Assessment of wave load reduction by breakwaters

Bachelor of Science Civil Engineering

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WATERSCHAP ZUIDERZEE LAND

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Preface

Dear reader,

This research was conducted to graduate from my Bachelor Civil Engineering at the University of Twente. Conducted at water authority Zuiderzeeland, from April 2023 until June 2023. The purpose of this research was to explore the effectiveness of breakwaters in reducing wave load on the dike. Motivated by the desire to gain insight into potential inclusion of breakwaters in flood defence assessments.

I would like to thank water authority Zuiderzeeland for allowing me the opportunity of conduction my research at their organisation, and express my sincere gratitude to all those who supported. A special thanks goes out to Marijke Visser, my supervisor at Zuiderzeeland, for her guidance, expertise, and support throughout this research journey. Her valuable insights and encouragement pushed me forward, challenging me to think critically and pushing the boundaries of my knowledge. I would also like to extend my thanks to Jord Warmink, my supervisor from the University of Twente. His input greatly assisted in crafting a concise and clear overview of my research.

Michiel Broenink
Lelystad, June 2023

Summary

This study aims to quantify the wave attenuating effect of different breakwater heights and establish a quantitative relationship between breakwater height and wave load reduction. By varying the breakwater height and observing the resulting variation in wave load on the dike, the effectiveness of breakwaters in mitigating wave impact is examined. The findings demonstrate that breakwaters exhibit a significant reduction in wave load, even when submerged. However, limitations exist in measuring the exact effect of breakwaters, and there is a knowledge gap regarding the profile of a breakwater after failure. Therefore, further research is strongly recommended to investigate the effectiveness of failed breakwaters, as they show promising load-reducing potential, even when submerged. Only with a comprehensive understanding of failed breakwaters and their impact on wave load breakwaters could be included in the safety assessment of primary flood defence structures.

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List of Symbols

α	Angle of incidence [°]
β	Wave direction relative to north [°]
θ	Lake level [m+NAP]
ϕ	Wind direction [°]
a, b, c	Revetment dependent exponents [-]
H_{m0}	Significant wave height [m]
P	Exceeding probability [-]
p	Probability of occurrence [-]
S	Load level - quantity [-]
s	Load level - value [-]
T	Return period [years]
T_p	Peak period [s]
u	Wind speed [m/s]
W	Water level - quantity [m+NAP]
w	Water level - value [m+NAP]

Chapter 1

Introduction

1.1 Problem Context

According to the Dutch Water Act, Article 2.12, the primary water barriers must be assessed at least once every twelve years. The assessment is intended to ensure their safety and compliance with the prescribed norms. The norms specify the accepted flood risk for areas protected by the primary embankment and are based on two elements: each individual benefits the same minimum protection level (Dutch: basisbeschermingsniveau), which is expressed as Local Individual Risk (LIR). The LIR is a political decision and is currently set at LIR $\leq 10^{-5}$ per year. The second principle is that high consequences of a flood result in a lower flood probability. This basically means that the higher the potential consequences of a flood, the higher the corresponding norms [7]. The Water Act stipulates different types of required reliability levels for flood defences. For segments providing direct protection from flooding, the requirements are formulated in terms of the probability of flooding. The probability of flooding is *'the probability of the loss of flood defence capacity in a dike segment causing the area protected by the dike segment to flood in such a way that fatalities or substantial economic damage occur'* [7]. Each dike segment has two values, an alert level and a lower threshold. The alert value (Dutch: signaleringswaarde) indicates the dike requires strengthening in the foreseeable future. The lower threshold (Dutch: ondergrens) is the minimum probability of flooding which the flood defence structure is designed to prevent. This is the maximum permissible value for the probability of flooding [7]. Both values are expressed as return periods. Where a return period of, for example, 1000 years means the probability of occurrence each year is 1/1000. The standards for dike segments in the Netherlands range from 1/1000 to 1/1,000,000 a year [7].

A flood defence system can consist of more elements than the primary embankment only. In some cases, there are additional structures providing protection, such as breakwaters. Breakwaters, or dams, are offshore structures that can serve several purposes. The most obvious purpose of a breakwater is to provide protection against waves. But they can also be built to reduce the amount of dredging required in a harbor entrance or to guide the currents in the channel or along the coast for example [8]. Because they are located outside the dike, they also impact the hydraulic load on the dike and therefore affect the assessment. But since breakwaters are often built only to protect a harbour, they have much higher allowed failure probabilities and management and maintenance is not based on the high norms of the dike. For those reasons, they are likely to fail far before the hydraulic boundary conditions of the primary structure are reached. Therefore, when assessing the dike it is often assumed the breakwater has already failed and thus has no reducing effect on the wave load at the dike. This is an unproven conservative assumption. It is indeed true that in almost any case the breakwater will fail before the hydraulic boundary conditions of the dike are reached. But this does not mean the remnant of the breakwater has no wave attenuating effect. The breakwater will not be there in its original shape, but it is not gone and probably eroded only until a certain height. The issue is that the remaining strength (Dutch: reststerkte) of breakwaters is unknown and to be on the safe side this conservative assumption is widely accepted. The motivation behind this research is to learn more about this remaining strength. If proven that breakwaters still significantly reduce wave load on the dike after failure, this could, or even should be included in the assessment and it can prevent unnecessary dike strengthening. Furthermore, there are situations where dike strengthening is possible only very limited due to multifunctional land uses, i.e. building on or close to the dike. Knowing whether or not breakwaters effectively reduce wave load on the dike even after failure could in such situations provide a solution through the construction or strengthening of a breakwater rather than the dike itself.

1.2 Research Scope

In this research, the wave reducing effect of breakwaters in the control area of water authority Zuiderzeeland will be studied, which roughly corresponds to the province of Flevoland. A cross-section of a typical flood defence structure consisting of a breakwater, foreshore and dike is given in Figure 1.1. Note that many flood defence structures controlled by Zuiderzeeland only consist of a dike. Dams and foreshores occur only in specific situations and they can have different purposes. Note that the words breakwater and dam are used interchangeably throughout the report.

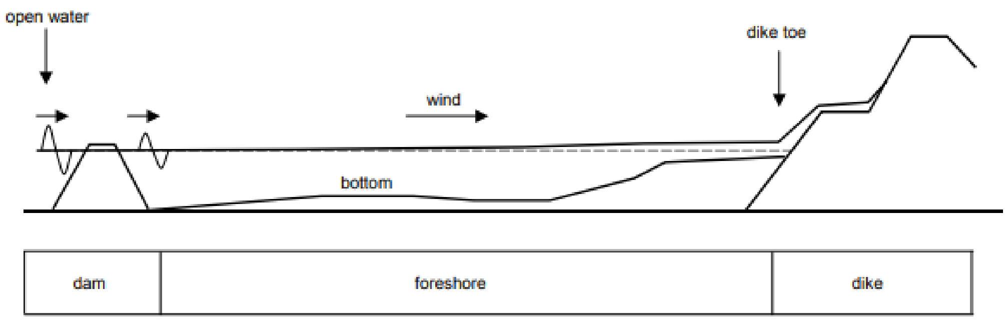


Figure 1.1: Cross-section of a dam/breakwater, foreshore and dike [1].

There are several types of breakwaters that can be divided into roughly two categories: the rubble mound and monolithic type breakwaters [9]. Rubble mound breakwaters consist of large heaps of loose elements with an armour layer of rock or concrete blocks, whereas monolithic breakwaters have a cross-section which acts as one block, for instance, a caisson [10]. Figure 1.2 depicts the representative cross-sections for all breakwater types defined in the Rock Manual [2]. Breakwaters under authority of Zuiderzeeland are of the type conventional rubble mound.

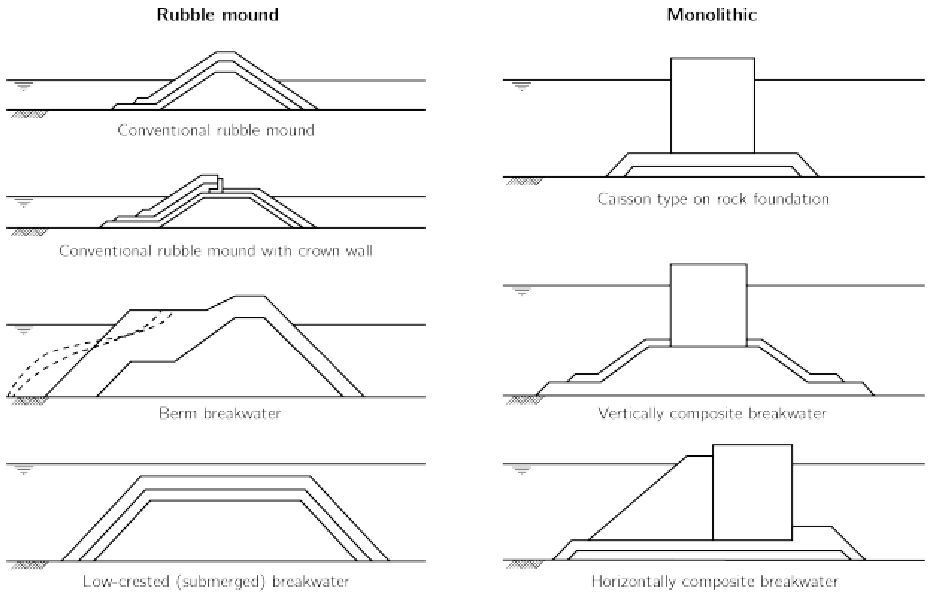


Figure 1.2: Typical cross sections of various types of breakwaters, with the rubble mound types on the left and the monolithic types on the right [2].

Rubble mound breakwaters are located throughout the entire province of Flevoland. Some of them are included in the assessment, but most are not. If a breakwater is incorporated in the assessment, it means the water authority assumes that during normative conditions for the primary embankment the breakwater still provides reduction in wave load. A list of breakwaters with their current state of assessment is provided in Table 1.1, Figure 1.3 shows were these breakwaters are located.

Table 1.1: Current state of breakwaters in Zuiderzeeland control area, the Roman numbers correspond to the numbers in Figure 1.3.

#	Location	State
I	Lemmer	Breakwaters not included in assessment
II	Urk	Included in the assessment of sluice and pumping station.
III	Schokkerhaven	Breakwaters not included in assessment
IV	Ramsdiep	Included in establishment hydraulic boundary conditions by Rijkswaterstaat, strength not assessed by Zuiderzeeland.
V	Ketelhaven	Breakwaters failed the assessment, dike on its own is not sufficient.
VI	Lelystad	Not included in previous assessment, might be included in the next.
VII	Oostvaardersdiep	Breakwater failed the assessment thus not included in dike assessment

Figure 1.3: Location of breakwaters in Zuiderzeeland control area. Primary flood defence structures provide protection against external water from the Northsea, Waddensea, the main rivers and the IJssel- and Markermeer. Regional flood defence structures provide protection against internal water from the rivers and canals [3]. Structures labeled as 'other' do not provide direct protection against water, but have been doing so in the past or might be in case the primary structures fail. A brief description of each case can be found in Table 1.1. Roman numbers in the figure correspond to the numbering in the table.

The last element of focus worth noticing is the type of dike revetment that will be considered. Wave load on a dike is influenced by the way dike revetment behaves while exposed to waves. This behaviour varies per type of revetment, e.g. grass and stone covering, but it can also vary per type of stones. With only a few exceptions, all dikes controlled by Zuiderzeeland are constructed with stone coverings on the lower side of the embankment. Therefore in this research, the effect of wave load reduction on stone covering will be studied. The interaction between waves and breakwater is not affected by this choice, but the determination of wave conditions at the dike does depend on the revetment type.

1.3 Research Field

There is plenty of literature available related to the design process or potential causes of failure of breakwater [11] [12] [13] [14]. However, the wave attenuation of these breakwaters after failure is rarely mentioned. There are some studies related to the effectiveness of submerged breakwaters. Submerged breakwaters are structures with a crest elevation below the local water level. Although they are well suited in situations where minimal visual intrusion is desired, one can not expect transmission coefficients as low as those achievable with structures with a crest height above water level [15]. The transmission coefficient is the ratio between the wave height behind and in front of the breakwater [16]. Several tests to assess the wave transmission coefficient of submerged breakwaters found that it is most sensitive to the depth of submergence, the incident wave height, and the crest width [15]. Note that these tests were conducted with incident waves perpendicular to the structure. Universal quantification of wave reduction by submerged breakwaters is not available because of the huge variety in wave reduction with changing circumstances such as crest depth and incident wave height. As a general conclusion, it can be said that the higher the breakwater, the lower the transmission coefficient [17]. Meaning we can expect the effectiveness of the breakwater to reduce significantly with reducing crest heights.

1.4 Research Objective

The objective of this study is to quantify the wave attenuating effect of different breakwater heights, to understand the quantitative relationship between breakwater height and the reduction of wave load.

1.5 Research Questions

To reach the objective two research questions are formulated:

1. What is the most suitable parameter to observe the effect of a breakwater on wave attenuation?
2. What is the relationship between breakwater height and the effectiveness of wave load reduction?

Chapter 2

Theory & Methode

This chapter elaborates on the method applied to quantify the effect of breakwaters and the theory behind it. Starting with an analysis of the failure mechanism of stone revetment. Subsequently, it will be mentioned how the effect of breakwaters can be measured and how this is linked to the failure mechanism. And lastly, the methodology behind the assessment of flood defence structures in the Netherlands will be dealt with.

2.1 Failure mechanism of stone revetment

A stone revetment, as commonly applied to water defences, typically consists of several layers that are intended to protect the underlying ground against erosion. In the most common stone revetments, these layers are as follows, as shown in Figure 2.1 [4]:

- A top layer of set stones (armour layer). These stones can be tightly fitted together (square concrete blocks), but they can also consist of columnar elements with the gaps between the columns filled with granular material.
- A granular layer, often serving as a leveling layer to smooth out irregularities in the underlying ground, but it can also be part of the filter intended to prevent the washing out of underlying layers.
- A geotextile, also referred to as filter fabric.
- A base layer, usually consisting of clay, but broadly graded granular materials have also been used (e.g., mine waste).

Figure 2.1: Stone revetment during construction [4]. Translation: inwasmateriaal = joint filler / backfill, toplaag van gezette zuilen = top layer of columnar concrete blocks, granulaire uitvullaag = granular layer, filterdoek = filter fabric, onderlaag (breed gegradeerd granulair materiaal) = underlayer (large graded granular material)

There are six initial failure mechanisms that can cause the armour layer to fail. These are top layer (armour) instability due to wave attack, where a stone is lifted out of the stone settlement (Dutch: steenzetting) due to breaking waves on the embankment. Instability due to longitudinal flow, where a stone is extracted from the settlement due to strong currents along the dike. Instability due to scour (erosion) occurs when breaking waves on the embankment deform the subsoil. Instability caused by material transport from the subsoil through the top layer, or material transport from the granular/filter layer through the top layer. And lastly failure of the toe construction [4]. Four of those six failure mechanisms are illustrated in Figure 2.2. This study focuses on the instability of the top layer due to wave attacks. Within wave attack, there is a distinction between wave run-down (Dutch: golfneerloop) and wave impact (Dutch: golfklap), both of them can cause stones to be pushed out of the settlement.

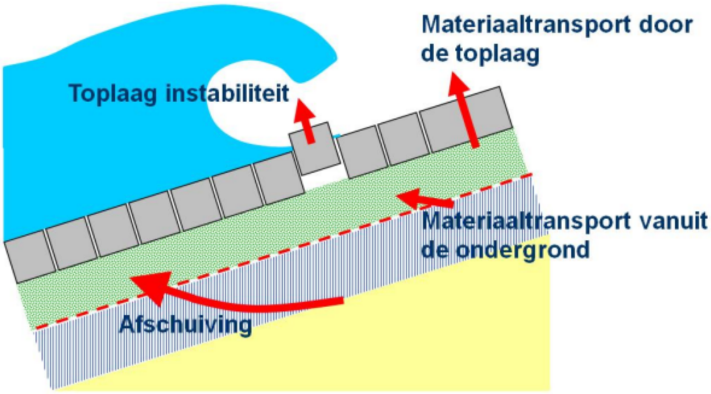


Figure 2.2: A few failure mechanisms of stone revetments. Translation: Toplaag instabilitieit = Instability of the top layer, Materiaaltransport door de toplaag = Material transport through the top layer, Afschuiving = Scour (erosion), Materiaaltransport vanuit de ondergrond = Material transport from the subsoil [4]

Wave run-down

Breaking waves induce a huge load on the revetment of the dike that can cause failure. This failure does not occur at the moment of impact, but when the wave has withdrawn. At this moment there is a wall of water on the embankment, with high pressure in zone A and low pressure in zone B (see Figure 2.3). The high pressure in zone A is transferred through the granular layer (filter) underneath the top layer, causing high pressure towards the top layer under zone B as well. Simultaneously the phreatic line in the filter is increased causing the water to flow downwards and outside (away from the embankment). This causes an upward pressure difference between zone A and zone B which tends to push stones out of the settlement. This situation persists in each wave for about 0.2 to 0.7 seconds, which can be enough for stones to be pushed out of the top layer bit by bit, or all at once by a large wave [4].

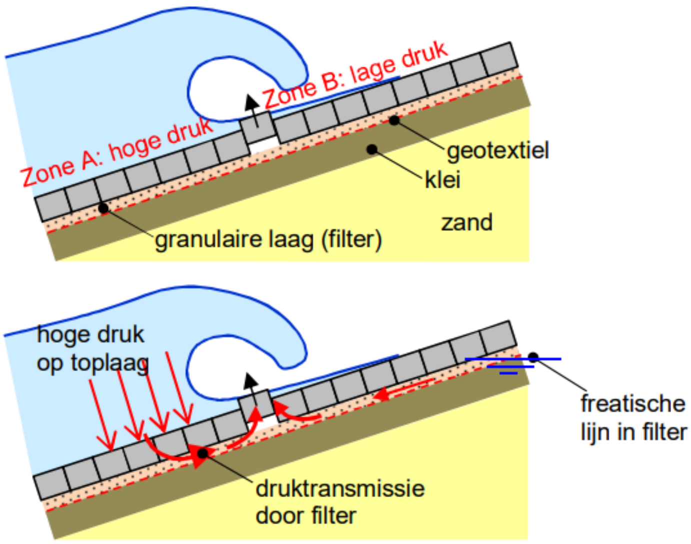


Figure 2.3: Wave down flow leads to a pressure differential across the top layer. Translation: hoge druk = high pressure, lage druk = low pressure, granulaire laag (filter) = granular layer (filter), klei = clay, geotextiel = geotextile, zand = sand, hoge druk op toplaag = high pressure on top layer, druktransmissie door filter = pressure transmission through filter, freatische lijn in filter = phreatic line in filter [4]

Wave run-down occurs for stone deposits with relatively small permeability of the top layer and large permeability of the filter, such as rectangular concrete blocks with narrow crevices. Modern stone deposits have such a high permeability that the moment of wave impact becomes normative [4].

Wave impact

The wave impact produces a short, approximately 0.1 to 0.3 seconds, but extremely high pressure on the embankment. This high pressure occurs only on a narrow strip but is via the filter transferred to surrounding zones creating an upward pressure difference that can cause stones to be pushed out of the settlement, as illustrated in Figure 2.4. The basic principle of this load is in other aspects equal to wave run-down [4].

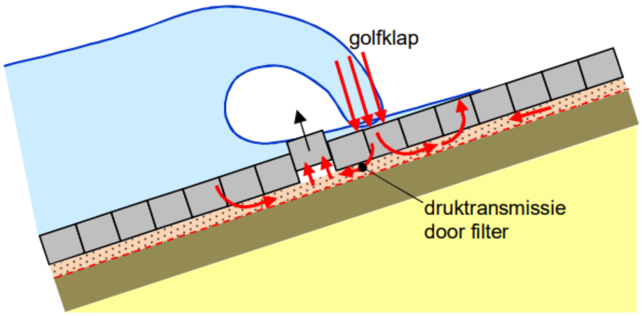


Figure 2.4: Wave transmission from wave impact through the filter resulting in upward pressure differences. Translation: Golfklap = Wave impact, Druktransmissie door filter = Pressure transmission through filter [4]

The magnitude to which both processes cause pressure differences across the top layer depends on the permeability of that layer and the underlying granular layer. A relatively permeable filter and impermeable top layer are unfavorable for the stability of the revetment. Since this results in larger pressure differences for equal wave conditions. A well-designed stone revetment consists of a low permeable filter and a high permeable top layer [4].

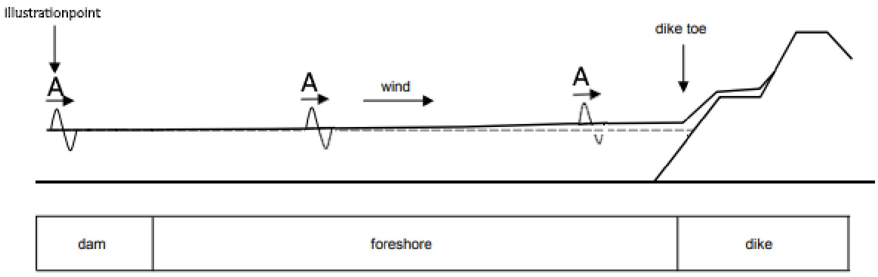
The strength of stone revetment depends on the weight of the top layer per square meter and the interaction between individual stones. The open spaces between individual stones are not included in the weight per square meter, even when they are filled with rubble. Meaning only the thickness of the layer and the material density are relevant. Interaction between stones depends on the characteristics and placement of the stones. Three important elements defining interaction are the friction between stones. Clamping, which occurs due to the normal force in the plane of the settlement and is further enhanced when deformation of the embankment surface occurs. And interlocking by means of a hollow-and-doll connection for example [4].

Because the magnitude of the failure mechanisms and the strength of the revetment depends on revetment characteristics and the placement of the stones. It can be said that waves are not the only factor determining the load on a dike, but the ability of the revetment to respond to these waves also plays a part.

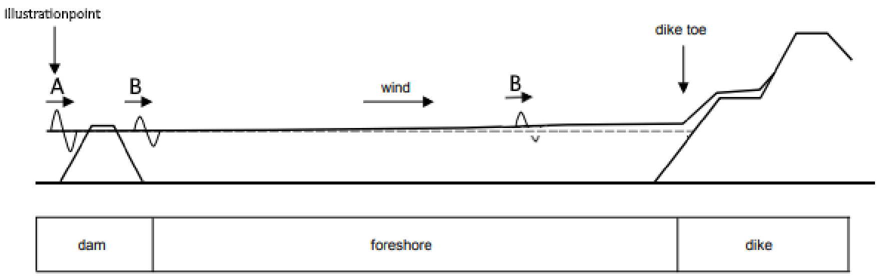
2.2 Physical effect of breakwaters

In the previous section, the failure mechanisms of a stone revetment are discussed. The magnitude to which the failure mechanisms occur depends on both wave- and revetment characteristics. The wave characteristics are affected by a breakwater. This section deals with the physical effect of breakwaters.

Figure 2.5 contains two schematizations of wave propagation towards a dike. Figure 2.5a illustrates how it is assumed waves behave if there is no breakwater and no foreshore. Since there is no breakwater, the behaviour of the waves is not affected between the illustration point and the toe of the dike. The illustration point, the outer left arrow in Figure 2.5, is the location for which Rijkswaterstaat provides the hydraulic boundary conditions. In Figure 2.5b a breakwater is implemented. This breakwater does affect the wave characteristics, meaning the waves at the illustration-point (A) are different than those reaching the dike toe (B). These different waves will cause a different load on the dike. How much the wave load varies depends on the breakwater height. To quantify the effect of the breakwater, different heights will be implemented and the resulting wave load will be compared.



(a) Wave propagation without breakwater



(b) Wave propagation with breakwater

Figure 2.5: Wave propagation affected by a breakwater (dam). Adapted from [1].

2.3 Assessment Flood Defence Structures in the Netherlands

So far the failure mechanisms of stone revetment and the effect of a breakwater in this process are discussed. The following section elaborates on the methodology and theory behind flood risk assessment as it is applied in the Netherlands. The focus will be on the quantification of waves and loads acting on the dike. This process is split up into three phases. It starts with quantifying water levels and wave conditions, i.e. the hydraulic boundary conditions. Secondly, the breakwater module is applied. This is optional and could also be a foreshore, but since this study aims to quantify the effect of breakwaters, foreshores are not mentioned anymore. Subsequently, the wave conditions at the revetment are determined in the load level module (Dutch: belastingmodule). This process is illustrated in Figure 2.6, where the three phases are marked by different colours. An Assessment and Design Toolkit (Dutch: Beoordelings- en Ontwerpinstrumentarium) is composed stating methods and models to be used for the assessment. This toolkit includes Hydra-NL, which is a probabilistic model that calculates the statistics of the hydraulic loads [18].

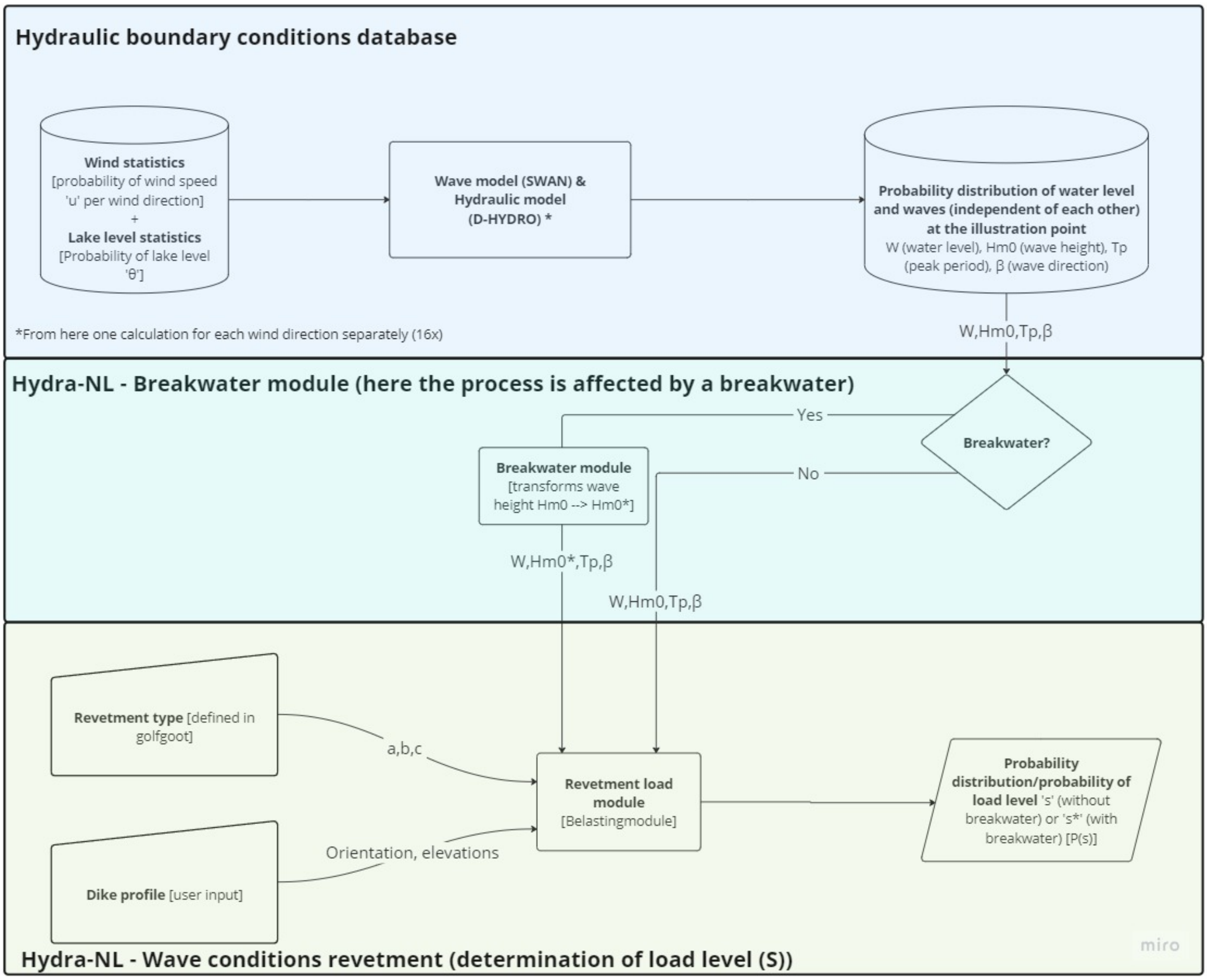


Figure 2.6: Flowchart water safety assessment.

Hydraulic Boundary Conditions database

The assessment starts with lake level statistics, specifying how often a certain lake level occurs. And wind statistics stating how often a wind speed occurs for each wind direction, where we distinct 16 wind directions. By means of a hydraulic model (D-HYDRO) for the lake level statistics and a wave model (SWAN) a probability distribution of lake level and waves (height, period, direction) at the illustration point is constructed. However, this lake level is not always equal to the actual water level at the illustration point. The lake level is the average water level over the entire lake. But due to wind a phenomena called skew (Dutch: scheefstand) occurs. The water level at a certain location is the lake level $+/-$ the skew in this point. The steepness of this skew, i.e. the difference between lowest and highest water level, depends on the wind speed and direction. Meaning that for each combination of lake level and wind speed a corresponding (ocal water level can be determined. The concept of skew is illustrated in Figure 2.7.

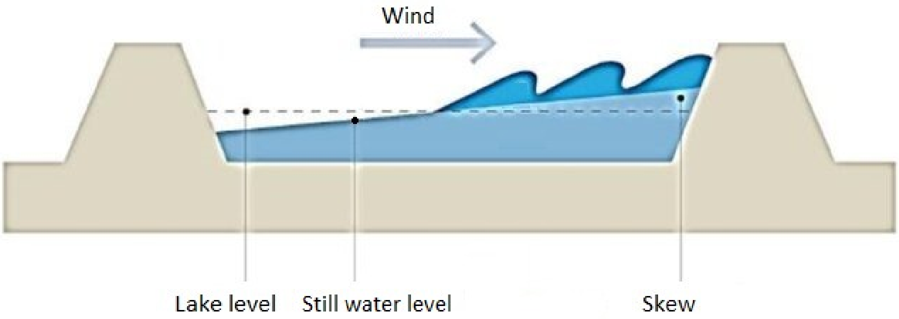


Figure 2.7: Illustration of lake level, water level, skew and wave run-up. Adapted from [5].

The water level and wave conditions at the illustration point are stored in the Hydraulic Boundary Conditions (Dutch: Hydraulische Randvoorwaarden) provided by Rijkswaterstaat. Models such as Hydra-NL allow the transformation of these conditions at the illustration point into load conditions as they act on the dike.

The water level and wave conditions are obtained as follows. For all 16 wind directions a diagram is created with the wind speed u in m/s on the x-axis and the lake level θ in m+NAP on the y-axis (see Figure 2.8). For each combination of wind speed and lake level the corresponding water level w, which is lake level + skew, and wave conditions (H_{m0}, T_p, α) are defined. The magnitude of the skew and the wave characteristics both depends on the wind speed. Since the probability distributions for lake level and wind speed are known and assumed to be independent of each other, the probability for each point in the diagram is determined by the product $p(u, \theta) = p(u) \times p(\theta)$. Since water level and wave conditions directly follow from the combination of lake level and wind speed the probability $p(u, \theta)$ is equal to the probability of water level and wave conditions $p(w, (H_{m0}, T_p, \alpha))$.

Hydra-NL breakwater module

So far we have determined probabilities of water levels and probabilities of wave conditions at the illustration point. Where the latter is done for each wind direction separately. In case there is no breakwater, these conditions are assumed to remain equal until affected by the dike itself. If a breakwater is implemented, these conditions do change. This change in conditions is determined using a primitive breakwater module, where it is assumed a breakwater only affects the wave height. Meaning the peak period and wave direction remain unchanged. Furthermore, Hydra-NL only allows varying the crest height. Meaning it is not possible to draw own profiles based on an expected remnant profile for example.

Hydra-NL load module

The load level S that corresponds to a return period is determined by the following process. From the database with hydraulic boundary conditions, the water level and wave conditions are defined, including their probability of occurrence. This load level depends on the wave characteristics only. Therefore the probability of a load level s is equal to the probability of the wave characteristics causing this load. Since each point represents a water level, and a load level, the probability of a single point is given by $p(w, s)$, see Equation 2.1. Figure 2.8 illustrates an example of such a diagram, this example might help in understanding the method used to determine the normative wave conditions and load level as it is explained below.

$$p(w, s) = p(w, (H_{m0}, T_p, \alpha)) = p(\theta, u) \tag{2.1}$$

The diagram is used to find a load level and corresponding normative wave conditions belonging to a certain return period and water level. Since each point contains a water level, one can draw a line through the points where this W is equal to a self-chosen value w. This line is called the isoline (Dutch: isolijn) $W = w$. Each point above the line corresponds to a combination of lake level and wind speed that causes water levels higher than W, each point underneath the line causes water levels lower than W. For the latter, it is assumed the corresponding load, denoted by capital S in Figure 2.8, is equal to zero. Now there is a selection in the data points based on water levels.

The value of S, denoted by s, depends on wave conditions only, as can be seen in Equation 3.1. If one would draw a vertical line, all points on this line would share an equal load level. All points on the left side, corresponding to lower wind speeds, would have a lower load level and vice versa all points on the right-hand side would have a higher load level.

Now a self-chosen return period is used to define the isoline $S = s$. The return period is transformed to an exceeding frequency $1/T$, denoted by capital P. This exceeding frequency corresponds to the size of the yellow plane in Figure 2.8, the smaller the exceeding frequency the smaller the plane. Since the plain is formed by both isolines $W = w$ and $S = s$, all points in the plane have a water level higher or equal to the chosen value of w, and a load level higher or equal to the value corresponding to the isoline $S = s$.

Both lower limits of the plane are now defined by the isolines, $W = w$ and $S = s$. The upper limit for both wind speed and lake level is defined by the probability distribution of both parameters. These distributions will eventually return a probability of zero when the values are increased. These values are the upper limit of the yellow plane.

The exceeding frequency is the summed probability of the probability of all points individually, located within the plane. Thus the smaller the plane, the smaller the sum of probabilities and the smaller the exceeding frequency. The isoline is drawn such that the sum of the probabilities of all points within the plane is equal to $P = 1/T$. The wind speed corresponding to this isoline is called the critical wind speed.

Now both isolines $W = w$ and $S = s$ are known, the load level and normative wave conditions corresponding to the return period can be determined. From all points on the boundary line, the border of the yellow plane, the one with the highest probability of occurrence is chosen, i.e. $p_{max}(u, \theta)$. This point is called the illustration point (Dutch: illustratiepunt). There are three different situations, as illustrated in Figure 2.8 and listed below. The water level and wave conditions corresponding to this illustration point are set as normative conditions.

1. IP 1: the illustration point is located on the intersection of isoline $S = s$ and $W = w$. The water level is equal to the specified water level and the load level is equal to the load level calculated for the specified exceeding frequency.

2. IP 2: the illustration point is on the isoline $W = w$, but above the critical wind speed. The water level is equal to the specified water level, but the load level is larger than the load level calculated for the specified exceeding frequency.

3. IP 3: the illustration point is located on the critical wind speed but above the isoline $W = w$. The water level is higher than the specified water level but the load level is equal to the

calculated load level for the specified exceeding frequency.

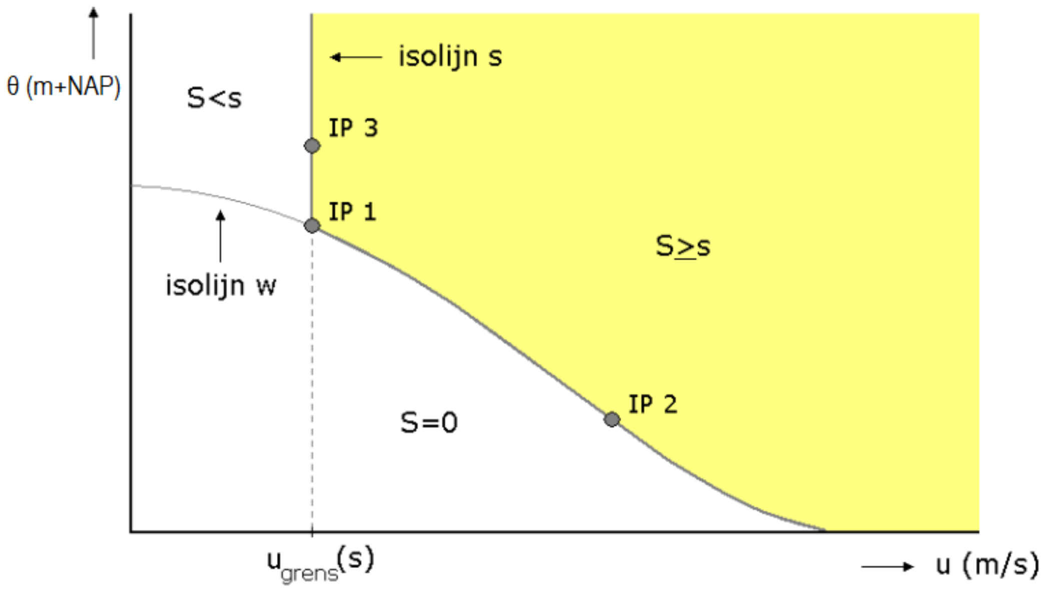


Figure 2.8: Example diagram containing isoline $W = w$ and $S = s$. All points on and above the isoline $W = w$ correspond to a water level equal to or greater than w. All points in the yellow plane correspond to a load level equal to or greater than s. This diagram includes one wind direction only.

In the above-explained process, only one wind direction is considered. In the actual assessment, there is a distinction between 16 different wind directions. The diagram in Figure 2.8 actually has a third dimension with the wind direction. The isoline $W = w$ is plotted for all these wind directions separately, meaning there are 16 unique lines. Because of this extra dimension, the yellow plane above the isoline becomes a 3D-object. The location of the isoline $S = s$ is now defined by the volume of this object, the sum of all probabilities within this object must be equal to the exceeding frequency $1/T$. Since an extra dimension is added, the probability of a single point in is not defined by $p(u, \theta)$, but the probability of this third parameter, wind direction ϕ, must be included resulting in a probability $p(u, \theta, \phi)$.

The normative conditions are not chosen from the boundary line but from a boundary surface. This surface is the boundary line as in Figure 2.8 but then extended over all 16 wind directions. All points on this surface contain a wind speed u, lake level θ, and wind direction ϕ. The probability of occurrence is denoted as $p(u, \theta, \phi)$ which is found by Equation 2.2.

$$p(s) = p(u, \theta, \phi) = p(u) \times p(\theta) \times p(\phi) \tag{2.2}$$

Chapter 3

Wave load assessment

To assess the wave load on a dike a parameter is needed representing this load. In order to find and justify the choice for such a parameter it is important to understand how a wave behaves and how it causes load on a dike. In this chapter, the behaviour of waves and the most relevant characteristics of waves will be discussed briefly. Followed by the choice for the most suitable parameter to quantify wave load reduction on a dike.

3.1 Wave characteristics

While looking out to the sea, it is clear that waves on the surface of a water body are not sinusoidal. The surface appears to be composed of random waves of various lengths and periods. Quantifying this surface is complex and requires simplifications. These simplifications lead to the concept of a wave spectrum. The spectrum gives the distribution of wave energy among different wave frequencies or wavelengths on the sea surface [19]. The characteristics of this spectrum define the magnitude to which wave run-down and wave impact occurs.

For the assessment of dikes the most relevant parameters of a spectrum are the significant wave height at the toe of the dike in meters, the peak wave period in seconds, and the angle of incidence in degrees [20]. Wave height refers to the overall vertical change in height between the wave crest (or peak) and the wave trough [21], and is expressed as significant wave height H_{m0}. The significant wave height is the average wave height of the highest 1/3 part of the waves [22]. The peak wave period T_p refers to the time interval between two successive peaks of the spectrum passing at a fixed point [21]. In the case of wave load on a dike, it is the time between two wave attacks on the dike. The angle of incidence $\angle\alpha$ is expressed in degrees relative to the perpendicular of the dike. The wave direction is given in degrees relative to the wind direction north (see Figure 3.1) [6].

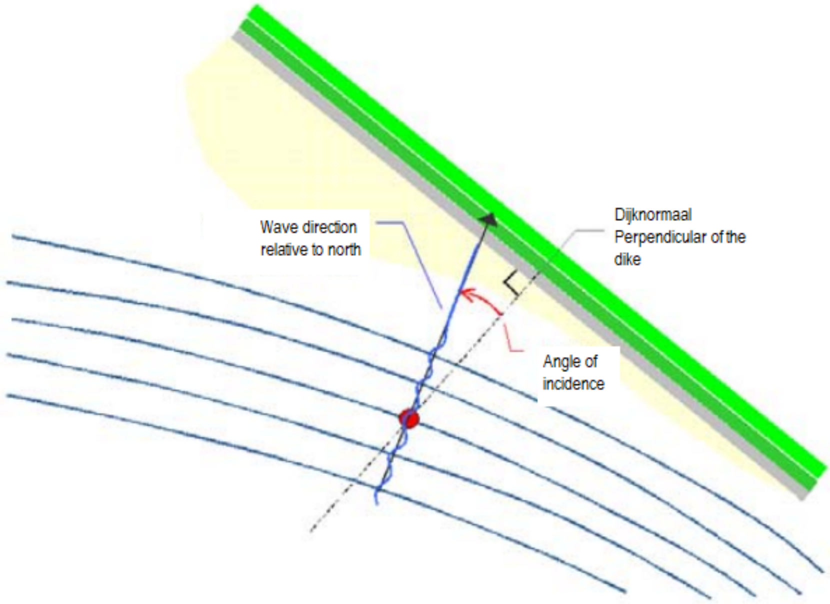


Figure 3.1: Definition of angle of incidence [6].

3.2 Wave load parameter

Initially, it was supposed the wave height would be a suitable parameter to measure the wave attenuation effect of breakwaters. Based on the assumption that there is a positive relationship between wave height and wave load on the dike. However, during the first calculations, an issue occurred following this assumption. It was found that increasing the breakwater height in particular cases would result in an increased wave height as well. Meaning according to this analysis and the assumption that wave height is a proper and direct measure of wave load, a lower breakwater can be more beneficial than a higher one. While clarifying these results it was found that there is a load level S based on significant wave height, peak period, and angle of incidence. Meaning it represents all three wave spectrum characteristics mentioned in Section 3.1.

Furthermore, as discussed in Section 2.1, load on a dike does not only depend on wave characteristics but also on the ability of the revetment to handle these waves. This ability depends on the characteristics of the stones and the way they form a settlement. As a consequence, different revetment types respond differently to the same waves. The two elements, wave characteristics and revetment type, together form the load level S. Since the load depends on the revetment, the load level can vary for different revetments even when exposed to exactly the same waves. Equation 3.1 provides the formula used to convert the wave conditions and revetment characteristics into a load level.

$$s = H_{m0}^{a} \times T_{p}^{b} \times (\cos \angle\alpha)^{c} \tag{3.1}$$

The parameters H_{m0}, Tp, and α represent the load acting on the dike. The coefficients a, b, and c define the relevance of that parameter towards the load experienced on the dike and they can vary per revetment. This variation implies that the wave characteristics, including significant wave height (Hm0), peak period (Tp), and angle of incidence (α), are considered differently. Increasing coefficient 'a' would amplify the influence of wave height on the load parameter. Adjusting coefficients 'b' and 'c' would affect the importance of peak period (Tp) and angle of incidence (α) respectively.

For concrete pillars (Dutch: betonzuilen) the values for a, b, and c are 1.0, 0.4, and 0.8 respectively [23]. Indicating significant wave height (Hm0) is considered as most important wave characteristic for pillars.

Chapter 4

Quantification of wave reduction

Chapter 2 explained the theory behind flood safety assessment in the Netherlands, and briefly mentioned where in this process a breakwater interferes. This chapter elaborates on the method used to quantify this interference. The purpose is to quantify the effect of breakwaters by means of a load relative to the load without a breakwater.

4.1 Dike profile

To study the wave attenuating effect of breakwaters a study case is required. Since wave load varies per location, the absolute wave load reduction of breakwaters is likely to vary per location as well. This is mainly caused by the orientation of the dike. For example, if hard western wind occurs more often and causes bigger storms than wind from other directions. A dike facing west will suffer heavier loads than a dike facing north-west since waves will attack the latter under an acute angle which causes less load on the dike than waves coming perpendicular at the dike. However, the wave attenuation effect of breakwaters is the same for each wind direction. The breaking of a wave is a physical process that does not vary for different locations. If it is found that wave load at one specific location significantly decreases due to the presence of a breakwater, this is likely to be the case at other locations as well.

As study case is chosen for cross-sectional profile 11.70, part of the Ketelmeerdijk. Due to restrictions in Hydra-NL regarded minimum and maximum allowed slope, the implemented profile slightly differs from the profile as documented. Both profiles can be found in Figure 4.1, where the adapted sections are given the colour blue and green. Table 4.1 contains the heights and distances for each slope section from right (lower elevations) to left (higher elevations) in the figure. The height is relative to NAP, the distance is measured from the core of the dike which is the outer left part in the figure.

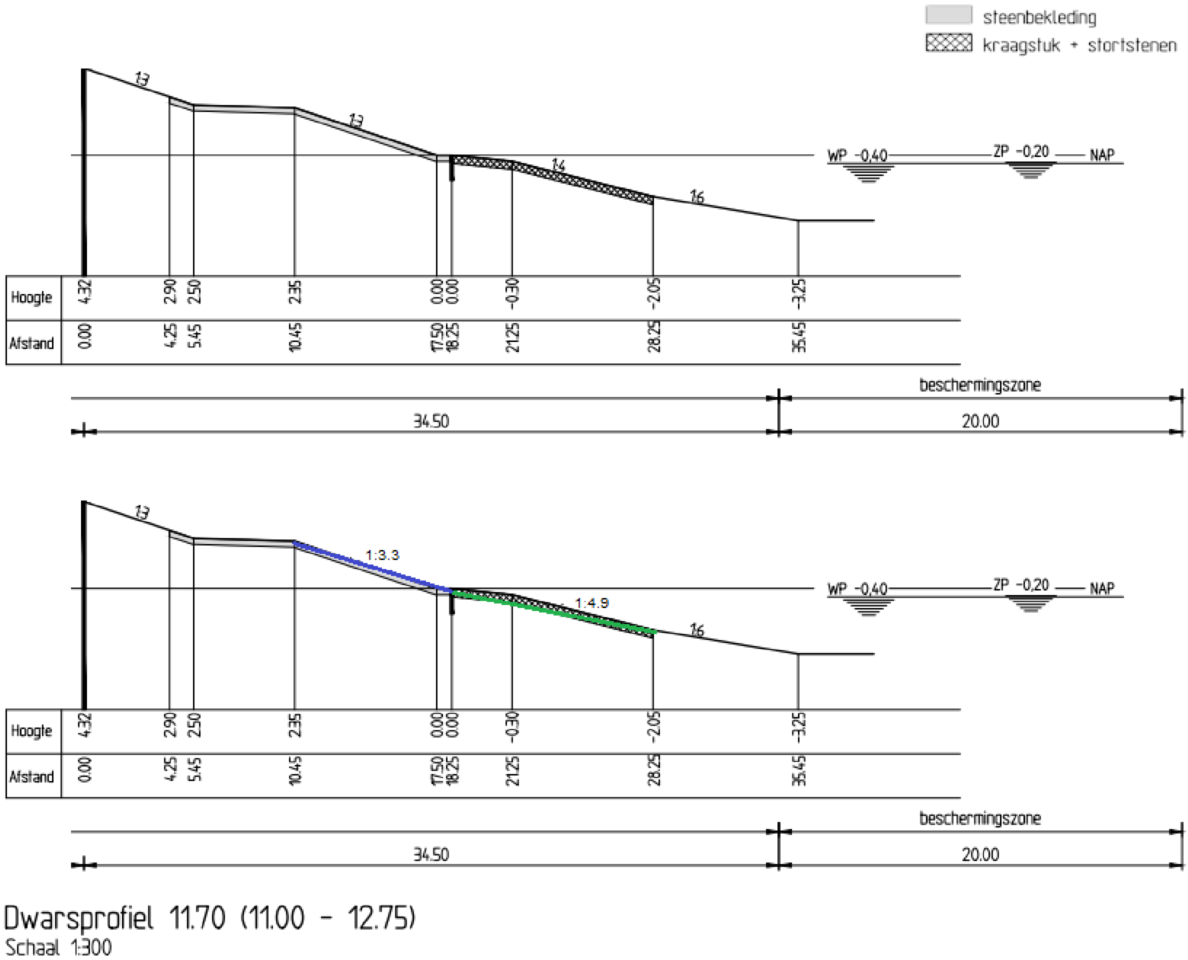


Figure 4.1: Actual and implemented cross-section of the outer embankment of dike profile 11.70 respectively. Heights relative to NAP, distances relative to the dikes core. Translation: Steenbekleding = Stone revetment, Kraagstuk + stortstenen = Toe section + stone armor, Dwarsprofiel = Cross Section, Hoogte = Height, Afstand = Distance, WP (Winter Peil) = Winter Level, ZP (Zomer Peil) = Summer Level, Beschermingszone = Protection zone

Table 4.1: Distance/height overview of implemented outer slope DP11.70.

From		**To**		
Distance [m]	**Height [m+NAP]**	**Distance [m]**	**Height [m+NAP]**	**Slope [1:..]**
-35.45	-3.25	-28.25	-2.05	6.00
-28.25	-2.05	-18.25	0.00	4.90
-18.25	0.00	-10.45	2.35	3.30
-10.45	2.35	-5.45	2.50	33.3
-5.45	2.50	-4.25	2.90	3.00
-4.25	2.90	0.00	4.32	3.00

4.2 Return periods

Initially six different return periods are considered to illustrate the effect of breakwaters on wave load. These are 100y, 300y, 1,000y, 3,000y, 10,000y and 30,000y. The reason all six periods were considered is to learn to what extent wave reduction varies for different circumstances. However, not all return periods are relevant for the assessment of a dike. For instance, knowing exactly the wave attenuation of a breakwater for a return period of 100 years is not that useful since the dikes are built for far greater periods. There are two norms for dikes surrounding the Flevopolder. The Randmeerdijken, on the eastern and southern border of Flevoland, have a lower limit (Dutch: grenswaarde) of 1,000 years and an alert level (Dutch: signaleringswaarde) of 3,000 years. The IJsselmeer-, Markermeer- and Ketelmeerdijken both have a lower limit of 10,000 years and an alert level of 30,000 years. Since there are hardly any breakwaters located at the Randmeerdijken, and the wave load on these dikes is significantly lower the focus will be on the return period of 10,000 year

4.3 Breakwater heights

To map the wave attenuating effect of breakwaters it is necessary to quantify wave load for a range of breakwater heights and see how it varies. Since breakwaters are not the primary flood defence structure and only provide additional protection they are significantly lower than the dike. Therefore it is only relevant to include breakwaters up to a crest height that is practically feasible. Most breakwaters controlled by Zuiderzeeland have a height of around 2m+NAP. For this study, a maximum height of 3m+NAP is considered. Since this study has the intention to learn about the effect of wave attenuation by breakwaters after failure, it is chosen to include breakwater heights until the bottom of the lake. The elevation of the bottom is determined by implementing breakwaters with extremely low crest heights which were compared with the simulation where no breakwater was implemented. Doing so it was found that all elevations of -5m+NAP and below result in the exact same wave load and wave conditions at the dike as the simulation without a breakwater (see Appendix A). From now on, the wave load corresponding to a breakwater height of -5m+NAP is taken as the reference situation, i.e. the wave load in case no breakwater is present.

The range of heights thus spans from a minimum of -5m+NAP until a maximum of 3m+NAP. To map the behaviour of the load level between these extremes the height is increased by steps of 0.5 meters, meaning 17 different breakwater profiles are evaluated. As mentioned in Section 2.3, Hydra-NL only allows to vary the crest height. Implying all profiles are equal in terms of shape and slope.

4.4 Water levels

Since this study focuses on the effect of wave load on stone revetment, the slope section covered with this revetment type will be considered only. Since stone covering is often located at the lower part of the dike close to the water surface, this is the most interesting part to analyse wave load. At dike profile 11.70 the stone covering is located between 0.0 m+NAP and 2.9 m+NAP. Therefore water levels starting at 0.0m+NAP, increasing with steps of 0.25 meters, up to 2.9 m+NAP will be analysed. The water level given as input to Hydra-NL is the water level at the illustration point, meaning it includes the skew effect.

4.5 Load reduction

Sections, 1.1, 2.3 and 3.2 respectively explain the norms, expressed as return period T, used to assess flood defence structures in the Netherlands. How these norms are transformed to an exceeding frequency $1/T$ and the relation between this exceeding frequency and the normative hydraulic conditions.

These hydraulic conditions consist of a lake level and waves. The lake level depends on the ratio of influx vs outflux of water in the lake, and wave characteristics result directly from wind speed and direction. For both the lake level and wind causing the waves there is a probability distribution based on measurements from the past. While assessing a dike for one specific return period, the probability of the hydraulic load is fixed. Meaning if the probability for a lake level becomes much smaller, the probability for waves has to become higher. Practically this means a high lake level brings along smaller waves than a lower lake level when the return period is fixed.

I.e. a high lake level brings along smaller waves than a lower lake level when the return period is fixed. Because of this probabilistic approach, not all lake levels are assessed for each return period. A small return period corresponds to a relatively high probability, meaning that extremely high lake levels are not likely to be reached. This phenomenon is illustrated in Figure 4.2 using different shapes of equal size.

The surface area of a shape represents the exceeding probability. The larger the area, the smaller the exceeding probability. Since the exceeding probability is found by 1/T, a larger area corresponds to a bigger return period. From now on, the size of this area will be referred to as probability space (Dutch: kansruimte). Once a return period is chosen for which the normative conditions will be determined, this probability space is fixed. The probability space is characterized by the probability for wind speed $p(u)$ and lake level $p(\theta)$. The product of these two will always be equal to 1/T, but their individual values can vary. Say there is an extremely high wind speed, there is only little probability space left for lake level, as illustrated in the center shape in Figure 4.2. The left shape, the square, illustrates a situation where the probability of lake level is equal to the probability of wind speed, indicating both moderate wind speed and lake level. The outer right shape indicates a situation with an extremely high lake level, and thus a relatively low wind speed.

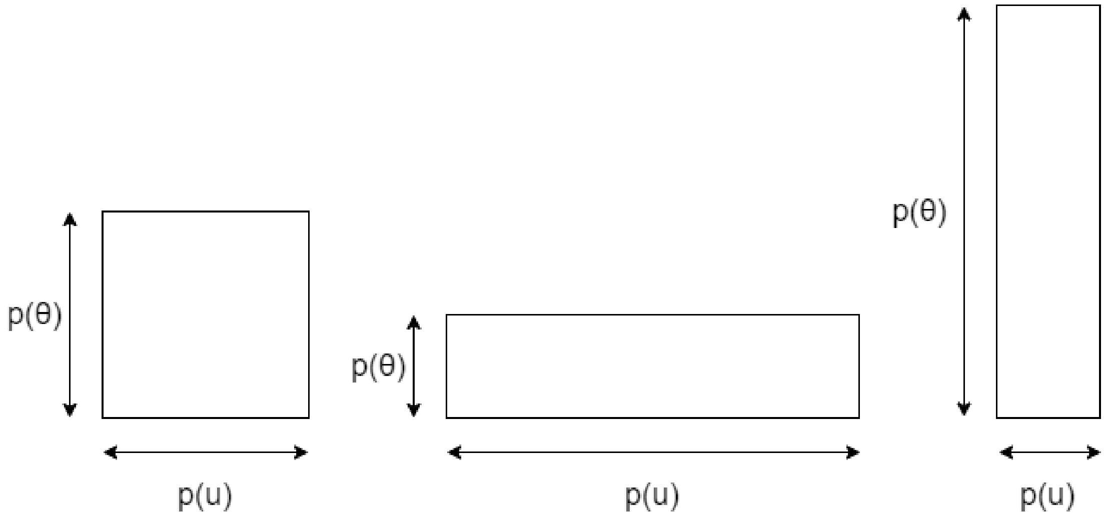


Figure 4.2: Visualisation of probability space. The size of the probability space is defined by the return period, how this space is split between wind speed and lake level can vary.

Figure 4.3 illustrates the load level for a return period of 10,000 years. This return period corresponds to an exceeding probability of 1/10,000. The x-axis contains the dam height, and the y-axis the load level. The higher the load level, the more load the revetment experiences. The plot contains two colours, each colour represents a different still water level (swl). I.e. each point represents the load level, with an exceedance probability of 1/10,000, experienced by the revetment for one specific water level and one specific dam height.

The blue and orange dots correspond to a still water level of 1.0m+NAP and 2.75m+NAP respectively. It can be seen that the load level line through the orange dots stagnates for lower breakwater heights. The water level of these orange dots is rarely exceeded within a period of 10,000 years, therefore the exceeding frequency is relatively small. To maintain the combined exceeding probability of 1/10,000, the exceeding frequency for wind speed must be relatively big. This higher exceeding frequency results in lower wind speeds and thus smaller waves. This corresponds to the most right shape in Figure 4.2, where the lake level seizes a big proportion of the probability space and there is only little left for wind speed.

This explains why extremely high water levels bring along relatively low normative wave conditions. But does not completely declare why this causes the load level line to deflect for those high water levels only. To understand this deflection the breaking of waves is relevant.

A wave breaks because of friction with the bottom of the lake [24], or in this case due to interaction with the breakwater. A smaller wave is less affected than a larger wave for the same water depth. Therefore, when the breakwater is nearly as low as the lake bottom, a small wave will not be affected while a large wave still got attenuated. In the figure, it can be seen that for all breakwater heights below -2m+NAP there is hardly any difference in wave load. I.e. for the load on the dike, it does not matter if the breakwater is -2m+NAP or if there is no breakwater at all because the small waves that come with this high water level only start to attenuate at breakwater heights of around -2m+NAP higher. On the contrary, the blue line representing a water level of 1.0m+NAP, which occurs more frequently, brings along larger waves. These higher waves are impacted by a breakwater much sooner than a small wave, and thus there still is a difference in load level corresponding to a breakwater height of e.g. -4m+NAP and -5m+NAP.

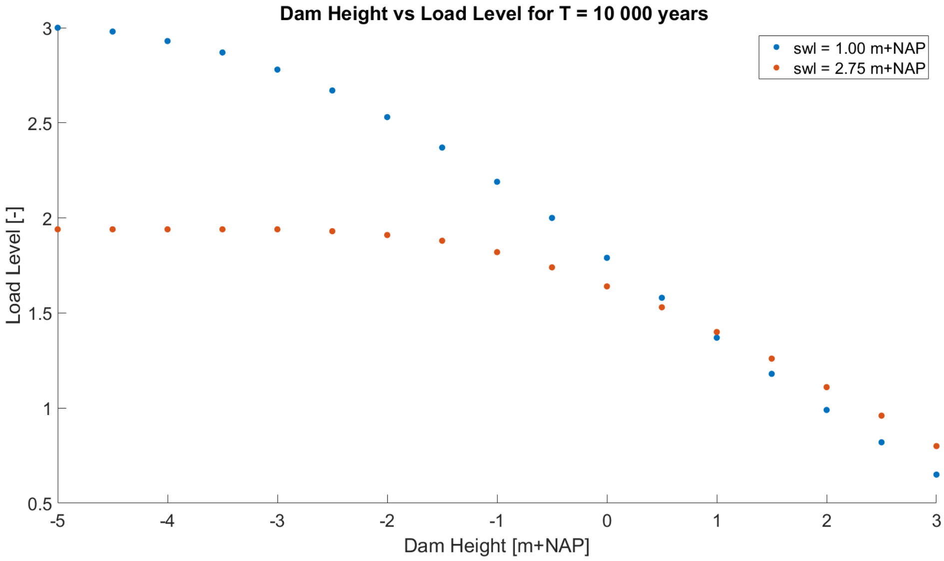


Figure 4.3: Load level for a return period of 10,000 years, swl = still water level. Deflection towards lower dam heights for swl = 2.75 m+NAP is caused by the limitation in probability space.

4.6 Reduction factor

The reduction in load level is given as relative load. The relative load level represents the percentage of load compared to the scenario without a breakwater. This relative load is found by dividing each load by the maximum load for the corresponding water level. A relative load of 100 percent means there is no reduction, i.e. the wave load is equal to the situation without a breakwater. A relative load of 0 percent means there is no load on the dike.

Figure 4.4 depicts the relationship between the relative load level and the breakwater height for a return period of 10,000 years. The relative load [%] is plotted on the y-axis against the dam height [m+NAP] on the x-axis. The different colours represent different water levels. Each data point in the figure corresponds to the relative load level for a particular breakwater height and water level, all pertaining to a 10,000-year return period. To enhance clarity, four still water levels (swl's) are included in the graph in Figure 4.4. For a comprehensive view including all analyzed water levels, please refer to Appendix B.2.

The graph exhibits a 3-degree polynomial shape, indicating a non-linear relationship between the relative load level and breakwater height. Furthermore, the observations reveal that for lower water levels, the reduction in load level is more significant. This is likely due to the decreased submergence depth causing the wave to interact with the dam earlier. The submergence depth is the distance between the dam and the water surface. The highest water level, of the analysed water levels, that occurs within a return period of 10 000 years is 2.75 m+NAP. In Figure 4.4 one can see the relationship between relative load and dam height does not follow the same pattern as for the other water levels. This is due to the limitation in probability space, as explained in Section 4.5. Where it is illustrated by a plot of the load level against the dam height (Figure 4.3). Because Figure 4.4 displays the relative loads rather than the absolute load levels, the deflection takes a different form.

These plots allow to determine quite accurately the effectiveness of breakwaters in very specific situations. The water level and dam heights should be known to indicate what the reduction load might be. To draw a more general conclusion one could analyse the behaviour of the relative load per submergence depth, which is done in Section 4.7.

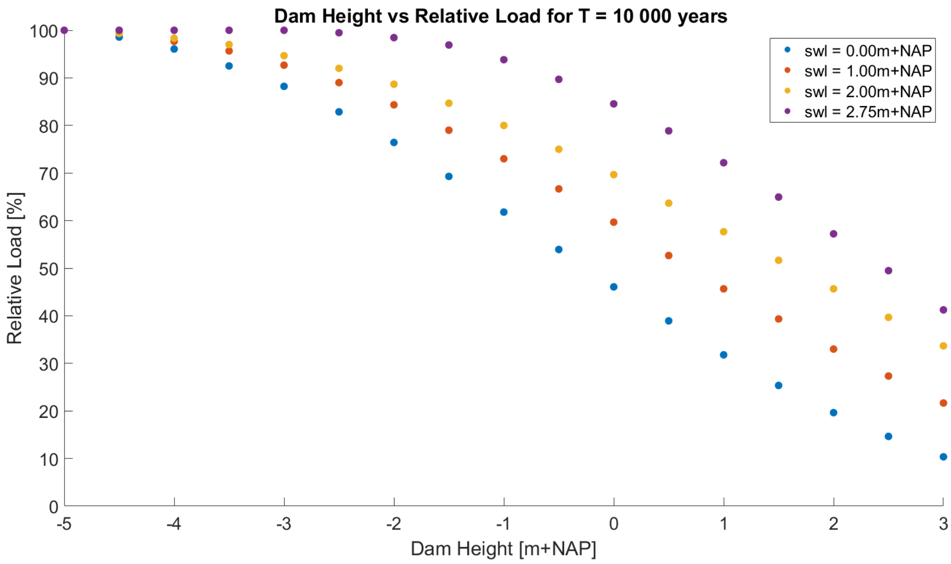


Figure 4.4: Relative load per still water level for a return period of 10,000 years.

4.7 Submergence depth

So far the effectiveness of breakwaters is expressed as a relative load per still water level for various dam heights. Meaning each point in the discussed graphs illustrates the reduction in load level provided by a specific breakwater height for a specific water level. To gain more insight into the effectiveness of breakwaters, this water level is replaced by the submergence depth. The submergence depth H_{sub} is the distance between the crest height of the breakwater and the still water level in meters, as illustrated in Figure 4.5. Note that a negative submergence depth indicates the breakwater reaches above the water level.

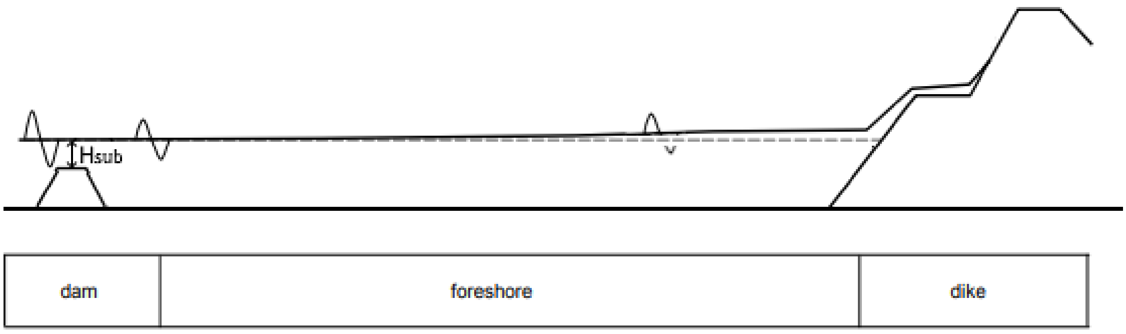


Figure 4.5: Illustration of submergence depth (H_{sub}) of a breakwater

In Figure 4.6 the relative load level is plotted against the dam height per submergence depth. Meaning each point of the same colour has different water levels, but the distance between crest height and still water level remains equal. Table 4.2 provides the relative load-interval per submergence depth. This interval contains all relative loads that are calculated for the corresponding submergence depth.

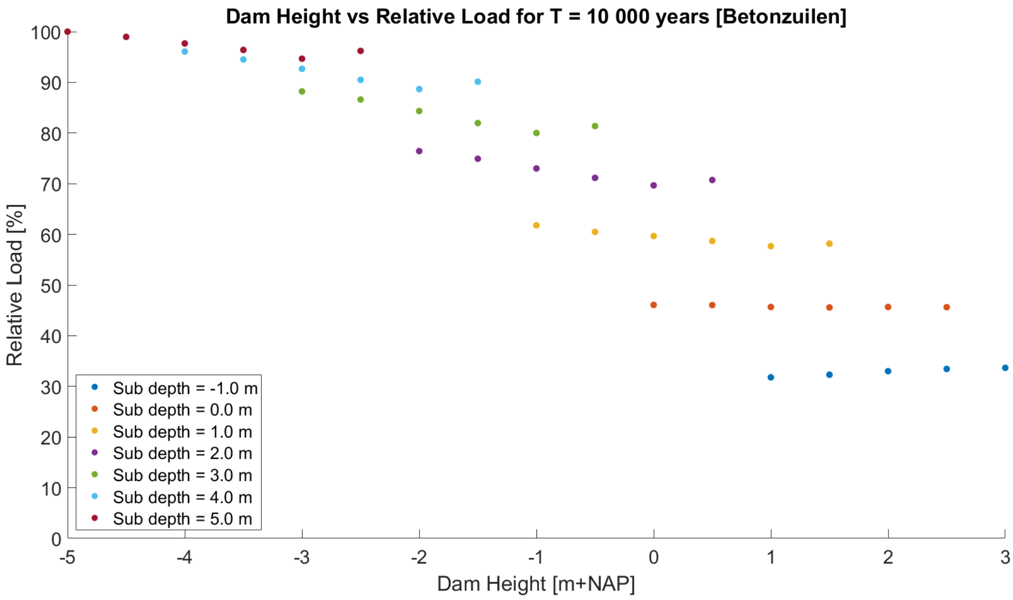


Figure 4.6: Relative load per submergence depth for a return period of 10,000 years.

Table 4.2: Relative load intervals per submergence depth for a return period of 10,000 years.

Submergence depth [m]	**Relative load-interval [%]**
5	95-100
4	89-96
3	80-88
2	70-76
1	58-62
0	45-46
-1	32-34

4.8 Application of the results

The results discussed so far indicate that breakwaters significantly reduce the wave load, even when submerged. This is relevant in the strength assessment of the stone revetment and might eventually be relevant in the assessment of the dike. Because failure of the revetment ultimately results in failure of the dike.

This research on its own might not be sufficient to be used in dike assessments. But it does provide reason for further research to quantify failure profiles of breakwaters and their effectiveness. This knowledge would broaden the possibilities in dike strengthening projects, for instance when expansion of the dike itself is not possible. In Figure 1.3 and Table 1.1 an overview was provided of breakwaters in Flevoland. The breakwaters in Lemmer, Schokkerhaven, Ketelhaven, Lelystad, and Oostervaardersdiep are not included in the assessment because they will fail before normative conditions of the corresponding flood defence structure are reached. An increased understanding of the failure of these breakwaters and the load reduction of the remnant profiles would allow to include them in the assessment of the primary flood defence structure preventing unnecessary strengthening.

Chapter 5
Discussion

Throughout this study, various simplifications and assumptions were made, particularly regarding the chosen methodology. While some of these simplifications may have potential implications for the results obtained, others are less likely to significantly impact the findings. In this chapter, the most crucial aspects of discussion will be dealt with, addressing the key points that require further examination and consideration.

The first point of discussion worth mentioning is the profile of breakwaters used in the analysis. An important assumption made in this research is that all breakwaters have the same shape. However, when a breakwater fails, it is likely to have a different shape compared to an intact breakwater. This aspect was not considered in this study, and it may have implications for assessing the effectiveness of failed breakwaters in wave load reduction. While this study did not specifically address the impact of failed breakwater shapes, it does provide important insights into the overall influence of submerged objects on wave loads. The findings demonstrate that the presence of an object underneath the water surface significantly affects the wave load on the dike. Considering this, it is reasonable to assume that even a failed breakwater, with a different shape from intact breakwaters, will still have a noteworthy effect on wave behavior. The remnants of a failed breakwater, despite their altered shape, are likely to cause wave energy dissipation and thus reduce the load on the dike.

The second point of discussion is the revetment type analysed in this study. The values of parameters a, b, and c, used to determine load level s, vary for different types of revetments. This variation implies that the wave characteristics, significant wave height H_{m0}, peak period T_p, and angle of incidence (α), are considered differently. Increasing coefficient 'a' would amplify the influence of wave height(H_{m0}) on the load parameter s, potentially leading to a larger load parameter (s). Adjusting coefficients 'b' and 'c' would affect the importance of peak period (T_p) and angle of incidence (α) respectively. When analyzing the wave reduction effects of breakwaters, it is important to note that Hydra-NL assumes breakwaters impact the significant wave height (H_{m0}) only. If the load parameter (s) is predominantly determined by parameters T_p and α, the reduction achieved through the implementation of a breakwater may be smaller compared to situations where the coefficient 'a' has a relatively higher value, indicating a stronger influence of wave height. Vice versa, if load is primarily defined by the significant wave height, introducing a breakwater will cause relative more load reduction. It should be noted that the findings of this study are specific to the analyzed revetment type, namely concrete pillars (Dutch: betonzuilen), and caution should be exercised when extrapolating the results to other revetment types, such as stone blocks or grass. Each revetment type may exhibit different responses to wave conditions due to their distinct structural properties and behaviors. The exact same method can be applied for different revetments, the only thing that changes in such an analysis are the values of parameters a,b,c in the formula for s.

This issue related to possible variations for different revetment types is partly caused by model assumptions regarded the effect of breakwaters on waves. The model utilized in this research incorporates a simplistic breakwater module that assumes breakwaters solely impact wave height, neglecting any influence on peak period and angle of incidence. The effect of the breakwater on both neglected wave characteristics might be less than it is on the significant wave height, but assuming there is no effect at all might be too simplistic and a more comprehensive study would be necessary when applying these results to dike strengthening projects.

The analysis is done at the Ketelmeerdijk. The relevance of the study's location in the obtained results is an important consideration. Conducting the study at a specific location raises questions about the generalizability of the results to other locations, such as different lakes or dikes with different orientations.

While it is true that the fundamental physics of wave breaking, and the interaction between waves and breakwaters remain consistent across different locations, the specific wave conditions and characteristics experienced at each location can vary significantly. This variation may have implications for the range of wave conditions evaluated in the study.

For instance, if the study was conducted in a relatively calm lake, it is likely that only smaller wave conditions were considered, leading to conclusions that are accurate for those specific wave conditions and storms. However, the applicability of these findings to stronger storms with larger waves may be less certain, as the study might not have examined or captured the behavior of such extreme wave conditions.

Chapter 6

Conclusion & Recommendations

6.1 Conclusion

Two research questions have been composed and answered in this report. The first question aimed to identify the most suitable parameter to observe the effect of a breakwater on wave load reduction.

It was found that significant wave height is not a suitable parameter to analyse the effectiveness of breakwaters. The magnitude of failure mechanisms depends on the wave characteristics of a wave spectrum, namely significant wave height (H_{m0}), peak period (T_p), and angle of incidence (α). And the ability of the revetment to deal with these specific waves. The latter is defined by the properties of the revetment themselves and the way they are put in place. This means that for equal waves but different types of revetment, or even different types of stone revetments, a different load can be experienced. A proper parameter is the load level (s), which is defined by the product of significant wave height, peak period, and angle of incidence (Equation 3.1). Where each factor has an exponent which can vary from 0 to 1. The value depends on the type of revetment and indicates how sensitive the revetment is for that specific wave characteristic.

The second question aims to quantify the relationship between breakwater height and its effect on the wave load. To answer this question the load reducing effect of breakwaters was expressed as relative load, both per water level and per submergence depth. The former graphs revealed a 3-degree polynomial shape, indicating a non-linear relationship between the relative load level and breakwater height. Furthermore, the observations revealed that for lower water levels, the reduction in load level is more significant (Figure 4.4). The latter gave a good view on the extent to which breakwaters significantly reduce wave load (Table 4.2). This ranges from a reduction in load level of approximately 55% for breakwaters with crest heights equal to the water level to a 24-30% percent reduction for 3-meter submerged breakwaters.

6.2 Recommendations

Based on the findings of this study, it is strongly recommended to incorporate breakwaters into the safety assessment of flood defence structures. The significant wave load reduction achieved by breakwaters, even when submerged, highlights their potential to enhance the performance of flood defence structures. However, before implementing these findings in real-world applications, further research is essential to address key knowledge gaps and uncertainties.

One crucial area for future research is the understanding of breakwater failure mechanisms and their remnant profiles. Investigating how breakwaters fail, the extent to which they erode, and the resulting profiles are vital factors to consider. Mark Klein Breteler, coastal structures specialist at Deltares, provided expectations regarded the extent to which a breakwater would erode. He mentioned a dam erodes roughly until 1 $\times$ the significant wave height (Hm0) underneath the water level, at the side of wave attack. This would mean for a water level of 2.0m+NAP and a significant wave height of 2.0m, the front side of the dam would erode until approximately 0.0m+NAP [25]. However, to comprehensively assess the contribution of failed breakwaters to wave load reduction, future research should consider the influence of breakwater shape on wave attenuation. This would involve studying the impact of varying breakwater shapes on wave energy dissipation in order to provide a more accurate assessment of their effectiveness after failure. Furthermore, understanding the erosion rates and the longevity of breakwaters during storm events is important, particularly in regions where storms have a relatively short duration.

Additionally, it is recommended to conduct further research to explore the behavior and performance of different revetment types beyond concrete pillars. This method can relatively easy be repeated for various revetment types, such as stone blocks or grass. This would contribute to a more comprehensive view and allow to apply the results in a broader context.

Furthermore, investigating the interaction between breakwaters and wave characteristics beyond wave height, such as peak period and angle of incidence, is essential for a more accurate assessment of their overall impact on wave load reduction. Incorporating these factors into future research will provide a more comprehensive understanding of breakwater performance.

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Appendix A

Breakwater Heights

This appendix provides the load levels for extreme low breakwater heights, where an height of NaN in the tables corresponds to the scenario without a breakwater. It can be seen that from crest heights of approximately -5m+NAP the load level remains equal.

Table A.1: Breakwater height and load levels for T = 1,000 year

Breakwater height [m+NAP]	Load level [-] swl = 0 m+NAP	Load level [-] swl = 1 m+NAP	Load level [-] swl = 1.5 m+NAP
-4	2.24	2.38	2.34
-5	2.29	2.41	2.36
-6	2.29	2.41	2.36
-7	2.29	2.41	2.36
-8	2.29	2.41	2.36
NaN	2.29	2.41	2.36

Table A.2: Breakwater height and load levels for T = 3,000 year

Breakwater height [m+NAP]	Load level [-] swl = 0 m+NAP	Load level [-] swl = 1 m+NAP	Load level [-] swl = 1.5 m+NAP
-4	2.45	2.38	2.67
-5	2.53	2.7	2.7
-6	2.55	2.7	2.71
-7	2.55	2.7	2.71
-8	2.55	2.7	2.71
NaN	2.55	2.7	2.71

Table A.3: Breakwater height and load levels for T = 10,000 year

Breakwater height [m+NAP]	Load level [-] swl = 0 m+NAP	Load level [-] swl = 1 m+NAP	Load level [-] swl = 2 m+NAP
-4	2.69	2.93	2.95
-5	2.8	3	3
-6	2.84	3.02	3
-7	2.84	3.02	3
-8	2.84	3.02	3
No dam	2.84	3.02	3

Table A.4: Breakwater height and load levels for T = 30,000 year

Breakwater height [m+NAP]	Load level [-] swl = 0 m+NAP	Load level [-] swl = 1 m+NAP	Load level [-] swl = 2 m+NAP
-4	2.9	3.18	3.28
-5	3.05	3.28	3.35
-6	3.11	3.31	3.37
-7	3.12	3.32	3.37
-8	3.12	3.32	3.37
No dam	3.12	3.32	3.37

Appendix B
Results

This appendix contains the results for all six analysed return periods.

B.1 Absolute load level

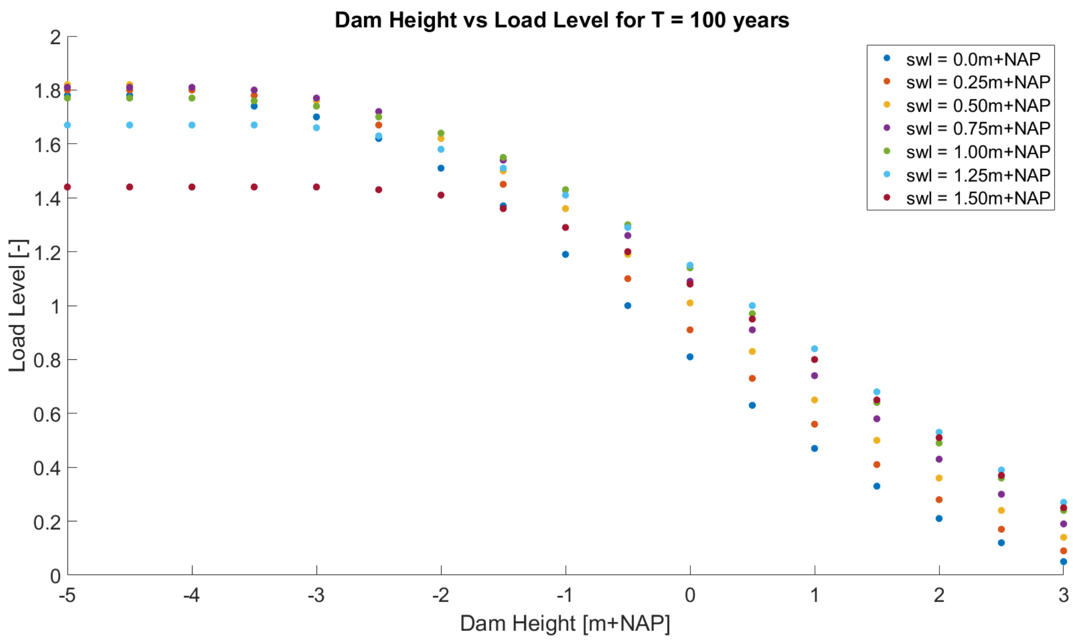

Figure B.1: Absolute load level for T = 100 year

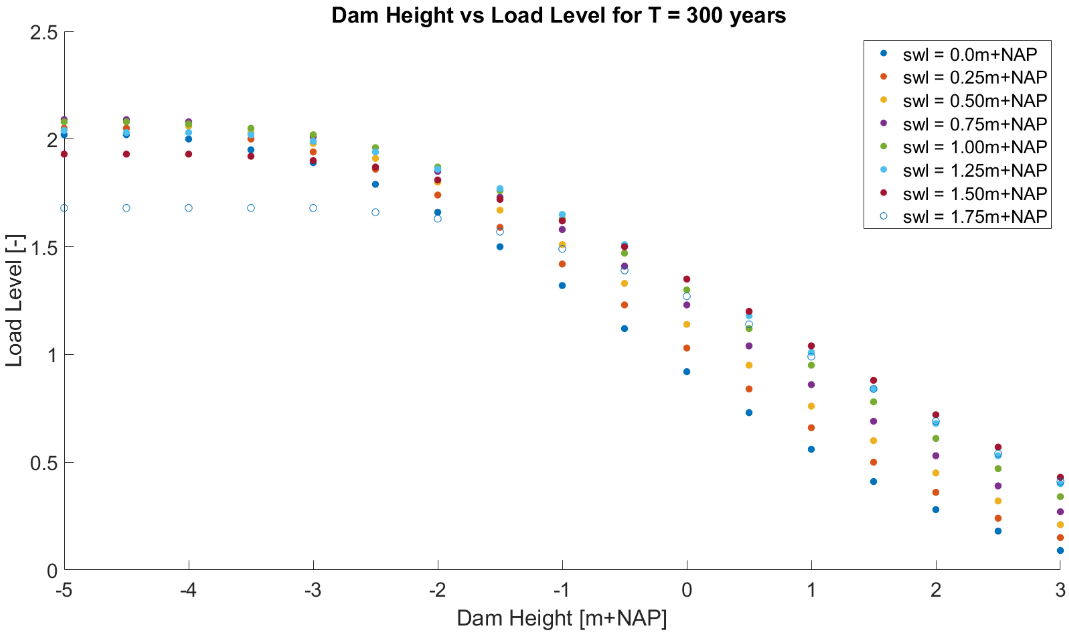

Figure B.2: Absolute load level for T = 300 year

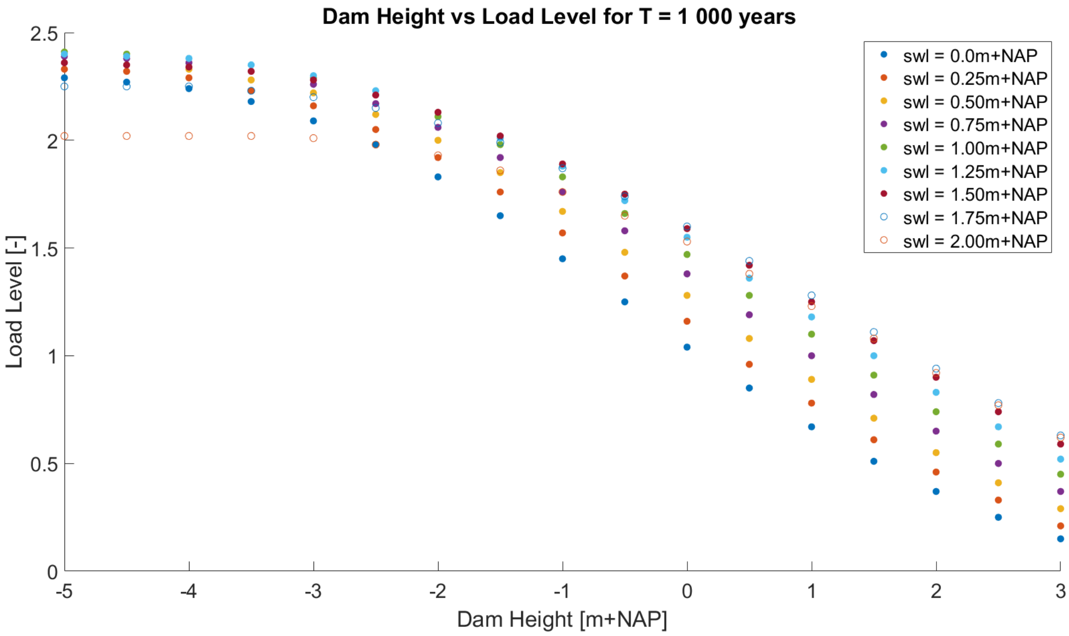

Figure B.3: Absolute load level for T = 1,000 year

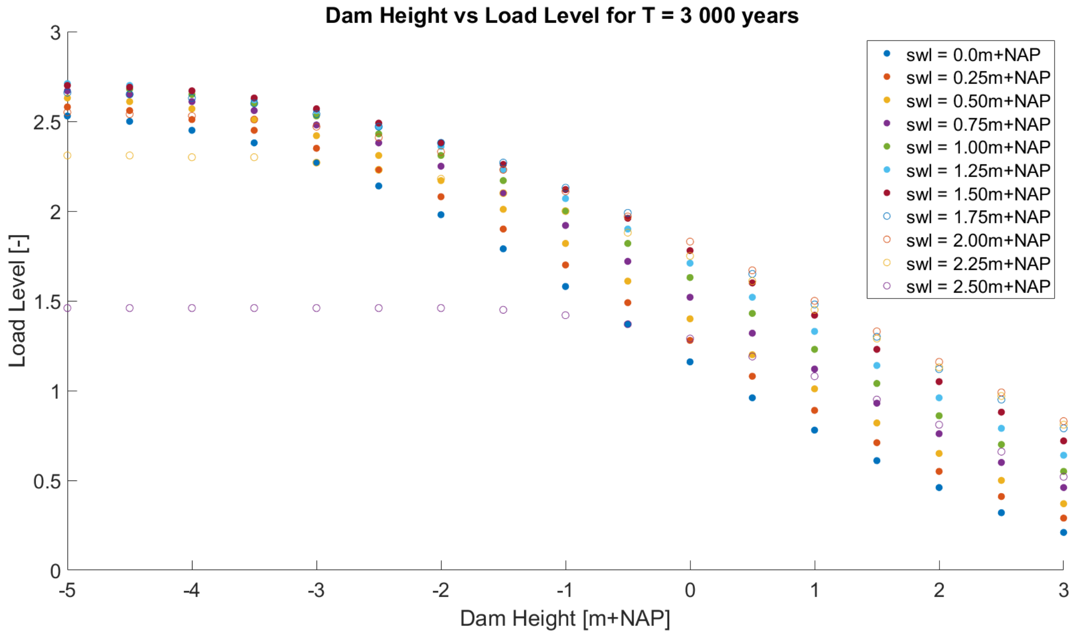

Figure B.4: Absolute load level for T = 3,000 year

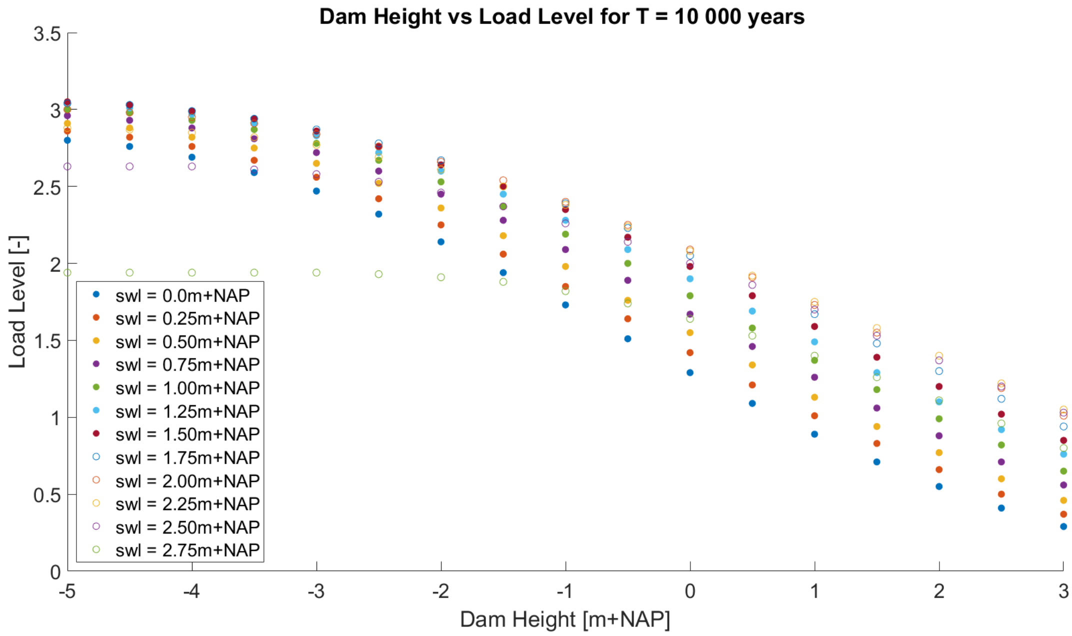

Figure B.5: Absolute load level for T = 10,000 year

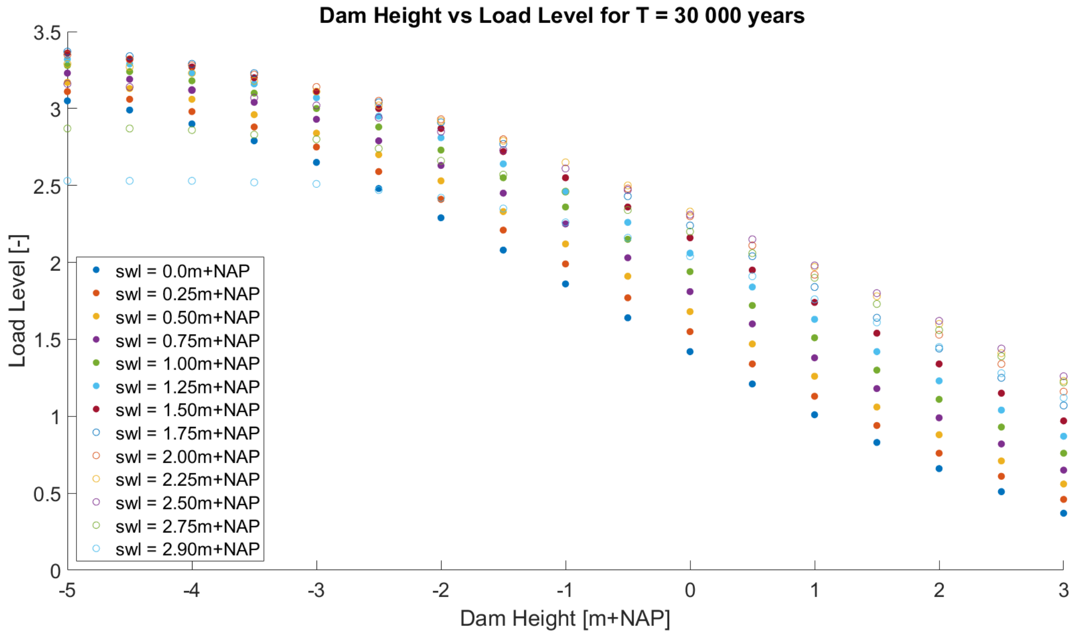

Figure B.6: Absolute load level for T = 30,000 year

B.2 Relative load level

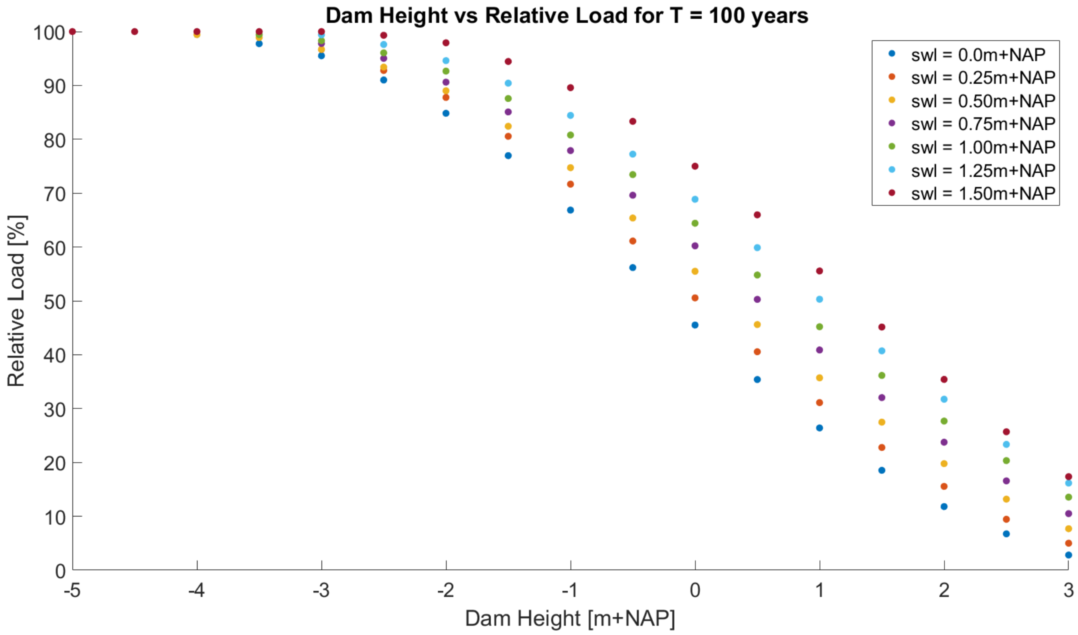

Figure B.7: Relative load level for T = 100 year

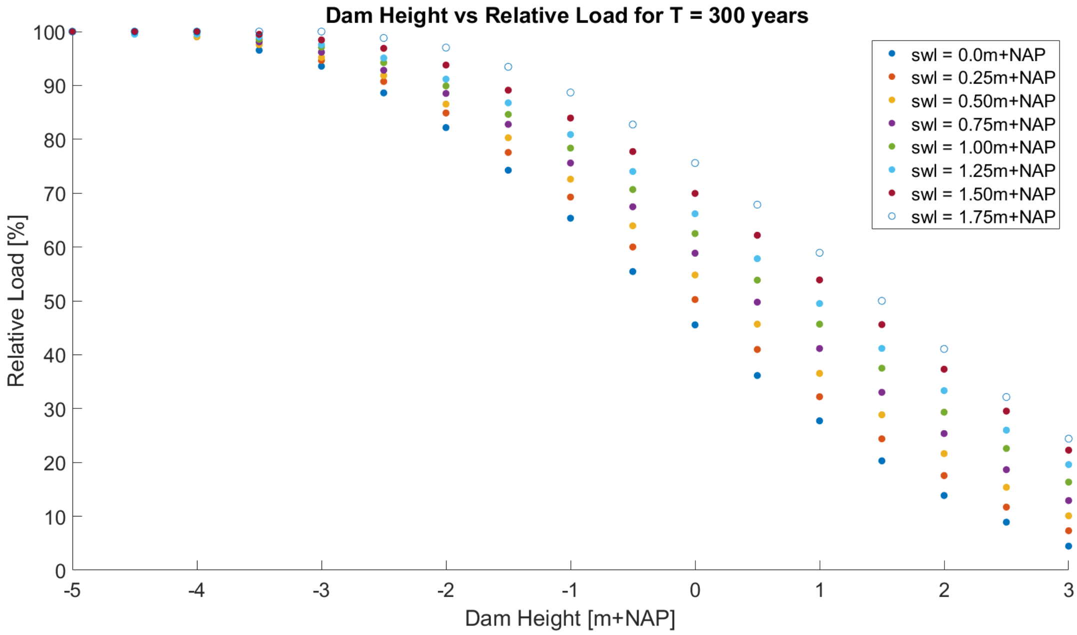

Figure B.8: Relative load level for T = 300 year

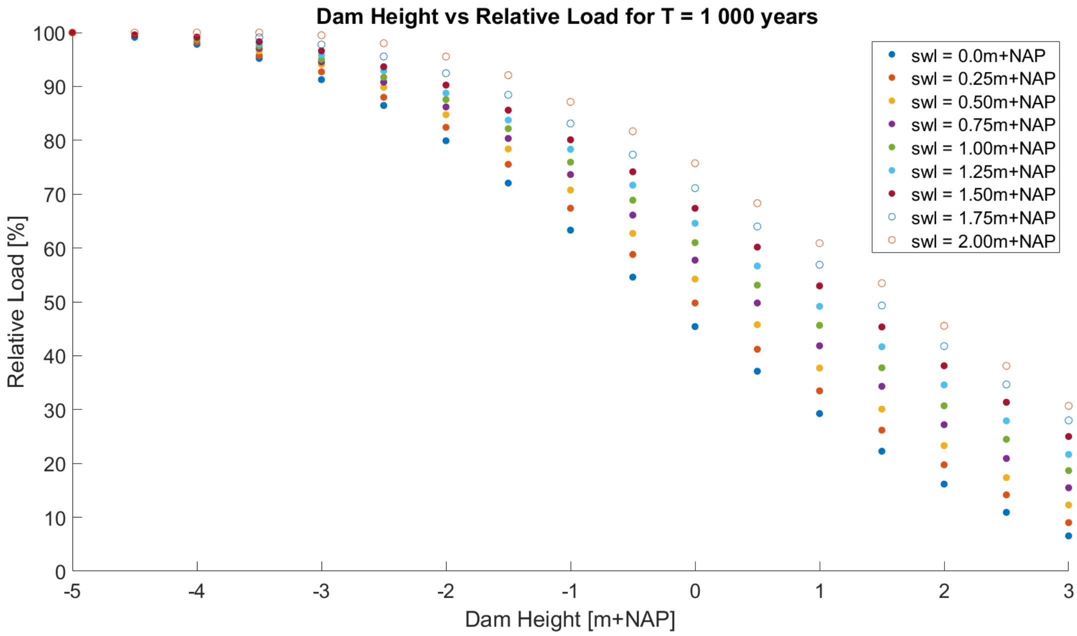

Figure B.9: Relative load level for T = 1,000 year

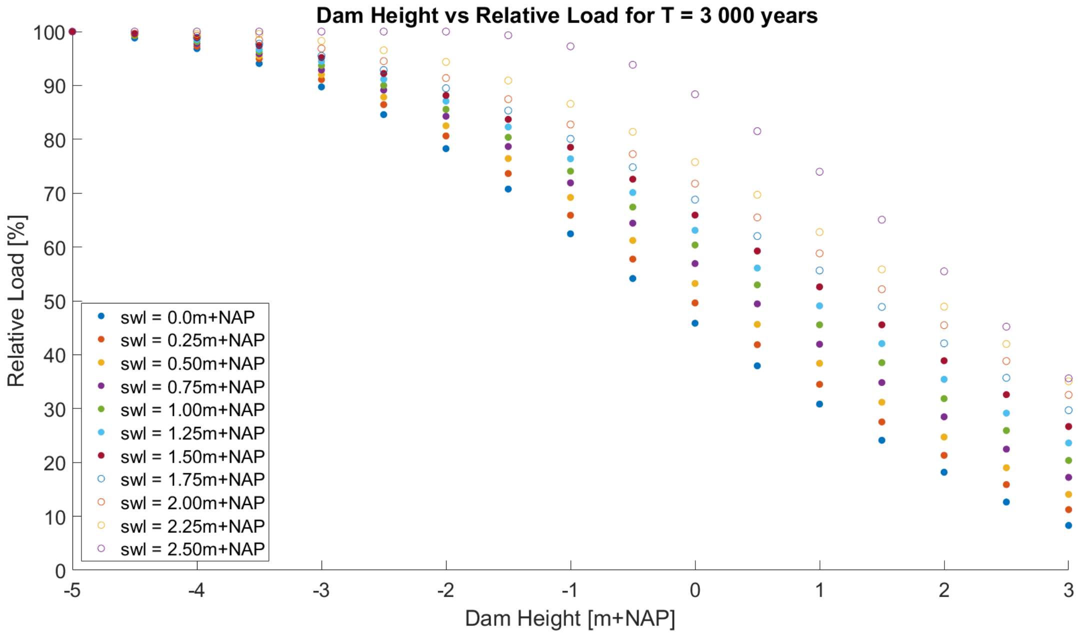

Figure B.10: Relative load level for T = 3,000 year

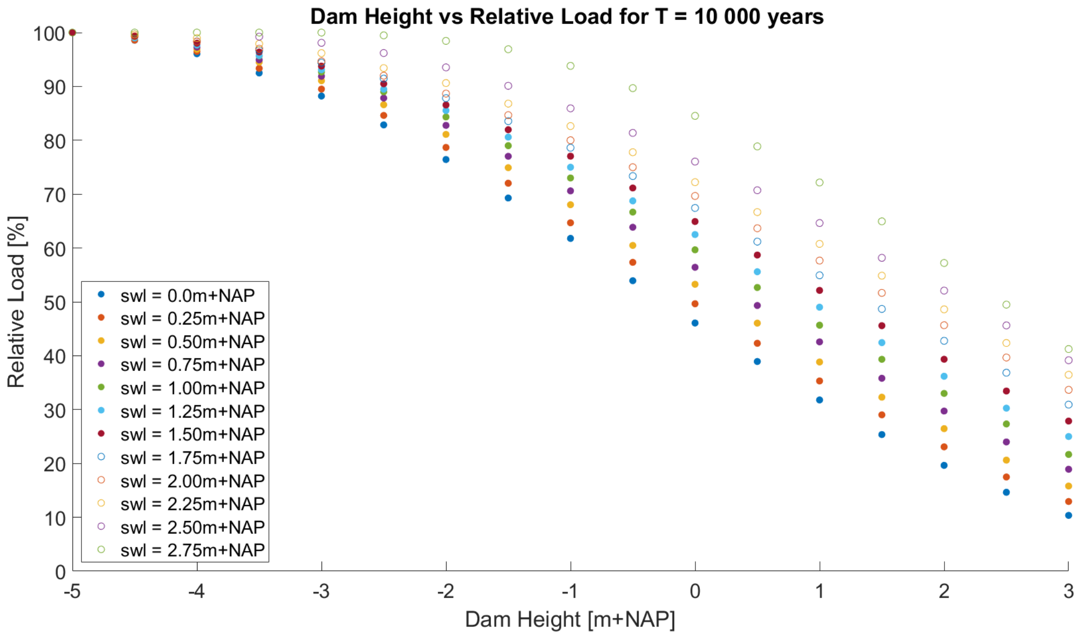

Figure B.11: Relative load level for T = 10,000 year

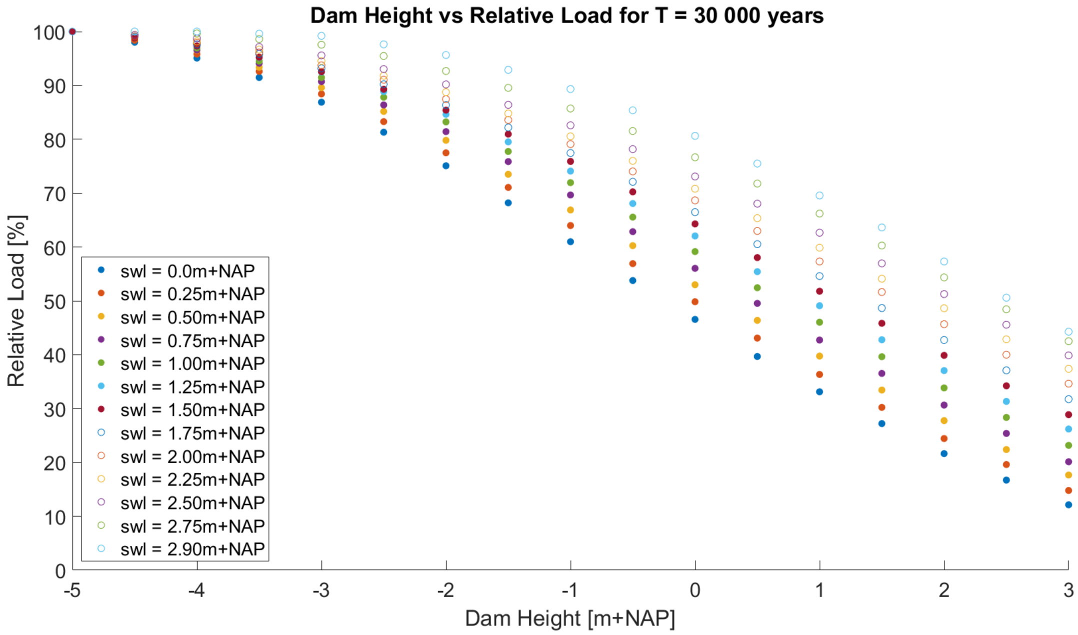

Figure B.12: Relative load level for T = 30,000 year

B.3 Submergence depth

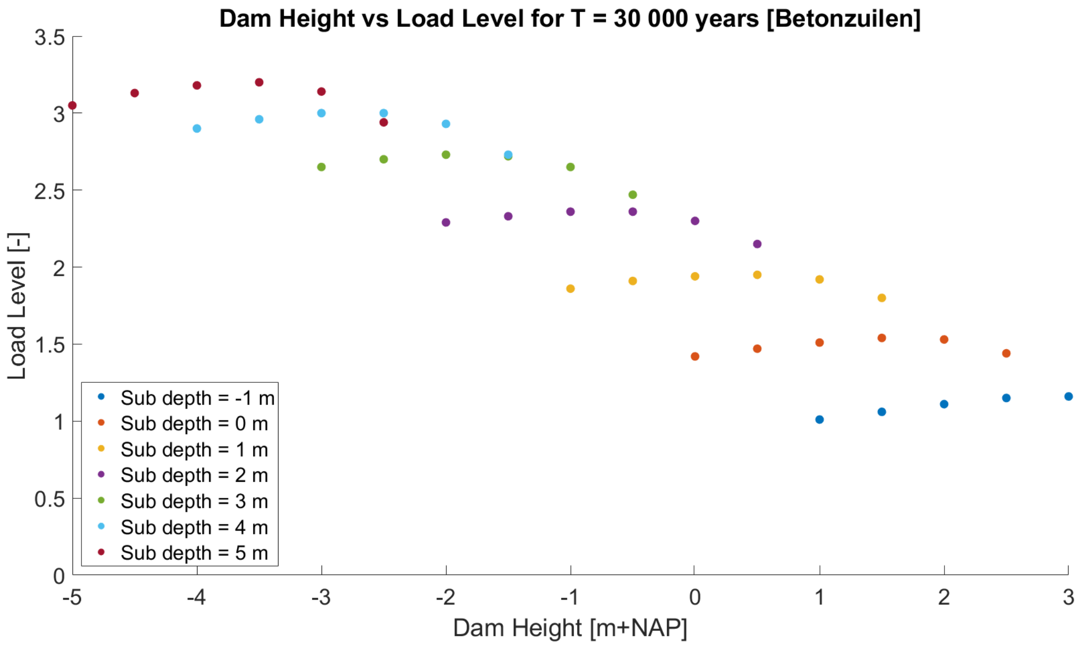

Figure B.13: Load level per submergence depth for T = 30,000 year

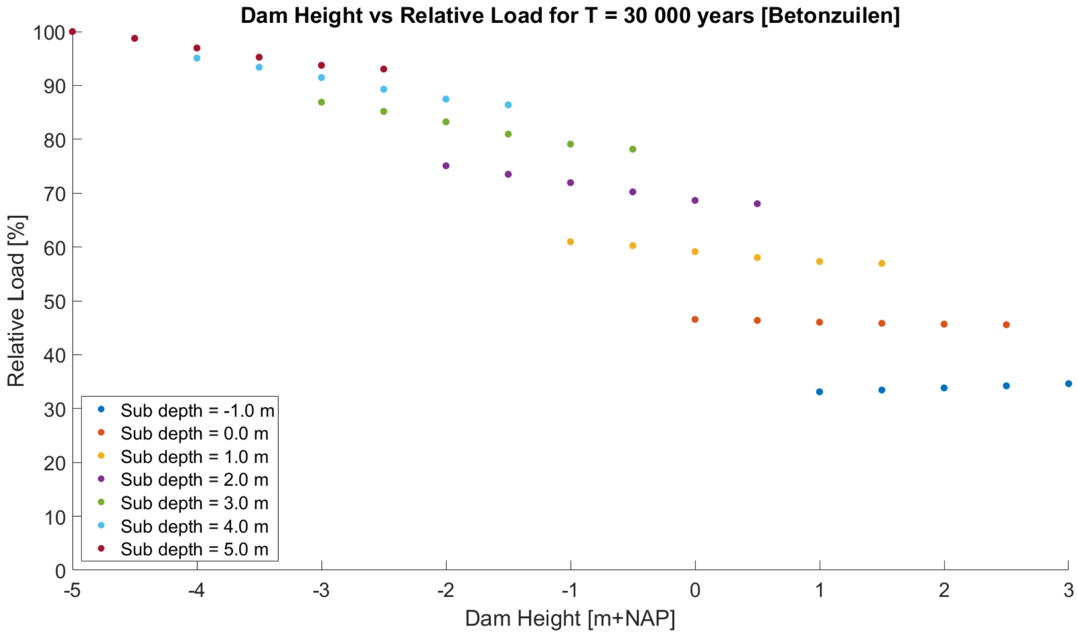

Figure B.14: Relative load per submergence depth for T = 30,000 year

Table B.1: Relative load intervals per submerged depth for a return period 30,000 years

Submerged depth [m]	Relative Load Interval [%]
5	100 - 93
4	95 - 86
3	87 - 78
2	75 - 68
1	61 - 57
0	47 - 46
-1	33 - 35