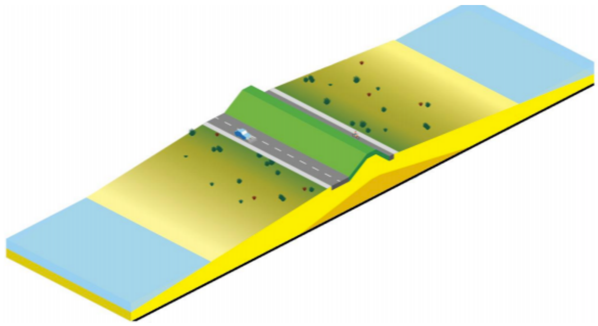
Bachelor Thesis Assignment

Effects of the Hydraulic loads at the Houtribdijk in a situation with and without a breaker bar



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Rijkswaterstaat supervisors: Ir. R.J.A Wilmink and Dr. E. Brand.

August, 2021

Department of Civil Engineering
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Internship at Rijkswaterstaat

Rijkswaterstaat
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Summary

The assignment will study the measured data at two locations on the Markermeer side, one location with a breaker bar and one location without a breaker bar. Both locations are exposed to hydraulic loads. In the situation with a breaker bar, the wave transmission and the breaker bar play a significant role in altering nearshore hydrodynamics. During the assignment, the wave transmission is studied in the location with a breaker bar. Also, fetch length, wind speed, and wave energy are defined to evaluate the main differences between both situations. The Markermeer lake is located on one side of the Houtribdijk, a levee where a natural based approach reinforcement was done. The challenge is that this is the first time in the world that this approach is made in a non-tidal area. The objective of the assignment is to evaluate the behaviour of the breaker bar and compare the findings to a situation without a breaker bar and preliminary theoretical research. The main research question of this research document is:

- What are the effects on wave loads of a rubble mound breaker bar?

The methodology is to consider the theoretical background to center the scope of the assignment, gain knowledge and understand the behavior of hydraulic load in shallow environments. Besides, a theoretical approach made in 2019 was done to estimate and help with the construction of the rubble mound breaker bar. In this study, the theoretical results will be compared with the main findings from the gathered data. The results show two main distinctions in the period of 22 days of March 2021. The distinction lays in calm and storm periods. The situation with a breaker bar demonstrates a wave reduction percentage of 80% during storm periods and a minimum percentage of 71% during calm periods. Also, the behavior of the breaker bar is analyzed under the influence of three parameters for the transmissivity of the the wave load, among them, the crest freeboard, the crest width, and the incident wave angle. The results, from the theoretical approach were also stated and compared with the results from the gathered data. The comparison lead to a positive relation between both findings, the main transmissivity of the wave load is by overtopping and through the structure. Also, the increase behind the breaker bar leads to a 2 cm of wave height. And, the wave reduction have a discrepancy being 58% for the theoretical approach and 71% for the analyzed data. Generally speaking, the main differences between the situation with and without a breaker bar is the wave load on the shoreline and the energy, from 3.4 [J/m2] to 81.5 [J/m2], respectively. Hence, it can be said that the effect of the breaker bar on the reduction of wave transmission is dominant, even considering that the transmitted wave height increases again due to the still existing fetch length of 0.1 [Km] behind the breaker bar at a speed of 6-8[m/s].

Acknowledge

"Put your heart, mind and soul into even your smallest acts. This is the secret of success" Swami Sivananda.

Throughout the writing of this Bachelor thesis assignment I have received a great deal of support and assistance. I would first like to thank my supervisor of the University of Twente, Dr. ir. B.W. Borsje, and my supervisors from Rijkswaterstaat Ir. R.J.A Wilmink and Dr. E. Brand whose expertise was invaluable in formulating the research questions and assessing the data. Your insightful feedback pushed me to sharpen my thinking and brought my work to a higher level.

I would like to thank University of Twente for all the academic support and for the pleasant study environment provided. In addition, I would like to thank Rijkswaterstaat for giving me the opportunity to execute my research assignment.

My sincere thanks to the Secretaria Nacional de Educacion Superior, Ciencia, Tecnologia e Innovacion (SENESCYT) for granting me a complete scholarship to study abroad and complete my Bsc bachelor assignment.

I would like to extend my hearted gratitude to my parents Maria Luisa Erazo and Luis Giovanni Salguero for their love, wise counsel and sympathetic ear. You are always there for me. Finally, I would like to extend my deepest gratitude to my brother and sister, my family and friends who made this experience a nice journey filled with love, and happy distractions to rest my mind outside of my research.

Contents

1 Introduction

1.1 General description

The Houtribdijk divides two artificial lakes; the IJsselmeer and Markermeer lakes. The construction of the levee started in 1956 and was finished until 1975 with a 25km-long levee [5]. Later, a road was implemented over the dike with an expected volume of 5000 vehicles per day, a cycle path, and a bus connection. Over the years, the Houtribdijk did not satisfy the current (legally defined) requirements for water safety, thereupon, reinforcement was necessary [4]. A nature-based approach is considered instead of using a traditional dike reinforcement called Building with Nature (BwN). For the reinforcement, 10 million cubic meters of sand were dredged at the Markermeer lake. (Figure 1) shows the last outcomes of the reinforcement, placed along the levee to protect it against flooding . The project is a pioneer since Building with Nature (BwN) has been done in the sea with tidal properties. Conversely, this scenario is different since the reinforcement was done in non-tidal lake areas. Figure 1 shows that the northern side was reinforced, where location 1 and location 2 can be seen. The breaker bar was highlighted on the figure with a blue dot and the location with no breaker bar with a yellow dot. The breaker bar is located there because it is designed to minimize the hydraulic load on the existing dike by absorbing the incoming wave attack. Furthermore, if the wave load is reduced, the maintenance at the levee will decrease. Additionally, Marker Wadden's finalization is accomplished, which creates new habitat for new flora and fauna.

FIGURE 1: Reinforcement 2019 [11]

1.2 Study Area

The Markermeer lake, in the central Netherlands, is a shallow lake with a mean depth of 4 [m NAP] and an area of 700 [Km^2]. Figure 2 highlights location 1 and location 2 on the Markermeer lakeside and the monitoring lakeside project. At location 1, a 1.5 Km long rubble–mound breaker bar with a crest elevation [0 m NAP] was built in front of the sandy reinforcement to decrease the hydraulic load on the sandy reinforcement on the Houtribdijk. Location 2, near Trintelzand is located where the shoreline is unprotected by a breaker bar.

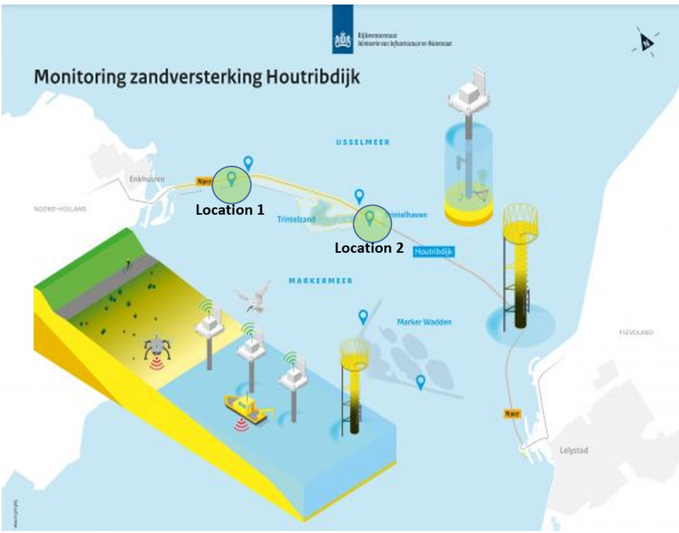

FIGURE 2: Monitoring Study Area [14]

The monitoring lakeside project focuses on measuring hydraulic data at both locations with the same amount of measurement stations, four in a shore parallel transect per location (Figure 4). A detailed description of the measurement stations can be seen in Figure 3 with the breaker bar. It displays location 1 with two offshore measurement stations FL68 & C, which gather the incident data from hydraulic loads before they meet the first breaking point. Onshore measurement stations A & B gather the transmitted data of the hydraulic loads traveling to the shoreline, second breaking point. At location 1, The breaker bar has a trapezoidal shape with a crest of 0 [m NAP], a crest width of 3 [m], and shoreward and seaward lateral slopes of 1:3. The structure is 1.5 [km] long, and 100 [m] offshore from the shoreline.

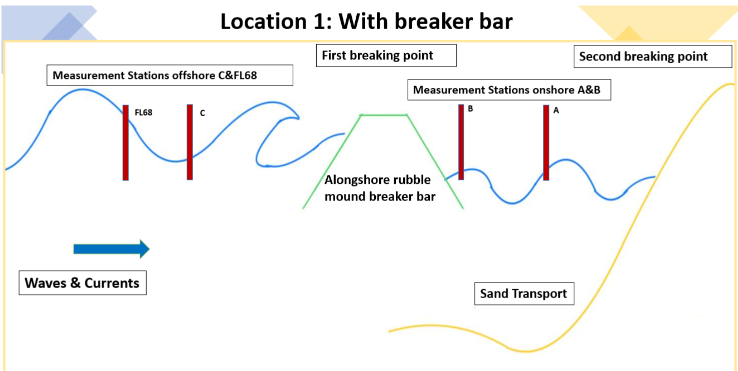

FIGURE 3: Conceptual cross section at location 1

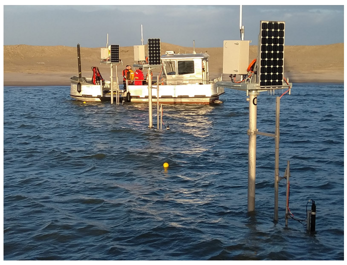

FIGURE 4: Each Monitoring Station contains sensors [14]

The conceptual cross-section at location 2 is shown in Figure 5. In the graph, there is one breaking point, the sandy shoreline. The location has the same four measurement stations located in shore parallel transect, the most offshore measurement station, FL67, and measurement stations A, B & C, most onshore. The measurement stations gather the same hydraulic data to analyze the wave load impact on the shoreline. The gathering data is a condition for this lake situation to compare the effects of a breaker bar with a situation without a breaker bar.

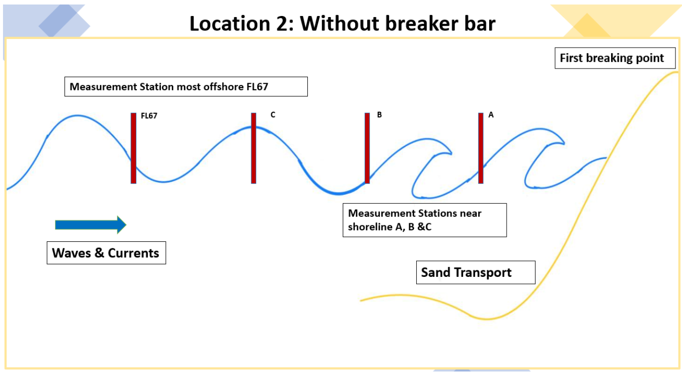

Figure 5: Conceptual cross section at location 2

1.3 Objective

There are two locations, one with a breaker bar and one without a bar. The objective is to assess the differences in gathered data of hydraulic loads in the situation with and without the breaker bar and relate the findings with preliminary research. To reach this objective, twenty-two days of data are used to quantify and describe the wave parameters, wave transformation, wave transmission over the breaker bar and evaluate the performance of a theoretical approach of transmission of the breaker bar on the Markermeer lakeside.

1.4 Research questions

1.4.1 Main Research

What are the effects on wave loads of a rubble mound breaker bar?

1.4.2 Sub research questions

- What is the difference between the incoming waves at location 1 and location 2?
- Is the wave reduction over the breaker bar, and what is the difference compared to the situation without a breaker bar?
- Are waves generated locally behind the breaker bar?
- How is the wave reduction influenced by the water level over the bar?
- What is the influence of wave direction onshore and offshore the breaker bar?

1.5 Theoretical framework

The theoretical background is used to define the main concepts of the scope of the research in both locations, as were showed in Figure 1. From Figure 3 the breaker bar is introduced and from Figure 3 and Figure 5 the waves are described since both will analyze the hydraulic load. Also, The description of specific variables of the waves, like wave height and wave period is done. To analyze and interpret the data to be gathered factors that affect the wave breaking in each breaking point are also included, such as, wind, bathymetry, effective fetch, surf similarity parameter, and wave transformation.

1.5.1 Rubble mound breaker bar

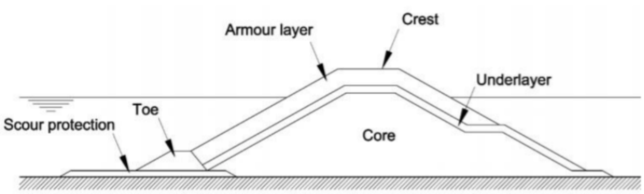

FIGURE 6: Rubble mound breaker bar [17]

A rubble mound breaker bar is quarried rock structure, usually protected by a cover layer of heavy armour stones or concrete armour units [17]. The general purpose of the breaker bar is to protect the anchorage or mooring of vessels from waves and currents attacks. Figure 6 shows the cross section of a breaker bar. The main body is usually built of wide-graded dredged or blasted material such as quarry run and armour layer cover, and the armour layer protects the crest. To maintain the slope's stability in case of erosion of the seabed, a toe and scour protection are usually constructed at the seaward side of the breaker bar.

1.5.2 Hydraulic loads: Waves

Waves will be explained with the main characteristics; wavelength, amplitude, wave height, wave steepness, and wave period. A wave is a water surface variation at a specific location. Figure 7 demonstrates the principal characteristics; the amplitude is the vertical water surface elevation, and the wave height is the distance between crests. Additionally, the wave steepness is the wave height divided by the wavelength and the wave period is when two consecutive crests pass a fixed location. Finally, the wavelength is the distance that a wave travels.

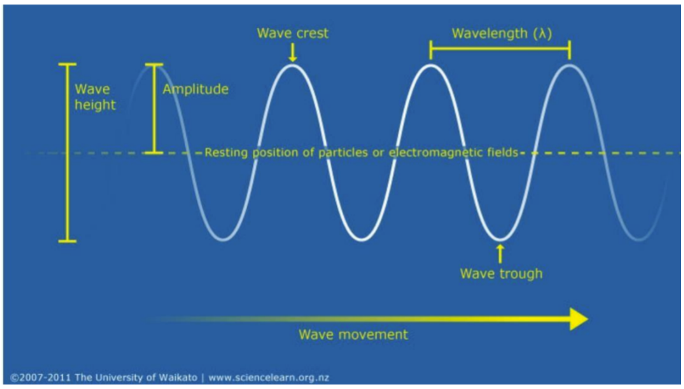

FIGURE 7: Schematization of a wave [21]

1.5.3 Wave breaking

The wave breaking occurs when the wave propagates towards a breaking point, and then the wave becomes unstable and breaks. The considered breaking points in this study are the rubble mound breaker bar and the shoreline. Wave breaking is when the water particle velocity exceeds the velocity of the wave crest (celerity) [9]. Wave breaking happens due to two breaking causes:

1. Steepness induced wave breaking.
2. Depth-induced wave breaking.

First, when the wave has become too steep to remain stable, it is denominated to be steepness-induced wave breaking. In other words, when the ratio wave height and wavelength become too large. Second, the depth-induced wave breaking. For shallow water, dimensionless limits were derived [9]. The limits are shown in Equation 1,Equation 2, and Equation 3 [18].
Breaking limit

$$\frac{Hb}{L} = 0.142 \cdot tanh(kh) \tag{1}$$

For deep water

$$tanh(kh) = 1 \quad \Longrightarrow \frac{Hb}{L} \approx 0.142 \cdot 1 \approx \frac{1}{7} \tag{2}$$

For shallow water

$$tanh(kh) = kh \implies \frac{Hb}{L} \approx 0.142 \cdot \frac{2\pi}{L} \cdot h \approx 0.88 \cdot \frac{hb}{L} \tag{3}$$

Breaker index (Miche reference) shallow water

$$\gamma b \implies \frac{Hb}{L} \approx 0.88 \cdot \frac{Hb}{L} \cdot h \approx 0.88 \cdot \frac{hb}{L} \tag{4}$$

breaker index (Significant wave height) shallow water

$$\gamma b, Hs = \frac{Hs}{hb} \approx 0.4 - 0.5 \tag{5}$$

1.5.4 Bathymetry

Bathymetry is the description of the ground surface below water [13]. It has a significant influence on the volume of rock needed and the hydraulic loadings, and it is a critical boundary condition for the geometric and structural design of rock structures. At coastal and lake environment sites, for example, the water depth can limit wave heights. Bathymetric data comprise series of points, each determined with three coordinates: horizontal (x,y) and vertical (z).

1.5.5 Wave transformation

When water level is less than 0.05L the wave is in shallow water. In shallow water the wave is non dispersive and wave celerity is only depended on local water level as:

$$C = \sqrt{gh} \tag{6}$$

As the first waves will arrive at shallow water the wave celerity will decrease because of the water level but the following wave is still at deeper water and has its original celerity. The following waves will catch up with the first waves, together the waves will concentrate wave energy and increase in wave height, this is shoaling. Another phenomenon is wave refraction. It is where waves with an oblique angle will rotate towards the coastal normal. At deeper water the wave celerity is higher which causes the most offshore wavefront to catch up with the closest wave front to the coast. And, thus, it results in rotating towards the coast.

1.5.6 Breaker bar wave interaction

Wave breaking and porous flow, two very complex physical phenomena, are part of the wave impacts on the rubble mound breaker bar [2]. Due to the reduced water depth on the lake landwards and increased wave steepness, wave breaking will be triggered when the wave face a sloping structure [7]. Several physical processes arise from the wave structure interactions. First, reflection, as Figure 8 shows, a substantial proportion of wave energy may be reflected from or absorbed by the structure. Second, transmission, a significant amount of wave energy go through and/or over the breaker bar [8]. Third, diffraction, the remaining wave energy will be transmitted by overtopping or by waves permeated through the structure at the head of the breaker bar, then waves turn into diffraction waves around the tips.

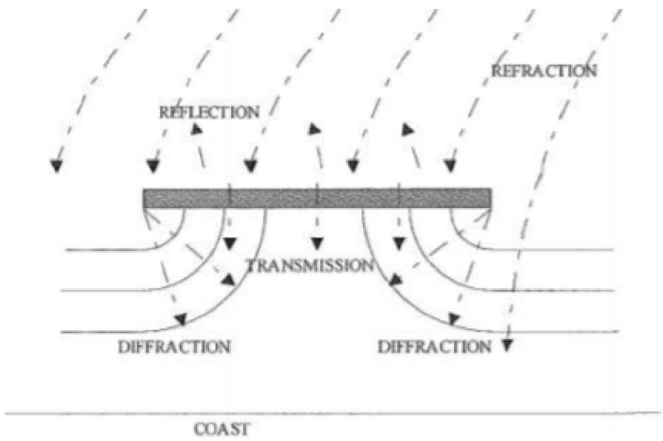

FIGURE 8: Wave near a breaker bar [7]

1.5.7 Wind waves

In the Markermeer, the primary mechanism of waves is wind [3]. The wave growth is affected by several parameters by wind forcing in relatively shallow water, among them, wind speed, wind duration, Fetch, and water level. The wave height can have different limiting factors, such as friction of the bed, area of generation has a limited fetch, or the duration of the wind.

1.5.8 Effective Fetch

The principle of the effective fetch is based on the assumption that wind transfers energy to the water surface in the wind direction and all directions within a particular directional sector on either side of the wind direction [1].

$$Fe = \frac{\sum Ri(\theta_i) \cdot \cos^2 \theta_i}{\sum \cos \theta_i} \tag{7}$$

Where θ is the angle between a certain fetch length and R the fetch length.

1.5.9 Surf similarity Parameter

The type of wave breaking is determined by three conditions of the ξ number[19]:

- Spilling breaking on gentle slopes for ξ<0.2;
- Plunging breaking with steep overhanging wave fronts 0.2<ξ<2.5;
- Collapsing and surging breaking waves on very steep slopes ξ>2.5.

$$\xi = \frac{tan(1.25)\ T\ tan(\alpha)}{\sqrt{s}\ (H_{1/3})^{0.5}} \tag{8}$$

Equation 8 shows how to obtain the surf similarity parameter, Where α is the slope angle of bottom structure, H significant wave height at toe of the structure (H_{m0}), The wave length stands by $L = 1.56(T)^2$, and T is for wave period or T_p.

2 Methodology

2.1 Theoretical approach

A theoretical approach focuses on the quantitative calculation of the functioning of the breaker bar, concerning wave reduction, the residual wave behind the breaker bar, and resulting wave load on the sandy maintenance. It follows the hypothesis that the breaker bar dampens wave action, and thus, wave force decreases on the shoreline as well as the maintenance. The purpose of the theoretical approach was to gain knowledge and experience about the breaker bar in the Markermeer [10]. This study follows the reference design guidelines made by Royal HaskoningDVH prior to the reinforcement and sharing this knowledge with Rijkswtaerstaat organization. And, it was done by making recommendations regarding the measurement and monitoring program around the Houtribdijk called LakeSIDE. In this assignment, the data generated by the LakeSIDE project are quantified, analyzed, and finally compared with the final results of the theoretical approach. It is important to understand how the theoretical approach was done, to consider the main assumptions and discuss them in the discussion section.

TABLE 1: Geometry and Design conditions[12]

Parameter	Symbol	Value (Source)
Slope outside	tan_α	1:3 (Rijkswaterstaat, 2016)
Crest width	B	3.0 [m] (Rijkswaterstaat, 2016)
Crest height	R_c	0 [m NAP]
Design Parameters		
Permeability coefficient	$P.$	0.6 [rubble mound breakwater] (Rock Manual, 2007)
Damage number	$S.$	10 (Rijkswaterstaat, 2016)
Stone density	ρ_r	2650 Kg/m3 (communication technical management, 1/22/2019)
Median mass rubble stones	M_{50}	200 Kg (communication technical management, 1/22/2019)
Median nominal stone diameter	D_{n50}	0.33 - 0.42 [m]

Table 1 shows the design conditions, during storms the Markermeer is dominant subject by waves, hence in the study it is assumed that the system is wave-dominated, and currents are disregarded in the approach[10]. The study looks at the hydraulic load (wave attack) occurring during storms with a return period around 0.1-year storm (a storm that occurs on average ten times a year), 1/1 year storm, 1/10 year storm, 1/50 year storm, and 1/100 year storm. Then, the calculation for the significant wave height (H_{m0}), peak wave period (T_p), and water level (wl) were derived from three logarithmic relations based on hydraulic loads from HydraNL as Equation 9, Equation 10 and Equation 11 shows .

$$H_s = 0.2637 log(T) + 0.9253 \tag{9}$$

$$T_p = 0.4972 log(T) + 5.1441 \tag{10}$$

$$wl = 0.3358 log(T) + 0.5679 \tag{11}$$

The use of wind conditions at the height of 10 m was added to the analysis, from the KNMI Schiphol station. The wind speed is used to calculate the wave growth between the foreshore dams and the sandy reinforcement and for the calculation, Equation 12 was used:

$$U_{10} = 2.5805 log(T) + 18.673 \tag{12}$$

In the study, it is assumed that incident waves are perpendicular between the breaker bar and the sandy reinforcement, hence the angle of wave incidence is taken as 0 degrees[10]. The fetch length and the wind speed, allows the wave growth to take place behind the breaker bar, towards the sandy reinforcement. The breaker bar in this study considers behind the breaker bar a fetch length of 320 [m] between the breaker bar and the sandy reinforcement. The next relationship is between the sea bed, water levels, and wind conditions. The storms cause the water level to skew over the Markermeer, the water level in the southwestern corner will drop, and the water level in Northeastern, during storm, water level will rise[10]. In addition, the bed position (z) is another important parameter to determine the water depth (h). Following equation 13 the emergent breaker bar is at position z = -1.5 m NAP. Table 2 summarizes the conditions used in this study [10].

Water depth

$$h = |z| + wl \tag{13}$$

Table 2: Data theoretical approach[10]

T(yr-1)	N.storm (h)	His (m)	Tp (s)	Wl (m)	Z (m)	h (m)	U10 (m/s)	Wind direction (m/s)	F (m)
0.1	6	0.66	4.6	0.23	-1.5	1.7	16.1	240	320
1	6	0.93	5.1	0.57	-1.5	2.1	18.7	240	320
10	6	1.19	5.6	0.90	-1.5	2.4	21.3	240	320
50	6	1.37	6.0	1.14	-1.5	2.6	23.1	240	320
100	6	1.45	6.1	1.24	-1.5	2.7	23.8	240	320

The formulas to determine the hydraulic interaction between wave load, wave reduction, wave growth, and wave refraction of the theoretical approach are described in this section. The formation of the waves behind the breaker bar is caused by two hydraulic interactions:

1. wave overtopping, mainly during storms
2. transmission of long waves

The transmission of the waves is due to the permeability of the structure. The wave transmission coefficient is conceptualized as Equation 14 shows.Equation 15 conceptualize the transmitted wave height and Equation 16 represents the percentage of wave reduction Conceptualization of transmission coefficient:

$$Ct = \frac{Ht}{Hi} \tag{14}$$

Conceptualization of transmitted wave:

$$Ht = Ct \cdot Hi \tag{15}$$

To percentage of wave reduction transmitted wave:

$$\text{Golf reductie} = (1 - Ct) \cdot 100\% \tag{16}$$

To quantify the results for the transmission coefficient of rubble mound breaker bar, Equation 17 is used:

$$C_t = -0.4 \cdot \frac{R_c}{H_{m0,i}} + 0.64(\frac{B}{H_{m0i}})^{-0.31} \cdot (1 - exp(-0.5\xi_\rho) \tag{17}$$

Wave breaking strongly depends on the ratio of the slope of the bottom or structure and the wave steepness. This ratio is known as the surf similarity parameter ξ [19]
Surf similarity parameter

$$\xi = \frac{tan(\alpha)}{[\frac{H_{1/3}}{L_o}]^{0.5}} = \frac{1.25T_{m-1,0} \cdot tan(\alpha)}{(H_{1/3})^{0.5}} \tag{18}$$

The residual wave which remains behind the breaker bar can grow further under the influence of the wind. Calculations of wave growth follows the Brettschneider equation [10].

Brettschneider equation

$$H = \frac{H_{s,groei}g}{U_{10}} = 0.24tanh(0.343 \cdot h^{1,14})tanh \cdot (\frac{4.41 * 10^{-40.79}}{tanh \cdot 0.343h^{1.14}})^{0.572} \tag{19}$$

2.2 Wave Measurement: Houtribdijk monitoring and research program

This section explains how the waves were measured by the Houtribdijk monitoring and research program [20]. It was noted based on a measurement that the Markermeer side at location 1 and location 2 was located at a depth of -3.5[mNAP] and -2.9 [mNAP], respectively. Each location has four measurement poles, one is a permanent offshore measurement pole (FL68 FL67), and the remaining measurement stations C, BA (most onshore), have two times three smaller nearshore measurements poles. The permanent offshore measurement station has a step gauge (STB) and an Acoustic Current Doppler Profiler (ADCP). The STB can measure every 4Hz the water levels from approximately -1.5 to +1.5 m via a series of contact sensors every 5cm. Due to the high sampling frequency, information about wave heights and wave periods can be obtained. Next, the ADCP, with a frequency of mean values, measures every 10 minutes by transmitting "pings" of sound, the flow velocity, and direction with 24 vertical layers with a layer thickness of 25 cm. In location 2, two extra instruments are utilized: Multi-Parameter Probe (MPP) and Vector Acoustic Doppler Velocimeter (ADV). On the one hand, The MPP gives one measurement every 10 minutes, and the gathered information is temperature, conductivity, pH, turbidity, and chlorophyll. On the other hand, the ADV measures with a frequency of 4Hz the pressure and velocity in three directions (East, North, and Upward). With these signals, wave height, wave period, and wave direction are estimated. Therefore, the MPP is not relevant for this research and is ignored. By adding the ADV at this offshore station, we can verify that the STP and ADV are in the same order, so it is possible to make the wave developments over all four stations in a row. The measured pressure, as Equation 20 shows, allows the calculation of the water pressure. The KNMI station, Lelystad, proportionated the air pressure, and the pressure offset compares the mean water level on ADV pressure and the mean water level of the nearest lakeside step gauge pressure, as Equation 22 shows. Then, knowing the water pressure (Equation 20), and the sensor's height, the NAP water level can be determined. Later, the difference between the normal NAP water level and the sensor NAP water level determines the wave amplitude.

$$p = \rho g h \tag{20}$$

$$waterpressure = pressureoffset - airpressure \tag{21}$$

$$Pressureoffset = lakesidestepgaugepressure - ADVpressure \tag{22}$$

The three mobile measurement stations are present close to the shoreline and got an Altimeter (ALT), Vector Acoustic Doppler Velocimeter (ADV), and/or Aquadopp HR (AQD). Measurement station two is equipped with an Aquadopp HR (AQD) for a more detailed analysis of the currents near the bottom. The AQD measures with a frequency of 4Hz with high resolution and with thin vertical layers. It provides detailed insight into the near-bottom flow and the frequent high pressure and/or velocity measurements wave information. Finally, Stations 13 are equipped with ADV to measure wave transformation, and the Altimeter (ALT) is present in the three mobile measurement stations to investigate how the bottom moves (accretion and erosion), giving one measurement every 10 minutes. Therefore, the ALT is not used relevant in this study case.

2.3 Analyses

In this section the description of the gathered variables and the formulas for the quantification of the main behavior and effects of the breaker bar are described, Table 3 gives a summary of the main parameters used for the quantification of the results. The research focuses on the effect of the hydrodynamics on the breaker bar and the shoreline at two locations. A theoretical background was provided in Section 1.5. to understand the behavior of the waves in shallow water conditions, such as wave breaking and wave transformation and effects from parameters like wind speed and fetch length on wave heights. The information explained in the theoretical background helps to understand the data as a general from stations most offshore at both locations. From the collected hydraulic data, analyses are performed focused on describing the behavior of the breaker bar regarding the wave load transmission. During the analysis of location 1, the formulas used in Section 2.1 are followed to quantify the data and compare the results. Equation 13 is utilized to quantify the water depth, considering each measurement station's design water level and water depth. Then, Equation 17, and Equation 18, are followed to quantify the breaking type and wave energy transmission over the breaker bar.

TABLE 3: Governing parameters[20]

Governing parameters of wave Transmission	Description	Data	Source
H_i	Incident wave height, typically H_{moi} ,at the toe of the structure;	Location 1 ADV_C	Lakeside project (Vuik, 2019)
H_t	Transmitted wave height, typically H_{moi} at the toe of the structure;	Location 1 AQD_B	Lakeside project (Vuik, 2019)
T_p	Peak wave period;	STB ADV_C AQD_B ADV_A	Lakeside project (Vuik, 2019)
S_{op}	wave steepness	$S_{op} = \frac{2\pi H_i}{gT_p^2}$	(Giuseppe, 2013)
R_c	Crest freeboard: positive upward, zero at still water level;	0 [m NAP.]	(Pluis, 2019)
B-h_c	Crest width;	3 [m]	
D_{n50}	Nominal diameter of armor units (in case of rubble mound structure);	0.33 [m] - 0.42 [m]	(Pluis, 2019)
K_t	Transmission coefficient,	Ht/Hi	(Giuseppe, 2013)
ξ_{op}	Breaker parameter,	$\xi_{op} = \frac{\tan(\alpha)}{\sqrt{S_{op}}}$	(Giuseppe, 2013)
tan_α	Seaward slope of the structure	1:3	(Pluis, 2019)
op	Subscript for offshore conditions and peak wave period	FL68 STB	
hb	Depth of refraction	STB ADV_C AQD_B ADV_A	(Rijn, 2016)
$Hs_{,HRD}$	Remaining wave load on sandy reinforcement	Location 1 and Location 2 ADV_A	(Rijn, 2016)
L	Wave length	$L = \frac{9.8T_p^2}{2\pi}$	(Cappietti Lorenzo, 2012)

3 Results

In this section will be describe the main results of the gathered data by the four measurement stations on location 1 and location 2. It starts by describing the main differences between both locations as a general. Then, the focus goes to Location 1, with a breaker bar, to describe the wave behaviour before and after the breaker bar. And, It is also described the influence of the external factors on the behaviour of the hydraulic loads and the breaker bar.

3.1 Analysis of the hydraulic loads on each location

Twenty-two days of field data (5-27 March 2021) were used to evaluate the behavior of hydraulic loads in a situation with and without the breaker bar. The wave height (H_{m0}), peak period (T_p), and water level (h) time series at location 1 and location 2 are shown (Figure 9). Figure 9 shows the offshore measurement station data (instrument STB). The average wave height (H_{m0}) at location FL67 is 0.26 and at FL68, 0.22, therefore with a ratio of 7:6. The wave period has a ratio of 10:9, taking from the data the average value of 2.2 [s] on location 2, whereas location 1 has an average peak period of 2.02 [s]. The water level shows a slight difference between both locations, giving a ratio of 1 and varying with an average value of -0.21 [m NAP] at location 2 and -0.23 [m NAP] at location 1 due to wave conditions. Hence, the hydraulic loads differ from the most offshore measurement station between location 1 and location 2.

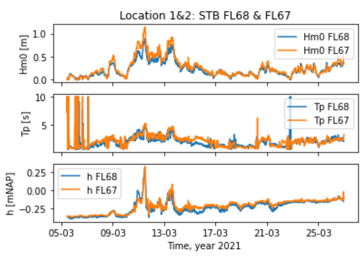

FIGURE 9: Significant wave height (H_{m0}), peak period (T_p), and water level (h)

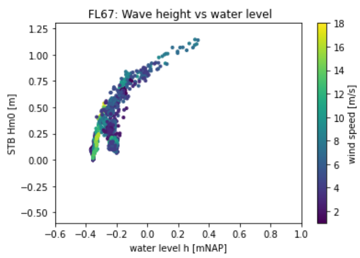

FIGURE 10: relationship between wave height (Hm0), water level (h) and wind speed for location 2

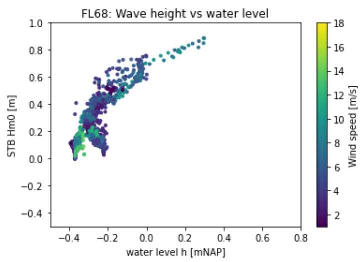

FIGURE 11: relationship between wave height (Hm0), water level (h) and wind speed for location 1

Figure 10, and Figure 11 shows that the conditions differ in the hydraulic load. FL67 has a lower water level of -0.5 [mNAP] while FL68 starts at a water level of -0.4 [mNAP]. The wind speed also differs at the beginning, FL67 shows a higher wind speed of $16 - 18$ [m/s] while FL68 shows at the beginning a wind speed of 14-16 [m/s]. Then, both figures follow same patterns with the difference of the maximum wave height reached by FL67 of almost 1.2 [m] and 0.9 [m] at FL68.
Table 4 gives the average values of the findings considering the twenty-two days of data for the main parameters: Wave height (Hm0), peak period (Tp) and water depth (h). It was done to give a general overview of the behavior of the data from most offshore measurement stations to onshore measurement stations at both locations. Results show that deep and shallow conditions are found in most stations. The results show no breaking by the bed level in both situations; still, there is wave transformation. The wave will start to transform, wave height, length and direction, until the wave breaks and dissipates its

energy.

TABLE 4: Parameters for Location 1 and Location 2 for effects of water level and wave length

	Location 1: FL68 STB	Location 1: FL68 ADV_C	Location 1: FL68 AQD_B	Location 1: FL68 ADV_A	Location 2: FL67 STB	Location 2: FL67 ADV_C	Location 2: FL67 AQD_B	Location 2: FL67 ADV_A
h [mNAP]	3.3	3.4	1.6	0.9	2.6	2.2	1.5	1.1
Deep/ Shallow	Deep and shallow conditions	Deep and shallow conditions	Shallow conditions	Shallow conditions	Deep and shallow conditions	Deep and shallow conditions	Shallow conditions	Shallow conditions
Breaking	No breaking	Breaking at breaker bar	Breaking at breaker bar	No Breaking	No breaking	No breaking	No breaking	No breaking
Celerity [m/s]	5.7	5.8	4.0	3.0	5.1	4.7	3.9	3.3
Energy [J/m^2]	91.2	37.6	1.0	3.4	134.0	92.3	88.9	81.5
Hm0 [m]	0.2	0.1	0.03	0.05	0.3	0.2	0.2	0.2

Table 4 shows the celerity in both locations, and it decreases as the water depth decreases, with the difference that location one is lower because of the presence of the rubble mound breaker bar. Section 1.5.6 describes the effect of the breaker bar on wave transmission. However, the wave height does not increase from offshore to onshore since not all the waves are at deep conditions in most offshore measurement stations (STB). Instead, it was a mixture of intermediate conditions. Another difference is the measured energy at both locations, most onshore. For instance, the period in one of the storm periods at measurement station A on March 11th gave an insight into the wave energy going landwards. At FL68, 0.2[m] of wave height with a peak period of 2.4[s] and wave energy of 72.1 [J/m^2] were found. Whereas in the same period at FL67, a wave height and peak period of 0.9 [m] 4.7 [s] average and energy of 979.2 [J/m^2] respectively, were found. Therefore, offshore stations have slightly different wave conditions and at FL67 the conditions are a bit more rough.

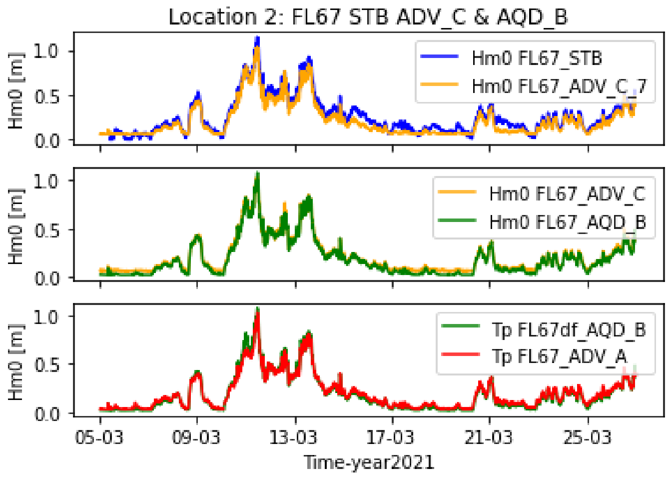

FIGURE 12: Significant wave height as measured at STB, ADV_C, AQD_B,and ADV_A

3.2 Effects of the rubble mound breaker bar

In this section, the analysis of location 1 was done to evaluate the effects of the breaker bar. Figure 13, top panel, demonstrates a general insight of how wave heights are reducing when going towards the two breaking points. The physical processes of the breaker bar consider measurement station C to analyze the data offshore the breaker bar. Next, the data from measurement station B to analyze the data onshore the breaker bar. Then, data from measurement station B and measurement station A analyze the process within the lee zone (area behind the breaker bar), and the data from measurement station A helps to understand the wave load traveling towards the shoreline.

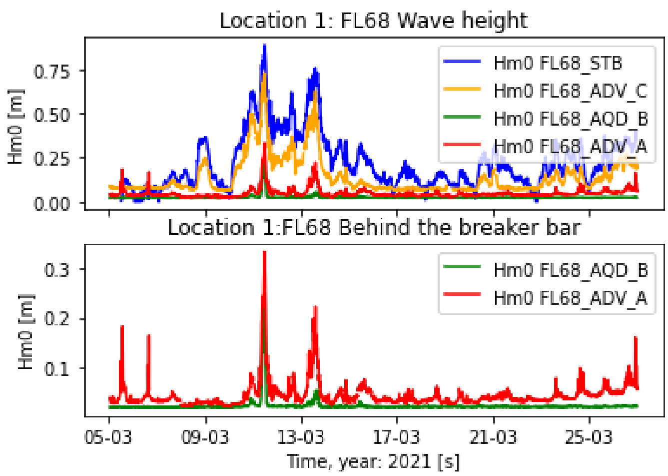

FIGURE 13: Wave height approaching the breaker bar

Figure 13 bottom illustrates that in the lee zone, the wave height will be substantially reduced but transmitted waves will continue to increase and propagate towards the shoreline. The wave energy at the shoreline was significantly reduced. The binding effect of the breaker bar that is important for this study is mainly the transmission since diffraction can hardly happen in this study. The reason is that there are hardly any openings, and the breaker bar is continuous. The growth of the wave height is by 2 cm as Table 4 shows from measurement station B to A due to local wind growth.

3.2.1 The hydraulic boundary conditions

The primary function of the breaker bar is to reduce the amount of energy in sheltered areas behind the breaker bars. Table 3 listed the governing parameters of wave transmission. Table 5 gives the average values of the main parameters, among them wave height [m] and peak period [s], in six different situations where two distinctions were made. One distinction represents the calm period, before March 10th and after March 13th. The second distinction considers storm periods between March 10th and March 13th due to heavy gusts that occurred. For instance, on March 11th, heavy gusts of wind of 75 Km/h happened [15]. Then, the quantification of the main parameters has been done, and the average value results are shown in Table 5.

TABLE 5: Summary of the ranges of parameters involved in wave transmission at measurement station 3, ADV_C

Date	Type	Hm0 [m]	Tp [s]	ξ	Kt [-]	Ht [m]	wave reduction %
March 5 -10	calm period	0.1	1.5	2.2	0.3	0.07	85.8
10-Mar	storm period	0.2	2.6	2.3	0.4	0.21	80.5
11-Mar	storm period	0.4	3.4	2.4	0.5	0.33	76.5
12-Mar	storm period	0.3	2.7	2.3	0.4	0.22	79.7
13-Mar	storm period	0.4	3.2	2.4	0.5	0.32	77.4
March 13- 27	calm period	0.1	1.7	2.3	0.3	0.08	84.8
March 10 highest	storm period	0.7	4.6	2.4	0.3	0.2	71.8

In this section, measurement station C is considered the incident data to the breaker bar. The wave height (H_{m0i}), peak period (T_p), and water level (h) time series at the offshore station are shown in Figure 14, left panel. During storms, the wave height at ADV_C reached between 0.3 [m] and 0.7 [m], and peak periods ranged from 2.6 [s] to 4.6 [s]. On the contrary, during calm periods, the wave height at C reached 0.1 [m], and spectral periods ranged from 1.5 [s] to 1.7 [s]. The water level measured at ADV_C was utilized as the reference value to evaluate the crest freeboard and then evaluate it with the transmitted wave height measured at AQD_B. That might occur at this site because, at this site, water depth outside the breakwater should limit incident wave heights to about the maximum measured during the storm. As the water level inside the breaker bar increases relative to that offshore, the data shown in Figure 14, left panel, bottom graph shows a slight increase in water level from -0.25 [mNAP] in the C to -0.20 [mNAP] at a ratio of 6:5.

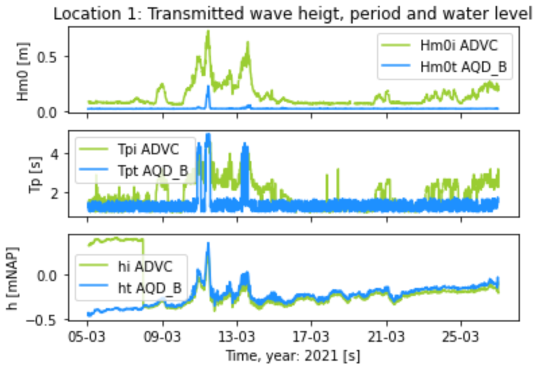

FIGURE 14: Overview of the incident and transmitted wave height (Hm0), peak period (Tp) and water level (h)

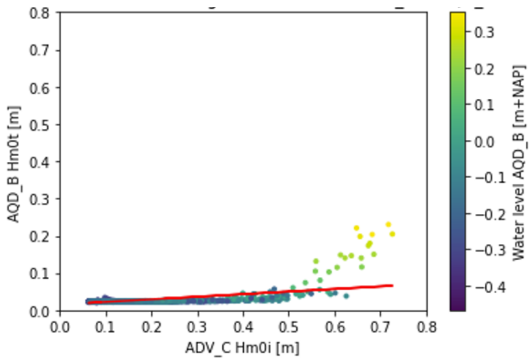

FIGURE 15: relation between the incident wave height and transmitted wave height and the correlation as water level increases

3.2.2 Breaking wave

The ξ number helps to recognize which type of braking occurs on the breaker bar following the conditions stated in Section 1.5.9. Table 5 shows, on average, the ξ number on the six different situations. The number varies from 2.2 to 2.3 during the calm period, and it varies from 2.3 to 2.4 during the storm. Condition two occurs since the values fall within the range $0.2 < \xi < 2.5$. The plunging breaking with steep overhanging wave fronts occurs.Figure 15, right panel shows in the z-axes the water level color scale, while in x and y axes, the wave height is illustrated in offshore measurement station (ADV_C) and onshore measurement station (AQD_B), respectively. Following the breaking point equation, the effect of the breaker bar starts to diminish when the wave height reaches 0.4 [m]. Several parameters influence the behavior, such as the crest height, crest width,and wave direction. Section 3.2.3 explains these influences. Figure 15 also shows the correlation between the wave height before and after the breaking point. Before 0.4, the wave height followed the same pattern, and after the breaking point, the scatter can be seen increasing up and down the regression line.

3.2.3 Influence of Breakwater freeboard (R_c) and crest width (B)

Breakwater freeboard (Rc) and crest width (B) are recognized as significant parameters affecting the transmission of waves over and through a breakwater. Table 5 shows the transmission coefficient. In a calm period, it is on average 0.3. Whereas in the storm period, the transmission coefficient can vary between 0.3 and 0.5. Figure 16, shows the overall picture of the transmission coefficient versus the relative freeboard. This graph shows the ratio between transmitted wave height measured at B station and the incident wave height measured at C station. The range of Rc/Hi plotted is limited to -6<Rc/Hi<6. It clearly shows that Kt shows higher maximum values when R_c/H_i reached relatively large values.

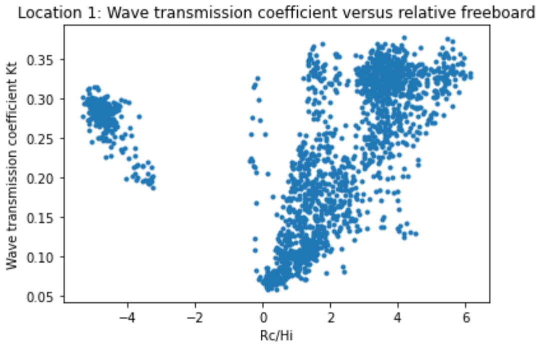

FIGURE 16: Wave transmission coefficient versus relative freeboard for the data set used in this study

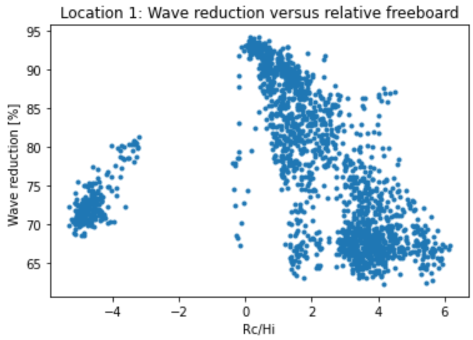

FIGURE 17: Percentage reduction of transmissivity versus relative freeboard

The calculation of the K_t is the ratio between the transmitted wave height measured at B and the incident wave height measured at C. The scatter shown on the left part of theFigure 16 and Figure 17 is from the days between 5 and 8 of march with a very high water levels of +0.3 [mNAP]. It is due to the fact that the measurement stations were not stationary, on the contrary the instruments were placed back. The right panel of Figure 16 shows the reduction percentage of the breaker bar concerning the crest freeboard ratio. Showing a negative relationship between the reduction percentage as the ration R_C/H_{m0i} increases.

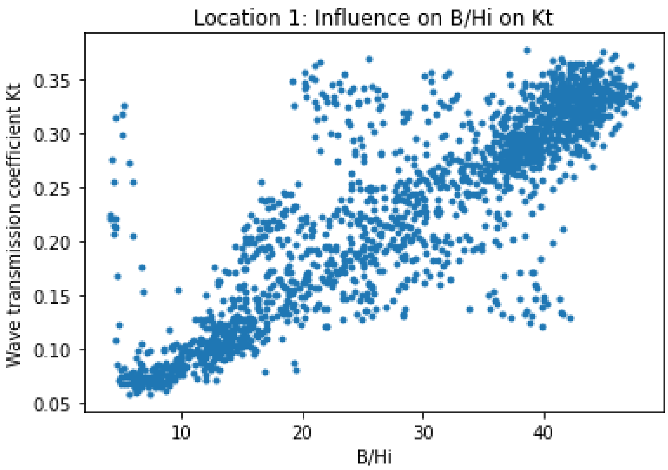

FIGURE 18: Influence of B/H_i on Kt for structures with zero freeboard

Generally speaking, Figure 18 shows a positive correlation between the crest width (B) and the incident wave height Hi ratio, measured at C and the transmission coefficient. When the B/H_i in x-axes goes up, the Kt also goes up. The crest width, on the one hand, does not change in the ratio calculation. The value of 3 [m], as Table 6 stated, was used. On the other hand, the other variable varies according to the available data, and as it decreases, the ratio will grow as the transmission coefficient.

Table 6: Summary of the averages of parameters involved in wave transmission at ADV_C

	Rc	Rc/Hi	B/L	ξ	Hi/Dn50	Hi/h	Sop
Average	0.16	1.55	1.11	2.28	0.43	0.04	0.03

3.3 Results Theoretical approach

The results show that the emerged breaker bar (OHD) is efficient at reducing wave heights [10]. The results state that the crest width in terms of wave reduction is more vital for the submerged breaker bar (OWD) than for the emerged breaker bar, while the crest height effect in terms of wave reduction is more vital for the OHD than for the OWD. When the crown of a dam is just below water level, the wave height behind the dam is limited. When the water level rises by 0.5 m and the crown of the dam disappear further underwater, a more significant wave remains behind the dam. However, the increase in the height of the residual wave is not as strong for the second ascent of 0.5 m. This explains why the crest height effect in terms of wave reduction decreases more for the OHD. Wave reduction percentages can be directly translated to residual wave heights behind the OHD. The lowest residual wave is 0.28 m for the OHD.

Table 7: Main findings theoretical approach [10]

Design conditions						
T (yr-1)	H.si (m)	Tp (s)	wl (m)	C.t (-)	Wave reduction (%)	H.st(m)
0.1	0.66	4.6	0.23	0.42	58	0.28
1	0.93	5.1	0.57	0.54	46	0.50
10	1.19	5.6	0.90	0.62	38	0.74
50	1.37	6.0	1.14	0.66	34	0.91
100	1.45	6.1	1.24	0.68	32	0.98

3.3.1 Wave growth and wave breaking on the sandy shore

The wave growth of the waves behind the fore dams for the OHD (at approximately 320 m from the sandy reinforcement) is up to 1-2 cm. The process of wave reduction through the fore water dams is thus dominant (compared to wave growth behind the front dam) in the residual wave load on the sandy reinforcement of the Houtribdijk. The residual wave load varies from 0.29 m to 1.00 m for the 1/0.1 year storm and the 1/100 year storm, respectively. Table7 shows that the spilling wave breaking occurs for all design conditions. This indicates a gradual decrease in the wave load on the sandy bank. The depth at which waves break is also calculated employing equations 14 and 3. As can be seen from the results, the waves start to break just below 0 [mNAP] water level.

TABLE 8: Main changes on the five design conditions[10]

Design conditions		OHD							
T (yr-1)	U_{10} (m/s)	Hst (m)	Hsgrowth (m)	H.s HRD (m)	ξ(-)	Zb (m)	wl(m)	Δwl %	$\Delta Hs, HRD(\%)$
0.1	16.1	0.28	0.01	0.29	0.15	-0.14	0.23	-	-
1	18.7	0.50	0.01	0.52	0.14	-0.10	0.57	147	79
10	21.3	0.74	0.01	0.75	0.13	-0.06	0.90	58	44
50	23.1	0.91	0.02	0.92	0.13	-0.04	1.14	27	23
100	23.8	0.98	0.02	1.00	0.13	-0.04	1.24	9	9

3.4 Comparison with forecast before construction

Table 8 shows the results [10] to estimate the effects of the breaker bar before the construction. In this section, the results from a return period of 0.1 (jr-1) are considered, since these conditions best match the most energetic conditions during survey period. Table 8 shows the results from Stefan, stating that the wave reduction is 58% with a wave height of 0.66 [m]. Results vary with the measured at the highest period since a reduction percentage of 71% for a wave height of 0.654 [m] is found. The theoretical approach has three main factors that affect the transmissivity of the wave load.
First, The crest freeboard, in design condition it is stated to be 0.3 [mNAP]; however, in the analysis of the results, the crest freeboard mainly depends on the variation of water level concerning NAP. The second factor is the water level. It can be seen from the measurements that the theoretical approach has a 0.23 m, whereas the data gives a value of -0.16 [mNAP]. The third factor is the ξ number, which states which breaking types do occur in the breaker bar slope, being 0.15 and 2.44, for the theoretical approach[10] and the gathered data respectively. Then, it can be said that the primary breaking is by spilling for the theoretical approach and the measured data by plunging breaking.
Finally, it can be concluded that the percentage is lower since the water level is high enough that the breaker bar acts as a submerged structure, allowing more of the incident wave load to go to the lee with a dominant transmission over the crest. In the gathered data, the breaker bar, on the contrary, behaves as an emerged breaker bar. Therefore the incident wave load will still have a crest freeboard to break the wave load and dissipate the energy. Hence the percentage of reduction is higher.Table9 shows the results of the preliminary research and the findings of the gathered data in calm and storm periods.

Table 9: Data theoretical approach

Study	T (jr-1)	Hs,i (m)	Tp (s)	wl (m)	Ct (-)	Golfreductie (%)	Hs,t (m)	Hs growth (m)	ξ
(Pluis, 2019)	0.1	0.66	4.6	0.23	0.42	58	0.28	0.01	0.15
Highest period		0.654	4.57	-0.16	0.28	71.83	0.18	0.0158	2.44
March 5-10		0.092	1.53	-0.16	0.34	85.76	0.013	0.010	2.22
March 10		0.23	2.56	-0.16	0.41	80.46	0.050	0.011	2.34
March 11		0.39	3.44	-0.16	0.46	76.52	0.097	0.047	2.39
March 12		0.25	2.69	-0.16	0.42	79.69	0.051	0.051	2.33
March 13		0.36	3.22	-0.16	0.45	77.37	0.085	0.018	2.35
March 13-27		0.10	1.72	-0.16	0.33	84.80	0.017	0.023	2.27

To understand the differences between both locations at the Markermeer lakeside, it is essential to understand that the primary mechanism which generates waves is wind, where several parameters, among them, wind speed (U), Fetch (F), wind duration (t), water level (h) affect the wave growth. The range of the parameters is summarized in Table 4. Results showed that the wave heights were higher at location 2 than location 1. Since the water depth does not vary too much due to the shallowness of the lake, the fetch length and wind speed affect the wave height. In location 2, the fetch length is higher than the fetch length in location 2. The wind speed, as shown in Table 10, is also higher at location 2 than in location 1 at a ratio of 9:6 on average.

Table 10: Summary of the estimated and measured results

Location	Effective Fetch length	Wind speed [m/s]	Water level [mNAP]	Calculation and figures
Loc1: FL67	12 [Km]	8-10 [m/s]	-2.84 [mNAP]	Appendix A
Loc2: FL68	6.7 [Km]	6-8 [m/s]	-3.5 [mNAP]	Appendix A
Loc2: FL68	0.1 [Km]			Distance from the breaker bar to the shoreline

Generally speaking, the wave transformation follows the same pattern at location 2 and location 1. The difference is the effect of the breaker bar in the transformation of the wave heights. In location 2, the drop of wave height is shallow and the main factor that affects the change is the variation of water depth since it decreases as the waves go landward to the shoreline. The wave energy has a considerable variation between the location 1 and location 2, from 3.4 [J/m2] to 81.5 [J/m2], respectively. Hence, it can be said that the effect of the breaker bar on the reduction of wave transmission is dominant, even considering that the transmitted wave height increases again due to the still existing fetch length of 0.1 [Km] behind the breaker bar at a speed of 6-8[m/s]. The section behind the breaker bar, going towards the shoreline, needs to be evaluated. As the study area is a lake, the factor with the most influence over the wave growth will be the wind direction and wind speed. In other times, the values change abruptly due to the storms that happened. The storm from March 10th to March 11th had gusts of wind of 75 km/h. Also, on March 11th, the wind speed increase to 100 Km/h, and then on March 13th, the lake experimented wind speeds between 90 km/h and 100 km/h [16].

4 Discussion

4.1 Wave transmission

Breakwater freeboard (Rc) and crest width (B) are recognized as major parameters affecting the transmission of waves over and through a breakwater. When overtopping can occur there are two dominant modes of wave transmission past a detached breakwater through a permeable core or over the crest [8]. There are three different conditions depending on the R_c. The first one is when the ratio is less than 0, when the breakwater acts as a submerged structure. In section 3 it was explained that the left part of Figure 16, and Figure 17, was not considered since the water level during the period March 5th until March 8th are not reliable. Then condition (2) and condition (3) applies.

1. Rc/Hm0$<$0) the dominant mode is the transmission over the crest.

2. 0$<$Rc/Hm0$<$2 If the structure is emerged, the dominant mode is by wave run up and overtopping against.

3. Rc/Hm0$>$2 for higher structures, transmission is through the break bar.

The behavior is divided between the second and third condition since it acts all the time as a low crested rubble mound breaker bar. Then in calm periods the condition (3) is followed, the main behavior of the transmission is through the breaker bar. Finally during storm conditions, from March 10th to March 13th the main condition is the second one, since the dominant mode is wave run up and overtopping. Therefore, wave transmission through the breaker bar and overtopping are the two phenomena that allow wave energy to pass over or through the breaker bar. It can be said that the theoretical approach was right as considering these two conditions as the main transmission modes. As the ratio between crest width and incident wave height increases, which only depends on the incident wave height since the crest width is considered 3 m constant on the calculation. Therefore, the influence is based on the incident wave height, if it decreases the ratio becomes bigger and the transmission coefficient increases showed in Figure 18.

4.1.1 Influence of wave direction

The incident angle is the wave direction measured at measurement station C offshore the breaker bar. The angle of incidence did not show up substantial influence for rubble mound structure, it can be concluded that the angle of wave attack has no or only marginal influence on the wave transmission coefficient. The influence of wave angle incidence was also followed in a study [6] establishing that for the lowest crest heights, there is a small tendency that the transmission coefficient decreases with increasing angle of wave attack.

5 Conclusion

Field measurements of wave conditions were made during a period of 22 days of a detached rubble mound breaker bar at Houtribdijk dike, on the Markermeer side in The Netherlands. The data were used to evaluate the performance in terms of transmissivity over and through the rubble mound breaker bar. The results led to the following conclusions:

1. The difference between the incoming wave at FL67 and FL68 is the wave height difference at a ratio of 7:6. For the parameters peak period (Tp) and water level (h), the FL67 data is higher than the Fl68 values. Considering the general local conditions, such as Fetch length, water level, and wind speed, such difference can be explained. At location 2 the Fetch length is 11 [Km] and 6.75 [Km] for location 1. Another difference is that at FL 68, and FL67 the water level varies from -0.4[mNAP] to -0.5[mNAP], respectively. Furthermore, another difference is the wind speed at the trend of both locations. There is a positive correlation between wave height and water level, but the scatter shows in the z color scale the wind speed being higher at FL67 with 16 [m/s] whereas FL68 shows 14 [m/s]. Therefore, FL67 with a lower water level, higher fetch length, and wind speed reaches a maximum wave height of 1.2 [m], and FL68 with a bit higher water level, lower fetch length, and low wind speed, reach a wave height of maximum 0.85 [m].

2. The wave reduction over the breaker bar decreases to 71% in the highest storm period on March 10th with a wave height of 0.65 [m] and a peak period of 4.5[s], leading to a transmitted wave of 0.18 [m] landwards. For location 2, there is no breaker bar, and the breaking factors are the shallow water and the friction of the bed. In this area the wave height also decrease from 0.93 [m] to 0.89 [m]. The comparison is made at the same extreme conditions and with the same instruments, ADV_C and ADV_A. During the calm period, non-storm conditions, a reduction of 75-85% was generated on the hydraulic load.

3. In the lee (area behind the breaker bar) the results of the increment of wave height between AQD_B, measurement station onshore the breaker bar and ADV_A, the measurement station near the shoreline demonstrated that the wave height increases due to the wind (local fetch). Increasing in average from a wave height of 2 [cm].

4. The water level affects the transmission of the wave load on the breaker bar. The water level is used to calculate the crest freeboard of the breaker bar and analyze the ratio between the crest freeboard and the incident wave, measured at C, offshore the breaker bar. As stated in the theoretical approach and based on the results of Section 3.3.1. the crest freeboard explains the type of transmissivity on the breaker bar, concluding that the main modes in this research study are overtopping and transmission through the structure. Then, as the water level decreases, a higher crest freeboard occurs, and as the ratio, RcHmoi increases, the reduction transmission percentage decreases. Consider a water level of -0.4 [mNAP], then the crest freeboard will become 0.4, and with an incident wave height of 0.1 [m], the ratio will be 4, following figure 13 a transmission coefficient between 0.20 and 0.35 will result in a reduction of 75% approximately.

5. The influence of the wave direction offshore the breaker bar is a slight tendency to decrease the transmission coefficient with increasing incident wave angle. Onshore the breaker bar is not possible to evaluate since the AQD does not measure the wave angle. Figure 17 in section 3.3.2. shows the direction of the incoming waves. It

is essential to establish the variation between 50 and 100 degrees, but some storm periods during this range do occur. The remaining graph shows the zero correlation, which decreases slightly as wave angle increases.

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