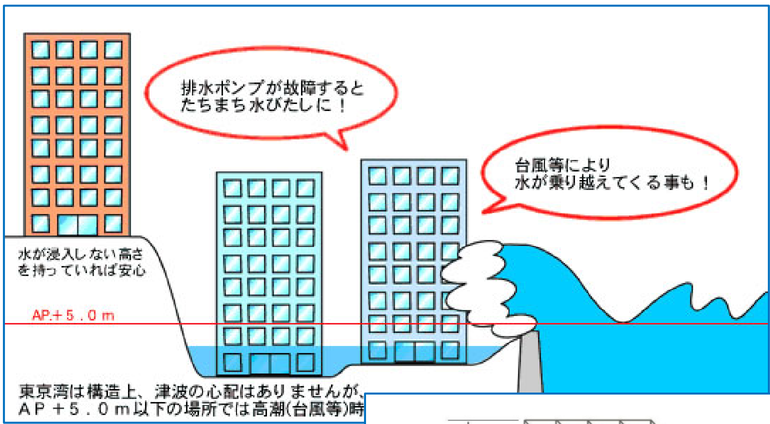
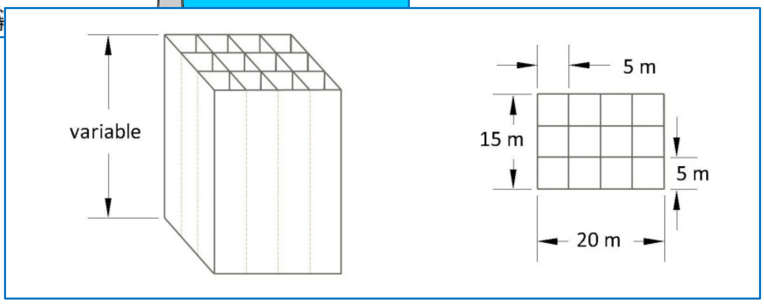
STORM SURGE BARRIER TOKYO BAY

ANALYSIS ON A SYSTEM LEVEL AND CONCEPTUAL DESIGN

MSc Thesis





María José Ruiz Fuentes

June 2014

Delft University of Technology

Challenge the future

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Student	María José Ruiz Fuentes (4230647)
Supervision	Prof. Dr. Ir. S.N. Jonkman (TU Delft) Ir. A. Van der Toorn (TU Delft) Ir. H.J. Verhagen (TU Delft) Dr. M. Esteban (University of Tokyo)

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Storm surge barrier Tokyo Bay

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α: slope

B: ship's beam

c_0: wave propagation velocity

C_{sf}: safety coefficient

c_1: pressure set-up coefficient

d: depth

d_r: discount rate

D: damage

DR: ship's draught

δ: friction angle sand-geotextile

F: fetch length

φ: wind direction

ϕ: internal friction angle

g: gravitational constant

g_r: interest rate

γ: density

h: height

h_w: water elevation

i: hydraulic gradient

l: length

L_{oa}: ship's length overall

L_{pp}: ship's length between perpendiculars

p: probability

p_r: local atmospheric pressure

$p_{r\infty}$: peripheral atmospheric pressure (typhoons)

PV: present value

q: discharge

r: ratio between tidal amplitudes inside a bay and in open sea

ρ_w: water density

t: time

T: period

v_s: wind speed

W_{bm}: width of basic or fundamental maneuvering lane

W_{wf}: additional width to account for wind forces

W_{cf}: additional width to account for current forces

W_{ym}: additional width to account for yawing motion

W_{dd}: additional width to account for drift

W_{fi}: additional width to account for interaction forces

W_{ba}: bank clearance

W_{pa}: additional width to account for two-ship interaction in passing

W_{ov}: additional width to account for two-ship interaction in overtaking

$\hat{\zeta}_k$: amplitude of sea water elevation inside a basin

$\hat{\zeta}_z$: amplitude of sea water elevation outside a basin

ω: frequency of water tidal motion

List of abbreviations

A.P.: Arakawa Peil

BC: boundary conditions

CC: climate change

IPCC: Intergovernmental Panel on Climate Change

HWL: high water level

JMA: Japan Meteorological Agency

LNG: liquefied natural gas

LWL: low water level

M: million

MWL: mean water level

SLR: sea level rise

T.P.: Tokyo Peil

VLCC: very large crude carrier

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PREFACE

This report is the result of the research carried out for my Master Thesis at Delft University of Technology. It explores the future of flood protection for Tokyo Bay. Within this context, a proposal for a storm surge barrier is made, developing a conceptual design for part of the barrier. It has been stimulating to work on a project that requires attention at different levels, starting with aspects related to decision-making and ending with a detailed design. I hope the passion I have put in my work has been transmitted to this thesis and make it a pleasant reading.

I would like to warmly thank the members of my Graduation Committee, Prof. Dr. Ir. S.N. Jonkman, Ir. A. van der Toorn, Ir. H.J. Verhagen and Dr. M. Esteban, for their guidance and valuable advice. Their enthusiasm and interest on the subject encouraged me to do my best, and their feedback was essential to give shape to this thesis.

My gratitude goes out to Vana Tsimopoulou as well, who generously shared her expertise in the field and gave me good advice over these last months.

Finally, I would like to thank my family and friends for their support. A special mention goes to my classmates and friends from TU Delft. Sharing student life with them has been an enriching and unforgettable experience.

María José Ruiz Fuentes

Delft, June 2014

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ABSTRACT

Tokyo Bay is an area exposed to storm surges generated by typhoons. Consequently lowlands along its coastline are subject to flood risk. Dike walls, levees and barriers have been developed by the Japanese to protect these coastal areas.

In the coming decades it is expected that a rise in sea level will occur. Also, climate change is expected to induce an increase in intensity of typhoons and therefore larger storm surges. These two circumstances would cause a need for the coast to be protected from higher water levels. The question arises whether the coastal defences of Tokyo Bay will be able to resist these future conditions in a satisfactory manner. In case there is a need for taking measures, a storm surge barrier could be an alternative course of action instead of dikes.

The aim of this project has been to determine whether a storm surge barrier is a good solution for the flood protection of Tokyo Bay in the future. Moreover, a proposal for a conceptual design has been developed. The design addresses the challenges that this barrier would bring, mainly the large depths and the high seismicity of the area.

Previous studies have analyzed the cost of upgrading coastal defences, considering conditions for the year 2100 and a typhoon of approximated return period of 100 years. In this report, this cost is compared to the cost of building a storm surge barrier, for the same scenario. Different locations have been studied for the barrier and the optimal one has been found to be close to the bay mouth, near cape Futtsu. The report also analyzes the convenience of using certain design conditions (sea level rise, typhoon return period). As a result, it is advised to consider as a safety standard larger return periods than the ones the Japanese have used up till now (100-200 years).

Besides the cost calculation, a multi-criteria analysis is performed in order to compare all the considered options. The result is that barriers do not surpass coastal defences upgrading, if a return period of 100 years is considered. However, in case larger design return periods are applied, as recommended here, the barrier is supposed to increase its advantages, compared to the upgrading of coastal defences. The result of these considerations is a proposal for a barrier close to the bay mouth and built for a return period of 500 years.

Once the overall design conditions are set for the hypothetical barrier, a layout is defined in order to satisfy the contemplated requirements and functions. The main requirements are flood protection (control water levels inside the bay during typhoons), navigational (allow shipping) and environmental (allow sufficient water exchange to protect the ecosystem inside the bay). The proposed barrier that fulfills these requirements is composed of a dam, a movable barrier part and a permanent opening.

Finally, a conceptual design is developed. Part of the barrier is a dam, which would cross a long and deep section of the bay. The aim is to present an innovative design that offers advantages compared to standard solutions (rubble mound section, use of geocontainers). This thesis proposes a sand-filled geotextile structure for the dam core. This design offers a good performance during the dam lifetime, at a price that makes it competitive. The geotextile structure is composed of vertical cells and is expected to offer interesting advantages, like cost savings compared to a quarry run standard solution. Also, the expected performance during its lifetime improves with respect to other geotextile technologies, such as geocontainers. The proposed design allows for compacting the filling and avoiding gaps in the core of the structure, which is expected to help the stability of the dam and reduce the damage in case an earthquake happens.

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1. INTRODUCTION

Tokyo Bay is the biggest urban area in Japan. It hosts a large population, a high number of industrial facilities and all types of business. Due to climate change effects (such as sea level rise and increase in typhoon intensity), storm surge levels are expected to be larger in the future and the existing coastal defences could be insufficient to protect this area.

In this Master's Thesis, a storm surge barrier will be studied as a solution to protect Tokyo Bay from future increased storm surge levels. The need for a barrier will be evaluated and a conceptual design will be developed.

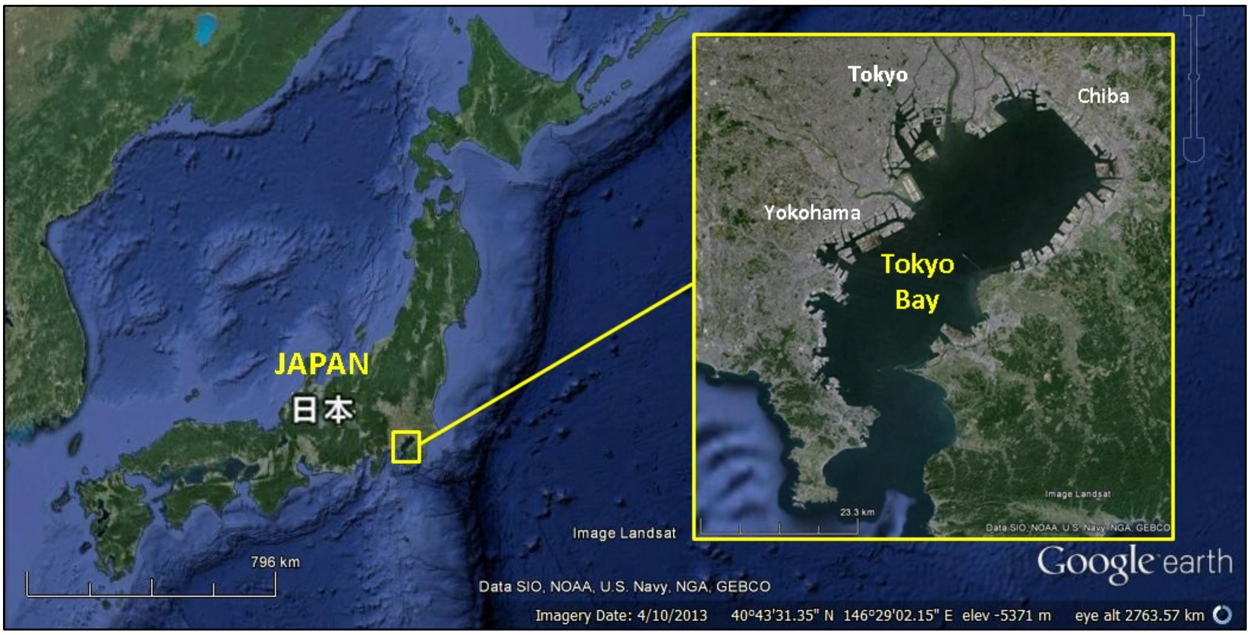

Figure 1-1: Tokyo Bay location (Google Earth, 2014).

1.1. PROBLEM STATEMENT

Due to its location, Tokyo Bay is an area notoriously prone to natural disasters. The proximity of a deep subduction zone (Japan Trench) makes the bay one of the areas with the highest earthquake intensity in the world. Besides that, Honshu Island is in the path of typhoons, which are tropical cyclones that grow in the Western part of North Pacific Ocean and move northwards later. Therefore, it is possible that major natural disasters occur. This would cause great losses since the area is densely populated and economic activity is highly concentrated there.

Regarding protection against flooding caused by typhoons, so far Japan has relied on the construction of coastal dikes, levees or walls. Also, gates and barriers have been built in rivers and navigational channels.

Studies have proven that expected sea level rise and increased typhoon intensity due to climate change may cause high water levels during storms that can surpass coastal defences elevation at some points. Therefore, existing coastal defences in Tokyo Bay might be insufficient in the future and need an upgrading (S. Hoshino, 2012). It could be necessary to take measures to keep the area protected from flooding. Upgrading coastal defences would imply the heightening of hundreds of kilometers of coastline and it represents a major investment. Therefore, a storm surge barrier appears to be an option worth to consider for protection of Tokyo Bay from flooding.

Moreover, so far Japanese policies have used the worst known event as the one for design, which for current coastal defences can be close to a return period of 100 years. Calculations for other Japan areas suggest that the optimal design return period is much larger (T. Takayama, 2004). The trend nowadays is to use large return periods for flood protection of developed areas. It is not clear which return period should be considered for this infrastructure design. This would reinforce the idea of needing measures, since not only worse conditions but more exigent safety standards could contribute to make existing coastal defences outdated.

In spite of being an appealing idea, the construction of a storm surge barrier in Tokyo Bay would face strong challenges. Typically, a storm surge barrier closure for a large length (as would be the case here), would mean to use a closure dam or wall and, at some point, a movable barrier to allow the exchange of water and navigation (see New Orleans example in Figure 1-2). In case of Tokyo Bay, the depths are larger than 60 meters in some areas, which complicates the implementation of the same solution.

Figure 1-2: New Orleans storm surge barrier (U.S. Army Corps of Engineers, 2014).

The earthquake resistance of the structure has to be guaranteed as well. This becomes difficult when large depths and material volumes are involved, as standard construction techniques and procedures can become largely expensive or simply unfeasible.

As can be seen, the design will confront the need to adapt to the challenging site conditions, overall large depths and seismicity.

1.2. OBJECTIVES

The goal of this MSc Thesis is to develop a conceptual design for a storm surge barrier in Tokyo Bay.

The research questions that will be answered are the following:

- Evaluate whether a storm surge barrier is a good solution for flood protection in Tokyo Bay in the future.
- Define the requirements that the storm surge barrier should satisfy.
- For the given the frame of reference, define an innovative barrier design, putting emphasis on reduction of construction costs and seismic performance.

The aim will be to develop a conceptual design for the closure dam that is expected to be part of the barrier**.** It will be studied how to reduce costs, specifically, by means of using a cheap filling material. In order to achieve that, the use of geosynthetics will be considered.

1.3. APPROACH

In order to answer the proposed research questions, a study of the available information and further elaboration is needed.

First of all, **Tokyo Bay** situation will be presented, along with the **problems** faced and the possible **solutions** to these problems, which include the storm surge barrier.

Later, the **need for a storm surge barrier** will be analyzed and which conditions this barrier should satisfy. A preliminary rough study of the feasibility of the barrier has been done (S. Hoshino, 2012), coming to the conclusion that a barrier could be a feasible solution compared to the high costs of upgrading coastal defences. This MSc Thesis will start from this point, using Hoshino's findings and figures.

Then, Tokyo Bay will be studied as a system, an ensemble composed of the areas to protect, the bay itself and the protection elements (coastal defences and barrier). The system will have certain given characteristics (**boundary conditions**) and others that will depend on the desired performance level (**requirements**).

An in-depth study regarding the chosen safety level is included as well. Different design return periods are analyzed with regards to costs and risk reduction. It has been considered interesting to observe the sensitivity to changes in this factor. This will be done by means of linking Hoshino's data of expected damage to return periods for water levels calculated in other studies (H. Kawai, 2008).

The solution considered by Hoshino (coastal defences upgrading) will be compared with other options, which are storm surge barriers situated in different locations. Later, for the barrier considered to be the best, a comparison of costs for different typhoon return periods will be done. To summarize, the options considered will comprise:

- different solutions (do nothing, upgrade coastal defences, storm surge barrier)
- different locations for a possible storm surge barrier
- different levels of protection (return periods) for the chosen storm surge barrier

The most suitable option will be chosen, by means of a multi-criteria analysis. This analysis will take into account the total costs of each alternative. In order to calculate total costs, investment and expected damage will be estimated for every solution. It is considered to be an interesting exercise, although it will be a simplified study (in order to adapt to the scope and extension of the thesis).

Once this analysis is done, the best alternative for storm surge barrier (in terms of location and design return period of the typhoon) will be selected for developing a detailed design. A **general layout** will be set. The dimensions of this general layout will be determined in a way that water levels inside the bay are kept under safe values.

As it will be described later in detail, the barrier is composed of different elements. It is expected, due to the large depths and length of the closure sections, that a part of the section will be a closure dam. A

conceptual design of the dam will be performed. The focus of the design phase will be to address the high costs due to large amounts of material needed, and how to reduce these costs while keeping an acceptable performance.

1.4. REPORT STRUCTURE

This report is composed by the following parts

1. Introduction. This section will provide a general overview of the subject.

2. Problem description. Here, data and relevant information will be given, in order to be used later for analysis and design. Tokyo Bay and its characteristics will be described, mainly the site conditions that are relevant for flood risk and the construction of a barrier. Also, studies related to the subject will be summarized here.

3. Analysis on a System level. This part is divided into different subsections:

- Analysis of the need for taking measures to improve flood protection
- Definition of what these measures could be
- Comparison between the different existing alternatives
- Choice of best solution
- Analysis of results and conclusions

4. Design phase. Using the characteristics of the previously selected alternative, a design of the dam stretch of the barrier will be developed in this section. First, the design approach will be introduced. Then, requirements and relevant boundary conditions will be summarized. The validity of the design will be checked regarding construction feasibility, construction costs and performance during lifetime (earthquake performance will have special relevance).

5. Conclusions and recommendations. Here, the design phase outcomes and relevant findings will be summarized. Also recommendations regarding further investigation will be given.

Calculations and some reference information have been separated in **appendices**, which include the required development of certain subjects. When necessary, at the beginning of each section, a graph detailing the structure of the chapter will be shown. There, the relation between chapter sections and appendices will be explained.

2. PROBLEM DESCRIPTION

Here, information related to Tokyo Bay and its situation regarding flood risk is given, providing insight into the problem and possible solutions. This is meant to give an overview of the situation and data that will be needed later.

Details on Tokyo Bay physical and economical environment are presented in first place.

Since the need for a storm surge barrier is going to be studied, an introduction to them is given, explaining their characteristics and functions.

Finally, relevant publications concerning the topic are examined. This review will include studies related to Tokyo Bay situation regarding flood risk. It will also include a list of international standards and guidelines that are used in the report.

2.1. TOKYO BAY PHYSICAL ENVIRONMENT

2.1.1. MORPHOLOGY

Tokyo Bay has a surface of 1,500 km^2. It is rectangular shaped, with an approximate length of 50 km and a width of 15 km in most of its length. The lands surrounding the bay are mostly a plain (called Kanto plain), except for the area close to the bay mouth, where there are larger ground elevations.

Figure 2-1: Kanto Plain physical map 3D view (Maphill, 2013).

Regarding the bathymetry, for most of bay surface, the bottom depths are not very large. However, in the South, large depths are present, up to 80 meters in the strait close to Cape Futtsu and more than 200 meters at the entrance of the Bay, between Tsurugisaki Cape and Sunosaki Cape (see A1-1 in Appendix 1).

2.1.2. GEOLOGY

There is not much information available on the characteristics of soils at the entrance of Tokyo bay. It is expected that no boreholes o geological transects can be found, since there are no structures built in the area and this information was not needed in the past.

In the figure below seabed characteristics and a geological map are presented. It can be seen that there is more rock in the southern part (black spots on the left, yellow areas on the right). The northern lands are composed by Holocene and Pleistocene deposits (blue areas) while in the south Neogene-Quaternary sedimentary rocks appear (yellow areas). Most of seabed in the bay is composed of sand and is covered by a muddy layer.

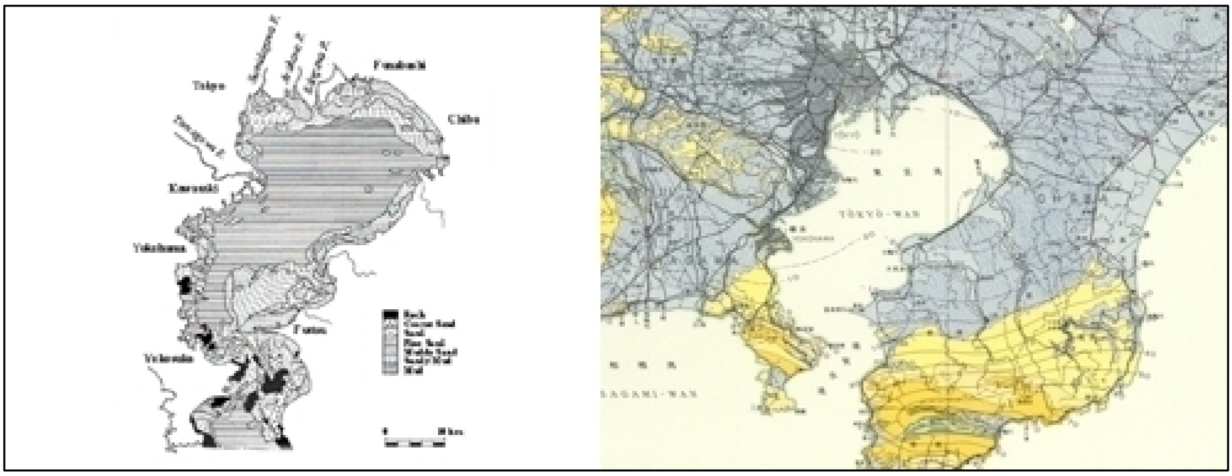

Figure 2-2: Seabed characteristics (Kaizuka, 1993) and geological map (Geological Survey of Japan).

2.1.3. RIVERS

The main rivers that flow into Tokyo Bay (River Bureau, 2014) are:

- Arakawa. It has a 2940 km^2 basin area, a peak discharge of 6000 m^3 and concentration time around 20h (Ahmed et al., 2011).
- Nakagawa. It has a 987 km^2 basin area.
- Tamagawa. It has a 1240 km^2 basin area, 4500 m^3/s peak discharge and concentration time around 12h (Nakagawa et al., 2012).
- Tsurumi. It has a 235 km^2 basin area, 1250 m^3/s peak discharge and concentration time around 3h (Terakawa, 2006).

2.1.4. TIDE

The average high spring tide level at Tokyo is A.P.+2.10 m (above Tokyo Bay Low Water Level) and tidal range value is 2.10 meters (Tokyo Metropolitan Government, 2013).

Tidal amplitude is approximately the same along the bay, according to measures and tidal simulations (see A1-3 in Appendix 1).

2.2. SOCIOECONOMIC ENVIRONMENT

2.2.1. POPULATION

The total population in Japan was 127,5M people in 2012, from which 37M live in Kanto area, surrounding Tokyo Bay. According to projections from Japanese Government (Statistics Bureau, 2013), in 2050 the total population of the country will be 97M. A decreasing trend starts in 2011 and in 2050 the annual rate is expected to be around -1%.

The area surrounding Tokyo Bay has very large population densities, the largest in the country. A distribution of population density can be found in the following figure.

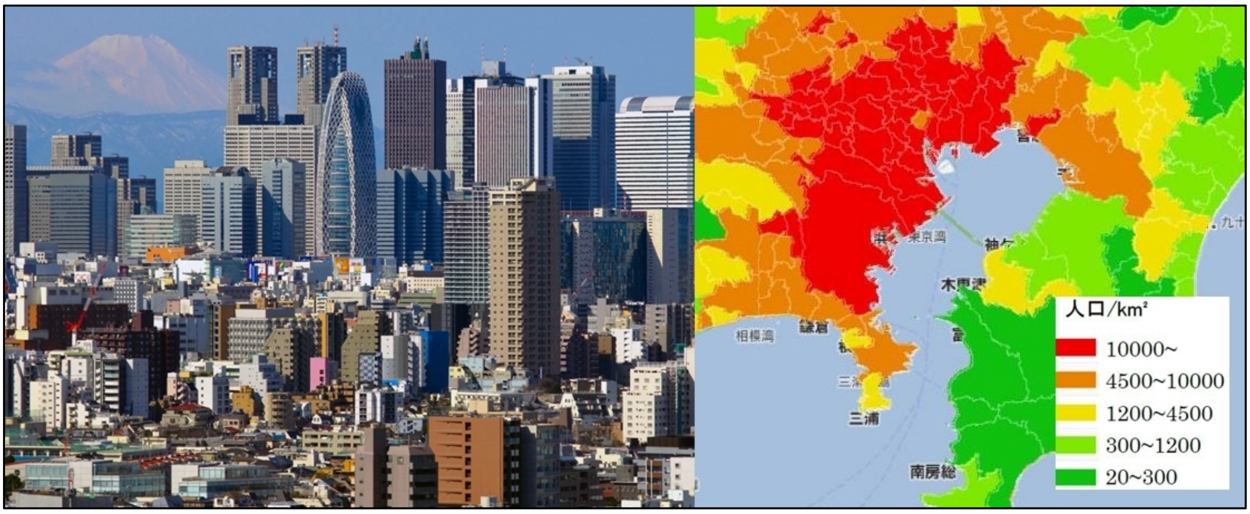

Figure 2-3: Tokyo skyline (left) and population density (per km²) in Kanto area (Hoshino, 2013).

2.2.2. ECONOMY

The Japanese GDP (Gross Domestic Product) is the third in the world, with 5,959 trillion $ in 2012 (World Bank, 2013) and about a 40% of it is generated in Tokyo Bay area. The expected economic annual growth is about 1% until 2040 and from this point the values will remain stable (Japan Center for Economic Research, 2007).

There is large international interaction in Japanese economy. Thus, a natural disaster that forces to stop activity not only would affect Japan, but there would be consequences for foreign economies too (V.Tsimopoulou, 2012).

Tokyo Bay can be considered important for the economy regarding several aspects, firstly due to the large amount of industries and ports that are located in the area. Fishery could be considered a relevant aspect too, due to the large importance of this activity in the traditional Japanese economy. In the past, fishery in Tokyo Bay was a major economic sector, but nowadays, the fish in Tokyo area comes mostly from outer islands (World Port Source, 2014). Currently about 5,000 fishermen (from prefectures of Tokyo, Kanagawa and Chiba) keep harvesting the Bay waters and 50,000 tons of fish are caught per year (T. Ueno, 2006).

Of special importance are seaweed farms as well. Due to the growing consumption and high prices, it is a sector on the rise. Several wind farms can be found in shallow water areas in Tokyo Bay (mostly close to Cape Futtsu).

2.2.3. PORTS AND NAVIGATION

Port facilities represent a large percentage of Tokyo Bay coastlines. The main Japanese ports are inside the bay: Ports of Yokohama, Chiba, Tokyo, Kawasaki, Yokosuka and Kisarazu.

These ports host large traffic and a significant part of Japanese economy depends on it. For example, Japan is only a 39% self-sufficient in food nowadays (Statistics Bureau, 2013).

The mouth of Tokyo bay is a congested waterway (Ikemachi M. et al., 2007). The ships enter Tokyo bay using Uraga Channel, being its dimensions: Breadth 1,400 m, Length 14.8 km, and Depth 23 m. The channel is giving service to approximately 600 ships per day (Cabinet Office, 2008).

2.3. FLOODS IN TOKYO BAY

2.3.1. PAST FLOODS

Tokyo is situated in a floodplain, between rivers and the sea, and has always been subject of flood risk. Measures against floods have been taken since the Tokugawa Shogunate, about 400 years ago, when it was decided to divert Edo River and make it flow to Pacific Ocean through Japanese East Coast instead of Tokyo Bay. Besides the morphological location characteristics (low elevation, proximity or rivers and sea), Tokyo Bay suffers the consequences of frequent typhoons, tropical cyclones that grow in the Western part of North Pacific Ocean and move Northwards reaching Japan several times a year. These characteristics make Tokyo Bay an area predisposed to flood disasters.

A large number of flood events have been registered in the past in Tokyo area (C40 Tokyo Conference on Climate Change, 2008). The most harmful storm surge event happened in 1917, when 87 km^2 were flooded, leaving 180,000 inundated houses and causing 1,524 deaths. This disaster was surpassed regarding material damage in 1958, due to floods caused by a typhoon (nº22 on Kano River). This time 311 km^2 were flooded and 340.000 houses were inundated, though the number of deaths was lower (203 people lost life).

Due to the relevance of the cities around Tokyo Bay, it is very important to keep flood protection levels in acceptable ranges and apply the available knowledge to protect population (and assets) in the area.

In the second half of 20th century no casualties due to floods were registered. Flood protection measures taken in last decades seem to have been effective. Nevertheless, several decades is not a period long enough to guaranty that current flood defences are sufficient to ensure the desired safety levels. The current state of coastal defences needs to be analyzed in order to have an idea of the existing safety margins and possible future performance.

2.3.2. EXISTING COASTAL DEFENCES

Gentle-slope levees and super levees are a widely used solution in Tokyo Bay, as they are perceived as safer and more suitable (environmentally, visually) than vertical walls. For example, super levees are being built nowadays in low elevation areas east of Sumidagawa River (Bureau of Construction, 2014).

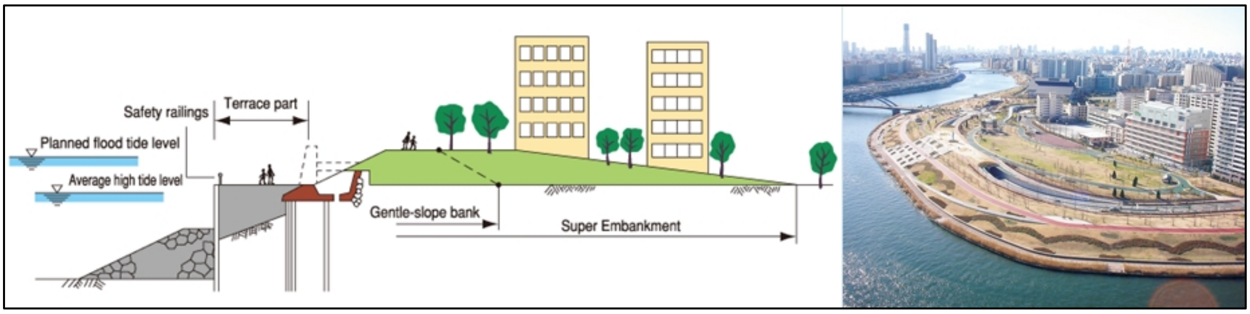

Figure 2-4: Installed super levee at Sumidagawa River (Bureau of Construction, 2014)

Vertical walls are used as flood protection as well. Also, flood gates have been built to protect river and channel banks against storm surges coming from open sea (see following figure).

Figure 2-5: Floodgates and walls in Tokyo Bay (S.N. Jonkman, 2014)

2.3.3. GOVERNMENT POLICIES

Tokyo has about 1.5M people living under the high-tide water level. For population and business to keep growing in the area, sufficient safety levels regarding flooding have to be guaranteed. There are active policies to protect from floods coming from rivers or storm surges (C40 Tokyo Conference on Climate Change, 2008). Nowadays, innovative solutions are applied jointly with walls and levees, like underground discharge channels and reservoirs. They help to increase Tokyo protection levels against floods.

In the past Japanese have designed their flood defences using as reference worst occurred events, which much times has meant to choose design events with return periods of several decades or around 100 years. But it seems that nowadays the approach is slowly changing and they have started to consider events with larger return period, i.e. 200 years for river defences, according to Japanese Government (Cooper and Matsuda, 2013). Nevertheless, they are still far from considering protection levels pursued in other countries, as The Netherlands, where the dikes protecting the country are designed for return periods between 1.250 and 10.000 years. This report will contemplate, in principle, a range of 100-500 years for design return period, which is close to Japanese practice.

2.4. TYPHOONS

A Typhoon is defined as a tropical storm in the region of the Indian or western Pacific oceans.

Typhoons affect Japan and specifically Tokyo bay area, where, as stated before, there is flood risk due to low ground elevations.

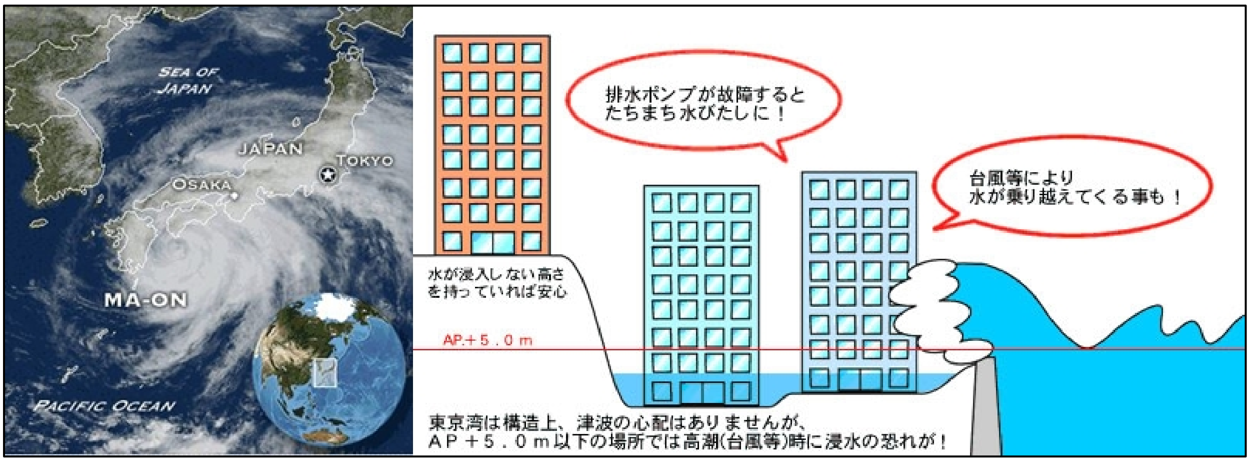

Figure 2-6: Typhoon Ma-On over Japan (Earthweek, 2011) and risk for lowlands (AT Tokyo Corporation, 2008).

In the following paragraphs, relevant data regarding typhoons will be presented.

2.4.1. RAINFALL

It is not clear whether extreme rainfall associated to typhoons does follow the same patterns as regular rainfall. However, it is difficult to find statistics on precipitation intensity that separate typhoon from normal precipitation. Anyway, it is interesting to check the extreme values, since these include typhoons as well (see A1-4 in Appendix 1).

Extreme typhoon rainfall tends to be concentrated in short periods. For Japanese typhoons there are measurements for rainfall intensity within 6 hours. For example, the 6 h record precipitation in Japan, 549.5 mm, was registered in Tokyo prefecture in October 2013 (Kitamoto, 2014).

2.4.2. STORM SURGE

A storm surge is an extraordinary rise of water level due to a storm. Part of the rise in water level is due to low atmospheric pressures, part is due to wind-induced piling up of water in low water depths.

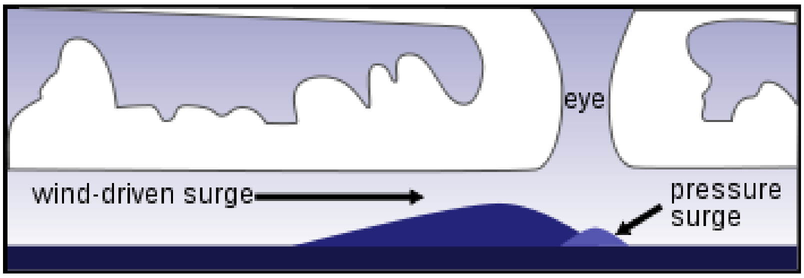

Figure 2-7: Storm surge composition (Graham and Riebeek, 2006).

Storm surge magnitudes in Japan have been studied by different authors. Values obtained from stochastic simulations can be observed in the following figure.

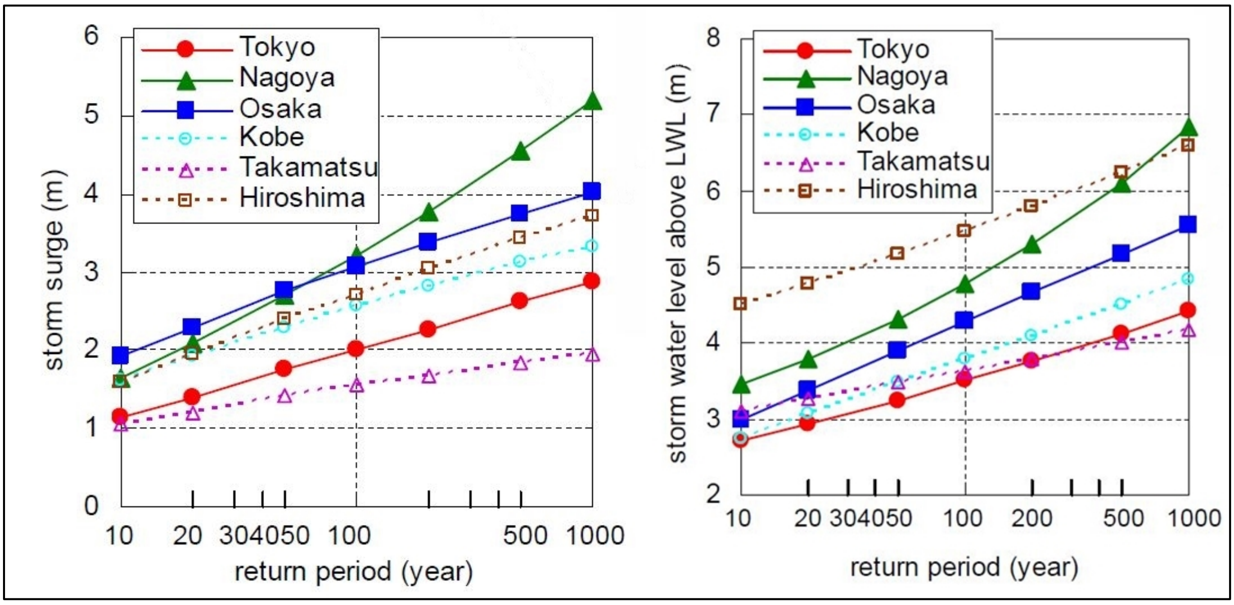

Figure 2-8: Storm surge (left) and storm water levels (right) for several locations in Japan (H. Kawai, 2008).

Typhoon surge duration is a relevant factor as well, regarding the design of a barrier. A value of 12 hours will be used for calculations, taking into account the available data, it is a conservative enough choice (Tokyo Metropolitan Government, 2013).

2.4.3. WAVES

Coupled waves-storm surge statistics were not found for Tokyo bay. Therefore, statistical values of waves will be used instead. The approximation is considered to serve well the purpose of this report, as waves are not expected to be a major parameter in the barrier definition.

Values from NOWPHAS Japanese network (Nagai, 2002) are used for calculations. Specifically, the significant wave height and period are calculated as the average of three measurement stations in the entrance of Tokyo Bay. The values can be found at A1-5 being the average result 5 meters of wave height, which is the value that will be used later.

2.4.4. EXPECTED INTENSITY INCREASE

Due to climate change typhoon intensity might increase (Knutson et al., 2010) and storm surges could be larger in the future in Japan. At the same time, the typhoon path is expected to move slightly northwards which would increase typhoon strikes on this country (H. Kawai, 2010). Several studies indicate that the typhoon intensity might increase while the frequency would descend, like the Stochastic Typhoon Model made for Osaka area (Yasuda, 2010).

There are studies about increasing trends in typhoon intensity that calculate representative parameters like wind velocities and pressure difference at the eye of the typhoon (H. Kawai, 2010).

Investigations have been carried out to estimate the expected damage associated to determined typhoon intensity. In Hoshino's MSc Thesis, the selected design typhoon is the most harmful registered in the 20th century, which corresponds approximately to a return period of 100 years. Then a simulation is done taking into account the expected increase in intensity, in order to calculate the consequences for Tokyo Bay area. An increase in storm surge is found as a result (see Figure 2-10).

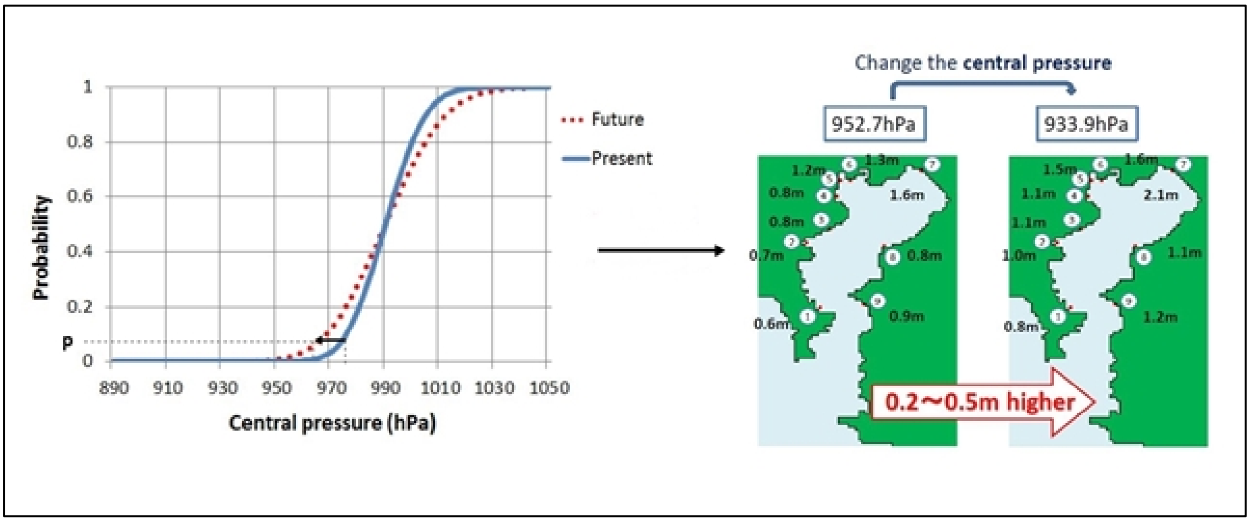

Figure 2-9: Change in typhoon characteristics (central pressure) and consequences for storm surge (Hoshino, 2013)

2.5. EARTHQUAKES

Tokyo bay is an area prone to earthquakes; due to its proximity to deep subduction zones (see Figure 2-11). They can be extremely destructive. A recent example is Tohoku earthquake in 2011, which caused many deaths and great economic losses.

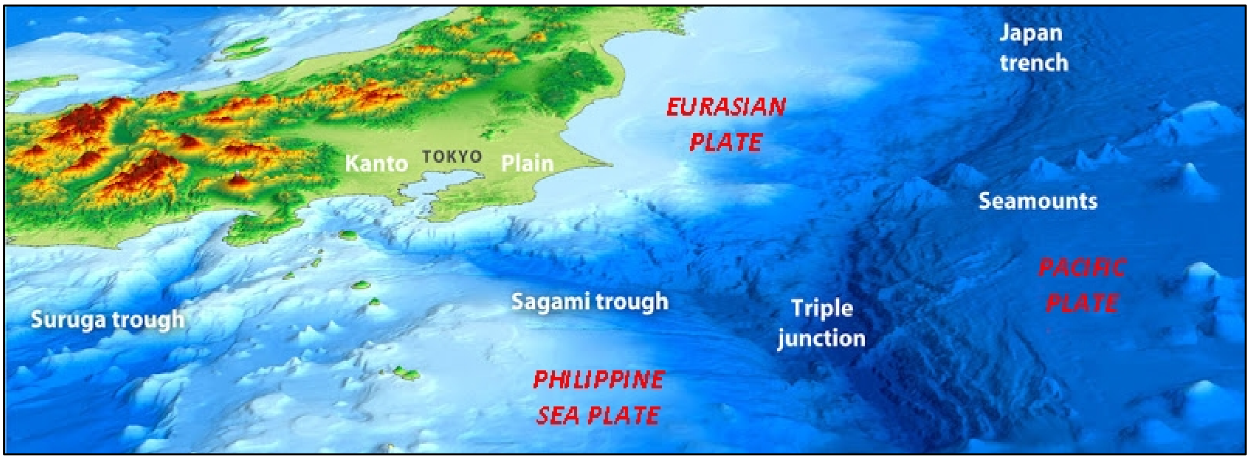

Figure 2-10: Tectonic plates in the region of the Japan Islands (Nippon Sekai, 2011).

In the past, strong earthquakes were considered unpredictable events, but this concept has changed and nowadays they are studied under a statistical approach (Takewaki, 2011). For example, the return period for the Tohoku Earthquake in 2011 is reported to be around 500-100 years. The great Kanto earthquake is assigned a return period of 220 years (Mahul and White, 2012). A graphical representation of return periods can be found in the figure below.

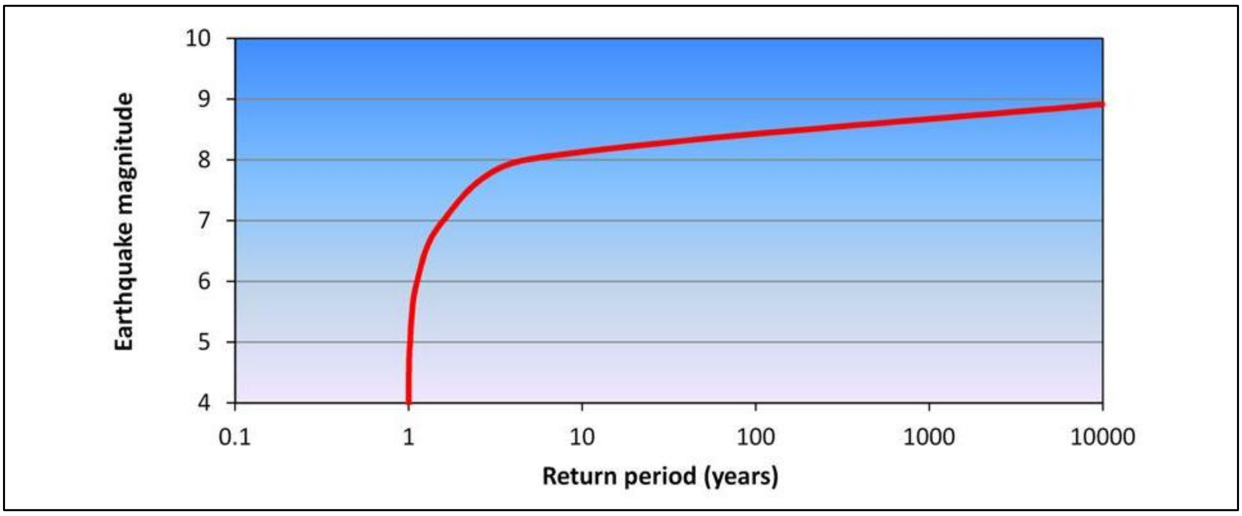

Figure 2-11: Return period versus magnitude for earthquakes in Japan, period 1900-2010 (Yegulalp, 2011).

The Headquarters for Earthquake Research Promotion regularly publishes information on National Seismic Hazard Maps for Japan. In these maps the probability of occurrence of earthquakes is linked with their intensity (see following figure).

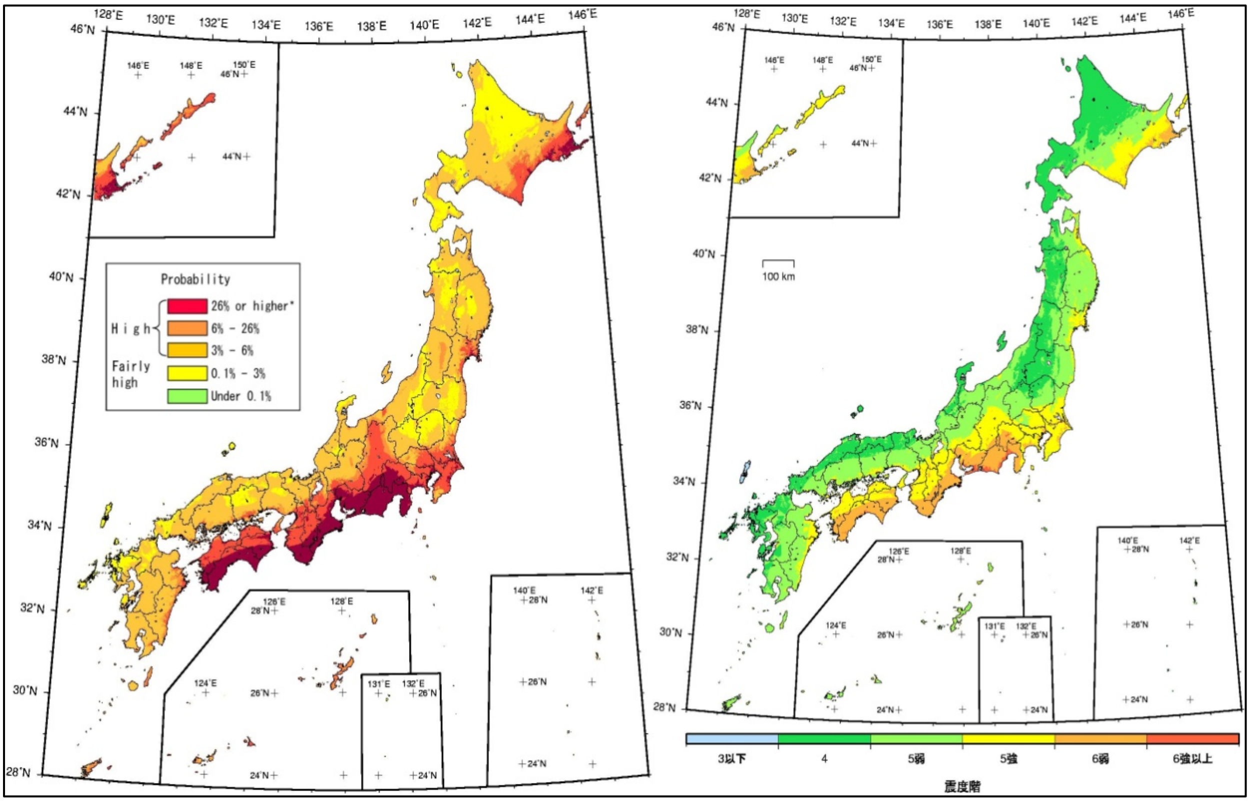

Figure 2-12: Exceedance probability of earthquake intensity 6 (JMA scale) in 30 years (left) and earthquake intensity for a 39% exceedance probability in 50 years (right) (Earthquake Research Committee, 2005).

As can be seen in the figure, Tokyo Bay is located in the area with largest seismic risk. Considerations regarding earthquake will be included later in calculations, as it is a relevant factor for design.

2.6. TSUNAMIS

A tsunami is a large sea wave that is caused by an underwater disturbance that mobilizes a large water body. Tsunamis are produced most of times by large and sudden displacements of the sea bottom, particularly at points where a subduction zone or a fault exist. Japan is an area close to these points and subject to tsunamis.

According to the National Oceanic and Atmospheric Administration's (NOAA) International Tsunami Information Center, all of the Pacific basin nations face the risk of tsunami. Between 1692 and 1998, 13 major tsunamis occurred in different parts of the world, and over 197,000 lives were lost (US National Oceanic and Atmospheric Administration, 2014).

Nevertheless, Tokyo Bay, due to its shape, is not as exposed as other parts of Japan and tsunami maximum heights are limited. For example, a simulation based on the Great Kanto Earthquake gives a tsunami wave height around 1.6 m close to Tokyo and 2.8 m close to Futtsu (see Appendix 1).

There are other studies that obtain lower tsunami heights, 1.5 meters around Tokyo port (T. Shibayama, 2013) or even lower than 1 meter (Wu, 2012).

Given the previous information, tsunamis are not considered to be the limiting factor for design regarding water levels. Tsunami height seems to be the same order of magnitude than storm surge, around 1.5-2.5 meters. The tsunami wave can also cause impact pressures on structures and this aspect will be considered later in design phase if necessary

2.7. STORM SURGE BARRIERS

Storm surge barriers are coastal defence structures that can protect tidal inlets, rivers and estuaries from occasional surge events.(…) These barriers are not applicable everywhere, but are best suited to tidal inlets with narrow mouths (UNFCCC, 1999).

Preventing coastal flooding is the main function of the structures called storm surge barriers. As stated above, they are particularly suitable for protection of tidal inlets with narrow mouths.

Lowlands close to tidal inlets or estuaries are vulnerable to storm surge events. The surge can result in extreme water levels and flooding. It can be more harmful when it coincides with high tide. For this type of coastal areas, the construction of a storm surge barrier allows a large reduction in the length of coastline to protect, which results in significant costs savings.

In the past, this concept was materialized by closing tidal inlets and estuaries completely, by means of building a closure dam. This method is still used nowadays, mainly in developing countries. Nevertheless, in developed countries, technology has evolved to more sophisticated and expensive structures, which have implemented other functions apart from flood protection. Storm surge barriers have now movable elements, such as gates, sluices or locks. These elements add other functions to the structure, such as making shipping possible. In the last decades environmental considerations are gaining importance. In order to protect the ecosystem, it is required to allow some water exchange between enclosed area and open sea. Nowadays storm surge barriers keep evolving and other functions are considered for them, such as energy generation.

Storm surge barriers are suitable for very specific conditions and have been applied in few places around the world. The Netherlands in particular has a long history of development and application of this type of projects (Afsluitdijk, Eastern Scheldt barrier, Maeslant barrier). Nowadays, other countries have developed projects for storm barriers; some of them still are in phase of design or construction (Thames barrier, MOSE project, New Orleans barrier).

Figure 2-13: Eastern Scheldt storm surge barrier (Source: www.architecture.about.com).

2.8. REFERENCE DOCUMENTS

2.8.1. PREVIOUS STUDIES

INCREASE IN FLOOD RISK:

Climate change is the source of changes in sea levels and typhoon intensity that would lead to need an upgrading of Tokyo Bay coastal defences.

Storm surges affecting Tokyo Bay are caused mainly by typhoons. Global warming is expected to influence **typhoon intensity**, as temperature differences (between ocean and atmosphere) are the cause of typhoon formation. Several authors have performed simulations to study changes in typhoon intensity, using global and regional climate models. From these simulations, some conclusions can be drawn, such as it is expected an increase in frequency of the strongest tropical cyclones. These extreme events could also have an increase in their intensity. For example, some models predict an increase in maximum wind speeds of 2-11% (Knutson et al., 2010).

Regarding the total number of typhoons, there are different interpretations, but it seems that the expected evolution depends on the studied location. For example, in the Atlantic basin the number is expected to decrease (Knutson et al., 2010), but there are studies that foresee an increase of the number of typhoons in the West Pacific area (Emanuel et al., 2008).

Besides that, **sea level rise** is another threat to face by coastal defences in Tokyo bay, as large values are predicted for the year 2100 that could make the height of coastal defences insufficient.

The lifetime of the foreseen structure will be around 100 years due to its importance and the large investment. Consequently, it seems reasonable to take values of SLR for the end of 21th century. Sea level rise predictions have been recently reviewed in the 5th report IPCC. The previous IPCC 4AR gave a range of 18-59 cm for SLR at the end of 21st century. In the new 5th report, the predictions are revised upwards and a range of 52-98 cm for a conservative scenario is considered (for year 2100). A value of 98 cm would have 17% probability of exceedance (which is the limit set in the report for the likely range of SLR). Besides that, some authors (Barrand et al., 2013) reflect about the possibility of Antarctic ice-sheets collapsing, which would mean extra centimeters of SLR (an average of 10-20 and a maximum of 40 cm). Therefore, a cautious decision would be to consider that SLR value could be around 1.10-1.50 m. This value would account for the collapse of ice-sheets, for which the probability of occurrence is unknown.

The **combination of sea level rise and an increase in typhoon intensity** is studied in S. Hoshino's MSc Thesis, where several specific scenarios are presented. The typhoon Taisho, from October of 1917, is used to simulate the consequences over Tokyo Bay. The obtained storm surge levels are combined with sea level rise producing different scenarios. Then, coastal defences' performance is evaluated. As the results lead to the conclusion that coastal defences might be insufficient to protect the area in the future, an estimation of the damage that could occur and the necessary investment to upgrade the defences is made.

Hoshino studied a typhoon that can be considered the worst in Tokyo Bay for around 100 years. The chosen design return period of the typhoon, regarding flood protection, is also subject of study. Several authors have calculated the values of expected storm surge for different return periods (H. Kawai, 2008) corresponding to Tokyo Bay and other large bays in Japan. Besides that, there are studies where the convenience of using a determined return period is analyzed (T. Takayama, 2004).

<u>EVALUATION OF DAMAGE PRODUCED BY FLOODS:</u>

The performed analysis is based in the methods described in (CUR-Publicatie-190, 1997). The reduction in risk provided by every alternative is supposed to be taken into account in the evaluation of the final costs.

There are recent studies that evaluate the costs associated to floods in large urban areas. For example, there are reports concerning New Orleans flood damage produced by Hurricane Katrina (Hallegatte, 2008), or damages in New York produced by hurricane Sandy. Also, there are studies that evaluate the consequences of the Tohoku Earthquake and Tsunami (V.Tsimopoulou, 2012). All these studies have been used as reference, for what concerns the amount of damage and the factors that are considered to have a relevant contribution to it.

<u>STORM SURGE BARRIERS:</u>

PIANC has done a compilation of the state of the art in “Design of movable weirs and storm surge barriers” (InCom, 2006).

There are few projects of storm surge barriers. An overview of the last ones executed can give an idea of the state of the art and trends. Relevant recent projects are the St. Petersburg flood barrier, Thames barrier or MOSE project in Venice. An increasing concern regarding environmental and aesthetic issues can be appreciated when comparing these projects and older barriers.

Research is carried out nowadays about new barrier typologies. For example, inflatable rubber dams have been tested for small water heads in rivers, with good results (Ramspol barrier). Other innovative ideas are being explored nowadays as well, such as the parachute-type barrier (van der Ziel, 2009).

2.8.2. STANDARDS AND GUIDELINES

The following standards or guidelines are used in this report:

- Design of movable weirs and storm surge barriers, from PIANC (InCom, 2006).
- Technical standards and commentaries for port and harbor facilities in Japan (Goda et al., 2002).
- Seismic design guidelines for port structures, from PIANC (MarCom, 2001).
- Harbour approach channel design guidelines (PIANC-IAPH, 2014).

3. ANALYSIS ON A SYSTEM LEVEL

3.1. INTRODUCTION

The structure of this chapter is shown in the following graph.

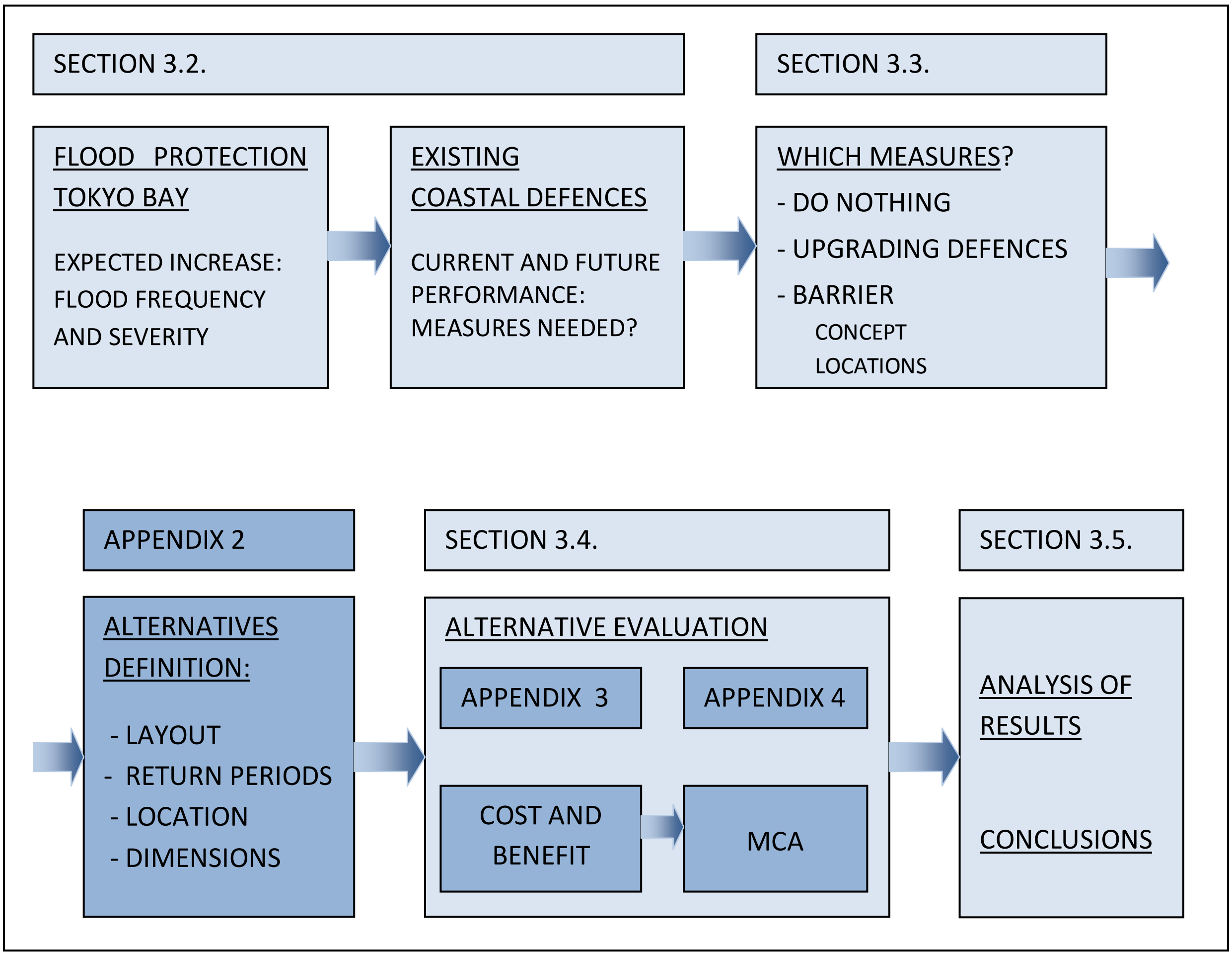

Figure 3-1: Chapter 3 structure.

This chapter is devoted to the analysis of the needed flood protection in Tokyo Bay. The aim is to study to which extend the bay needs protection and which are the most convenient measures to take.

First of all, the current situation of Tokyo Bay regarding flood risk will be presented. This includes an analysis of current and future performance of coastal defences. In case it is considered necessary to take action due to coastal defences being insufficient to protect the bay, a proposal for different measures will be done and the characteristics of these measures will be defined in order to compare them later. The comparison will be done by means of a multi-criteria analysis. Finally, the results will be summarized and analyzed. Advice will be given regarding the most favorable alternative, and also concerning the safety standards used for design of coastal defences.

Since water levels will be used frequently, later on in this report, a graph explaining them has been included so it can be used as reference (see Figure 3-2). The water level known as **A.P. will be used in this document as reference for all water elevations**, current and future, unless it is stated otherwise. It corresponds to present day low tide level.

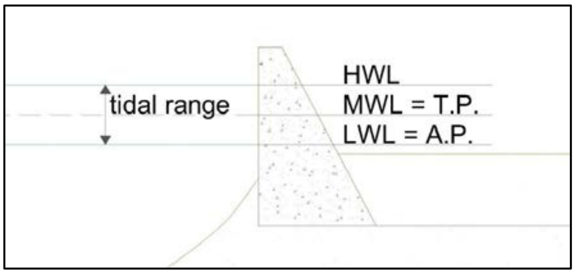

Figure 3-2: Reference water levels in this document.

3.2. CURRENT AND FUTURE PERFORMANCE OF COASTAL DEFENCES

As already stated, it is expected that climate conditions will produce larger flood elevations in the future. Also, the convenience of using a specific design return period is not clear. Thus, it should be studied how the coastal defences respond to future climate conditions and different return period events.

Regarding these considerations, the question to answer is whether coastal defences are can withstand expected future water elevations. In case they are not, the different alternatives of action should be evaluated.

The performance of coastal defences will be analyzed for different scenarios:

- Now and in the future (year 2100, with an increased typhoon and including sea level rise).
- For a 1/100 year typhoon and for a 1/500 year typhoon.
- For two future scenarios of sea level rise: 0.59 m and 1.90 m.

First, coastal defences elevation is calculated in each point, using data from Hoshino's MSc Thesis. Then, this coastal defences elevation is compared with water elevations produced by storm surge, in the cases mentioned above.

For storm surge elevations for 1/100 year typhoon, data from Hoshino's MSc Thesis is used. For 1/500 year typhoon, the 1/100 year data are extrapolated using the ratio provided by other studies (H. Kawai, 2008), where storm surge for Tokyo is represented for a range of return periods (see Figure 3-3).

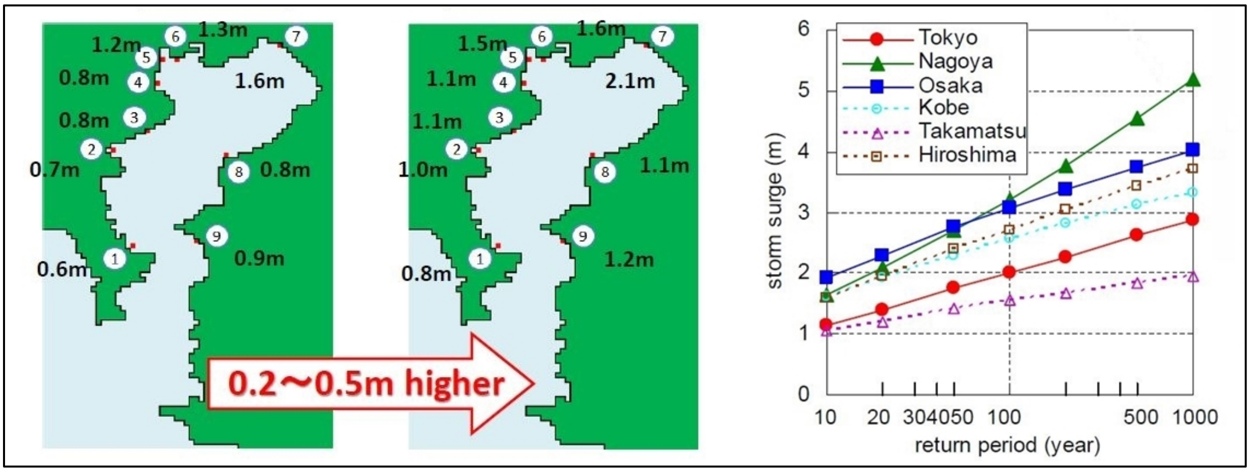

Figure 3-3: Storm surge level at different locations, now and in 2100 (Hoshino, 2013) and different return periods (H. Kawai, 2008)

Using the data from the figure above, for all the specified locations (1 to 9) it has been estimated the performance of coastal defences in different conditions. Storm surge elevations have been compared to coastal defences elevations (also obtained from Hoshino's MSc Thesis). The performance is evaluated below, according to a scale defined in Figure 3-4. The results are shown in Table 3-1. and Table 3-2.

As can be seen in the following table, regardless the typhoon return period, coastal defences will not be sufficient to withstand storm surges in year 2100, in case the rise in sea level exceeds certain limits. Their elevation is surpassed (at some points) by storm surge levels when considering SLR of 1.50 m or larger. Since 1.50 m is the chosen design value for SLR (see 2.8.1), it is considered that there is need for taking measures.

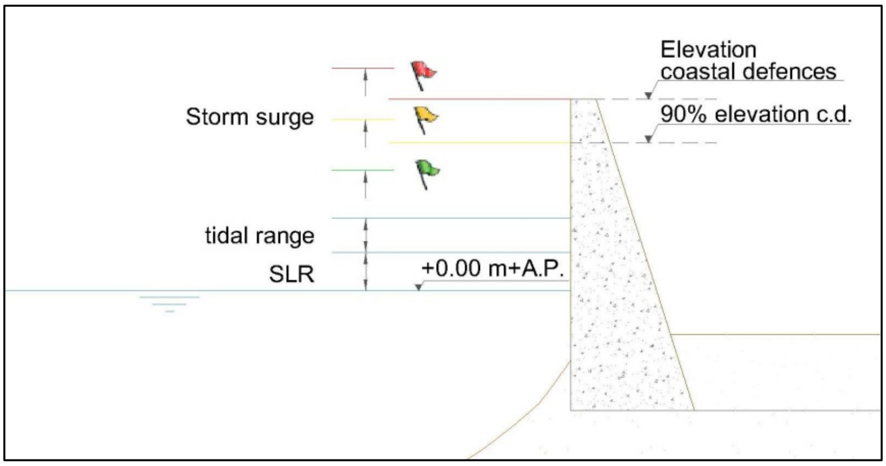

Figure 3-4: Coastal defences performance related to elevation of storm surge.

Table 3-1: Water elevations during typhoon in Tokyo Bay, for 1/100 year typhoon.

COASTAL DEFENCES		TYPHOON T=100yr					
CURRENT STATE		NOW		YEAR 2100			
LOCATION	Defences elevation MWL (m +T.P.)	Storm surge (m) (*)	Water elevation LWL (m +A.P.)	Storm surge increased CC (m)	Water elevation LWL+0.59 (m +A.P.)	Water elevation LWL+1.50 (m +A.P.)	Water elevation LWL+1.90 (m +A.P.)
1	3.89	0.60	2.70	0.80	3.49	4.40	4.80
2	4.39	0.70	2.80	1.00	3.69	4.60	5.00
3	4.79	0.80	2.90	1.10	3.79	4.70	5.10
4	5.29	0.80	2.90	1.10	3.79	4.70	5.10
5	5.69	1.20	3.30	1.50	4.19	5.10	5.50
6	5.69	1.30	3.40	1.60	4.29	5.20	5.60
7	6.29	1.60	3.70	2.10	4.79	5.70	6.10
8	4.69	0.80	2.90	1.10	3.79	4.70	5.10
9	4.39	0.90	3.00	1.20	3.89	4.80	5.20

Table 3-2: Water elevations during typhoon in Tokyo Bay, for 1/500 year typhoon.

		TYPHOON T=500yr					
CURRENT STATE		NOW		YEAR 2100			
LOCATION	Defences elevation MWL (m +T.P.)	Storm surge (m)	Water elevation LWL (m +A.P.)	Storm surge increased CC (m)	Water elevation LWL+0.59 (m +A.P.)	Water elevation LWL+1.50 (m +A.P.)	Water elevation LWL+1.90 (m +A.P.)
		(*)x1.35		Increase proportional to (H. Kawai, 2008)			
1	3.89	0.81	2.91	1.08	3.77	4.68	5.08
2	4.39	0.95	3.05	1.35	4.04	4.95	5.35
3	4.79	1.08	3.18	1.49	4.18	5.09	5.49
4	5.29	1.08	3.18	1.49	4.18	5.09	5.49
5	5.69	1.62	3.72	2.03	4.72	5.63	6.03
6	5.69	1.76	3.86	2.16	4.85	5.76	6.16
7	6.29	2.16	4.26	2.84	5.53	6.44	6.84
8	4.69	1.08	3.18	1.49	4.18	5.09	5.49
9	4.39	1.22	3.32	1.62	4.31	5.22	5.62

3.3. FLOOD PROTECTION ALTERNATIVES

Since coastal defences might not be enough to protect Tokyo Bay in year 2100, the measures to take are studied.

In the following section, the possible alternatives for action are described in detail. Their characteristics will be presented in order to proceed to a comparison between them. Sufficient definition needs to be given so a multi-criteria analysis and costs calculations can be performed later.

3.3.1. ALTERNATIVES

It has been shown in previous section that coastal defences might be not sufficient in year 2100. If this is the case, measures will be needed to protect the area in the future. The most immediate option to consider is the upgrading of the existing coastal defences. This possibility has been studied by Hoshino's MSc Thesis, and the result was that the investment was significant and the undertaking complex. This leads to consider other options, for example a storm surge barrier.

Concerning the possible **courses of action**, the most plausible ones seem to be:

- Do nothing.
- Upgrade coastal defences.
- Build a storm surge barrier.

Also, the design typhoon is an important choice when considering which the necessary measures are. As stated before, Japanese practice uses periods close to 100 years. But this approach changes in other countries. For example, Dutch practice is much closer of a statistical approach that theoretically minimizes losses. This could lead to the choice of a much longer return period. For the analysis that will be performed here it is decided to study return periods between 100 and 500 years, so the final choice is close to Japanese methods.

Hence, the chosen **return periods** to study will be:

- 100 years, because the data available for comparison was obtained for this return period
- 500 years, in order to confirm whether there are substantial changes in the analysis outcome when increasing the return period.

Finally, another important point, regarding the choice of alternatives in case of the barrier, is the **barrier location**. This factor has large influence in the final costs, since variations in building costs and in risk can be considerable when changing location.

3.3.2. UPGRADING COASTAL DEFENCES

In case coastal defences become insufficient regarding the desired level of protection, upgrading them is one of the options. This way they can reach a height sufficient to protect the land from design storm water levels.

In this report, the upgrading considered in Hoshino's MSc Thesis is considered. A study of the needs was done there and an evaluation of the costs as well. The investment calculated by Hoshino will be used later for a comparison with the other proposed alternatives.

Hoshino considers the following costs for the upgrading of coastal defences:

- Build new levees and elevate the existing ones at some areas.
- Elevate port area platforms outside the protection of dikes.

The length of considered levees and platform surfaces can be observed in the following figure.

Figure 3-5: Upgrading of coastal defences, from left to right: Tokyo, Kawasaki and Yokohama ports.

The results are summarized in the following table.

Table 3-3: Cost of upgrading coastal defences

		Coastal levees		Waterside land	**Upgrading coastal defences (M€)**
		Build a new one (M€)	Seismic reinforcement (M€)	Raise the ground level (M€)	
TOKYO	Tokyo Port	42.49	688.58	137.89	**868.96**
KANAGAWA	Kawasaki Port	25.67	422.54	479.37	**927.58**
	Yokohama Port	40.91	669.80	243.99	**954.70**
TOTAL					**2751.24**

This is done for a typhoon with an estimated return period of about 100 years and considering an increased intensity in respect to the current one. Besides that, several SLR scenarios are considered.

However, there is no data for a return period of 500 years. The calculation of the investment corresponding to 500 years is complex enough to not allow a direct extrapolation form Hoshino's data. It will be left out of the scope of this thesis and no further elaboration will be done regarding this.

3.3.3. STORM SURGE BARRIER

First of all, the requirements to fulfill are studied. These requirements will determine the elements that give shape to the barrier. Using these elements, a general layout is proposed.

Later, several barrier locations are studied. All of them have the same layout that was proposed in section 3.3.3.1. The conditions for definition of barrier characteristics will be the same as the ones considered for the upgrading of coastal defences, so the comparison is possible. Therefore, a typhoon with a return period of 100 years is chosen as design event. Also, a SLR of 1.90 m is used.

For all the studied locations the main dimensions are determined (for 100 years). For the ones that are found more favorable, calculations for 500 years will be completed, as will be explained later on.

3.3.3.1. REQUIREMENTS

The requirements that the storm surge barrier has to satisfy in case of Tokyo Bay are:

- Prevent flooding due to storm surges in the inner part of the bay.
- Allow navigation, since there are major ports located inside the bay.
- Allow wildlife crossing and water exchange, in order to protect the bay ecosystem.
- Allow river discharge to flow to open sea.
- Allow inspection and maintenance activities.
- All these functions have to be guaranteed for a certain lifetime. According to the importance of the structure and the large investment that would be needed, a lifetime of 100 years is set.
- Economic factors (minimize investment and maintenance costs).

Other desired characteristics could be:

- Allow land transport between the two edges of the barrier (road connection). This function could be needed, but the possibility will not be explored here, as it seems unlikely due to the low development of the areas on the west part of the bay).

- Create amenity (recreation, landmark, etc). This aspect should be considered in case the project is executed, but no further elaboration will be done here.

There are other functions that storm surges can offer, however for this case study they are not needed or applicable:

- Separate fresh and sea water.
- Land reclamation.
- Generation of energy.

These listed requirements determine the barrier layout and, specifically, the elements that compose the barrier. In case of Tokyo bay, in order to satisfy the previous conditions, the barrier would be composed of a number of elements:

- **Closure**, in order to keep water from entering the bay during storms. A closure dam would be used, in principle, to accomplish this function together with movable gates.
- **Permanent opening**, which is the part that remains open during a storm surge event. This part will be as large as possible for several reasons:
  - Allow navigation. The use of elements such as sluices can give service to navigational needs. However, it will not be the case here, since the intensity of shipping traffic is very high and a permanent open channel should be provided in normal conditions. It would be desirable that this navigational opening can be left open during storms as well, so there is no need to close it with gates (the navigational channel will have large depths and gates would be expensive to place there).
  - Environmental requirements. Wildlife crossing has to be allowed and also a sufficient water turnover for the inner bay ecosystem to be preserved. Currents help to keep the oxygen content of waters and also avoid increases in temperature. Moreover, if the water exchange is too low, the tidal range would be reduced inside, altering the biological processes that are related to these periodic oscillations of water level.
  - Economical reasons (costs savings).

  Nevertheless, the permanent opening size cannot be as large as desired, since it is limited by the tolerable rise of water level behind the barrier. During a storm surge event only a small amount of water is allowed to enter the bay, so the inner water levels do not rise above certain limits. This maximum amount of water will determine the maximum size of the permanent opening.

- **Movable barrier**, which allows temporal closures in case the different requirements for the opening conflict. If the opening has to be different during a storm surge event and during normal conditions, this element will be necessary. This part of the barrier remains open during normal conditions and has to be closed during a typhoon, to reduce the amount of water that enters the bay.
- A **road,** over the top of the barrier is only considered in principle for maintenance purposes. However, in case of building a barrier, the need for a causeway that communicates both sides of the bay should be studied. In case of communicating both sides, the road design should deal with the problem of how to cross over the permanent opening.

The characteristics of the main elements are explained in more detail in next section.

PERMANENT OPENING

The part that can be left open during a storm surge event is the permanent opening.

As long as it is possible, it is desirable to design the permanent opening so the navigational channel can be included in it. The navigational channel will require large depths for shipping and it can be complicated to close these large depths with gates. This circumstance can be avoided by locating the navigational channel in the area corresponding to the permanent opening.

Hence, for a first estimation, the permanent opening of the barrier is given the dimensions of the navigational channel. In case later in the calculations it is found that the opening dimensions are excessive (so they allow to enter the bay a too large discharge), it would be necessary to consider reduction of the permanent opening.

For channel width calculation, Harbour approach channels design guidelines (PIANC-IAPH, 2014) have been used. In these guidelines, a Japanese Design Method and Japanese standards are included. The design vessels are given there as well. Due to the importance of the ports inside the bay, the largest ship for each category is chosen as design vessel. Also, a two way channel is envisaged. A proper calculation of the channel capacity would involve complex calculations. However, in order to do this, much more information would be needed regarding traffic characteristics and this is considered out of the scope of this Thesis. The result of calculations is a width of 625 meters (see A1-7 in Appendix 1 for details).

Not having available data on the design vessel considered by ports in Tokyo Bay, the actual channel depth, 23 meters, is compared with other important projects. For example, Maasvlakte 2 will allow the entrance of vessels with a draught up to 22.5 meters (Port of Rotterdam, 2014), or the Third Set of Locks of Panama Canal will have a depth of 18.30 meters (Impregilo, 2009). Taking all this into account, a draught of 25 meters is selected.

This navigational opening might need a revetment that protects it from high current velocities during normal tidal cycles or high waters events, while the water flows in and out the bay. This aspect however will be out of the scope of this thesis and no further attention will be devoted to it.

In principle the permanent opening will be located in a deep area, and it would be interesting to locate it in the same position than the existing navigational channel (this way, the navigational channel would not need a change in its layout).

MOVABLE BARRIER

The first step to define the movable barrier is to calculate the opening that is required due to environmental reasons. This section will remain open during normal conditions, allowing for water and wildlife exchange.

The determination of the opening due to environmental reasons is a complex issue that needs detailed examination. A complete study is out of the scope of this thesis, consequently an approximation will be made. This opening has to guaranty a sufficient water exchange and tidal range inside the bay. Similar projects and guidelines have been studied, in order to find useful references. For example, for Eastern Scheldt barrier, a 18% of the original cross section remains open due to environmental reasons (A. van der Toorn, 2013). The aim is to keep a 87% of the original tidal range inside the estuary, which is considered the minimum in order to keep acceptable conditions to maintain the ecosystem. The minimum tidal range inside the basin (87%) is the value taken as reference, in order to use it in the calculations for Tokyo Bay.

Once the tidal range inside the bay is fixed, it is possible to define the open cross section that corresponds to it. This section has to remain open during normal conditions. But as it was noted before, during a typhoon, only the permanent opening (625*25m) is allowed. Therefore the rest needs to be closed. This section is the movable barrier.

This movable barrier will be composed of elements that allow alternative opening and closure. Specifically, several types of gates could be appropriate for this design (InCom, 2006):

- Vertical sliding gate.
- Horizontal roller gate.
- Horizontal axis sector gate.
- Flap gate.
- Visor gate.
- Inflatable dam.

This selection has been done while taking into account certain considerations:

- Dimensional requirements for movable barrier are not as strict as for navigational section (no horizontal or vertical clearance required); therefore, in principle, the range of alternatives is larger.
- Simplicity and keeping gates and mechanical devices out of water are appreciated characteristics.
- Since the shallow area is not very long, a design that reduces piles or eliminates it would help to keep the gates stretch shorter and therefore not needing a large dam under their foundation.
- It is vital that gates perform well under earthquake and the possible differential settlements need to be dealt with. In order to achieve that, a solution based on adjacent elements that move independently could be convenient.

Only these preliminary considerations regarding the movable barrier definition will be presented in this report. Gates design is out of the scope of this thesis and will not be further developed.

Whenever possible, the movable barrier will be located in the shallow part of the section. This helps:

- Avoiding a large dam under the barrier.
- Facilitate water exchange and currents in the shallow area where seaweed farms are located (seaweed needs minimum conditions regarding water turnover).

DAM

Deducting the elements that have already been mentioned, the rest of the barrier length will be a closure dam. In a first approach, it is assumed that two types of cross sections will be used (for different depths), a rubble mound dam and a composite structure (see next section).

3.3.3.2. PROPOSED BARRIER LAYOUT FOR TOKYO BAY

Taking into account the requirements stated in previous section, a preliminary barrier layout is defined. A sketch can be found in the following figure. Also, several cross sections of the different elements are marked. These cross sections are sketched later in this section as well.

This layout is an initial approximation and will be further developed in the following sections of this report.

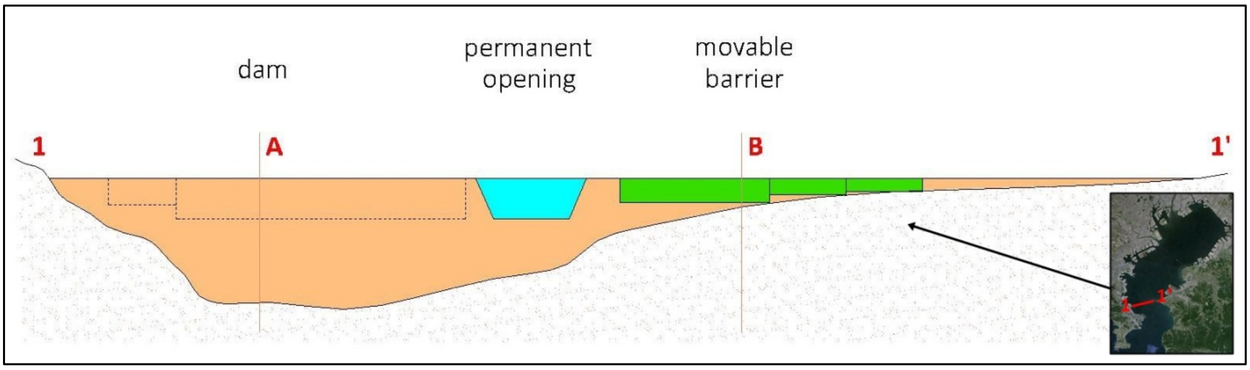

Figure 3-6: Barrier longitudinal layout and cross sections (A,B,C) at different points.

The barrier will be a dam for most of its length. The dam will consist of a rubble mound or a composite section (see Figure 3-7). A study of these two types of cross section will be carried out in Chapter 4, in order to decide which one is more appropriate.

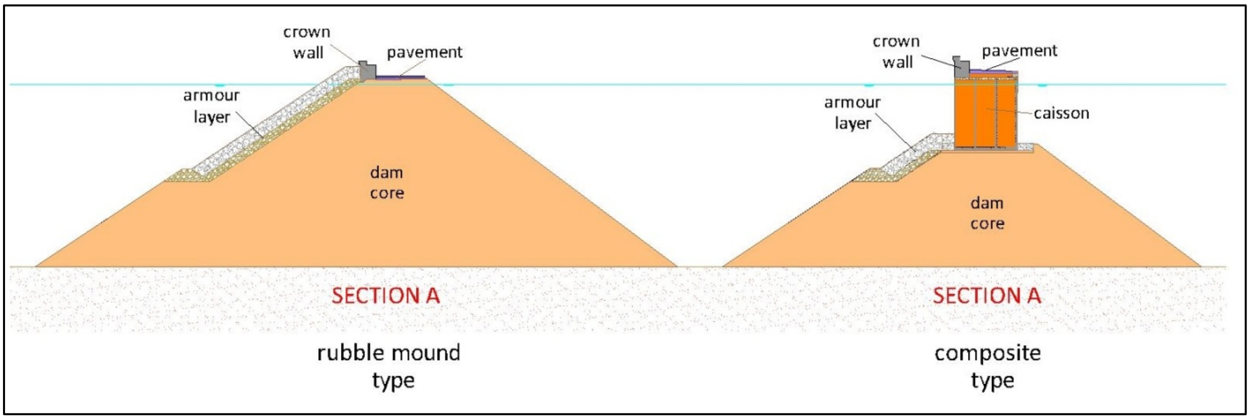

Figure 3-7: Dam cross section

Also a movable barrier (see Figure 3-8) is foreseen. It is assumed that this movable barrier will consist on gates or an inflatable dam, as explained in the previous section. However, this part of the barrier will not be defined in detail.

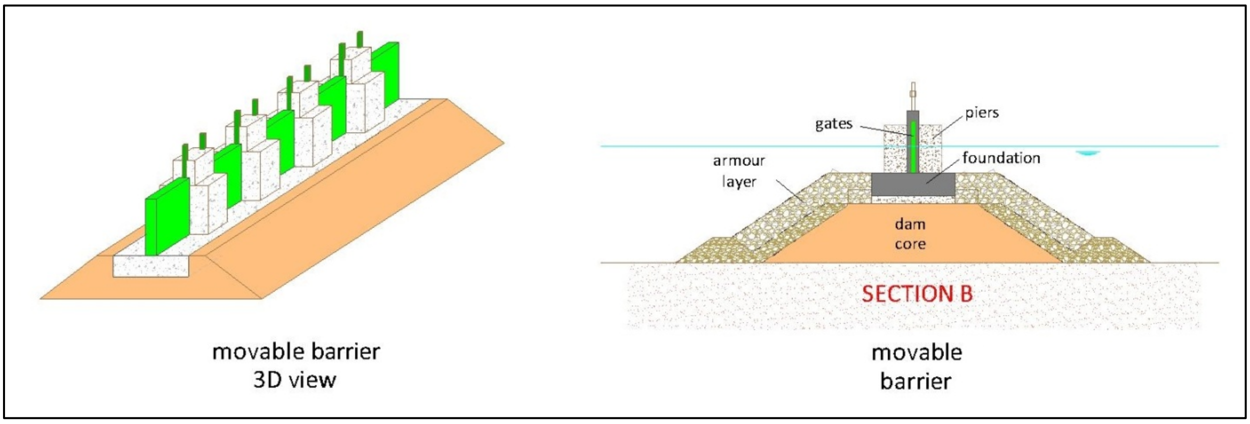

Figure 3-8: Movable barrier view and cross section

This layout will be used from this point onwards in the analysis carried out for the barrier, considered at a system level. The dam cross section will be studied in detail later in the design chapter.

3.3.3.3. POSSIBLE LOCATIONS

Six possible locations are initially proposed (see Figure 3-9). Their position has been chosen mainly to minimize the barrier depth and length, while protecting the bay area to a different degree.

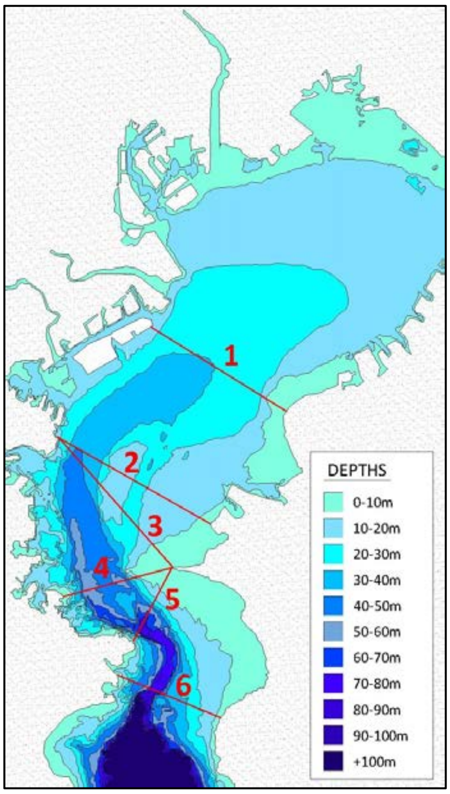

Figure 3-9: Barrier location alternatives and water depths.

In the following paragraphs the chosen locations and their main characteristics will be presented.

LOCATION 1

Even though depths are lower than for other locations, cross section is still very large. Therefore it is not clear that there will be a reduction in costs, compared to other locations. Besides that, this location leaves about half of the bay coastline unprotected. In principle, this location is not very favorable.

LOCATION 2

This location is chosen to avoid depths larger than 40 meters. It protects Tokyo and half of Chiba coastline.

Industrial facilities are located at the barrier edges, so they would need relocation. The barrier would be very “visible” since it is close to the most populated areas.

LOCATION 3

This location also avoids depths larger than 40 meters, protecting a larger area than number 2. Still, the length of cross section makes it expensive and part of Yokohama is left exposed.

The barrier is crossing a seaweed farm area in the east end. The area (Cape Futtsu) is a park and it might be a problem to invade a natural area. Besides that, the barrier is close to an industrial area in the west part.

LOCATION 4

Location 4 protects a larger area than 3 (port of Yokohama and surroundings), having a shorter length and still avoiding depths larger than 60 meters.

LOCATION 5

This is the shortest stretch for closure, but it crosses areas with large depths (up to 80 meters).

LOCATION 6

This location would protect a larger area than 5, having a length slightly larger and the same depths. This could be interesting in case there is urban development foreseen in the area close to the bay mouth. If it is not the case, it does not make sense to place the barrier here (more expensive but no significant increase in protection).

3.3.3.4. DIMENSIONS

The barrier is assumed to have the layout presented before (section 3.3.3.2). It will be composed of a dam, a movable barrier and a permanent opening. With this barrier composition, some dimensions need to be defined in order to calculate the investment that each location would imply. The basic dimensions can be found below:

- Slopes, that are set at 1:2.5 to give sufficient stability, according to research done related to breakwaters in seismic areas (Memos and Protonotarios, 1992).
- Depths, given by bathymetry (see Appendix 1, A1-1).
- Freeboard (pending).
- Use of caissons on top of dam, for depths larger than 50-60 m, in a first approach.
- Total surface of permanent opening (pending).
- Total surface of gates in movable barrier (determined by minimum tidal range to maintain in the basin, see Appendix 2).

Gates surface depends on what is called environmental section, which is the cross section that needs to remain open in normal conditions, in order to allow the minimum necessary water exchange between the ocean and the bay (due to environmental reasons). This is related to the reduction in tidal range that will take place when closing the bay, as less water goes inside. Once this surface is known, the gates surface can be calculated, as it will be the remaining surface after subtracting the permanent opening.

FREEBOARD AND PERMANENT OPENING

As stated before, all the main dimensions have already been set in advance, except for freeboard and permanent opening. These two will depend on the increase in water level that can be allowed inside the bay during extreme water level events. The design storm surge is used for this calculation. The magnitude of tsunami and storm surge is quite similar for this case study, while the duration of storm surge is larger than the tsunami (hours compared to minutes). Therefore, it is expected that the storm surge will cause a larger amount of water to enter the bay and will be the limiting factor. The design storm surge will be used to calculate **freeboard** and the **opening during typhoons**.

In case that a storm surge develops, the barrier gates will be closed, but yet some water will enter the bay via the permanent opening or over the barrier, in form of overtopping. There will be also other contributions to the increase in water level inside the bay, as local surge generated inside the bay or rivers discharge contribution. The allowable increase in water levels is limited by the height of coastal defences inside the bay, so the maximum amount of water that can enter the bay can be calculated. Since the other contributions can be determined, the amount of water from overtopping and permanent opening can be calculated by subtracting the other contributions from the total.

As they are two unknowns, there are several combinations that would allow fulfilling the requirement of not surpassing the coastal defences elevation. It is considered that variations of freeboard within a reasonable range (usual freeboards given to dams or breakwaters) will not produce large variations in the total cost, so freeboard will be fixed first, and then the permanent opening will be calculated.

A general sketch explaining all the elements involved in this calculation is presented at Figure 3-10. It can be observed how the limitation for water levels comes from the elevation of coastal defences, and how the barrier characteristics (opening and freeboard) are related to these water levels.

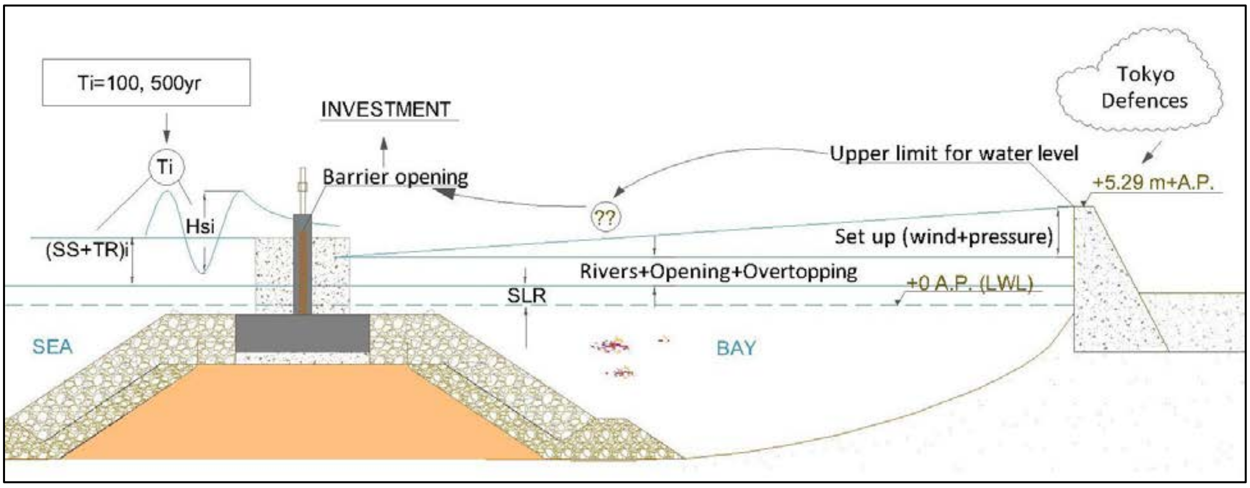

Figure 3-10: Calculation of design water levels for different return periods

In order to define the opening, some calculations regarding water elevations must be done. There are several contributions to the increase in water level inside the bay, being the upper limit the coastal defences height. So, once this limit is set, and the rest of contributions are known, the permanent opening and barrier freeboard can be calculated (see following figure).

Once this critical elevation is known (point 5 in Figure 3-3, see Table 3-1 for elevation values), and the freeboard (overtopping) over the barrier is fixed, the permanent opening can be calculated as it is the only unknown left (see Figure 3-11).

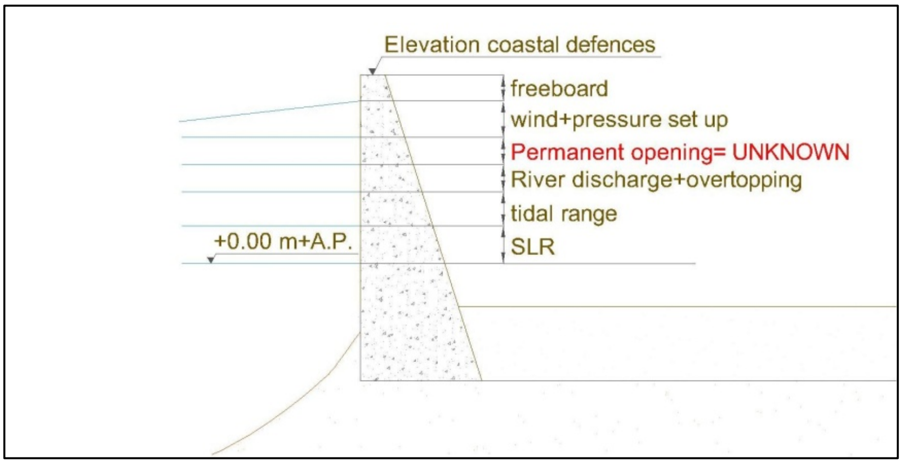

Figure 3-11: Sketch of water levels used to calculate permanent opening in the barrier

The elements that participate in this calculation are:

- **Coastal defences elevation**: known to be +5.29 meters. The elevation of coastal defences at Tokyo area (see Figure 3-3, points 4, 5 and 6) is chosen as upper limit for the increase in water level inside the bay, at the point where a largest storm surge is expected (point 5). The reason for choosing this point is that Tokyo is, by large, the most important location, with the larger concentration of population and assets affected in case of flooding.
- **Sea level rise**: set at 1.90 meters, the same value accounted for in the calculation for upgrading of coastal defences.
- **Wind and pressure set up**: even if there is a barrier closing the bay mouth, some wind and pressure set-up still can be built inside when the typhoon crosses the bay. It is considered too conservative to add maximum wind and pressure set-up, as maximum pressure set-up is expected close to the eye of the typhoon, and maximum wind set up is close to areas with high wind velocities. Therefore, set-up has been calculated at these two points and the maximum value is chosen. For wind set-up, a formula depending on fetch is used. For pressure set-up calculation, a formula depending on pressure gradients for the design typhoon is chosen. The result is a set-up of 1.45 m for 100 year and 1.80 m for 500 year return period (see A2-1 for more details).
- **River discharge**: It is considered that discharges start to rise at the same time so they peak within the storm surge duration. The closure time is set in 12 hours, according to storm surge duration (see 2.4.2). It is considered that all the rivers will arrive to their maximum discharges, except for Ara River; since the concentration time is 20 h, the reached discharge will be 12/20 of the peak discharge. The discharges used are the ones detailed in 2.1. Taking an average discharge of 50% of peak discharge for the whole period of closure, the volume

of water contributed by rivers is calculated and the result is an increase of 0.30 m in water level (see A2-1).

- **Overtopping**: calculated from the chosen freeboard that the barrier will have over the extreme storm surge level (2m). With this and wave data (5 m height, period 8 s, see A1-5, Appendix 1), the amount of overtopping can be calculated. The volume of overtopping is estimated as a volume per meter length. Knowing the total length of the barrier and considering a constant overtopping over the duration of the storm surge, the total volume of water contributed can be calculated. This ends in a value of 0,07 m. As the amount is quite low, the approximation is considered to be good enough (see A2-1).
- **Freeboard.** Overtopping the coastal defences is not considered as failure and only the exceedance of certain water level will be. A small freeboard of 0.50 meters is considered, as safety margin.

The result of this calculation can be found in the following table:

Table 3-4: Allowable increase in water levels inside the bay.

	T=100yr		T=500 yr	
Elevation coastal defences (A.P.)	5.29	m	5.29	m
SLR	1.90	m	1.90	m
Wind + Pressure setup	1.45	m	1.80	m
River discharge	0.30	m	0.30	m
Overtopping	0.07	m	0.07	m
Freeboard	0.50	m	0.50	m
Allowable increase	**1.06**	**m**	**0.72**	**m**

This allowable increase left (last row in the previous table) corresponds to the water that enters the bay via the permanent opening, which is the only contribution not accounted for at this point.

There will be a water flow entering the bay due to the head in water elevations outside and inside the bay. The conservative assumption is made that maximum storm surge will coincide with high tide, and that they have similar period (12h for tide and for storm surge, which can be assimilated to a sinusoidal movement). Therefore, the two of them together can be modeled as a harmonic movement of sea water elevation, which starts at low water level (the moment of barrier closure). The conditions allow then to model the bay as a discrete system with storage and resistance (see figure below), since the length of the bay is small compared to the wave length. There is formulation that relates the following parameters for a discrete system: amplitude of water elevations inside (ζ_k) and outside (ζ_z) the bay, surface of bay (A_k), surface of gap through which the water enters the bay (A_s). Since all the other parameters are known, it is possible to estimate the surface of the opening (A_s).

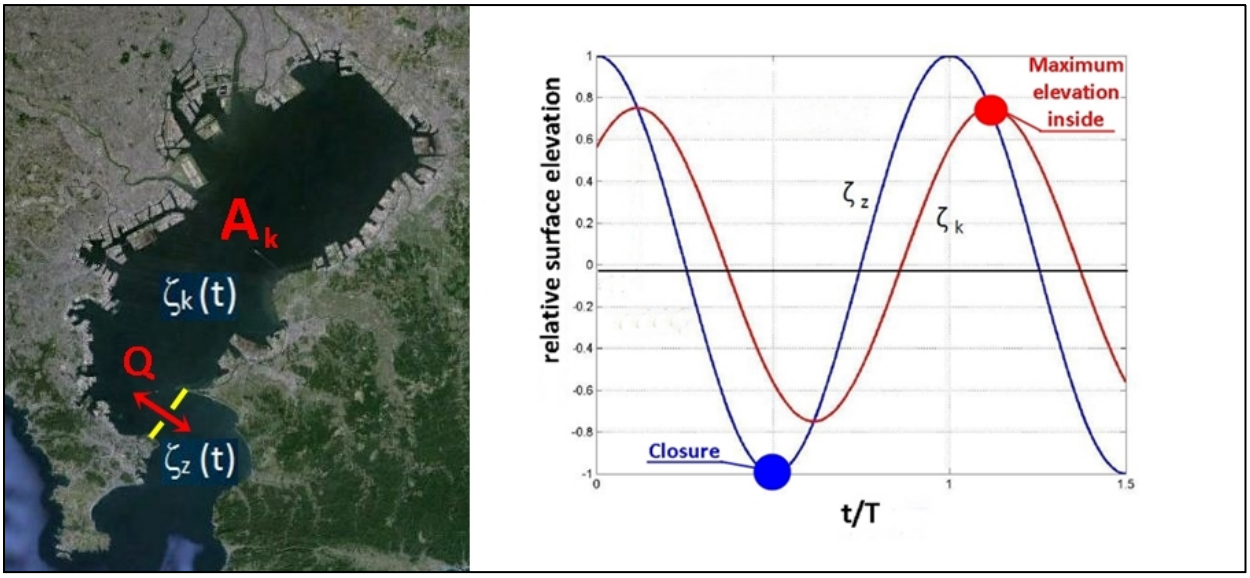

Figure 3-12: Discrete system approximation.

Using this approximation, the surface of the opening during the design typhoon is calculated for each barrier location and return period considered. The details can be found in A2-1. It is considered that part of the gates could fail during the closure; therefore, the opening during typhoon is going to be composed of the permanent opening and a 10% of the environmental section that remains open during storm due to failure of gates.

ENVIRONMENTAL OPENING

Later, the same discrete system approximation (see Figure 3-12) can be applied again to calculate the **environmental section**.

In this case, the condition to check is not a storm surge event, but operation under normal conditions. The barrier needs to have an open section that allows keeping a minimum tidal range inside the bay. Therefore, elevations inside and outside the bay are known. The only unknown now is the surface of the section through which the water enters the bay (As). The calculations are performed for all the considered barrier locations (see A2-2 for details).

Once the environmental section is known (opening during normal conditions) and the opening during typhoon is known, the gates surface and permanent opening can be obtained.

FINAL RESULT

As a result of all these calculations, permanent opening and movable barrier dimensions are set. A summary of the results can be found in the following table. Here, it has to be noted that the total opening during typhoon is composed of the permanent opening and 10% of the surface of gates (due to failure when closing).

Table 3-5: Storm surge barrier dimensions, return period 100 years.

DIMENSIONS RETURN PERIOD 100 YEARS					
	Environmental section (m2)	Permanent opening (m2)	Gates (m2)	Open length (m)	Gates length (m)
LOCATION 1	18,750	4,375	14,375	175	958
LOCATION 2	27,000	6,050	20,950	242	1,397
LOCATION 3	28,500	6,525	21,975	261	1,465
LOCATION 4	33,000	7,325	25,675	293	1,712
LOCATION 5	33,750	7,875	25,875	315	1,725
LOCATION 6	37,500	8,750	28,750	350	1,917

Table 3-6: Storm surge barrier dimensions, return period 500 years.

DIMENSIONS RETURN PERIOD 500 YEARS					
	Environmental section (m2)	Permanent opening (m2)	Gates (m2)	Open length (m)	Gates length (m)
LOCATION 1	18,750	1,875	16,875	75	1,125
LOCATION 2	27,000	2,925	24,075	117	1,605
LOCATION 3	28,500	2,775	25,725	111	1,715
LOCATION 4	33,000	2,950	30,050	118	2,003
LOCATION 5	33,750	3,500	30,250	140	2,017
LOCATION 6	37,500	3,750	33,750	150	2,250

These dimensions will be used to calculate construction costs for every different location and return period.

3.4. ALTERNATIVE SELECTION

First, total cost will be calculated for each alternative (section 3.4.1). Besides cost, other factors are taken into account in the comparison. In order to account for these factors, the alternatives are evaluated in a multi-criteria analysis in which they are given a score that represents their goodness (section 3.4.2). Finally, the ratio score/cost is calculated and the option obtaining the higher ratio is selected as the proposed solution.

The alternatives that will be compared here are:

- Doing nothing.
- Upgrading coastal defences.
- Barrier location 1.
- Barrier location 2.
- Barrier location 3.
- Barrier location 4.
- Barrier location 5.
- Barrier location 6.

For all of these alternatives, a return period of 100 years is taken (for the comparison with the upgrading of coastal defences to be valid). The analysis of the alternatives will give a total cost and a score for each one of them. Then, they will be compared and the best alternative will be chosen (see example in following figure).

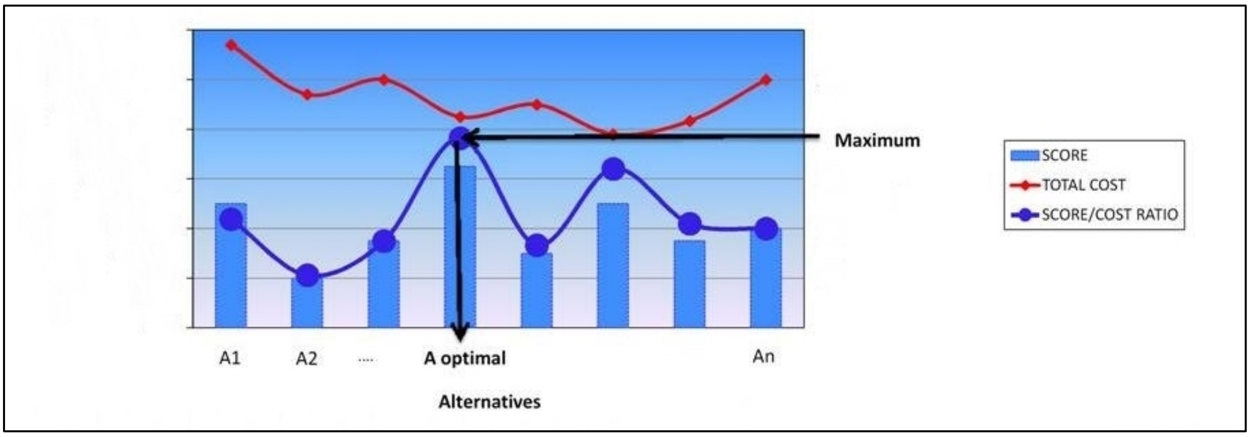

Figure 3-13: Conceptual graph of alternative selection using multi-criteria analysis and costs.

Lastly, for the most favorable barrier option, the comparison between two return periods (100 and 500 years) will be done in order to analyze the relation between total cost and return period.

3.4.1. COST-BENEFIT ANALYSIS

3.4.1.1. PROCEDURE

The aim is to find the total cost to take into account for each alternative. This total cost consists of the sum of investment and residual risk.

Only construction costs are included as **investment**. Other costs, like maintenance have been ignored since they are considered to be of similar range for all the alternatives. Furthermore the calculation would be complex and it is considered out of the scope of this Thesis.

Residual risk is the amount of risk that still exists after applying a specific measure. Risk is defined here as a measure of the expected damage, which is calculated as the damage that one event may cause multiplied by the probability of occurrence of this event. Here, the events are typhoons that cause different flood levels. As flood levels versus probability of occurrence is a continuous function, the total risk inherent to a specific measure comes from integrating the area under the curve of flood damage versus probability.

Adding these two concepts total costs are found and the alternatives can be compared (see following figure). In order to add the two concepts, the amounts are put in units that can be considered equivalent, namely equivalent quantities for year 2014. Therefore, construction costs prices are updated to 2014, and residual risk amounts are reduced to Net Present Value as well.

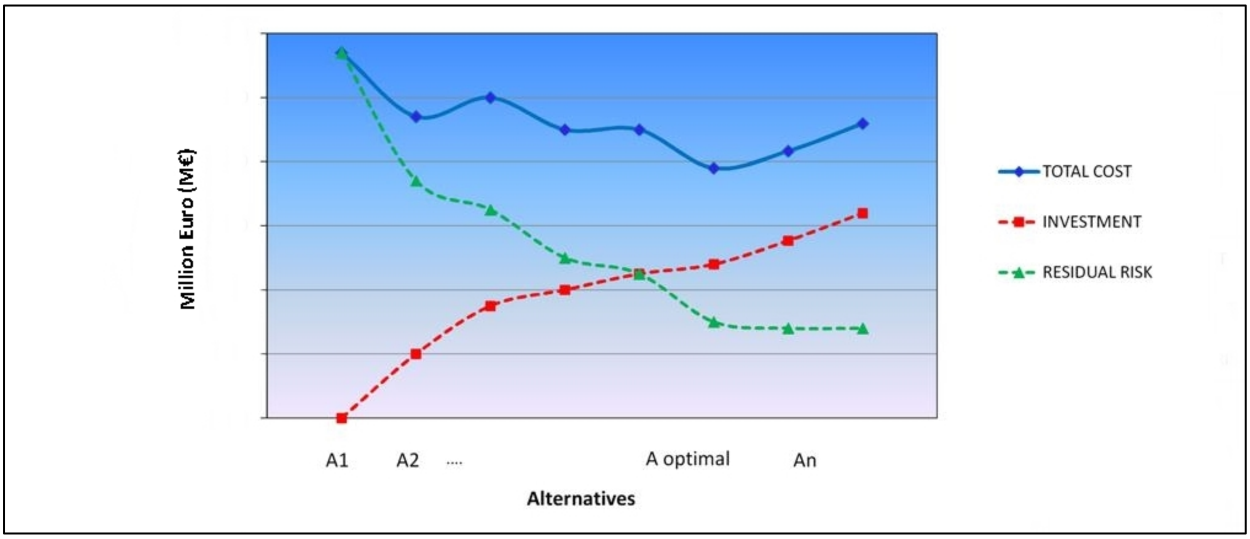

Figure 3-14: Conceptual graph of comparison of alternatives regarding total cost.

It has to be noted though that the final decision will not be only based in costs but will take other aspects into account. Finding the total costs is just the first step in the decision process. Later, a MCA will be performed in order to add other considerations to the final decision.

3.4.1.2. INVESTMENT

Investment in case of upgrading coastal defences is already known from Hoshino's MSc Thesis.

The investment related to the barriers needs to be calculated, for the different locations and return periods. As it has been stated before, the investment is considered to be equivalent to the construction cost.

In order calculate construction costs, the layout defined in 3.3.3.2 and dimensions previously calculated in section 3.3.3.4 are used here. The barrier has been divided in several parts, to give a sufficiently accurate result:

- Dam.
- Movable barrier.
- Extra cost due to earthquake resistance measures.

The cost of **dams** has been calculated dividing it in several elements for which the prices are known (see Table 3-7). Prices from international port projects are provided by French University ESITC Caen and several databases are used (CYPE Ingenieros S.A., 2014). The prices have been updated to 2014 and then translated to Japanese values using the price of concrete as reference (The Asahi Shimbun, 2013), which was a 80% of the prices for Europe. A 30% is added in the end, in account for indirect costs. This way, the total unit costs are found.

All the figures will be expressed in Euros from this point on, except the data coming from Hoshino's MSc Thesis, which is given in yens. This data is converted to Euros in calculations, using the following exchange rate: *1€=141.2¥* (this rate will be used for all currency translations).

Table 3-7: Unit prices dam.

ITEM		Unit direct prices Europe 2014 (€)	Unit direct prices Japan 2014 (€)	Unit total prices Japan 2014 (€)
- Dam				
Dredging (mud or sand)	m3	6.00	4.75	**6.18**
Quarry run/stones	m3	32.00	25.36	**32.96**
Concrete in armour layer H25/35	m3	364.27	288.65	**375.24**
Concrete in crown wall H25-35 (4m height)	m3	200.00	158.48	**206.02**
Bituminous pavement º	m2	16.50	13.07	**17.00**

These prices have been applied to the material volumes of the dam, giving a total cost for it.

Regarding cost of **barriers**, the prices (meter length*meter height*water head) are provided by Professor S.N. Jonkman. These prices have been applied to the dimensions previously calculated.

Finally, **earthquake reinforcement** is roughly estimated as a 10% extra cost. At this stage this is considered a sufficient margin, since for buildings the figures are between 1 and 5% (Kawashima, 2013).

The detailed calculations and explanations can be found at A3-2, in Appendix 3. A summary of the results can be found in the following table.

Table 3-8: Investment for every alternative.

RETURN PERIOD	ALTERNATIVE	INVESTMENT (M€)
100 years	DO NOTHING	0.00
	UPGRADE DEFENCES 1/100yr	2751.24
	BARRIER LOCATION 1 1/100yr	4465.05
	BARRIER LOCATION 2 1/100yr	5158.81
	BARRIER LOCATION 3 1/100yr	5289.96
	BARRIER LOCATION 4 1/100yr	6295.92
	BARRIER LOCATION 5 1/100yr	5533.74
	BARRIER LOCATION 6 1/100yr	6391.67
500 years	BARRIER LOCATION 5 1/500yr	7113.98

Since location 5 is expected to be the most favorable one (shortest stretch to close, largest area protected), the investment has been calculated as well for a return period of 500 years. This will allow analyzing later which return period is more interesting, in case a barrier is built.

3.4.1.3. DAMAGE

The data found in Hoshino's MSc Thesis (see Figure 3-15) is used in this section. In Figure 3-13, storm surge elevation is linked to damage, for Tokyo and Kanagawa prefectures.

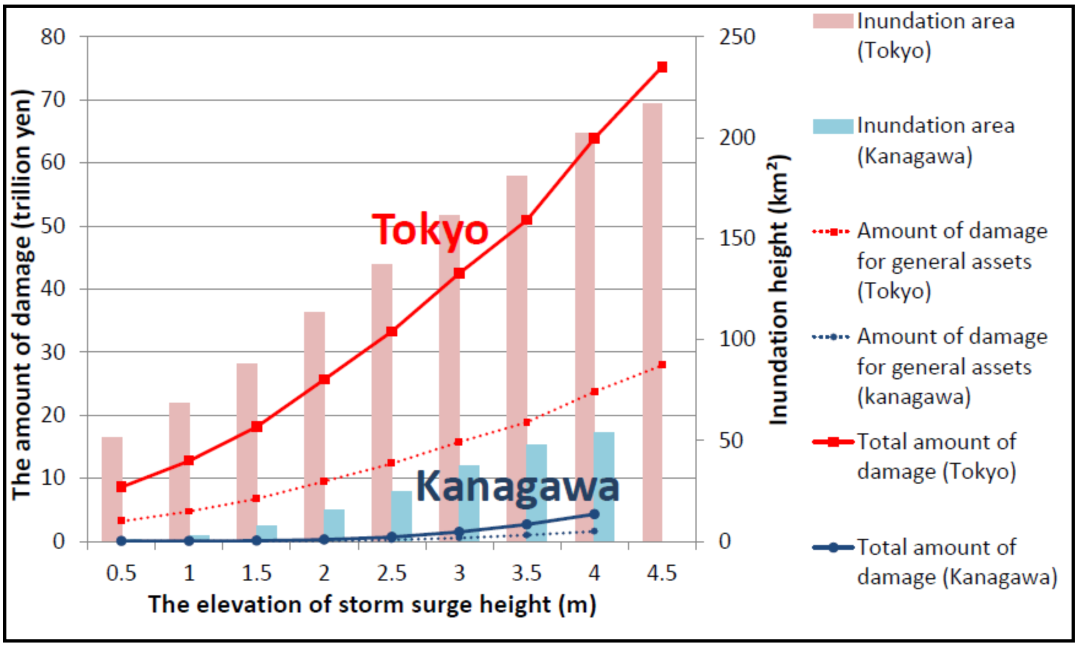

Figure 3-15: Amount of damage for different water levels Tokyo and Kanagawa (Hoshino, 2013).

This graph has been completed the following way:

- Storm surge elevations have been linked to absolute water level elevations and to return periods using the graphs calculated by Kawai (see Figure 2-9). Also, an extrapolation to slightly larger water levels has been done, in order to cover levels corresponding to the 1/1000 years storm surge (the aim is to work with values up till 1/500 years, therefore the estimation should reach or surpass this value).
  The 1/1000 years storm surge is set as the maximum possible damage for simplicity. The sensitivity of results to this chosen maximum has been checked and variations are very low, so the approximation can be considered sufficiently accurate for this analysis. For example, taking 1/10000 years instead of 1/1000 years leads to differences in damage values of about 2-3% (see Figure 3-17 in next section, for a graphical representation).
  The result can be found at Table 3-9, where the **relation between probability of occurrence and water levels** is given. These water levels can be directly related to the ones in Figure 3-15. For example, the event with a return period of 100 years is related to a water level of 4.5 m in year 2100 and to a damage valued at 75 trillion ¥ in Tokyo.

Table 3-9: Water levels during typhoon (storm surge + tide) for year 2100.

T (yr)	NOW		YEAR 2100
	Storm water level above LWL (m+A.P.) *From Kawai*	Storm water level above MWL (m+T.P.)	Storm water level above MWL (m+T.P.) *adding SLR=1.90 m*
10	2.70	1.57	3.5
20	2.95	1.82	4.0
50	3.20	2.07	4.2
100	3.50	2.37	4.5
200	3.70	2.57	4.7
500	4.20	3.07	5.2
1000	4.50	3.37	5.5

- Only Tokyo and Kanagawa prefectures have been included in the estimation of damage in Figure 3-15. These prefectures cover the western part of the bay. To account for the complete bay coastline, the prefecture of Chiba has to be included as well. An **estimation of damage for Chiba** prefecture has been added, using maps of flooded surfaces. The amount of damage has been estimated from the values of Tokyo for damage per flooded square meter. A coefficient is applied to this ratio, accounting for relation in population density between the two prefectures (it has been taken roughly as 1/10). It has to be noted here that the approximation made using flooded surfaces is far from accurate. However, due to the small percentage that Chiba represents in the total damage, the estimation is accepted and added to the final result. The detailed calculation can be found at A3-3 in Appendix 3.
- Finally, an **estimation of losses associated to casualties** has been done. According to a Japanese Government estimation, for an inundated area of 280 km^2 in Tokyo, up to 7600 deaths would be expected (Japan Times, 2013, The Yomiuri Shimbun, 2010).
  A check on these figures can be performed using loss of life estimations for The Netherlands (Jonkman, 2007). Using a population density of 6.000 people/km2 (Tokyo Metropolitan Government, 2010) for Tokyo, a flooded area of 280 km^2 would affect 1,68M people. Considering that 20% can be evacuated and 20% can find shelter, 1M people would be exposed and casualties would be about 0.5%, that is to say 5376. This value is quite close to the provided by Japanese Government and it is decided to use an average value of 6500 deaths corresponding to 280 km^2 of inundated area (23casualties/m^2). For the different possibilities considered in this report, the number of fatalities will be extrapolated in order to assign a value to the different inundated surfaces (weighing this value using population density in different prefectures as well). More details can be found at A3-3 in Appendix 3.

With all the above considerations, a relation between return periods (probability) and damage for each affected prefecture has been developed (see Figure 3-16). This relation can be used to draw residual risk graphs, as it will be explained in the following section.

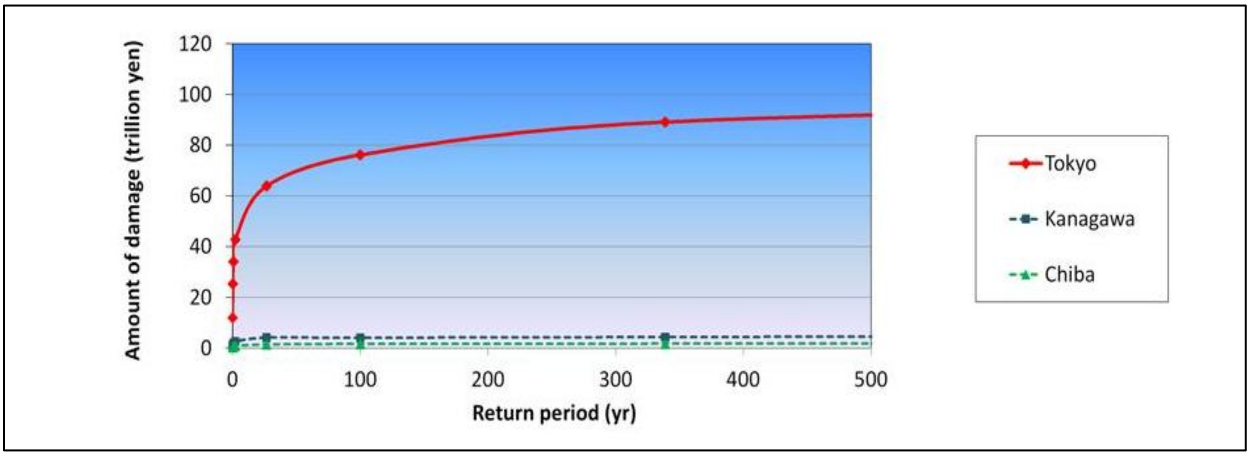

Figure 3-16: Damage for prefectures of Tokyo, Kanagawa and Chiba, in relation to storm surge return period.

3.4.1.4. RESIDUAL RISK

Once the material damage is known, residual risk for every alternative (do nothing, upgrading defences, barriers) is calculated.

The relation between probability of occurrence and damage has already been obtained in previous section (Figure 3-16). Through this, the associated risk can be calculated. The total risk for the case considered will be the area under the curve that represents damage as a function of probability of occurrence.

For an alternative that increases the level of protection in an area, the total risk is reduced in respect to the previous situation. In case the alternative is protecting against an event of probability of occurrence p, the damage function changes. For all events with probability of occurrence larger than p, the damage will be zero; therefore the total area under the curve is reduced. For a graphic explanation of this concept, see the following figure.

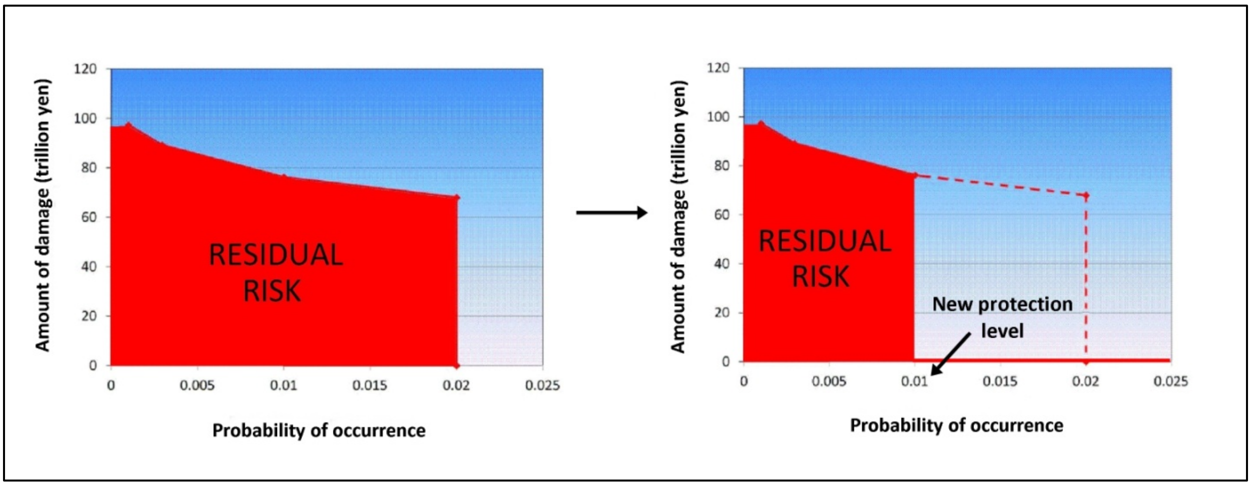

Figure 3-17: Residual risk corresponding to a protection designed for a return period of 50 years.

It has to be noted that, for all the alternatives, the existing coastal defences are assumed to remain there and operational, also in case a barrier is built. This assumption is made since it is difficult to estimate the state of the defences by the time the barrier is built. This means that the start of damage corresponds to the water level that surpasses the elevation of these coastal defences. For Tokyo area, which is taken as reference, the elevation of coastal defences is around 5,30m +A.P. (see Table 3-1), which is equivalent to +4.3 m T.P. As can be seen in Table 3-9, this is between 50 and 100 years of return period. Thus, 50 year has been chosen as the minimum return period for which there is damage, which is considered a conservative decision.

Each studied option leaves some areas unprotected, where the amount of risk remains constant, and a protected area, where there is a reduction in risk (due to the increase in protection level). Total risk for each alternative is the sum of these two parts (see Figure 3-18).

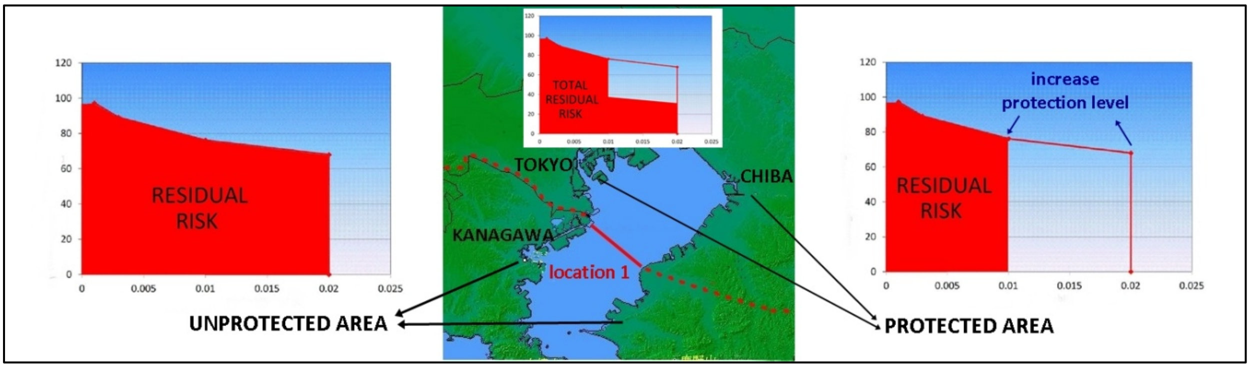

Figure 3-18: Residual risk for a specific barrier location.

Residual risk is calculated as the area under each damage curve. The total risk for each alternative is the sum of the two parts, protected and unprotected area. Residual risk calculations have been done for design return periods of 100 and 500 years. The largest amount of risk (area below damage curve) is concentrated under return periods from 50 to 1000 years. This concept is illustrated in Figure 3-17, where it can be seen that already a 50% of risk (area under the curve) is reduced when increasing the protection level from 50 to 100 years (probability 0.02 to 0.01). Increases in protection level larger than 1000 years would cause a small risk reduction in percentage. Detailed calculations for each location can be found at A3-3 in Appendix 3.

Due to the level of accuracy of the available data, it has been considered that calculating total residual risk for each alternative would not add much to the analysis. Therefore, only locations 1 and 5 have been calculated. For locations 2,3 and 4 the risk is calculated interpolating between 1 and 5 (taking into account the length of protected coastline for each one). For location 6, the risk is considered to be the same than for 5, since the protected assets are the same (the increase in protected coastline means nothing in terms of risk, since it is an area that does not have large settlements).

When the residual risk (from material damage and loss of life) is calculated for each alternative and return period, the quantities are added and reduced to Net Present Value. This has been done considering a growth rate of 1%, an interest rate of 2% and 100 years for barrier lifetime.

Also, a percentage accounting for indirect losses is added. In this case, a factor of 2 is used to calculate it (Hallegatte, 2008). The results can be found in the following table.

The detailed calculations can be found at A3-3 in Appendix 3. A summary is shown in the following table.

Table 3-10: Residual risk as per action alternative and return period.

ALTERNATIVE	Net Present Value Direct Losses (M €)	Indirect losses (Direct Losses *2) (M €)	TOTAL RESIDUAL RISK (M €)
Do Nothing	3801	7602	**11403**
Upgrading 100yr	2469	4938	**7407**
Location1 100yr	2519	5039	**7558**
Location5 100yr	2454	4908	**7362**
Location5 500yr	560	1121	**1681**

Adding the residual risk to the investment previously calculated in 3.4.1.2, an estimation of the total cost of each alternative is found. The total cost will be used in the multi-criteria analysis (see next section) for choosing the most favorable solution.

3.4.2. MULTI-CRITERIA ANALYSIS

3.4.2.1. PROCEDURE

In order to select the most advantageous alternative, a multi-criteria analysis is performed. The objective is to assess the suitability of the different alternatives. For every alternative there are a number of factors that contribute to its goodness and whose influence has to be quantified, by means of giving it a score. Once all the influences are quantified, it is possible to make a decision on the most favorable alternative, based on the numerical results obtained in the analysis. For the MCA methodology, guidelines from "Multi-criteria analysis: a manual" (Government, 2009) are adopted.

The chosen method to perform the analysis is the Linear Additive model, which is conceptually simple. The basis of the analysis is to add a relative weight to each criterion that takes part of the analysis. This weight will multiply the score given to this criterion.

For criteria definition and their weighing PIANC guidelines (InCom, 2006) are used as model. The final criteria choice and their relative weights can be found in the following table.

Table 3-11: Criteria for evaluation of alternatives and relative weights.

CATEGORIES	CRITERIA	Weight
TECHNICAL	Solution reliability	0.29
	Operations	0.17
	Navigation	0.17
	Maintenance	0.07
ENVIRONMENT	Currents/sediments	0.02
	Pollution	0.01
	Landscape	0.05
	Land occupation	0.04
SOCIAL	Population feeling protected	0.10
	Other social	0.08

Each alternative is given a score for every criterion, which represents its level of performance relative to this criterion. For that, a numerical scale is created. For each number in this scale a precise level of performance is defined. In the following table, the numerical scale for assigning scores is presented.

Table 3-12: Scale for assigning scores to evaluation criteria.

SCORES	ALTERNATIVE PERFORMANCE
4	ideal conditions
3	good conditions, without significant problems
2	few problems that can be solved
1	problems difficult to resolve
0	problems that could proscribe location

Once every alternative is given weighed scores for all the criteria, these weighed scores are added, resulting in a performance matrix where each alternative gets a mark. The highest mark indicates that the alternative is better than the rest, regarding the analyzed criteria. Detailed assignment of scores and performance matrix can be found at Appendix 4 . The final score given to each alternative can be found below, in Table 3-13.

3.4.2.2. RESULTS

A final index is calculated and used to choose the best alternative. This index is the “value over costs ratio” (Ligteringen and Velsink, 2012). This allows introducing the previously calculated costs in the analysis and deciding which alternative is the most favorable.

Results from scoring all the alternatives are modulated using the total costs obtained in 3.4.1. These results are summarized in the following table. It should be reminded that only for locations 1 and 5 the calculations of residual risk were performed, the rest (grey in the table) are extrapolations based on the different degree of protection that each alternative provides (length of coastline covered).

The benefit over cost ratio has also been added to the table, as it is considered to be useful for the evaluation of alternatives. For each alternative, benefit is defined as the difference between initial (equivalent to do nothing) and residual risk. For this ratio, cost is defined as the investment needed.

Table 3-13: Multi-criteria analysis performance matrix, T=100 years.

RETURN PERIOD	ALTERNATIVE	INVESTMENT (M€)	RESIDUAL RISK (M€)	TOTAL COST (M€)	BENEFIT/ COST RATIO	SCORE	SCORE /COST RATIO
100 years	DO NOTHING	0	11403	11403	-	1.61	**141**
	UPGRADE 1/100yr	2751	7407	10159	**1.45**	2.41	**237**
	LOCATION 1 1/100yr	4465	7558	12024	**0.86**	1.60	**133**
	LOCATION 2 1/100yr	5159	7480	12639	**0.76**	2.12	**168**
	LOCATION 3 1/100yr	5290	7460	12750	**0.75**	2.07	**162**
	LOCATION 4 1/100yr	6296	7401	13697	**0.64**	2.17	**158**
	LOCATION 5 1/100yr	5534	7362	12896	**0.73**	2.28	**177**
	LOCATION 6 1/100yr	6392	7362	13754	**0.63**	2.15	**156**

It can be seen that the upgrading of coastal defences gets the highest punctuation, being the best alternative regarding the score/cost ratio. This option gets also the best benefit over cost ratio.

Among the different barrier locations, number 5 scores the best. This is the reason why location 5 has been analyzed for larger return periods, in order to see whether total costs are reduced.

As the reduction in residual risk is substantial between 100 and 500 years for location 5 (see Table 3-10), the total cost decreases for 500 years. The comparison can be seen in the following figure. It can be appreciated how taking a larger return period results in a significant reduction of total costs.

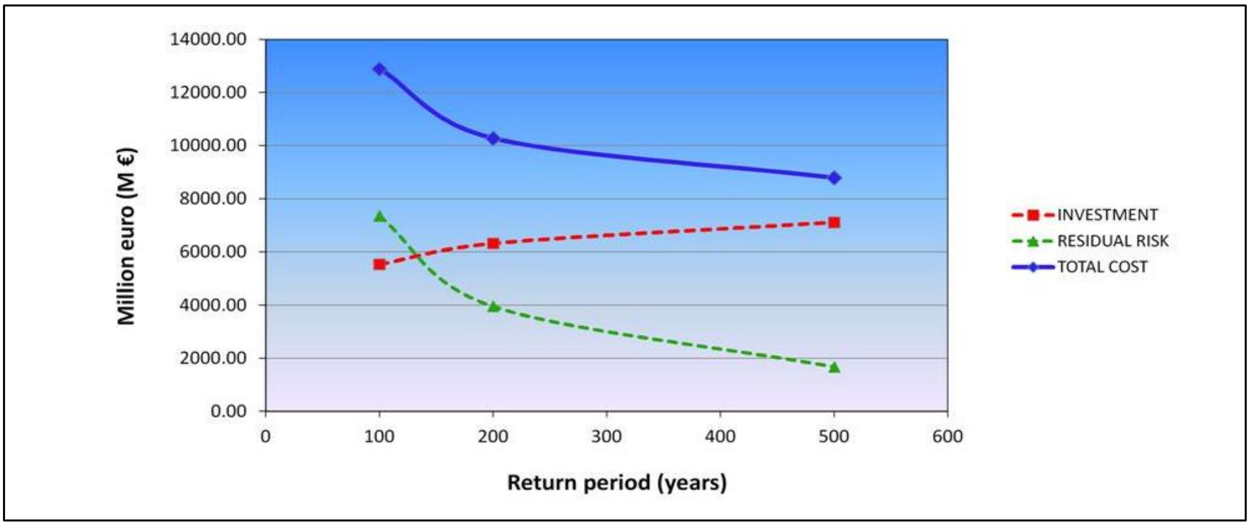

Figure 3-19: Total costs location 5 for different return periods.

Since it seems that considering a return period of 500 years is a better option for the barrier, the comparison between the barrier (just location 5 this time) and the other alternatives is done for this return period. The results are shown in the following table.

Table 3-14: Multi-criteria analysis performance matrix, T=500 years.

RETURN PERIOD	ALTERNATIVE	INVESTMENT (M€)	RESIDUAL RISK (M€)	TOTAL COST (M€)	BENEFIT /COST RATIO	SCORE	SCORE/ COST RATIO
500 years	DO NOTHING	0	11403	11403	-	1.61	**141**
	UPGRADE 1/500yr	??	1680.94	??	-	2.41	-
	LOCATION 5 1/500yr	7114	1681	8795	**1.37**	2.28	**259**

In this case, the comparison is not complete, since there are no data for the upgrading of coastal defences. The barrier is clearly preferred to the option of doing nothing. The benefit over cost ratio is favorable for location 5 too, being the reduction in risk larger than the necessary investment.

3.5. ANALYSIS OF THE OUTCOME

3.5.1. CONCLUSIONS

An analysis has been done on present and future performance of coastal defences in Tokyo Bay. According to the results, these defences give sufficient protection nowadays, considering typhoons up to 1/500 years. However, due to sea level rise and increase in typhoon intensity, it is possible that these **coastal defences will need an upgrading** in the future to keep preventing floods.

Even though this is a rough approximation, the obtained results are clear enough to state, before going into more detail, that **taking action is cost effective (less costly than doing nothing) ,** since there are options with lower total cost than doing nothing. This holds for both studied return periods, 100 and 500 years (see Table 3-13 and Table 3-14).

Looking at residual risk graphs, it is clear that choosing a return period larger than 100 years (Japanese are already considering 1/200 years for river floods) a significant reduction in risk is achieved. Since the increase in investment is small compared with risk reduction, improvement of flood protection implies a **benefit which increases when considering larger return periods** in design.

Regarding which solution is best; at first glance the barrier built in location 5, for a return period of 500 years seems to be the best option. However, the upgrading of coastal defences for this return period has not been calculated so it is not possible to make a proper comparison. At **first glance upgrading coastal defences seems to be more economical than building a barrier**, according to the data available for comparison (1/100 years alternatives). The difference between upgrading coastal defences and a barrier for a return period of 100 years is of 2,700 M€, from a total cost of 12,800M€ (about 20%). This is a margin that could be reduced if a more accurate study of costs.
Regarding this last consideration, some important remarks need to be done:

- In case a return period of 500 years is chosen, it is expected that upgrading of coastal defences would be needed in a longer stretch than what was calculated in Hoshino's MSc Thesis. Design water levels will be 0.5 meters higher (H. Kawai, 2008) than the values used for cost calculation of upgrading coastal defences included in this report. This would mean 0.5 meter extra of rising sea defences. The total length that would need this protection is unknown, but the total coastline protected by barrier in location 5 is more than 240km. Taking figures for the Netherlands, increase 1 meter in urban area would cost about 18M€/km and 9M€/km in rural areas (Jonkman et al., 2013). Considering urban area a 50% of the coastline, this would mean an extra investment of 1,650M€, while the difference for the barrier is about 1,600M€. Therefore, the increase in investment cost is the same order for both alternatives. Nevertheless, there are other factors (see below) that could increase the total cost of upgrading coastal defences.
- As there is a large proportion of coastline used as port and industrial area, operations between sea and land take place frequently and certain "permeability" of the coastline has to be ensured. At certain points dikes and flood walls could be an obstacle for this. Hence, movable barriers could be necessary at some points instead of dikes or flood walls (increasing largely construction and operational costs).

- In the affected industrial areas, not only a ground elevation or dike enlargement would be needed, it is possible that certain facilities need adaptation and this should be accounted for in costs estimations.
- The cost of upgrading coastal defences itself could be underestimated. For example:
    - It is considered an elevation of soil in port areas, operation that cannot be done at once for a whole port and presumably will include some phasing in the process. Therefore, these partial platform build ups will entail temporal adaptation of facilities (roads, pipelines, etc) and its ulterior demolition, which is an extra cost.
    - Not only walls but movable gates or access to the coastline might need upgrading, at a higher cost than a mere concrete wall.
    - Heighten the levees will reduce the “permeability” of the land-sea interface. This will also involve costs and inconvenience, which are difficult to quantify without further analysis.
- Movable barrier is the main cost driver, representing up to 73% of total investment (location 5 and 6, see A3-2). In this report, the included calculations to determine their dimensions are very rough, so it is expected that more accurate definition of water flow and levels could lead to some reduction in barrier costs.
- The chosen return period is important to decide which solution is better. It is expected that, at some point, the barrier will be cheaper than coastal defences upgrading. Some more centimeters are a small increase in the barrier cross-section, while for coastal defences, the situation gets more complicated. Solutions will be more difficult to execute, as a large percentage of the coastline to protect are industrial ports (dikes cannot be simply enlarged, there might be need for movable defences that allow port operations and this will increase significantly construction and maintenance costs)

Taking into account the previous considerations it is expected that, **at some point, a barrier would become the best solution**.

Among all studied alternatives for the barrier, **location 5 is the most favorable**. There are disadvantages for locations 1, 2 and 3, which protect only part of urban settlements and interfere with activities that are developed inside the bay. Among remaining options, 4, 5 and 6, location 5 presents the lowest investment and minimum total cost. Location 6 only allows protecting an extra area with no industrial facilities or urban settlements. Location 4 on the contrary could be an option worth to consider in case the depth in location 5 proves to be an important obstacle (so far, prices for both alternatives have been taken the same, considering no extra costs generated by going from 60 to almost 80 meters depth). It is considered that the difficulty added by extra depth will be compensated by the larger reduction in risk that location 5 achieves (the difference between the two areas is that location 5 protects Yokosuka area).

Other remarkable reflections regarding the performed analysis are:

- Comparisons here are done for a sea level rise of 1.90 meters. The value considered reasonable for design would be around 1.50 meters, in the light of new predictions from Climate Change experts. Still, the values are considered to be close enough for this analysis to be valid.

- Peak river discharges are expected to increase in some areas in the future, in case urban development continues (increase in run-off coefficients). This should be taken into account in a more detailed study about water levels definition.
- In case new urban or industrial settlements are foreseen in the area between location 5 and 6, location 6 could be an interesting option, since the building costs for the barrier are almost the same than for location 5.
- The calculation for barrier dimensions and costs is based on setting the freeboard of the dam first, and then calculating the permanent opening. It has been considered that further investigation in this direction would not add much to the aspects that this thesis is analyzing. However in a real case study, it would be interesting to calculate how an elevation of the freeboard influences the total building costs. An elevation of the freeboard implies larger material volumes in the section and larger construction costs. On the other hand, it allows for a larger permanent opening (due to the reduction on overtopping) and this means a reduction of material volumes as well (which could be particularly interesting in case this means a reduction on gates surface).

A more extensive and precise analysis needs to be done about this subject, since the approximation made is very rough and the obtained results are quite close for different options. This is a first estimation aiming to give a good overview of the situation, in order to tackle a conceptual design for a storm surge barrier.

As a conclusion, compared to inaction or upgrading of coastal defences, a storm surge barrier for flood protection in Tokyo Bay is an option worth to consider. However, a more detailed analysis is needed to arrive to a final conclusion. It seems that 500 years return period is more favorable that 100 years, but for this safety level, the cost of upgrading coastal defences is unknown, so it cannot be compared to the option of building a barrier. Besides that, it has to be noted that the decision is strongly dependent on the chosen SLR scenario; the barrier provides a good solution for a specific range of rise in sea level. This is due to several reasons:

- It is only from certain amount of SLR (+1.50 m) onwards that the coastal defences height is surpassed by the design storm surge, and the need for measures arises.
- In case SLR arrives to certain levels, storm surge built up inside the barrier would reach the coastal defences height, despite the presence of the barrier. For the design conditions, the allowable increase in water level inside the bay is around 1 m during a storm. Therefore, in case the SLR arrives to one extra meter, sea defences will be reached even with a complete closure of the bay. This means that for SLR larger than 2.50 m, an upgrading of coastal defences (or other extra measures) would be needed anyway.

Taking into account all the above considerations, location 5 is chosen to develop the barrier conceptual design, for a return period of 500 years.

3.5.2. PROPOSED BARRIER LAYOUT FOR DESIGN PHASE

According to the previous section conclusions, location 5 has been selected as the location to develop a conceptual design.

Several criteria taken into account in the layout definition are:

- Navigation channel (permanent opening) located in same position as current channel whenever possible.
- Movable barrier located at shallow areas, so a large dam under it is avoided. This decision could change when having more information about soil characteristics, as there could appear an area which is recommended to avoid.
- Movable barrier located close to permanent opening. Since the navigational channel is composed of an opening and gates, it in order to keep all the gates together, the permanent opening should be located close to the gates.
- In areas with large depths, a composite section with caissons is considered.

For the chosen return period, the permanent opening has a length of 140 meters (see Table 3-5), which is shorter than the necessary navigational channel width (625 meters). Therefore, the navigational channel would need an additional stretch where gates would be installed, so it can be closed in case it is needed.

The movable barrier would be located at the shallow part of the cross-section, if possible. This way, large depths of dam under the gates foundation are avoided. Besides that, there are seaweed farms located in the shallow platform close to Cape Futtsu, so the gates proximity can help to keep good conditions for seaweed growing; facilitating water exchange in the area (this point would need further study).

Finally, a dam will close the rest of the cross section. The dam is defined as a rubble mound section, but in the deepest areas (more than 50 meters), the option of using a composite cross section (including caissons) is considered in order to save material.

The main issue posed by this cross section is the large depths. An interesting option could be to slightly change the layout to avoid the deepest area. Going from a length of 7000 meters to 7,600 meters, the maximum depth is reduced from 80 meters to 60-65 meters (see following figure, yellow line).

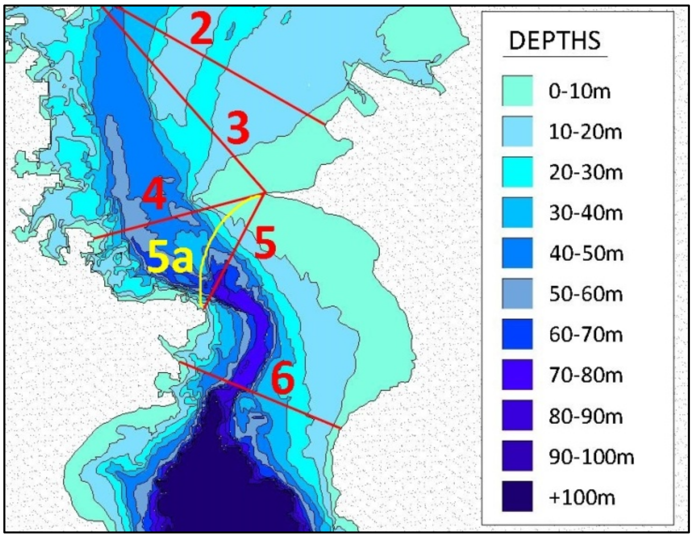

Figure 3-20: Location 5 variant.

Given the more favorable conditions, location 5a is selected to be developed in the design phase.

The focus of the design phase will be the dam. Despite the use of caissons, there will be dams up to 40 meters. Therefore, the main issues to solve for the dam are **large material volumes** and **earthquake resistance**.

A detailed longitudinal section of location 5a can be observed in Figure 3-21. Permanent opening and movable barrier cross-section are indicated as well.

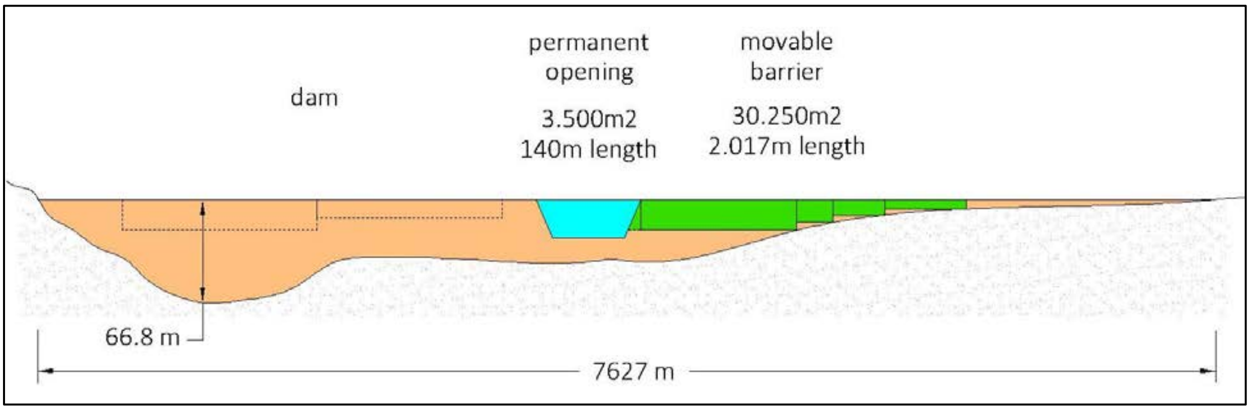

Figure 3-21: Detailed layout location 5a.

4. DAM DESIGN

4.1. INTRODUCTION

The structure of this chapter is shown in the following graph.

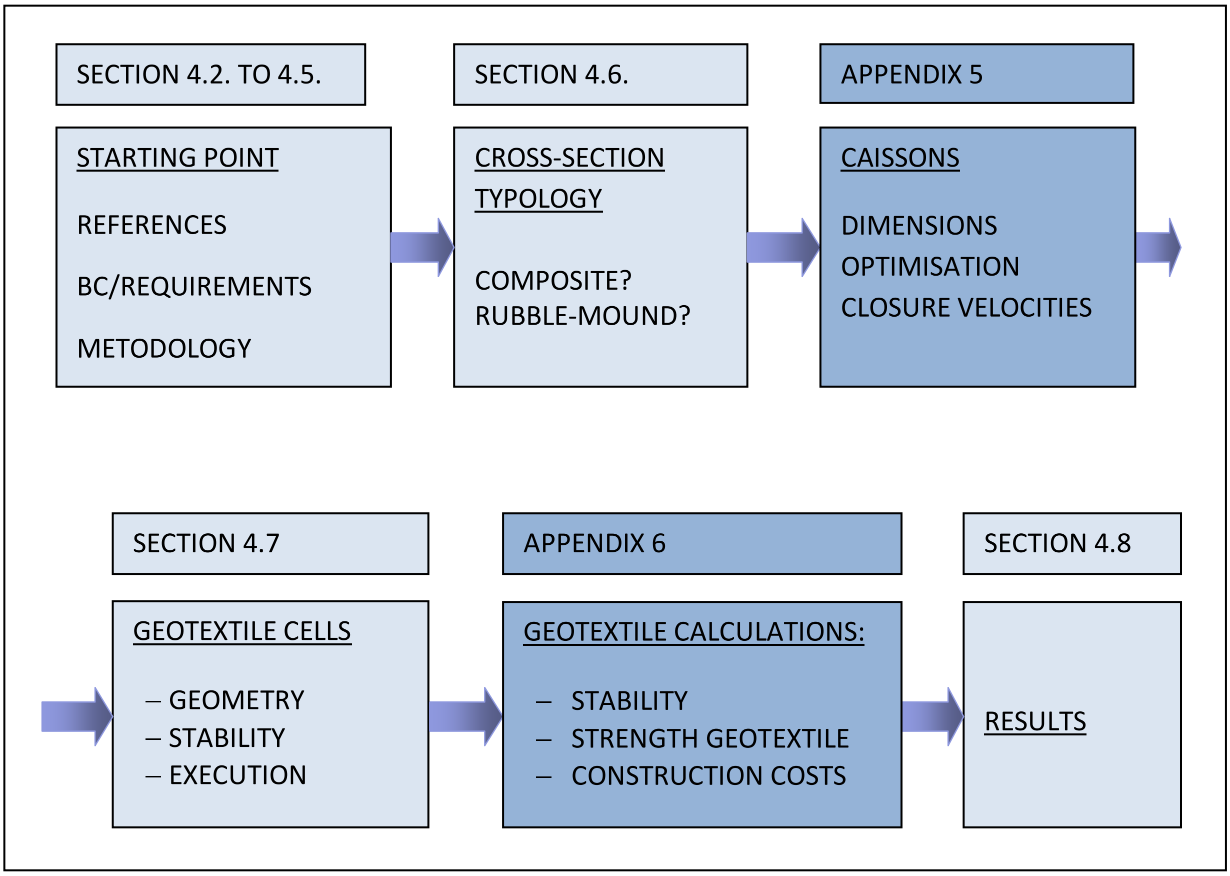

Figure 4-1: Chapter 4 structure.

This chapter is devoted to the definition of a conceptual design. The dam that is part of the barrier is the element studied here. The aim is to introduce innovative components that allow construction at sufficiently low costs, as well as optimizing the structure performance for the specific conditions of this case study, namely large depths and high seismicity.

First of all, information about similar structures is given as reference. Then, requirements and boundary conditions are described in order to proceed with the definition of cross section. The cross section definition includes a study of the suitable typology, a proposal for an innovative design and the stability calculations needed to confirm the validity of the proposal. Later, construction process and costs are studied in order to confirm the feasibility of the design.

4.2. REFERENCES

4.2.1. PREVIOUS STUDIES

In case a storm surge barrier is considered as a solution for protection against flooding in Tokyo Bay, the main difficulty faced is probably the large dimensions. These dimensions imply large construction costs and vulnerability to earthquakes (which will result in large costs as well). There are studies related to structural behavior of similar elements when affected by earthquakes (Selcuk et al., 2012). There are also more general compilations of knowledge regarding marine structures and seismic design (see 2.8.2). In general, recommendations for a good seismic performance are focused on the use of high quality material, densification of soils or improvement of permeability conditions.

Regarding the optimization of the dam, there are studies and documented experiences about the use of a cheap material as filling (see Figure 4-2). An example can be the use of geomats filled with sand (Yan and Chu, 2010, Kan and Liu, 2009). Regarding the seismicity issue, it is of special interest the design developed by A. Van der Plas in his MSc Thesis (A. van der Plas, 2007). The thesis is focused on the design of a tsunami barrier, where the problem of earthquake resistance is addressed. It is proposed the use of geocontainers to improve the sand core seismic performance. The confinement of sand is expected to reduce the consequences of liquefaction. The applicability of this technology to this case study will be considered later in this report, comparing it with the proposed solution.

Other application of geosynthetics allowing cheap material filling can be the use of vertical cells (Pilarczyk, 2000). Vertical cells are developed in two main ways:

- Vertical cell web, supported by steel frame. These elements are called "superbags". It was only tested in the past for small elements and there are no further developments, at least related to the present case of dams situated in large depths and open sea.
- Hydraulically-filled geomembrane bags for land reclamation.

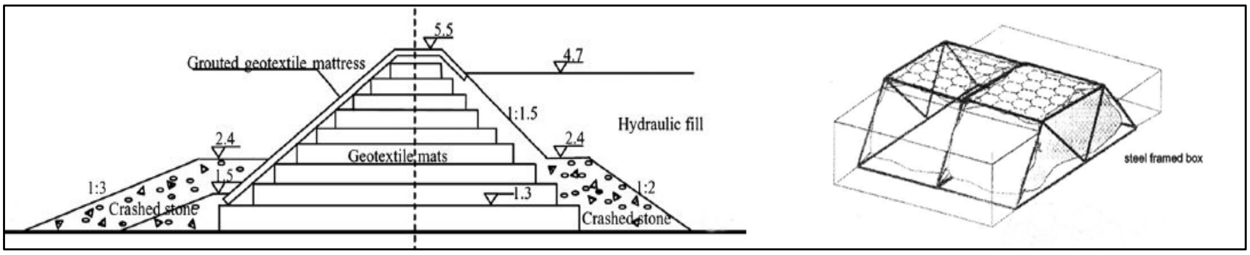

Figure 4-2: Geomats (left) and "superbags" (right) applied to construction of dikes and dams.

All these concepts will be explored in order to decide on their applicability to this case of study.

4.2.2. PROVEN TECHNOLOGY

Techniques used in closures that have large depths are of interest for this case of study. Also, of special interest are technologies that are developed for similar type of structures, in order to improve performance related to earthquake or tsunami.

Regarding dams or breakwaters located in **large depths**, it is frequent the use of a composite cross section (a rubble mound base and a concrete caisson on top). Several examples of composite breakwaters are found in Japan (Kamaishi Breakwaters have a maximum depth of 63 meters), or in areas with steep continental shelves (like the volcanic Canary Islands in Spain, with several breakwaters located in depths larger than 50 meters). This layout allows for large savings in material and compensates for the use caissons, which are expensive.

The structural resistance to **earthquakes** is a topic that has been largely studied, mainly in areas with high seismicity, like Japan or California, for example. Measures for dams are summarized in studies and guidelines, like "Seismic design and performance criteria for large storage dams" (Wieland, 2012). The more widely applied are the following:

- Avoid the use of material predisposed to liquefaction.
- Substitution of materials in the foundation.
- Use of drains and filters.
- Excess of material in the crest to compensate for expected settlements.

Substitution of materials, use of drains and filters are applied to avoid liquefaction of foundation materials. Regarding the core of the dam, the advice is to avoid certain materials or adding extra volume to compensate for future unwanted settlements that can occur in case of earthquake.

Another way to improve the characteristics of a sandy soil is the use of vibrolances to make a dynamic compaction and achieve a densification. It allows to reach a 70-85% of relative sand density (Haywardbaker, 2014). The effects of an earthquake in sand with these densities are supposed to be less destructive, since:

- Settlements will be lower.
- Liquefied percentage of the soil will be less.

There are studies that confirm the positive effect that compaction has on sandy soils regarding the consequences or large earthquakes. There is documented evidence deriving from past earthquakes in Japan that densification can be more effective than measures aimed to dissipate water pressures (drains) (Harada et al., 2014).

Regarding **tsunamis**, the concept of closing a bay to reduce tsunami heights inside is already known in Japan. It is precisely the failure of one of these structures, the Kamaishi breakwaters, which has lead to specific research on how to improve the performance of composite cross sections under tsunami loads (Matsushita, 2013). In case of a composite cross section, the largest damage due to a tsunami is produced by the sliding and loss of the caissons on top. This sliding failure has been studied and it has been concluded that it can be avoided by placing a dissipative structure on the inner side of the caisson. By doing so, the wave energy is dissipated, not causing a scour hole that is responsible for the failure of the caissons most of times. A study of scour holes dimensions due to tsunamis has been done for the Tohoku earthquake (Bricker et al., 2012).

4.3. REQUIREMENTS

4.3.1. FUNCTIONAL REQUIREMENTS

General requirements are studied in the System Level Analysis chapter. Some of them apply to the design of the dam:

- Structure lifetime: 100 years.
- The structure has to be stable during construction and in operational conditions.

- The structure has to be earthquake resistant, up to an earthquake of several hundred years return period (see Japanese standards for port and harbour facilities). This is the standard for high seismic resistant structures that need to retain their function after the earthquake.
- Safety level regarding storm surge: 500 year design return period of typhoon, meaning a joint value of 3.7 m for storm surge and tide together.
- Freeboard of 2 meters during a storm surge event, a requirement given by the limitation to overtopping values.
- The top of the dam should be accessible for vehicles, in order to reach any point with heavy equipment if needed (maintenance, repairs). A road is foreseen to satisfy this requirement. The minimum elevation is chosen 1 meter above high water level, therefore is set at + 6.60 m A.P. (1.90 m SLR + 3.70 m storm surge and tide + 1 m freeboard). The chosen width is 8 meters, so there is sufficient space for road.

Other specific requirements that were not previously mentioned are:

- Ensure that in case there seepage through the dam core, due to water heads inside and outside the barrier, the core material will be stable and the amount of water that flows does not affect the safe water levels inside the bay.
- Water head due to tide, storms or tsunamis should not produce liquefaction at any location of the dam.

4.3.2. CONSTRUCTION MATERIAL CHARACTERISTICS

Here, a small summary with the relevant material properties is presented.

For the filling of dam core, quarry run and dredged sand are used. Values considered standard are assigned to them (see table below).

Table 4-1: Construction materials characteristics.

	Unit saturated weight Loose/Compacted		Internal friction angle	Friction angle with geotextile	Cohesion	Porosity Loose/Compacted	
	γ_{sl} (kN/m^3)	γ_{sc} (kN/m^3)	ϕ (º)	δ (º)	c (kPa)	n_l	n_c
Quarry run	18.00	-	35.0	25.0	0.00	0.40	-
Dredged sand	16.00	21.00	40.0	25.0	0.00	0.43	0.20

4.4. BOUNDARY CONDITIONS

4.4.1. BATHYMETRIC CONDITIONS

The longitudinal section to close is 7627 meters long and around 67 meters deep for location 5a. The chosen layout and cross section is detailed in the following figure. This layout will be used when calculating total volumes of material, for construction process definition.

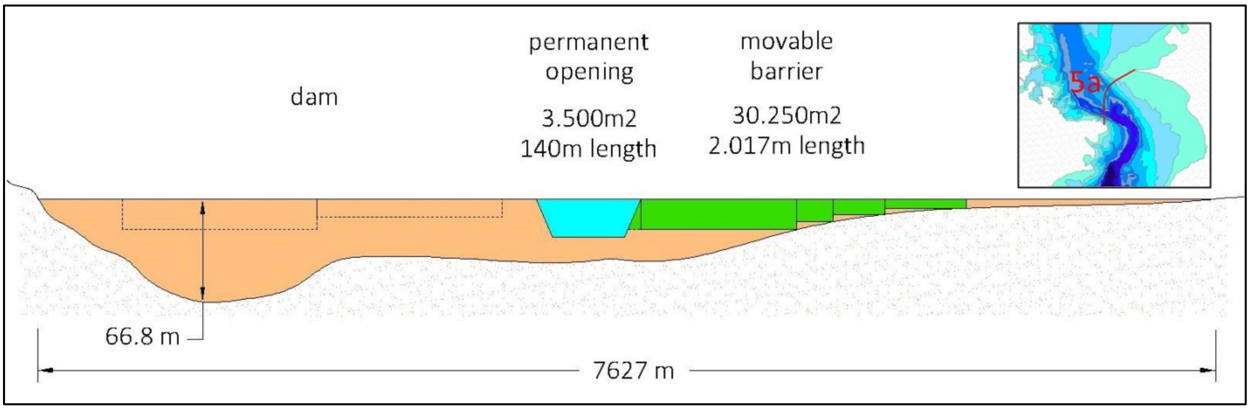

Figure 4-3: Barrier location and general layout.

For stability calculations, the deepest cross section is used, since the aim of this thesis is to offer a solution that addresses the problem of large depths. Therefore, the focus will be the performance of the design for the deepest part of the closure section.

4.4.2. GEOTECHNICAL CONDITIONS

As stated in 2.1, not much information is available. There are wide sandy deposits on the sea bottom (with presence of silt and mud). Also rocky outcrops in the area suggest that the rocky bottom is not far from the seabed. Anyway, dam foundation is out of scope of this thesis, so this aspect will not be studied in detail.

It must be noted here that the foundation is assumed to be given the necessary treatment if needed. Soil characteristics are improved and liquefaction does not occur in case of an earthquake. Thus, in the calculations that will be performed later, it is considered that the foundation remains solid and supports the dam section.

4.4.3. HYDRAULIC LOADS

A summary of the hydraulic boundary conditions is shown in the following table.

Table 4-2: Hydraulic boundary conditions.

	Height	Period
Tide	2.1 m	12 h
Waves	5.0 m	8 s
Storm surge and tide combined	3.7 m	12 h
Tsunami	2.5 m	Several minutes

Where needed, a tsunami height of 2.5 m is considered. It is the largest value given in all the consulted references (see 2.6). Since the magnitude and duration of tsunami are lower than the values of the storm surge, it is expected that the last one will be limiting for calculations.

4.4.4. EARTHQUAKE LOADS

The design requirements regarding earthquake resistance are expressed by means of choosing a seismic coefficient. This seismic coefficient will be used in calculations in order to take into account seismic forces. It will be applied to earth pressures acting on a surface, resulting in an increase of these pressures.

Japanese standards for port and harbour facilities assign a seismic coefficient depending on the location of the structure and its importance. Here, the corresponding seismic coefficient used in calculations should be the one corresponding to the Kanto area, and to a type of structure that has the highest importance.

As a reference, according to these standards, for “high seismic resistance structures” situated over a fault plane, the seismic coefficient should be at least 0.25. In these standards two methods are proposed to calculate the seismic coefficient. For the first method the coefficient is the result of multiplying partial coefficients related to the location and the importance of the structure. The second method gives a formula where the calculation depends on the peak ground acceleration on the surface α, which is equal to 1.5 (Goda et al., 2002). In this report an average of the two methods is used, leading to a value for the seismic coefficient of 0.33.

The choice of this seismic coefficient implies, according to the followed standards, that the target earthquake is the one chosen in the “Regional Disaster Prevention Plan” for the area.

4.5. METHODOLOGY

As the main goal is to optimize the design to arrive to a low-priced solution, the main elements that define the cross section are studied. These elements are the core of the dam and the potential use of caissons (composite cross section).

It has to be remarked that the goal is not a complete definition of the dam, but to check the feasibility and define the elements that would help to lower the costs.

First, the **typology of cross section** is chosen. The possibility of using a composite cross section is considered. Calculations are performed to find whether a composite cross section is more economic than a rubble mound cross section and, if this is the case, which caisson dimensions are optimal. Also construction feasibility is taken into account. Thus, the calculations to be done are:

- **Comparison of costs** for rubble mound solution and composite cross section (for a range of depths and, within each depth, for a range of caissons depth). This will allow defining the dimensions of caissons to use.
- **Check of closure velocities**, once the dimensions of the caissons are defined and the closure gap is known. This will allow knowing whether the defined caissons can be sunk using standard procedures.

Once it is determined whether the use of caissons is recommended, the next step is to define the core of the dam. For this purpose, the use of a **geotextile structure** is considered. In order to arrive to a satisfactory definition, the following steps will be followed:

- Definition of **tentative geometry**, according to material characteristics and construction procedures.
- **Stability** checks. It is necessary to confirm the strength of geotextile sheets, stability of single geotextile elements and overall stability.
- **Feasibility** of construction. It is necessary to ensure that the defined geometry can be built.

Finally, **construction costs** of the defined geometry will be calculated, in order to compare them to a standard solution. This way, it will be determined whether the proposed solution is good.

All these steps will be further described in next sections.

4.6. CROSS-SECTION TYPOLOGY

Two options are proposed in principle, based on the current practice for dams or breakwaters in large depths (see 4.2):

- Rubble mound section
- Composite cross section, consisting of a rubble mound dam and a caisson on top

As stated before, there are examples in Japan of composite cross sections that failed. For example, a number of port breakwaters and bay mouth breakwaters failed due to Tohoku Earthquake and Tsunami in 2011 (MarCom, 2012). Anyway, these recent failures were mainly due to the tsunami that occurred jointly with the earthquake. The cause for failure is presumed to be the wave overtopping the dam, producing a fall of the back surface pressure and rise of instability by scouring the mound (Arikawa, 2012). Scour depths have been studied after the event and conclusions have been drawn regarding how to predict and prevent this phenomenon (Bricker et al., 2012, Matsushita, 2013). Therefore, measures can be taken to protect the structure from this type of failure. In any case, since the expected tsunami height in Tokyo Bay is small and the wave will not overtop the dam, a composite section is worth to consider.

In a first approximation, only construction costs will be compared for the two options. It has to be noted though, that in case of failure of the caissons, the repairing costs would be high, but taking this into account would require further study that is considered out of the scope of this MSc thesis. Therefore, the choice will be made based on construction costs only. This comparison is explained in the following section.

4.6.1. CHOICE OF TYPOLOGY

A comparison of construction costs has been carried out in order to make a decision. A rubble mound cross section has been compared with a composite cross section. In order to do that, different elements have been identified in the cross section (see Figure 4-4), being assigned a unit price (same prices from chapter 3 are used here). This has been done for:

- different depths (d)
- different caisson depth (d1), for a given depth

For each value of d and d1, the volume of each element can be calculated and, knowing the prices, the cost can be found for each option.

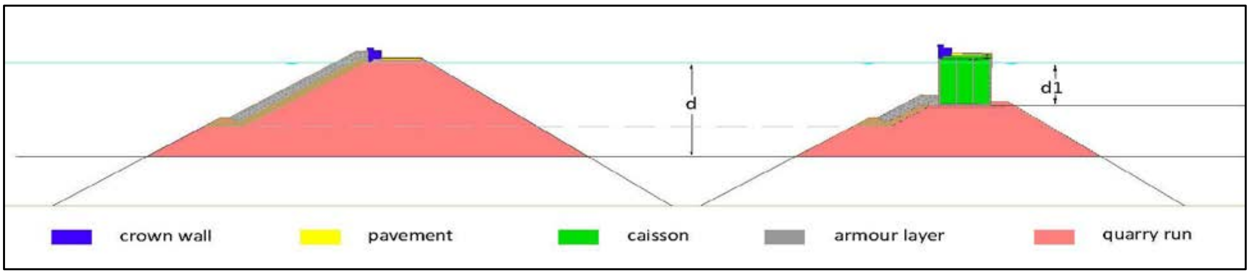

Figure 4-4: Cross section elements for cost calculation(rubble mound on the left, composite on the right)

Varying these two parameters, a graphical comparison of costs has been obtained (see Figure 4-5). All the performed calculations can be found at A5-1, in Appendix 5.

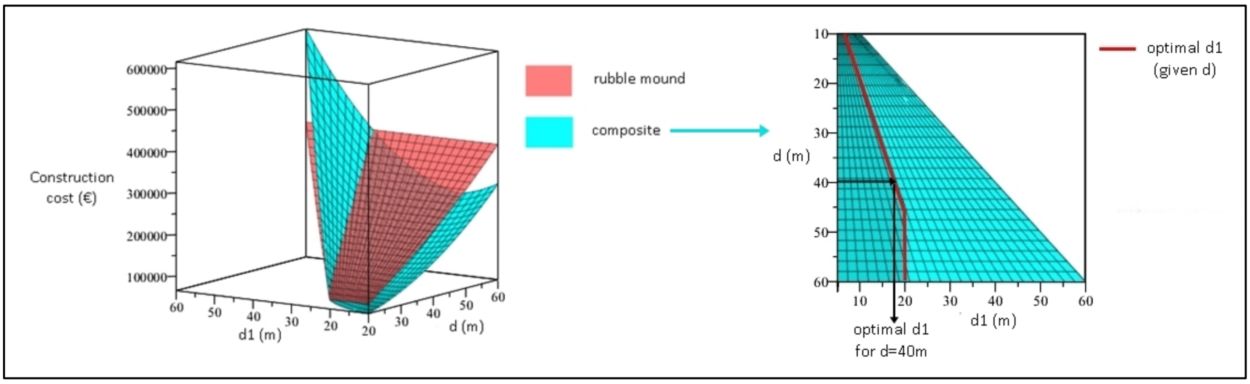

Figure 4-5: Construction costs depending on depths (left) and optimal caisson depth d1 depending on d (left)

It can be appreciated that for each depth (d), the blue surface (composite cross section) has always lower points than the orange one (rubble mound). Therefore the minimum costs correspond to the composite solution. Within the composite solution, the optimal (lowest) cost is found by an optimization of the surface. For every sea bottom depth (d), the optimal caisson depth (d1) can be found at Figure 4-5 (figure on the right, red line).

This study is considered valid for depths larger than 15-20 meters, due to the assumptions made when defining the unit prices (regarding dimensions, materials and construction procedures). For lower depths, different construction methods and indirect costs should be taken into account, likely increasing the price of the use of caissons (very small caissons have larger unit price per volume, and also help to reduce less the volume of material needed. As the detailed study that would be needed is out of the scope of this thesis, it will be assumed that for low depths a rubble mound solution is the most inexpensive.

The composite cross section is chosen as a good option for large depths. It would be interesting to build caissons of different depths in order to be close of the optimal caisson dimensions. But for construction feasibility reasons, probably just two or three steps in dimensions would be considered. Here, for areas between 40-65m deep, a caisson of 20 meters depth would be good, for areas between 20-40 m, a caisson of 15-17 meters could be used.

4.6.2. CLOSURE WITH CAISSONS

It is necessary to do one more check before defining the dam as a composite section. The velocities in the closure process need to be under certain limits. The velocities for caisson placement are limited to approximately 0.3- 0.5 m/s (Verhagen et al., 2009). These values are also valid for caissons with large dimensions (Díaz, 2008). This could difficult the placement operation and constitute an obstacle for the execution of this design. The caissons considered in this case study are large ones and the sinking process can easily last several hours. Therefore it is important to have enough time with low velocities, so the caissons can be placed.

The most unfavorable situation is the placement of the last caisson, when the gap is reduced to the permanent opening and the movable barrier opening. The results in this case are the following (see Figure 4-6 and section A5-2 in Appendix 5 for detailed calculations):

- Maximum velocities in the gap around 3.5 m/s
- Duration of slack water (velocities lower than 0.5 m/s) around 33 minutes (blue stripe in the following figure)

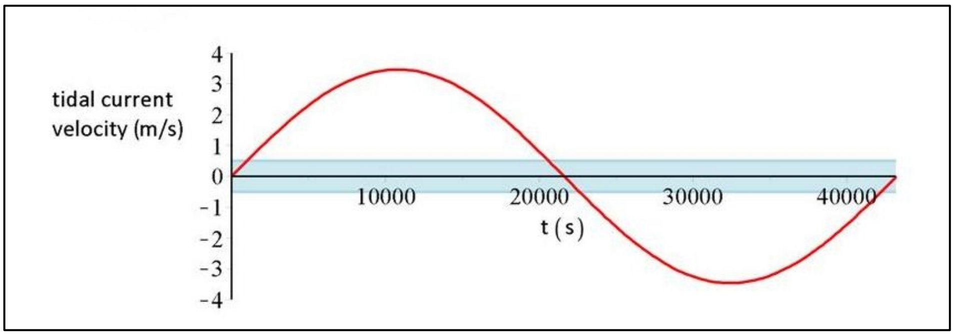

Figure 4-6: Tidal current velocity placement last caisson (smallest opening).

For large caissons the sinking operation can last several hours, so a more detailed study would be necessary in order to define the equipment needed and the real cost of using caissons in this design. A rubble mound section could be the option in case the velocities restrain the use of caissons. This aspect would need further study, but it will not be developed here. Consequently, a rubble-mound section is chosen and will be used from now onwards.

4.7. USE OF GEOTEXTILE IN THE DAM CORE

As stated before, the use of geosynthetics is studied here, in order to propose a design that allows a reduction in costs and still offers a good performance during the lifetime of the structure. Geotextile can be used to confine a low-priced material in the core of the dam. Sand from dredging is assumed to be this inexpensive filling material. Using a geotextile structure will allow, in principle, building steeper slopes (reduction in costs). Besides that, the sand confinement will improve the seismic behavior of the structure.

One of the options to consider is the use of geocontainers, as proposed by A. van der Plas in his MSc thesis (see 4.2.1). However, due to the large volume of the dam, this method implies that hundreds or thousands of bags should be filled, transported and dropped into the ocean, which is a disadvantage due to the large number of operations and transport times. Therefore, the idea of reducing the number of construction steps arises.

Besides that, there is an inherent inaccuracy in the placement of geocontainers dropped from a barge. This inaccuracy can cause unwanted tolerances in the limits of cross section, as well as the presence of gaps in the dam core. There are studies that investigate geocontainers placement and these resultant inaccuracies (de Groot et al., 2003). A graphical idea is given in the following figure.

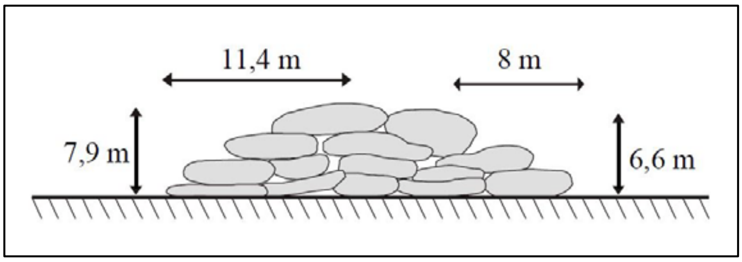

Figure 4-7: Cross section of geocontainer stack (de Groot et al., 2003)

As it has been stated before, the large number of bags and the inaccuracy of the placement method are the disadvantages to overcome. The number of bags could be reduced by enlarging the size (a standard geocontainer can have a diameter of 4-5 meters and a maximum length of 50 meters). The transport by barges and inaccuracy of placement can be avoided by doing the filling on site. Also, the on-site filling process allows considering the vibrocompaction of sand at the same time that the cells are being filled, reducing the possibility of liquefaction in case of earthquake.

A bag that could be filled on site is close to the concept of geosynthetics vertical cells (see section 4.2.1). Nevertheless, vertical cells have only been proposed for very small elements and there are no references about their performance or details about construction. Here, the goal is to adapt the concept and define a feasible design. This means, starting from a marketed material (geotube, geotextile), to design a geometry that is compatible with its strength and maximum dimensions. Also, dimensions and material characteristics should be compatible with the construction process.

As stated before, a product that is already in the market is used as a reference. This way, a standard price can be used to calculate construction costs. Characteristics and specifications of the chosen product will be used in calculations in order to confirm that the proposed geotextile elements can resist the design loads.

Due to similarities with geocontainers, the use of the same type of material is proposed. In principle, any geotextile in the market used for this application could be considered. Specifically, the woven geotextile ACETex® PP, fabricated by ACE Geosynthetics, has been chosen. ACE Geosynthetics has provided assessment regarding production feasibility of the designed cells, with the required dimensions. It has been considered important to confirm that the design does not pose specific complications in the manufacturing process.

The chosen product specifications are summarized in the following table.

Table 4-3: Woven geotextile specifications

Product	Designation	Nominal Tensile Strength (kN/m)	Static Puncture Resistance CBR (N)	Apparent Opening Size O_{95} (mm)	Permittivity 50 mm head (1/sec)
ACETex® PP	GT200-II PP	200	23000	0.30	0.45

The proposed design consists in geotextile elements, fabricated with the above mentioned woven geotextile, and composed by vertical cells. A structure made of square cells is defined. The geometry is intended to be simple and easy for fabrication, so the production costs can be competitive.

These elements will be placed at the core of the dam and filled with sand. Several of these elements will be needed to complete the cross section, and they will have different heights. For a graphic representation, see Figure 4-8.

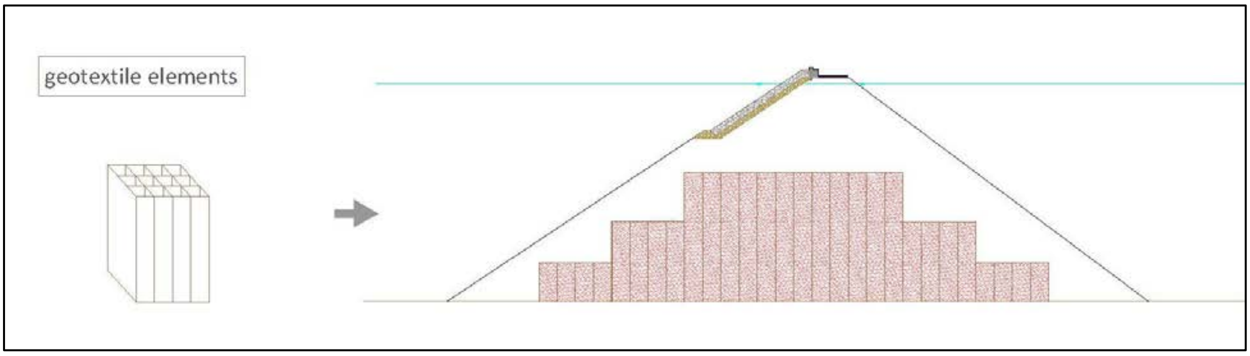

Figure 4-8: Geotextile caisson geometry and location at the dam core

Once materials, geometry and construction process are defined, it is possible to estimate costs and compare them to a solution that applies proven technology. Regarding proven technology for dams in seismic areas, there are several measures prescribed to improve earthquake resistance for similar structures (see 4.2.2). However, these measures are mainly aimed to avoid liquefaction of foundations. It is generally assumed that in the core of the structure high quality material will be used. A standard rubble mound section is applied in many cases. Therefore, the proposed design will be compared to a standard rubble mound cross-section made of quarry run.

Since the geotextile structure is supposed to outperform geocontainers for this type of conditions, they will be compared as well in order to confirm that the assumption is correct.

4.7.1. METHOD DESCRIPTION

The steps to follow are:

- Firstly, tentative **geometry** of the geotextile cells has to be defined. For that, similar elements used nowadays are taken as a reference. With this reference, the cell size is set (needed for stability calculations), and the density of geotextile on the dam core can be obtained (needed for cost calculations later).
- Once the geometry is defined, calculations regarding **stability** and geotextile strength are done, in order to check the validity of the chosen dimensions. Calculations also help to define the

limits for the unequal filling of the cells, which is needed for the definition of construction process.

- Later, the **construction process** is described in detail, mainly estimating filling rates (dredging) which lead to a definition of equipment needed and duration of works.
- Finally, with all this information, it is confirmed that the initial assumption about **costs** is correct and the costs are lower than in case of a standard rubble mound/geocontainer solution.

In case the initial dimensions are not valid, iterations of this whole process would be needed.

In order to perform the calculations, the following theoretical background is used:

- Japanese guidelines "Technical standards and commentaries for port and harbour facilities in Japan" (Goda et al., 2002). They are used to calculate earth pressures, specifically the dynamic pressures generated during an earthquake.
- For the pressure distribution of the sand inside the vertical cells, the Janssen formula for pressures in silos is used.

4.7.2. CELL GEOMETRY

In order to achieve low costs and large stability, the dimensions of the geotextile elements and individual cells have to be as large as possible. Certain conditions limit these dimensions though, like strength of geotextile or capacity of equipment that will handle the elements.

A first estimation is done by taking dimensions similar to products that already exist in the market. For example, ordinary geocontainers have a maximum diameter of 4-5 meters, approximately. Besides that, these dimensions are convenient since they are close to the reach of a vibrolance (Davies and Schlosser, 1997), which would be used to compact the sand while the filling is done. Therefore, the calculations will begin with a cell of 5*5 meters and undetermined height. In principle, it is assumed that the cell will reach the necessary height, since, according to ACE Geosynthetics, for this case study the geotextile piece can be produced as large as needed. In the following figure a sketch of dimensions is presented.

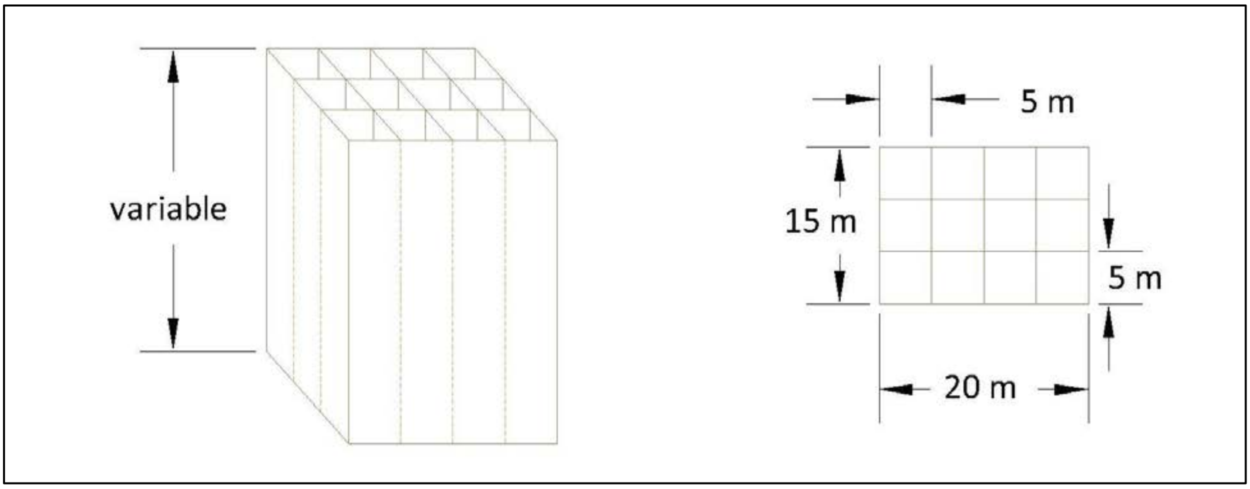

Figure 4-9: Geotextile cells geometry.

4.7.3. STABILITY

In order to check the stability of the proposed structure, several load cases are considered:

- Construction phase.
- Normal operation.
- Earthquake occurrence.
- Storm surge induced by a typhoon.
- Tsunami.

Calculations need to be done at different levels:

- **Sand filling stability**. Here, the situation during typhoon is critical, since the difference in water levels between the inner and the outer part of the dam can create a pressure difference along the vertical column of sand filling in the geotextile cell. If the pressure difference is high enough it can cancel the weight of the sand and liquefaction will occur.
- **Quarry run grains stability**. The sand filled structure is much less permeable than the quarry run. When there is a water head difference between the inner and outer part of the bay, water will tend to move through the quarry run. The hydraulic gradient and water velocities need to be calculated in order to corroborate that the quarry run material is stable under the design water heads.
- **Geotextile resistance**. The two cases considered limiting are the construction phase (when filling a cell while the adjacent one is still empty) and earthquake occurrence (when dynamic forces can induce extra pressures on the geotextile sheet)
- **Stability of geotextile elements**. Here, sliding stability is checked. It is considered that overturning will not occur, as the structure is deformable and large settlements and sand losses would occur instead. Here, earthquake is considered to be the limiting situation.
- **Overall stability**. Since the slopes have been chosen low enough, it is considered that this check is not necessary.

All these calculations are explained and developed in the following sections.

4.7.3.1. STABILITY OF SAND UNDER WATER LOADS

The stability of the dam core material can be compromised in case of large differences in water levels outside and inside the bay. In case certain conditions happen, there is a possibility that liquefaction occurs. This would be due to the relative low permeability of sand. In case the permeability at the base of the dam is larger than in the core, water pressures can be transmitted to the inner base of the dam. If this happens, a sand filled cell in the core of the dam would be subjected to a larger water pressure on the bottom than on the top. This difference in water pressures would result in an uplift net force that would reduce the effective weight of sand. This could reach a point where the extra vertical water pressure cancels the submerged weight of the core filling. In this case, liquefaction may occur.

Since the dam is not impermeable, the distribution of water pressures would need more elaboration. Also, the permeability of the dam foundation would be needed here. Due to the lack of information and in order to simplify the case, a conservative assumption is made. Namely, that during the storm surge event, the permeability of the dam foundation is higher than the dam permeability. This would allow a vertical gradient in pressures on the dam core; since the pressure at the bottom of the cell would be the one on the sea side; while the pressure at the top of the cell would be the one at the bay side (see following figure).

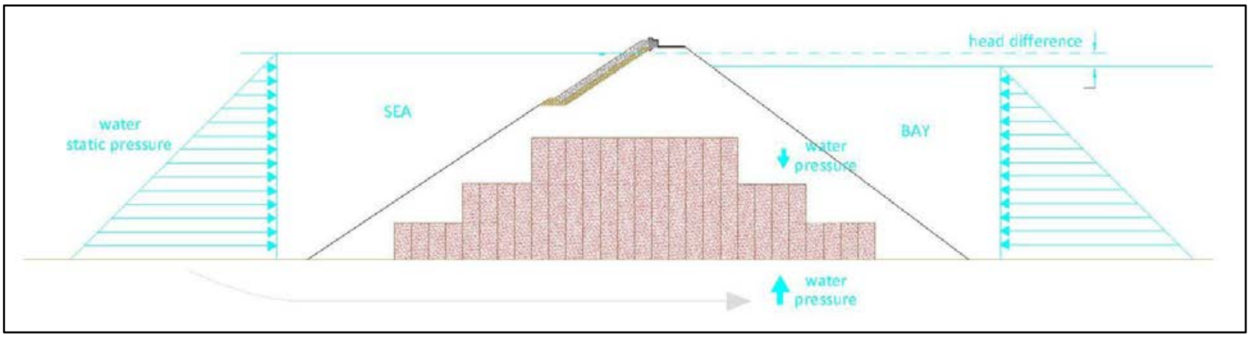

Figure 4-10: Storm surge induced water pressures on dam core

In the dam core, two materials are used: sand filling for geotextile cells and quarry run. It is assumed that the quarry run used for the cover is permeable enough so it is not under extra pressures. However, in case of sand filling, liquefaction can be a problem. Hence, it has been calculated whether a reasonable quarry run cover has enough weight to counteract the induced vertical pressures on the sand. The submerged weight of quarry run has to be larger than the gradient in water pressure.

In order to know whether the structure is stable, it is necessary to calculate first the hydraulic gradient that is acting upon it. For that, the difference in water levels outside and inside the bay has to be known. The limiting situation would be a storm surge, where the difference in water levels is the highest. Despite the amplitude of the water level motion is 3.7 m for the design storm surge, inside the bay the water is also moving, so the maximum difference in water levels could be lower. Therefore, it is necessary to do an estimation. In Figure 4-10 water levels during typhoon inside (dark blue) and outside the bay (light blue) can be observed. The difference between water levels has been plotted in red, having a maximum at 2.8 meters (see A6-1 in Appendix 6 for details). This value will be used in the calculations.

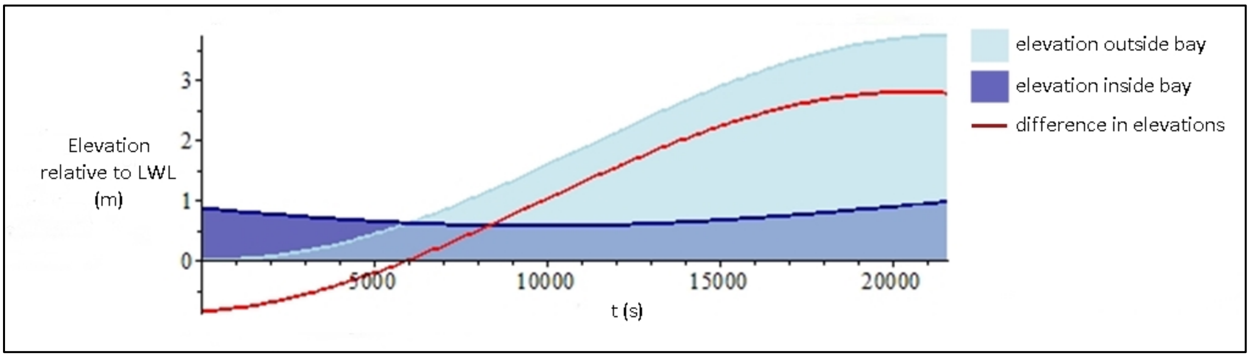

Figure 4-11: Water level difference among inner and outer part of the bay, for the design storm surge.

Now water head and therefore hydraulic gradient are known. The extra vertical pressures caused by this hydraulic gradient can be calculated. These extra pressures will be counteracted by the extra weight provided by the quarry run layer on top of the sand. Knowing the weight needed, the thickness of quarry run over the sand can be determined.

Complete calculations can be found at A6-1 in Appendix 6. The result is that 3.5 meters of quarry run add a weight that is sufficient to counteract possible vertical water pressure gradients. Therefore, the sand filling will be stable under these conditions.

4.7.3.2. STABILITY OF QUARRY RUN UNDER WATER LOADS

The difference in water levels inside and outside the bay causes a hydraulic gradient and water flow through the core of the dam (see figure below). This water flow may cause instability of quarry run, if the grains are carried away by the water.

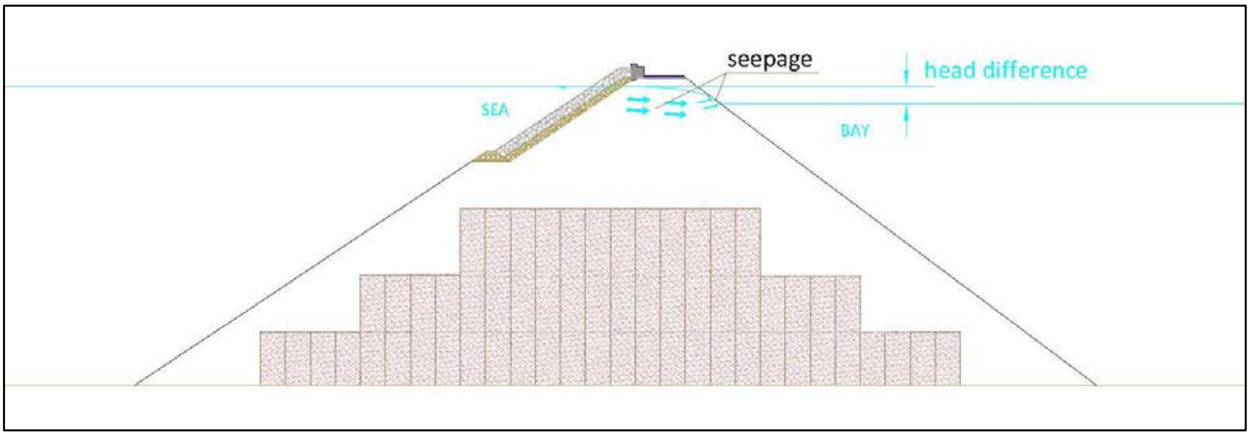

Figure 4-12: Quarry run stability in case of water head difference

With the previously calculated value for water head difference, the stability of quarry run grains is checked. Two calculations are developed:

MICRO-STABILITY OF INNER SLOPE

From equilibrium of forces on a soil element on the slope surface (see Figure 4-12), a critical slope can be obtained for the given hydraulic gradient. The case of perpendicular seepage is considered here, where the equilibrium reads $\tan \emptyset \geq \frac{\sin \alpha}{\cos \alpha - i}$, being Ø the angle of repose, i the hydraulic gradient and α the slope.

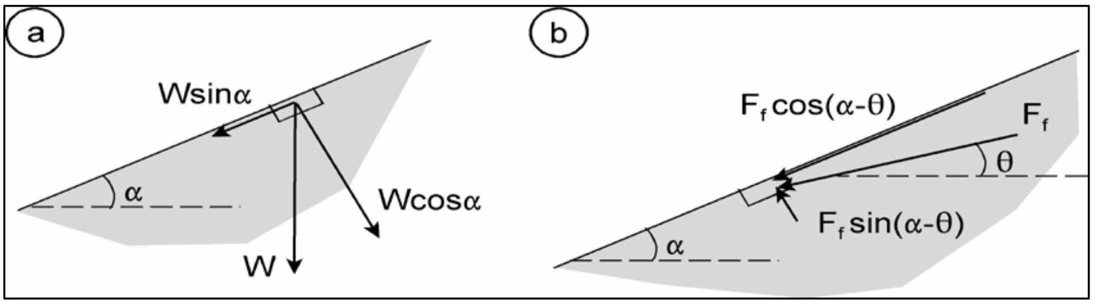

Figure 4-13: Forces on slope with porous flow (Schiereck, 2001)

Applying this equation it is found that the slope low enough to be stable (see A6-2 in Appendix 6 for details). Since very low values where set for seismic reasons, it was expected that no instability would appear here.

STABILITY OF GRAINS IN THE DAM CORE

For the velocities that will occur in case of water head differences, the critical particle diameter that is stable is calculated. Forchheimer equation $u_f = k \cdot i^{\frac{1}{p}}$ is used to calculate the water velocity, as the hydraulic gradient i is known. This is done applying a coefficient p that is considered typical for quarry run (Schiereck, 2001). Once the filter velocity (u_f) is known, the water velocity in the pores can be estimated dividing it by the porosity. Finally, the Shields relation for the threshold of motion is applied to this velocity, so the critical particle diameter that can be washed is found (see A6-2 in Appendix 6 for details).

Here, the calculations give as result that the particles larger than 2 centimeters will be stable. In order to avoid material losses it would be necessary to take measures, for example:

- Use of material with a certain gradation, without small diameters (this material would be expensive though).
- Placement of a geometrically closed granular filter, in order to keep the quarry run grains inside the dam core.
- Placement of a geotextile covering the quarry run core, in order to avoid the water flow through the dam core.

4.7.3.3. GEOTEXTILE RESISTANCE

Given geotextile strength and cells geometry (5*5 meters), it is calculated which loads the geotextile can resist without breaking.

Two limiting situations are contemplated (see Figure 4-11):

- Building phase. The strength of geotextile will determine the maximum height in which the cell can be filled while the adjacent cells are still empty. This calculation will set the maximum head difference while filling.
- Operational phase. Earthquake occurrence is considered to be the critical situation. With the extra forces involved, it has to be checked that the geotextile cell will not break. Cells in the edge of the structure are considered to be the most unfavorable since the soil is confined only at one side and there will be an imbalance in the transmission of dynamic forces at both sides of geotextile.

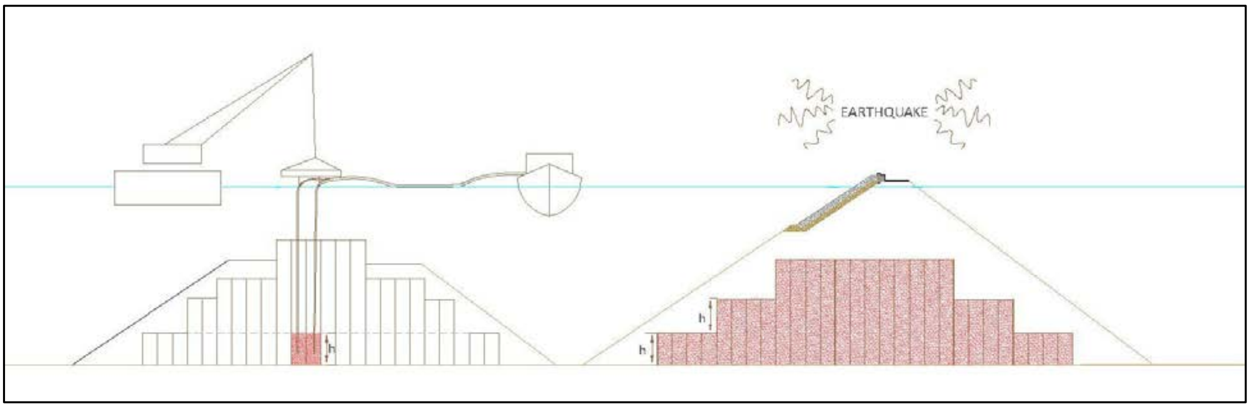

Figure 4-14: Limiting situations regarding strength of geotextile: building process (left) and earthquake (right)

In order to perform the calculations, standard values for geotechnical parameters are chosen for quarry run and sand (see 4.3.2).

For geotextile, parameters from the product ACETex® PP are used for calculations. This has been done with the advice of ACE Geosynthetics engineering department, which considers this product to be the most suitable for such application. The chosen nominal strength of geotextile is 200 kN/m, which is the largest one within the offered range of products.

It has to be noted that the provided nominal strength is reduced in calculations by the use of safety factors (Leshchinsky et al., 1996). In this particular case the reduction accounts for seam strength. Chemical degradation is not taken into account since the geotextile will be confined inside the dam core and will not be exposed to degrading agents. Once safety factors are applied, the maximum geotextile working tensile force is compared to the force induced on it by the loads (static and dynamic forces).

The chosen load cases are described below.

<u>CONSTRUCTION PHASE - FILLING</u>

The situation when filling a cell while the adjacent ones are empty is considered to be critical for building process. The sand filling causes horizontal pressure on the geotextile sheet. While the adjacent cell is empty, this is not counteracted by any other load, so pressure on geotextile reaches a maximum in this situation. This maximum pressure which depends on the column height is compared to the geotextile working tensile strength. This way, the maximum head in sand filling is found.

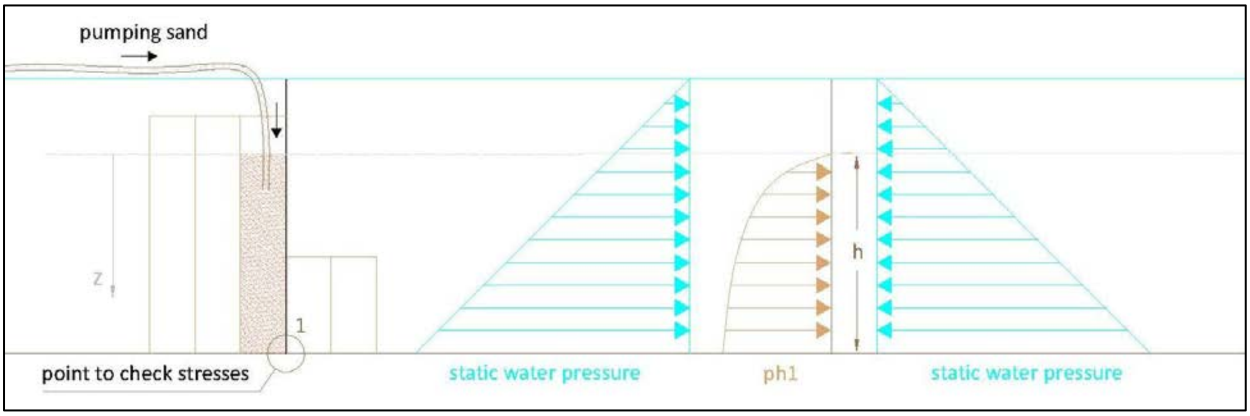

Figure 4-15: Stresses on both sides of geotextile during filling process

Several tentative filling heights are tested in calculations. The corresponding horizontal pressure is calculated using the Janssen formula for silos, where horizontal pressure is function of the column height.

It is found that the geotextile cells resist a maximum of 10 meters head in sand filling (see A6-3 in Appendix 6 for details), given the chosen geometry of 5*5 meters cells. Therefore, the filling process needs to be done respecting this limit.

<u>OPERATIONAL PHASE - EARTHQUAKE</u>

The main risk here is earthquake induced accelerations increasing stresses on the geotextile. As the sand is confined within the cell, it is expected that it will be subject of dynamic forces that will affect the geotextile sheet. On the external side, the quarry run used as a cover is not confined. For simplicity, it is assumed that the dynamic forces that act on quarry run cancel the horizontal pressure on the geotextile. Therefore, a significant increase in the pressure at the bottom of the cell will occur (see following figure).

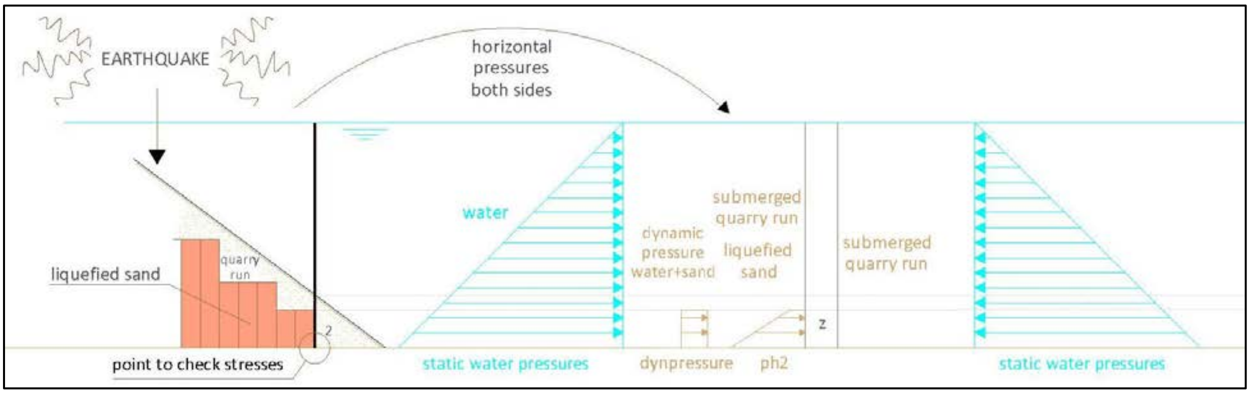

Figure 4-16: Stresses on both sides of geotextile during earthquake

Now, it is necessary to calculate the stability when the dam is already built and under critical loading (earthquake). The edge of the geotextile structure is expected to be the critical point here, since the material is confined only at one side of the geotextile sheet. This leads to an unbalance in forces that will cause extra tension on the geotextile. The maximum height that the edge cell can have is calculated. For a first approximation 10 meters are chosen, since the construction will be done in 10 meter steps and this way the process of filling the edge cells is simplified.

For this edge cell of 10 meters height, the pressures on geotextile sheet are calculated. A conservative assumption is made, considering that the sand is liquefied. Then, the forces to take into account are the hydrostatic sand pressure and the dynamic forces induced by the earthquake. These dynamic forces are modeled as a percentage of the static forces, using the seismic coefficient that has been calculated before (see 4.4.4). Then, the total horizontal pressure is compared to the geotextile strength.

The outcome of calculations is that, in case of earthquake and liquefaction of sand within cells, the geotextile alone cannot withstand the pressures (see A6-3 in Appendix 6). In consequence, some kind of reinforcement is needed. Two options are considered in a first approach:

- Use of several geotextile layers or a stronger material, which would lead to a larger geotextile resistance.
- Reinforcement using stainless steel cables with the geotextile.

For simplicity, the stainless steel cables are chosen, since the material costs are known and they can be incorporated to the total costs calculation. A 10 mm diameter cable is considered, leading to a reinforcement of 2 cables per meter height, which will resist the excess of pressure (see A6-3 in Appendix 6).

This reinforcement is applied to the edge cell. For inner cells, since the material is confined (and densified) at both sides of geotextile, it is considered that the reduction in pressure at one side will not occur since sand and geotextile would move together.

4.7.3.4. STABILITY OF GEOTEXTILE ELEMENTS

The sliding stability of one geotextile element under earthquake is studied. This is the limiting situation, as no sliding or overturning are expected in normal conditions. It has to be remembered that it is assumed that no liquefaction occurs in the foundation of the dam during earthquake.

In principle the element dimensions are chosen such as it can be handled with standard waterborne equipment. Since the individual cells are 5 meters long, a maximum of 3-4 cells (15-20 meters) is contemplated. The length of the element is set on 15 meters in the calculations.

Then, the sliding stability under earthquake is checked, by means of comparing the destabilizing horizontal dynamic forces with the stabilizing friction forces (submerged weight multiplied by friction coefficient). In case the friction forces are larger, the element is expected to remain in place and not move.

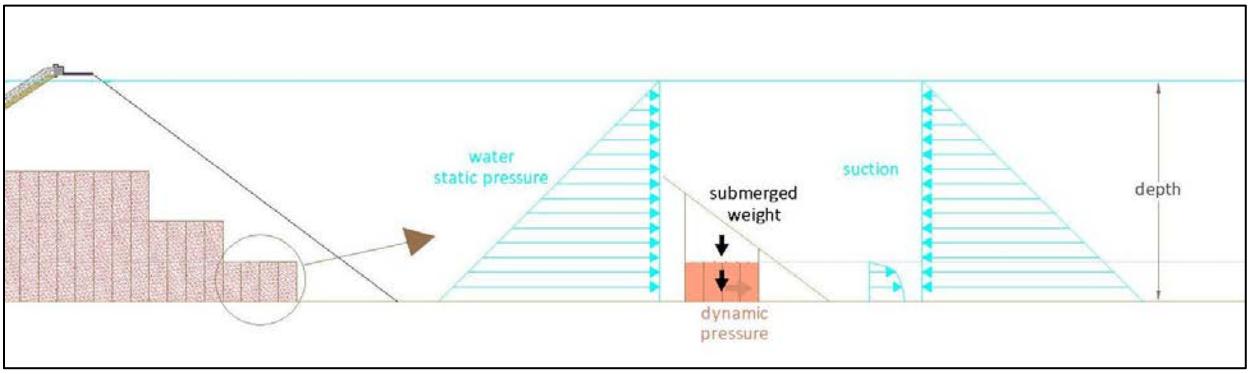

Figure 4-17: Sliding stability under earthquake of a geotextile element.

The result of sliding stability check is positive. Therefore the considered size of the elements is sufficient to keep them in place in case of an earthquake.

4.7.4. FINAL DIMENSIONS

Once the stability calculations have been carried out, it is possible to define a final geometry. Also, details like the sealing of cells and the stainless steel cables are introduced. A complete cross section is detailed in the following figure.

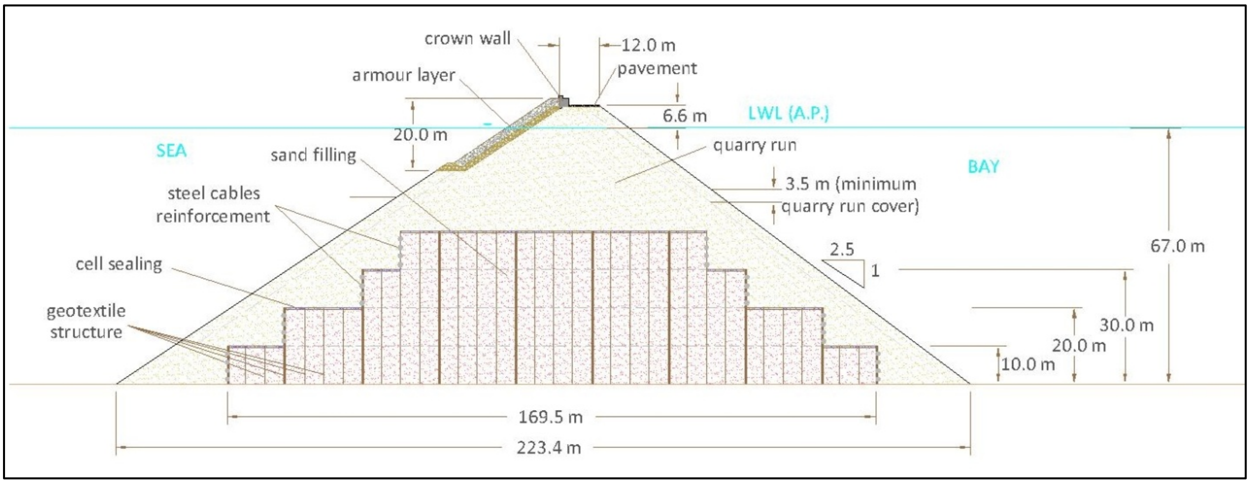

Figure 4-18: Dam cross- section

4.7.5. CONSTRUCTION PROCESS

Once the dimensions of geotextile elements are defined, the execution process can be confirmed.

Basically the construction phases will be:

1. **Placement.** The geotextile elements must be placed first. The size of these elements (20 meters long) should be manageable using a crane on a pontoon or tugs.
It is expected that some ballast will be needed to help with the placing operation, and also later to keep the geotextile in place (see following figure).

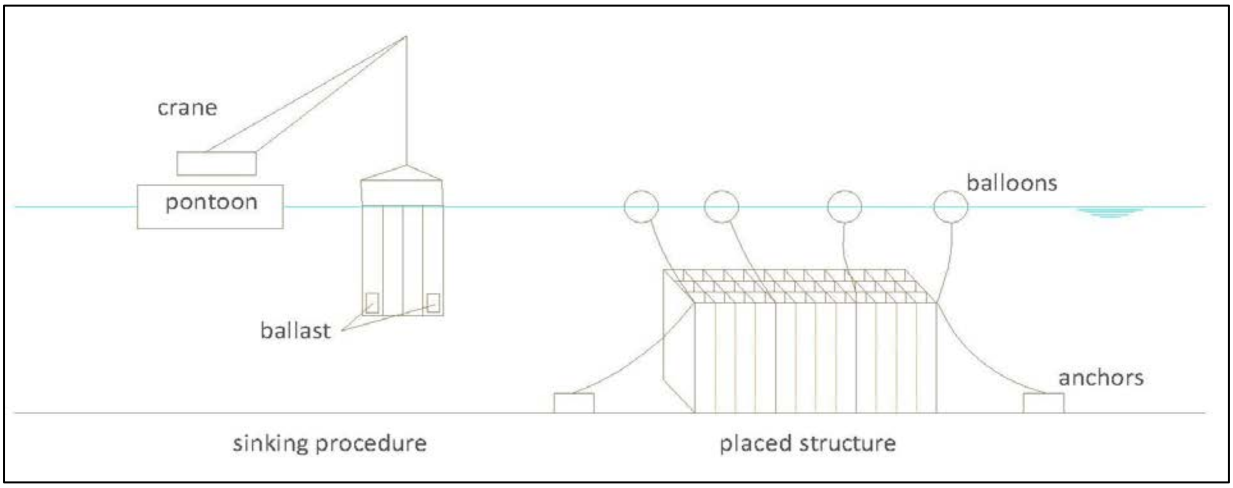

Figure 4-19: Placement of geotextile elements

2. **Cells filling**. Here, the previously calculated requirements have to be taken into account (maximum head difference between cells). Cells will be filled with dredging slurry, preferably several adjacent cells at the same time, to improve stability and take advantage of the auxiliary equipment (crane, pontoons). While they are filled, a vibrolance will be introduced in the cell, in order to compact the sand that is deposited.
Every time the filling reaches 10 meters height, the equipment will need to move to adjacent cells to keep filling them, so the maximum head is not surpassed at any point. The number of displacements per day will depend on the number of cells that can be filled simultaneously. For example, filling 10 meters of 6 cells simultaneously is equivalent to 1,500 m^3, which is considered to be enough, as it will be explained later in section 4.7.6 Construction rates.

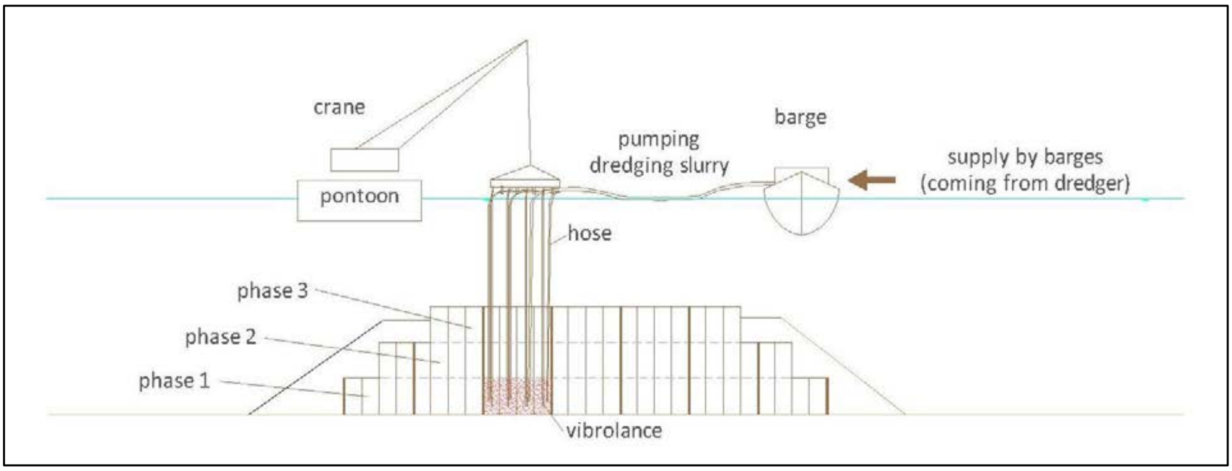

Figure 4-20: Cells filling and compaction

3. **Cell sealing**. As soon as there is enough space between the two activities, the cells will be sealed with a bituminous product and later covered with quarry run.
4. **Finish quarry run cover**. This will be done with the help of standard waterborne (barges) or land-based equipment (trucks).

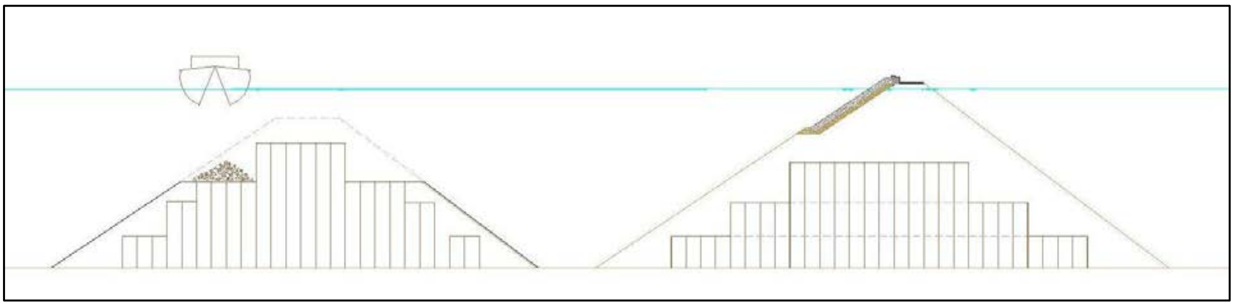

Figure 4-21: Quarry run placement.

It has to be noted that velocities in the cross section will grow while closing the gap. At the moment the closure starts, the maximum current velocity will be 0.5 m/s, with the cross section completely open (see Appendix 5, A5-2). Finally, when only the gaps for permanent opening and barrier gates are left, current velocities will reach 3.5 m/s. It is possible these strong currents make the use of the geotextile too expensive at some point while closing the gap, since the auxiliary equipment and anchorage could be too costly. The calculation of this point is related to anchorage calculations, which are considered out of the scope of this MSc thesis.

4.7.6. CONSTRUCTION RATES

The use of standard waterborne equipment has been considered. The following construction means are included in the calculation of costs:

- Dredger. It is unknown which type of dredger would be used, but a standard suction dredger with a production rate of 300 m^3/h is chosen for calculations. The dredger will work an average of 10 hours per day and will produce 3,000 m^3/day. The rest of the equipment detailed below will adapt to this work pace.
- Barge. A team of 4 barges is assumed to transport the dredged material from the dredger to the pumping location, where the sand is placed in the geotextile structure.
- Pontoons. Two pontoons of large dimensions are necessary to host the crane and the auxiliary equipment in the location where the sand is pumped into the geotextile cells. An extra pontoon is used to help with the anchoring works.
- Crane on pontoon. A crane is needed on the pontoon, in order to move the hoses and vibrolances that are introduced in the geotextile cells. The crane also places the geotextile structure before filling.
- Tug. It would help the works when installing anchoring and balloons.
- Divers. A team of divers assisting the anchoring works is foreseen.
- General operators. They are the human team that will help with the crane operations, handling pumps, hoses or vibrolances.

In order to make a realistic approximation, it has been assumed that the dam core would be executed in approximately 2 years. Accounting for a 6 day week and a 20% of interruptions due to bad weather conditions, this means that a rate 30,000 m^3/day should be achieved. Since every team working in filling is placing 3,000 m^3/day (the production of one dredger), 10 teams are assumed to be working simultaneously. As is was stated before, the equipment working in cells filling will need to move every 1,500 m3, in case 6 cells are filled simultaneously. Therefore, in order to place 3,000 m3 of sand per day, every team needs to move the hoses once per day, which is considered to be feasible.

In reality, it is likely that instead of having 10 working teams the equipment would be optimized. Besides that, more than 6 cells could be filled at the same time and allow a reduction of operations. Therefore, some reduction in costs could be achieved. However, this level of detail is assumed to be enough for the purpose of this thesis.

4.7.7. COSTS

For cost calculation, the previously defined cross section geometry (4.7.4) and the estimated construction rates (4.7.6) are taken into account. It should be emphasized that the costs are calculated per one meter length and for a cross section of 67 meters depth, which is the one considered representative, as this study focus on solving the problem for dams with large depths. Therefore, particularities of sections with low depths are not addressed here.

The following table summarizes the approximate building costs that have been calculated for the dam core. This table compares a standard rubble mound section with geocontainers and geotextile cells. It has to be noted that only the granular material body is considered here, since crown wall, pavements or armour layer will be present in all cases.

Table 4-4: Construction cost comparison between alternatives for the dam core.

	€/meter length
QUARRY RUN CORE	284,000
GEOCONTAINERS CORE	245,000
GEOTEXTILE CELLS CORE	236,000

More details on these costs calculations are given at A6-5, in Appendix 6.

4.8. RESULTS

The performed calculations have provided the following results:

- The strength of the selected product is sufficient for this application.
- The designed structure withstands earthquake induced forces with a reinforcement supporting the geotextile edges.
- The designed structure withstands hydraulic loads with the help of a quarry run cover and the necessary measures to prevent seepage (material gradation, granular filter, geotextile).
- The total cost of using geotextile cells is lower than for the quarry run case and the geocontainers.

Several remarks can be done regarding the comparison between geotextile cells and the other considered options:

- Compared to **quarry run**, the geotextile cells have the disadvantage of being new technology that requires practical validation, but they still have important advantages:
  - In a country densely populated which has small surface of natural areas, it could be preferred to apply a new technology rather than open new quarries for material supply. In this case of study, the needed material volumes are quite high (more than 13M m^3 of core material for the whole dam). Therefore, this consideration might be important.
  - Since the material is confined and compacted in the geotextile cells, lower settlements would be expected in case of an earthquake and a smaller volume of material would be needed for repairs.
  - As it was mentioned before, the total cost is expected to be lower for the geotextile cells.
- Compared to **the use of geocontainers**, the geotextile cells have the disadvantage of being new technology, but they may be preferred due to certain reasons:
  - Geotextile cells allow controlled placement of sand filling. Geocontainers dropped with barges fall in a disorderly distribution. This effect is more important for large depths. Gaps would be unavoidable between the dropped geocontainers, which would result in lower quality of the final structure. Besides that, the built cross-section would have an extra volume due to the large tolerances in placement of geocontainers. This would result in extra costs.
  - Geotextile cells allow controlling the density of the sand filling, since it can be compacted using vibrolances and at the same time, the achieved densities can be measured.
  - The expected seismic behavior of geocontainers is worse, due to the above mentioned gaps between cells and the lower density of the sand filling. Larger movements and settlements are expected in case an earthquake occurs.
  - The calculated construction cost is lower for geotextile cells, though it has to be noted that they are very close. More study should be done in order to clarify which are the possibilities regarding construction process and which is the real difference in costs.

Finally, it has to be noted that there are important aspects that still would need study, for a complete definition of the dam, but these aspects are out of the scope of the thesis and are not developed here. The points that would need more definition are:

- <u>Foundation</u>. It would be necessary to know if the soil needs treatment or substitution and where. Bearing capacity and probability of liquefaction should be studied. The permeability of soils in the dam foundation should be studied as well.
- <u>Design of the anchoring</u>. The high current velocities are a challenge for the filling of geotextile cells. Anchoring is proposed as a solution, but further study and calculations would be needed regarding this matter. It is necessary to define the anchoring and the possible current velocity limits for the application of the geotextile structure.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. CONCLUSIONS

The study, calculations and analysis carried out for this Master Thesis have lead to fruitful conclusions. Parts of them were already introduced in the system level analysis section (see 3.5) as they were necessary to give a context to the subsequent design phase. Here, a more general statement will be made.

In the following paragraphs, the conclusions are presented and linked to the research questions that were raised at the beginning of this report.

IS A BARRIER A GOOD SOLUTION FOR PROTECTING TOKYO BAY AGAINST FUTURE FLOODS? UNDER WHICH CONDITIONS?

- A **storm surge barrier can be a good solution to protect Tokyo Bay,** since under certain conditions it is the most inexpensive measure.
- Among the studied possibilities, location 5 is the optimal one regarding total costs. This location is close to the bay mouth and protects almost the whole bay coastline.
- Among the studied return periods, 500 years is the most favorable since it minimizes total costs. Moreover, when considering this return period, the values of benefit start to surpass the costs (ratio benefit over costs larger than 1).
- The need for the barrier is strongly related to the fulfillment of certain conditions, mainly a minimum rise in sea level of 1.5 meters.
- In order to determine whether the barrier is really the best solution, the cost of upgrading of coastal defences for a design return period of 500 years should be calculated. This way the two alternatives can be properly compared.
- Finally, a storm surge barrier is a solution which makes sense from a point of view that implies a long term perspective (100 years lifetime) and a very large investment. It could be easier to implement some phasing in the upgrading of coastal defences and make smaller investments every time. However, this could represent an increase in total cost in the long term. Due to these reasons, the decision will be strongly conditioned by the intergenerational policy of Japanese Government.

WHICH IS THE BEST TYPE OF BARRIER?

- In order to fulfill the required functions, the barrier should be composed of several elements, including a closure dam, a movable barrier and an opening for navigational purposes.
- According to calculations, it is possible to leave open part of the section. Only dimensions have been defined for this **permanent opening**, and no further study has been done. It has to be noted though that this channel would be subjected to large current velocities, during storm surge events and also during normal conditions. Therefore, this opening would need protection against scour.
- Regarding the **dam**, it has been studied whether a composite or a rubble mound cross section is more advantageous. The conclusion is that a rubble mound section is preferred, since the large current velocities on site would make impossible to place the caisson using standard

procedures. This dam would need to cover a stretch with large depths and would have a large volume (more than 13 Mm^3).

- No detailed study has been done regarding the **movable barrier**. It could be composed of gates or inflatable elements, but they are out of the scope of the thesis and no further attention has been devoted to them. The only relevant remark regarding this subject concerns the navigational channel. This channel should have a width close to 600 meters and during a storm surge event only 140 meters would be left open. Therefore, a temporal closure would be needed during typhoons. In case this is done using gates, the large depths needed for navigation would be a challenge for the design.

CONCEPTUAL DESIGN (DAM DESIGN BASED ON GEOTEXTILE CELLS)

- The proposed geotextile structure is **stable and resists the design loads.**
- Regarding costs, the structure is **economically feasible**, since it reduces costs compared to the other studied alternatives (quarry run, geocontainers core).
  Here, it has to be noted that a simplified cost estimation has been performed; therefore the result has to be taken as positive but not decisive. Due to the uncertainties and details that have not been taken into account, the costs for the different alternatives could be more similar than the result obtained here. However, as the geotextile cells are new technology that needs more study and optimization, it can be assumed that further investigation will lead to reduction in the calculated costs.
- Regarding the **comparison with quarry run and the use of geocontainers**, geotextile cells offer several advantages at a lower cost (for a first approximation). These advantages could compensate the drawback of being new technology that still needs practical validation.
- The top part of the cross-section, which is made of **quarry run**, requires attention as well. In case large water heads occur, significant seepage might happen. This means that small grains would be washed away. In order to avoid material losses, it would be necessary to take measures. These measures could be to choose a quarry run gradation with large diameters, to place a geometrically closed granular filter or a geotextile layer. This would make the dam more expensive than using a standard quarry run core.

5.2. RECOMMENDATIONS FOR FURTHER STUDY

Some recommendations for further study can be done regarding the subject at hand. The topic itself is very wide and there are issues related to this research that could not be addressed to start with. Besides that, interesting questions have arisen through the work process and when finding the results.

BARRIER CONSIDERED ON A SYSTEM LEVEL

- In this thesis, some assumptions have been made regarding typhoon statistics and expected damage caused by a typhoon. Further study would be needed in order to work with more precise information. This way a more accurate cost benefit analysis can be performed.
- According to the obtained results, a return period of 500 years minimizes total costs. However, the analysis does not clarify if larger return periods could be even more favorable. Therefore, the optimal return period might be larger than 500 years and this could be subject of further study.

- Further study in detail to calculate precisely barrier dimensions would be necessary. For example, permeability of the dam or its bottom foundation was not taken into account but their contribution to water levels should be studied as well.
- In performed calculations, the freeboard of the dam was set first and then the permanent opening was calculated. It would be worth studying how total costs vary while changing the dam freeboard (allowed overtopping), and keeping the same water levels inside the bay.
- The effects of the barrier in sediment transport have not been studied here, but would need to be taken into account in case the barrier is built. These effects may influence the evaluation of alternatives goodness and also the final design of the chosen alternative.
- The part related to movable barrier has not been developed here and can be subject of further study as well (calculation of environmental section, gates design, etc).

DAM DESIGN

On a **general** note, further investigation can be done regarding the following topics:

- It has been assumed that sand will be an available and relatively cheap material for filling. It should be confirmed that supply of sufficient volume of sand can be guaranteed and at a reasonable cost. Sand filling represents a 56% of the dam core, which means that approximately 7.5Mm^3 of sand would be necessary for the project.
- Water flow through the barrier has quite high current velocities. This is relevant for operational conditions, but also for construction methods applied. The question arises whether is possible to sink caissons with current velocities up to 3.5 m/s, which is more or less the maximum that would be found at the closure gap. The feasibility of this operation can be studied, as well as the development of a construction method that makes it possible.
- In case a composite cross-section is considered for the dam, not only construction costs are relevant for comparing it with the rubble mound cross section. Reparation costs in case of an earthquake should be studied as well, to be added to total costs.
  First of all, it should be determined whether reparation in case of earthquake is possible at all for the composite cross section (what if the caissons are lost, fallen along the slope of the dam).
  Also, the calculation of a damage function (related to intensity and probability of earthquake) would help to determine the total expected damage during the structure lifetime. This would allow a detailed comparison between the two types of cross-section.
- It has been mentioned that due to seepage through the dam core, filters or an expensive rock fill could be needed. This issue could be studied, including how seepage and overall stability are affected by the height of the geotextile structure inside the dam core. A very high geotextile structure would act as an impermeable core and could stop or divert the seepage.

Since the limiting load cases and certain assumptions have been chosen on the conservative side, to prove the feasibility of the geotextile structure, there is room left for improvement in the design of geotextile cells. Modifications can be done and more detailed calculations such as:

- In general, further study and calculations would be needed in order to check the behavior of geotextile cells regarding settlements and deformations in case of earthquake.
- The sealing of geotextile cells has been considered in order to prevent sand losses in case liquefaction occurs, but it has not been defined or studied in detail. Further research can be

done regarding the probability and extent of liquefaction in case of earthquake. The sealing of cells could be avoided or modified if sand losses or settlements are sufficiently small.

- There is room left for optimization regarding the construction method. A sensible construction procedure has been established and checked here, as economically feasible. However, it can be investigated whether certain modifications lower the cost and make it optimal. Certain parameters could be studied here, for example variations in cells geometry, size of geotextile elements, or working rates.
- Since the largest loads will act on the edges, a more detailed calculation of pressures acting in the core of the structure could allow an optimization in the design. This would imply modifying geometry or materials (larger size of cells, lower strength) for central cells.
- Regarding the necessary reinforcement at the edge of the structure, a solution with stainless steel cables has been considered for simplicity, but there are other options that could be studied. The production of a stronger geotextile sheet can be considered. For example, this could be achieved by using a geotextile reinforced with fibers.
- Concerning the geotextile there are other aspects that can be improved. The working tensile strength of geotextile is calculated by reducing the nominal strength by 2, which is the seam safety coefficient. This coefficient is quite high compared to other safety coefficients used in engineering calculations, which suggests that there is room for improvement regarding this aspect. Further study on how to do it could be done, in order to identify the source of uncertainties and reduce it.

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Appendix 1

BOUNDARY CONDITIONS AND REQUIREMENTS

APPENDIX 1 – BOUNDARY CONDITIONS AND REQUIREMENTS

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List of Figures

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A1-1. BATHYMETRY

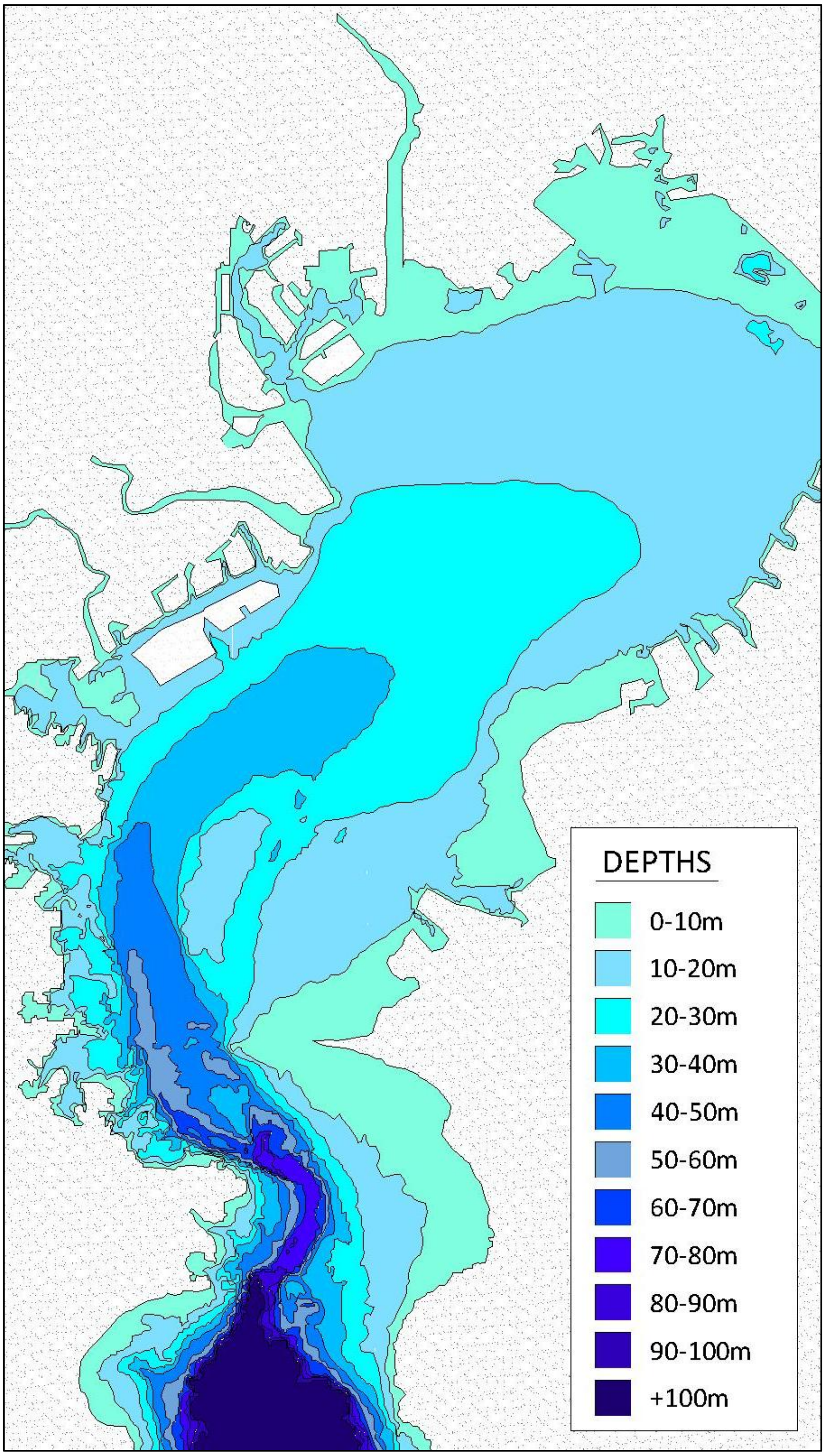


Figure A1 1: Bathymetry Tokyo Bay

A1-2. GEOLOGICAL MAP

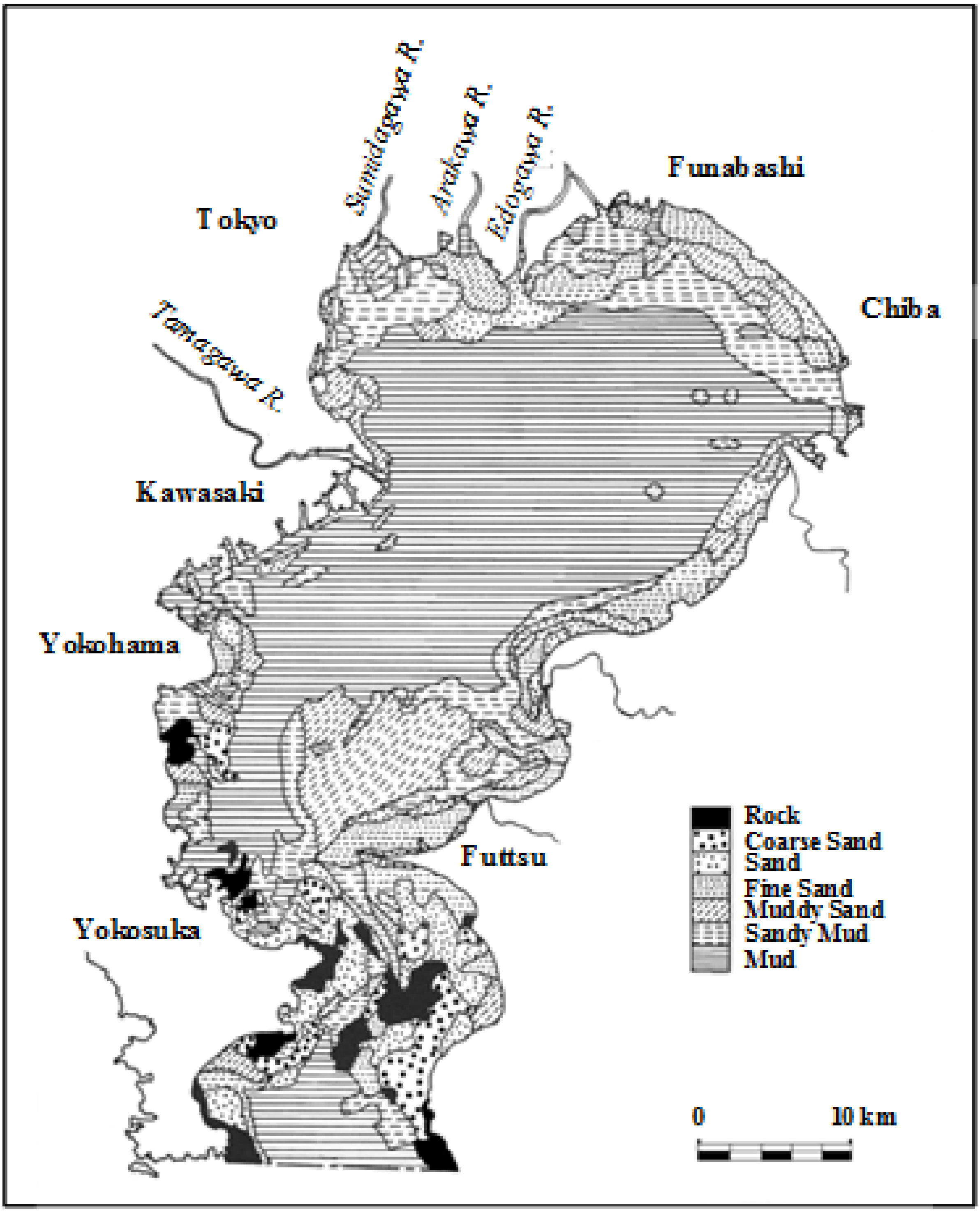


Figure A1 2: Investigated bed characteristics before 1959 by Secretariat of Committee on Development in Tokyo Metropolitan Area (Kaizuka, 1993)

A1-3. TIDES

Tides data obtained from simulation and observations are shown in the following figure.

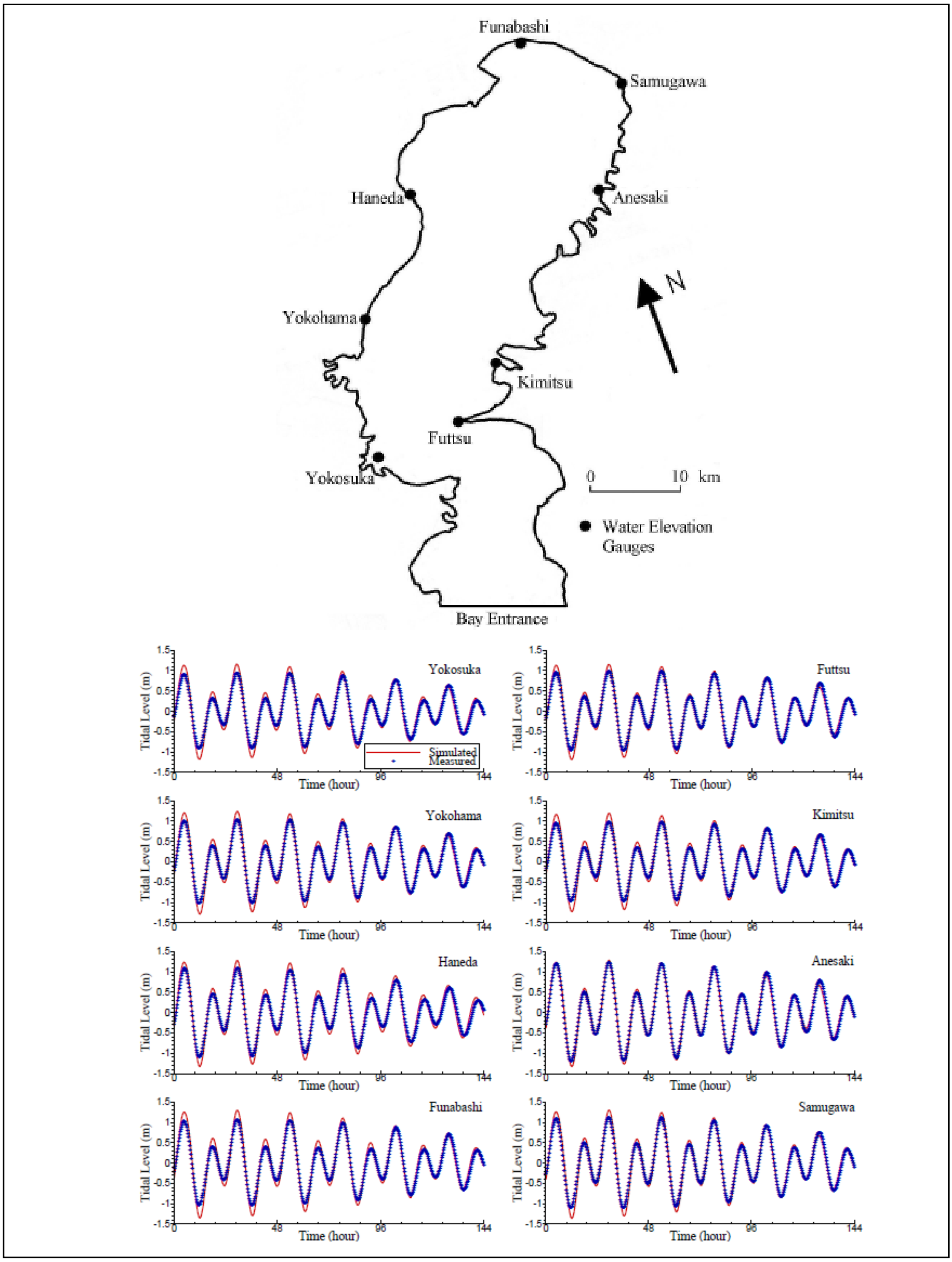


Figure A1 3:Observed vs. simulated tidal levels in Tokyo Bay (W. Wu, 2004)

A1-4. RAINFALL

In The following figure, statistics on extreme rainfall in Japan are shown.

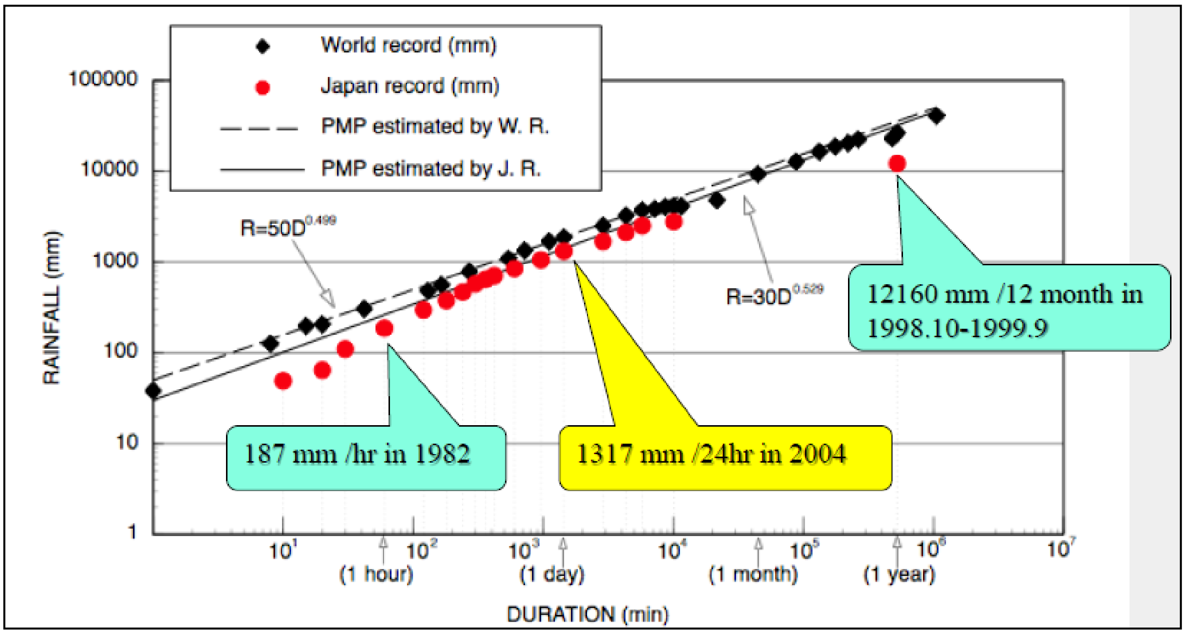
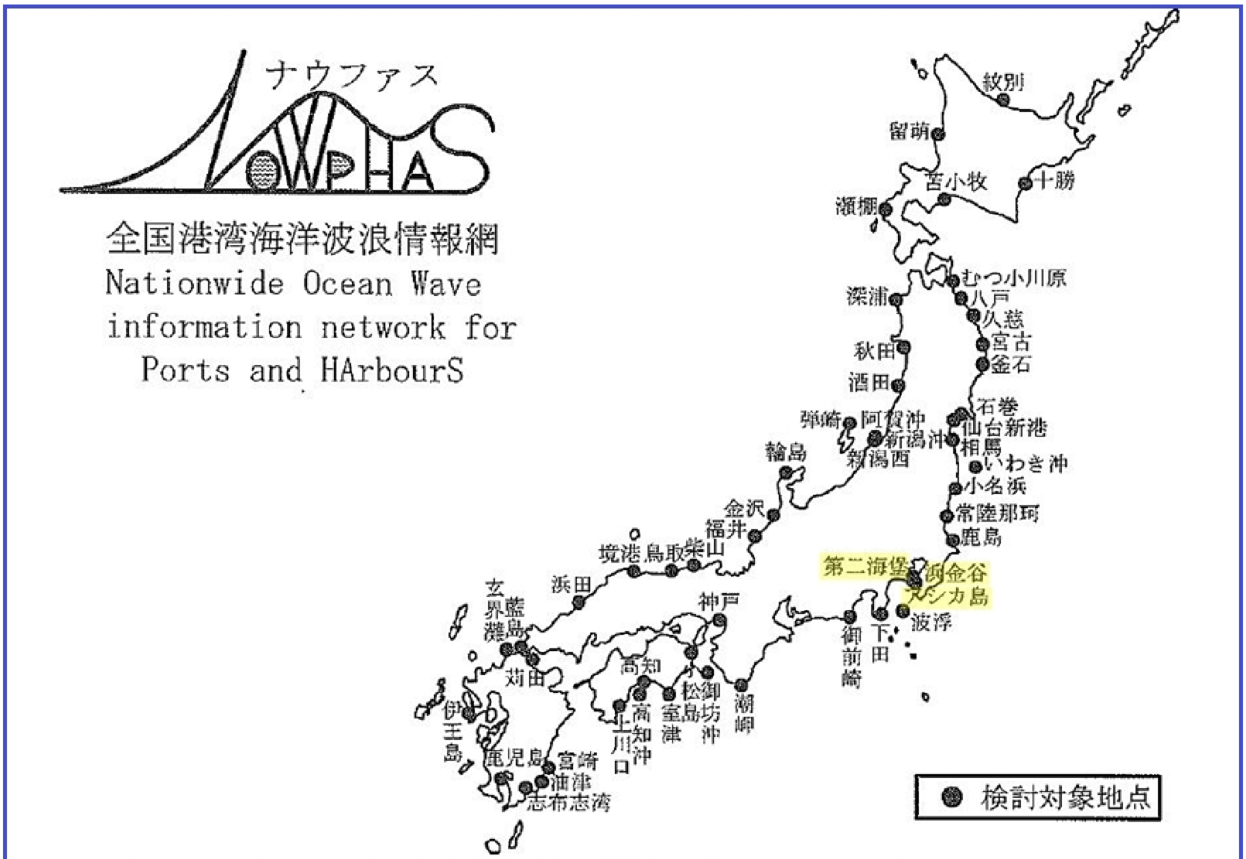


Figure A1 4: Extreme precipitation values for different rainfall durations in Japan (Takara, 2012)

A1-5. WAVE CLIMATE DATA

Wave data are retrieved from NOWPHAS report (Nagai, 2002), where the parameters for wave climate (significant wave height and period) are calculated from data obtained from buoys.

There are three buoys in the Tokyo Bay area. The buoy locations and the wave height and period calculated can be found below. Also, the measurement periods are specified for each buoy.



地点名	波高計機種	水深 (m)	最大有義波および対応最高波				起時	発生要因
			$H_{1/3}$(m)	$T_{1/3}$(s)	H_{max}(m)	T_{max}(s)		
浜金谷	USW	-22.0	7.30	11.7	10.06	12.2	85年 7月 1日 6時	台風8506号
第二海堡	USW	-16.5	1.86	4.9	3.29	4.1	95年 4月23日12時	寒冷前線の通過
アシカ島	USW	-21.7	6.12	8.0	-	-	98年 9月16日 8時	台風9805号

整理番号	地点名	既往最大有義波検討対象期間	波高・周期統計期間	波向統計期間
37	浜金谷	1972.09.05.20 ～ 1990.09.25.16	1972.09.05.20 ～ 1990.09.25.16	
38	第二海堡	1991.01.11.14 ～ 1999.12.31.22	1991.01.11.14 ～ 1999.12.31.22	
39	アシカ島	1991.12.25.18 ～ 1999.12.31.22	1991.12.25.18 ～ 1999.12.31.22	

Figure A1 5: Buoy data Tokyo Bay (Nagai, 2002)

A1-6. TSUNAMI HEIGHTS

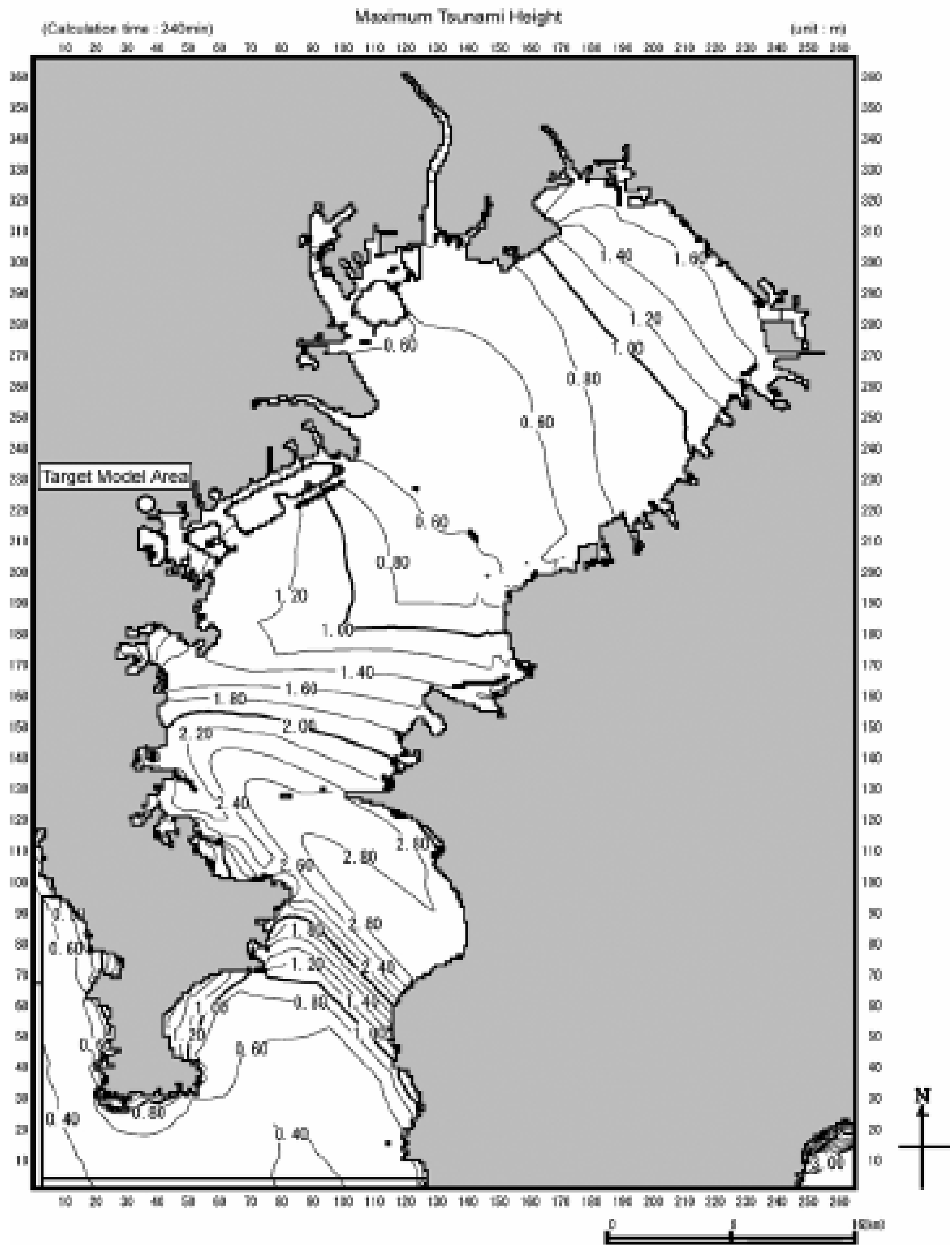


Figure A1 6: Maximum tsunami height in Tokyo Bay due to Great Kanto Earthquake in 1923 (T. Yasuda, 2004)

A1-7. WIDTH OF NAVIGATIONAL CHANNEL

For channel width calculation, Harbour approach channels design guidelines (PIANC-IAPH, 2014) are used. In these guidelines, a Japanese Design Method and Japanese standards are included and will be the base for calculations (Appendix G of PIANC guidelines).

A two-way channel is envisaged, which would suffice to serve the actual traffic (data on vessel traffic and an analysis involving queuing theory would be needed here in order to obtain a precise result). The design ship is chosen within the list provided in the guidelines (largest ship for each category). Navigational environment and sectional conditions are described in the following figure.

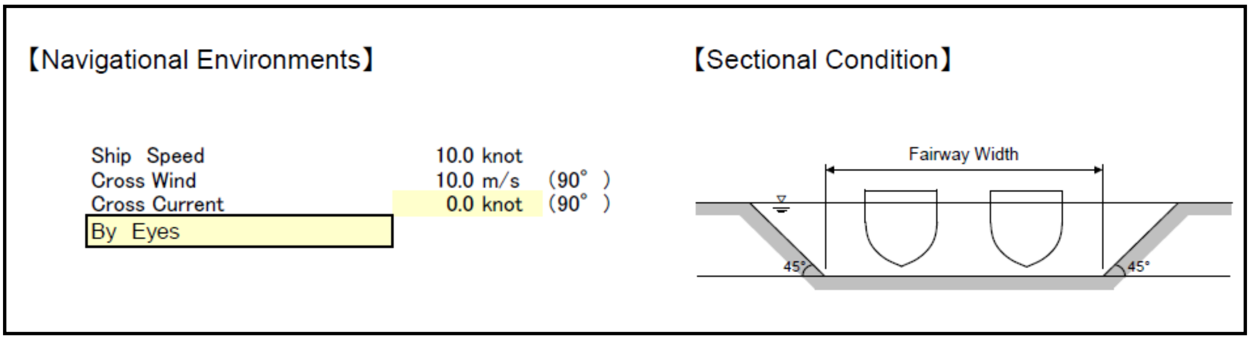


Figure A1 7: Navigational and sectional conditions for channel width calculation

According to previously presented conditions, channel width is calculated for different types of vessels (design vessels are given by Japanese standards). The minimum navigation channel width is 625 meters. The details of calculations are shown in the following table.

Table A1 1: Channel width calculation

		SHIP TYPE					
TWO WAY CHANNEL		**CONTAINER**		**VLCC**		**LNG**	
	Loa	299.9	m	333	m	283	m
	Lpp	283.8	m	316	m	270	m
	B	40	m	60	m	44.8	m
	DR	14	m	20.4	m	10.8	m
Wtotal for risk=(Wbm+Wif)*Csf		**473.6**		**624.2**	10.4	**536.9**	12.0
Wtotal=(Wbm+Wif)		430.5	10.8 (*B)	480.1	8.0 (*B)	413.0	9.2 (*B)
Wbm	Wwf+Wcf	41.0		61.2		45.8	
	Wym	4.8		4.8		4.8	
	Wdd	109.7		122.1		104.4	
	Wwf+Wcf+Wym+Wdd	155.5		188.1		155.0	
	a	2.0		2.0		2.0	
Wbm=a(Wwf+Wcf+Wym+Wdd)		311.1	7.8 (*B)	376.2	6.3 (*B)	309.9	6.9 (*B)
Wif	Wba	14.4		12.9		12.4	
	Wpa	105.0		91.0		90.7	
	b	1.0		1.0		1.0	
	Wov	169.1		155.7		150.1	
	c	0.0		0.0		0.0	
Wif=Wba+bWpa+cWov		119.4	3.0 (*B)	103.9	1.7 (*B)	103.1	2.3 (*B)
Csf		1.1		1.3		1.3	

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Appendix 2

WATER LEVELS AND PRELIMINARY LAYOUT

APPENDIX 2 – WATER LEVELS AND PRELIMINARY LAYOUT

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A2-1. PERMANENT OPENING CALCULATION

As stated in section 3.3.3.2, the first step to determine dimensions of barrier permanent opening is to calculate the allowable increase in water level inside the bay during the design storm surge events.

The steps to follow are:

- Set return period (in this case, 100 years and 500 years)
- Calculation of storm water levels at the barrier location, corresponding to the chosen return period
- Set critical point regarding flooding and determine the height of coastal defences at this point
- Calculation of water levels inside the bay, taking into account sea level rise, contribution of rainfall and river discharges, overtopping and set-up (due to wind and pressure)
- Check whether coastal defences are higher than the calculated water level and if there is a margin for further level increase
- Using the previous information, calculate increase in the inner bay water level due to the permanent opening

A sketch explaining these steps can be found in the figure below.

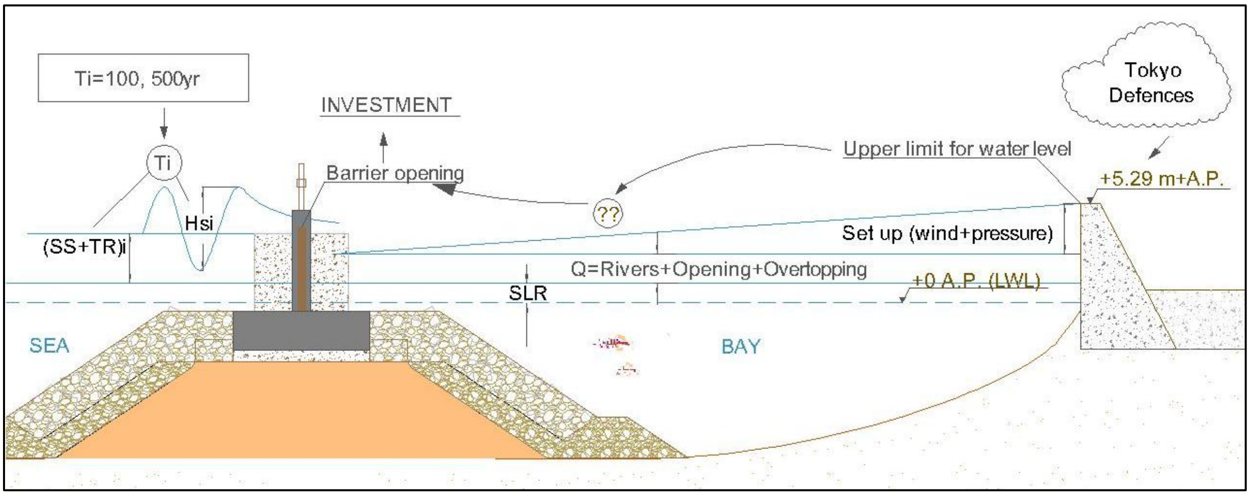

Figure A2 1: Calculation of design water levels for different return periods.

This process starts with the definition of design storm water levels at the barrier location. For the outer part of the barrier, knowing the assigned return period, there is a corresponding water level composed by storm surge and tide which can be found (see following figure).

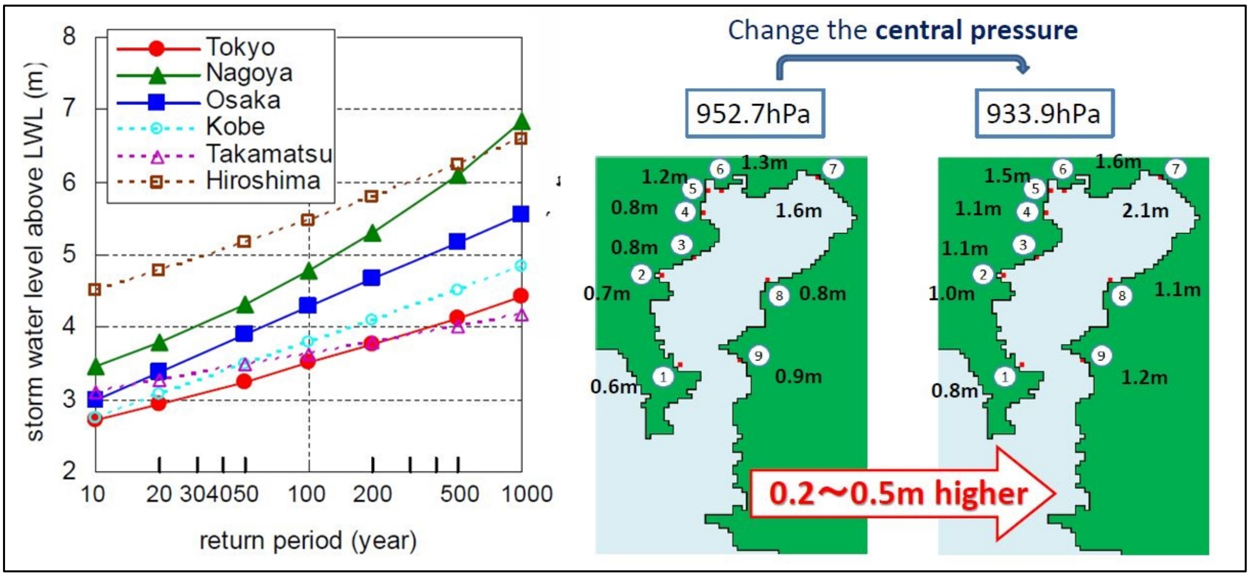

Figure A2 2: Storm surge + tide in Japanese cities (H. Kawai, 2008) and storm surge distribution in Tokyo bay (S. Hoshino, 2012).

Water levels at Figure A2 2 (left) have been calculated for Tokyo and not the barrier location. A correction will be introduced based on the ratio Tokyo/ barrier location (right), in order to have the design water level at the barrier location (see Table A2 1). Since the area around Cape Futtsu is considered to be the most likely location for the barrier, the calculation has been done only for this point. In case a different location is chosen, a review of the calculations would be needed.

Table A2 1: Storm surge + tide joint values at the bay entrance.

	Tokyo	**Cape Futtsu**	**Ratio**
Hoshino: Storm surge (m)			
1917 typhoon (point 5)	1.2	**0.9**	0.8
intensified CC (point 5)	1.5	**1.2**	0.8
Kawai: storm surge (m)			**difference**
T=100yr	2.0	1.6	-0.5
T=500yr	2.9	2.2	-0.7
Kawai: Storm surge + tide (m)			
T=100yr	3.6	**3.2**	
T=500yr	4.4	**3.7**	

The final water level in the **outer part of the barrier** will be composed by the storm surge and tide, adding the chosen value for sea level rise.

Next step in the calculation is to determine the maximum allowable water level inside the bay. Here, the most unfavorable point along coastal defences is chosen. The point to study is going to be Tokyo, as it is close to the maximum storm surge and the maximum concentration of population and assets. The critical point for elevation is taken there. Coastal defences height is defined at the lowest point (number 4, see Table 3-1).

The chosen return period is 100 years in principle. At some points, a comparison with 500 years has been done, as a barrier designed for 500 years has been included in the comparison of possible alternatives.

The chosen closure time is 12h. According to retrieved data, storm surge duration can be approximately this long. It is considered that the closing is done at low tide, so tidal peak would be within this 12h period as well.

During the closure time it is considered that some water is entering the bay. There will be an increase in water level **inside the bay** due to several contributions:

- Sea level rise
- Wind and pressure set-up built inside the bay
- River discharge and rainfall contribution
- Overtopping at the barrier
- Discharge entering through the barrier permanent opening

The allowable amount of water entering the bay is calculated setting as upper limit the minimum height of coastal defences at Tokyo. The previously mentioned contributions to increase in water level are calculated. Then, the rest of allowable water level increase would enter through a permanent opening whose dimensions are calculated, in order to set a permanent open section.

The different contributions to increase in water levels inside the bay are calculated below.

WIND AND PRESSURE SET-UP

Even if there is a barrier closing the bay mouth, some wind and pressure set-up still can be built inside when the typhoon crosses the bay.

It is considered too conservative to add both values, as maximum pressure set-up is expected close to the eye of the typhoon, and maximum wind set up is probably closer to areas with high wind velocities (see Figure A2-2-1). Therefore, set-up has been calculated at these two points and the maximum value is chosen.

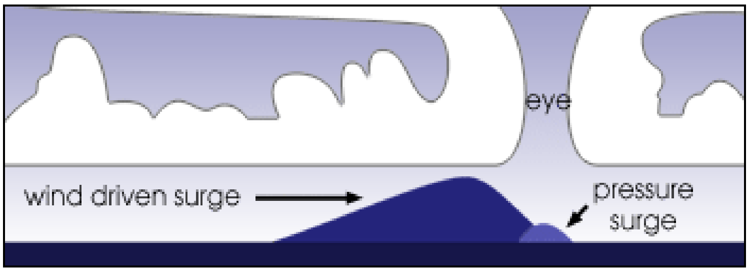

Figure A2 3: Wind and pressure surge, from http://earthobservatory.nasa.gov/Features/Hurricanes/

For **pressure set-up** calculations, the following formula is used (Inazu et al., 2006):

$$\Delta h_p = \frac{-(p_\infty - p)}{\rho_w \cdot g}$$

c_1 : constant between 0.01 – 0.04, an average value of 0.025 is chosen

pr_∞ : for peripheral pressure a value of 1013hPa is used, corresponding normal atmospheric pressure

pr : local pressure

In the eye of the typhoon, the pressure value is taken from Hoshino's MSc Thesis, corresponding to Isewan Typhoon increased intensity.

At the point with maximum velocities, a pressure drop of 2/3 in the eye is used (see figure A2-2-2).

ρ_W: water density

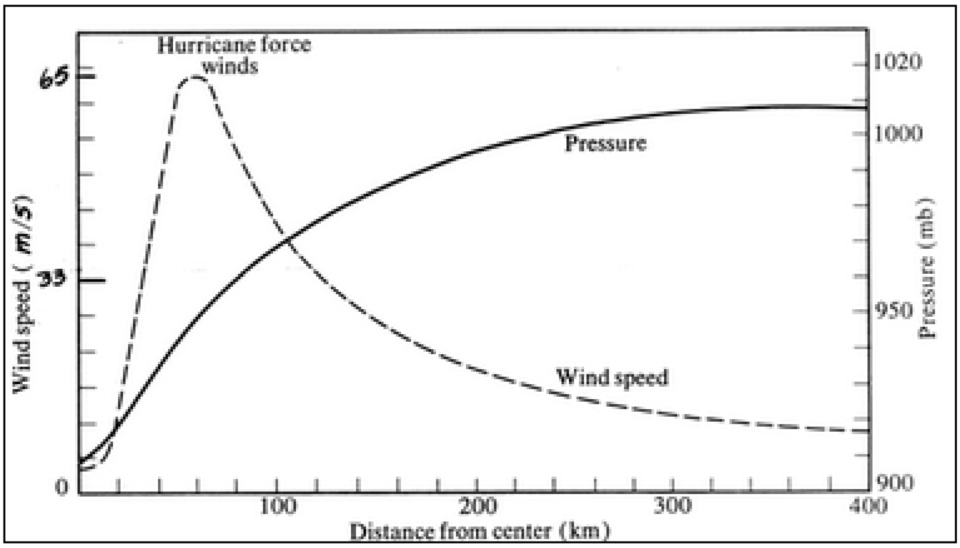

Figure A2 4: Pressure distribution in a hurricane (ATOC University of Colorado)

For **wind set-up**, the following formula is used (TU Delft, 2010):

$\Delta h_w = 0.5 * \kappa * F_{set-up} * \cos\varphi * \frac{V_s^2}{gd}$, where $\kappa = c_w * \frac{\rho_{air}}{\rho_{water}}$

c_w: constant between 0.8*10-3 - 3*10-3

φ: direction of wind approaching the coast (30º for maximum fetch in this case)

F_{set-up}: fetch length of wind set-up, 29,000m here, chosen as the most unfavourable situation regarding the study point which is Tokyo (See Figure)

V_s: wind speed at 10m height, approximated as 45 km/h

d: average water depth at the shallow area, taken as 15m (see bathymetry)

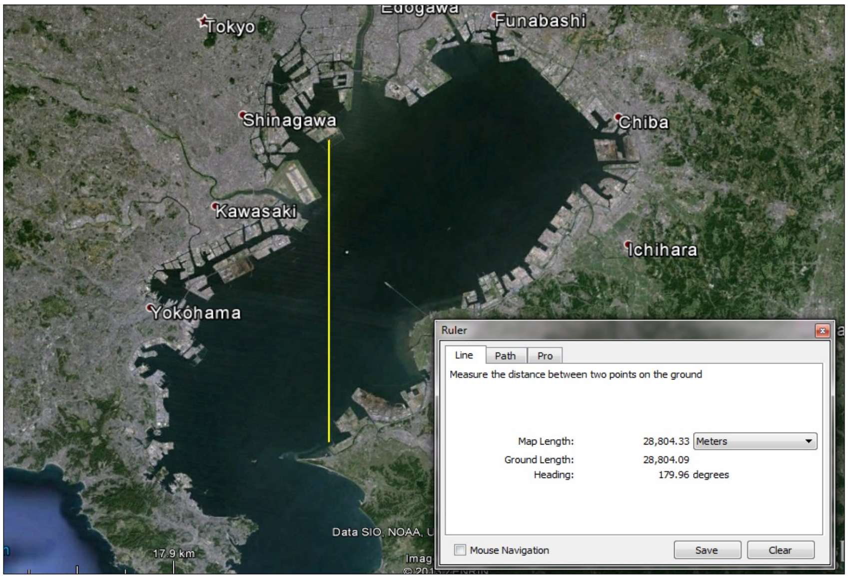

Figure A2 5: Fetch used in wind set-up calculation (Source: Google Earth).

Once the values are calculated for a return period of 100 years, estimation for a return period of 500 years is done. The values for 100 years are multiplied by the ratio between storm surges in Tokyo for 500 and 100 years, which is supposed to be similar. Final results can be found in the following table.

Table A2 2: Pressure and wind set-up estimation for Tokyo.

	POINT MAXIMUM VELOCITY (r=50000m, v=45m/s)		**POINT MINIMUM PRESSURE (r=0, v=0)**	
	100 yr	**500 yr**	**100 yr**	**500 yr**
Wind set-up				
ρa	1.225		28800	
ρw	1030		0.000003	
cd	0.0013		0	
v	45		9.8	
f	29000			
g	9.81			
D	10	estimation:	10	estimation:
		(**)		(**)
h	**0.93**	**1.15**	**0.00**	**0.00**
Pressure set-up				
Inazu et al.				
p∞ (hPa)	1013		1013	
pc (hPa)	933.9		933.9	
pr (hPa)	960.3		933.9	
rm (m)	50000		50000	
Pressure drop (hPa)	53	estimation:	79	estimation:
Δhp (m)	**0.53**	**0.65**	**0.79**	**0.98**
Total set-up (m)	**1.45**	**1.80**	**0.79**	**0.98**

(**): ratio storm surge 100 yr /500 yr for Tokyo, found in (H. Kawai, 2008)

The area with maximum wind velocities has larger values of total set-up and will be used for further calculations (1.45m for 100years and 1.80m for 500yr).

RIVER DISCHARGES AND RAINFALL CONTRIBUTION

All discharges are known, except for Nakagawa River (see 2.1.). For this one, a discharge is estimated proportional to the other ones, regarding the basin surface.

It is considered that rainfall starts at the same time in all basins and discharges start to rise at the same time so they peak within the storm surge duration. As the closure time is set in 12 hours, it is considered that all the rivers will arrive to their maximum discharges, except for Ara River: since the concentration time is 20 h, the reached discharge will be 12/20 of the peak discharge. It is assumed an average discharge of half the peak discharge for all rivers during the whole closure period.

Taking this into account, the expected increase in water level due to river discharge contribution is presented in next table.

Table A2 3: River discharge contribution to increase in water level

PEAK DISCHARGES AT t=12h (m3/s)	
Tamagawa	4500
Arakawa	3600
Tsurumi	1250
Nakagawa	3615
% equivalent constant discharge	50
Duration (s)	43200
Volume (m3)	2.80E+08
Bay area (m2)	9.22E+08
Increase water level (m)	**0.30**

OVERTOPPING

Overtopping has been checked with the help of EurOtop website calculation tool (http://www.overtopping-manual.com/).

A wave height of 5 m and a period of 8s have been chosen as reasonable values. This choice corresponds to the average significant wave height calculated based on the data registered by buoys at the entrance of Tokyo bay (Nagai, 2002). There is data registered for three buoys in the area close to the bay entrance (which is supposed to be the location for implantation of the barrier). A summary of this information can be found at Appendix 1. Buoys situated in the East registered larger wave heights, therefore this area is supposed to be more exposed to waves. As the barrier will cross the bay from one side to another, an average between the exposed part and the protected part has been chosen as the value for calculations.

The barrier is supposed to be composed of a dam and a movable barrier. The calculations are done for the total length of the barrier, for both types of cross section. Later, a percentage of the total length will be applied to the obtained results, in order to find the total amount of overtopping. The overtopping values are considered to be constant along the 12 h duration of the storm surge. Applying this to the whole length of the barrier, the amount of water that enters is found, and consequently, the increase in water level.

The check has been done for a continuous slope 1:2.5 for dams, which is considered appropriate for dam slopes in seismic areas (see 3.5.2). The movable part of the barrier is defined as a vertical wall.

The freeboard is set in 1 and 2 meters in calculations. Once the results are known, the most convenient value will be chosen.

OVERTOPPING IN DAM

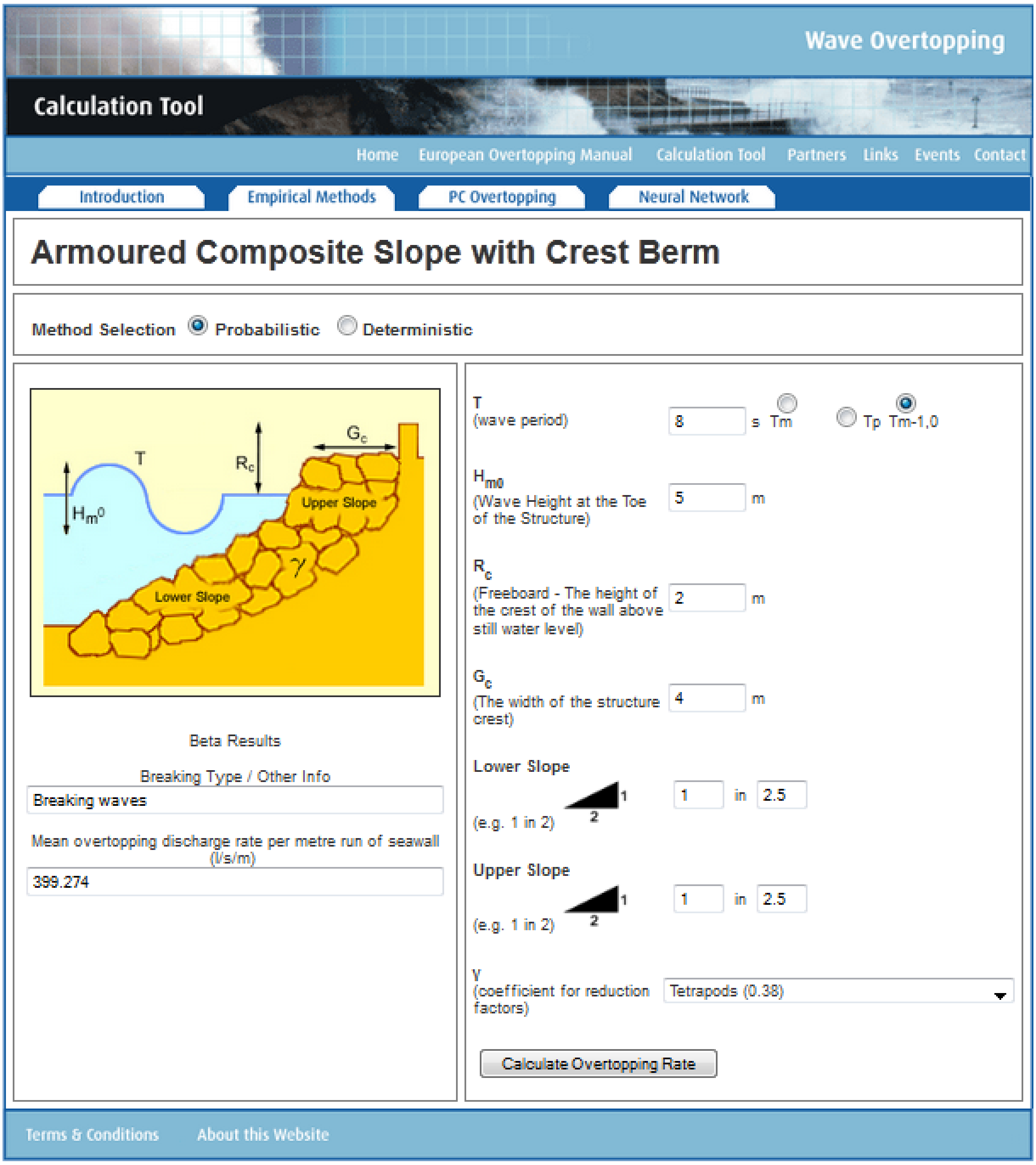

Figure A2 6: Overtopping calculations for dam, 1m freeboard (+1m crown wall)

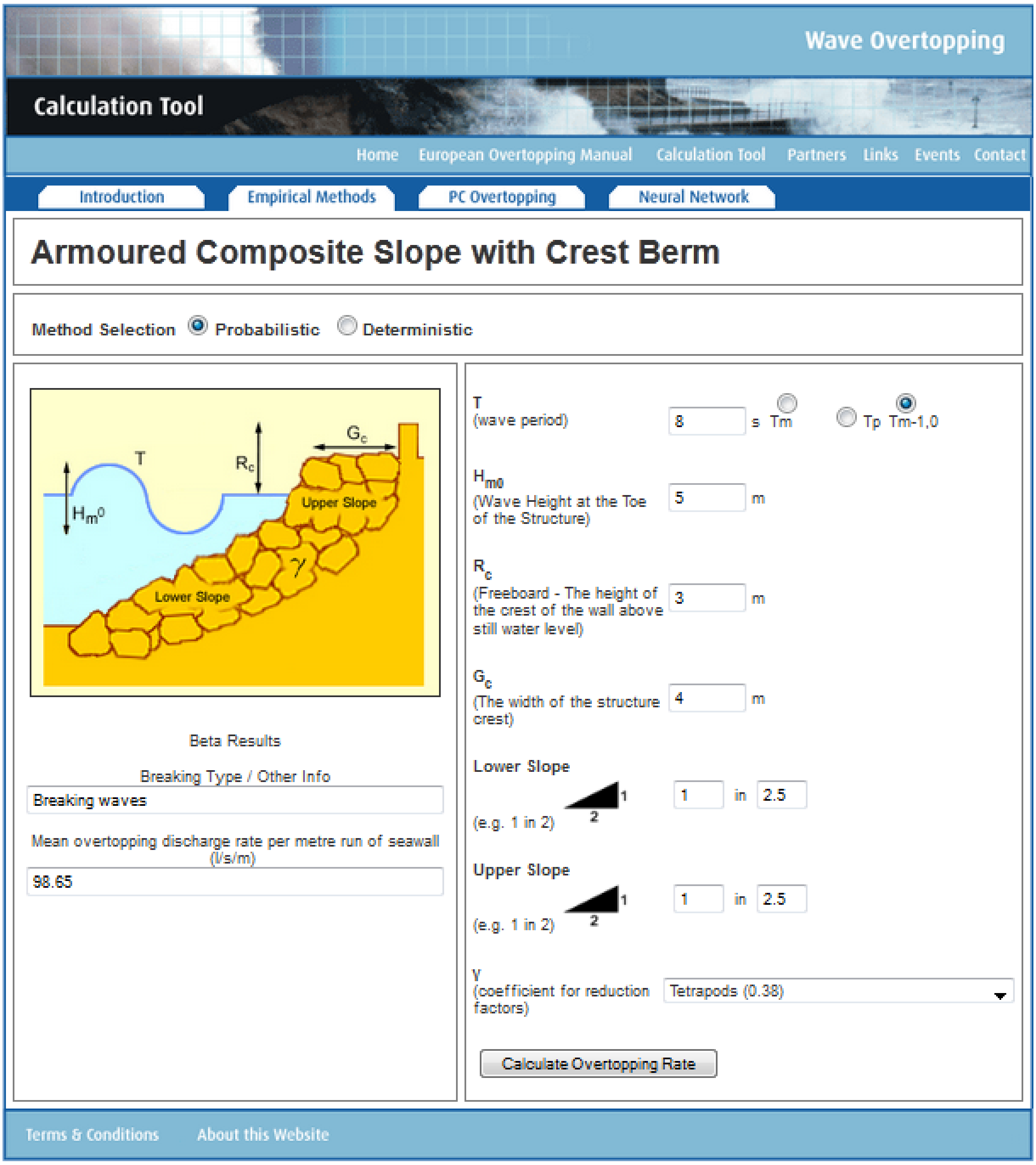

Figure A2 7: Overtopping calculations for dam, 1m freeboard (+1m crown wall)

MOVABLE BARRIER OVERTOPPING

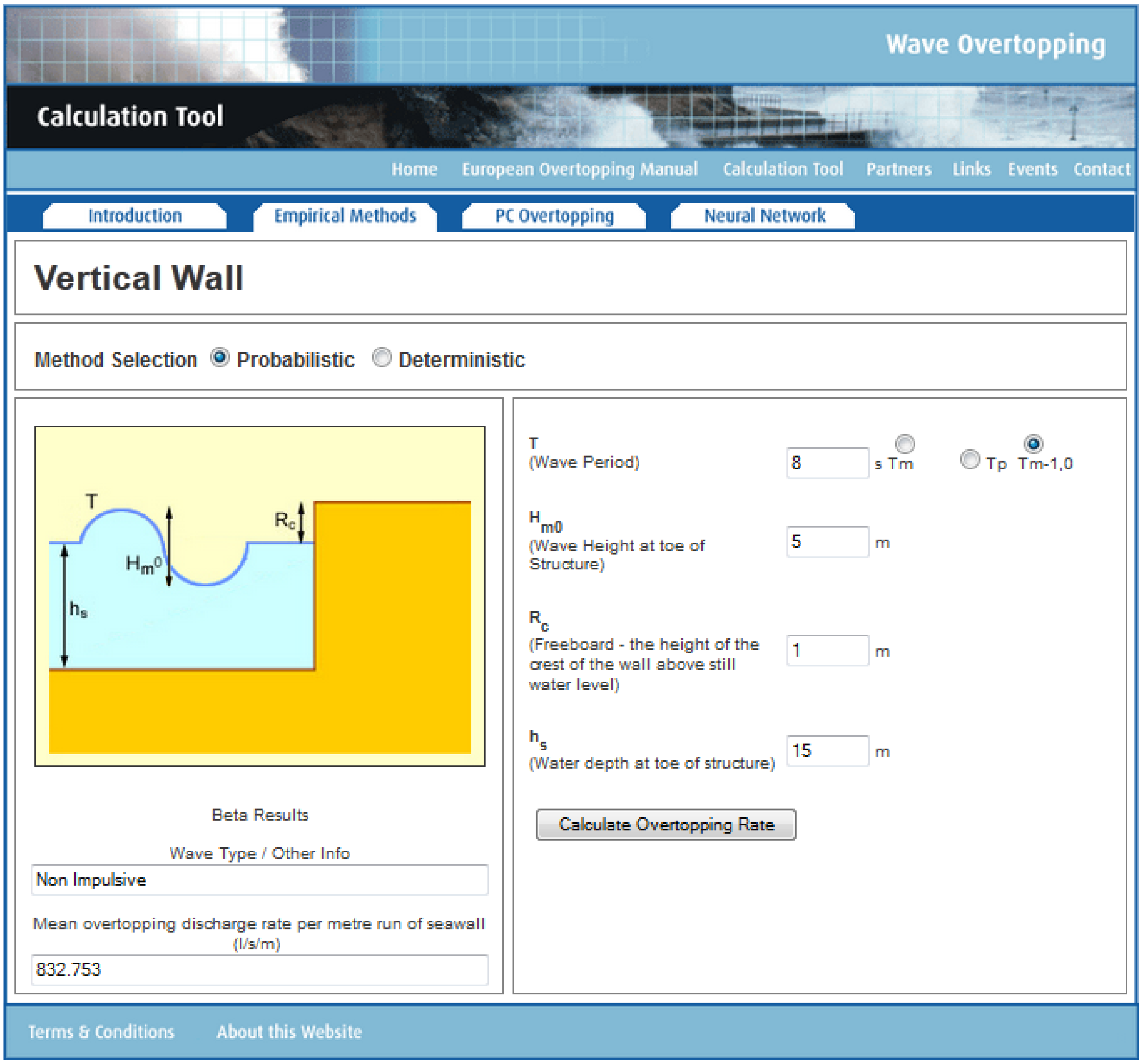

Figure A2 8: Overtopping calculations for movable barrier, 1m freeboard

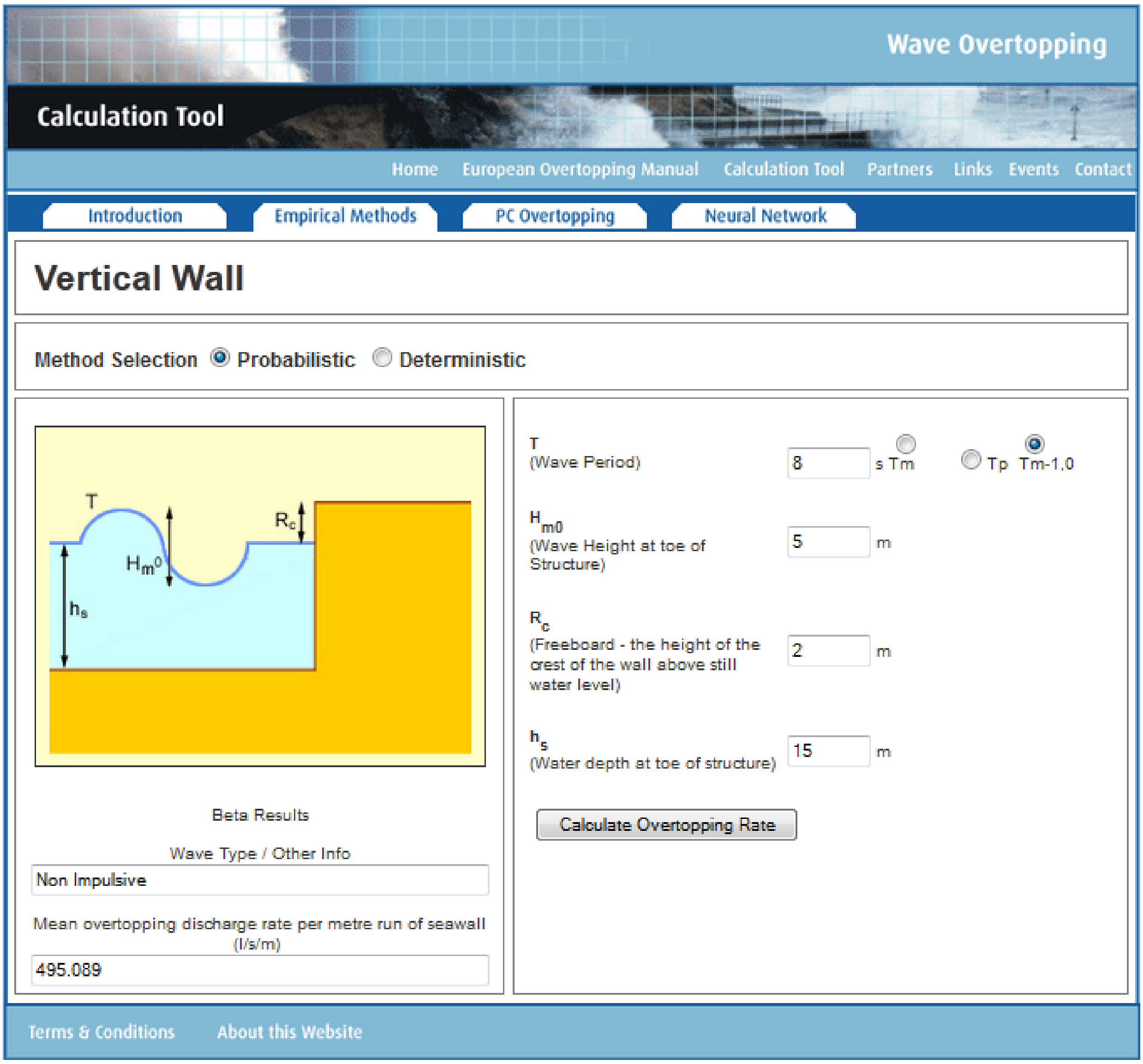

Figure A2 9: Overtopping calculations for movable barrier, 2m freeboard

In the following table the obtained results are summarized. Given a sufficient freeboard, overtopping values are low, so a more accurate calculation is not necessary at this stage. A freeboard of 2 m (over high water level during storm surge) will be considered from this point on. The corresponding water level increase is approximated considering the overtopping amount constant along the 12 h of closure. As the dam is considered to cover 70% of the barrier length (this should be checked later, when the final location is decided), the weighed increase in water level is of 7 centimeters.

Table A2 4: Overtopping estimation depending on freeboard.

	WAVE CLIMATE DATA				OVERTOPPING VOLUME ESTIMATION			
Freeboard (m)	**Hs (m)**	**T (s)**	**Duration (s)**	**Overtopping (l/s/m)**	**Barrier length (m)**	**Volume (m3)**	**Bay area (m2)**	**Increase water level (m)**
DAM								
1	5	8	43200	400	4900	8.47E+07	9.22E+08	0.09
2	5	8	43200	99	4900	2.10E+07	9.22E+08	**0.02**
GATES								
1	5	8	43200	833	2100	7.56E+07	9.22E+08	0.08
2	5	6	43200	495	2100	4.49E+07	9.22E+08	**0.05**

It has to be noted that the whole bay area has been accounted for in the calculations, while this value should vary for each barrier location. But, since the final increase is 7 cm, it is expected that the variation between locations will be less than 10 centimeters and it is not worth to extend the calculations.

ESTIMATION OF PERMANENT OPENING

Using the previously calculated data, an estimation of the allowable permanent opening is been made. In this section, a barrier location close to Cape Futtsu area is considered (close to location 5), in order to explain the followed procedure.

First of all, the allowable increase in water level due to water entering through the opening has been calculated. The point of departure is the elevation of coastal defences at the selected critical point. Then, previously calculated increases in water level are subtracted to calculate the remaining margins. The results for both operations are detailed in the following tables.

Table A2 5: Elevation coastal defences at Tokyo.

COASTAL DEFENCES ELEVATION AT TOKYO (lowest point: Shinagawa)		
Elevation Compared to storm surge	2.60	m
Tide	0.97	m
SLR	0.59	m
Coastal defences		
Elevation T.P.(MWL)	4.16	m
Elevation A.P. (low tide)	**5.29**	**m**

Table A2 6: Allowable increase in water levels due to permanent opening.

	T=100yr		T=500 yr	
Elevation coastal defences (A.P.)	5.29	m	5.29	m
SLR	1.90	m	1.90	m
Wind + Pressure setup	1.45	m	1.80	m
River discharge	0.30	m	0.30	m
Overtopping	0.07	m	0.07	m
Freeboard	0.50	m	0.50	m
Allowable increase	**1.06**	**m**	**0.72**	**m**

Once the allowable increase in water level is known, an estimation of the water entering the bay through the permanent opening has been made. The length of this permanent opening has been chosen so that the increase in water level is equal or lower than the allowable increase defined in Table A2-1-6.
In the following calculations, water head will be the value for storm surge + tide combined, approximated for Cape Futtsu (see Table A2-1-1).

Two different methods have been used in order to check the amount of water entering the bay:

1. Considering the bay and the gap in the barrier as a discrete system with storage and resistance.

The storm surge combined with tide is treated as a sinusoidal forcing *during the period* between closure (low tide) and maximum elevation inside the bay (see Figure A2-2-8).

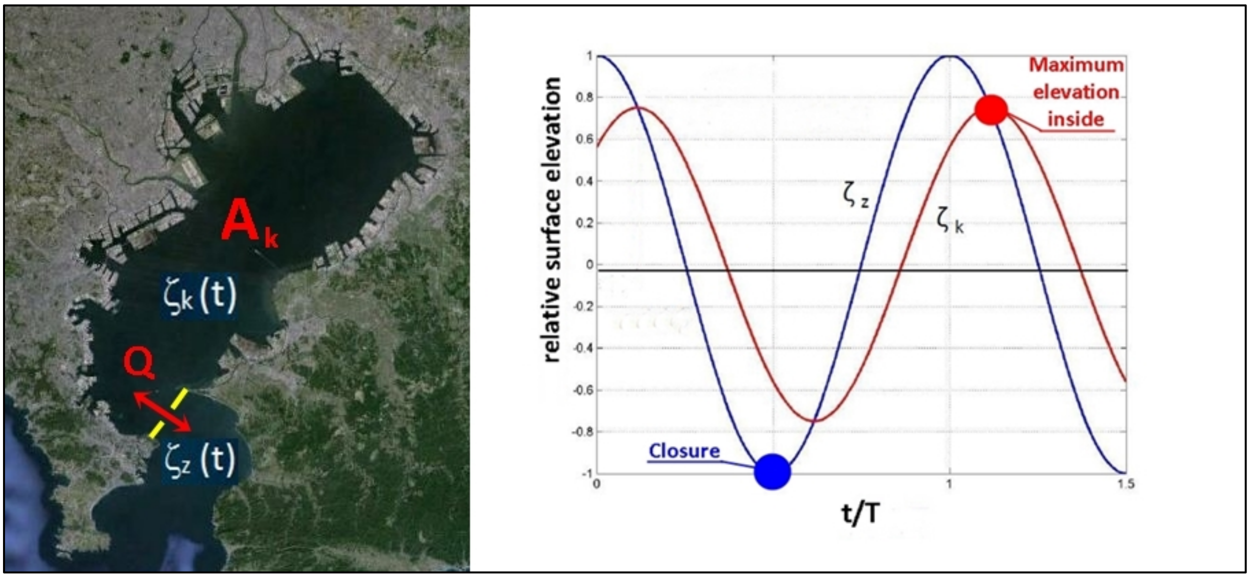

Figure A2 10: Tokyo bay assimilated to a discrete system

This method is applicable to tidal basins when the horizontal size of the system is small compared to wave length. The maximum water level inside the bay is calculated as a percentage of the maximum water level outside the bay, using this formulation:

$$\zeta_k = \hat{\zeta}_k \cdot \cos(\omega t - \vartheta) = r \cdot \hat{\zeta}_z \cdot \cos(\omega t - \vartheta)$$

$$\omega \cdot \tau = \frac{8}{3\pi} \cdot \chi \cdot \frac{A_k^2 \cdot \omega^2 \cdot \hat{\zeta}_z}{g \cdot A_s^2} \cdot r = \Gamma \cdot r$$

$$r = \cos\vartheta = \frac{1}{\sqrt{1+(\omega\cdot\tau)^2}} = \frac{1}{\sqrt{1+(\Gamma\cdot r)^2}} = \frac{1}{\sqrt{2}\cdot\Gamma}\cdot\sqrt{(\sqrt{1+4\cdot\Gamma^2}-1)}$$

Where:

$\hat{\zeta}_k$ =amplitude of sea water elevation inside the basin

$\hat{\zeta}_z$ =amplitude of sea water elevation outside the basin

$\omega = \frac{2\pi}{T}$; being T the period of the water motion, considered here to be 12 h (M-2 tide, and storm surge can be assimilated to a period of 12h, there would only be one "oscillation")

r= ratio between amplitudes

Using this method, the ratio r is calculated and consequently the water elevation amplitude inside the bay. The allowable increase in water level is two times the amplitude of water elevation inside the bay associated to storm surge and tide outside. As can be seen in Figure A2-2-8, low water levels inside and outside the bay are supposed to be different, but it is assumed that the value is small enough to be ignored at this stage and will not be calculated.

Table A2 7: Increase in water level due to permanent opening T=100 years (discrete system approximation)

RETURN PERIOD 100 YEARS			
length	475	m	permanent opening dimensions
depth	25	m	
T	12	h	M-2 tide+ storm surge 12 h
ω	0.00014537		2π/T
χ	0.5		(gap)
Ak	922000000	m^2	
As	11875	m^2	
ζz	1.6	m	0.5 *(storm surge + tide together T=100yr)
Γ	8.69		
r	0.33		
ζk	0.52	m	
Δh	1.04	m	
CONDITION: h =	**1.06**	**m max!!!**	

Table A2 8: Increase in water level due to permanent opening T=500 years (discrete system approximation)

RETURN PERIOD 500 YEARS			
length	275	m	permanent opening dimensions
depth	25	m	
T	12	h	M-2 tide+ storm surge 12 h
ω	0.00014537		2π/T
χ	0.5		(gap)
A_k	922000000	m^2	
A_s	6875	m^2	
ζz	1.9	m	0.5 *(storm surge + tide together T=500yr)
Γ	30.86		
r	0.18		
ζk	0.33	m	
Δh	0.67	m max!!!	
CONDITION: ζk =	**0.72**	**m max!!!**	

2. Considering constant water head and constant water velocity

The amount of water crossing the gap in 12 hours is calculated considering a constant velocity of $\sqrt{g \cdot d}$ and a constant head of 3.2m (combined values of storm surge + tide, 3.7m for 500 year return period). The constant head is divided in two 6h steps, first 6 hours Δh = 3.2m and, in the following 6 hours period, half of the calculated water increase is subtracted from the water head (through an iterative process).

Table A2 9: Increase water level due to permanent opening T=100 years (constant head and velocity approximation)

RETURN PERIOD 100 YEARS			
t	43200	s	(12h)
g	9.8	m/s^2	
d	25	m	
l	475	m	
c0	15.7	m/s	
Δh	3.2	m	
q	23420.0	m^3/s	
V	912979176	m^3	2steps!
A	922000000	m^2	
Increase water level	**0.99**	**m**	
CONDITION: increase =	**1.06**	**m max!!!**	

Table A2 10: Increase water level due to permanent opening T=500 years (constant head and velocity approximation)

RETURN PERIOD 500 YEARS			
t	43200	s	(12h)
g	9.8	m/s^2	
d	25	m	
l	235	m	
c0	15.65	m/s	
Δh	3.7	m	
q	13784.5485	m^3/s	
V	549410354	m^3	2steps!
A	922000000	m^2	
Increase water level	**0.60**	**m**	
CONDITION: increase =	**0.72**	**m max!!!**	

Both methods give similar results, which indicate that the approximation is good. As expected, taking constant water heads and velocities gives more conservative values. For both, it is found a valid permanent opening of 475*25m in case of T=100yr and 235*25m in case of T=500yr.

In the following tables, the results of permanent opening for different locations are shown, for 100 and 500 years. The method used in these calculations is the previously explained discrete system approximation.

Table A2 11: Allowable permanent opening (T=100yr)

ALLOWABLE PERMANENT OPENING T=100 years							
Barrier location	**1**	**2**	**3**	**4**	**5**	**6**	**Units**
Length	250	350	375	425	450	500	m
depth	25	25	25	25	25	25	m
T (2)	12	12	12	12	12	12	h
ω (3)	0.000145	0.000145	0.000145	0.000145	0.000145	0.000145	
χ (4)	0.5	0.5	0.5	0.5	0.5	0.5	
A_k	489000127	704735628	747377409	849187074	881796158	983113648	m^2
A_s	**6250**	**8750**	**9375**	**10625**	**11250**	**12500**	**m^2**
$ζ_z$ (5)	1.58	1.58	1.58	1.58	1.58	1.58	m
Γ	8.83	9.36	9.17	9.21	8.86	8.92	
r	0.33	0.32	0.32	0.32	0.33	0.33	
ζk	0.52	0.50	0.51	0.51	0.51	0.51	m
Δh	1.03	1.00	1.01	1.01	1.03	1.03	m
CONDITION	**h=1.06 m maximum**						

(1): permanent opening length
(2): M-2 tide and storm surge with a duration of 12 h

(3): 2π/T
(4): gap
(5): 0.5 *(storm surge + tide together for a T=100yr)

Table A2 12: Allowable permanent opening (T=500yr)

ALLOWABLE PERMANENT OPENING T=500 years							
Barrier location	**1**	**2**	**3**	**4**	**5**	**6**	**Units**
length	150	225	225	250	275	300	m
depth	25	25	25	25	25	25	m
T	12	12	12	12	12	12	h
ω	0.000145	0.000145	0.000145	0.000145	0.000145	0.000145	
χ	0.5	0.5	0.5	0.5	0.5	0.5	
Ak	489000127	704735628	747377409	849187074	881796158	983113648	m^2
As	**3750**	**5625**	**5625**	**6250**	**6875**	**7500**	**m^2**
ζz	1.87	1.87	1.87	1.87	1.87	1.87	m
Γ	29.17	26.93	30.29	31.67	28.23	29.48	
r	0.18	0.19	0.18	0.18	0.19	0.18	
ζk	0.34	0.36	0.34	0.33	0.35	0.34	m
Δh	0.69	0.72	0.68	0.66	0.70	0.68	m
CONDITION	**h=0.72 m maximum**						

In the following tables, a summary of the results is shown. In the permanent opening is included the allowable failure of gates, so the final width of the open channel is lower than the previously calculated.

Since the movable barrier should allow failure of a small percentage of its units, there is a water inflow in the bay during storm surge events that was not accounted for previously. This leads to a reduction of the permanent opening. It is assumed that a 10% of gates cross can fail when closing. This value has to be subtracted from the permanent opening, which would be reduced.

Table A2 13: Permanent opening final dimensions T=100 years

PERMANENT CHANNEL RETURN PERIOD 100 YEARS					
	Environmental section (m^2)	**10% failure (m^2)**	**Permanent channel (m^2)**	**Final perm. channel (m^2)**	**Channel width (m)**
LOCATION 1	18,750	1,875	6,250	4,375	175
LOCATION 2	27,000	2,700	8,750	6,050	242
LOCATION 3	28,500	2,850	9,375	6,525	261
LOCATION 4	33,000	3,300	10,625	7,325	293
LOCATION 5	33,750	3,375	11,250	7,875	315
LOCATION 6	37,500	3,750	12,500	8,750	350

Table A2 14: Permanent opening final dimensions T=500 years

PERMANENT CHANNEL RETURN PERIOD 500 YEARS					
	Environmental section (m^2)	**10% failure (m^2)**	**Permanent channel (m^2)**	**Final perm. channel (m^2)**	**Channel width (m)**
LOCATION 1	18,750	1,875	3,750	1,875	75
LOCATION 2	27,000	2,700	5,625	2,925	117
LOCATION 3	28,500	2,850	5,625	2,775	111
LOCATION 4	33,000	3,300	6,250	2,950	118
LOCATION 5	33,750	3,375	6,875	3,500	140
LOCATION 6	37,500	3,750	7,500	3,750	150

A2-2. ENVIRONMENTAL SECTION

Once the permanent opening has been calculated, the open section during normal conditions has to be set. Due to environmental reasons there is need to keep a minimum water turnover and a minimum tidal range inside the basin. This condition is the base to calculate the movable barrier needed. This movable barrier will be open during normal conditions and will close in case of storm surge (leaving open to water flow just the permanent opening section).

Since a percentage of current cross section needs to remain open in normal conditions due to environmental reasons, reference values for that opening have been investigated. Values for Eastern Scheldt storm surge barrier are used as a reference in this report. There, a 18% of the original cross section remains open due to environmental reasons (A. van der Toorn, 2013, De Bok et al., 2001). In this case different locations are studied, in which the ratios closed bay area compared to closure section can be very different from Eastern Scheldt case. A parameter that reflects better the maintenance of a minimum water exchange is the reduction in tidal range inside the bay. A rough calculation to determine the opening that allows a minimum tidal range will be performed, following the discrete system approximation that has been used before. A tidal range of 87% will be used, the same value than for Eastern Scheldt.

Keeping this reduction in tidal range as reference, calculations have been made for each location. The same approximation used for permanent opening is done again, comparing the bay to a discrete system with storage and resistance. In order to perform the calculation, the ratio between tidal range inside and outside the barrier is fixed (r=0.87, the value set for Eastern Scheldt). Once this ratio is fixed, the needed opening in normal conditions is calculated. This gives a different result from the first guess. For example in case of location 1 (which only closes half of the bay), a value much lower than a 18% of the total section is enough to keep the tidal range, due to the small enclosed surface to fill.

In the following pages, the results for each barrier location are shown (100 and 500 years). After calculating the environmental section, the gates dimensions are calculated by subtracting the permanent opening from this cross section and assuming a gate depth (see last table for summary of results).

Table A2 15: Environmental opening calculation Location 1.

TIDAL RANGE REDUCTION NORMAL CONDITIONS							
Barrier location	**1**	**2**	**3**	**4**	**5**	**6**	**Units**
length	1250	1800	1900	2200	2250	2500	m
depth	15	15	15	15	15	15	m
T	12	12	12	12	12	12	h
ω	0.000145	0.000145	0.000145	0.000145	0.000145	0.000145	
χ	0.5	0.5	0.5	0.5	0.5	0.5	
A_k	489000127	704735628	747377409	849187074	881796158	983113648	m^2
A_s	**18750**	**27000**	**28500**	**33000**	**33750**	**37500**	**m^2**
ζ_z	1.05	1.05	1.05	1.05	1.05	1.05	m
Γ	0.65	0.66	0.66	0.64	0.66	0.66	
r	0.87	0.87	0.87	0.87	0.87	0.87	
ζ_k	0.91	0.91	0.91	0.92	0.91	0.91	m
Tidal range inside	1.83	1.83	1.82	1.84	1.82	1.82	m

Table A2 16: Environmental section for all locations.

ENVIRONMENTAL SECTION / TOTAL CROSS SECTION			
	Total cross-section (m^2)	**Environmental section (m^2)**	**% of total cross section**
LOCATION 1	292,236	18,750	6%
LOCATION 2	287,403	27,000	9%
LOCATION 3	315,058	28,500	9%
LOCATION 4	274,459	33,000	12%
LOCATION 5	240,560	33,750	14%
LOCATION 6	313,788	37,500	12%

Next step would be to check flow velocities for open cross-sections (permanent opening and environmental sections). This could lead to consider some measures aimed to allow navigation (changing entrance shape to limit velocities) or to prevent scour (bed protection). These measures will not be discussed here. However, the velocity check will be done later in the design phase, in order to account for its influence in the elements to be defined.

Gates dimensions are calculated from environmental section as explained previously, subtracting the permanent channel from it.

In the following tables a summary of gates dimensions is offered. Gates length has been estimated considering a depth of 15 meters. This value is intended to be used as guidance, in order to work with it in the process of setting the barrier general layout. The height of the barriers could be different, and several heights could be used, but this will not be addressed here.

Table A2 17: Gates cross section and dimensions T=100 years.

GATES 15m RETURN PERIOD 100 YEARS				
	Environmental section (m^2)	Permanent channel (m^2)	Gates section (m^2)	Gates length (m)
LOCATION 1	18,750	4,375	14,375	958
LOCATION 2	27,000	6,050	20,950	1,397
LOCATION 3	28,500	6,525	21,975	1,465
LOCATION 4	33,000	7,325	25,675	1,712
LOCATION 5	33,750	7,875	25,875	1,725
LOCATION 6	37,500	8,750	28,750	1,917

Table A2 18: Gates cross section and dimensions T=500 years.

GATES 15m RETURN PERIOD 500 YEARS				
	Environmental section (m^2)	Permanent channel (m^2)	Gates section (m^2)	Gates length (m)
LOCATION 1	18,750	1,875	16,875	1,125
LOCATION 2	27,000	2,925	24,075	1,605
LOCATION 3	28,500	2,775	25,725	1,715
LOCATION 4	33,000	2,950	30,050	2,003
LOCATION 5	33,750	3,500	30,250	2,017
LOCATION 6	37,500	3,750	33,750	2,250

A2-3. PRELIMINARY LAYOUT

With the information available so far, it is possible to define a preliminary layout of the barrier, in order to analyze possible locations and costs. A summary of results can be found in the following tables, which has been calculated for an open channel 25 meters deep (in order to use it as navigation channel) and movable barrier 15 meters deep (as a first approximation).

Table A2 19: Dimensions of gates and permanent opening (T=100 years)

	Environmental section (m^2)	Open channel (m^2)	Gates (m^2)	Open length (m)	Gates length (m)
LOCATION 1	18,750	4,375	14,375	175	958
LOCATION 2	27,000	6,050	20,950	242	1,397
LOCATION 3	28,500	6,525	21,975	261	1,465
LOCATION 4	33,000	7,325	25,675	293	1,712
LOCATION 5	33,750	7,875	25,875	315	1,725
LOCATION 6	37,500	8,750	28,750	350	1,917

Table A2 20: Dimensions of gates and permanent opening (T=500 years)

	Environmental section (m^2)	Open channel (m^2)	Gates (m^2)	Open length (m)	Gates length (m)
LOCATION 1	18,750	1,875	16,875	75	1,125
LOCATION 2	27,000	2,925	24,075	117	1,605
LOCATION 3	28,500	2,775	25,725	111	1,715
LOCATION 4	33,000	2,950	30,050	118	2,003
LOCATION 5	33,750	3,500	30,250	140	2,017
LOCATION 6	37,500	3,750	33,750	150	2,250

As can be seen, the permanent opening has been reduced and it is smaller than the navigational channel (which has a width of 625 meters). This means that a part of the navigational channel would need to be closed during typhoons. In any case, this circumstance will not be addressed in this report.

Appendix 3

TOTAL COSTS

APPENDIX 3 – TOTAL COSTS

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A3-1. ALTERNATIVE LOCATIONS DETAILS

Locations are specified in the following map. All cross sections are taken in the same direction AA' (West to East).

Figure A3 1: Cross section locations.

Vertical dimensions have been scaled 10 times compared to horizontal dimensions.

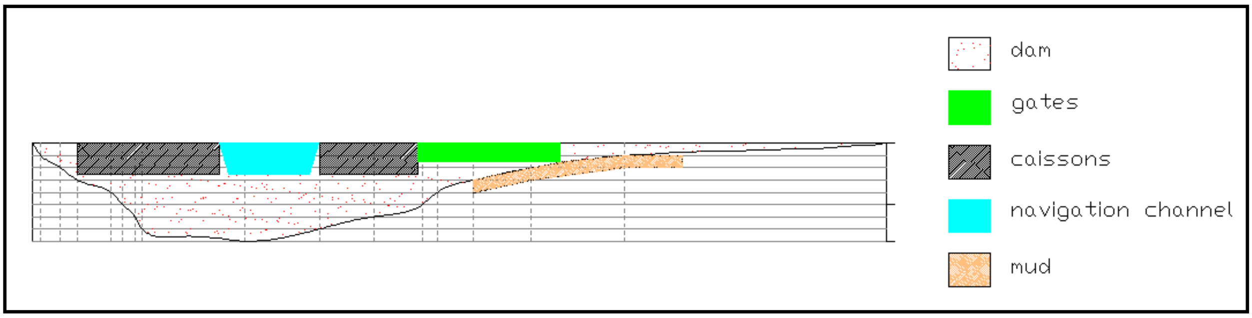

Figure A3 2: Example cross section.

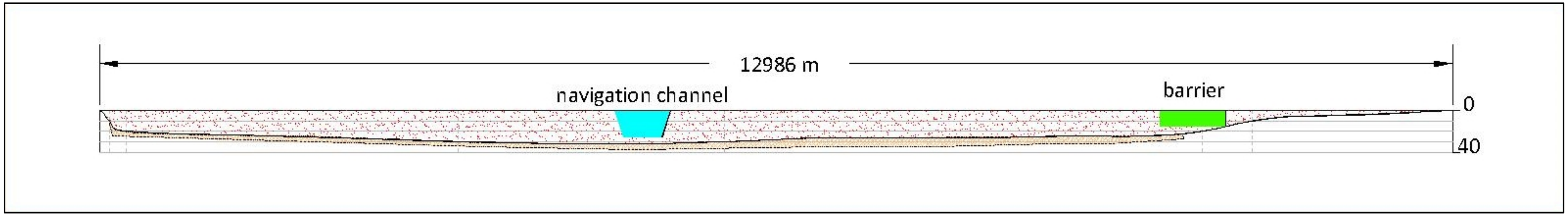

Figure A3 3: Location 1 layout

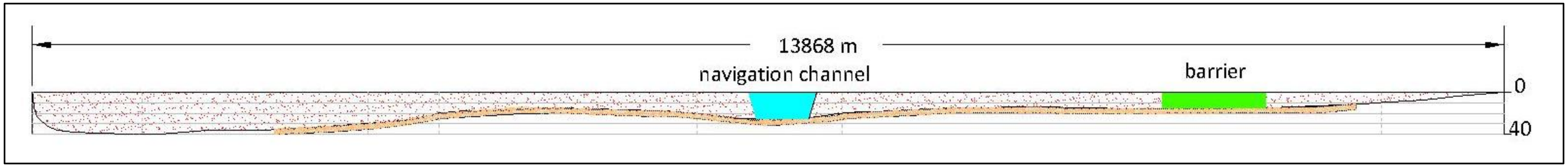

Figure A3 4: Location 2 layout

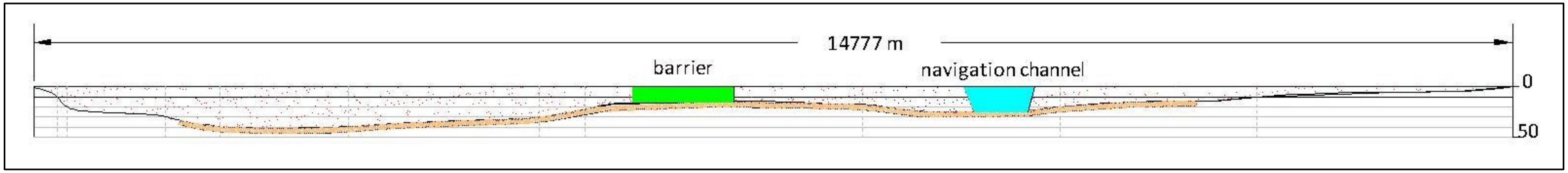

Figure A3 5: Location 3 layout

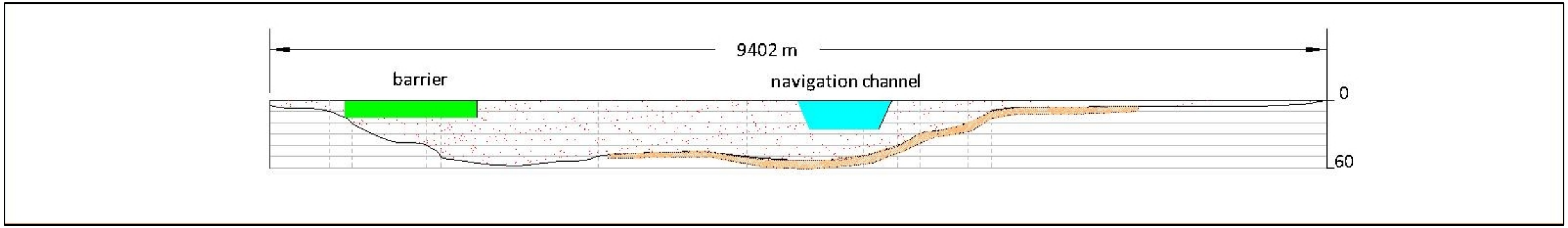

Figure A3 6: Location 4 layout

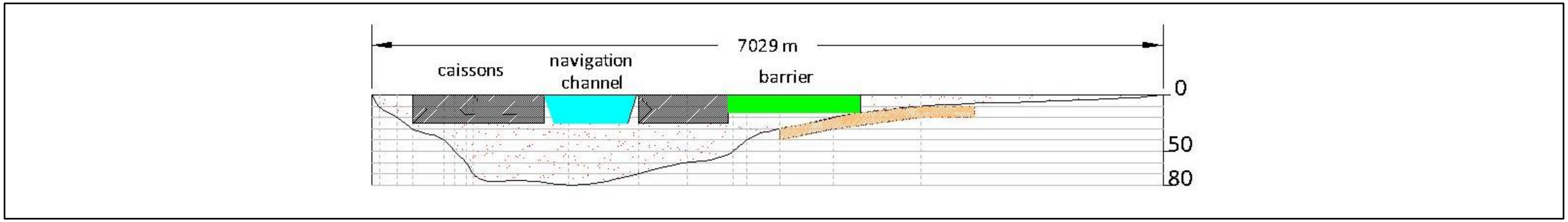

Figure A3 7: Location 5 layout

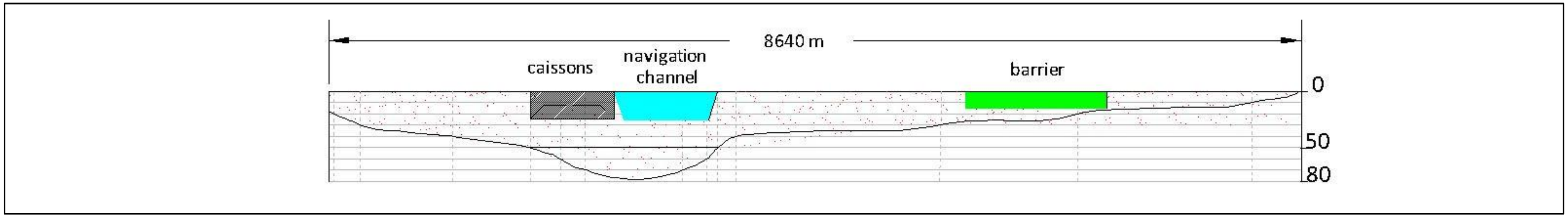

Figure A3 8: Location 6 layout

A3-2. INVESTMENT

For the previous described locations, the investment needed for building a barrier has been calculated. The cost has been divided in several parts, to give a more accurate result:

- Dam
- Movable barrier
- Caissons (in composite cross sections)
- Dredging
- Extra cost due to earth quake resistance measures

The **cost of dams** has been calculated using the following information and criteria:

- The cost of dams has been divided in rubble-mound dam (including armour layer up to - 20m depth, bituminous pavement and crown wall when needed), caissons and dredging.
- Two types of dam have been measured:
  - 15m width at the top, for normal dams
  - 40m width at the top, for dams under gates or caissons (these dams do not include crown wall or bituminous pavement)
- Both types of dams have a slope 1:2.5, recommended for seismic areas for some authors (Memos and Protonotarios, 1992)
- To simplify, it has been calculated the cost for rubble mounds in intervals of 5m depth. The average depth of the dams has been measured in the cross sections and the applied price is the closest step.
- Prices from international port projects are provided by French University ESITC Caen and several databases are used (CYPE Ingenieros S.A., 2014). The prices have been updated to 2014 and translated to Japanese values using the price of concrete as reference (The Asahi Shimbun, 2013). A 30% is added in the end, in account for indirect costs. This way, the total unit costs are found (see following table).

Table A3 1: List of prices for dams

		Unit direct prices 2014 (€)	**Unit direct prices Japan 2014 (€)**	**Unit total prices Japan 2014 (€)**
PRICES (euro)				
- Dam				
Dredging (mud or sand)	m3	6.00	4.75	**6.18**
Quarry run/stones	m3	32.00	25.36	**32.96**
Concrete in armour layer H25/35	m3	364.27	288.65	**375.24**
Concrete in crown wall H25-35 (4m height)	m3	200.00	158.48	**206.02**
Bituminous pavement	m2	16.50	13.07	**17.00**
REFERENCE: concrete (material)				ratio:
concrete H35/45		114.58	90.79	0.79

- Dam depths are given in current bathymetry (the values in the bathymetry are assumed to be referenced to Low Water Level) and calculated cross sections. To calculate the total depth, it is necessary to add:
  - 3m accounting for dredged base where necessary. The presence of mud (see Geological profile in Appendix 1) has been represented in the drawings and accounted for in measurements.
  - SLR (1.90m)
  - Freeboard over LWL (A.P.). According to Appendix 2, the freeboard in case of dam and gates is 2m. This value is added to design storm surge water levels. It will not be added for dam stretches under gates or caissons, since here the freeboard is already accounted for in the upper element)
- Caissons are used to avoid dam depths larger than 60 meters. They have 2m of freeboard, like dams.
- Armour layer estimated 2 meters. This is the thickness of an Xbloc armour layer with 5m design wave height (Geological Survey of Japan). The depth of the armour layer is taken as 20 meters, as a conservative assumption.

Regarding **cost of barriers**, the following remarks need to be done:

- Prices for barrier (meter length*meter height*water head) are provided by Professor S.N. Jonkman
- Gates height is given by cross section calculation. Depth is set at 15 meters (wet cross section). In order to obtain the total height, it is necessary to add:
  - Barrier head (3.2m for 100 yr, 3.7m for 500 yr)
  - Freeboard (2m)

Earthquake reinforcement is roughly estimated as a 10% extra cost. At this stage this is considered enough, since for buildings the figures are between 1 and 5% (Kawashima, 2013).

Table A3 2: Dam costs per meter length

- DAM COST 2014 (euro/m length)			DEPTH (m)											
			5		**10**		**15**		**20**		**25**		**30**	
			15	40	15	40	15	40	15	40	15	40	15	40
	Dimensions:													
	top width		15	40	15	40	15	40	15	40	15	40	15	40
	slope 1:		2.5		2.5		2.5		2.5		2.5		2.5	
	base width		40	65	65	90	90	115	115	140	140	165	165	190
	unit cost 2014(€)	**un**												
Quarry run - land equipment	**32.96**	**m3**	4532	8653	13185	21426	20767	30656	25712	35601	30656	40545	35601	45490
Quarry run - waterborne equipment	**32.96**	**m3**	0	0	0	0	8158	10631	25052	31645	46067	56780	71201	86035
Concrete in armour layer H25/35 (*)	**375.24**	**m3**	8335	8335	16671	16671	25006	25006	33342	33342	41677	41677	41677	41677
Concrete in crown wall H25-35 (4m height)	**206.02**	**m3**	3296		3296		3296		3296		3296		3296	
Bituminous pavement	**17.00**	**m2**	187		187		187		187		187		187	
	TOTAL	**€/m**	**16351**	**16988**	**33340**	**38097**	**57415**	**66293**	**87589**	**100588**	**121884**	**139003**	**151963**	**173202**

- DAM COST 2014 (euro/m length)			DEPTH (m)											
			35		**40**		**45**		**50**		**55**		**60**	
			15	40	15	40	15	40	15	40	15	40	15	40
	Dimensions:													
	top width		15	40	15	40	15	40	15	40	15	40	15	40
	slope 1:		2.5		2.5		2.5		2.5		2.5		2.5	
	base width		190	215	215	240	240	265	265	290	290	315	315	340
	unit cost 2014(€)	**un**												
Quarry run - land equipment	**32.96**	**m3**	40545	50434	45490	55379	50434	60323	55379	65268	60323	70212	65268	75157
Quarry run - waterborne equipment	**32.96**	**m3**	100457	119411	133832	156907	171328	198523	212945	244260	258682	294118	308539	348096
Concrete in armour layer H25/35 (*)	**375.24**	**m3**	41677	41677	41677	41677	41677	41677	41677	41677	41677	41677	41677	41677
Concrete in crown wall H25-35 (4m height)	**206.02**	**m3**	3296		3296		3296		3296		3296		3296	
Bituminous pavement	**17.00**	**m2**	187		187		187		187		187		187	
	TOTAL	**€/m**	**186163**	**211522**	**224483**	**253963**	**266923**	**300524**	**313485**	**351206**	**364166**	**406008**	**418968**	**464930**

Table A3 3: Barrier costs (I)

T=100 years		
	storm surge-tide	
barrier head (m):	**3.20**	m +A.P.
	1.90	m for SLR
	3.97	m + current T.P (DESIGN W.LEVEL)
	2.00	m freeboard
DAM:	**5.97**	m freeboard over current T.P.
BARRIER:	**4.07**	m freeboard over future T.P.

LOCATION 1		**LOCATION 2**		**LOCATION 3**	
12986	m length	13868	m	14777	m
292236	m^2	287403	m^2	315058	m^2
18750	env. Section	27000	env. Section	28500	env. Section
175	permanent op.	242	permanent op.	261	permanent op.
25	depth	25	depth	25	depth
588	nav. Barrier	521	nav. Barrier	502	nav. Barrier
24.1	height	24.1	height	24.1	height
20	depth	20	depth	20	depth
174	movable barrier	702	movable barrier	796	movable barrier
19.1	height	19.1	height	19.1	height
15	depth (MSL)	15	depth (MSL)	15	depth (MSL)

PRICES												
			Year	**Depth**	**Width**	**Total price 2014**		**MILLION EURO**		**MILLION EURO**		**MILLION EURO**
1	**Dam**	**ml**		**5.00**	**15.00**	**16351.32**		0		0		0
					40.00	**16988.44**		0		0		0
				10.00	**15.00**	**33339.76**		0		0		0
					40.00	**38097.33**		0		0		0
				15.00	**15.00**	**57415.37**	2182	125		0		0
					40.00	**66293.40**		0		0		0
				20.00	**15.00**	**87589.25**		0	6446	565	4774	418
					40.00	**100587.73**		0		0		0
				25.00	**15.00**	**121883.58**		0		0	1465	179
					40.00	**139002.51**		0	980	136		0
				30.00	**15.00**	**151962.87**		0	2912	443	3910	594
					40.00	**173202.25**		0		0		0
				35.00	**15.00**	**186162.61**	4854	904		0	2896	539
					40.00	**211522.44**		0		0		0
				40.00	**15.00**	**224482.80**	4703	1056	2751	618		0
					40.00	**253963.07**		0		0		0
				45.00	**15.00**	**266923.43**		0		0		0
					40.00	**300524.16**		0		0		0
				50.00	**15.00**	**313484.52**		0		0		0
					40.00	**351205.70**		0		0		0
				55.00	**15.00**	**364166.06**		0		0		0
					40.00	**406007.69**		0		0		0
				60.00	**15.00**	**418968.05**		0		0		0
					40.00	**464930.13**		0		0		0
			Year		**Price**	**Total price 2014**						
2	**Dredging**	**m^3**	**2010.00**		**6.18**	**7.00**	30384	0	30288	0	30465	0
3	**Caissons**	**m^3**	**2010.00**		**110.00**	**129.00**		0		0		0
4	**Movable barrier**	**m.m.m**	**2010.00**		**30180.00**	**35306.00**	55919	1974	82953	2929	87204	3079
5	**Earthquake resistance**	**%**				**0.10**	4059	406	4690	469	4809	481
					TOTAL			**4465**		**5159**		**5290**

	UNIT COST	**EURO**	**UNIT COST**	**EURO**	**UNIT COST**	**EURO**
DAM	euro/m	177584	euro/m	134532	euro/m	132619
CAISSONS	euro/m	22066250	euro/m	22066250	euro/m	22066250
BARRIER	euro/m	532463	euro/m	383469	euro/m	370593
TOTAL PER METER LENGTH	euro/m	343836	euro/m	371994	euro/m	357986
TOTAL SQUARE METER SECTION	euro/m2	15279	euro/m2	17950	euro/m2	16790

Table A3 4: Barrier costs (II).

T=100 years		
	storm surge-tide	
barrier head (m):	**3.20**	m +A.P.
	1.90	m for SLR
	3.97	m + current T.P (DESIGN W.LEVEL)
	2.00	m freeboard
DAM:	**5.97**	m freeboard over current T.P.
BARRIER:	**4.07**	m freeboard over future T.P.

LOCATION 4		**LOCATION 5**		**LOCATION 6**	
9402	m	7029	m	8640	m
274459	m^2	240560	m^2	313788	m^2
33000	env. Section	33750	env. Section	37500	env. Section
293	permanent op.	315	permanent op.	350	permanent op.
25	depth	25	depth	25	depth
470	nav. Barrier	310	nav. Barrier	413	nav. Barrier
24.1	height	24.1	height	24.1	height
20	depth	20	depth	20	depth
1085	movable barrier	1312	movable barrier	1366	movable barrier
19.1	height	19.1	height	19.1	height
15	depth (MSL)	15	depth (MSL)	15	depth (MSL)

PRICES							**LOCATION 4**		**LOCATION 5**		**LOCATION 6**	
			Year	**Depth**	**Width**	**Total price 2014**		**MILLION EURO**		**MILLION EURO**		**MILLION EURO**
1	**Dam**	**ml**		**5.00**	**15.00**	**16351.32**		0		0		0
					40.00	**16988.44**		0		0		0
				10.00	**15.00**	**33339.76**		0		0		0
					40.00	**38097.33**		0		0		0
				15.00	**15.00**	**57415.37**	2985	171	2690	154	1731	99
					40.00	**66293.40**		0		0		0
				20.00	**15.00**	**87589.25**	672	59		0		0
					40.00	**100587.73**		0		0	1250	126
				25.00	**15.00**	**121883.58**		0	382	47		0
					40.00	**139002.51**		0	1170	163		0
				30.00	**15.00**	**151962.87**		0		0		0
					40.00	**173202.25**	1170	203	534	92		0
				35.00	**15.00**	**186162.61**	887	165		0		0
					40.00	**211522.44**	618	131		0		0
				40.00	**15.00**	**224482.80**		0		0	4047	908
					40.00	**253963.07**		0		0		0
				45.00	**15.00**	**266923.43**		0		0		0
					40.00	**300524.16**		0	798	240		0
				50.00	**15.00**	**313484.52**		0		0	237	74
					40.00	**351205.70**		0	833	293	1345	472
				55.00	**15.00**	**364166.06**		0	644	235		0
					40.00	**406007.69**		0		0		0
				60.00	**15.00**	**418968.05**	3293	1380		0		0
					40.00	**464930.13**		0		0		0
			Year		**Price**	**Total price 2014**						
2	**Dredging**	**m^3**	**2010.00**		**6.18**	**7.00**	14370	0	5100	0		0
3	**Caissons**	**m^3**	**2010.00**		**110.00**	**129.00**			1080000	139	503550	65
4	**Movable barrier**	**m.m.m**	**2010.00**		**30180.00**	**35306.00**	102392	3615	103900	3668	115147	4065
5	**Earthquake resistance**	**%**				**0.10**	5724	572	5031	503	5811	581

		Location 4		Location 5		Location 6	
TOTAL			**6296**		**5534**		**6392**
		UNIT COST	**EURO**	**UNIT COST**	**EURO**	**UNIT COST**	**EURO**
DAM		euro/m	219055	euro/m	173454	euro/m	195153
CAISSONS		euro/m	22066250	euro/m	22066250	euro/m	22066250
BARRIER		euro/m	368075	euro/m	310216	euro/m	326622
TOTAL PER METER LENGTH		euro/m	669637	euro/m	787272	euro/m	739776
TOTAL SQUARE METER SECTION		euro/m2	22939	euro/m2	23004	euro/m2	20369

Table A3 5: Barrier costs (III)

T=500 years				
	storm surge-tide			
barrier head (m):	**3.70**	m +A.P.		
	1.90	m for SLR		
	4.47	**m + current T.P (DESIGN W.LEVEL)**		
	2.00	m freeboard		
DAM:	**6.47**	**m freeboard over current T.P.**		**(volume calc.)**
BARRIER:	**4.57**	**m freeboard over future T.P.**		**(surface calc.)**

LOCATION 1		**LOCATION 2**		**LOCATION 3**	
12986	m length	13868	m	14777	m
292236	m^2	287403	m^2	315058	m^2
18750	env. Section	27000	env. Section	28500	env. Section
75	permanent op.	117	permanent op.	111	permanent op.
25	depth	25	depth	25	depth
688	nav. Barrier	646	nav. Barrier	652	nav. Barrier
25.0	height	25.0	height	25.0	height
20	depth	20	depth	20	depth
208	movable barrier	744	movable barrier	846	movable barrier
19.6	height	19.6	height	19.6	height
15	depth (MSL)	15	depth (MSL)	15	depth (MSL)

PRICES												
			Year	**Depth**	**Width**	**Total price 2014**		**MILLION EURO**		**MILLION EURO**		**MILLION EURO**
1	**Dam**	**ml**		**5.00**	**15.00**	**16351.32**		0		0		0
					40.00	**16988.44**		0		0		0
				10.00	**15.00**	**33339.76**		0		0		0
					40.00	**38097.33**		0		0		0
				15.00	**15.00**	**57415.37**	2182.00	125		0		0
					40.00	**66293.40**		0		0		0
				20.00	**15.00**	**87589.25**		0	6446.00	565	4774.00	418
					40.00	**100587.73**		0		0		0
				25.00	**15.00**	**121883.58**		0		0	1465.00	179
					40.00	**139002.51**		0	980.00	136		0
				30.00	**15.00**	**151962.87**		0	2912.00	443	3910.00	594
					40.00	**173202.25**		0		0		0
				35.00	**15.00**	**186162.61**	4854.00	904		0	2896.00	539
					40.00	**211522.44**		0		0		0
				40.00	**15.00**	**224482.80**	4703.00	1,056	2751.00	618		0
					40.00	**253963.07**		0		0		0
				45.00	**15.00**	**266923.43**		0		0		0
					40.00	**300524.16**		0		0		0
				50.00	**15.00**	**313484.52**		0		0		0
					40.00	**351205.70**		0		0		0
				55.00	**15.00**	**364166.06**		0		0		0
					40.00	**406007.69**		0		0		0
				60.00	**15.00**	**418968.05**		0		0		0
					40.00	**464930.13**		0		0		0
	50 cm extra freeboard	**m^3**				**32.96**	1064892.00	35	987467.50	33	974750.00	32
			Year		**Price**	**Total price 2014**						
2	**Dredging**	**m^3**	**2010.00**		**6.18**	**7.00**	30384	0	30288	0	30465	0
3	**Caissons**	**m^3**	**2010.00**		**110.00**	**129.00**		0		0		0
4	**Movable barrier**	**m.m.m**	**2010.00**		**30180.00**	**35306.00**	68042	2402	98242	3469	105108	3711
5	**Earthquake resistance**	**%**				**0.10**	4522	452	5262	526	5473	547
						TOTAL		**4974**		**5788**		**6021**

	UNIT COST	**EURO**	**UNIT COST**	**EURO**	**UNIT COST**	**EURO**
DAM	euro/m	177584	euro/m	134532	euro/m	132619
CAISSONS	euro/m		euro/m		euro/m	
BARRIER	euro/m	2682136	euro/m	2495942	euro/m	2477821
TOTAL PER METER LENGTH	euro/m	383066	euro/m	417392	euro/m	407433
TOTAL SQUARE METER SECTION	euro/m2	17022	euro/m2	20140	euro/m2	19110

Table A3 6: Barrier costs (IV)

T=500 years				
	storm surge-tide			
barrier head (m):	**3.70**	m +A.P.		
	1.90	m for SLR		
	4.47	**m + current T.P (DESIGN W.LEVEL)**		
	2.00	m freeboard		
DAM:	**6.47**	**m freeboard over current T.P.**		**(volume calc.)**
BARRIER:	**4.57**	**m freeboard over future T.P.**		**(surface calc.)**

LOCATION 4		**LOCATION 5**		**LOCATION 6**	
9402	m	7029	m	8640	m
274459	m2	240560	m2	313788	m2
33000	env. Section	33750	env. Section	37500	env. Section
118	permanent op.	140	permanent op.	150	permanent op.
25	depth	25	depth	25	depth
645	nav. Barrier	623	nav. Barrier	613	nav. Barrier
24.6	height	24.6	height	24.6	height
20	depth	20	depth	20	depth
1143	movable barrier	1186	movable barrier	1433	movable barrier
19.6	height	19.6	height	19.6	height
15	depth (MSL)	15	depth (MSL)	15	depth (MSL)

		PRICES					**LOCATION 4**		**LOCATION 5**		**LOCATION 6**	
			Year	**Depth**	**Width**	**Total price 2014**		**MILLION EURO**		**MILLION EURO**		**MILLION EURO**
1	**Dam**	**ml**		**5.00**	**15.00**	**16351.32**		0		0		0
					40.00	**16988.44**		0		0		0
				10.00	**15.00**	**33339.76**		0		0		0
					40.00	**38097.33**		0		0		0
				15.00	**15.00**	**57415.37**	2985.00	171	2690.00	154	1731.00	99
					40.00	**66293.40**		0		0		0
				20.00	**15.00**	**87589.25**	672.00	59		0		0
					40.00	**100587.73**		0		0	1250.00	126
				25.00	**15.00**	**121883.58**		0		0		0
					40.00	**139002.51**		0	1170.00	163		0
				30.00	**15.00**	**151962.87**		0		0		0
					40.00	**173202.25**	1170.00	203		0		0
				35.00	**15.00**	**186162.61**	887.00	165	733.00	136		0
					40.00	**211522.44**	618.00	131		0		0
				40.00	**15.00**	**224482.80**		0		0	3809.00	855
					40.00	**253963.07**		0		0		0
				45.00	**15.00**	**266923.43**		0		0		0
					40.00	**300524.16**		0	798.00	240		0
				50.00	**15.00**	**313484.52**		0		0	237.00	74
					40.00	**351205.70**		0	833.00	293	1594.00	560
				55.00	**15.00**	**364166.06**		0	798.00	291		0
					40.00	**406007.69**		0		0		0
				60.00	**15.00**	**418968.05**	3293.00	1,380		0		0
					40.00	**464930.13**		0		0		0
	50 cm extra freeboard	**m^3**				**32.96**	953462.50	31	629440.00	21	837395.00	28
			Year		**Price**	**Total price 2014**						
2	**Dredging**	**m^3**	**2010.00**		**6.18**	**7.00**	14370	0	5100	0		0
3	**Caissons**	**m^3**	**2010.00**		**110.00**	**129.00**			1080000	139	503550	65
4	**Movable barrier**	**m.m.m**	**2010.00**		**30180.00**	**35306.00**	141397	4992	142487	5031	159435	5629
5	**Earthquake resistance**	**%**				**0.10**	7132	713	6467	647	7436	744
						TOTAL		**7845**		**7114**		**8179**

	UNIT COST	EURO	UNIT COST	EURO	UNIT COST	EURO
DAM	euro/m	219055	euro/m	181788	euro/m	198851
CAISSONS	euro/m		euro/m	80625	euro/m	80625
BARRIER	euro/m	2791526	euro/m	2780891	euro/m	2751674
TOTAL PER METER LENGTH	euro/m	834431	euro/m	1012089	euro/m	946696
TOTAL SQUARE METER SECTION	euro/m2	28585	euro/m2	29573	euro/m2	26067

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A3-3. DAMAGE AND RISK ESTIMATION

It is necessary to know the reduction in risk provided by each alternative, in order to use this information in their analysis and comparison.

First, the amount of damage corresponding to different water levels has to be found. Once this information is available, risk can be calculated from graphs damage versus occurrence probability. Then, residual risk for each alternative is obtained from these damage curves.

A3-3-1. DAMAGE CURVES

Hoshino's MSc Thesis results have been extended using flood maps (extrapolating damage values to different flooded surfaces). taking into account:

- Flooded surfaces in Saitama prefecture are ignored. There is a large distance to the shoreline and the presence of physical obstacles will make difficult for the water to arrive there.
- Values for Chiba prefecture are weighed, as the concentration of population and industrial facilities is lower than in Tokyo and Kanagawa prefectures, so the surfaces are not equivalent regarding economical damage due to flooding. The ratio used is 1/10.

Flood maps of Tokyo bay have been used (see following figure) to calculate inundated areas in each prefecture for different water levels. The calculated flooded surfaces in these flood maps have been compared to Hoshino's data. Then, an extrapolation has been done for Chiba prefecture, which was not accounted for previously. A remark that needs to be done is that the considered water levels include sea level rise (1.90m) in order to compare the barrier alternatives with the study done on upgrading of coastal defences. The damage starts when increase in water level is larger than coastal defences height in the area (the elevation at Tokyo is used in all cases, to simplify the calculation).

Detailed calculations and damage curves can be found later in this appendix. The total amount of damage is the sum of material damage and loss of life, which can be found at Tables A3-3-1-2 and A3-3-1-4.

As mentioned in 3.4.1, an exchange rate of *1€=141.2¥* is used here.

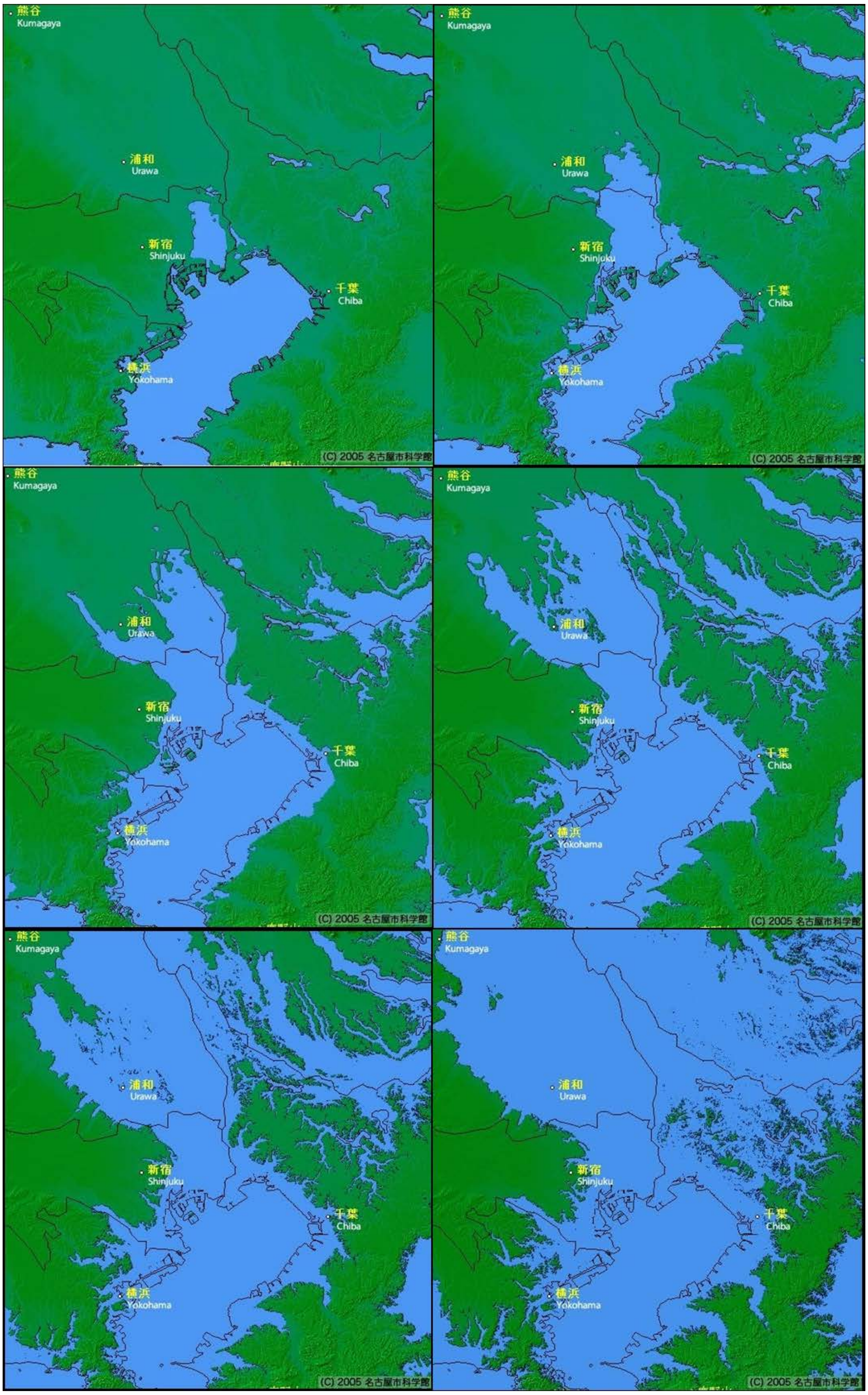


Figure A3 9: Flooded areas for 0, 1, 2.5, 10 and 20 m of water level increase (Nagoya City Science Museum, 2005)

In the previous flood maps, the inundated areas are measured (see table below).

Table A3 7: Inundated area (km^2) in all prefectures for different water levels

Return period (yr)	Water elevation (m+ T.P.)	PREFECTURE TOKYO		KANAGAWA		CHIBA	
	0.0						
	1.0	**0**	**km^2**	**0**	**km^2**	0	km^2
	2.0	**70**	**km^2**	**5**	**km^2**	27	km^2
	2.5	115	km^2	15	km^2	45	km^2
	3.0	**138**	**km^2**	**25**	**km^2**	91	km^2
	4.0	**160**	**km^2**	**35**	**km^2**	105	km^2
100	4.5	**205**	**km^2**	55	km^2	166	km^2
	5.0	220	km^2	56	km^2	197	km^2
1000	5.5	244	km^2	60	km^2	208	km^2

Data from Hoshino's MSc Thesis

An extrapolation is done from the given damage amounts (yellow) in order to complete the table for damage in each prefecture.

Table A3 8: Damage (trillion yen) in all prefectures for different water levels

Return period (yr)	Water elevation (m+ T.P.)	PREFECTURE TOKYO		KANAGAWA		CHIBA	
	0.0					(*)	
	1.0	**12**	**trillion ¥**	**0**	**trillion ¥**	0	trillion ¥
	2.0	**25**	**trillion ¥**	**1**	**trillion ¥**	0	trillion ¥
	2.5	34	trillion ¥	2	trillion ¥	1	trillion ¥
	3.0	**42**	**trillion ¥**	**3**	**trillion ¥**	1	trillion ¥
	4.0	**63**	**trillion ¥**	**4**	**trillion ¥**	1	trillion ¥
100	4.5	**75**	**trillion ¥**	4	trillion ¥	1	trillion ¥
	5.0	88	trillion ¥	4	trillion ¥	1	trillion ¥
1000	5.5	96	trillion ¥	5	trillion ¥	2	trillion ¥

Data from Hoshino's MSc Thesis

(*) proportional surface Kanagawa/relation population density

LOSS OF LIFE ESTIMATION

Loss of life is calculated as well, in order to add it to total damage evaluation.

As stated previously in section 3.4.1.3 a rate of 23 casualties/m^2 is used for calculations. For the different possibilities considered, the number of fatalities will be extrapolated from this initial value. The results can be found in the following tables.

Table A3 9: Casualties in all prefectures for different water levels

		PREFECTURE					
Return period (yr)	**Water elevation (m+ T.P.)**	**TOKYO**		**KANAGAWA**		**CHIBA**	
		density:	6000.0	density:	3639.0	density:	1175.0
	1.0	0	casualties	0	casualties	0	casualties
	2.0	1610	casualties	70	casualties	120	casualties
	2.5	2645	casualties	209	casualties	201	casualties
	3.0	3163	casualties	349	casualties	410	casualties
	4.0	3680	casualties	488	casualties	475	casualties
100	4.5	4715	casualties	767	casualties	749	casualties
	5.0	5060	casualties	776	casualties	886	casualties
1000	5.5	5617	casualties	832	casualties	936	casualties

Table A3 10: Loss of life value (trillion yen) in all prefectures for different water levels

		PREFECTURE					
Return period (yr)	**Water elevation (m+ T.P.)**	**TOKYO**		**KANAGAWA**		**CHIBA**	
	0.0						
	1.0	0	trillion ¥	0	trillion ¥	0	trillion ¥
	2.0	0	trillion ¥	0	trillion ¥	0	trillion ¥
	2.5	1	trillion ¥	0	trillion ¥	0	trillion ¥
	3.0	1	trillion ¥	0	trillion ¥	0	trillion ¥
	4.0	1	trillion ¥	0	trillion ¥	0	trillion ¥
100	4.5	1	trillion ¥	0	trillion ¥	0	trillion ¥
	5.0	1	trillion ¥	0	trillion ¥	0	trillion ¥
1000	5.5	1	trillion ¥	0	trillion ¥	0	trillion ¥

A3-3-2. RESIDUAL RISK CALCULATION

Once the damage is known (and related to water levels and return periods) for each prefecture, residual risk for every alternative is calculated. Residual risk is calculated as the area under the curve Damage versus Probability of occurrence. These curves are presented below, for the considered alternatives (do nothing, barrier).

Table A3 11: ALTERNATIVE: DO NOTHING. Damage (trillion yen) in unprotected area (EVERYTHING)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
		0				
		1	12	0	0	**13**
		2	25	1	0	**26**
		2.5	34	2	1	**36**
		3	42	3	1	**45**
		4	63	4	1	**68**
0.01	100	4.5	75	4	1	**80**
		5	88	4	1	**93**
0.001	1000	5.5	96	5	2	**102**

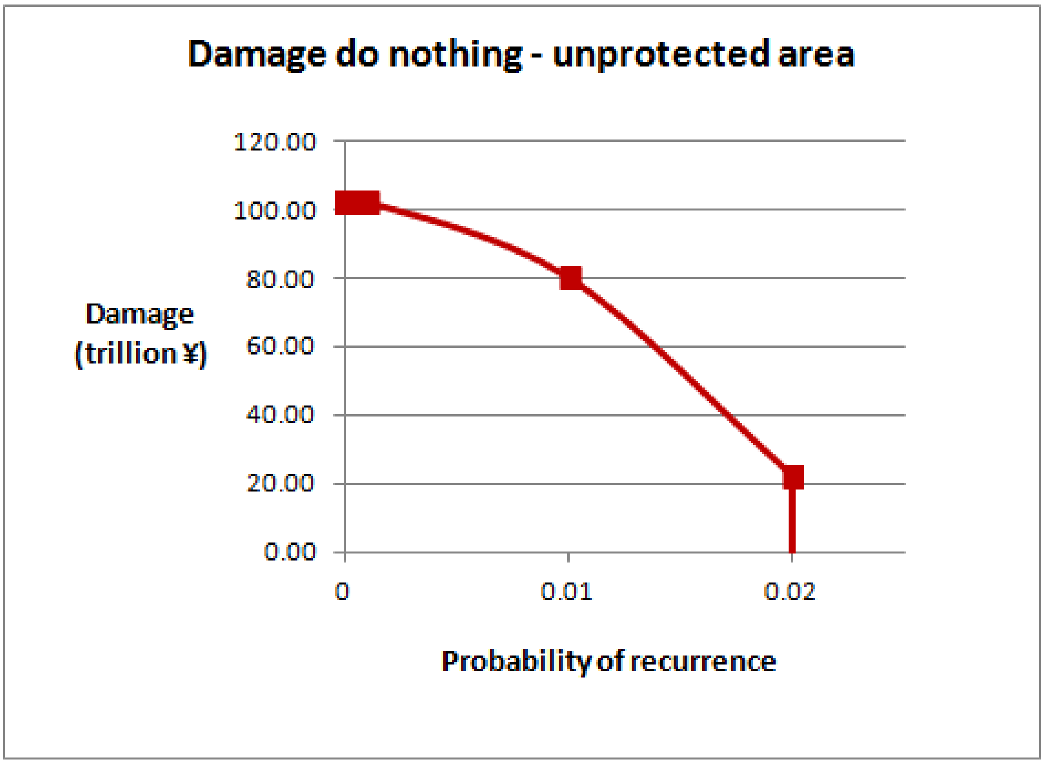


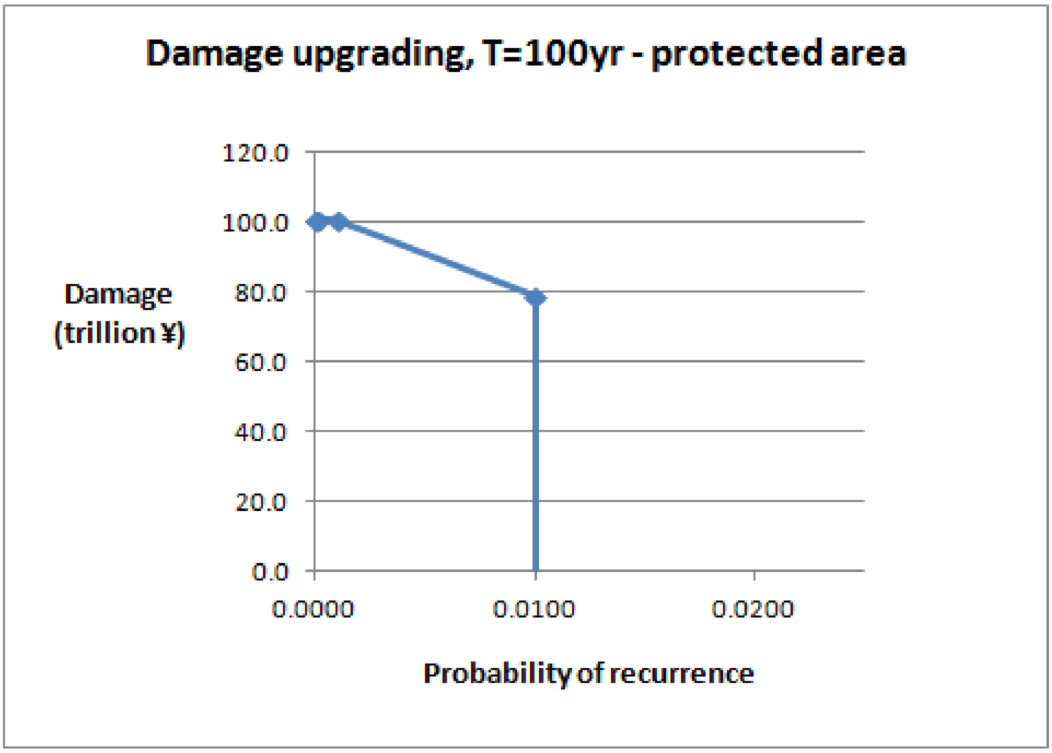
Figure A3 10: Damage curves protected/unprotected area. ALTERNATIVE: DO NOTHING

Table A3 12: ALTERNATIVE: UPGRADING. Damage (trillion yen) in protected area (0.5 CHIBA+TOKYO+ 0.8 KANAGAWA + SAITAMA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
		0				
		1	12	0	0	**12**
		2	25	1	0	**26**
		2.5	34	1	0	**35**
		3	42	2	0	**44**
		4	63	3	1	**67**
0.01	100	4.5	75	3	1	**79**
		5	88	3	1	**92**
0.001	1000	5.5	96	4	1	**101**

Table A3 13: ALTERNATIVE: UPGRADING. Damage (trillion yen) in unprotected area (0.5 CHIBA+0.2 KANAGAWA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
1		0				
		1	0	0	0	**0**
		2	0	0	0	**0**
		2.5	0	0	0	**1**
		3	0	1	0	**1**
		4	0	1	1	**1**
0.01	100	4.5	0	1	1	**1**
		5	0	1	1	**2**
0.001	1000	5.5	0	1	1	**2**



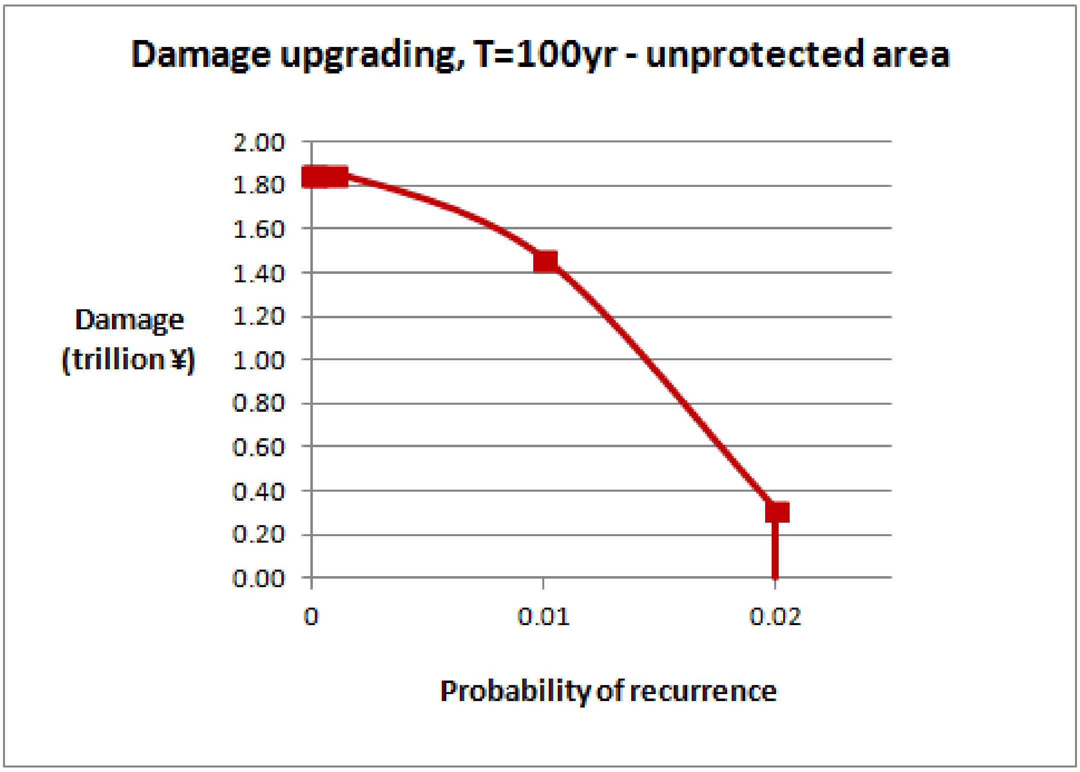


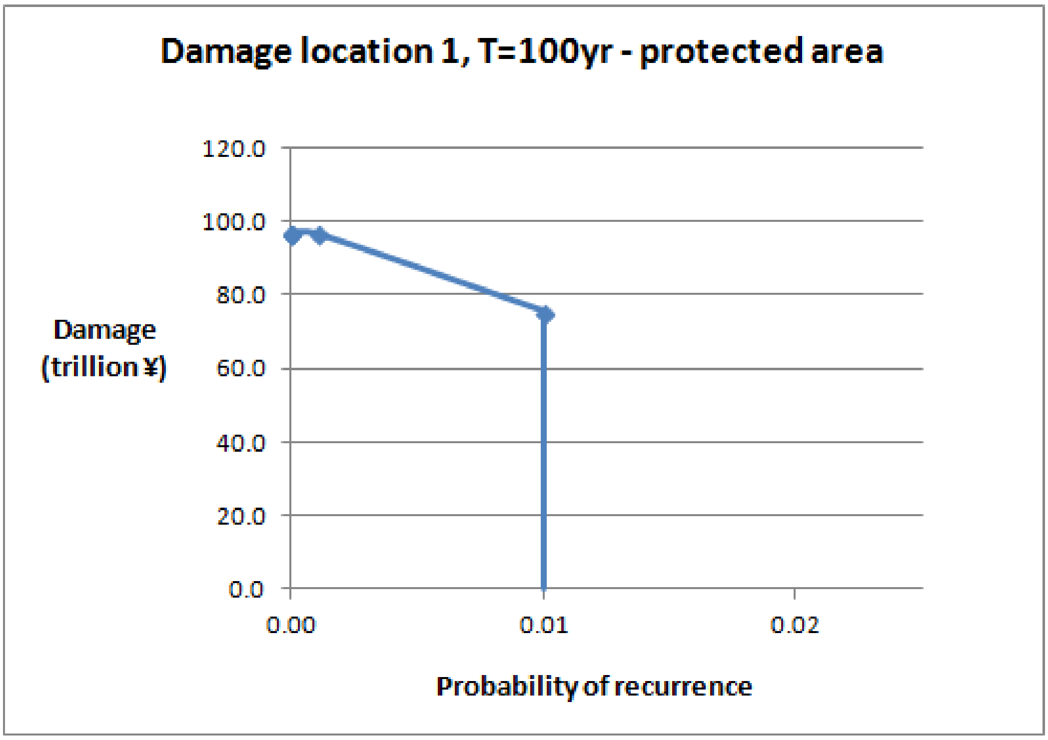
Figure A3 11: Damage curves protected/unprotected area. ALTERNATIVE: UPGRADING T=100YR

Table A3 14: ALTERNATIVE: LOCATION 1 T=100YR. Damage (trillion yen) in protected area (0.5 CHIBA+TOKYO+SAITAMA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
1		0				**0**
		1	12	0	0	**12**
		2	25	0	0	**25**
		2.5	34	0	0	**34**
		3	42	0	0	**42**
		4	63	0	1	**64**
0.01	100	4.5	75	0	1	**76**
		5	88	0	1	**89**
0.001	1000	5.5	96	0	1	**97**

Table A3 15: ALTERNATIVE: LOCATION 1 T=100YR. Damage (trillion yen) in unprotected area (0.5 CHIBA+KANAGAWA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
1		0				**0**
		1	0	0	0	**0**
		2	0	1	0	**1**
		2.5	0	2	0	**2**
		3	0	3	0	**3**
		4	0	4	1	**5**
0.01	100	4.5	0	4	1	**5**
		5	0	4	1	**5**
0.001	1000	5.5	0	5	1	**6**



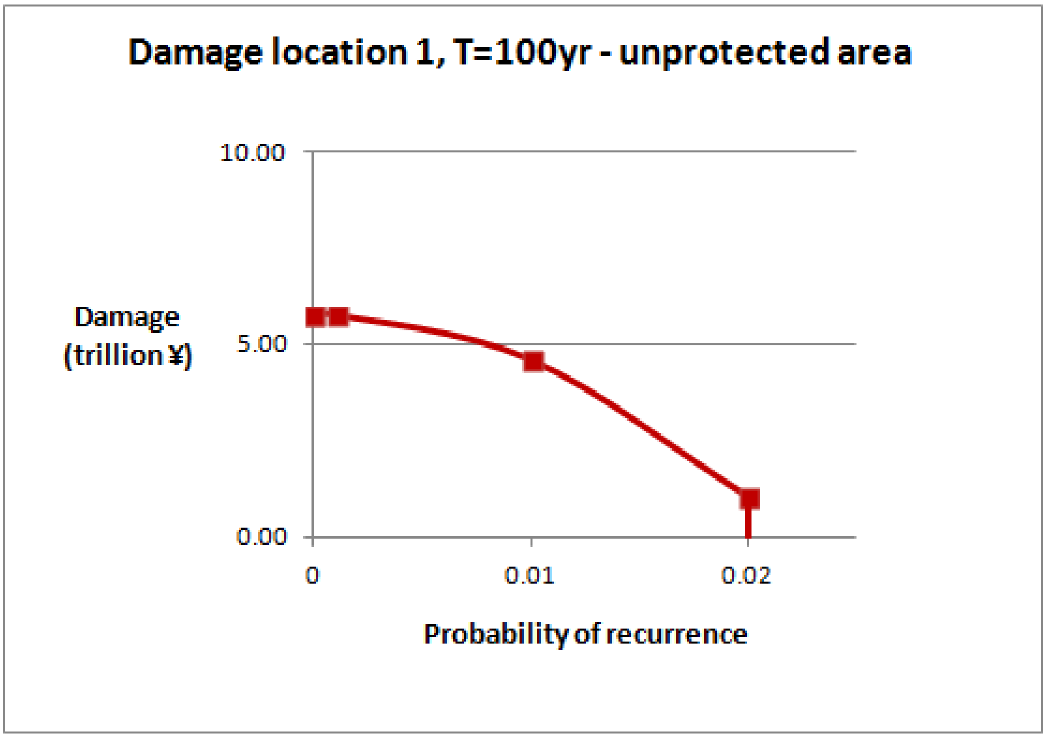


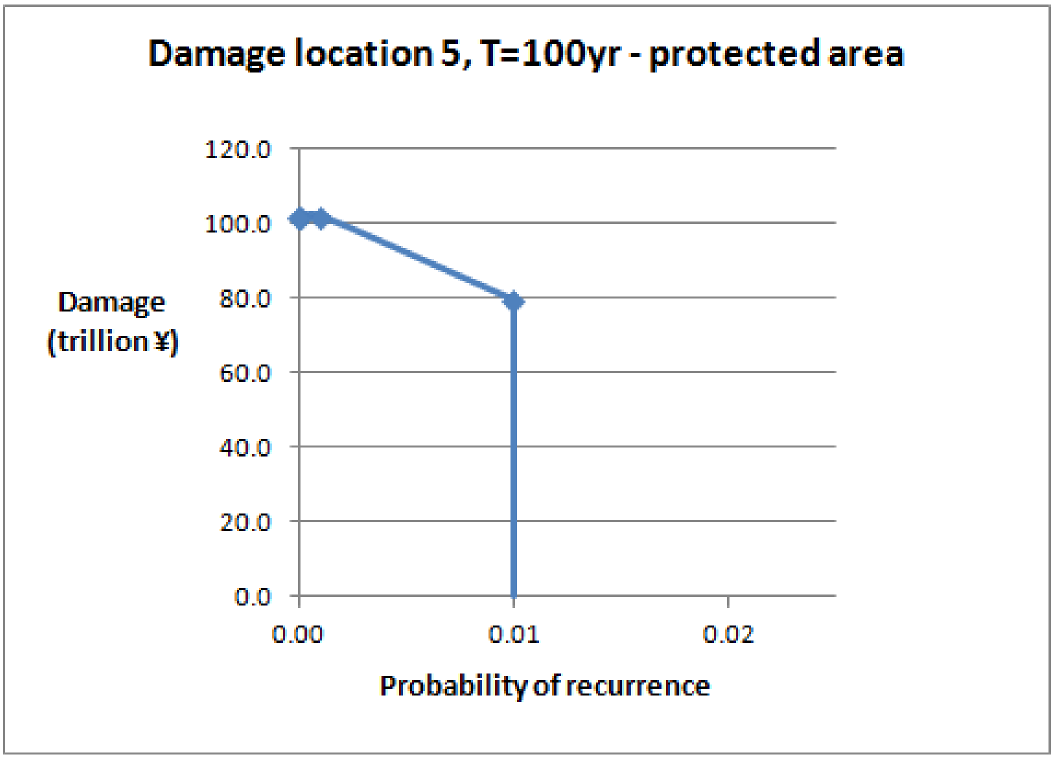
Figure A3 12: Damage curves protected/unprotected area. ALTERNATIVE: LOCATION 1 T=100YR

Table A3 16: ALTERNATIVE: LOCATION 5 T=100YR. Damage (trillion yen) in protected area (0.9 CHIBA+TOKYO+SAITAMA+0.9 KANAGAWA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
1		0				
		1	12	0	0	**13**
		2	25	1	0	**26**
		2.5	34	2	1	**36**
		3	42	2	1	**45**
		4	63	4	1	**68**
0.01	100	4.5	75	3	1	**80**
		5	88	4	1	**93**
0.001	1000	5.5	96	4	2	**102**

Table A3 17: ALTERNATIVE: LOCATION 5 T=100YR. Damage (trillion yen) in unprotected area (0.1 CHIBA+0.1 KANAGAWA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
1		0				
		1	0	0	0	**0**
		2	0	0	0	**0**
		2.5	0	0	0	**0**
		3	0	0	0	**0**
		4	0	0	0	**1**
0.01	100	4.5	0	0	0	**1**
		5	0	0	0	**1**
0.001	1000	5.5	0	0	0	**1**



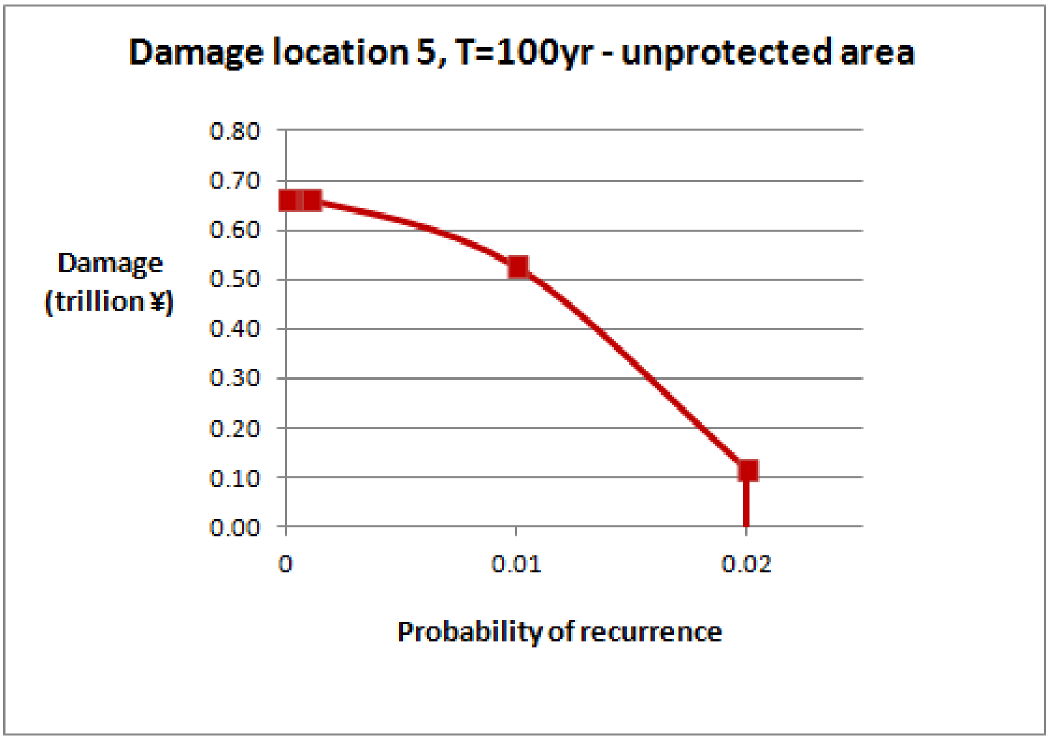


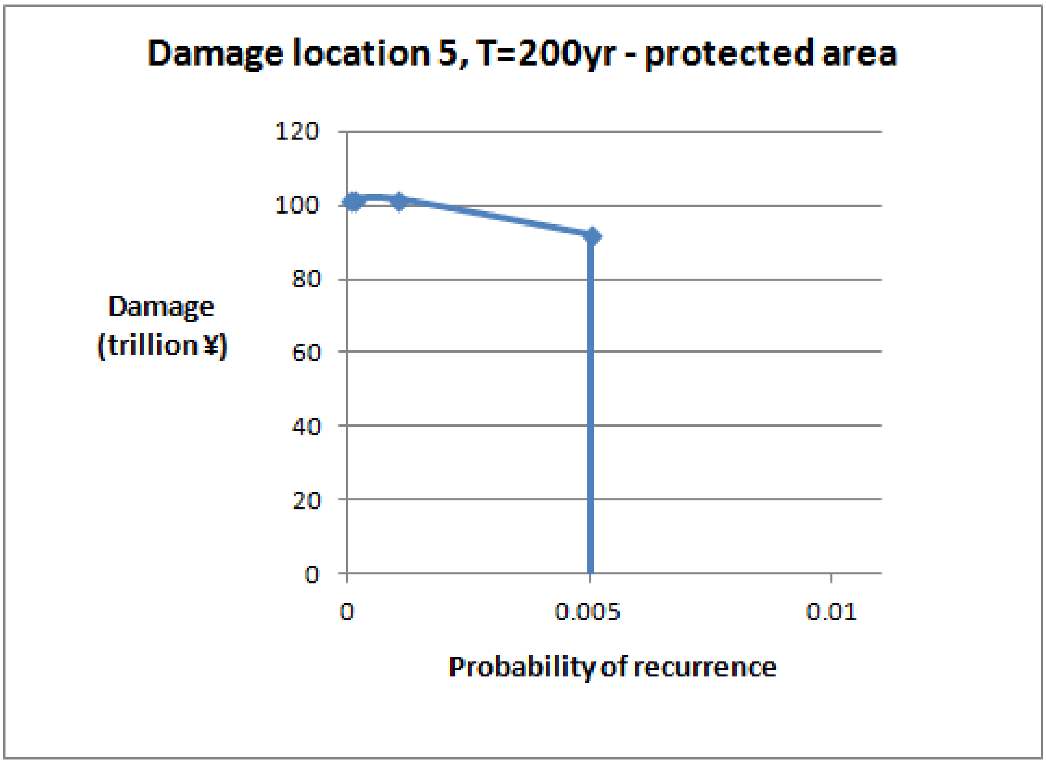
Figure A3 13: Damage curves protected/unprotected area. ALTERNATIVE: LOCATION 5 T=100YR

Table A3 18: ALTERNATIVE: LOCATION 5 T=200YR. Damage (trillion yen) in protected area (0.9 CHIBA+TOKYO+SAITAMA+0.9 KANAGAWA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
		0				
		1	12	0	0	**13**
		2	25	1	0	**26**
		2.5	34	2	1	**36**
		3	42	2	1	**45**
		4	63	4	1	**68**
0.01	**100**	**4.5**	**75**	**3**	**1**	**80**
		5	**88**	**4**	**1**	**93**
0.001	1000	5.5	96	4	2	**102**

Table A3 19: ALTERNATIVE: LOCATION 5 T=200YR. Damage (trillion yen) in unprotected area (0.1 CHIBA+0.1 KANAGAWA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
		0				
		1	0	0	0	**0**
		2	0	0	0	**0**
		2.5	0	0	0	**0**
		3	0	0	0	**0**
		4	0	0	0	**1**
0.01	100	4.5	0	0	0	**1**
		5	0	0	0	**1**
0.001	1000	5.5	0	0	0	**1**



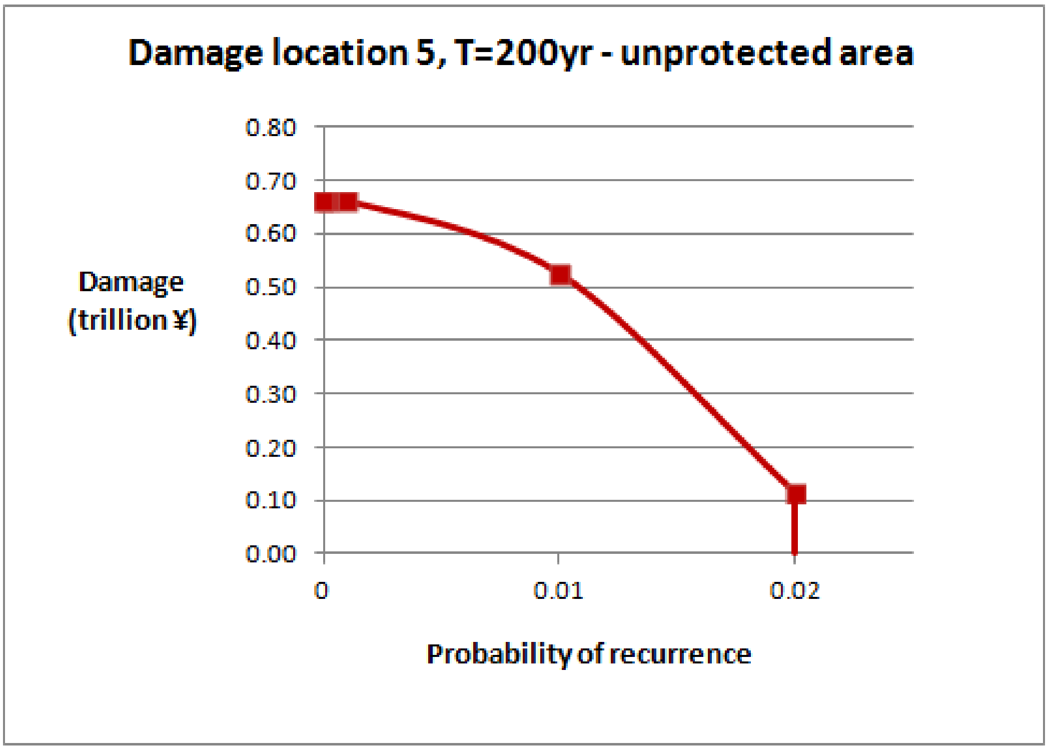


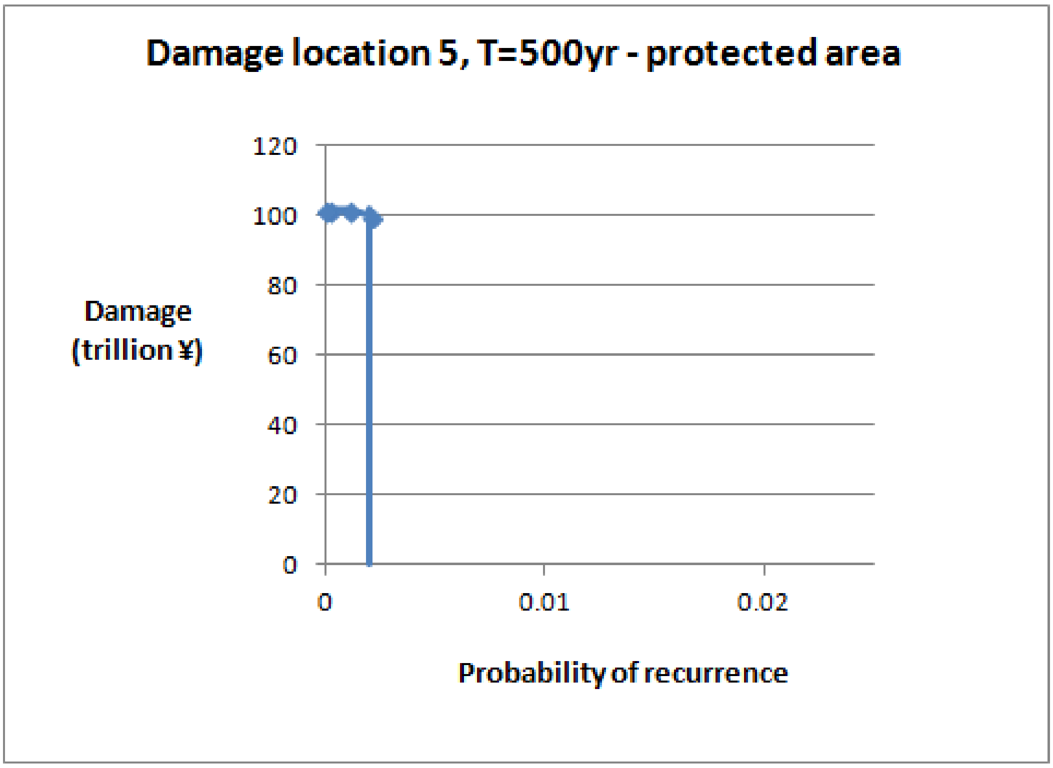
Figure A3 14: Damage curves protected/unprotected area. ALTERNATIVE: LOCATION 5 T=200YR

Table A3 20: ALTERNATIVE: LOCATION 5 T=500YR. Damage (trillion yen) in protected area (0.9 CHIBA+TOKYO+SAITAMA+0.9 KANAGAWA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
		0				
		1	12	0	0	**13**
		2	25	1	0	**26**
		2.5	34	2	1	**36**
		3	42	2	1	**45**
		4	63	4	1	**68**
0.01	100	4.5	75	3	1	**80**
		5	88	4	1	**93**
0.001	1000	5.5	96	4	2	**102**

Table A3 21: ALTERNATIVE: LOCATION 5 T=500YR. Damage (trillion yen) in unprotected area (0.1 CHIBA+0.1 KANAGAWA)

Probability	Return period (yr)	Water elevation (m+ T.P.)	TOKYO	KANAGAWA	CHIBA	TOTAL
		0				
		1	0	0	0	**0**
		2	0	0	0	**0**
		2.5	0	0	0	**0**
		3	0	0	0	**0**
		4	0	0	0	**1**
0.01	100	4.5	0	0	0	**1**
		5	0	0	0	**1**
0.001	1000	5.5	0	0	0	**1**



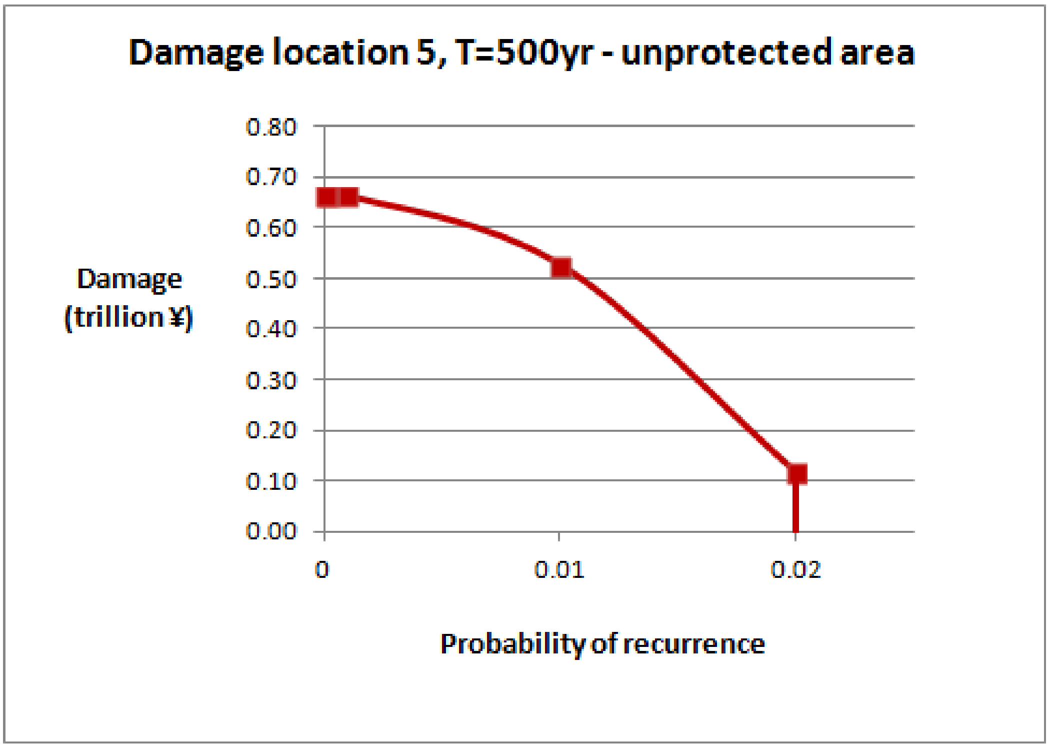


Figure A3 15: Damage curves protected/unprotected area. ALTERNATIVE: LOCATION 5 T=500YR

TOTAL RESIDUAL RISK

The area under the damage curves represents the residual risk. It has been calculated for each alternative the corresponding risk associated to material damage and loss of life (see second and third columns in Table A3-3-2-12 below).

After calculating residual risk, the values are reduced to Net Present Value (CUR-Publicatie-190, 1997):

$$PV(t) = D_x \cdot (1 + g_r)^t \cdot P_x \cdot (\frac{1}{1 + d_r})^t$$

where:

PV(t): net present value

D_x: damage

P_x: probability

g_r: interest rate. For this factor, experts foresee a long-term value of 1% (Nikkei Asian Review, 2014, Yamada et al., 2013).

d_r: discount rate. A value of 2% is expected, average of 1% return of public bonds and 3% return of private investment (Leventhal and Dolley, 1994).

After that, indirect losses are added as well, obtaining the final amount of risk related to each alternative. There is uncertainty about how to evaluate indirect losses. Their relation with direct losses changes as the amount of damage increases. A factor of 2 will be used here (Hallegatte, 2008).

A summary with results can be found in the following table. Only locations 1 and 5 have been calculated. In terms of residual risk, 6 has the same values as 5. For locations 2,3 and 4, an interpolation between 1 and 5 will be done, proportional to the length of coastline protected.

Table A3 22: Residual risk associated to every alternative

ALTERNATIVE	P*D Material damage (M €)	P*D Loss of life (M €)	P*D Total (M €)	t (yr)	g_r (%)	d_r (%)	PV(t) (M €)	Indirect losses (factor: *2) (M €)	TOTAL RESIDUAL RISK (M €)
Do Nothing	10181	195	10376	100	1	2	3801	7602	**11403**
Upgrading 100yr	6613	132	6745	100	1	2	2469	4938	**7407**
Location1 100yr	6748	138	6886	100	1	2	2519	5039	**7558**
Location5 100yr	6573	128	6701	100	1	2	2454	4908	**7362**
Location5 200yr	3532	69	3601	100	1	2	1319	2638	**3956**
Location5 500yr	1501	31	1532	100	1	2	560	1121	**1681**

Appendix 4

MULTI-CRITERIA ANALYSIS

APPENDIX 4 – MULTI-CRITERIA ANALYSIS

TABLE OF CONTENTS

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A4-1. INTRODUCTION

A multi-criteria analysis is performed in order to find the most favorable solution. The Linear Additive method is used, which is considered to be accurate enough for a preliminary design stage.

The followed procedure consists on the following steps:

- Identify the relevant criteria an assignment of relative weight
- Define a performance scale for giving scores and assign scores to the alternatives, regarding the established criteria
- Add partial scores in order to obtain the final result for each alternative

A4-2. WEIGHING

In principle, PIANC guidelines (InCom, 2006) are used to define criteria and their relative weights. It is considered to be accurate enough at this stage of the process. However, in case further study is done, a review of the relative weights could be necessary.

Table A4 1: Scores Multi-criteria analysis

CATEGORIES	CRITERIA	Weight
TECHNICAL	Solution reliability	0.29
	Operations	0.17
	Navigation	0.17
	Maintenance	0.07
ENVIRONMENT	Currents/sediments	0.02
	Pollution	0.01
	Landscape	0.05
	Land occupation	0.04
SOCIAL	Population feeling protected	0.10
	Other social	0.08

A4-3. SCORING

A4-3.1. METODOLOGY

A simple way to assign scores is defined. The scores are given according to a numerical scale that covers all possible performance levels that an alternative can reach, regarding one relevant criterion. Scores justification can be found in the table below.

Table A4 2: Scores Multi-criteria analysis

SCORES	ALTERNATIVE PERFORMANCE
4	ideal conditions
3	good conditions, without significant problems
2	few problems that can be solved
1	problems difficult to resolve
0	problems that could proscribe location

A4-3-2. ASSIGNMENT OF SCORES

Solution reliability

In terms of reliability, it is considered that a general failure is more difficult to happen in case of upgrade of coastal defences, since there are plenty of structures and kilometers of dike involved. In case of a barrier a general failure is still not very probable either (most likely, a breach will occur that will lead to lower water levels than in case there is no barrier).

DO NOTHING:0; UPGRADE:2; LOCATION 1:; LOCATION 2:;

LOCATION 3:2; LOCATION 4:2; LOCATION 5:2; LOCATION 6:2

Operations

Regarding operations, a solution that interferes the least with navigation and other operations along the coastline is preferred. Operations inland are not affected, except for the case of doing nothing, where the flooding of large areas would be a frequent event which would cause operational issues.

Concerning these aspects, building a barrier allows a more "permeable" coastline (in case of ports, navigational channels, etc), since it is not necessary to heighten existing walls and levees (and for a large length). The downside is that crossing the barrier complicates navigation. In the end, it is considered that upgrading coastal defences will interfere in a larger grade.

DO NOTHING:1; UPGRADE:2; LOCATION 1:2; LOCATION 2:2;

LOCATION 3:2; LOCATION 4:2; LOCATION 5:2; LOCATION 6:2

Navigation

Doing nothing would have no effects in navigation. Upgrading coastal defences will not have an effect either (it will interfere with loading/unloading operations, but this was taken into account in previous paragraph). Building a barrier though, would affect vessels traffic, since crossing the navigation channels would require extra safety measures. All locations would have a similar effect since the navigation channel will be the same dimensions.

DO NOTHING:4; UPGRADE:4; LOCATION 1:2; LOCATION 2:2;

LOCATION 3:2; LOCATION 4:2; LOCATION 5:2; LOCATION 6:2

Maintenance

Doing nothing implies no maintenance works are needed. Upgrading coastal defences will require the maintenance of dikes and movable barriers along the coastline (large lengths to control, long displacements, etc). The barrier as an option requires frequent maintenance works as well, since it has to be ensured that the gates are operational when needed, but these works are concentrated at one point, so they require less resources.

DO NOTHING:4; UPGRADE:2; LOCATION 1:3; LOCATION 2:3;

LOCATION 3:3; LOCATION 4:3; LOCATION 5:3; LOCATION 6:3

Currents/sediments

In case of doing nothing, the situation regarding currents or sediment transport will not change. Upgrading coastal defences will have a small effect compared to barriers; still, it might imply to place some new barriers in rivers, so there is some interference. Barriers will affect currents and sediment transport patterns, but to precisely quantify this effect hydraulic modeling would be needed. In this case, the barriers that close a smaller fraction of the whole bay get a higher score since they affect a smaller area.

DO NOTHING:4; UPGRADE:3; LOCATION 1:3; LOCATION 2:2;

LOCATION 3:2; LOCATION 4:2; LOCATION 5:2; LOCATION 6:2

Pollution

Here, pollution related not only to air, water or soil, but noise and light is included. Dams and barriers will cause different type of pollution (getting at least a score of 2, for problems that can be solved):

- In case of dams, which are made from granular material extracted from quarries, the activities involved are the extraction process, transport and underwater placement of the core material. All these operations will produce emissions and water turbidity.
- In case of movable barriers, possibly the main issue is the pollution created in the production process of the various elements of the structure (cement and steel).

Further study would be needed to determine which alternative is more pollutant, since it is not possible to determine it at first glance. Cross section dimensions will be used to give scores, since dams and movable barriers are considered to have a similar weight (concrete and steel are more pollutant but granular material in dams has a larger volume and adds water turbidity to other types of pollution). The scores will be assigned giving an extra point to the smallest cross sections which need less material (5). Upgrade of coastal defences obtains same score as the barriers (material volumes and expected pollution are lower, but they will be produced closer to population settlements and therefore more significant). Do nothing is considered not to cause extra pollution.

DO NOTHING:4; UPGRADE:2; LOCATION 1:2; LOCATION 2:2;

LOCATION 3:2; LOCATION 4:2; LOCATION 5:3; LOCATION 6:2

Landscape

Regarding landscape, the best alternatives are considered the ones that imply no intervention, intervention that allows aesthetic measures to be taken or is out of sight. It is considered that even if the barrier is aesthetically attractive, the citizens will not like to see their horizon cut this way.

Do nothing obtains a 4, for ideal conditions (no intervention). Upgrade of coastal defences obtains a 1 for significant problems (raising dikes or levees will limit the views along the whole coastline). Barriers get a 3 for good conditions, except locations 3, 4 and 5 that start in Cape Futtsu, which is a natural protected area and due to this, presumably of a larger value in terms of visual quality. Location 1 is very close to Tokyo and runs parallel to Aqualine highway, getting the worse possible score.

DO NOTHING:4; UPGRADE:2; LOCATION 1:0; LOCATION 2:3;

LOCATION 3:2; LOCATION 4:2; LOCATION 5:2; LOCATION 6:3

Land occupation

In principle, barriers will use more space than upgrading defences, most of it, underwater. Barriers will affect to the natural environment (displacing wildlife at some points), while upgrading will interfere with population settlements and activities in a higher degree. Doing nothing obtains the best score here, as it does not require occupation of any land.

Also, certain activities will need relocation. Some of the barriers affect industrial facilities, mainly ports and refineries. These are the alternatives located in the inner part of the bay (1, 2 and 3). There is also affection to seaweed farms that are located close to cape Futtsu (locations 3, 4 and 5).

DO NOTHING:4; UPGRADE:2; LOCATION 1:2; LOCATION 2:2;

LOCATION 3:2; LOCATION 4:2; LOCATION 5:2; LOCATION 6:2

Population feeling protected

Do nothing obtains the worst score and upgrade gets a 3 for good conditions. The different barriers will get a growing score depending on the percentage of protected population.

DO NOTHING:0; UPGRADE:3; LOCATION 1:1; LOCATION 2:2;

LOCATION 3:2; LOCATION 4:3; LOCATION 5:4; LOCATION 6:4

Other social

Economic development during and after construction, recreational activities, tourism or Japanese pride are included in this concept.

Do nothing obtains the worst score being highly harmful for all the factors mentioned above. For example, it would mean economic losses, not only due to floods but the unsafe environment would be bad for business development (many activities would leave looking for safer places). Upgrading of coastal defences will pose some problems at it decreases the permeability of the coastline. Barriers pose also some problems, since they interfere with recreational activities and would require relocation of agricultural/industrial facilities in some cases (location 2).

DO NOTHING:0; UPGRADE:2; LOCATION 1:2; LOCATION 2:2;

LOCATION 3:2; LOCATION 4:2; LOCATION 5:2; LOCATION 6:2

A4-4. PERFORMANCE MATRIX

After assigning weights and scores to every alternative, all of them get a final mark. Then, the cost is added to the analysis by means of a score/cost ratio. This ratio is the final parameter that indicates which is the most advantageous solution.

As both upgrading and barriers have maintenance and operational costs and they are difficult to evaluate at this point, they will be left out of the analysis (they are assumed to be in a similar range).

Table A4 3: Performance Matrix

			DO NOTHING		UPGRADE		LOCATION 1		LOCATION 2	
CATEGORIES	**CRITERIA**	**Weight**	**Score**	**Weighed**	**Score**	**Weighed**	**Score**	**Weighed**	**Score**	**Weighed**
TECHNICAL	**Solution reliability**	0.29	0	0.00	2	0.58	1	0.29	2	0.58
0.7	**Operations**	0.17	1	0.17	2	0.34	2	0.34	2	0.34
	Navigation	0.17	4	0.68	4	0.68	2	0.34	2	0.34
	Maintenance	0.07	4	0.28	2	0.14	3	0.21	3	0.21
ENVIRONMENT	**Currents/sediments**	0.02	4	0.08	3	0.06	3	0.06	2	0.04
0.12	**Pollution**	0.01	4	0.04	2	0.02	2	0.02	2	0.02
	Landscape	0.05	4	0.20	1	0.05	0	0.00	3	0.15
	Land occupation	0.04	4	0.16	2	0.08	2	0.08	2	0.08
SOCIAL	**Population feeling protected**	0.10	0	0.00	3	0.30	1	0.10	2	0.20
0.18	**Other social**	0.08	0	0.00	2	0.16	2	0.16	2	0.16
		TOTAL:		**1.61**		**2.41**		**1.60**		**2.12**

			LOCATION 3		LOCATION 4		LOCATION 5		LOCATION 6	
CATEGORIES	**CRITERIA**	**Weight**	**Score**	**Weighed**	**Score**	**Weighed**	**Score**	**Weighed**	**Score**	**Weighed**
TECHNICAL	**Solution reliability**	0.29	2	0.58	2	0.58	2	0.58	2	0.58
0.7	**Operations**	0.17	2	0.34	2	0.34	2	0.34	1	0.17
	Navigation	0.17	2	0.34	2	0.34	2	0.34	2	0.34
	Maintenance	0.07	3	0.21	3	0.21	3	0.21	3	0.21
ENVIRONMENT	**Currents/sediments**	0.02	2	0.04	2	0.04	2	0.04	2	0.04
0.12	**Pollution**	0.01	2	0.02	2	0.02	3	0.03	2	0.02
	Landscape	0.05	2	0.10	2	0.10	2	0.10	3	0.15
	Land occupation	0.04	2	0.08	2	0.08	2	0.08	2	0.08
SOCIAL	**Population feeling protected**	0.10	2	0.20	3	0.30	4	0.40	4	0.40
0.18	**Other social**	0.08	2	0.16	2	0.16	2	0.16	2	0.16
		TOTAL:		**2.07**		**2.17**		**2.28**		**2.15**

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Appendix 5

CROSS SECTION TYPOLOGY

APPENDIX 5 – CROSS SECTION TYPOLOGY DEFINITION

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A5-1. OPTIMISATION REGARDING COSTS

It has been studied whether a rubble mound dam or a composite dam cross-section is less costly.

Using the software Maple™, the costs have been calculated as a function of depths and later compared.

Two variables are considered here. The first one is the cross section depth d. The second variable is the caisson depth d1 (see Figure A5-1-1).

The studied cross sections have been divided in several elements with different unit prices. The total cost is found multiplying unit prices and corresponding volumes for each cross section. The prices used here are the same as the ones used in Appendix 3. For a graphical description of elements location and variables in the study, see Figure A5-5-1. The prices are summarized in the following table.

Table A5 1: Unit prices dam construction.

	units	Item	unit cost (€)
	un	Concrete in crown wall	3296.00
	un	Bituminous pavement	187.00
	m^3	Caisson	129.00
	m^3	Concrete in armour layer	375.24
	m^3	Quarry run	32.96

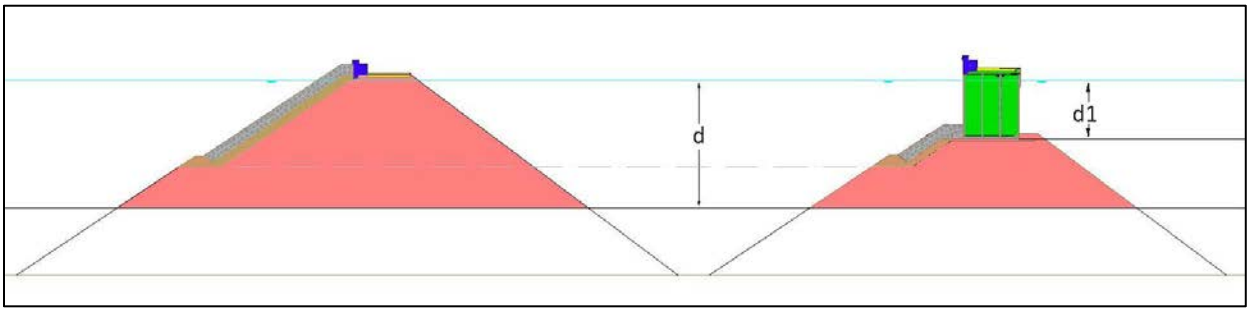


Figure A5 1: Unit prices dam construction.

The relationship between costs and depths d and d1 has been calculated, taking into account the following conditions:

- Crown wall is considered to be the same for the two sections, a concrete mass of 16 m^2.
- Bituminous pavement is the same in the two options, 8 m width.
- Armour layer has a porosity of 45%, a thickness of 2 meters and it extends up to -20 m (A.P.) maximum.
- Minimum caisson width is 12 m (4 m crown wall and 8 m pavement).
- The relationship caisson width/depth is 1.
- The comparison is done for a range of depths between 10 and 65 meters, as for less than 10 meters it is considered that the assumed unit costs and dimensions are no longer valid.
- All the calculations are meant for 1 meter in length.

The costs for the dam were previously studied in Appendix 3.. The function cost versus depth still has to be calculated for the composite cross section. Both are presented in the following paragraphs.

<u>RUBBLE MOUND DAM COSTS</u>

From A3-2, in Appendix 3, a formula where costs depend on depth (d) is obtained (see following figure). This is made by fitting a polynomial to the points already calculated.

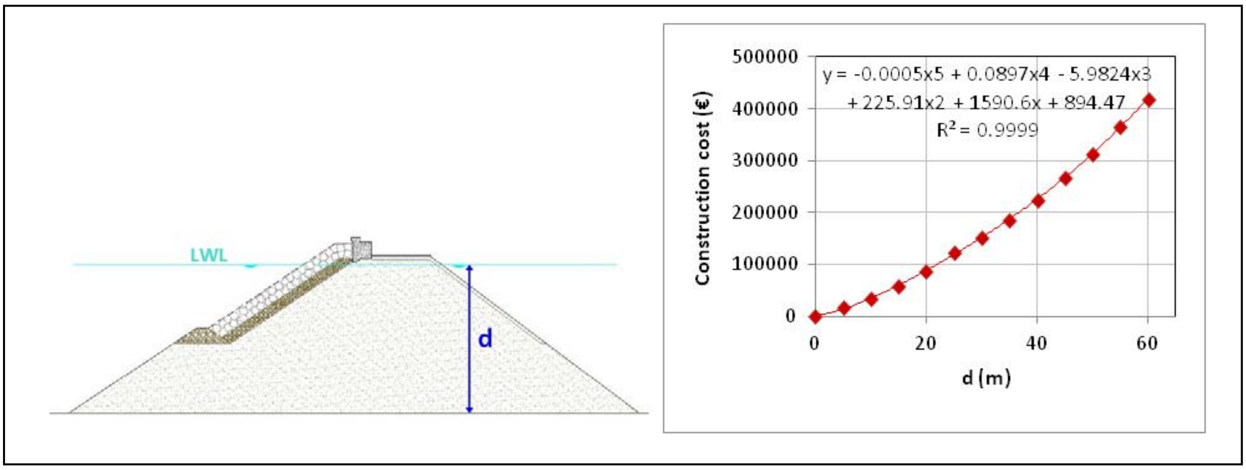


Figure A5 2: Rubble mound dam section

<u>COMPOSITE DAM COSTS</u>

Within the composite cross-section type, for each cross section depth, there is a range of possible costs, depending on the chosen caisson depth d1 (see following figure).

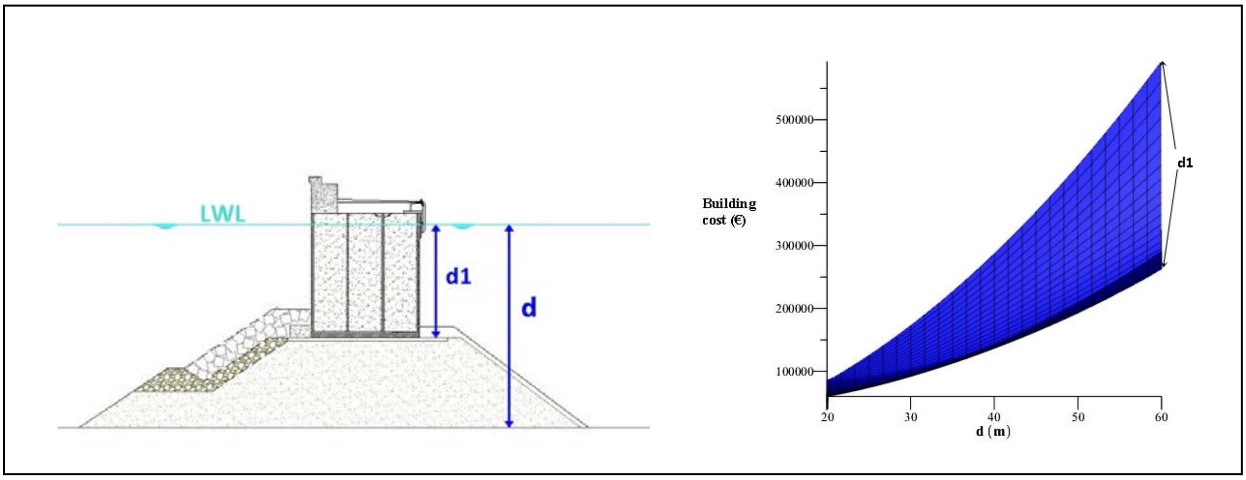


Figure A5 3: Composite dam section - for each depth d, there is a range for d1 (from 0 to d), with different costs (see next figure)

The formula used for cost calculation is detailed in the Maple™ code below.

restart;
Variables:
d:bottom depth under low water level
d1:caisson depth under low water level
Input data:
mw:minimum caisson width (to allow road at the top) $mw := 12$:
r:ratio depth/width caisson $r := 1$:
m:lateral margins at caisson edge $m := 4$:
s:slopes dam $s := 2.5$:
crownwallp: price of crown wall $crownwallp := 3296$:
pavp: price of pavement $pavp := 187$:
upcal:unit price concrete armour layer (per cubic meter) $upcal := 375.24$:
upc:unit price caisson (per cubic meter) $upc := 129$:
upqr:unit price quarry run (per cubic meter) $upqr := 32.96$:

COMPOSITE SECTION:

Composite-concrete armour layer:

$Lal := \left(\left((d - d1) \cdot 2.5 \right)^2 + (d - d1)^2 \right)^{0.5}$: length of armour layer

$length1 := piecewise(Lal \leqslant 4, 4, Lal > 4, Lal)$: minimum length set at 4 meters, at the top

$ccal := piecewise\left(d \leqslant 20, length1 \cdot 3 \cdot upcal \cdot 0.55, d > 20, \left(\left((20 - d1) \cdot 2.5 \right)^2 + (20 - d1)^2 \right)^{0.5} \cdot 3 \cdot upcal \cdot 0.55 \right)$:

Composite-caisson:

$$cc := piecewise\left(d1 + 5 \leqslant \frac{mw}{r}, mw \cdot (d1 + 5) \cdot upc, d1 + 5 > \frac{mw}{r}, (d1 + 5)^2 \cdot r \cdot upc \right) :$$

Composite-quarry run:

$$cq := piecewise\left(d1 + 5 \leqslant \frac{mw}{r}, \left((mw) + m \cdot 2 + 2 + s \cdot (d - d1) \right) \cdot (d - d1) \cdot upqr, d1 + 5 > \frac{mw}{r}, \left((d1 + 5) + m \cdot 2 + 2 + s \cdot (d - d1) \right) \cdot (d - d1) \cdot upqr \right) :$$

Composite-Total:

$composite := crownwallp + pavp + ccal + cc + cq;$

$$3483 + \left(\begin{cases} 619.1460 \left(\begin{cases} 4 & \left((2.5\,d - 2.5\,d1)^2 + (d - d1)^2 \right)^{0.5} \leq 4 \\ \left((2.5\,d - 2.5\,d1)^2 + (d - d1)^2 \right)^{0.5} & 4 < \left((2.5\,d - 2.5\,d1)^2 + (d - d1)^2 \right)^{0.5} \end{cases} \right) & d \leq 20 \\ 619.1460 \left((50.0 - 2.5\,d1)^2 + (20 - d1)^2 \right)^{0.5} & 20 < d \end{cases} \right) + \left(\begin{cases} 1548\,d1 + 7740 & d1 \leq 7 \\ 129\,(d1 + 5)^2 & 0 < -7 + d1 \end{cases} \right) + \left(\begin{cases} 32.96\,(22 + 2.5\,d - 2.5\,d1)\,(d - d1) & d1 \leq 7 \\ 32.96\,(-1.5\,d1 + 15 + 2.5\,d)\,(d - d1) & 0 < -7 + d1 \end{cases} \right) \quad \textbf{(1)}$$

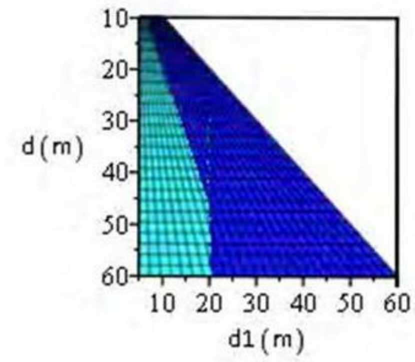
COST OPTIMISATION

Having the cost formula for the two types of cross section, it can be found which option is more inexpensive. In case of a given cross section depth (d) the composite cross section has the lowest costs. It can be found (by optimization of the curve) for which caisson depth d1 the cost is minimal.

The Maple™ graphs that allow this comparison can be found on next page.

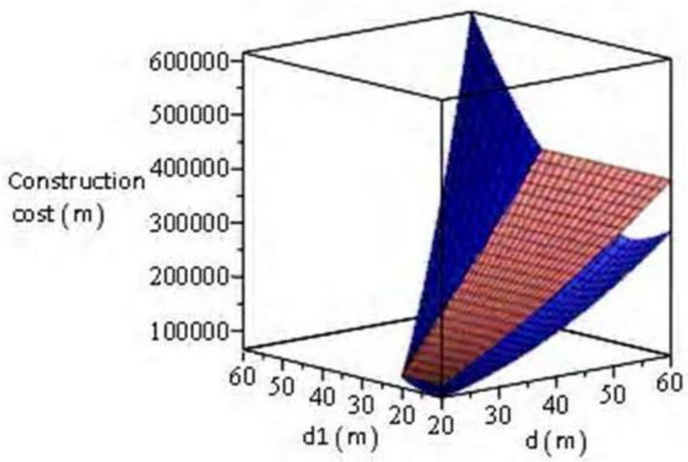
Optimal caisson depth d1 for a given cross section depth d:

mind1 := *diff*(composite, d1) :
plot3d([*mind1*, 0], *d* = 10 ..60, *d1* = 5 ..*d*, *color* = [*blue*, *cyan*]);



Rubble mound vs. composite section:

plot3d([*composite*, rubblemound], *d* = 20 ..60, *d1* = 10 ..*d*, *color* = [blue, *orange*]);



RESULTS

The following graph represents construction costs for the two cross section typologies, depending on two variables, d and d1:

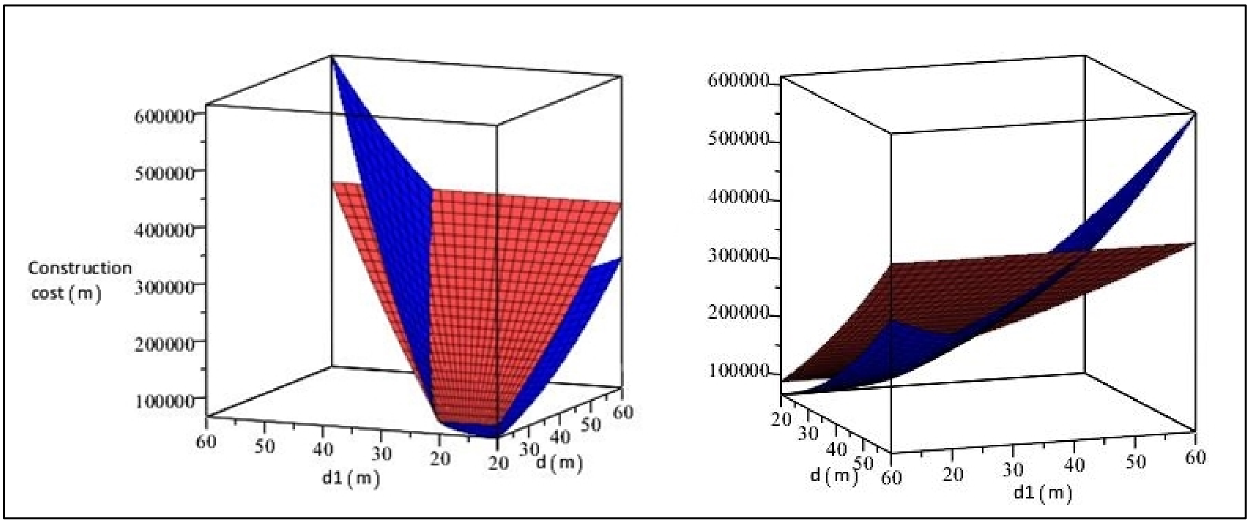


Figure A5 4: Cost (€/meter) for a rubble mound dam (orange) and a composite dam (blue), depending on bottom depth (d) and caisson depth (d1)

In Figure A5-1-4, it can be observed that, for any depth d, the minimum cost is always found for a composite cross section. Choosing a bottom depth (d), the composite solution (blue) has always points with lower cost than the dam (orange).

Taking only the part corresponding to the composite cross section (blue one in the previous graphs), a graph has been developed, where the optimal caisson depth (d1) is found for each bottom depth (d). The result can be found in the following figure.

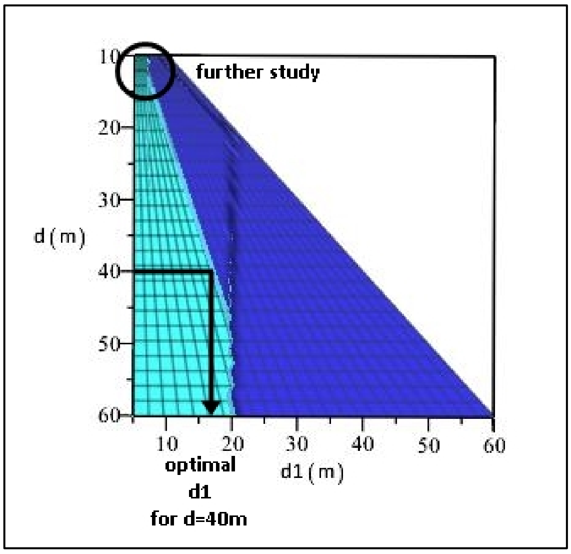


Figure A5 5: Optimal caisson depth (d1) related to bottom depth (d), for a composite dam section

In Figure A5-1-5, the circle marks an area needing further definition. A more detailed calculation would be necessary for depths lower than 10-15 meters, since the dimensions of the cross section have been estimated for large depths. Besides that, construction process can be different, as well as prices, due to smaller material volumes. Therefore, it will be assumed that for low depths a rubble mound solution will be preferable.

In principle, due to these results, a caisson with a depth of 20 meters would be chosen for large cross section depths.

A5-2. CLOSURE WITH CAISSONS

Once the dimensions of the caissons are defined, the feasibility of construction has to be checked. This means to calculate the current velocities in case of closure and check that they are in an acceptable range. The conditions taken into account for this check are:

- Movable barrier open
- Total cross section including movable barrier and permanent opening surface

These conditions are the same as the ones that should be studied to check navigation: barrier open and permanent opening together make the cross section.

The check for velocities is done using the discrete system approximation (see APPENDIX 2). The check is done for the two situations presented in Figure A5-2-1.

Table A5 2: Velocities in closure cross section when closure starts.

VELOCITIES IN THE GAP (WHEN STARTING THE CLOSURE)			
length		m	original opening dimensions
depth		m	
T	12	h	semi-diurnal tide
ω	0.0001454		$2\pi/T$
χ	0.5		(gap)
A_k	881796158	m^2	
A_s	**243386**	**m^2**	
ζ_z	1.05	m	0.5*tidal range outside
Γ	0.01		
r	1.00		
ζ_k	1.05	m	
Tidal range inside	2.10	m	$2*\zeta_k$
% original tidal range	1.00		same ratio Eastern Scheldt post-barrier
Q_{max}	134585.67	m^3/s	
v_{max}	**0.55**	**m/s**	

Table A5 3: Velocities in closure cross section when placing last caisson.

MAXIMUM VELOCITIES IN THE GAP (PLACING LAST CAISSON)			
T	12	h	semi-diurnal tide
ω	0.0001454		$2\pi/T$
χ	0.5		(gap)
A_k	881796158	m^2	
$\mathbf{A_s}$	**33750**	$\mathbf{m^2}$	
ζ_z	1.05	m	0.5*tidal range outside
Γ	0.66		
r	0.87		
ζ_k	0.91	m	
Tidal range inside	1.82	m	$2*\zeta_k$
% original tidal range	0.87		same ratio Eastern Scheldt post-barrier
Q_{max}	116926.51	m^3/s	
$\mathbf{v_{max}}$	**3.46**	**m/s**	

Using the values obtained for the placement of last caisson, the evolution of velocities with time has been calculated with Maple™. Tidal velocities have been assumed sinusoidal.

CURRENT VELOCITIES WHEN PLACING LAST CAISSON

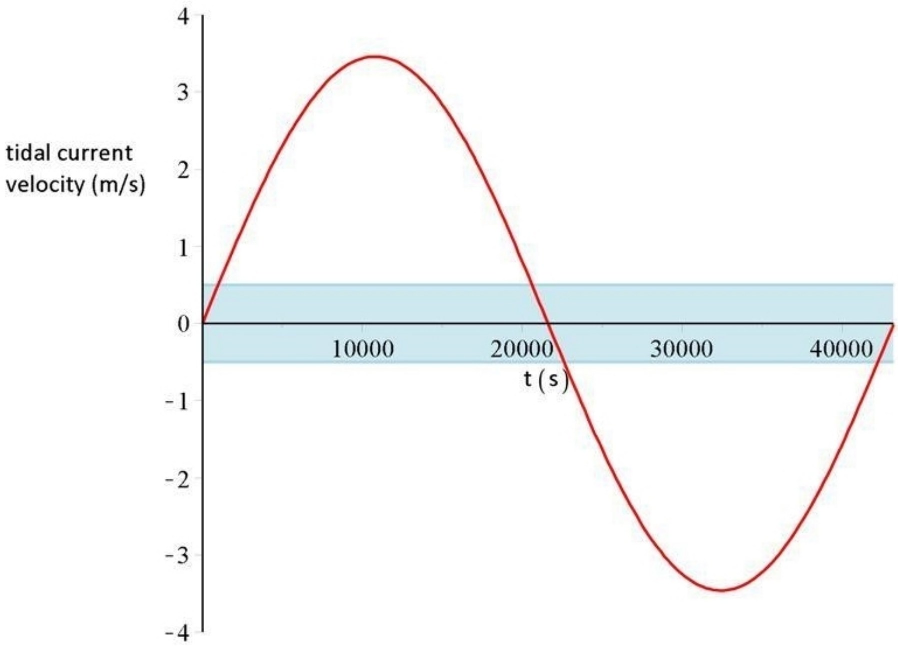
restart;

$T := 12 \cdot 3600 : \omega := \frac{6.28}{T} : vmax := 3.46 :$ all units meters and seconds

Available operational window for caisson placement . Maximum allowed velocity: 0.5m/s (blue stripe in graph below)

$v := vmax \cdot \sin(\omega \cdot t) : upperlimit := 0.5 : lowerlimit := -0.5 :$

plot({*v*, *upperlimit*, *lowerlimit*}, *t* = 0 ..*T*, *color* = ["LightBlue", "LightBlue","Red"], *filled* = [true,true, false]);



$t1 := fsolve\left(\text{upperlimit} = \text{v}, \text{t} = \frac{\text{T}}{4} .. \frac{3 \cdot T}{4}\right) : t2 := fsolve\left(\text{lowerlimit} = \text{v}, \text{t} = \frac{\text{T}}{4} .. \frac{3 \cdot T}{4}\right) :$

$intervalminutes := \frac{(t2 - t1)}{60};$

$$33.25 \qquad (1)$$

In brief, when starting the closure process, maximum velocities are already higher than the maximum recommended for placing caissons. And for the last caisson they would reach a value of 3.46 m/s. Moreover, duration of water slack (velocities lower than 0.5 m/s) is 33 minutes. This will difficult the use of caissons in the cross section, as it is explained in the main report.

Appendix 6

DAM CALCULATIONS

APPENDIX 6 – DAM CALCULATIONS

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A6-1. STABILITY OF SAND FILLING UNDER WATER LOADS

Here, the possibility of liquefaction, induced by water head differences, will be studied. As explained in the report, the worst case scenario is the maximum head difference induced by a storm surge (during a typhoon of 500 years return period).

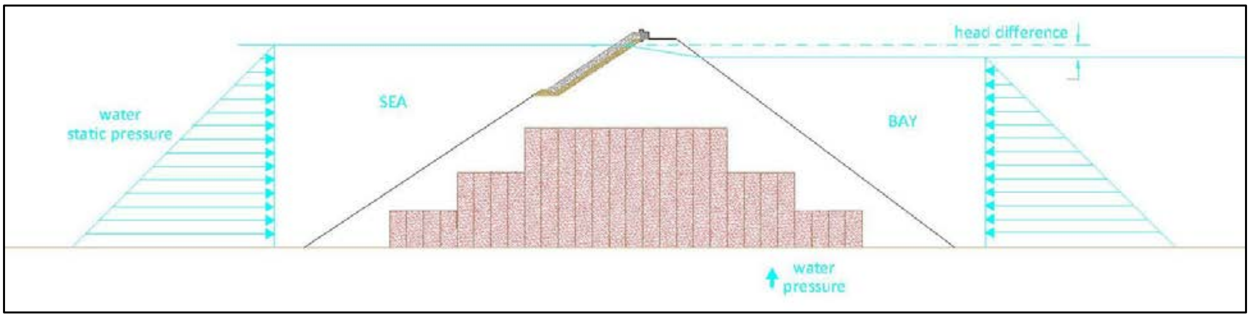


Figure A6-3-1: Water head difference during typhoon and induced vertical water pressures

In order to find out, calculations using Maple™ have been carried out.

Firstly, the water difference has to be calculated. The maximum elevation that the storm surge will reach is not equivalent to the head difference, as water levels inside the bay also change with time. For a more accurate value, it is calculated where is the maximum difference between water elevations:

Data used here comes from Appendix 2, corresponding to location 5 and a typhoon with a return period of 500 years (see Table A2-2-18).
Variables (all units kg, m, s, degrees):

restart;

$T := 12 \cdot 3600 : \omega := \dfrac{6.28}{T}$:

$\chi := 0.5$:

$Ak := 8.82 \cdot 10^{8}$:

$\zeta := 1.87$:

$\Gamma gamma := 28.23$:

$r := 0.19 : \theta := \arccos(r)$:

$\zeta k := 0.35$:

Also from Appendix 2 and corresponding to location 5, the parameters related to normal tidal oscillations are taken:

$\zeta n := 1.05$:

$\Gamma gamman := 0.66$:

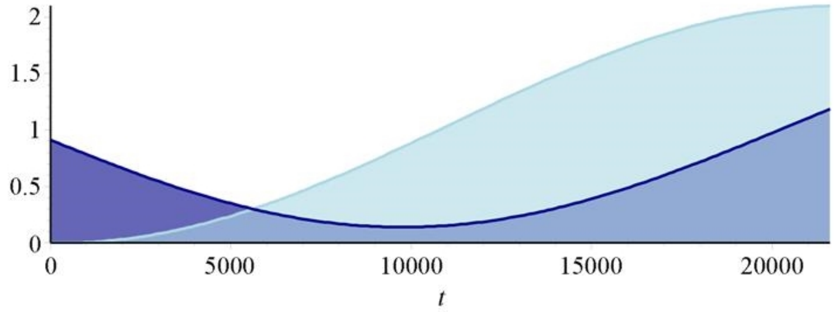
$rn := 0.87 : \theta n := \arccos(r)$:

$\zeta kn := 0.91$:

Water elevations inside and outside the bay are modelled as sinusoidal motions. Low water level in the ocean is taken as elevation 0.
The moment of closure is taken as t=0. For an approximation of water levels, first it is necessary to know which are the levels at t=0. The closure is assumed to happen when the storm surge arrives to the bay and the water levels start to increase. This is assumed to occur at low tide in the sea, so storm surge and tide coincide at their maximum point (they have the same period). The water levels at t=0 are:

$$elevbayn := \zeta kn \cdot \cos(\omega \cdot t + T - \theta n) + \zeta zn;$$
$$0.91\cos(0.00\,t + 43198.62) + 1.05 \quad \textbf{(1)}$$
$$elevsean := \zeta zn \cdot \cos(\omega \cdot t + \mathrm{T}) + \zeta zn;$$
$$1.05\cos(0.00\,t + 43200) + 1.05 \quad \textbf{(2)}$$

$plot\left(\{elevsean, elevbayn\}, t = 0\,..\,\frac{T}{2}, color = [\text{"LightBlue"}, \text{"DarkBlue"}], filled = [\text{true}, \text{true}]\right);$



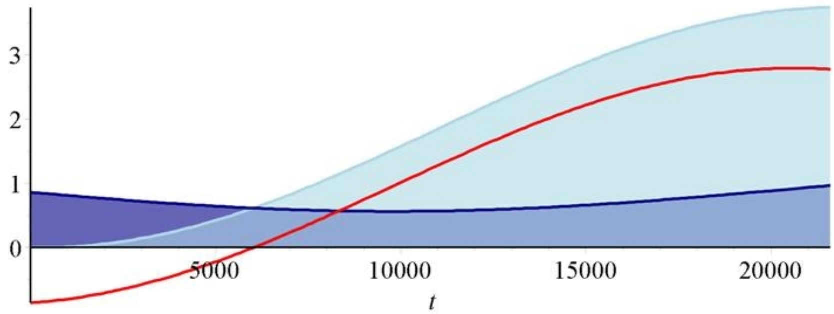
The elevations outside and inside the bay when there is low tide outside (t=0) are:

$$elevsea0 := 0 : elevbay0 := \zeta kn \cdot \cos(\omega \cdot 0 + T - \theta) + \zeta zn;$$
$$0.91 \quad \textbf{(3)}$$

When the storm surge arrives, the amplitude of oscillations changes. It is assumed that the storm surge starts at low water and then, the two motions are added during a semi-period.

$$elevbay := \zeta k \cdot \cos(\omega \cdot t + T - \theta) + elevbay0 :$$
$$elevsea := \zeta z \cdot \cos(\omega \cdot t + \mathrm{T}) + \zeta z :$$
$$difference := elevsea - elevbay;$$
$$1.87\cos(0.00\,t + 43200) + 0.96 - 0.35\cos(0.00\,t + 43198.62) \quad \textbf{(4)}$$

$plot\left(\{elevsea, elevbay, difference\}, t = 0\,..\,\frac{T}{2}, color = [\text{"LightBlue"}, \text{"DarkBlue"}, \text{"Red"}], filled = [\text{true}, \text{true}, \text{false}]\right);$



The maximum difference between water elevations can be calculated, by means of optimisation of the difference function. The considered time range goes from the closure point (minimum water level at sea) to half period more, which is enough to find a maximum of this harmonic function.

$$maxdiff := diff(difference, t);$$

$$-0.00 \sin(0.00\, t + 43200) + 0.00 \sin(0.00\, t + 43198.62) \quad (5)$$

$$maximum := fsolve\left(\text{maxdiff} = 0, \text{t} = 0 .. \frac{T}{2}\right);$$

$$20594.96 \quad (6)$$

$$t := maximum : difference;$$

$$2.79 \quad (7)$$

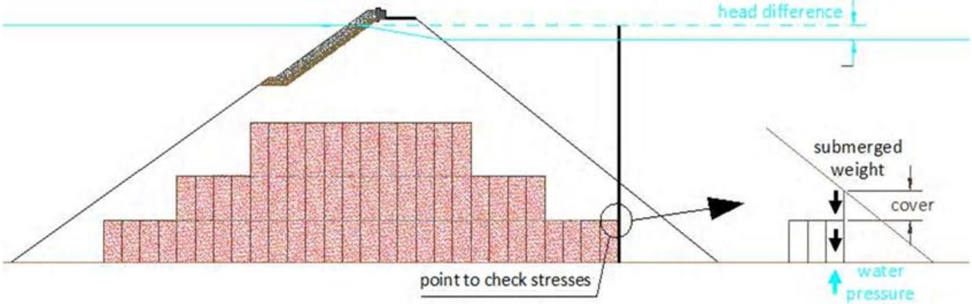
The maximum difference in water levels is therefore 2.8 meters. Now, the stability of the sand column can be analyzed:

restart;
Variables (all units kg, m, s, degrees):
γw: density of water $\gamma w := 1000$:
γc: density of compacted sand $\gamma c := 2100$:
γl: density of loosely packed sand $\gamma l := \frac{0.6}{0.8} \cdot \gamma c$:
nl: porosity of loosely packed sand $nl := 0.43$:
γs: density of submerged sand $\gamma cs := \gamma c - \gamma w : \gamma ls := \gamma l - \gamma w$:
γq: density of quarry run $\gamma q := 1800 : \gamma qs := \gamma q - \gamma w$:
hd: head difference in water levels between sea and bay $hd := 2.8$:

CASES TO CHECK:

A. TYPHOON

The water head difference is assumed to act on the core of the dam. The most unfavorable area is the sand on the bay side, where the water level is minimum. Within this area, the limiting point to check is the top of the sand filling (the bottom will have more weight on it to counteract the uplift caused by water head). An sketch of the situation can be found in the following figure.



Since liquefaction has to be avoided, the effective pressure on the sand has to be larger than zero. This will happen if the submerged weight of quarry run over the sand can compensate the uplift pressure due to water head difference. Therefore, from the balance of vertical pressures at the check point:

$$cover := \frac{hd \cdot \gamma w}{\gamma qs};$$

$$3.50 \quad (1)$$

In consequence, a cover larger than 3.50 meters will be sufficient to avoid liquefaction induced by water levels difference during a typhoon.

A6-2. STABILITY OF QUARRY RUN UNDER WATER LOADS

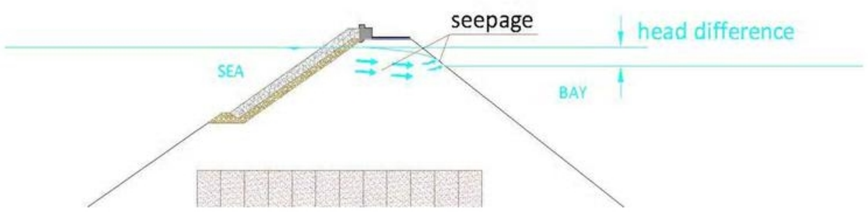
Here, the possibility of grains in the dam core being washed away by water flow is studied.

restart;
Variables (all units kg, m, s, degrees):
γw: density of water $\gamma w := 1000$:
φq: friction quarry run $\varphi q := 35$:
α: slope $\alpha := \arctan\left(\frac{1}{2.5}\right)$:
nq: porosity of quarry run $nq := 0.4$:
γq: density of quarry run $\gamma q := 1800$:
Δ: relative density $\Delta := \frac{(\gamma q - \gamma w)}{\gamma w}$:
k: permeability of quarry run core $kq := 0.1$: valid for small rock and turbulent flow
p: coefficient applied to rewrite Forchheimer equation as uf=k(i)^(1/p), p=1 for laminar flow and p=2 for turbulent flow $p := 2$
ψ: critical Shields parameter $\psi := 0.06$:
g: gravitational constant $g := 9.81$:
hd: head difference in water levels between sea and bay $hd := 2.8$:
x: horizontal distance $x := 20$:

CHECKS:

A.MICRO-STABILITY OF SLOPE

The micro-stability of the outer grains on the slope is checked in case of a large head in water levels occur between the inner and outer side of the dam. In this case, the top part of the dam core, formed by quarry run, could be subject to seepage. Perpendicular seepage is assumed in order to perform the calculations. The aim is to find whether the pressure gradients associated to the design head in water level are larger than the maximum allowed for the slope stability.



In this case, the equation for stability is: tan φq = sin α / (cos α - i)
From this equation, the maximum gradient for which the slope is stable can be obtained:

$$istable := \cos(\alpha) - \frac{\sin(\alpha)}{\tan\left(\frac{\varphi q \cdot 6.28}{360}\right)};$$

$$0.40 \tag{1}$$

This value needs to be compared with the gradient associated to the design water heads.

$$i := \frac{hd}{x};$$

$$0.14 \tag{2}$$

$$evalb(i \leq istable);$$

$$true \tag{3}$$

The design slope is low enough to guarantee stability under perpendicular seepage.

B. GRAINS WASHING

The Forchheimer equation gives a measure of the filter velocity, and therefore the real velocity in the pores. The equation can be written in the following form so it gives the value of filter velocity depending on the hydraulic gradient:

$$uf := kq \cdot i^{\frac{1}{p}};$$

$$0.04 \tag{4}$$

This gives the real velocity in the pores:

$$u := \frac{uf}{nq};$$

$$0.09 \tag{5}$$

Applying the Shields relation for the threshold of movement, it can be calculated which particles will put in motion. The equation reads u*c^2=Δgdψ. Taking ψ=0.06 for the start of motion, the diameter of particles that will move can be calculated.

$$criticaldiameter := \frac{u^2}{\Delta \cdot g \cdot \psi};$$

$$0.02 \tag{6}$$

The result is that particles smaller than 0.02 m, namely 2 centimeters, could be put in motion and washed.

A6-3. GEOTEXTILE STRENGTH

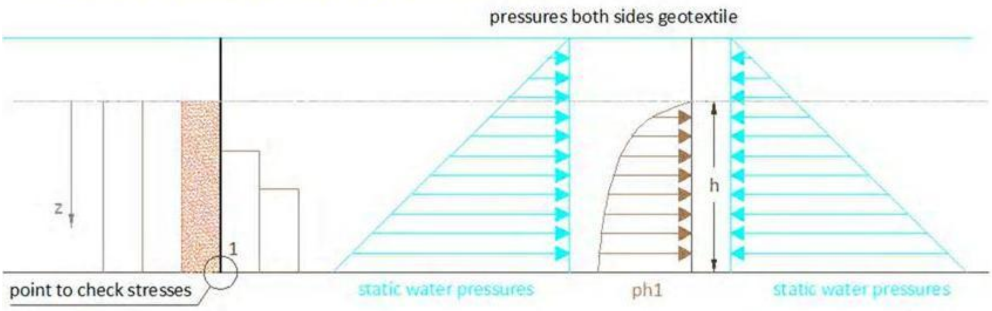
restart;
Variables (all units kg, m, s, degrees):
φs: friction angle sand $\varphi s := 40$:
φq: friction quarry run $\varphi q := 35$:
δ: friction angle sand vs. geotextile $\delta := 25$:
γw: density of water $\gamma w := 1000$:
γc: density of compacted sand $\gamma c := 2100$:
γl: density of loosely packed sand $\gamma l := \frac{0.6}{0.8} \cdot \gamma c$:
nl: porosity of loosely packed sand $nl := 0.43$:
γs: density of submerged sand $\gamma cs := \gamma c - \gamma w$: $\gamma ls := \gamma l - \gamma w$:
γq: density of quarry run $\gamma q := 1800$: $\gamma qs := \gamma q - \gamma w$:
ka: active earth pressure coefficient for quarry run $ka := \tan\left(\frac{3.14}{4} - \frac{\varphi q \cdot 6.28}{2 \cdot 360}\right)$:
L: horizontal dimension cell $L := 5$:
h: head difference in sand $h := 10$:
A: horizontal cross section area $A := L^2$:
U: perimeter $U := 4 \cdot L$:
Res: geotextile tensile strength (Kg/m) $Res := 20000$: $fseam := 2$: $Resworking := \frac{Res}{fseam}$;

$$10000 \qquad (1)$$

ks: sand coefficient ph/pv $\mu s := \frac{\tan\left(\frac{6.28 \cdot \delta}{360}\right)}{\tan\left(\frac{6.28 \cdot \varphi s}{360}\right)}$: $m := \sqrt{1 - \mu s^2}$: $ks := \frac{\left(1 - \sin\left(\frac{6.28 \cdot \varphi s}{360}\right)\right)}{\left(1 + \sin\left(\frac{6.28 \cdot \varphi s}{360}\right)\right)}$:
k: seismic coefficient (average of two orientative values given in Japanese guidelines)
$regional := 0.15$: $subsoil := 1.2$: $importance := 1.5$: $k1 := regional \cdot subsoil \cdot importance$:
$k2 := \frac{1.5^{0.33}}{3}$: $k := \frac{(k1 + k2)}{2}$:

CASES TO CHECK:

A. FILLING OF CELLS DURING CONSTRUCTION.



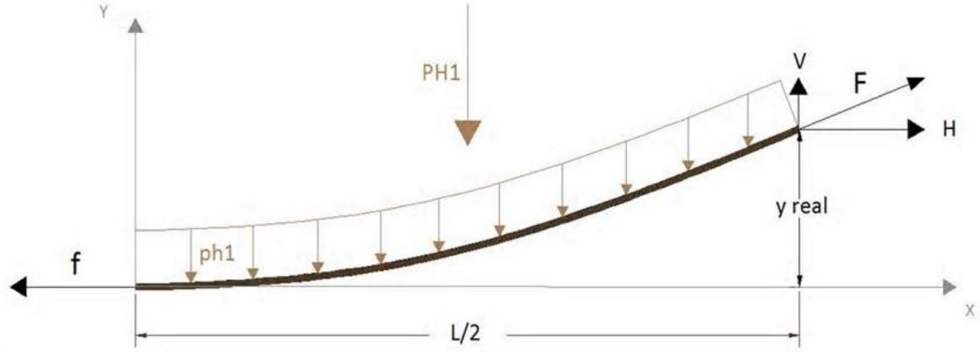
In this case, the maximum pressure to withstand is for one cell that is filled while the adjacent ones remain empty. The submerged pressure of the sand (submerged and compacted) at the bottom point acts as load (the hydrostatic water pressure cancels since it is acting on both sides).

The horizontal pressure (kg/m[2]) for a silo shaped cell is (Janssen):

$$ph1 := \frac{\gamma s}{\tan\left(\frac{6.28\cdot\delta}{360}\right)}\cdot\frac{A}{U}\cdot\left(1-\mathrm{e}^{-\left(\frac{U\cdot ks\cdot\tan\left(\frac{6.28\cdot\delta}{360}\right)}{A}\cdot z\right)}\right):$$

$$2950.40 - 2950.40\,\mathrm{e}^{-0.08z} \qquad \mathbf{(2)}$$

The geotextile has to be able to withstand this pressure. Thus, for a given h, it is possible to calculate the induced tensile forces on the geotextile and compare it with its strength. In order to do that, the strength and the maximum deformation of the geotextile are used. In the following calculations, the last meter of geotextile next to the bottom is the element considered.



The geotextile deformation has been assumed to be linearly related to the applied tension and it has to be calculated as well (the maximum is y=1.30m for L=5m, corresponding to a maximum deformation of 17%, according to product specification).

$ymax := 1.30 : z := h : ph1;$

$$1639.78 \qquad \mathbf{(3)}$$

$$PH1 := \frac{ph1\cdot L}{2};$$

$$4099.46 \qquad \mathbf{(4)}$$

$$yrealm := solve\left(\sqrt{\left(\frac{PH1\cdot L}{4\cdot y}\right)^2 + (\mathrm{PH1})^2} = \frac{Res\cdot y}{ymax}, y\right):$$

$yreal := yrealm[1];$

$$0.61 \qquad \mathbf{(5)}$$

Once the deformation is known, equilibrium of forces can be applied to find the maximum tension at the geotextile F:

$\Sigma\mathrm{M}=0 \Rightarrow f := \frac{PH1\cdot L}{4\cdot yreal}:$

$\Sigma\mathrm{H}=0 \Rightarrow H := f:$

$\Sigma\mathrm{V}=0 \Rightarrow V := PH1:$

$F := \sqrt{H^2 + V^2};$

$$9364.03 \qquad \mathbf{(6)}$$

$$\mathbf{(7)}$$

This maximum tension at the edge of the cell has to be lower than the working resistance (both

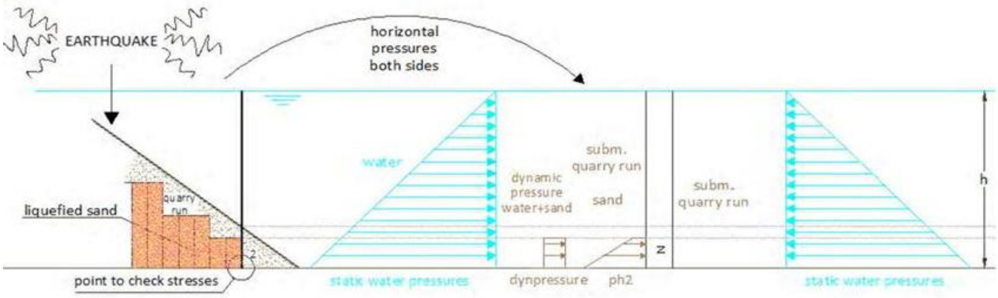
magnitudes in kg)
$evalb(F \leqslant Resworking);$

$$true \tag{8}$$

Therefore, for the chosen h=10 m, the condition is fulfilled. The geotextile is able to resist this head difference between cells during the filling process.

B. EARTHQUAKE, INCLUDING LIQUEFACTION OF SAND INSIDE GEOTEXTILE CELLS.

Here, the cells are already built and filled. The maximum head is found at the edge cell. Several values will be considered for the height of the cells at the edge, as it has already been calculated that the strength is enough for filling (smaller heights would lead to larger filling costs of the edge cell). A 3m cover is considered over the sand cells, made of quarry run (so the slopes can be steeper).
Static and dynamics pressures are represented here. Note that all the static forces are represented. Then, for the dynamic forces, the ones along the depth of the geocell are presented, since above the cell, they do not affect the equilibrium of the cell anymore . It is also considered that the dynamic pressure that affects the quarry run outside the cell does not induce any forces on the geotextile.



In this case, the maximum pressure to withstand in the bottom is calculated from the balance accounting for static and dynamic pressures.
The most unfavourable point is considered to be the cell at the cross section edge, since the non confined soil in the edge is supposed to have more freedom to move,
so the reduction in pressures at the external side is expected to be larger.
As a simplified model, it is proposed to consider static pressures (of liquefied sand and solid quarry run) and add to them:
- dynamic pressures on the left side, acting on the cell (towards the right direction in the above figure)
- dynamic pressures on the right side, on the quarry run. These pressures added to the static pressures can be expressed by means of a different active pressure coefficient. Since the critical situation here is not adding to the earth pressure but subtracting, for simplicity it will be considered that the quarry run pressures on the geotextile are cancelled by dynamic horizontal forces.
The dynamic pressure is considered to be uniformly distributed over the depth and the mass affected is the whole cell, considered as a block.
For the bottom of the sand cell, the pressures are calculated below (calculations according to 7.7.5 Japanese design guidelines).

$depthsand := 10 : z := depthsand :$

$$ph2 := \frac{\gamma s}{\tan\left(\frac{6.28\cdot\delta}{360}\right)} \cdot \frac{A}{U} \cdot \left(1 - e^{-\left(\frac{U\cdot 1\cdot \tan\left(\frac{6.28\cdot\delta}{360}\right)}{A}\cdot z\right)}\right);$$

$$2879.49 \tag{9}$$

$dynpressuresand := k \cdot \chi \cdot (L);$

$$3418.04 \tag{10}$$

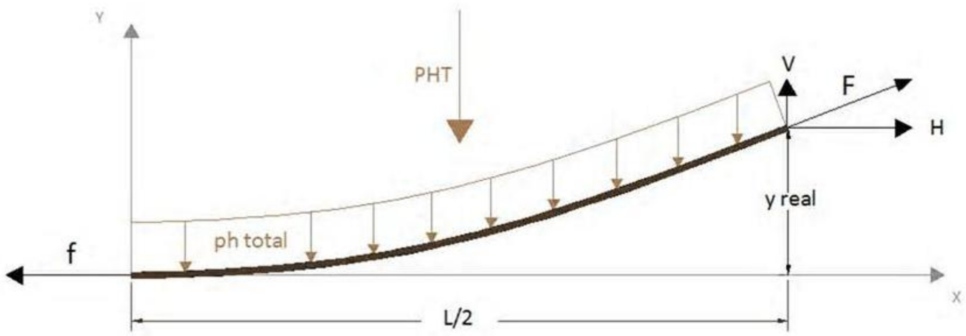
$submsandqrun := 0:$

(11)

$phtotal := ph2 + dynpressuresand;$

$$6297.54 \tag{12}$$

This pressure is equivalent to the following tensile force on the geotextile:



$PHT := \dfrac{phtotal \cdot L}{2};$

$$15743.84 \tag{13}$$

$yrealm := solve\left(\sqrt{\left(\dfrac{PHT \cdot L}{4 \cdot y}\right)^2 + (\mathrm{PHT})^2} = \dfrac{Res \cdot y}{ymax}, y\right):$

$yreal := yrealm[1];$

$$1.38 \tag{14}$$

Since the maximum deformation is reached, from now on, the given maximum of y=1.30 is used (assuming that 17% is the maximum deformation regardless the tensile strength of the geotextile). The equilibrium of forces gives the tension that the geotextile has to withstand:

$\Sigma\mathrm{M}{=}0 \;\; f := \dfrac{\mathrm{PHT} \cdot L}{4 \cdot ymax}:$

$\Sigma\mathrm{H}{=}0 \;\; H := f;$

$$15138.31 \tag{15}$$

$\Sigma\mathrm{V}{=}0 \;\; V := \dfrac{ph2 \cdot L}{2};$

$$7198.73 \tag{16}$$

$F2 := \sqrt{H^2 + V^2};$

$$16762.76 \tag{17}$$

This has to be lower than the working tensile strength:

$evalb(F2 \leqslant Resworking);$

$$\mathit{false} \tag{18}$$

Needed tensile resistance for geotextile (kN/m):

$$Res2 := \frac{F2 \cdot fseam}{100};$$

$$335.26 \tag{19}$$

Needed number of stainless steel cables to help the geotextile to withstand the tension (in 1 meter height, cables 10mm diameter and 60KN yield strength each one):

$$cable10mm := \frac{(F2 - Resworking)}{100 \cdot 60};$$

$$1.13 \tag{20}$$

Therefore, the addition of two cables per meter would allow the geotextile to resist the loads generated during an earthquake.

A6-4. STABILITY OF GEOTEXTILE ELEMENTS

Here, the sliding stability of one single geotextile element is studied.

$restart;$

Variables (all units kg, m, s, degrees):

φs: friction angle sand $\varphi s := 40$:

φq: friction quarry run $\varphi q := 35$:

δ: friction angle sand vs. geotextile $\delta := 25$:

γw: density of water $\gamma w := 1000$:

γc: density of compacted sand $\gamma c := 2100$:

γl: density of loosely packed sand $\gamma l := \frac{0.6}{0.8} \cdot \gamma c$:

nl: porosity of loosely packed sand $nl := 0.43$:

γs: density of submerged sand $\gamma cs := \gamma c - \gamma w : \gamma ls := \gamma l - \gamma w$:

γq: density of quarry run $\gamma q := 1800 : \gamma qs := \gamma q - \gamma w$:

ka: active earth pressure coefficient for quarry run $ka := \tan\left(\frac{3.14}{4} - \frac{\varphi q \cdot 6.28}{2 \cdot 360}\right)$:

L: horizontal dimension cell $L := 5$:

A: horizontal cross section area $A := L^2$:

U: perimeter $U := 4 \cdot L$:

Res: geotextile tensile strength (Kg/m) $Res := 20000 : fseam := 2 : Resworking := \frac{Res}{fseam};$

$$10000 \qquad (1)$$

ks: sand coefficient ph/pv $\mu s := \frac{\tan\left(\frac{6.28 \cdot \delta}{360}\right)}{\tan\left(\frac{6.28 \cdot \varphi s}{360}\right)} : m := \sqrt{1 - \mu s^2} : ks := \frac{\left(1 - \sin\left(\frac{6.28 \cdot \varphi s}{360}\right)\right)}{\left(1 + \sin\left(\frac{6.28 \cdot \varphi s}{360}\right)\right)}$:

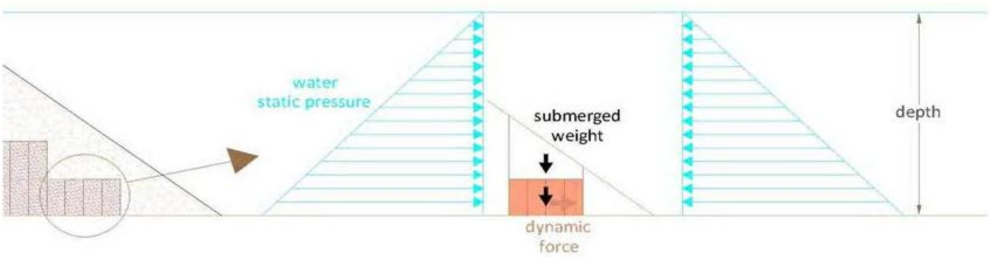
k: seismic coefficient (average of two orientative values given in Japanese guidelines)

$regional := 0.15 : subsoil := 1.2 : importance := 1.5 : k1 := regional \cdot subsoil \cdot importance$:

$k2 := \frac{1.5^{0.33}}{3} : k := \frac{(k1 + k2)}{2}$:

CASES TO CHECK:

A. EARTHQUAKE



Since the element is about to start sliding, it is assumed that it will loose contact with the cells behind. Hence no lateral pressures are applied on it. Only the weight of materials and the dynamic force are contemplated. Also, water suction is added, which is considered to be a conservative decision since suction is in principle the result of a water mass being pushed (here the presence of quarry run could lessen the effect, since it is an obstacle for water movement).

$depthsand := 10:$

$dynforcesand := k \cdot \gamma c \cdot (4 \cdot L \cdot depthsand);$

$$1.37\ 10^5 \tag{2}$$

$hztal := dynforcesand;$

$$1.65\ 10^5 \tag{3}$$

$$subweightqrun := \frac{\left(3 + 3 + \frac{4 \cdot L}{2.5}\right)}{2} \cdot 4 \cdot L \cdot \gamma qs:$$

$subweightsand := depthsand \cdot 4 \cdot L \cdot \gamma cs:$

$totalweight := subweightqrun + subweightsand;$

$$3.32\ 10^5 \tag{4}$$

$$frictionresistance := (totalweight) \cdot \tan\left(\frac{\varphi s \cdot 6.28}{360}\right);$$

$$2.78\ 10^5 \tag{5}$$

this has to be lower than the working tensile strength (both in kg)

$evalb(hztal \leqslant frictionresistance);$

$$true \tag{6}$$

In consequence, the element is stable regarding a possible sliding during earthquake.

A6-5. COSTS

The costs are calculated for the following alternatives:

- Standard rubble mound section.
- Geotextile structure in the core, covered by quarry run.
- Geocontainers in the core (same volume as geotextile structure) covered by quarry run.

In this comparison, costs will be calculated for one meter length of the following cross section. Dimensions are the same for all the alternatives. The only difference is the volume in the center, which can be occupied by quarry run, geotextile cells or geocontainers.

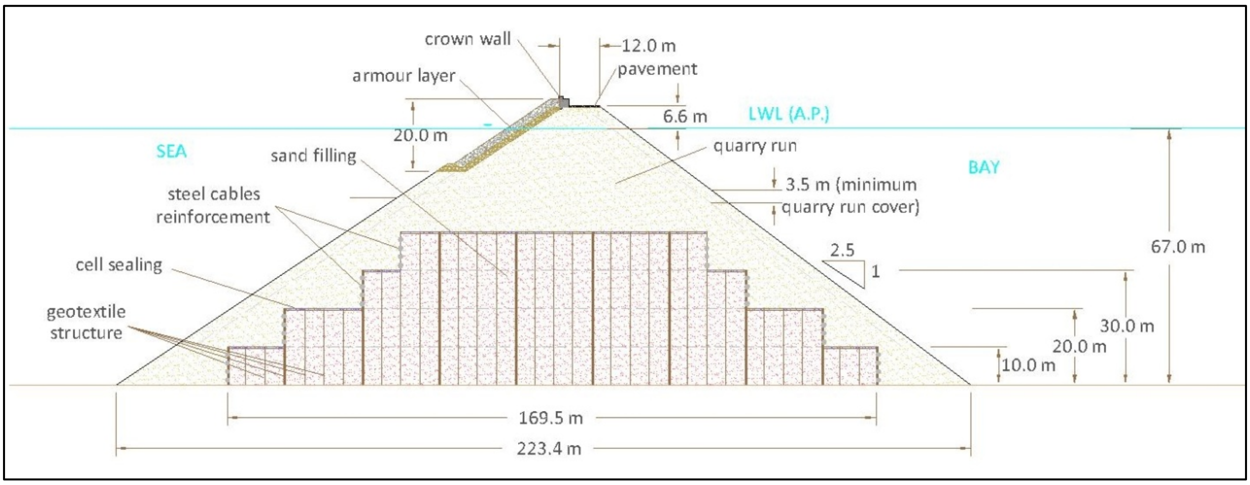


Figure A6 1: Cross section for cost comparison purposes.

The prices previously used in this report (3.4.1.2) are applied here. The only new prices included in this section are:

- Geotextile price, chosen as 3€/m2, which is well above standard woven geotextile prices.
- Sand placement auxiliary operations for geocontainers, which are described in Table A6 1
- Sand placement auxiliary operations for geotextile cells, which are described in Table A6 2.

Once all the prices are determined, they are applied to the amount of materials in one meter length of dam. The final result can be found at Table A6 3.

Table A6 1: Cost of sand placement operations for geocontainers.

Total dam core volume		13304157	m^3				
approximated execution period		2	years				
6 day week + 20% bad weather		482	days				
production rate needed/day		27601.99	m^3/day				
chosen production rate		30000.00	m^3/day	(equivalent to ~6 meters			
Approximate rate of advance		6	m/day	of geocontainers)			
geocontainer 5mdiameter*50mlength		981.25	m^3				
number of geocontainers to drop / day		30.57	units	3.5h filling (300m3/h)+2h transport			
time 1 cycle barge		8	h	+0.5h positioning and drop			
number of barges needed		30		+2h preparation			
1crane+1barge handle 981 m3/day, 30 teams needed							
extra transport sand to working dock: 4 extra barges handle 300m3/h (40 needed for the daily rate)							
	units	length	width	heigth	quantity	unit price	total
COST ONE DAY (€)							
pontoon	day				0.00	3000	-
crane	day				30.00	5000	150,000
operators	day				30.00	2000	60,000
barge	day				70.00	4000	280,000
tug	day				0.00	8000	-
divers	day				0.00	2000	-
material	unit				0.10	5000	500
(protection sheet on barge, etc.)							
							490,500
COST/CUBIC METER (€/m3)							
extra operations	m3						16.35

Table A6 2: Cost of sand placement operations for geotextile cells.

Total dam core volume		13304157	m^3				
approximated execution period		2	years				
6 day week + 20% bad weather		482	days				
production rate needed/day		27601.99	m^3/day				
chosen production rate		30000.00	m^3/day	(equivalent to ~6 meters of geotextile			
Approximate rate of advance		6	m/day	structure)			
2 pontoons+crane+4 barges handle 300m3/h (the production rate of one small dredge)							
approximately 10 teams would be needed (working 10 h/day)							
1 extra team with 1 crane+pontoon+tug+divers will make the placement of geotextile structures and anchors							
	units	length	width	heigth	quantity	unit price	total
COST ONE DAY (€)							
pontoon	day				22.00	3000	66,000
crane	day				11.00	5000	55,000
operators	day				11.00	2000	22,000
barge	day				40.00	4000	160,000
tug	day				10.00	8000	80,000
divers	day				10.00	2000	20,000
material	unit				1.00	5000	5,000
(balloons, anchors, etc)							
							408,000
COST/CUBIC METER (€/m3)							
extra operations	m^3						13.60

Table A6 3: Cost comparison between quarry run, geocontainers and geotextile cells

BUILDING COST COMPARISON							
Calculated per meter length of dam (60m depth,15m width at the top, slope 2.5)							
	units	length	width	height	quantity	unit price	total (€)
QUARRY RUN CORE							
Quarry run	m3		190.00	60.00	8619.00	32.96	284,082
							284,082
GEOCONTAINERS							
Quarry run	m2		3736.00		3736.00	32.96	123,139
Geotextile	m2	0.40	4883.00		3906.40	3.00	11,719
Sand (dredged)	m3				4883.00	6.18	30,177
Extra operations							
Sand placement	m3				4883.00	16.35	79,837
							244,872
GEOTEXTILE CELLS							
Quarry run	m2		3736.00		3736.00	32.96	123,139
Geotextile	m2	0.40	4883.00		1953.20	3.00	5,860
Cables	kg	60.00	4.00	0.62	147.89	4.00	592
Sand (dredged)	m3				4883.00	6.18	30,177
Extra operations							
Sand placement	m3				4883.00	13.60	66,409
Ballast blocks	m3	14.00	2.00		28.00	158.48	4,437
bituminous seal	m2	140.00			169.50	32.50	5,509
							236,122