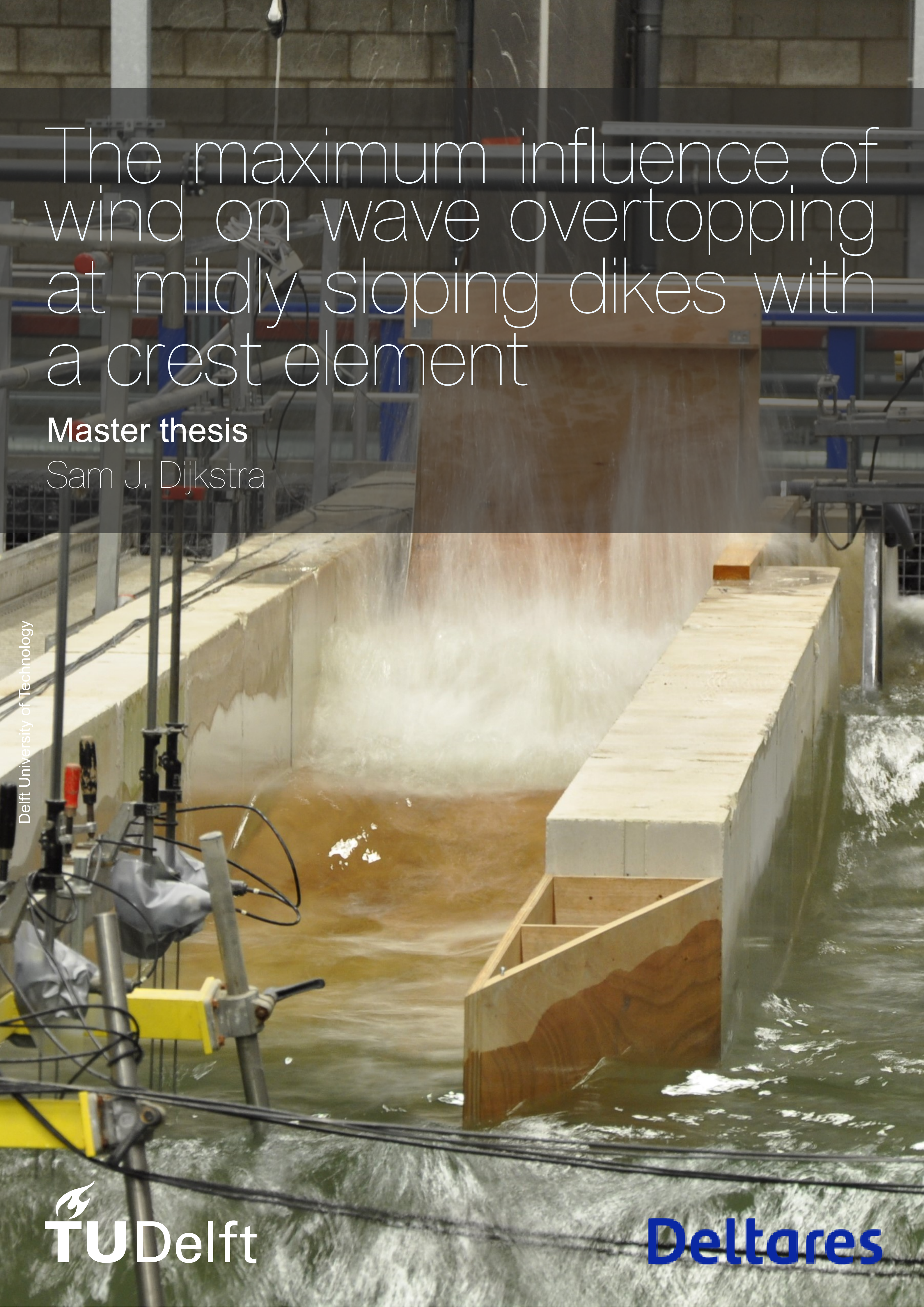

The maximum influence of wind on wave overtopping at mildly sloping dikes with a crest element

Master thesis

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The maximum influence of wind on wave overtopping at mildly sloping dikes with a crest element

by

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This thesis is confidential and cannot be made public until August 31, 2024

Preface

Dear reader,
After a year of hard work I present to you my master thesis, which is written as a finalization of the Civil Engineering master at the TU Delft. The research was conducted on behalf of Deltares, who provided an internship allowance and their testing facilities. I would like to thank Deltares for this. Furthermore, I would like to express my gratitude to model technicians Wesley Stet and Peter Alberts for constructing the scale model and assisting with the measurements. Also I want to thank the members of the assessment committee for their feedback throughout the project. Lastly I want to thank Ruben van der Bijl for making his data available for this research, this made it possible to include an analysis on a broad data-set.

Sam J. Dijkstra
Delft, August 2023

Summary

The rising sea level in combination with land subsidence, resulting in an increase in relative sea level rise, is becoming an increasing problem for the dikes in the Netherlands. When the dikes are not reinforced to account for this increase in loads, there is a risk that the dikes do not satisfy the Dutch standards anymore in the future (van Gent, 2019). A consequence of this is that the safety of the dikes can no longer be guaranteed, making the probability of failure of the dikes increase and thus the chance of a flooding of the hinterland more likely.

Using a crest element on top of the dike is an effective and relatively cheap way to reinforce the Dikes (Hogeveen, 2021). When a crest element is used, an overtopped wave can generate spray when the jet generated by wave breaking impacts on the crest element. This spray can be transported by the wind over the crest element, causing additional overtopping due to the wind (see figure 1), which can increase the overtopping discharge by a factor 6.3 (Wolters and van Gent, 2007) . The wind influence is most severe in the medium to low overtopping regime (a dimensionless discharge $q^*[-] < 10^{-3}$, mainly relevant for the users of the dike (e.g. pedestrians)), because for higher discharges the amount of water flowing over the crest element is large, leading to little to no impact on the crest element and thus limited spray generation.

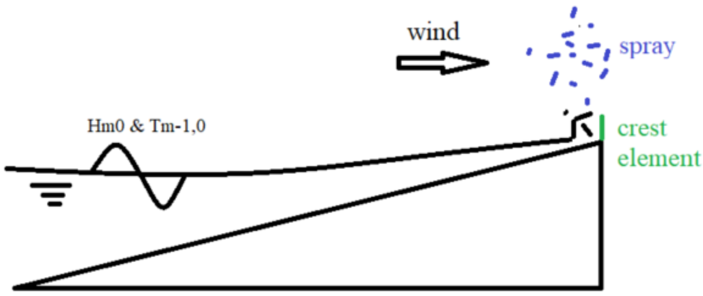

Figure 1: Schematic of wind influenced overtopping

The aim of the research was to better understand this influence of the wind on wave overtopping on dikes with a crest element. Therefore, small scale physical model tests were performed on a relatively mild dike slope of 1:6 with a crest element, resulting in breaking waves on the dike slope (breaker parameter $\xi_{m-1,0}$ < 2). Because there are no scaling rules for the wind, the maximum influence of the wind was studied for which no scaling of the wind is needed. This was done by using a rotating paddle wheel that transports the water that sprays up in the air over the crest element and therefore simulates maximum wind conditions. Note that with maximum wind conditions, all the water that sprays up in the air is transported over the crest element by the wind (/paddle wheel).

In the research, two heights of the crest elements were tested and the tests were performed with and without a promenade in front of the crest element. Furthermore a data-set of a former study done by van der Bijl (2022) was used, the only difference being the relatively steep dike slope of 1:3 he tested with. By combining the two data-sets, the influence of the dike slope was studied.

Analysis of the data-set obtained in this research (1:6 slope) resulted in wind influences varying from 1.2 till 5.74, with the wind influence mainly depending on the magnitude of the dimensionless overtopping discharge (a smaller discharge leads to a higher wind influence). With the wind influence defined as q_w/q, with q_w being the wind influenced overtopping discharge and q the overtopping discharge without wind. Moreover, the higher crest element generally lead to higher wind influences. For the promenade it was found that, in combination with the higher crest element, it lead to an increase in wind influence.

In combination with the lower element, no difference in wind influence with/without a promenade was found. Regarding the dike slope, generally no differences in wind influences between the relatively steep (1:3, van der Bijl (2022)) and mild (1:6) dike slope was found. Because no noteworthy differences were found between the two dike slopes, a general wind amplification factor was determined based on the combined data-set containing both slopes. The factor is an improvement on the factor defined by van der Bijl (2022)/van Gent et al. (2023):

$$\gamma_{wind}\,(bestfit) = 0.051\; q^{*-0.281} + 1 \tag{1}$$

$$\gamma_{wind}\,(bestfit) = \begin{cases} 0.0665\; q^{*-0.217} + 1, & \text{if } 0.25 \leq h_w^* \leq 0.40 \\ 0.102\; q^{*-0.231} + 1, & \text{if } 0.40 < h_w^* \leq 0.80 \end{cases} \tag{2}$$

With $q*$ the mean dimensionless overtopping discharge [-], $h_w^* = h_{wall}/H_{m0}$, h_w^* the dimensionless crest element height, h_{wall} the crest element height [m] and H_{m0} the significant wave height [m]. As can be seen, 2 factors were formulated. For the second formulation the improvement was especially gained for the low interval ($0.25 \leq h_w^* \leq 0.40$). Quite some uncertainty was found in the data, to deal with this for design purposes, 1 standard deviation can be included in the formulations.

Lastly, the data obtained in this research without wind was useful to comment on the reduction effect of the crest element and promenade in combination with breaking waves. For the promenade (in combination with a crest element) a constant reduction factor of 0.96 was found. For the crest element in combination with breaking waves it was concluded that, regarding the data obtained in this research, it is best to use no reduction factor. So the crest element only contributes to the crest freeboard. In contrary, Van Doorslaer et al. (2016) found that it is best to use a reduction factor of 0.92. No noteworthy differences were found between the two studies, so further research is needed on this topic.

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Nomenclature

Symbols

Symbol	Definition	Unit
q	Mean overtopping discharge discharge	[m^3/s/m]
q_w	Mean overtopping discharge discharge with maximum wind	[m^3/s/m]
q^*	Dimensionless overtopping discharge	[-]
q^*_w	Dimensionless overtopping discharge with maximum wind	[-]
$H_{m0}(= H_{m0i})$	(Incoming) significant spectral wave height	[m]
$T_{m-1,0}$	Spectral wave period	[s]
$s_{m-1,0}(= s_0)$	Wave steepness base on H_{m0} and $T_{m-1,0}$	[-]
h_{wall}	Height of the crest wall	[m]
$tan(\alpha)$	Seaward dike slope	[-]
B	Promenade width in front of the crest wall	[m]
g	Gravitational acceleration	[m/s^2]
R_c	Crest height above still water level	[m]
$\xi_{m-1,0}(= \xi_0)$	Breaker parameter/Irribarren number	[-]
γ_b	Berm reduction factor	[-]
γ_f	Reduction factor for roughness	[-]
γ_β	Reduction factor for oblique wave incidence	[-]
γ_v	Reduction factor for a vertical wall on a dike slope	[-]
d_h	Water level at the center of the berm	[m]
B_{berm}	berm width	[m]
L_{berm}	Breaker parameter/Irribarren number	[-]
x	Parameter to account for the berm position with respect to water line	[m]
β	incoming wave angle	[°]
$L_{m-1,0}$	Wave length corresponding to $T_{m-1,0}$	[m]
γ_{prom}	Reduction factor for a promenade	[-]
γ_{prom_v}	Reduction factor for a promenade and crest wall combined	[-]
d	Water depth	[m]
T_p	Peak period	[s]
u	velocity scale	[m/s]
L	Length scale	[m]
n	Scale factor	[-]

1
Introduction

1.1. Motivation

Recently in February 2022 storm ”Eunice” reached the Netherlands. The storm with wind gusts up to 145km/h reached the Netherlands (KNMI, 2022). No noteworthy damage to the dikes was observed, but it made people think of an important day in Dutch history: January 31st 1953, the day is also referred to as ”de Watersnoodramp”. That day 1836 people died and 72000 citizens were evacuated from their homes (Rijkswaterstaat, n.d.). Heavy winds up to 97km/h with wind gusts up to 144km/h, in combination with spring tide, caused an extreme storm surge (KNMI, n.d.). Many dikes were not designed on the extreme conditions reached that day, leading to dike breaches and a flooding of 1500 squared kilometer of the hinterland.

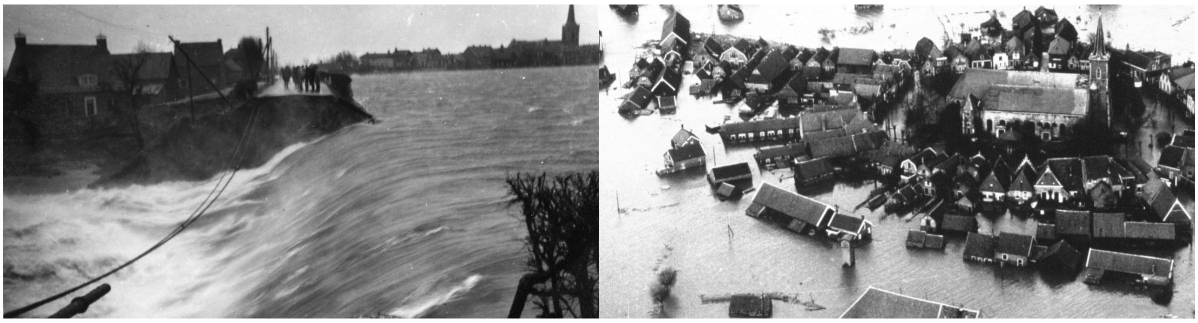


Figure 1.1: Dike breaching during the ”Watersnoodramp” (Rijkswaterstaat, n.d.)

The disaster was the start of high standards regarding the protecting of the Netherlands against the water. Also plans were made for the famous Dutch Deltaworks, which still contributes to the protection of the Netherlands against the water. This year, 70 years later, the disaster is still nationally memorized and the importance of protecting the Netherlands against the water in the future is shown.

To make this protection in the future possible and not be surprised by the magnitude of the extreme conditions like in 1953, there is a need to account for the sea level rise. The rising sea level in combination with land subsidence, resulting in an increasing relative sea level rise, is becoming an increasing problem for the dikes in the Netherlands. Due to the increase in relative sea level rise in the future, the loads on dikes increases. When a dike is not reinforced to account for the increase in loads, there is a risk that dikes in the Netherlands do not satisfy the dutch standards anymore in the future (van Gent, 2019). A consequence of this is that the probability of dike failure increases and thus the probability of a flooding of the hinterland increases. To keep the chance of a flooding of the hinterland as low as it is at the moment, the dikes must be reinforced by for instance increasing the crest height. Increasing the crest heights for all the dikes in the Netherlands is an expensive measure and takes a lot of time. Next to that, to increase the crest height, the dike needs more space, space that is scarce in the Netherlands. This makes it interesting to look at other alternatives (see e.g. van Gent, 2019). Part of the alternatives is the use of a crest element.

A crest element on top of a dike is an effective and relatively cheap way to reinforce the dikes (Hogeveen, 2021). However with a crest element, an overtopped wave can generate spray, which than can be transported by the wind, causing wind influenced overtopping (Wolters and van Gent, 2007) (see figure 1.2). Currently the design methods ignore the effect of wind which can lead to an underestimate of the amount of wave overtopping and the required crest level.

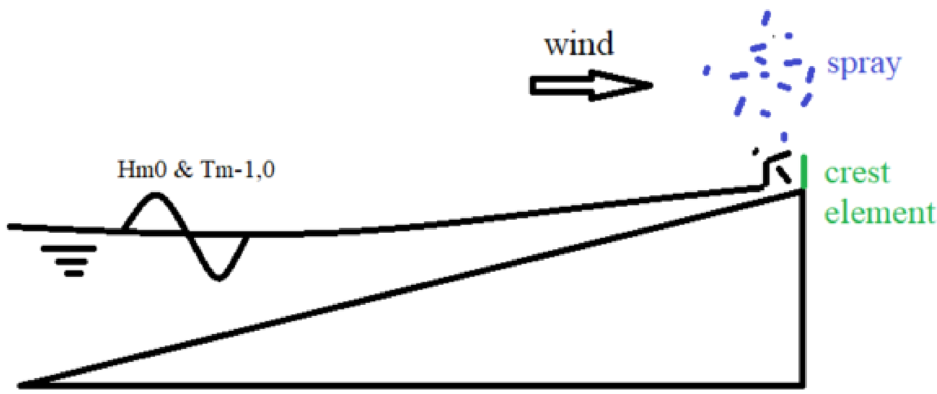


Figure 1.2: Schematic of wind influenced overtopping

The wind influence is the largest during stormy conditions, like in 1953 and with the recent storm "Eunice". During a storm the wind speeds are most severe and when the wind blows onshore, the wind leads to a set-up of the water level and an increase in the wave height. This in combination with a high tidal water level leads to decisive circumstances with overtopping discharges which are possibly increased by the influence of wind.

Some research is done on the influence of wind on wave overtopping, resulting in influence's varying from $q_w/q = 1.3$ till 6.3, with q_w being the wind influenced overtopping and q the overtopping without wind (Wolters and van Gent, 2007). From their study was concluded that the wind influence was most severe in the medium to low overtopping regime (a dimensionless discharge ($q^*[-]$) between E-3 and E-6). This regime is relevant for the safety of the users of the dike (e.g. pedestrians) and can cause damage to structures behind the dike. The range of wind influences can partly be explained by the structure types studied, but it also indicates that the influence of wind on wave overtopping is not yet fully understood. Further research can contribute to the understanding of the influence of wind on wave overtopping.

More recently a study was done by a colleague master student (van der Bijl, 2022) on the influence of wind on overtopping. He did small scale physical model tests on a dike slope of 1:3, resulting in non-breaking waves on the dike slope ($\gamma_b * \xi_{m-1,0} >\approx 2$, with γ_b the reduction factor for a berm and $\xi_{m-1,0}$ the breaker parameter), with a crest element and an optional promenade in front of the crest element. He found wind influences varying from 1 to 4 and he showed the reducing potential of a crest element and promenade for non-breaking waves. Because the outcome of van der Bijl (2022) is limited to non-breaking waves on a relatively steep 1:3 slope, this research will shift the focus to breaking waves ($\gamma_b * \xi_{m-1,0} <\approx 2$) on a relatively mild 1:6 slope. The crests elements and promenade tested with will be the same as the ones van der Bijl (2022) used, which makes it possible to compare the results of this study with his results.

1.2. Objective

In this research physical model tests are performed on a relatively mild dike slope of 1:6, resulting in breaking waves on the dike slope. The tested configurations are with 2 different crest element heights, with and without a promenade. The goal of the research is to extend the limited knowledge on the influence of wind on wave overtopping at dikes with a crest element. To extend the knowledge, attention is given to the following topics:

Crest element height and promenade in front of the crest element
The influence of the location of the crest element (with and without a promenade) on the additional amount of overtopping due to wind is studied. Next to that, the influence of the height of the crest element and the combined effect of the promenade and the height of the crest element is studied. This results in the following research questions:

With respect to the maximum influence of wind on wave overtopping on mildly sloping dikes:

- What is the influence of the promenade in front of a crest element?
- What is the influence of the height of the crest element?
- What is the combined effect of the crest element height and promenade in front of the crest element?

Wind effect on steep versus flat slopes with a crest element
The influence of the slope on the additional amount of overtopping due to wind will be studied. This results in the following research question:

- With respect to the maximum influence of wind on wave overtopping on mildly sloping dikes, what is the influence of the seaward slope angle of the dike?

1.3. Method

The goal of the research is to better understand the influence of wind on wave overtopping. To do so, dike configurations with the varying components: with/without a promenade, seaward slope angle and crest element height are studied. A data-set is needed including those components, with and without wind. To obtain these data, physical model tests are performed. The output of these tests are analysed and compared with literature. This analysis forms the basis for the conclusions with respect to the research questions. For a more detailed description of the method is referred to chapter 3.

1.4. Report outline

Chapter 2: Literature study
This chapter starts with a general description of wave overtopping and introducing the wave overtopping formula that is used as a theoretical background. Different factors can have an influence on the overtopping discharge, these factors including the crest element en promenade are treated. As last the knowledge available about the influence of wind on wave overtopping are part of this chapter.

Chapter 3: Physical model tests
In chapter 3 the method used to find an answer to the research questions are described. The choice for a small scale physical model are explained. Furthermore the set-up of the model, hydraulic conditions, test program and scaling of the model are part of this chapter.

Chapter 4 & 5: Data Analysis & discussion
The analysis of the data obtained during the physical model tests are split in two chapters. In chapter 4 the focus is more on the dimensionless wave overtopping discharges, to see if this is in line with the expectations based on TAW (2002) and EurOtop (2018). In chapter 5 the focus is shift to the wind influence, defined as the overtopping discharge with maximum wind influence divided by the overtopping discharge without maximum wind influence. This chapter is finalised with the determination of an improved wind amplification factor.

Chapter 6: Conclusion and recommendations
The core of the report is finished with an overview of the conclusions that are drawn from the analysis and discussion of the data. Moreover recommendations for further research are given.

2

Literature study

The literature study starts with explaining the relevance of wave overtopping and the methods available to predict the amount of wave overtopping. An empirical method is selected (TAW, 2002) to analyse further. For the most aspects influencing overtopping, TAW (2002) uses reduction factors. Reduction factors (e.g. for roughness or a crest element) are applied inside the exponential part of the empirical formula (2.2) and are usually not larger than 1, a factor of 1 is the same as no reduction. A factor higher than 1 would mean additional overtopping due to an element. It will be briefly described how those reduction factors should be determined following TAW (2002), after which more recent literature will be analysed for each reduction factor. From the recent literature is concluded that most reduction factors from TAW (2002) are not taking into account all parameters that are relevant for the specific reduction factor. Taking into account all relevant parameters can improve the accuracy of a reduction factor, which can lead to a more efficient design. The last reductive elements that will be treated are a crest element and a promenade, which will be studied in this research (in combination with the wind influence). Finally the literature on the influence of wind will be studied, a distinction will be made between the influence of wind without a vertical structure and with a vertical structure. Because as will become clear, a vertical structure is essential for the wind to become important. Finally the relevance of this research will be summarized as a consequence of the studied literature.

2.1. Wave overtopping

Wave overtopping is one of the key processes to take into account in dike design as it is one of the main processes causing dike failure (Chen et al., 2022). The overtopping discharge can erode the landward slope of a dike, this slope is designed on less severe loads and is therefore typically a weak part of a dike. An example of failure of the landward slope of a dike due to wave overtopping is shown in figure 2.1. This damage was caused by the release of a relatively large overtopping volume on the landward grass covered slope of a sand dike (EurOtop, 2018). In this research the focus will be on relatively small overtopping discharges, as for those amounts the wind influence is the largest (Wolters and van Gent, 2007). The relatively small overtopping discharges can also lead to the failure mechanism depicted in figure 2.1, however this is less likely due to the smaller loads. The relatively small amounts of overtopping are more relevant for the safety of the users of the dike (e.g. pedestrians) and the small amounts can cause structural damage to buildings behind the dike (Wolters and van Gent, 2007).

Figure 2.1: Example of dike failure due to overtopping with a simulator (EurOtop, 2018)

As explained, wave overtopping is an important process to take into
account in the design of a dike. There are multiple methods available to predict the amount of overtopping: Empirical models, analytical models, PC-overtopping, Eurotop database, Eurotop Neural Network prediction tool, numerical modelling and physical modelling (EurOtop, 2018). The empirical models will be further analysed, because an empirical model gives best insight in the factors influencing wave overtopping and it forms a good theoretical basis as comparison for the physical model data that will be collected further on in this research.

In empirical models the mean overtopping discharge (q) typically is the main parameter. Although in reality wave overtopping is not a continuous process. Between 2 overtopping waves can be for instance 10 waves that do not contribute to the overtopping (EurOtop, 2018). As Koosheh et al. (2021) explains, this 'wave-by-wave' way of looking at overtopping can be included by studying the following parameters: individual maximum overtopping volume, overtopping flow velocity and thickness over the crest or overtopping flow velocity and thickness over the landward slope. Especially the overtopping flow velocity and layer thickness are found to be the most important parameters causing failure at the landward dike slope (Schüttrumpf, 2001; van Gent, 2002a; van Gent, 2002b; Schüttrumpf and Oumeraci, 2005). So corresponding dike failure, individual overtopping waves are more relevant than the mean overtopping discharge.

In this research the influence of wind is mainly studied by measuring the mean overtopping discharge with physical model tests. Looking at the mean discharge over the test duration makes it easier to compare data with/without wind and with other researches like the one of van der Bijl (2022). Next to that, the mean overtopping discharge is still an important parameter used in dike design. Because typically when a dike is designed, a maximum allowed mean overtopping discharge is defined, generally varying from 0.1 l/s per m to 10 L/s per m. Mainly depending on the structure type/cover, the maintenance of the cover and the significant wave height (TAW, 2002;EurOtop, 2018).

To study the mean overtopping discharge using an empirical model, multiple possibilities are available. EurOtop (2018) mentions 2 formulas:'old' (TAW, 2002) and 'new'(EurOtop, 2018) for sloping structures, both generally formulated as follows:

$$(q^* = \frac{q}{\sqrt{g * H_{m0}^3}}) = a * exp(-(b\frac{R_c}{H_{m0}})^c) \tag{2.1}$$

With on the left hand side the dimensionless discharge q^* $[-]$, determined by the mean discharge q $[m^3/s/m]$, the gravitational acceleration g $[m/s^2]$ and the spectral significant wave height H_{m0} $[m]$. On the right hand side the crest height above still water level R_c $[m]$ and the wave height, both forming the (dimensionless) relative crest freeboard. a, b, c are calibration factors. In the old formula, c is equal to 1, which makes the equation an exponential function. In the new formula the coefficient is equal to 1.3, making it a Weibull shaped function. den Bieman et al. (2021), who compared different methods to predict wave overtopping, concluded that the TAW (2002) has a higher accuracy than the EurOtop (2018) formula. Important to note is that they also noted that both formula's result in a large amount of scatter. The reliability of overtopping predictions (90% confidence interval) shows to be a factor 2.5 for large amounts of overtopping and 20 for small amounts of overtopping (Van Doorslaer et al., 2016). As the TAW (2002) formula has the highest accuracy, this formula will be further analysed and used as theoretical background in this research.

2.2. Empirical design formula and reduction factors

2.2.1. Empirical design formula

The TAW (2002) formula for breaking wave conditions is given in equation 2.2 with its maximum for non-breaking wave conditions shown in equation 2.5. In TAW (2002), breaking waves are defined as $\gamma_b * \xi_{m-1,0} <\approx 2$ and non-breaking waves as $\gamma_b * \xi_{m-1,0} >\approx 2$. Where breaking refers to breaking on the slope, not to wave breaking on the foreshore. For severe wave breaking on the foreshore, the mentioned overtopping formula is not valid.

$$\frac{q}{\sqrt{g * H_{m0}^3}} = \frac{0.067}{\sqrt{tan\alpha}} * \gamma_b * \xi_{m-1.0} * exp(-4.75 * \frac{R_c}{\xi_{m-1.0} * H_{m0} * \gamma_b * \gamma_f * \gamma_\beta * \gamma_v}) \quad (2.2)$$

$$\xi_{m-1.0} = \frac{tan\alpha}{\sqrt{s_0}} \quad (2.3)$$

$$s_{m-1,0} = \frac{2\pi H_{m0}}{gT_{m-1,0}^2} \quad (2.4)$$

With maximum for non-breaking wave conditions:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = 0.2 * exp(-2.6 * \frac{R_c}{H_{m0} * \gamma_f * \gamma_\beta * \gamma_v}) \quad (2.5)$$

In this formula, $tan\alpha\ [-]$ is the seaward dike slope, $\xi_{m-1,0}\ [-]$ the Iribarren number(/breaker parameter), $s_{m-1,0}\ [-]$ the wave steepness, $T_{m-1,0}\ [s]$ the spectral wave period and $\gamma_b, \gamma_f, \gamma_\beta, \gamma_v\ [-]$ reduction factors for respectively the use of a berm, roughness, incoming wave angle and a vertical wall.

2.2.2. Reduction factors

"Reduction factors are always derived as the difference between a structure with and a structure without influence element" (van Doorslaer et al., 2015). An accurate reduction factor is needed because it leads to an efficient and safe design of a structure. When the reduction factor overestimates the reducing effect, there is a risk of under dimensioning the structure, which can ultimately lead to dike failure. On the other side, when a too low reduction effect is estimated with the reduction factor, there is a risk of over dimensioning the structure, which increases the costs of the structure and can increase the space needed for the structure, space which can be scarce at the project area. An accurate reduction factor can be reached by including the parameters it depends on in the equation, something that is not always the case as will be shown in this section. First a berm, roughness and oblique wave incidence will be treated. This will not be done very extensively, because these aspects will not be studied in this research. However, this will give an overall view of the most important aspects influencing overtopping and in what way they recently are improved. Which shows the relevance of studying reductive aspects in general and can give insights for the crest element and promenade studied in this research. Finally a more extensive study of literature concerning these last two aspects will be done.

Berm, $\gamma_b[-]$

A berm has a reductive effect on wave overtopping. The berm is included in TAW (2002) with a reduction factor equal to:

$$\gamma_b = 1 - \frac{B}{L_{berm}}(0.5 + 0.5cos(\pi\frac{d_h}{x}))\ with\ 0.6 \leq \gamma_b \leq 1.0 \quad (2.6)$$

The equation is valid for a berm slope between horizontal and 1:15 and a berm width lower than 1/4 times the wave length, to deal with a berm exceeding those thresholds and for a more detailed explanation of the reduction factor, is referred to TAW (2002). The meaning of the different terms in the equation are shown in figure 2.2.

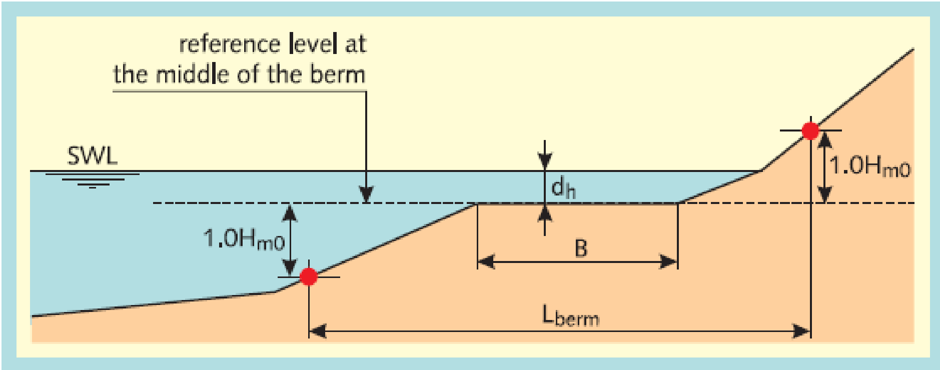


Figure 2.2: Berm reduction factor definitions (TAW, 2002)

Recent literature by Chen et al. (2020) on the reductive effect of a berm (and roughness) mentions that the reduction factor from TAW (2002) is not optimal. From their research they concluded that the permeability and wave steepness is of influence on the berm reduction in addition to the parameters as included in TAW (2002). Showing a larger reduction for a relatively low wave steepness and impermeable berm. To increase the accuracy of the reduction factor, they propose different formulas for an impermeable and permeable berm including the effect of the wave steepness. Chen et al. (2020) also studied the combined effect of the combination of roughness and a berm and proposed a method to account for this which performed substantially better than existing methods. This improvement by looking at the combined effect of reductive elements will also be obtained in the reduction factors that will be treated next. This makes it interesting to look at the combined effect of reductive elements, like the promenade and crest element studied in this research. The influence of a berm won't be studied in this research, therefore a constant dike slope will be used in the physical model tests without a berm ($\gamma_b = 1$).

Roughness, $\gamma_f[-]$
Adding roughness to a slope is commonly used to reduce the amount of wave overtopping. TAW (2002) includes the influence of roughness in their formula by applying a reduction factor. The general way given to determine the reduction factor is by consulting an extensive table which shows the reduction factor for different top layers of the slope. The content of the table is based on large scale studies with irregular waves, with values varying from 0.55 for rubble mount (large reduction) and 1.0 for smooth layers like asphalt. Essential for the effectiveness of roughness is the location and area of the slope that is covered with roughness elements, because too far away from the still water line results in little to no reduction effect. In TAW (2002) also special attention is given on how to deal with roughness elements on slopes (like blocks and ribs) and armour rock slopes.

Due to the importance of the reducing effect of the roughness on wave overtopping (Chen et al., 2020), during the years lots of research was done on the topic. A small selection of researches will be shortly treated. From Capel (2015) and van Steeg et al. (2018) can be concluded that the roughness is not constant as it seems from the table in TAW (2002), but that it depends on other factors than only the protection type. There are different ways to introduce roughness on a slope. van Steeg et al. (2018) who studied stair shaped revetments, gives a reduction factor depending on the wave height and overtopping discharge. Capel (2015)'s study about block revetments with enhanced roughness (by applying a spacial pattern of blocks) gives a reduction factor depending on the overtopping discharge and wave steepness. By selecting two studies about the roughness, it can be seen that the parameters included in the reduction factors can differ somewhat. However, both factors include the overtopping discharge in the formulation, so for example for the stair shaped revetments, the wave height is also (indirectly) included in the formulation. During this research the influence of roughness will not be studied, therefore a smooth dike slope will be used ($\gamma_f = 1$).

Oblique wave incidence, $\gamma_\beta[-]$
The angle in which the waves reach the structure reduce the amount of wave overtopping. Like van

Gent (2020) says: "Oblique wave attack can significantly reduce the amount of wave overtopping at coastal structures compared to perpendicular wave attack". The formulation used in TAW (2002) is formulated as follows (short-crested waves) with a minimum of $\gamma_\beta = 0.736$:

$$\gamma_\beta = 1 - 0.0033|\beta| \; for \; and \; 0^\circ \leq |\beta| \leq 80^\circ \gamma_\beta = 1 - 0.0033 * 80 \; for \; |\beta| > 80 \tag{2.7}$$

For $80^\circ \leq |\beta| \leq 110^\circ$ the H_{m0} and $T_{m-1,0}$ are additionally multiplied in equation 2.1 with a reduction factor dependent on the wave angle.

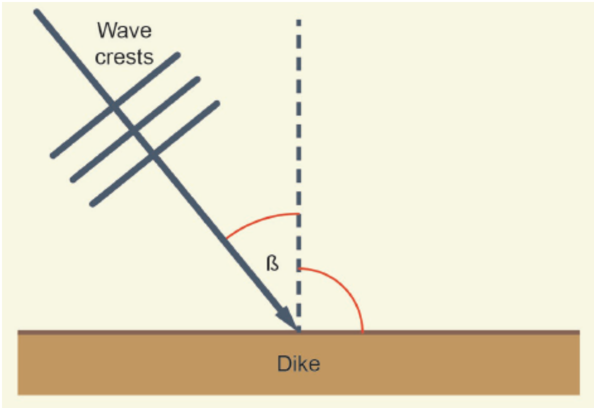


Figure 2.3: Definition oblique wave angle (TAW, 2002)

New researches say the reductive effect of oblique wave incidence can be predicted with a lower error (van Gent and van der Werf, 2019 and van Gent, 2020), by taking into account the combined effect with other reductive dike elements like a berm and roughness. The combination of oblique wave incidence and for instance a berm can result in severe overestimation of the wave overtopping when using the method from TAW (2002) (van Gent, 2020). In van Gent and van der Werf (2019) and van Gent (2020), a new formulation for the reduction factor for oblique wave incidence is proposed which is dependent on the angle (β) and non-dimensional berm width (B/H_{m0}), which clearly outperformed the γ_β formula from TAW (2002) and other methods. The method was consistent with the test data of multiple dike configurations. In this research the influence of oblique wave incidence won't be studied, therefore tests will be performed with normally incident waves ($\gamma_\beta = 1$).

Crest element, $\gamma_v[-]$

For the use of a crest element to reduce the amount of overtopping, TAW (2002) prescribes a reduction factor of 0.65 for a vertical wall on a slope (so this doesn't have to be on the top of the dike), when the wall has an certain angle ($\alpha_{wall}[^\circ]$), interpolation is needed with the following equation, with a maximum of 1 for an 45 degree angle.

$$\gamma_v = 1.35 - 0.0078 * \alpha_{wall} \tag{2.8}$$

Furthermore TAW (2002) says that when using a vertical wall, the breaker parameter ($\xi_{m-1,0}$) must be determined by using an average slope of the dike including the vertical wall. This was based on a data-set in which the vertical wall was part of the dike slope and thus influencing the breaking process. In this research a crest element will be used located on top of the dike, not influencing the breaking process. The applicability of equation 2.8 is limited and depends on some dimensions of the dike and the wall, the location of the wall on the dike and the reduction factor can only be used for steep slopes between 1:2.5 and 1:3.5. Moreover, this reduction factor is only included in the equation for breaking wave conditions, what suggests that a crest element doesn't have an influence on the overtopping due to non-breaking wave conditions. So there are a lot of limitations to this approach as described in TAW (2002).

EurOtop (2018) mentions more recent studies which result in reduction factors also for non-breaking wave conditions and a wider variety of dike configurations. The added information is based on van

Doorslaer et al. (2015) and Van Doorslaer et al. (2016). van Doorslaer et al. (2015) did tests for non-breaking wave conditions with a crest element, crest element with parapet, a stilling wave basin and a promenade, which all appeared to be effective reduction measures. Multiple reduction factors are deduced depending on the dike geometry (e.g. the combination of a crest element and a promenade) and the dimensionless height of the crest element (h_{wall}/R_c).

Van Doorslaer et al. (2016) gives additional information and adds test for breaking wave conditions. They did small scale model tests with a mild smooth dike slope of 1:6 with a crest element on top of the dike. In this research tests will be performed with similar wave conditions, crest elements and the same slope. Van Doorslaer et al. (2016) state the approach from TAW (2002) to determine the breaker parameter with the average slope including a crest element, is not optimal when the crest element is relatively small and located on top of the dike. Because in that case, the crest element does not influence the breaking process.

Van Doorslaer et al. (2016) did tests for 3 different heights of the crest element, of which no clear difference was observed. This observation resulted in a constant γ_v of 0.92 for breaking wave conditions and a dike slope of 1:6, which is much higher than the factor of 0.65 mentioned before from TAW (2002). This difference can be explained by the before mentioned different configurations and conditions the factors are based on. From this is concluded that for a crest element on top of a dike, the method of TAW (2002) overestimates the influence of a crest element.

Van Doorslaer et al. (2016) recommends to use the method from TAW (2002) (including the usage of the average slope) only for $h_{wall}/R_c > 1$, so for relatively large walls with the foot below sea water level. For non-breaking wave conditions the methods proposed in van Doorslaer et al. (2015). For breaking wave conditions with a relatively small wall and the foot of the wall above the water level ($h_{wall}/R_c < 1$), the actual slope can be used and the factor of 0.92. Based on these considerations, the method as it is described in TAW (2002) is not applicable on the configurations of this research. The factor from Van Doorslaer et al. (2016) should be used, as this factor is determined based on model tests with the same configuration.

From these resources it was learnt that the influence of a crest element (vertical wall) on the amount of wave overtopping mainly depends on the presence of breaking or non-breaking wave conditions, the location of the wall on the dike with respect to the water level and for non-breaking wave conditions the relative height of the wall. Adding a promenade in front of a crest element can also have an influence.

Promenade

As mentioned, van Doorslaer et al. (2015) also studied the influence of a promenade. An example of a promenade is shown in figure 2.4. A promenade is located at crest level, in contrast to a berm which is located on the dike slope. For the influence of a promenade, no specific guidance is described in TAW (2002). Therefore van Doorslaer et al. (2015) used the berm reduction factor (γ_b) to account for a promenade, but they showed this over predicts the reduction effect for a promenade. This can be explained by the fact that the behaviour of a promenade and berm differs essentially. By the location of a berm on the dike slope, it influences the average dike slope and breaking process. A promenade is located on top of a dike, having no influence on the average dike slope/breaking process.

The reducing potential of a promenade is shown by van Doorslaer et al. (2015) and a reduction factor (equation 2.9) is proposed for non-breaking waves depending on the dimensionless promenade length ($B/L_{m-1,0}[-]$), which is the promenade width ($B[m]$) divided by the wave length ($L_{m-1,0}[m]$) corresponding to the mean spectral wave period ($T_{m-1,0}[s]$). For non-breaking waves this reducing effect slightly increases by the promenade length.

$$\gamma_{prom} = 1 - 0.47 * \frac{B}{L_{m-1,0}} \; (non-breaking\ waves) \tag{2.9}$$

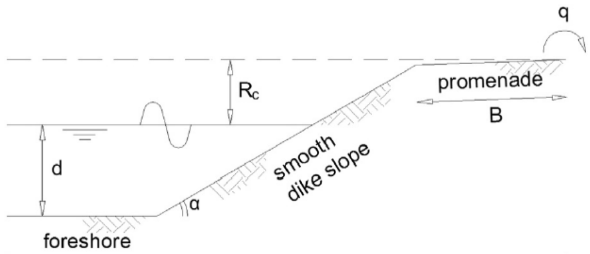


Figure 2.4: Example of a promenade (van Doorslaer et al., 2015)

For breaking waves, the method of using the berm reduction factor is given as best option in EurOtop (2018), because there is no decent alternative available. However as sated, the behaviour of a berm differs a lot from the behaviour of a promenade. Presumably, keeping the over prediction for non-breaking waves in mind, using the berm reduction factor for the use of a promenade with breaking waves will also not be accurate.

Promenade and crest element
"The physical process of a wave hitting a wall is different when a promenade is present in between the top of the dike slope and the wall" (van Doorslaer et al., 2015). Therefore it is likely that just multiplying a factor for a promenade and a crest element is not accurate. van Doorslaer et al. (2015) concluded that the combination of the two is an effective way the reduce overtopping, they propose a factor of 0.87 to the multiplication of the two reduction factors:

$$\gamma_{prom\_v} = 0.87 * \gamma_v * \gamma_{prom} \; (non - breaking \; waves) \tag{2.10}$$

For breaking waves no accurate method is available, rather than using a multiplication of the berm reduction factor and the reduction factor for a crest element.

2.3. Influence of wind on wave overtopping on dikes without a crest element

In the previous sections knowledge was gained about wave overtopping in general and the factors reducing overtopping. The focus will now shift to the influence of wind on wave overtopping, the main topic of this research.

In the empirical formulations from TAW (2002) and EurOtop (2018) the influence of wind is not included. Wind is only included indirectly, because wind set-up can cause the design water level to be higher and wind influences the wave conditions. The reason for not including the wind influence is found in literature from a long time ago. de Waal et al. (1997), who studied vertical seawalls, found that the wind influence could be relevant for low overtopping regimes and vertical structures. Because a vertical structure causes an upward directed spraying of the water which can contribute to the overtopping discharge by the wind. Ward et al. (1997), who studied a 1:3 slope without a crest element, found wind is negligible for high overtopping regimes and/or low wind speeds. Those statements were later on confirmed by Lorke et al. (2012), who tested with different wind speeds and crest levels on dikes with a 1:3 and 1:6 slope. Thus for small wind speeds and/or high overtopping discharges the influence of wind is limited, and a vertical structure is needed for the wind to become important. This explains why there was no (direct) wind influence included in the general TAW (2002) formula.

The influence of wind on wave overtopping becomes relevant for small overtopping discharges and vertical structures. The behaviour of a crest element on a coastal structure is somehow comparable to the behaviour of the vertical structure de Waal et al. (1997) did their tests on, because both causes wave motion to be in the vertical direction before wave overtopping occurs. Therefore, it is reasonable to assume that wind can also have an influence on dikes with crest elements. Wolters and van Gent (2007), who tested the maximum influence of wind on dikes with a crest element, proved this is the case.

As mentioned in the introduction, adding a crest element to coastal structures can become more interesting in the future (Hogeveen, 2021). This makes it relevant to zoom in on studies done on the influence of wind on wave overtopping on dikes with a crest element.

2.4. Influence of wind on wave overtopping on dikes with a crest element

Wind can have an influence on wave overtopping in the following way. When a wave breaks on the dike slope, a jet is generated. When the jet reaches the crest element an upward spraying motion occurs. Which can potentially contribute to the wave overtopping if the onshore wind speeds are large enough (see figure 1.2). When there is no crest element or other vertical structure that causes an upward directed spray, this wind influence is limited (Wolters and van Gent, 2007).

Not much research is done yet on the influence of wind on wave overtopping on dikes with a crest element, the before mentioned study by Wolters and van Gent (2007) is being a pioneer. In their research experiments where done on wind affected overtopping in a wave flume on dikes slopes with a crest element of varying heights. Rough and smooth sloped structures were tested with slopes of 1:2 and 1:1.5. Important to note is that this is 3 to 4 times steeper than the 1:6 slope of this research. Nonetheless, the described process of spray generation which can contribute to the overtopping due to the wind is the same as in this research. They studied the relevant low overtopping regime ($q^* < 2 * 10^{-4}$).

All the parameters Wolters and van Gent (2007) studied appeared to have an influence on the overtopping behaviour with wind: the mean overtopping discharge, wave steepness, relative water depth at the structure, relative crest element height, relative freeboard, slope angle and the roughness factor. Therefore the influence of all those parameters, except the roughness, will be studied in this research. Furthermore Wolters and van Gent (2007) found a maximum wind influence of $q_w/q = 6.3$ and a minimum influence of $q_w/q = 1.3$.

Other researches with different dike configurations result in wind influences in between that range. Ward et al. (1997) tested a 1:3 dike slope without a crest element, which resulted in a wind influence of around $q_w/q = 2$. de Waal et al. (1997) studied vertical seawalls and found a factor of around $q_w/q = 3.2$. A later study mentioned a factor q_w/q between 1 and 4 after analysing and combining model and field data of 2 rubble mount breakwaters and a vertical seawall (de Rouck et al., 2005). It can be concluded that there is some spreading in the wind influence found by the different studies, mainly determined by the structure type.

Wolters and van Gent (2007) found the following trends in their data, which will be used as a reference to compare and validate the data obtained in this research with:

- The q_w/q ratio decreases for a larger q.
- The influence of wind mainly depends on the dimensionless freeboard ($R_c/Hm0$), a larger dimensionless freeboard means a larger wind influence. For an increase in crest element height they found the same trend.
- No significant differences for the tested 1:2 and 1:1.5 slopes were found. However for the flatter slope the wind influence seemed to be somewhat larger. This is in contrast with what Ward et al. (1997) found.
- Without a crest element the influence of wind becomes much lower ($q_w/q << 1.5$).
- In general a higher q_w/q ratio's for smooth slopes (best visual for the 1:2 slope).
- An increase of the wind effect by a higher crest element height can be counteracted by for instance a decrease in the wind effect by a rougher or steeper slope.
- The wind effect is independent of the wave steepness (no relation Iribarren and q_w/q was found).

As a consequence of the study by Wolters and van Gent (2007), a more recent study was done by van der Bijl (2022), who studied the influence of wind on wave overtopping on a relatively steep slope of 1:3

and in general non-breaking wave conditions. The research in this report, with a relatively flat slope of 1:6 and in general breaking waves on the slope, is a follow up on his study. van der Bijl (2022) tested with the same wave conditions, promenade and crest element heights as will be done in this research. Also the output of his tests was in the same relevant low overtopping regime as this research (approximately $10^{-6} \leq q^* \leq 10^{-3}$). He found ratio's of $1 \leq q_w/q \leq 4$, stating that for lower dimensionless discharges than the mentioned regime the influence could be higher. He develops a influence factor depending on the overtopping discharge without wind ($q\ [m^3/s/m]$), by using the discharge, he accounts for all parameters concerning the wave conditions and the dike configuration as included in the formula from TAW (2002). The general reduction factor for wind he defined is:

$$\gamma_{wind} = 0.011 * q^{*-0.43} + 1\ (1:3\ slope) \tag{2.11}$$

With its applicability limited to the conditions he tested for, testing the factor on a wider range of data is needed to verify further applicability. In this research, an improved factor will be determined with a wider applicability.

Concerning the trends in his data, he observed a decrease in the wind effect for an increase in significant wave height and noticed the same trend for a decrease in wave steepness. This is because both result in an increase in overtopping discharge and like Wolters and van Gent (2007) concluded, the wind effect is lower for higher discharges. Moreover he found that, just like Wolters and van Gent (2007) found, an increase in crest element height results in an increase in the maximum wind effect. Corresponding the use of a promenade, van der Bijl (2022) found that by adding a promenade, the wind effect only increases for a relatively high crest wall. For the lower crest element he tested with, he didn't observe that trend. This is explained by reasoning that for a low element, the area above the promenade can more easily fill with water. When this part is filled with water, the remaining part of the wave can more easily flow over the crest element with less spray. As stated before, spray generation is an important factor for the magnitude of the wind influence.

2.5. Extending the knowledge

From the studied references regarding the wind influence can be concluded that the literature available is based on limited knowledge, because the test data the literature is based on is bounded by the conditions tested for. Especially for a mild slope and breaking wave conditions no information (/test-data) is available regarding the influence of wind, making it worthwhile to study such a slope and conditions. Next to that, for the wind influence to become important, a vertical wall (e.g. a crest element) is needed and the overtopping should be in the low to medium overtopping regime (approximately $10^{-6} < q^* < 10^{-3}$).

It was concluded by Hogeveen (2021) that a crest element can be an effective and relatively cheap way to reinforce a dike. Furthermore, van Doorslaer et al. (2015) showed that for non-breaking waves, a combination of a promenade and a crest element is an adequate way to reduce overtopping.

These findings lead to the studied topic of this research. In which the wind influence will be studied for a dike with a mild slope of 1:6 with a varying crest element height and optional promenade, with breaking waves on the slope.

3

Physical model tests

This chapter gives more insight in the method used to find answers to the research questions. First the choice of a small scale physical model will be explained. Subsequently the general set-up of the physical model and the way the model works will be described. Thereafter the hydraulic conditions and the used test program will be justified. Attention will be given to the wave conditions, water levels and dike configurations. Finally an overview of the magnitude of the tested dimensionless parameters will be given. The rest of the chapter will consist of an elaboration on the scaling and test procedure.

3.1. The physical model

3.1.1. Model selection

Studies done on wave overtopping are typically done with the help of numerical models, small/full scale models or by doing real life ”prototype” measurements. In this section it will become clear why a small scale physical model suits best to the topic of this research, by stating the shortcomings of the other methods and advantages of the model used.

There are multiple numerical models available to predict overtopping. This includes numerical simulation models like OpenFOAM that mimic the wave interaction with the structure (see for instance Mata and van Gent (2023)). Other models based on machine learning techniques exists, like a Overtopping Neural Network (see e.g. van Gent et al., 2007) or recently developed improved XGBoost model (see e.g. den Bieman et al., 2021). To perform good, these models must first be calibrated and validated by physical model data. As told by den Bieman et al. (2021), for the XGBoost model to perform good on for instance oblique wave attack, the model must first be calibrated and validated on a wider range of data. The model is unsuitable to study the in section 2.5 mentioned topics, as wind influence is not included in the models. Obtained data in this research can however later on be used to improve the XGBoost model.

Furthermore, doing prototype measurements is not feasible. Because to study the influence of wind, you need measurements with and without wind, with the rest of the conditions being constant. In real life this is impossible, since the conditions are not controllable and severe wave overtopping events are scarce. Physical model tests in a laboratory can produce the required conditions. Physical model tests can be performed in small or in full scale. For small scale model tests it is essential that the model behaves in the same manner as it would be in the real scale. When this is not possible, more expensive full scale model tests can be performed. In this study the maximum influence of wind is studied. By studying the maximum influence, it is possible to use a small scale model (see section 3.1.4).

3.1.2. Model set-up

The set-up of the physical model is schematically shown in figures 3.1 and 3.2. The model was built by model technicians Wesley Stet and Peter Alberts in the Pacific Basin located at Deltares. Inside the basin a flume was built (see figure 3.3), in which the model was constructed. More pictures and a technical drawing of the set-up made by Wesley Stet can be found in appendix A.

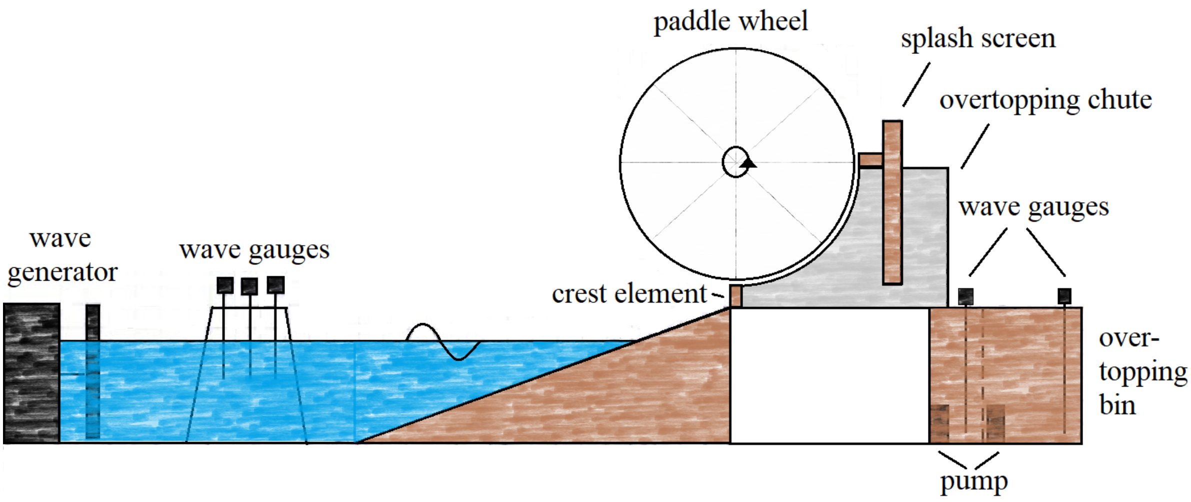


Figure 3.1: Schematic overview of the model set-up; not to scale

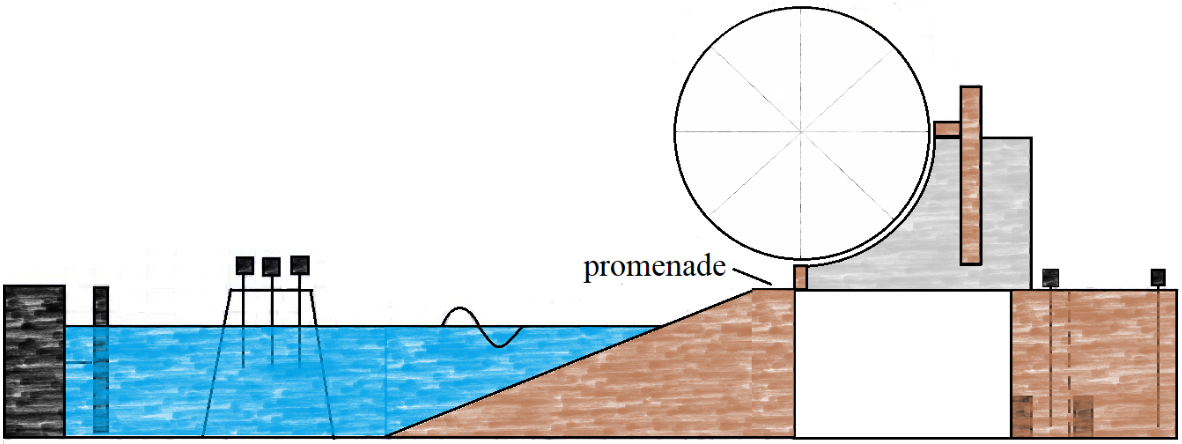


Figure 3.2: Schematic overview of the model set-up with a promenade; not to scale

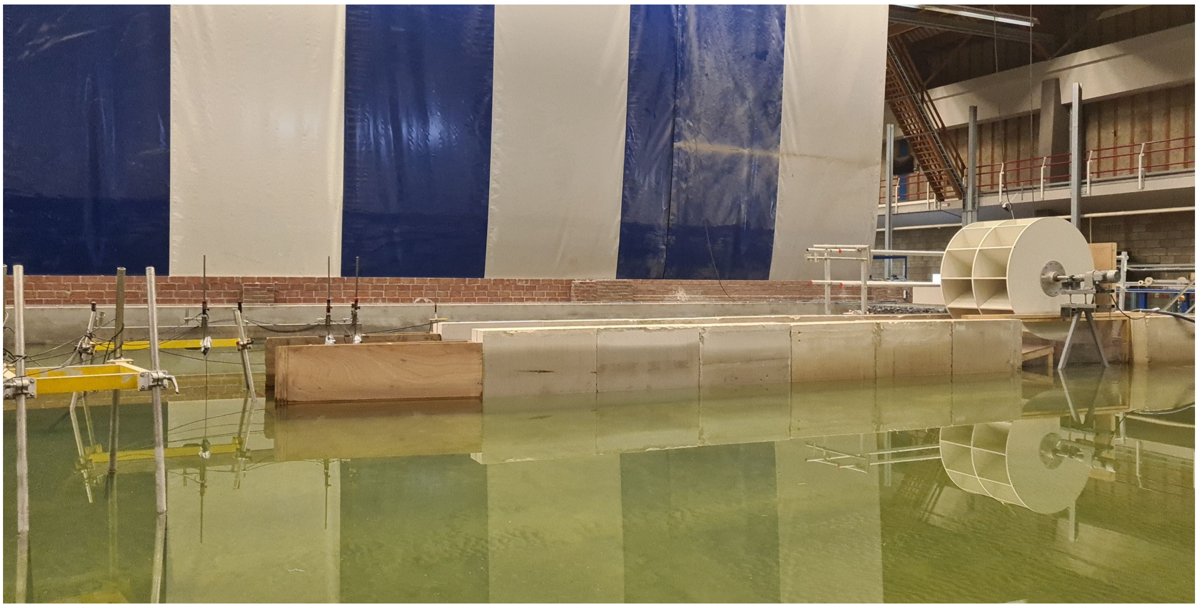


Figure 3.3: Picture of the model set-up

The model set-up works as follows. A JONSWAP-spectrum of waves are generated by the wave generator, the waves enter the flume and at the beginning of the flume the incoming and reflected wave height is measured with 3 wave gauges. The waves start shoaling on the dike slope and eventually the waves break. Some of the jets generated by the breaking waves reach the crest element. When they reach the crest element, a part of the water goes over the crest element and a part returns into the flume. The part that goes back into the flume consists of water that returns back into the flume without an upward motion (schematically and heavily simplified shown in figure 3.4 in blue) and a part that first moves upward (green in figure 3.4, also referred to as spray in the previous chapters). Without wind, the water that exceeds the crest level (green) partially goes back into the flume. With wind, a larger part of the water exceeding the crest level can contribute to the overtopping. The contribution is maximum when all this upward going water is transported over the crest element by the wind. This maximum condition is simulated with the paddle wheel. The situation without wind is modeled without paddle wheel.

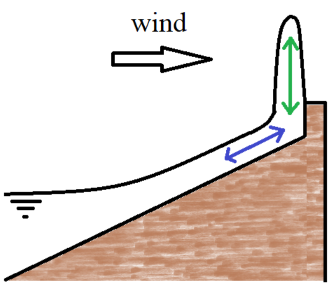


Figure 3.4: Simplified schematic of the jet (generated by wave breaking) returning into the flume

The overtopping water goes via the overtopping chute into the overtopping bin. The overtopping bin is equipped with wave gauges to measure the overtopping volume. Also, the overtopping bin is equipped with a pump. The pump is used to empty the bin after each measurement. Furthermore the pump is automatically turned on when the bin is fully filled with water during a measurement. The data collected during the time the pump is on is removed during post-processing. Because the amount of pumping was kept to a minimum, this had no negative effect on the obtained results (see intro chapter 4).

3.1.3. Hydraulic conditions and test program

In the previous sections it was shown which model to use and how the set-up of the model works and looks like. In this section the choice for the magnitude of the different parameters tested with will be explained. Furthermore the used test-program will be part of this section.

Wave conditions

The wave conditions are the same as van der Bijl (2022) and can be found in table 3.1. The latter study did a comparable research to his master thesis, only with a steeper slope of 1:3. By using the same wave conditions, the data of this research with a flatter slope of 1:6 can be compared to the data with a slope of 1:3.

Water level

The water level was determined with the help of the TAW (2002) formula. With the formula the dimensionless overtopping discharge (including confidence interval) was estimated for the different wave conditions, with the water level being the only parameter which could be adjusted. Next to the formula, two considerations were part of the determination of the water level:

As mentioned in the literature study (chapter 2), it is important to stay in the right overtopping regime (approximately $10^{-6} < q^* < 10^{-3}$) to study the wind influence. When the amount of overtopping is not approximately in this regime, a limited wind influence can be observed. A too low amount of overtopping may result in a large scatter in data due to scale effects, a too high amount of overtopping results in a limited wind influence (Wolters and van Gent, 2007) while the tests may be out of the range of practically relevant. The water level was determined following this criterion. From this followed that for a water level of 0.8 meter, nearly all wave conditions where estimated to be in the right regime. For some of the wave conditions it was estimated that the q* would be in a too low regime, for these conditions tests were also performed for a water level of 0.9 meter.

For other wave conditions a relatively large amount of overtopping was estimated. For these tests it was, next to the overtopping regime, important to take the amount of liters overtopping during a test into account, which turned out to be decisive. When this amount was too large, the overtopping bin would fill too fast and too much pumping during a test was needed. Because the time the pump is on is removed from the test during post processing, the test time would be too short to obtain useful results.

The above mentioned considerations lead to the water levels shown in the test program (table 3.1). Using those water levels resulted in a data-set with the overtopping discharge in the right regime and with little to no pumping.

Test program

The first test runs consisted of tests with and without wind, maximum and minimum wave overtopping conditions and different water levels. These tests in combination with the before mentioned considerations resulted in the use of the test program depicted in table 3.1. In the table H_{m0} corresponds to the spectral incoming wave height, $T_{m-1,0}$ to the spectral wave period, T_p to the peak wave period, $s_{m-1,0}$ to the spectral wave period determined with $T_{m-1,0}$ (see equation 2.4), $\xi_{m-1,0}$ to the breaker parameter determined with $s_{m-1,0}$ (see equation 2.3) and d to the water depth, all measured at the toe of the dike slope.

$Index\ nr.$	$H_{m0}\ [m]$	$T_{m-1,0}\ [s]$	$T_p\ [s]$	$s_{m-1,0}\ [-]$	$\xi_{m-1,0}\ [-]$	$d\ [m]$
1b	0.1	1.79	1.97	0.02	1.18	0.90
2	0.125	2.00	2.20	0.02	1.18	0.80
3	0.15	2.19	2.41	0.02	1.18	0.80
4b	0.1	1.46	1.61	0.03	0.96	0.90
5	0.125	1.63	1.80	0.03	0.96	0.80
6	0.15	1.79	1.97	0.03	0.96	0.80
7b	0.1	1.27	1.39	0.04	0.83	0.90
8b	0.125	1.41	1.56	0.04	0.83	0.90
9b	0.15	1.55	1.70	0.04	0.83	0.90
10	0.175	2.37	2.60	0.02	1.18	0.80
11	0.2	2.53	2.78	0.02	1.18	0.80
12	0.175	1.93	2.13	0.03	0.96	0.80
13	0.2	2.07	2.27	0.03	0.96	0.80
14	0.175	1.67	1.84	0.04	0.83	0.80
15	0.2	1.79	1.97	0.04	0.83	0.80

Table 3.1: Test program

Tests were performed for multiple relevant configurations (see section 2.5). The 4 configurations, shown in figure 3.5, are the same as van der Bijl (2022), the only difference is the slope: 1:6 (the slope of van der Bijl (2022) was 1:3). The first configuration is without a promenade and with a 5cm crest element. The second configuration is without a promenade and an 8cm crest element. The third is with a 15cm promenade and a 5cm crest element and the last configuration is with a 15 cm promenade and an 8cm crest element. For each configuration tests are performed without wind and with maximum wind (simulated with the paddle wheel).

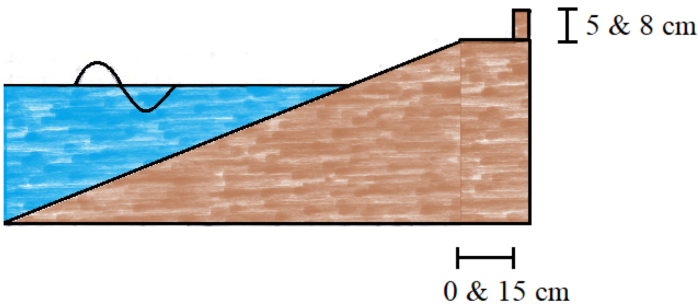


Figure 3.5: Schematic of tested crest elements and promenade; not to scale

Test duration
The test duration's were determined such that a minimum of 1000 waves would be generated during each test. This threshold is widely used in physical model tests in hydraulic engineering and is proven to simulate the sea state well.

Tested parameters
In table 3.2 the range of the magnitude of the different parameters are shown and in figure 3.6 the different parameters are defined.

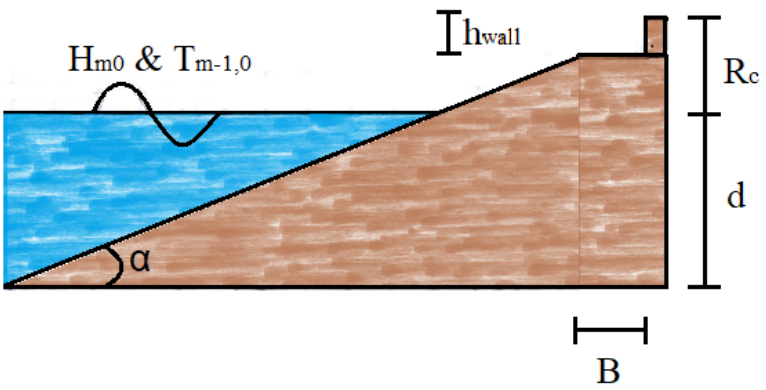


Figure 3.6: Definition sketch of the model set-up

Tested parameters	Symbol	Magnitude
Mean overtopping discharge [-]	q^*	$1.45E-6 \ - \ 1.64E-3$
Wave steepness [-]	$s_{m-1,0}$	$0.02-0.041$
Relative promenade width [-]	B/H_{m0}	$0-1.53$
Relative freeboard [-]	R_c/H_{m0}	$0.74-2.86$
Relative crest element height [-]	h_{wall}/R_c	$0.2-0.44$
Seaward slope [-]	$tan(\alpha)$	$1:6$
Breaker parameter [-]	$\xi_{m-1,0}$	$0.83 \ - \ 1.18$
Crest freeboard [m]	R_c	$0.15 \ - \ 0.28$
Crest wall height [m]	h_{wall}	$0.05 \ and \ 0.08$
Promenade width [m]	B	$0 \ and \ 0.15$
Significant incoming wave height [m]	H_{m0}	$0.098 \ - \ 0.202$
Mean spectral wave period [s]	$T_{m-1,0}$	$1.26 \ - \ 2.50$
Water depth [m]	d	$0.8 \ and \ 0.9$

Table 3.2: Magnitude of the tested parameters in the physical model tests (1:6 slope)

3.1.4. Froude scaling & scale effects

Now the test program is known, the model must be scaled. To be able to rely on the output of a scale model, it is important that the model reflects reality well. It is most of the time not possible to scale all the forces acting on a structure at the same time, therefore the hydrodynamics should be scaled by the predominant forces. In this research the main force acting on the structure is caused by waves. Due to the large gradient in water level caused by the waves, viscosity and surface tension are not significant. Inertia and gravitational forces are the dominant forces to take into account. Therefore Froude law should be used (Schiereck and Verhagen, 2019):

Froude scaling

The Froude law consist of an inertia term divided by a gravitational term:

$$\frac{u_m}{\sqrt{g_m L_m}} = \frac{u_p}{\sqrt{g_p L_p}} \tag{3.1}$$

In the Froude law, the velocity (u) divided by the square root of the gravitational acceleration (g) and length scale (L) of the model (subscript m) should be equal to the prototype (subscript p). Furthermore a scale factor is introduced:

$$n_L = \frac{L_p}{L_m} \tag{3.2}$$

All length scales should by scaled according to this scale factor. Moreover as said before, the gravitational acceleration stays constant. Combining equation 3.1 and 3.2, results in the following scale factor for the velocity:

$$n_V = \frac{u_p}{u_m} = \sqrt{n_L} \tag{3.3}$$

Because time is equal to the length scale divided by the velocity scale, the time scale is:

$$n_T = \frac{n_L}{n_V} = \sqrt{n_L} \tag{3.4}$$

Based on a sea dike with a freeboard of 6 meter, the model in this research can be seen as a scale model of a factor between 21.4 and 40 depending on the configuration. Although if the test results are made non-dimensional, the results can be used for any scale.

Scale effects

Like Dong et al. (2021) says: "The scale difference in the interactions between wind and spray can lead to differences in overtopping measurements between laboratory and field conditions". Next to that, scaling rules for the wind don't exist. Therefore to account for the wind, de Waal et al. (1997), Wolters and van Gent (2007), and van der Bijl (2022) and this research chose to study the maximum influence of wind with the help of a paddle wheel. When looking at the maximum influence of wind, no scaling of the wind is needed. Also, when looking at loads, typically engineers design their structure on maximum loads, so it is also relevant to look at this maximum additional wave overtopping load due to the wind. Thus the scale effects of this method are limited, however the use of the paddle wheel leads to the following model effect.

The paddle wheel used and the rotation speed of 22 revolutions per minute is the same as Wolters and van Gent (2007) did their tests with. They estimated the efficiency of the wheel to be above 90%. This means that when testing the maximum wind condition with the paddle wheel, it is possible that the wheel misses 10% of the over topped water. Based on visual observations and recordings of tests this estimate seems reasonable.

3.1.5. Test procedure

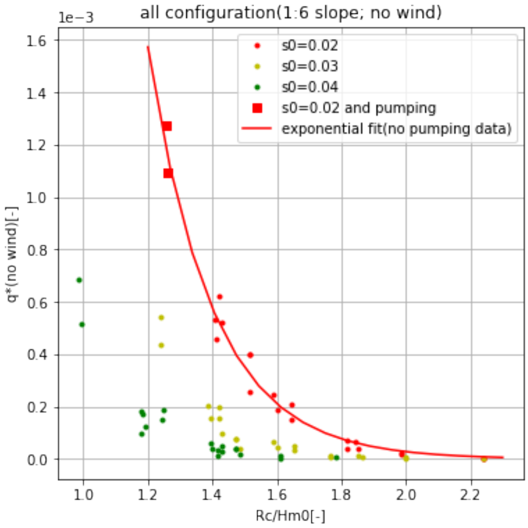
In this section the relevant parts of the test procedure will be shortly explained. At the start of the testing phase, input files for the wave generator containing H_{m0i}, T_p, test duration and the wave spectrum type were generated. At the start of each test, the correct input file was selected for the wave generator. Furthermore it was checked if the overtopping bin was pumped empty, so the new measurement starts at zero. For the tests with wind it was checked if the paddle wheel was rotating. At the start of each testing day was checked if the water level was correct, because especially after a weekend off it was possible that some of the water evaporated, resulting in a lowering of the water level. The water level was corrected when the maximum deviation of 1 millimeter was reached.

During the tests was checked if the waves were measured correctly. The wave gauges have a certain range of wave height they can measure, the level of the wave gauges was adjusted such that the waves stayed in between the range. During the measurements also the post processing and validation of the previous measurements took place. It was possible that the input wave height and the actual measured incoming wave height differed somewhat, if this difference was larger that 2 millimeter, the measurement was performed one more time with an adjusted gain factor (the gain factor is used to calibrate the wave generator). Furthermore the processing consisted of putting all the output in excel and process it to the dimensionless overtopping discharge.

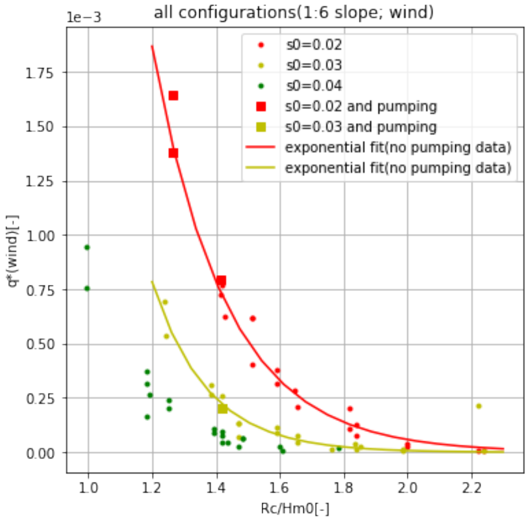
4 Data Analysis & discussion: The wave overtopping discharge

In the previous chapter is described how the physical model data is obtained. In this chapter the data will be compared with literature to validate if the trends in the data are as expected (section 4.1) and if the data is in line with what is expected from theory (section 4.2). This analysis will clarify that the data forms a solid foundation for the further analysis of the data with respect to the research questions about the influence of wind, what will be part of the next chapter (chapter 5). Furthermore (improved) reduction factors will be determined for the use of a crest element and/or a promenade in combination with breaking waves on the dike slope.

Before the analysis starts, it will be verified if the pumping had an influence on the obtained results. For relatively large amounts of overtopping, sometimes pumping was needed the empty the overtopping bin, because otherwise the bin would flood and the amount of overtopping could no longer be measured. The water levels were chosen in such a way that for only 6 tests pumping was needed. The time the pump was on during a measurement, was removed during post-processing. Resulting in a incomplete time series due to the pumping. The maximum amount of pumping allowed during a test was 2 times, which resulted in a removal of 6% of the time series for 1 test. For the other tests the maximum removal was 3% of the test duration. To verify if this had an influence on the obtained results, an exponential function is plotted through the data where pumping was not needed in figure 4.1. As can be seen, the data-points where pumping was needed (indicated with a square) do not deviate from the exponential trend of the rest of the data and is thus useful for the analysis.



(a) without wind



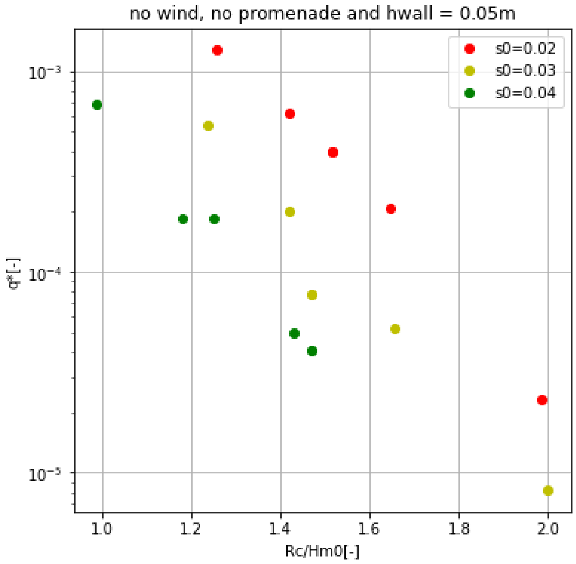
(b) with wind

Figure 4.1: Influence of pumping on the dimensionless discharge

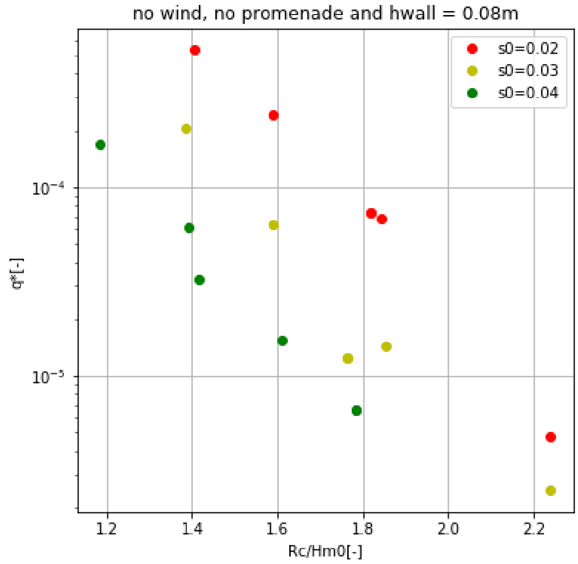
4.1. Validation of the trends in the data (1:6 slope)

To validate if the trends in the data are as expected, the dimensionless discharge is plotted against the dimensionless crest freeboard in figures 4.2 and 4.3 for every dike configuration (without and with wind), with a different colour for every wave steepness. From these graphs a lot is learned about the trends in the data. To start it is visible that, when looking at a constant wave steepness, the discharge decreases for an increase in dimensionless crest freeboard. Furthermore it is shown that, for a constant freeboard, the discharge decreases for an increase in wave steepness. Both trends are in line with what is expected based on the TAW (2002) formula and EurOtop (2018).

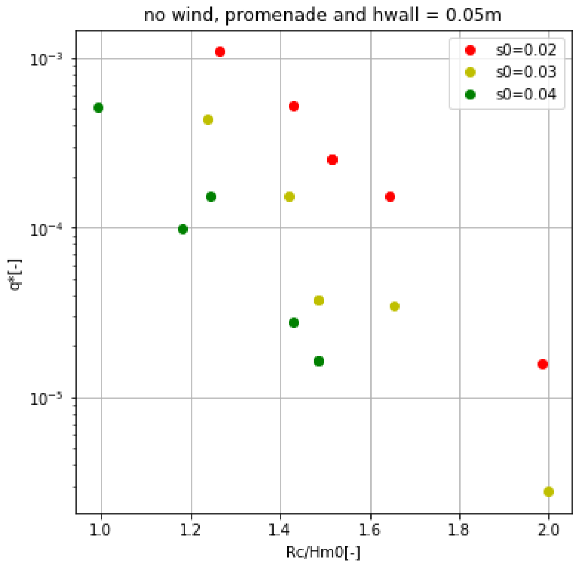
Next it can be seen that for a constant wave steepness, the discharge has a linear decreasing trend on the logarithmic scale. This linear dependency on the logarithmic scale results in a exponential dependency a normal scale. This corresponds to the exponential dependency in the TAW (2002) formula. Now it is known that the trends of the wave conditions in the data are as expected, the data will now quantitatively be compared with the TAW (2002) formula and other relevant literature from the literature study.



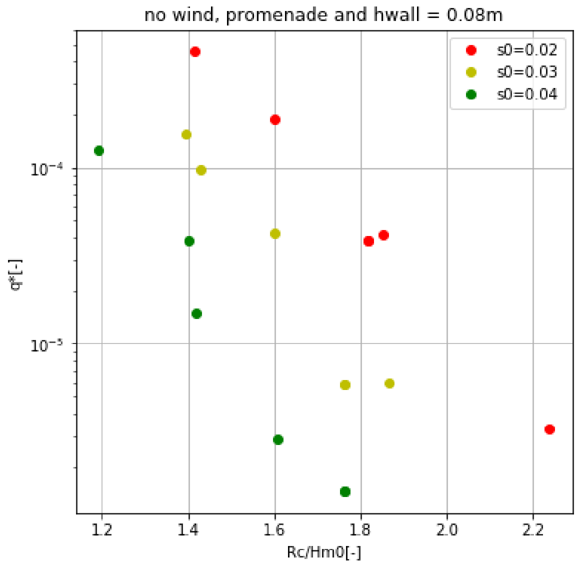
(a) -



(b) -

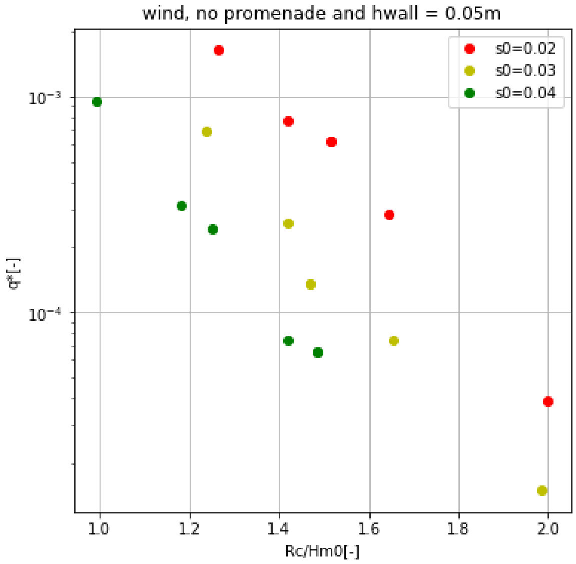


(c) -

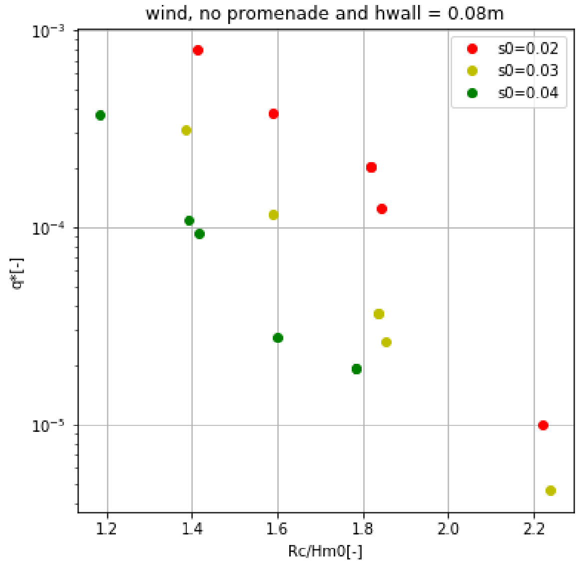


(d) -

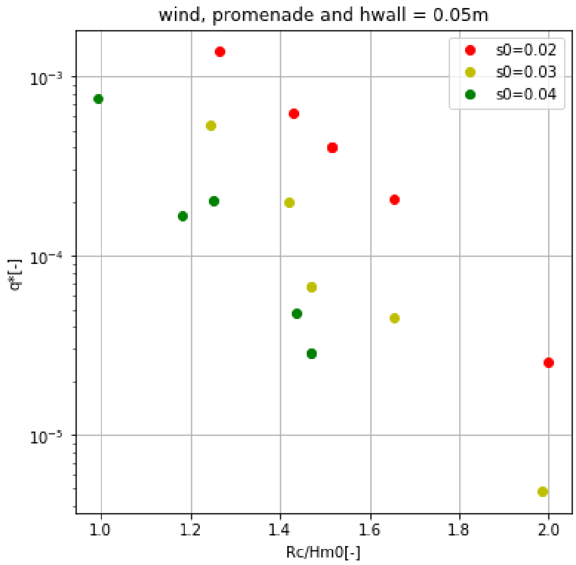
Figure 4.2: Graphs of dimensionless crest freeboard vs model data without wind



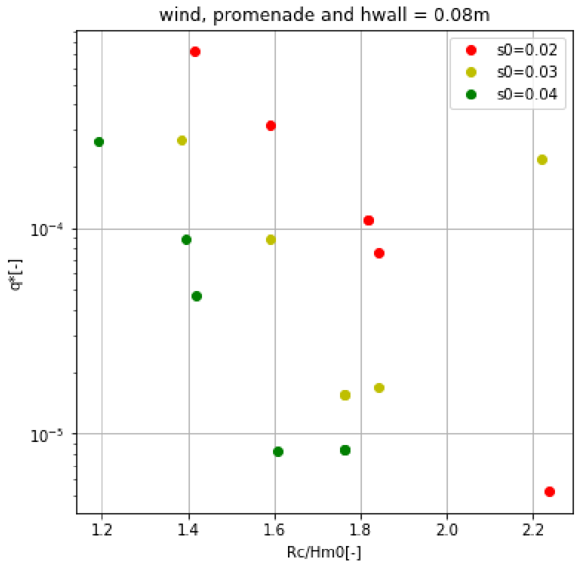
(a) -



(b) -



(c) -



(d) -

Figure 4.3: Graphs of dimensionless crest freeboard vs model data with wind

4.2. Comparing the overtopping discharge with literature (1:6 slope)

Now it is known that the trends in the data with respect to the wave conditions are in line with the expectations, now the overtopping discharges without wind influence will be quantitatively compared to the theoretical knowledge gained from the literature study.

4.2.1. Wave overtopping with a crest element

In section 2.1 and 2.2.1 the choice for applying the TAW (2002) formula as theoretical background was discussed. Thereafter in section 2.2.2 it was described how to account for the reduction due to a crest element. When the general method of TAW (2002) to account for a vertical wall on a dike slope ($\gamma_v = 0.65$) is used to account for an emerged crest element, Van Doorslaer et al. (2016) found that for breaking waves the reductive effect of a crest element is strongly overestimated. Because the conditions Van Doorslaer et al. (2016) tested for are similar to the conditions in this research (breaking waves, $tan\alpha = 1/6$ and a relatively small emerged crest element on top of the dike ($h_{wall}/Rc < 1$)), and the slope of this research is outside the applicability range of 1:2.5 - 1:3.5, it was decided that it is not relevant to zoom in on this factor in this section. More relevant is to see how the proposed factor by Van Doorslaer et al. (2016) of $\gamma_v = 0.92$, based on the before mentioned similar conditions, fits to the data in this research. In this section it will be validated if his findings are in line with the data obtained in this research.

To do so, the theoretical estimations with the reduction factor of 0.92 are plotted against the physical model data without a promenade and without maximum wind influence in figure 4.4. Because Van Doorslaer et al. (2016) determined the factor using the TAW (2002) formula, the factor can be used without a conversion. When Van Doorslaer et al. (2016) would have determined the factor using the EurOtop (2018) formula, special attention was needed by applying the reduction factor. Because in that case the factor was determined by a power of $c = 1.3$ in equation 2.1, in the used TAW (2002) formula the power is $c = 1$.

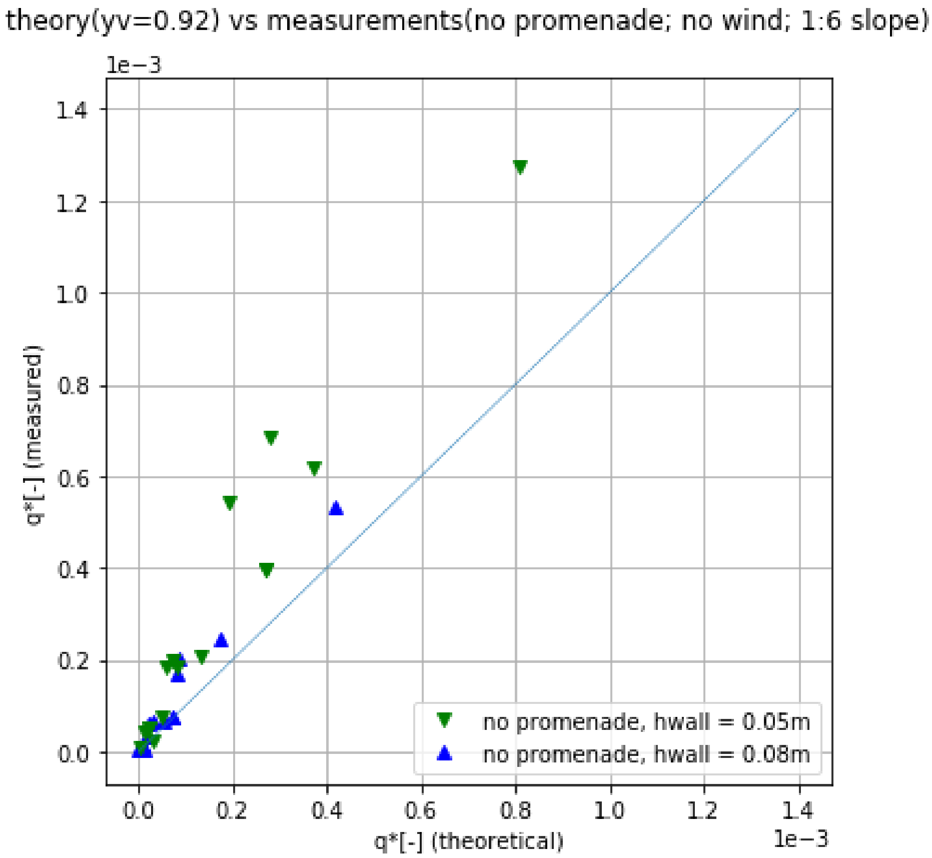


Figure 4.4: Theory (TAW (2002); Van Doorslaer et al. (2016):$\gamma_v = 0.92$) versus no promenade/no wind data

The first thing noticed is that there is one point with a larger theoretical and measured discharge than the rest. This was for the largest tested wave height of 0.2 meter and the relatively low crest element of 0.05 meter. Because the discharge is larger for both the theoretical and measured discharge this is expected. Another thing that stands out is that all data points lie relatively close to the theoretical estimated discharges. Furthermore, the data has a linear dependency with theory. This is in line with the expectations, since it was shown in section 4.1 that the trends in the obtained data behaves in the same way as the TAW (2002) formula. Moreover, most data-points lie above the 1:1 line. So the theory generally underestimates the wave overtopping discharge, only a few of the low discharges in the bottom left corner lie below or on the 1:1 line. This makes it interesting to see if the data maybe fits better to the theory when no reduction factor for a crest element is applied (see figure 4.5). As this will increase the theoretical overtopping discharge and translate the data towards the 1:1 line. In that case the crest element only has an influence on the theoretical discharge by increasing the crest freeboard (R_c).

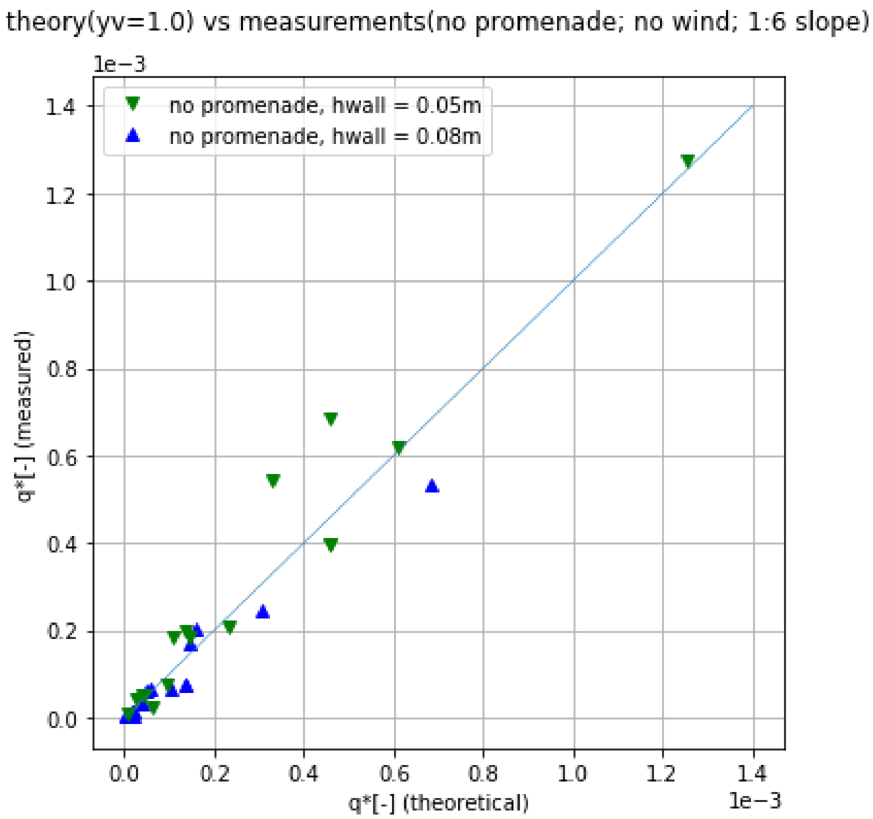


Figure 4.5: Theory(TAW (2002); $\gamma_v = 1.0$) versus no promenade/no wind data

On the eye it seems that this makes the TAW (2002) formula with no reduction factor fits well to the data. To make a quantitative comparison between the two, the Root Mean Squared Logarithmic Error (RMSLE) will be used:

$$RMSLE = \sqrt{\frac{\sum_{i=1}^{N}(log(q^*_{measured}) - log(q^*_{calculated}))^2}{N}} \tag{4.1}$$

With $q^*_{measured}$ the dimensionless overtopping discharge measured in the physical model tests, $q^*_{calculated}$ the estimated theoretical amount of overtopping and N the number of tests. The RMSLE is a widely used approach to account for the error in data. By using the logarithm of the discharge, the error is robust to outliers. In the Root Mean Square Error (RMSE) the same formula is applied without the logarithm. When there is an outlier using the RMSE the error will blow up. The RMSLE for $\gamma_v = 0.92$ is equal to 0.763. When no reduction factor is applied ($\gamma_v = 1.0$) the RMSLE is equal to 0.448. From this is concluded that that it is better to use no reduction factor rather than the $\gamma_v = 0.92$ proposed by Van Doorslaer et al. (2016).

In figure 4.6 it is shown that using no reduction factor is the optimal way to account for a crest element with breaking waves on the dike slope. The dotted line corresponds to the TAW (2002) formula and the uninterrupted line is the best fit to through the data. It is shown that the calibration constants (a) are nearly equal to each other (4.7 versus 4.75), showing that using no reduction factor is indeed the optimal way to account for a crest element regarding the conditions tested for.

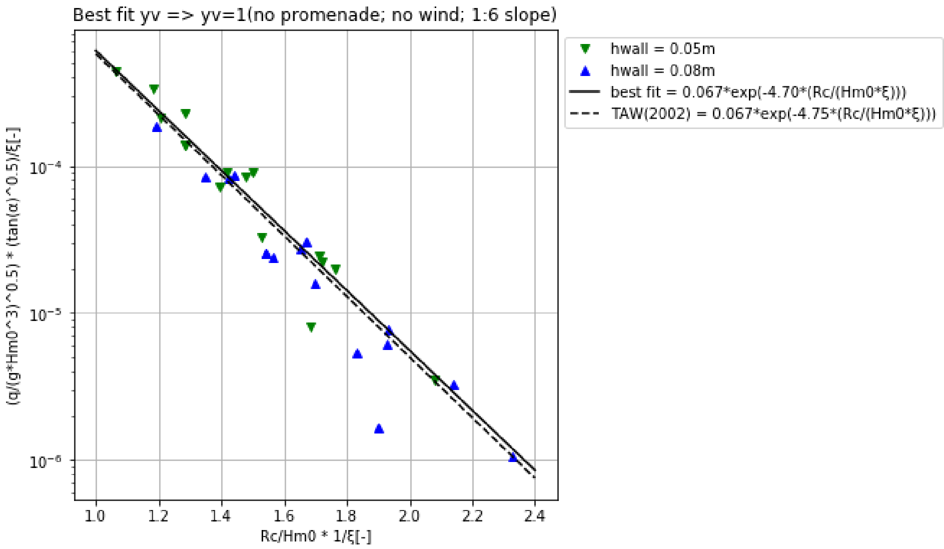


Figure 4.6: Best fit reduction factor for a crest element (breaking waves)

In figure 4.6 it can be seen that there certainly are some data-points that show some additional reduction effect due to the crest element, what is in line with the findings of Van Doorslaer et al. (2016). However this is especially found for the higher dimensionless free boards divided by the breaker parameter ($R_c/H_{m0}/\xi_{m-1,0}$), where Van Doorslaer et al. (2016)'s data doesn't include values higher than approximately 1.7. Furthermore the main trend shows no additional reduction effect, what is not in line with what Van Doorslaer et al. (2016) found. Because despite the mentioned small difference in $R_c/H_{m0}/\xi_{m-1,0}$, the conditions tested for are approximately equal and both studies used the same number of tests (both 30 tests) to come to a conclusion, no explanation for the difference in outcome between Van Doorslaer et al. (2016) and this research are found. It is recommended to use no reduction factor for the crest element until more knowledge is gathered.

Moreover, when the crest element doesn't have an extra reducing effect in the form of a reduction factor, the crest element only has a reducing effect by its contribution to the crest freeboard. But in that case also the dike slope can be extended to the crest element height like depicted in figure 4.7 to have the same reducing effect. Only this configuration has the advantage that the upward directed spray is limited and thus a limited addition due to the wind. Although still some spray is generated as the water impacts from the extended part on the back side of the crest (this is expected to be less).

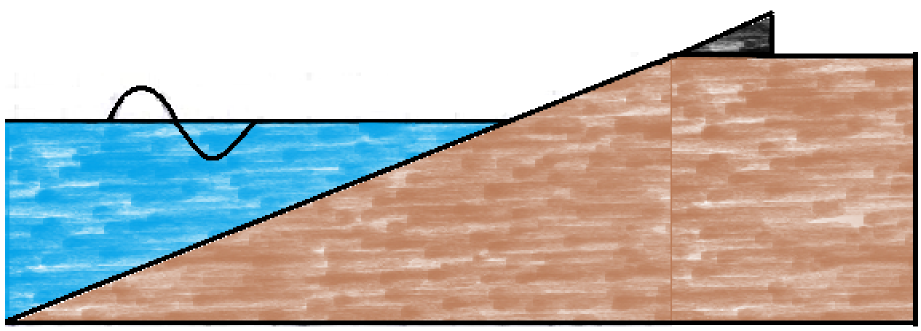


Figure 4.7: Figure of extending the dike slope in stead of the use of a crest element

4.2.2. Wave overtopping with a promenade and crest element

From section 2.2.2 was concluded that the combination of a promenade and a crest element has a reducing potential for non-breaking waves on the slope. van Doorslaer et al. (2015) proposed a reduction factor to account for a promenade and one for the combination of a promenade and a crest element. For breaking waves no such reduction factors are available. Therefore it will be shown that the promenade also has a reducing effect for breaking waves (in combination with a crest element) and a reduction factor will be determined.

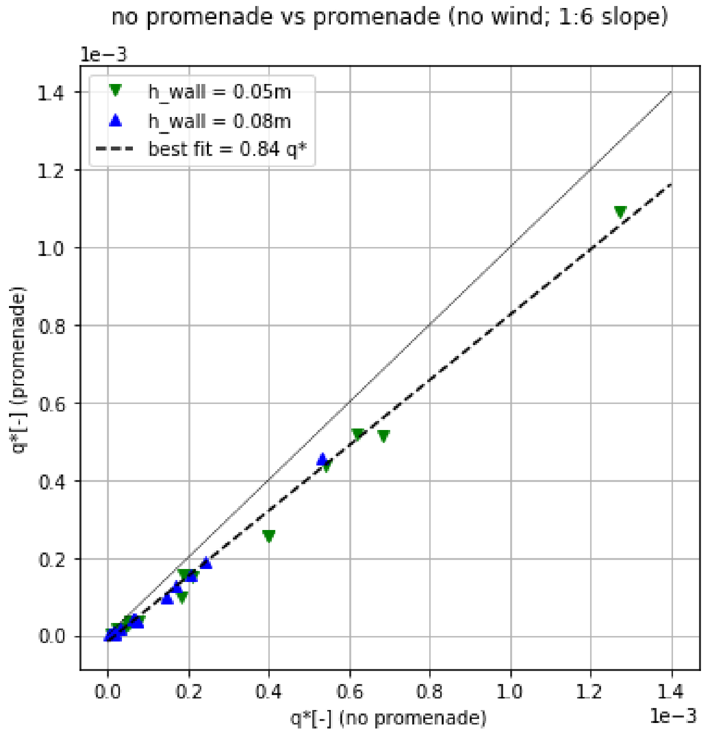


Figure 4.8: No promenade versus promenade physical model data

In figure 4.8 it is shown that the configurations with a promenade generally lead to lower discharges. Moreover this reduction due to the promenade is rather constant, something that was also found for non-breaking waves by van Doorslaer et al. (2015). To determine the magnitude of this reducing effect, a best fit function is plotted through the data without a promenade and a function through the data with a promenade in figure 4.9. The best fit functions are in the same form as the TAW (2002) function (see equations 2.1 & 2.2), but the constant b is kept as a calibration constant, making it possible to determine the reduction factor from it. For the configurations without a promenade this results in a calibration constant of b = 4.7 and with a promenade in b = 4.88, this results in a constant reduction factor of 0.96 when a promenade is used (equation 4.2).

$$\gamma_{prom\_v} = 0.96\ (breaking\ waves) \tag{4.2}$$

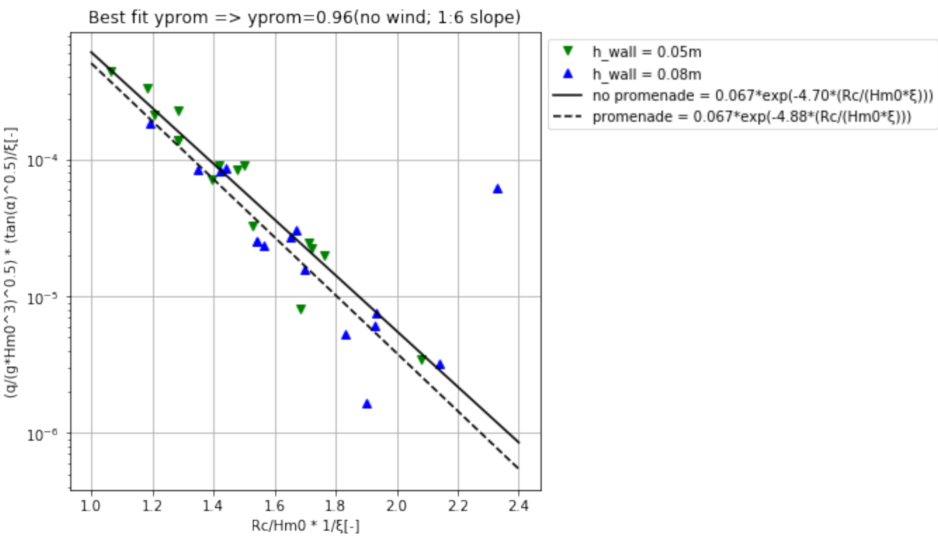


Figure 4.9: Best fit reduction factor for a promenade (in combination with a crest element) (breaking waves)

This factor is determined for dikes with a combination of a crest element and a promenade. When the factor is applied without a crest element, one should be aware that there is interaction between the crest element and the promenade. An example of an interaction is that for large amounts of overtopping the triangle shaped space between the promenade and crest element can fill with water and the rest of the water will flow over this triangle, leading to a situation where the effect of the promenade is lost. Another example is that the promenade slows down the water flowing over it, leading to a lower impact on the crest element. van Doorslaer et al. (2015) determined that for non-breaking waves, the combination of a promenade and crest element leads to an additional reduction effect of 0.87 (see equation 2.10). So possibly when only a promenade is used, the reduction is less than the 0.96 determined here. Tests without a crest element are needed to verify this.

Another thing to keep in mind is that in this research the aim was on relatively low overtopping discharges to study the influence of wind. The factor is thus determined on data with relatively low overtopping discharges. When looking at higher discharges, what often happens when designing a structure, is is more likely that overshooting occurs (see figure 5.10) and the reducing effect of the promenade can be less.

5

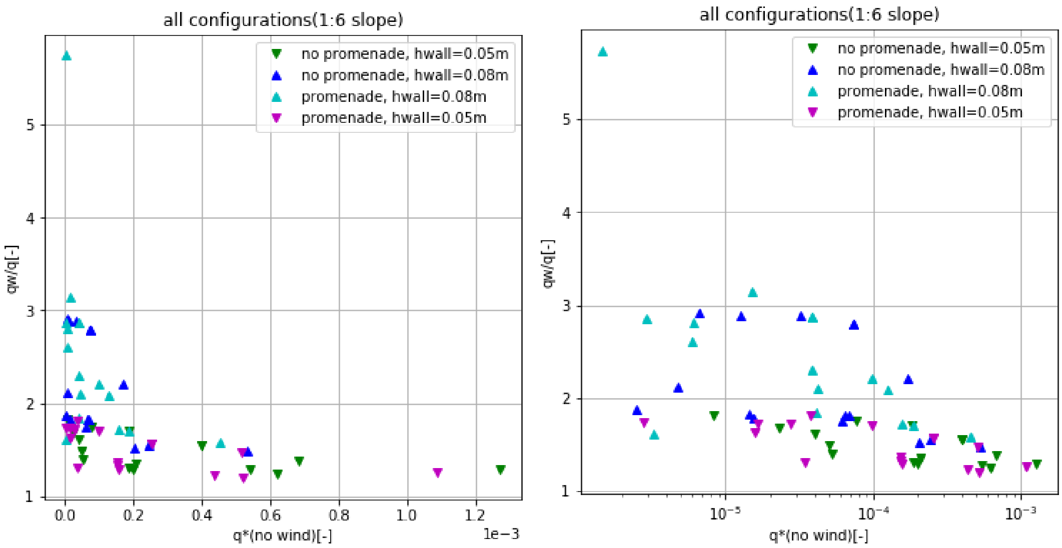
Data analysis & discussion: The maximum wind influence

In the second part of the data analysis the focus will be on the main topic of this research: the maximum influence of wind. The maximum influence of wind is determined by dividing the dimensionless overtopping discharge with maximum wind influence ($q*_w$) (obtained with testing with the paddle wheel) by the dimensionless overtopping discharge without maximum wind influence ($q*$) (no paddle wheel). This results in q_w/q ratio's which will be analysed in this chapter. First a general view on all the data combined will be discussed. Thereafter will be analysed if there is a dependency on certain wave conditions and dike configurations. The latter will form the basis for conclusions regarding the research questions about the dependency of the influence of wind on: the crest element height, the presence of a promenade and the dike slope. The chapter will be finalised with the determination of a general applicable wind amplification factor.

5.1. General view on the data (1:6 slope)

In this section the focus will be on the main trends in the wind data and the maximum and minimum values found. In the sections hereafter will be zoomed in on specific dike configurations to find an answer to the research questions.

To start, the maximum wind influence found was 5.7, as is shown in figure 5.1. A wind influence with this magnitude is only found in 1 test. After this the largest wind influence found is 3.2, the difference between these two is large. There are more tests resulting in approximately the same magnitude as the 3.2, making it a plausible output. However the large difference between 5.7 and 3.2 makes it questionable if the value of 5.7 is useful. The value of 5.7 corresponds with the lowest measured discharge of 1.45E-6. Wolters and van Gent (2007) found discharges lower than 2E-6 to be too much affected by scale effects, making it possible that also in this case the discharge is too much affected by scale effects. Because the data-point follows the trend of the rest of the data and is inside the range of wind influences found in the literature study, the point of 5.7 will be seen as a reliable measurement in the further analysis.



(a) all data

(b) all data on a logarithmic scale

Figure 5.1: Wind influence factor plotted against the discharge without wind (all configurations)

Corresponding the lowest found wind influence, there are a lot of tests resulting in a wind influence between 1.2 and 2. What stands out in this region of the graph is that all downward facing triangles, corresponding to a 5cm crest element, are located in between this range. There is a clear difference between the data with a 5cm and 8cm crest element, this is in coherence with what Wolters and van Gent (2007) and van der Bijl (2022)/van Gent et al. (2023) found. They found the wind influence to be for a large amount determined by the crest element height (and crest freeboard in general). A more detailed analysis of the influence of the height of the crest element specifically can be found in section 5.3.

Now is learned about the maximum and minimum values, the focus will shift to the dependency on the dimensionless overtopping discharge. From the literature it was learned that the dimensionless overtopping discharge is an important parameter when studying the influence of wind on wave overtopping. Too high discharges result in a little to no spray generation, something that is essential for the wind influence to become relevant. To see if this dependency on the dimensionless overtopping discharge is also found in the data, all wind data is plotted in figure 5.1 against the overtopping discharge without wind.

In the figures the same dependency on the overtopping discharge is observed as in literature (Wolters and van Gent, 2007 and van der Bijl, 2022/van Gent et al., 2023). van der Bijl (2022) plotted the same figure for his data with a 1:3 slope and the trend is very similar, although his tests resulted in somewhat less spreading. An explanation for this is that due to the different slopes, he tested with breaker parameters 2 times the breaker parameters in this research. Therefore his tests move more towards the non-breaking regime (as defined in 2.1) and approximately 25% of his tests are located inside this regime. In the breaking waves regime tested in this research, non-linear effects play a larger role during the wave breaking, causing somewhat more spreading in the obtained discharges. More about the comparison with the 1:3 slope can be found in section 5.3.

It is noticed that the wind influence increases for a decrease in discharge, a trend that is matching with the findings by Wolters and van Gent (2007) and van der Bijl (2022)/van Gent et al. (2023). Moreover there is an exponential dependency on the dimensionless overtopping discharge. When the dependency is exponential, a linear trend is expected on the logarithmic scale. As can be seen, a linear trend can be observed, despite the before mentioned spreading/scatter. Although when looking at the data per configuration, a clearer linear trend can be observed with limited scatter for the 5cm crest element. More information about the dependency on the configuration can be found in section 5.3.

5.2. Dependency on the wave conditions (1:6 slope)

During the physical model tests, tests were performed with varying wave conditions. In this section the influence of these varying conditions on the wind effect will become clear. In figure 5.2 the wind influence is plotted against the dimensionless crest freeboard.

Dimensionless crest freeboard

Beforehand it was expected that a larger freeboard results in less wave overtopping and thus a larger wind influence, due to the dependency on the dimensionless discharge shown in section 5.1. This trend of the dimensionless freeboard was also found by Wolters and van Gent (2007) and van der Bijl (2022). When focusing on a constant wave steepness (see figure 5.2), there is an increasing trend visible. The lower wind influences are found for a low dimensionless freeboard and higher wind influences are found for a higher dimensionless freeboard.

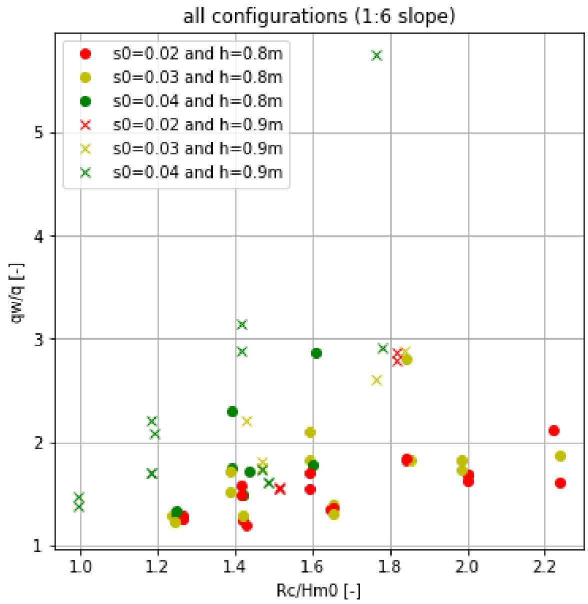


Figure 5.2: Wind influence factor plotted against the dimensionless crest freeboard (all configurations)

But some of the data deviate from this trend. For the 0.02 and 0.03 wave steepness, the maximum wind influence found is for a dimensionless freeboard a bit higher than 1.8. For higher dimensionless freeboards the wind influence reduces a relatively large amount. So where Wolters and van Gent (2007) and van der Bijl (2022) generally found the largest wind influences for the largest dimensionless crest freeboard, this pattern is not found everywhere the graph. For the 0.04 wave steepness the pattern looks more like the one Wolters and van Gent (2007) and van der Bijl (2022) found, although for this wave steepness the highest dimensionless freeboard presence in the data is 1.78. For the wave steepness of 0.02 and 0.03 the decreasing pattern is only visible for dimensionless freeboards above 1.87. It is unknown if for the 0.04 wave steepness the same pattern is present for dimensionless freeboards above 1.87.

Water depth variations

A possible explanation for this is found by having a closer look to the water depths used during the tests in figure 5.2. When the tests with a water level of 0.9 meter are not taken into account, the expected pattern is observed in the figure, despite one outlier for the 0.03 wave steepness. What attracts attention is that most of the highest found wind influences are found for a water level of 0.9 meter. This results in the presumption that there is a dependency on the water depth. Unfortunately there is no useful data in the data-set to verify if this statement is true.

Fortunately van der Bijl (2022) did some useful tests with non-breaking waves to make a first analysis on the influence of the water level. He did tests with a varying water level for the configuration with a 5 centimeter crest element. So the changes in the dimensionless crest freeboard are only caused by water level variations. The outcome of those tests are shown in 5.3. When observing the figure, there seems to be a maximum for a certain water level, for higher dimensionless freeboards induced by the water level the wind influence reduces a lot. This observation strengthens the presumption on the dependency on the water level and can possibly explain the pattern that was found in the data of this research in figures 5.2.

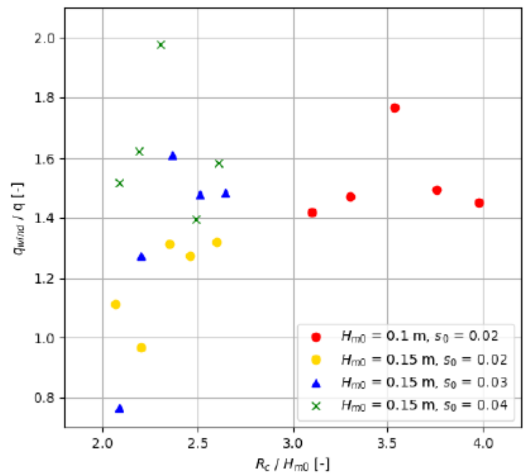


Figure 5.3: Wind influence factor plotted against the dimensionless crest freeboard, variations in freeboard are caused by a change in water level only (5cm crest element, non-breaking waves)

To conclude, it is possible to explain this possible dependency on the water depth by the influence of the water depth on the overtopping discharge. A larger water depth leads to more overtopping and thus a lower wind influence due to the dependency of the wind influence on the overtopping discharge. However the different water depths were used, so all discharges are in the same relevant overtopping regime (relevant for the wind influence). Therefore the discharges with the 0.9 meter water depth were not necessary higher than with the 0.8 meter water depth, which is also shown in figure 5.4. This makes it not likely that this explanation explains the possible dependency on the water depth.

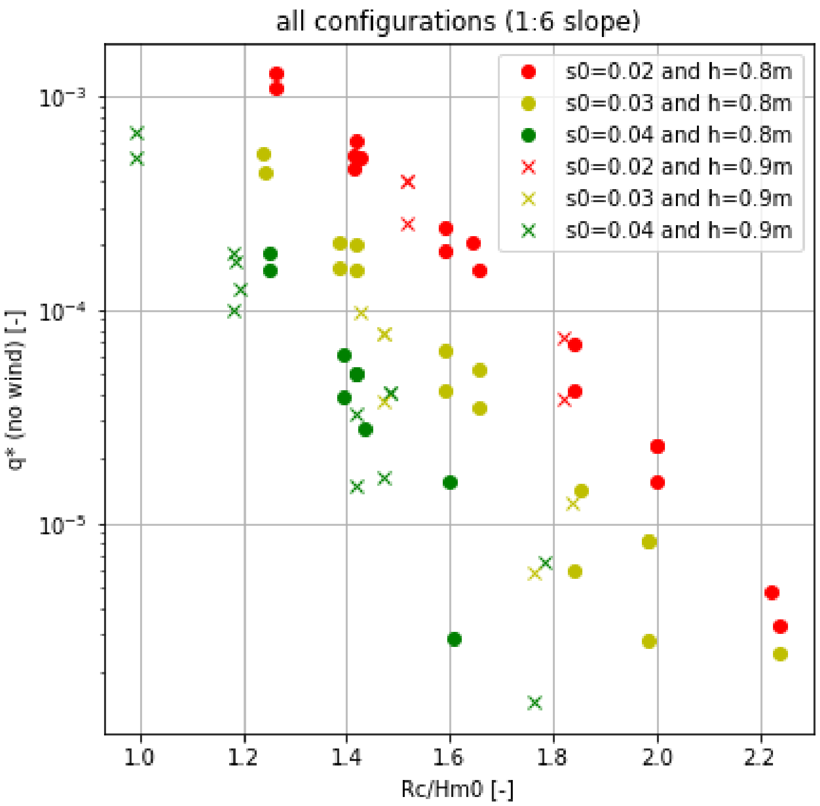


Figure 5.4: The wave overtopping discharge plotted versus the dimensionless crest freeboard including the dependency on the water depth

Wave steepness

The wave steepness is an important parameter regarding wave overtopping. It contains the wave height and wave length/period and in combination with the dike slope it determines the type of wave breaking. All tests were performed with the same plunging type of wave breaking, but the wave steepness (/breaker parameter) was varied. Because the dike slope is constant and the breaker parameter is determined by the dike slope divided by the square root of the wave steepness (equation 2.3), analysing both parameters separately is redundant. Therefore the focus will be on the wave steepness.

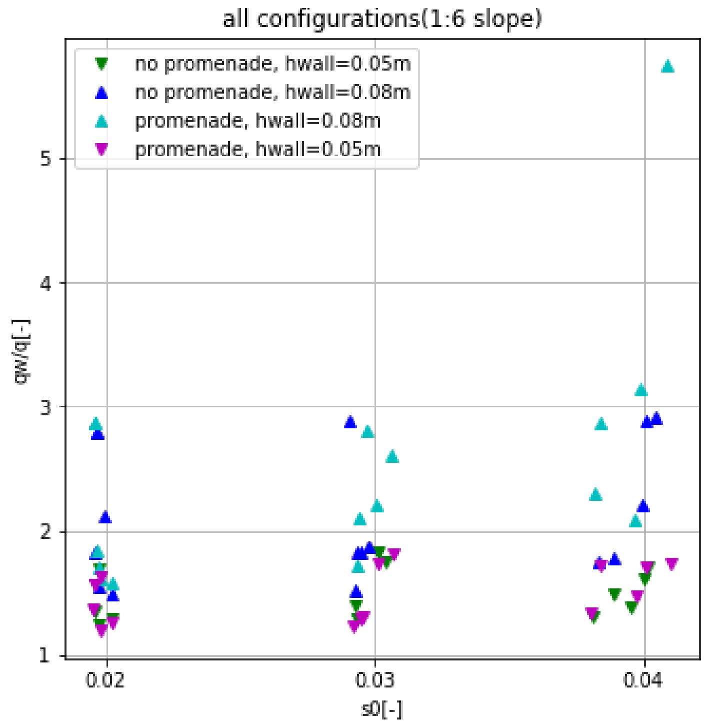
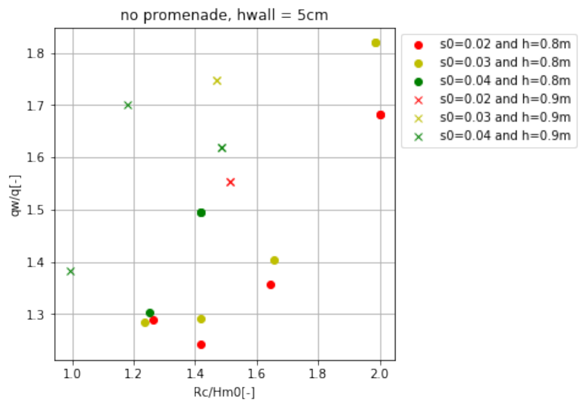
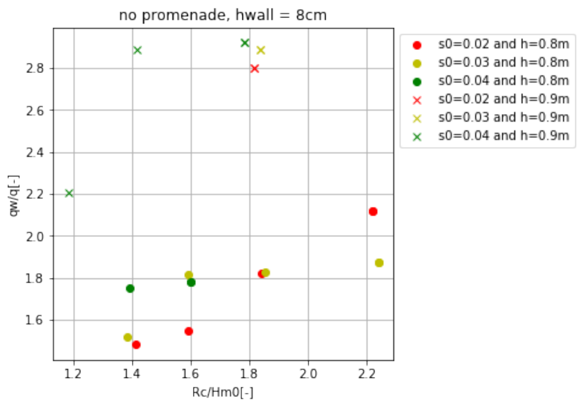


Figure 5.5: Wind influence factor plotted against the wave steepness (all configurations)

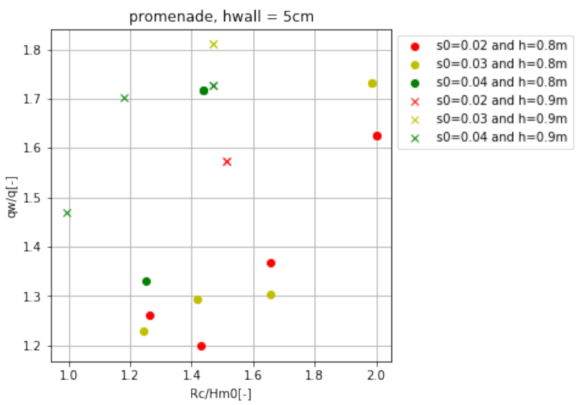
In section 4.1 it was found that for a constant relative crest freeboard, the discharge decreases for an increase in wave steepness. Because a decrease in discharge leads to an increase in wind influence (see figure 5.1), it was expected that an increase in wave steepness leads to a lower discharge and thus a higher wind influence. When looking at a constant dimensionless crest freeboard this trend can be observed in figure 5.6 for some of the dimensionless freeboards/configurations, but the trend is not obvious enough to conclude this dependency is found in general in the data. Also when looking at figure 5.5 where all data is plotted, no dependency on the wave steepness is found. Every wave steepness results in wind influences of the same order of magnitude. So no clear dependency on the wave steepness and breaker parameter is found, something that Wolters and van Gent (2007) also found in their data. In contrary, van der Bijl (2022) found the expected trend more pronounced in his data (1:3 slope).



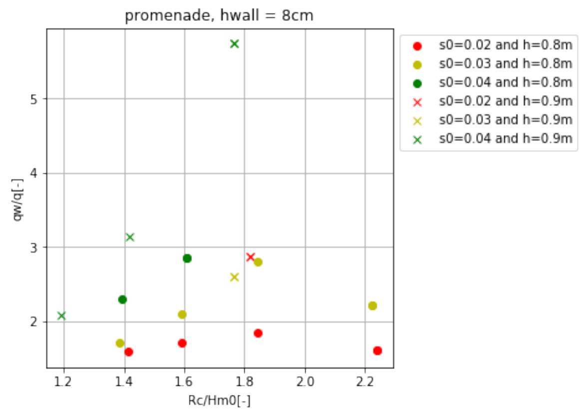
(a) no promenade, $h_{wall} = 5cm$



(b) no promenade, $h_{wall} = 8cm$



(c) promenade, $h_{wall} = 5cm$



(d) promenade, $h_{wall} = 8cm$

Figure 5.6: Wind influence factor plotted against the dimensionless freeboard

5.3. Dependency on the dike configuration (1:6 slope & 1:3 slope)

As explained in the literature study, the dike configuration can determine for an important amount the magnitude of the influence of wind on wave overtopping. So is a vertical structure needed to redirect the jet generated by breaking waves on the dike slope in an upward direction ("spray generation"). During the physical model tests a crest element was used which causes this upward direction, a crest element which can become increasingly important in the future (van Gent, 2019;Hogeveen, 2021). Wolters and van Gent (2007) (1:1.5 and 1:2 slope) and van der Bijl (2022)/van Gent et al. (2023) (1:3 slope), of which the conclusions of the last two are based on the same physical model tests, found that the height of the crest element has an important effect on the influence of wind. The wind influence increases for a higher crest element. Furthermore van der Bijl (2022) and van Gent et al. (2023) found an increase of the wind influence due to the addition of a promenade in front of the crest element for relatively high crest elements (8cm), for the relatively lower crest element (5cm) that trend was not observed. Remember that in this research tests are performed with the same crest elements and promenade (15cm). The validity of those dependencies are all limited to relatively steep slopes, for the relatively flatter slopes (1:6) those dependencies were not validated yet. In this section will be shown how the wind influence depends on the crest element height and promenade for the relatively flatter slope. Furthermore the influence of the dike slope on the wind influence will become clear.

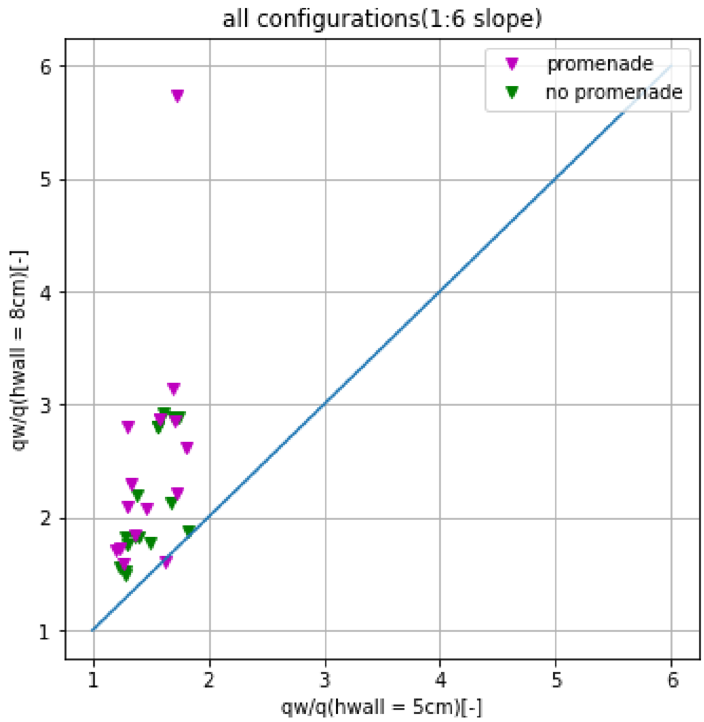


Figure 5.7: Wind influence for different crest element heights (1:6 slope)

Crest element

In figure 5.7 the wind influence for the 1:6 slope is plotted, with on the x-axis the 5cm element and on the y-axis the 8cm crest element. As is shown, the higher 8cm crest element clearly results in larger wind influences than the 5cm crest element. This is in line with what was found for the flatter slopes. An explanation for this is that the increase in crest element height enhances the amount of spray generated and thus increases the wind influences. As was explained in section 3.1.2, the jet generated by wave breaking that reaches the crest element, can be split in a part that goes over the crest element, a part that moves upward ("spray") and a part that inverses direction and goes back into the flume without an upward motion. Due to a higher crest element, the ratio of spray divided by the part that goes over the crest element increases, leading to a higher wind influence. Moreover, the dependency on the crest element height is in line with the found dependency on the overtopping discharge. A higher crest element leads to a lower overtopping discharge and thus an increase in wind influence.

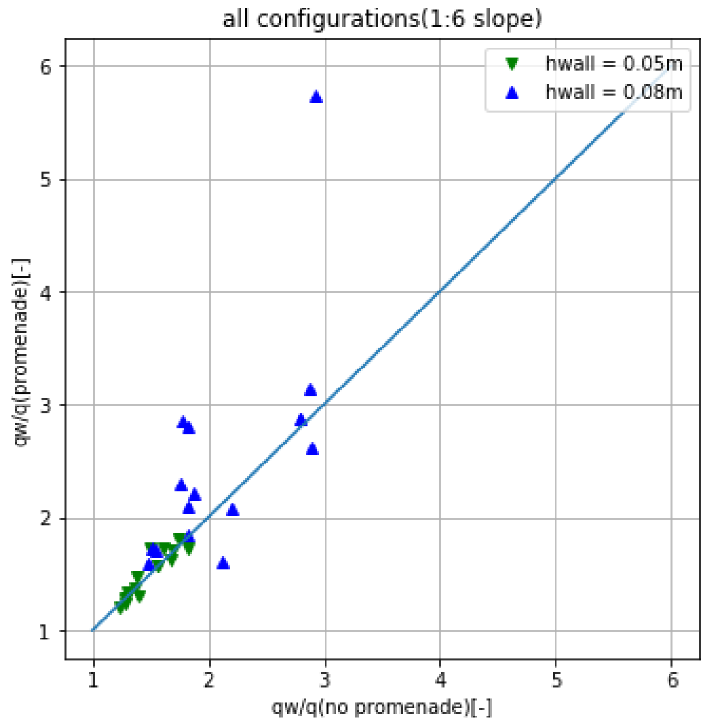


Figure 5.8: Wind influence regarding the use of a promenade(1:6 slope)

Promenade
The same graph is plotted regarding the use of a promenade. To start, it is expected that a promenade reduces the overtopping discharge (see figure 4.8) which result in an increase in the wind influence due to the showed dependency on the overtopping discharge. However, in figure 5.8 it is shown that for the relatively low crest element (green), the influence of a promenade is limited. Although for most conditions the wind influence is a small amount higher without a promenade. In contrary, for the relatively high crest element (blue) there is a more pronounced influence present. Most data lie on or above the 1:1 line, meaning more wind influence when a promenade is used. Although there are also 3 tests were the opposite is shown, a lower wind influence when a promenade is used, however this is less pronounced. For the data-point in the top of the figure it was reasoned before that it is probably too much affected by scale effects.

A comparable outcome was found by van der Bijl (2022)/van Gent et al. (2023) for the 1:3 slope, shown in figure 5.9. The only difference is that for the 1:3 slope, there is a more pronounced dependency on the promenade for the relatively low crest element. Especially for the 1:3 slope it stands out that for the relatively low crest element, the wind influence is larger when no promenade is present, while for the relatively high crest the dependency is the opposite.

van der Bijl (2022) gives two possible explanations for the difference for the different crest element heights. The first explanation has to do with the area above the promenade, in front of the crest element. When a jet reaches the crest element, this area fills with water. When this area is fully filled with water, the remaining part flows over the water volume and crest element, so less spray is generated. For the lower crest element this area is filled with water more quickly, because the volume that has to be filled with water is lower, resulting in a decrease of the wind effect due to the promenade. For the relatively high crest element this is less likely to happen, because the volume that has to be filled with water is larger.

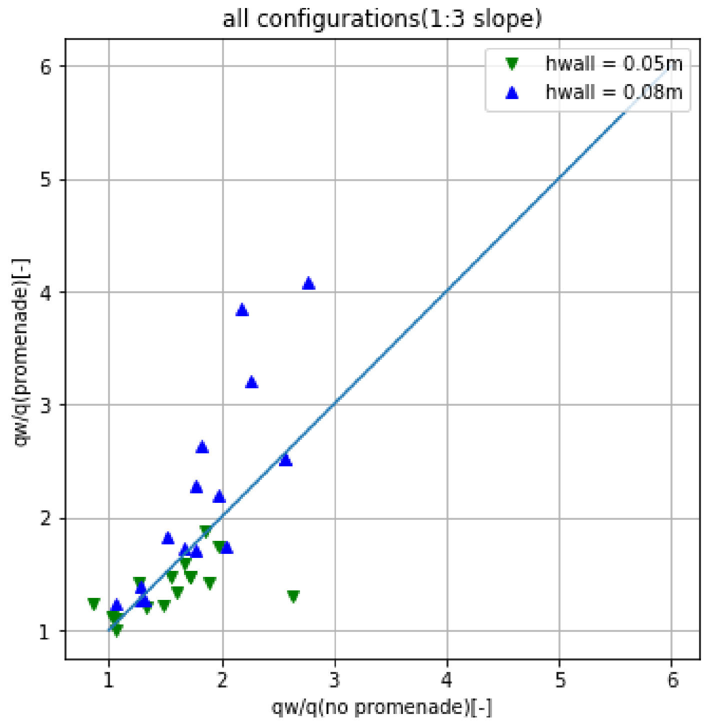


Figure 5.9: Wind influence regarding the use of a promenade(1:3 slope)

The second explanation has to do with overshooting, as shown in figure 5.10. For the lower crest element it is more likely that overshooting occurs. Overshooting takes place when the jet approaches the promenade and crest element with a high speed. When the dike slope is imaginary extended towards the crest element, the jet will follow this extended line en thus shoots over the crest element, without impact on the crest element. Without the impact, limited spray is generated and thus the wind influence reduces. This is less likely to occur for the flatter 1:6 slope, because the imaginary line is more flat and thus crosses the 5cm crest element as is shown in the figure. Thus the jet still results in an impact and spray generation. However the impact is less, because the crest element height the jet impacts on is reduced due to the overshooting and it was already shown that a lower crest element leads to less wind influence.

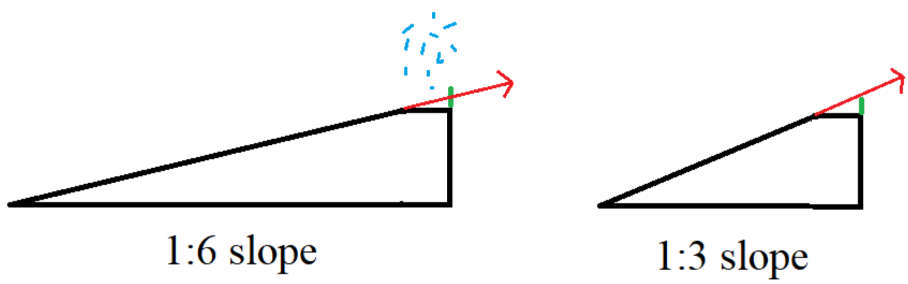


Figure 5.10: Schematic of overshooting

Following this reasoning, the second explanation seems to be most logical, because for the 1:3 slope with a 5cm crest element overshooting results in limited to no impact and thus higher wind influences without a promenade (green, figure 5.9). For the 1:6 slope and 5cm crest element the impact increases resulting in a wind influence approximately equal with and without a promenade (green, figure 5.8). For both slopes with a relatively high 8cm crest element and a promenade (blue) the impact and wind influence increases even more, resulting in a higher wind influence when a promenade is used. Regarding the first explanation, both volumes are equal for both slopes, so for the likeliness of that explanation no difference in the 1:3 and 1:6 slope is expected.

Dike slope

From the previous already a lot was learned on the similarities and differences between the 1:6 and 1:3 slope. The analysis will be finalized with an one on one comparison of the two dike slopes in figure 5.11. For this comparison, tests with index number 11 (see table 3.1) are removed from the data-set. van der Bijl (2022) found these tests to be too much affected by the pumping for the 1:3 slope, which resulted in deviating discharges.

When observing the figure, there is some spreading around the 1:1 line, with a higher concentration of points at the left side. This means that for most wave conditions the relatively flat 1:6 slope result in a higher wind influence, but no obvious dependency is shown. In figure 5.12 the data is plotted per configuration to see if a more obvious trend can be found. From this is learned that the higher concentration on the 1:6 slope side is present for every configuration, except for the configuration without a promenade and 5cm crest element, where the data is equally spread.

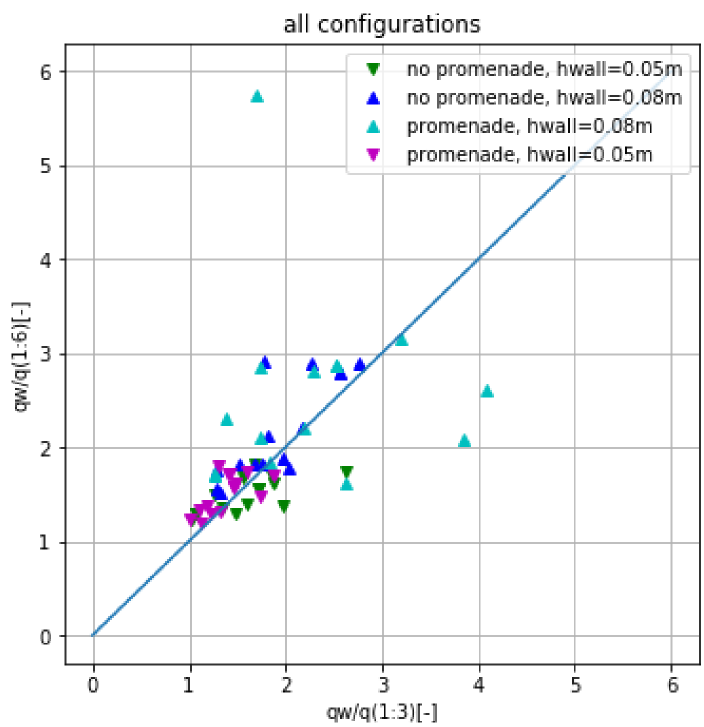
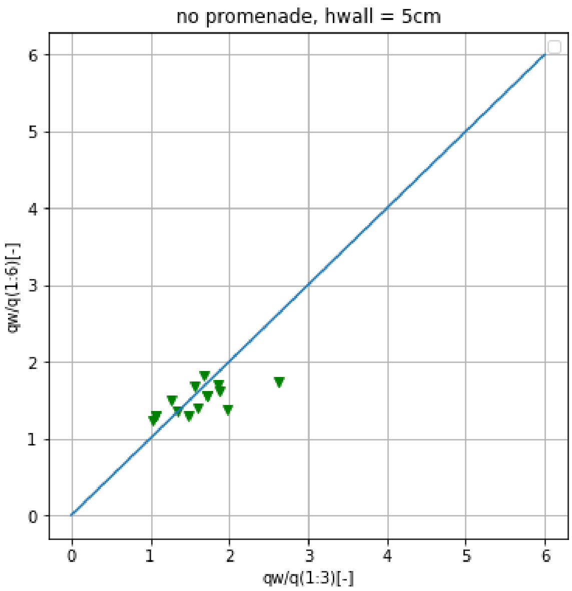
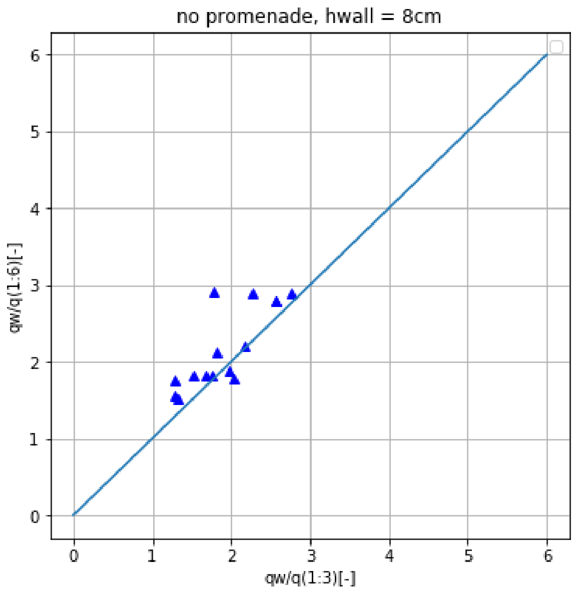


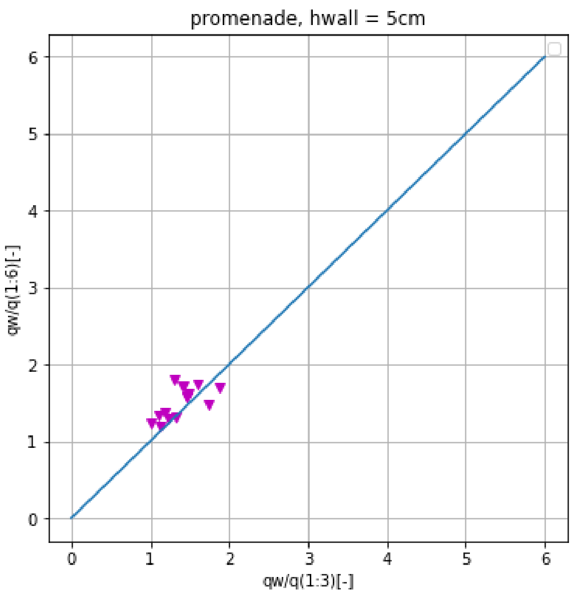
Figure 5.11: Influence of the slope on the wind effect



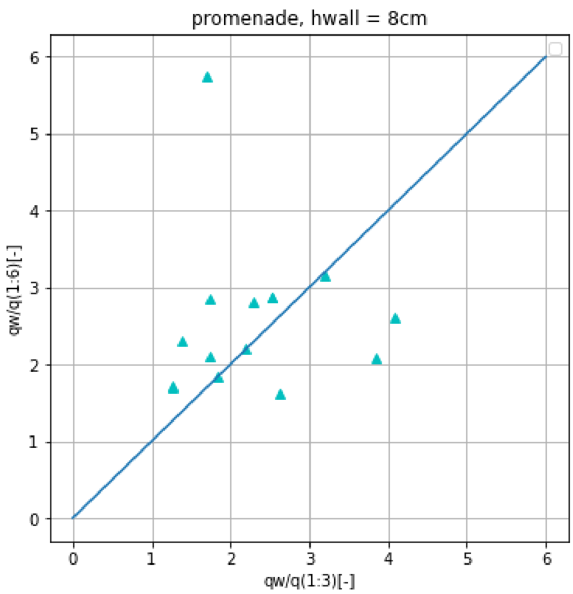
(a) no promenade, $h_{wall} = 5cm$



(b) no promenade, $h_{wall} = 8cm$



(c) promenade, $h_{wall} = 5cm$



(d) promenade, $h_{wall} = 8cm$

Figure 5.12: Influence of the slope on the wind effect per configuration

Lastly it is interesting to see the difference between breaking and non-breaking waves. 16 tests (4 per configuration) of the 1:3 slope data-set contained tests with non-breaking wave data ($\xi_{m-1,0} \approx 2.36$). The same tests were part of the 1:6 slope data-set, but due to the different slope this resulted in breaking waves ($\xi_{m-1,0} \approx 1.18$). Those breaking versus non-breaking data is plotted against each other in figure 5.13.

It is known that breaking waves lead to lower overtopping discharges because energy is dissipated. This should thus lead to a higher wind influence. In figure 5.13 it is shown that despite two outliers, all breaking waves resulted in higher wind influences, confirming the expectation.

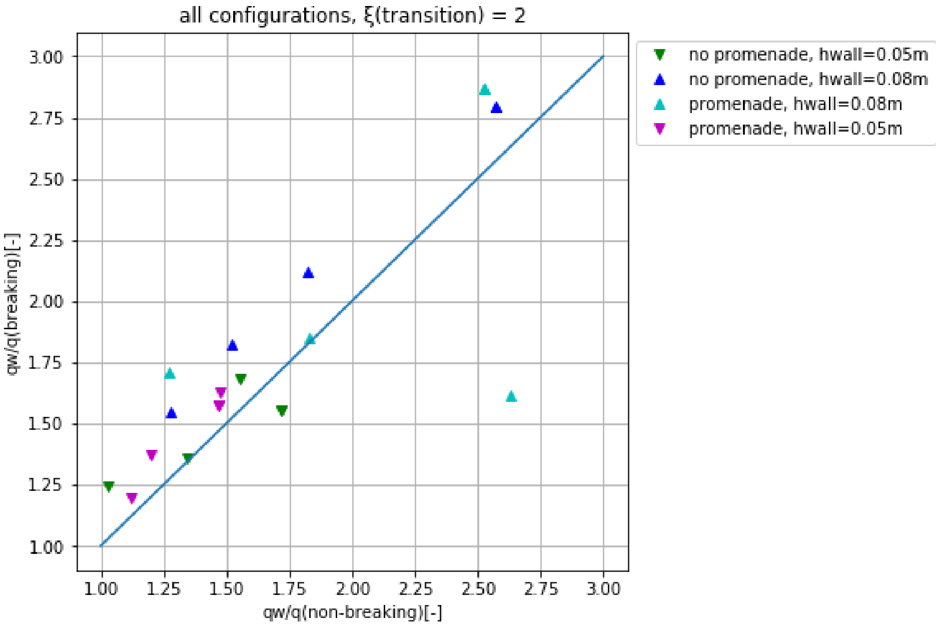


Figure 5.13: Wind influence: breaking versus non-breaking

5.4. The optimal wind amplification factor (1:6 & 1:3 slope)

In the analysis it is shown that the dimensionless discharge and the height of the crest element are important parameters regarding the influence of wind on wave overtopping. It was shown that the higher crest element and relatively lower overtopping discharges generally resulted in higher wind influences. van der Bijl (2022) only included the dimensionless discharge in the amplification factor. Therefore, to improve the amplification factor of van der Bijl (2022), the influence of the crest element height will be added to the formulation. Furthermore it was shown that non-breaking waves generally lead to lower wind influences, this effect is included because for non breaking waves the maximum formula is used (equation 2.5) to determine the overtopping discharge. This leads to higher discharges and thus lower wind influences. For the other studied parameters no strong dependency was found on which the factor could be improved.

Tested parameters	Symbol	Magnitude
Mean overtopping discharge (no wind) [-]	q^*	$4.76E-7 \ - \ 1.73E-3$
Wave steepness [-]	$s_{m-1,0}$	$0.02-0.042$
Relative promenade width [-]	B/H_{m0}	$0-1.53$
Relative freeboard [-]	R_c/H_{m0}	$0.74-3.98$
Dimensionless crest element height [-]	h_{wall}/H_{m0}	$0.25-0.8$
Seaward slope [-]	$tan(\alpha)$	$1:6 \ and \ 1:3$
Breaker parameter [-]	$\xi_{m-1,0}$	$0.83 \ - \ 2.36$
Crest freeboard [m]	R_c	$0.15 \ - \ 0.48$
Crest wall height [m]	h_{wall}	$0.05 \ and \ 0.08$
Promenade width [m]	B	$0 \ and \ 0.15$
Significant incoming wave height [m]	H_{m0}	$0.098 \ - \ 0.202$
Mean spectral wave period [s]	$T_{m-1,0}$	$1.248 \ - \ 2.507$
Water depth [m]	d	$0.6 \ - \ 0.9$

Table 5.1: Magnitude of the tested parameters with van der Bijl (2022) data included (1:3 and 1:6 slope)

To come to an optimal wind factor, the data of the 1:6 slope of this research and the data of the 1:3 slope of van der Bijl (2022) will be combined, resulted in the parameter ranges as included in table 5.1. For this combined data-set, 1 amplification factor will be determined depending on the dimensionless discharge. This factor and the error of the factor will be used as a comparison to quantify the improvements that will be done. In figure 5.14 this factor (equation 5.1) is plotted, which results in a RMSE of 0.483.

$$\gamma_{wind}\ (best\ fit) = 0.051\ q^{*-0.281} + 1 \tag{5.1}$$

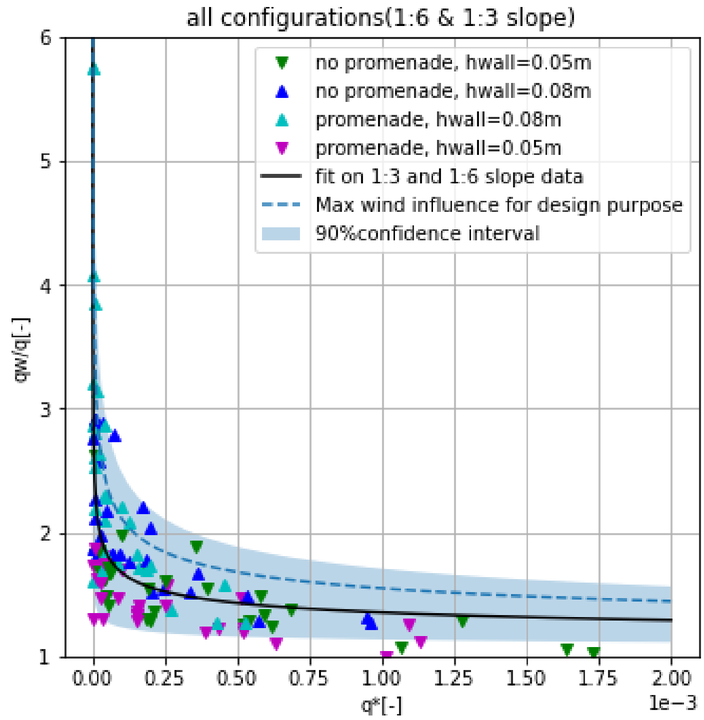


Figure 5.14: Wind amplification factor combined data-set

Note that the function type used is the same as van der Bijl (2022)/van Gent et al. (2023) used and that it fits well to the data. For extreme high values of the dimensionless discharge the function goes to 1, meaning no wind influence. This is needed because for extreme high discharges the water flows over the crest element with no to little spray generation and thus no to little influence of the wind. For low discharges the function rises hard, what corresponds to the data. In the extreme case, only spray is generated and no water flows directly over the crest element, what is in line with the function going to infinity. However it should be noted that for extreme low amounts of overtopping the wind influence is not practically relevant anymore. Furthermore 90% confidence interval is plotted assuming a normal distribution for the calibration constants, with $\sigma(0.051) = 0.025$ and $\sigma(0.281) = 0.0257$.

Factor 5.1 should be an improvement on the factor determined by van der Bijl (2022)/van Gent et al. (2023) (see factor 2.11), as factor 5.1 is calibrated on the combined data-set (1:3 and 1:6 slope) and the factor by van der Bijl (2022)/van Gent et al. (2023) is calibrated on the 1:3 slope data only. Both factors are plotted in figure 5.15 and the improvement is shown by the reduction in RMSE, 0.618 for the van der Bijl (2022)/van Gent et al. (2023) factor and 0.483 for factor 5.1.

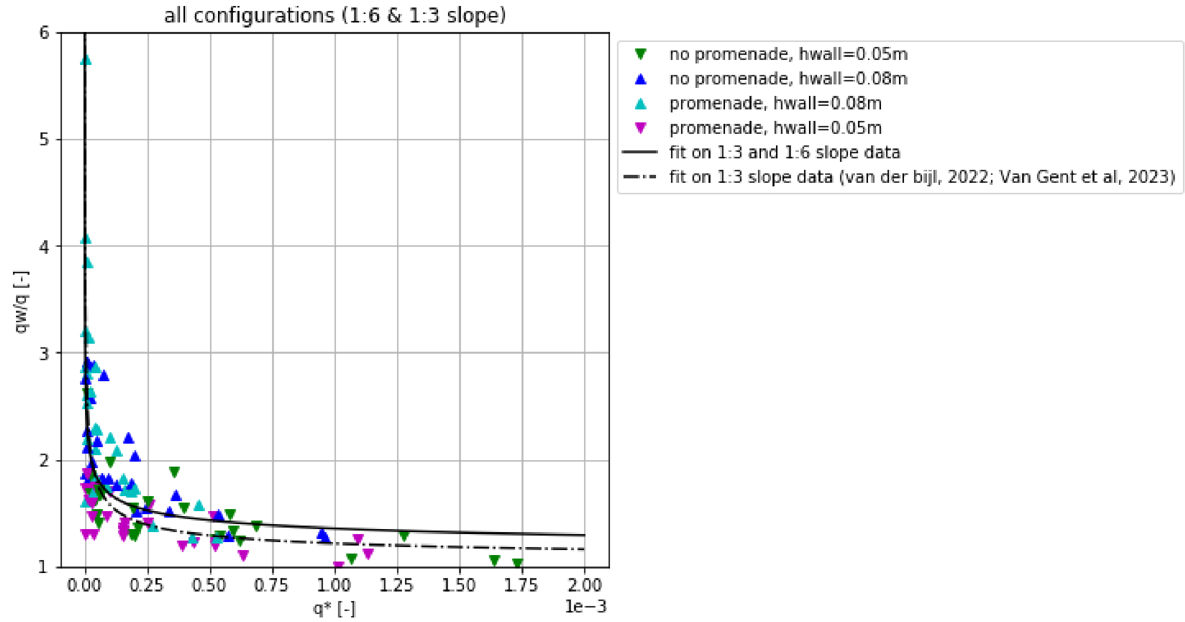


Figure 5.15: Old (van der Bijl, 2022/van Gent et al., 2023) and new wind amplification factor combined data-set

Now the dependency on the crest element height will be added to the equation. In figures 5.1 and 5.7 it was shown that the higher crest element generally leads to higher wind influences for approximately the same overtopping discharges. To add this dependency to the formulation, the formulation will be split in an interval for relatively low dimensionless crest element heights ($h_{wall}* = h_{wall}/H_{m0}[-]$) and an interval for relatively high dimensionless crest elements. Combining the dependency on the dimensionless discharge and dimensionless crest element in 1 function was also applied, but the lowest obtained RMSE by doing this was 0.684, showing no improvement. The 5 centimeter crest element corresponds to dimensionless crest element heights between 0.25-0.5 and the 8cm crest element between 0.4-0.8. There is some overlap on the dimensionless crest elements and the optimal transition point needs to be determined.

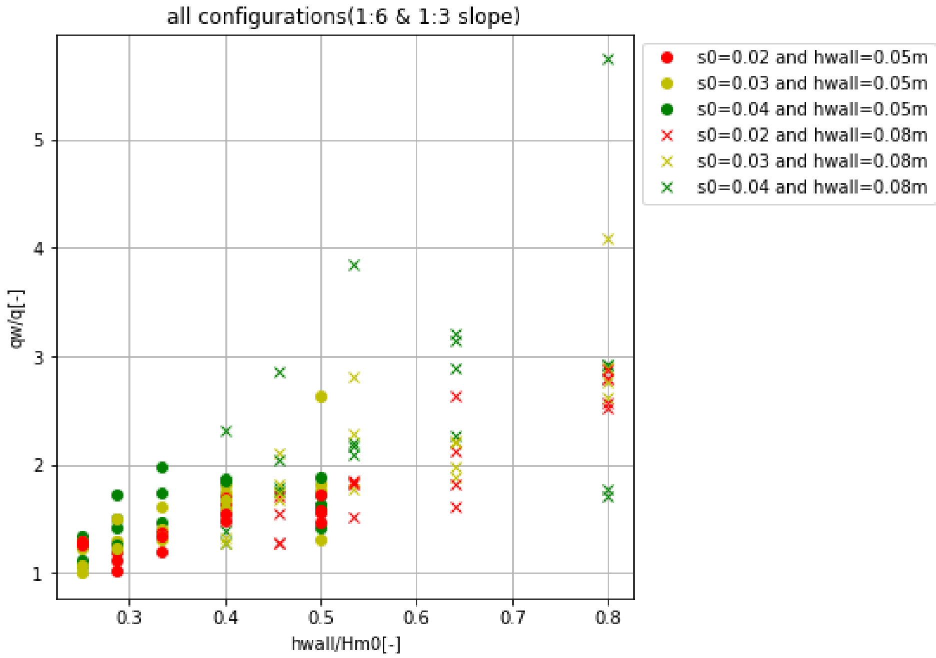
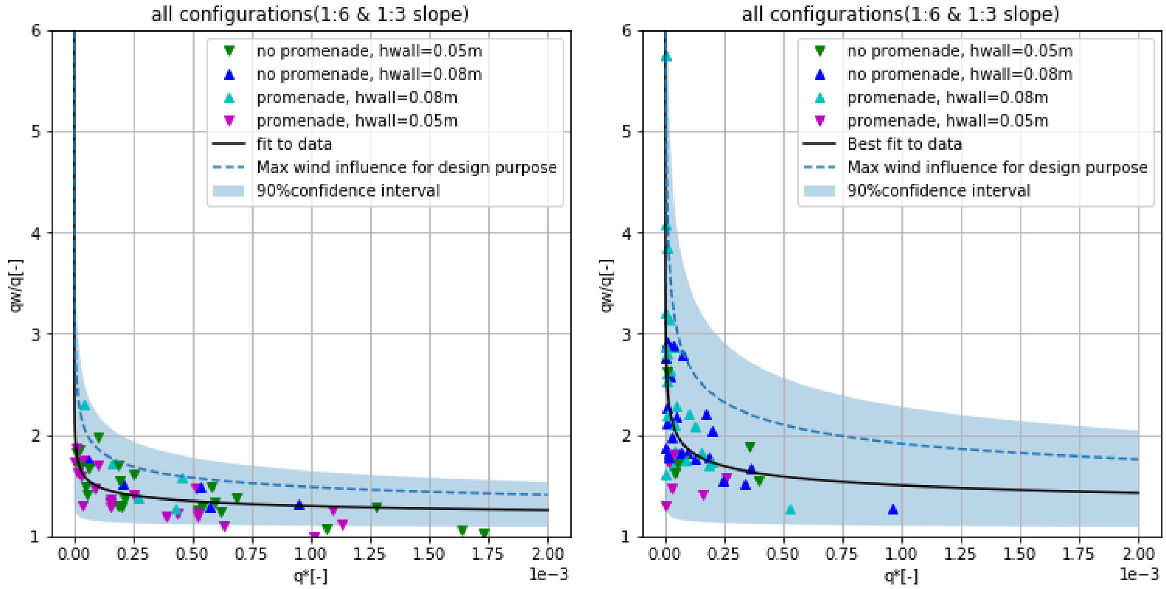


Figure 5.16: Wind influence factor versus dimensionless crest element height (1:3 and 1:6 slope)

In figure 5.16 the wind influence is plotted against the dimensionless crest element height. It can be seen that especially for dimensionless crest element heights above 0.4 the uncertainty in wind influence increases. The transition is chosen around this point, so a relatively high accuracy (low uncertainty) can be obtained for this interval. Including the $h_{wall}* = 0.4$ data in both the low or high interval resulted in approximately equal RMSE's (0.179 versus 0.155 for the low interval and 0.591 versus 0.544 for the high interval). Including the data in the low interval results in an approximately equal division of the data between the two intervals, resulting in enough data-points in both intervals to have a solid foundation for the derived equation. Therefore the transition between the intervals is chosen as is shown in equation 5.2 and figure 5.17. For the low interval, the RMSE is reduced from 0.295 for factor 5.1, to 0.189 for factor 5.2. For the high interval no improvement is gained (0.608 (factor 5.1) versus 0.587 (factor 5.2)). The ranges of applicability of equation 5.2 are shown in table 5.1 and the RMSE's are summarized in table 5.2.

$$\gamma_{wind}\,(bestfit) = \begin{cases} 0.0665\; q^{*-0.217} + 1, & \text{if } 0.25 \leq h^{*}_{wall} \leq 0.40 \\ 0.102\; q^{*-0.231} + 1, & \text{if } 0.40 < h^{*}_{wall} \leq 0.80 \end{cases} \tag{5.2}$$



(a) $0.25 \leq h^*_{wall} \leq 0.40$

(b) $0.40 < h^*_{wall} \leq 0.80$

Figure 5.17: Wind amplification factor including influence dimensionless crest element height

	$0.25 \leq h^*_{wall} \leq 0.80$	$0.25 \leq h^*_{wall} \leq 0.40$	$0.40 < h^*_{wall} \leq 0.80$
van der Bijl (2022)/van Gent et al. (2023)	0.618	-	-
Factor 5.1	0.483	0.295	0.608
Factor 5.2	-	0.189	0.687

Table 5.2: Overview of RMSE's regarding 1:6 & 1:3 slope data-set: van der Bijl (2022)/van Gent et al. (2023) (factor 2.11), general factor (factor 5.1) and h^*_{wall} splitted interval (factor 5.2)

It is shown that for the low interval is chosen to use the same type of function as the high interval, while it can be argued that there is not enough data supporting the upward going trend for the lower discharges. There are 3 arguments explaining this. Firstly, as was mentioned before, this upward going trend is expected from theory. Secondly, it is expected that the low and high interval behave in the same manner. Lastly, as is shown in figure 5.18, there are only 3 data-points in the low interval with a dimensionless discharge lower than E-5, while for these discharges the highest wind influences are found in the high interval. These 3 points are too less to conclude the trend is not present.

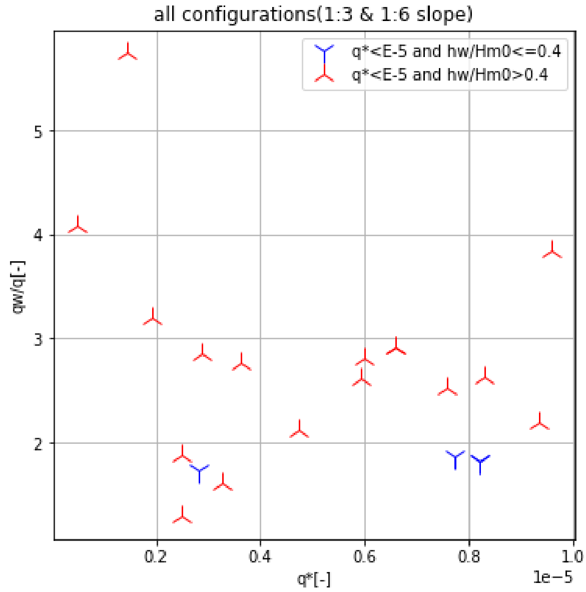


Figure 5.18: low discharges present in the intervals (1:3 and 1:6 slope)

Moreover the 90% confidence interval is plotted in figure 5.17, with $\sigma(0.0665) = 0.0207$, $\sigma(0.217) = 0.0316$, $\sigma(0.102) = 0.0707$, $\sigma(0.231) = 0.0591$. What attracts attention is that the confidence interval in figure 5.17b is quite broad and in figure 5.17a quite narrow. It was noted before that for the higher crest elements, the spreading in wind influences is quite large for a certain dimensionless discharge (see figure 5.1), which is also found back in figure 5.17b. For the lower crest element the spreading was quite small. This difference explains the difference in confidence interval found in the two plots.

For design purposes it is recommended to include 1 standard deviation in the equation's. This is plotted with a dotted blue line in figures 5.14 and 5.17. The equations including 1 standard deviation are as follows:

$$\gamma_{wind}\ (design) = 0.076\ q^{*-0.307} + 1 \tag{5.3}$$

$$\gamma_{wind}\ (design) = \begin{cases} 0.0872\ q^{*-0.249} + 1, & \text{if } 0.25 \leq h_w^* \leq 0.40 \\ 0.173\ q^{*-0.290} + 1, & \text{if } 0.40 < h_w^* \leq 0.80 \end{cases} \tag{5.4}$$

5.5. Applicability

Concluding this chapter, it will be discussed when and how to apply the wind amplification factor. To start, the factor is determined for dikes with a crest element. Without a crest element or another vertical structure which causes the water to spray in the vertical direction, the wind influence is limited and no wind amplification factor should be applied. When this vertical structure is too much deviating from the configuration tested in this research, for example a vertical seawall, it is recommended to consult the references mentioned in the section 2.3 and 2.4 of the literature study.

For comparable structures it is recommended to use one of the amplification factors derived in this research. The factor can be used for both breaking and non-breaking waves, different dike slopes (validated for an 1:3 and 1:6 slope) and is validated for the parameter ranges mentioned in table 5.1. When applying the factor outside the tested ranges for the dimensionless discharge and the dimensionless crest element height special attention is needed.

For extreme low discharges the factor goes to infinity. Testing with those extreme low discharges is not possible with the methods available at the moment due to the domination of scale effects and there is therefore no data available for those discharges. For those extreme low discharges it is important to keep in mind that the maximum wind influence found in this research is 5.74 and maximum factor found in literature is 6.3 (Wolters and van Gent, 2007) for a comparable configuration. Also the wind influence is limited to the water available to be sprayed up in the air. The maximum amount of water available is equal to the amount of overtopping when the crest element is not present, because this is the amount of water that reaches the crest element and can potentially spray up in the air. For extreme high discharges the factor goes to 1 and the factor can safely be used.

Moreover a clear dependency on the crest element height was found. The higher crest element lead to higher wind influences. For dimensionless crest element heights above the defined range, it can be argued that even higher wind influences are found (see figure 5.16). More tests are needed to verify this.

Finally, the wind amplification factor is determined by inserting the dimensionless mean overtopping discharge in the equation. The wind amplification factor obtained by this can be used to multiply with both the dimensionless discharge and/or the mean discharge. Because the wind amplification factor is defined as the overtopping discharge with wind divided by the overtopping discharge without wind.

5.6. Review on the used physical model

The results of this study are based on the performed small scale physical model tests. For some aspects of the model concessions had to be made (e.g. a paddle wheel to model the maximum wind effect), which need to be discussed. For the relevant aspects it will be shown that the concessions have a limited to no influence on the obtained results, resulting in a reliable output:

- The foreshore is horizontal. If in reality wave breaking occurs on the foreshore, while in the model relatively deep water is applied, differences may occur. This limits the range of applicability of the results. Nevertheless, the present results with deep water at the toe are considered to give only a first estimate for conditions with wave breaking on the foreshore. These estimates may still be rather good if the wave conditions at the toe are similar to those tested, whether wave breaking occurs at the foreshore or not.
- The paddle wheel used and the rotation speed of 22 revolutions per minute is the same as Wolters and van Gent (2007) did their tests with. They estimated the efficiency of the wheel to be above 90%. This means that when testing the maximum wind condition with the paddle wheel, it is possible that the wheel misses 10% of the over topped water. Based on visual observations and recordings of tests this estimate seems reasonable.
- Tests are done with a overtopping chute of 0.9 meter and 0.5 meter, while the width of the slope is 1 meter. It is assumed that tests done with the 2 chutes are approximately representative for the full width of the slope. This assumption proved to be appropriate in earlier studies. The assumption was also validated by 1 test where a relatively large amount of overtopping without wind was expected (test 13, no promenade, crest element height of 0.05 meter). That test was chosen to be representative for tests for which the 0.5 meter chute was used. For that specific test, 1 test was executed with the 0.9 meter chute and 1 test with the 0.5 chute. This resulted in approximately equal dimensionless overtopping discharges of $6.90E{-4}$ and $6.96E{-4}$ respectively. Also it was visually observed that for relatively large amounts of overtopping, the water was evenly distributed over the width.
  For the relatively low amounts of overtopping, it was visually observed that the overtopped water was sometimes not evenly distributed over the width. By using the 0.9 meter chute for these tests (90% of the slope width), the consequences of this are limited.
- For relatively large amounts of overtopping, sometimes pumping was needed the empty the overtopping bin, because otherwise the bin would flood and the amount of overtopping could no longer be measured. The water levels were chosen in such a way that for only 5 tests pumping was needed. The time the pump was on during a measurement, was removed during post-processing. Resulting in a incomplete time series due to the pumping. The maximum amount of pumping allowed was 2 times, which resulted in a removal of 6% of the time series for 1 test. For the other tests the maximum removal was 3% of the test duration. In the beginning of chapter 4 it was shown that this didn't lead to deviating discharges and thus this data was useful.

- The waves were measured with 3 waves gauges, with a linear analyses the incoming wave height is determined from the 3 wave gauges. Based on knowledge available at Deltares, the assumption of linear analysis provides accurate estimates of the wave heights, with negligible differences compared to application of a non-linear analysis to separate incident and reflected waves.

6

Conclusion and Recommendations

6.1. Conclusion

The goal of this research was to get a better understanding in the influence of wind on wave overtopping on mildly sloping dikes with a crest element. To do so, small scale physical model tests were performed on a dike slope of 1:6. The outcome of these tests in combination with the former comparable study done by van der Bijl (2022) with a 1:3 slope, resulted in a large data-set containing the dependency on the crest element height, promenade, dike slope and the wave conditions. From this data-set the following wind amplification factor's were determined:

$$\gamma_{wind}\ (best\ fit) = 0.051\ q^{*-0.281} + 1 \tag{6.1}$$

$$\gamma_{wind}\ (bestfit) = \begin{cases} 0.0665\ q^{*-0.217} + 1, & \text{if } 0.25 \leq h_w^* \leq 0.40 \\ 0.102\ q^{*-0.231} + 1, & \text{if } 0.40 < h_w^* \leq 0.80 \end{cases} \tag{6.2}$$

Where the main difference between equations 6.1 and 6.2 is that equation 6.2 has a higher accuracy for $0.25 \leq h_w^* \leq 0.40$. The standard deviations of the equations are: $\sigma(0.051) = 0.025$, $\sigma(0.281) = 0.0257$, $\sigma(0.0665) = 0.0207$, $\sigma(0.217) = 0.0316$, $\sigma(0.102) = 0.0707$, $\sigma(0.231) = 0.0591$ and the factors are being an improvement in comparison with the factors defined in van der Bijl (2022) and van Gent et al. (2023).

The wind amplification factors can be used for both dike slopes and is defined as the overtopping discharge with wind divided by the overtopping discharge without wind. Note that this ratio is the same for both the dimensional and non-dimensional overtopping discharge, although in the shown formulations the non-dimensional discharge must be used. Also factors for design purposes was determined, containing 1 standard deviation. Because the influence of wind causes an increase in load, these factors leads to higher wind influences:

$$\gamma_{wind}\ (design) = 0.076\ q^{*-0.307} + 1 \tag{6.3}$$

$$\gamma_{wind}(design) = \begin{cases} 0.0872\ q^{*-0.249} + 1, & \text{if } 0.25 \leq h_w^* \leq 0.40 \\ 0.173\ q^{*-0.290} + 1, & \text{if } 0.40 < h_w^* \leq 0.80 \end{cases} \tag{6.4}$$

When applying the amplification factor, the non-dimensional overtopping discharge must be determined first (including all influencing factors, e.g. a berm). This non-dimensional overtopping discharge is applied in the formulation. The overtopping discharge with wind is determined by multiplying the overtopping discharge (dimensional or non-dimensional) with the amplification factor.

To come to the wind amplification factor, research questions were defined to better understand the wind influence regarding the relatively mild 1:6 slope, van der Bijl (2022) did this for the 1:3 slope. The conclusions with respect to the research questions about the influence of the promenade, crest element and dike slope regarding the maximum wind influence will be treated below.

Crest element height
First the crest element height, tests were performed with two heights of the crest element, of which the higher crest element generally lead to higher wind influences. This was in line with the studied literature. Due to the higher crest element leading to higher wind influences, also the spreading increased for the higher crest element. The dependency on the element height is included in the amplification factor and the increase in spreading (uncertainty) can be found back in the before mentioned standard deviations.

Promenade in front of a crest element
Secondly the promenade in front of the crest element. The only literature available on this was the study by van der Bijl (2022). For the relatively low crest element no influence was found, for the high element somewhat higher wind influences are found with a promenade (1:6 slope). van der Bijl (2022) found for the low crest element, somewhat higher wind influences without a promenade and for the higher element the opposite (1:3 slope). The difference in behaviour for the low element is explained with the concept of overshooting as shown in figure 6.1. Overshooting is more likely to occur with the low element and the 1:3 slope, because spray generation due to impact on the crest element is needed for the wind influence to become important. For the low element and 1:3 slope this leads to the somewhat higher wind influences without a promenade. With the low element and 1:6 slope there still is some impact on the element, explaining that there is no difference with or without a promenade. This effect was not large enough to include it in the amplification factor, although the promenade leads to a small reduction of the dimensionless discharge and thus a small increase of the wind amplification factor due to the promenade.

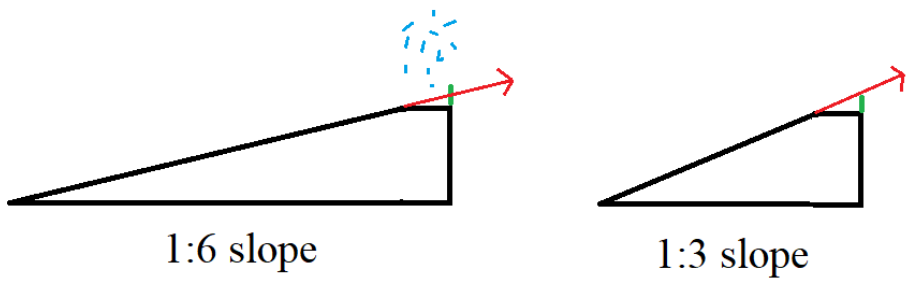


Figure 6.1: Schematic of overshooting with the relatively low element

Dike slope
The last research question was about the influence of the dike slope with respect to the influence of wind on wave overtopping. No clear dependency was found on the dike slope. For some of the tests the 1:6 slope resulted in somewhat higher discharges and some of the tests the 1:3 slope. This was approximately equally spread, but with a somewhat higher concentration with larger wind influences for the 1:6 slope.

The dike slope has an effect on the wind influence, because it determines in combination with the wave steepness the type of wave breaking on the dike slope. Breaking waves are defined as $\gamma_b * \xi_{m-1,0} <\approx 2$ and non-breaking waves as $\gamma_b * \xi_{m-1,0} >\approx 2$ (reduction factor for a berm times the breaker parameter). Following this definition, the breaking waves tests generally lead to higher wind influences. This is in line with the expectations, because breaking waves generally lead to lower overtopping discharges which lead to higher wind influences. Besides the influence on the wave overtopping discharges and consequently on the influence factor for wind, no additional influence of breaking versus non-breaking waves is observed.

Reduction factor for a crest element (breaking waves)
While analysing the data without wind influence, it became clear that the reduction factor for a crest element with breaking waves on the dike slope, determined by Van Doorslaer et al. (2016) and as in-

cluded in the latest version of the overtopping manual (EurOtop, 2018), did not match the data obtained in this research. In those references it is recommended to use a constant reduction factor of 0.92, while the data in this research suggests that it is better to use no reduction factor. So only the height of the crest element has a contribution to the crest freeboard. No noteworthy differences between the tests in this research and the research of Van Doorslaer et al. (2016) are known, so the origin of the difference is not known. It is recommended to use no reduction factor until more knowledge is gained on this topic.

Reduction factor for a promenade (breaking waves)
The effect of a promenade with breaking waves on the dike slope was yet unknown. With the data-set obtained in this research it was possible to derive a reduction factor for the use of promenade:

$$\gamma_{prom_v} = 0.96 \; (breaking \; waves) \tag{6.5}$$

Important to note is that the factor is determined for dikes with a combination of a promenade and a crest element. When the factor is applied without a crest element it is important to be aware that there is quite some interaction between the crest element and the promenade. van Doorslaer et al. (2015) found that for non-breaking waves the combination of a promenade and crest element leads to an additional reduction of 0.87. So when only a promenade is present without a crest element the reduction effect can be less.

6.2. Recommendations for further studies

- With the data obtained without wind influence a reduction factor was determined for the use of a promenade in front of the crest element with breaking waves on the dike slope (section 4.2.2). This was determined in combination with a crest element. It was argued that with only a promenade and no crest element it is expected that the reduction effect is less. It is recommended to verify this with additional physical model tests. Furthermore van Doorslaer et al. (2015) found for non-breaking waves the reduction effect to depend on the promenade length. Because a constant promenade length was used, it was not possible to study the influence of the promenade length in this study. It is recommended to study the effect of the length of the promenade in combination with breaking waves.
- It was found that it is best to use no reduction factor for the use of a crest element in combination with breaking waves on the dike slope regarding the data obtained without wind (section 4.2.1). While Van Doorslaer et al. (2016) found a reduction factor of 0.92. There are no noteworthy differences between the two studies explaining the inequality in outcome. It is endorsed to do additional physical model tests with a wide variety of conditions, so a definite answer can be given on when which factor should be used.
- In section 5.2 it was found that the wind influence possibly has an (unknown) dependency on the water depth. The data obtained was not usable to give a useful explanation for this. More physical model tests focusing on this possible dependency on the water depth are recommended.
- In section 5.4 the amplification factor for the influence of wind was defined which was split in a formulation for relatively high dimensionless crest elements and low dimensionless crest elements. It was shown that in the low interval the data with the lowest discharges is limited ($q^* < 10^{-5}$), while for these discharges the highest wind influences can be expected. To have a stronger substantiation for the formulation used for the low interval it is recommended to do additional tests for the low interval aiming for these low discharges.
- As mentioned in the same section 5.4, the uncertainty for the high interval is relatively large. In this high interval the largest wind influences are found. For approximately the same dimensionless discharge, the wind influence can vary from approximately 1.5 to 5.7, explaining the large uncertainty. Due to the wide variety of data the factor is based on, it is expected that this uncertainty can not be reduced a lot by additional physical model tests. Therefore it is recommended to use the design formulas given, which shows a good fit to the highest wind influences found. In the future it is expected that artificial intelligence models can result in a decrease in the uncertainty.
- Finally this study and the former study done by van der Bijl (2022) was about a combination of a promenade and crest element. The combination of other dike elements like a crest element

and a berm can be studied with respect to the influence of the wind. As it was learnt in the literature study that the combination of reducing elements can behave somewhat different than just multiplying reduction factors. Making it plausible to assume that the influence of the wind can also behave differently.

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A

Figures of model set-up

A.1. Technical drawings of the model

In the following technical drawing are shown of the model set-up, a schematic with definitions is shown in 3.1. The technical drawings are made by Wesley Stet, a model technician working at Deltares. In the first figure A.1, blue corresponds to the paddle wheel, red to the overtopping bin and in black the concrete blocks which formed the wave flume inside the basin. The frame supporting the paddle wheel is also shown in black.

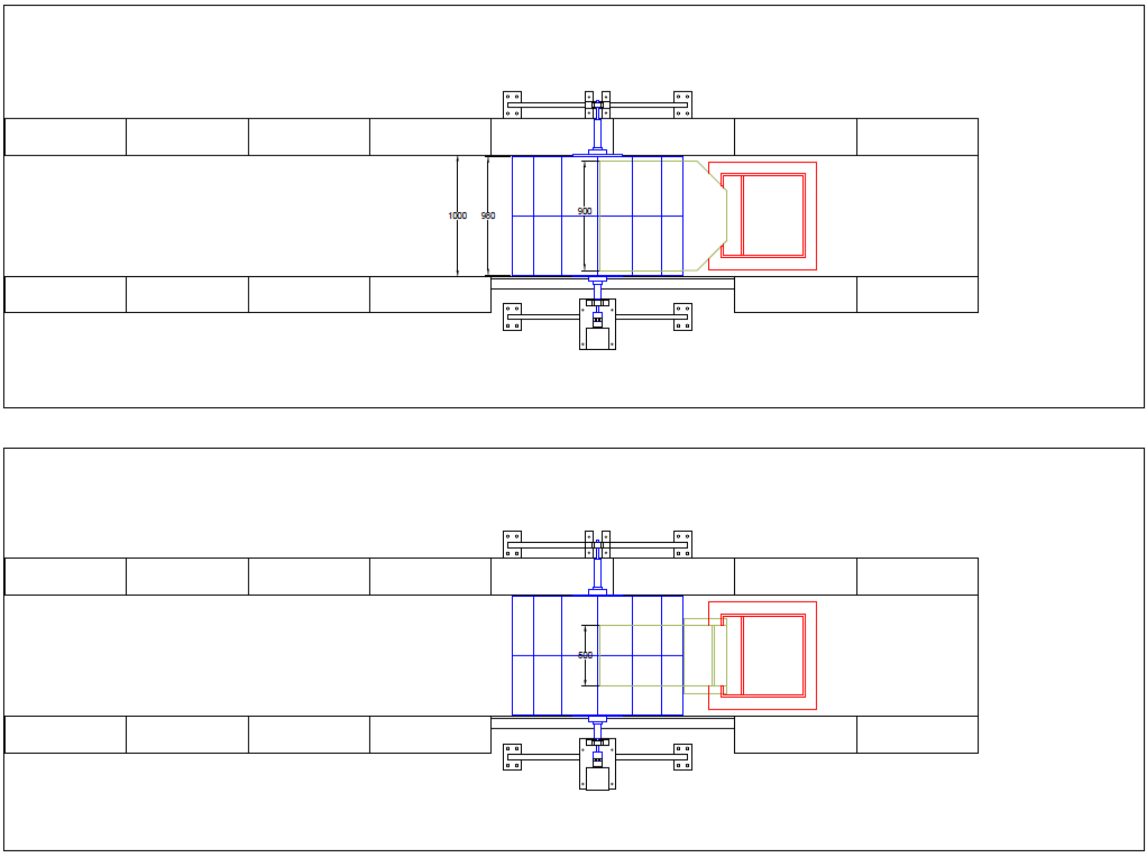

Figure A.1: Technical drawing of the model set-up: top view

In the second figure A.2, the colours indicate the same elements of the set-up as in the first figure. The dike slope is added in green and the structure in between the paddle wheel and the overtopping bin is the overtopping chute and splash screen .

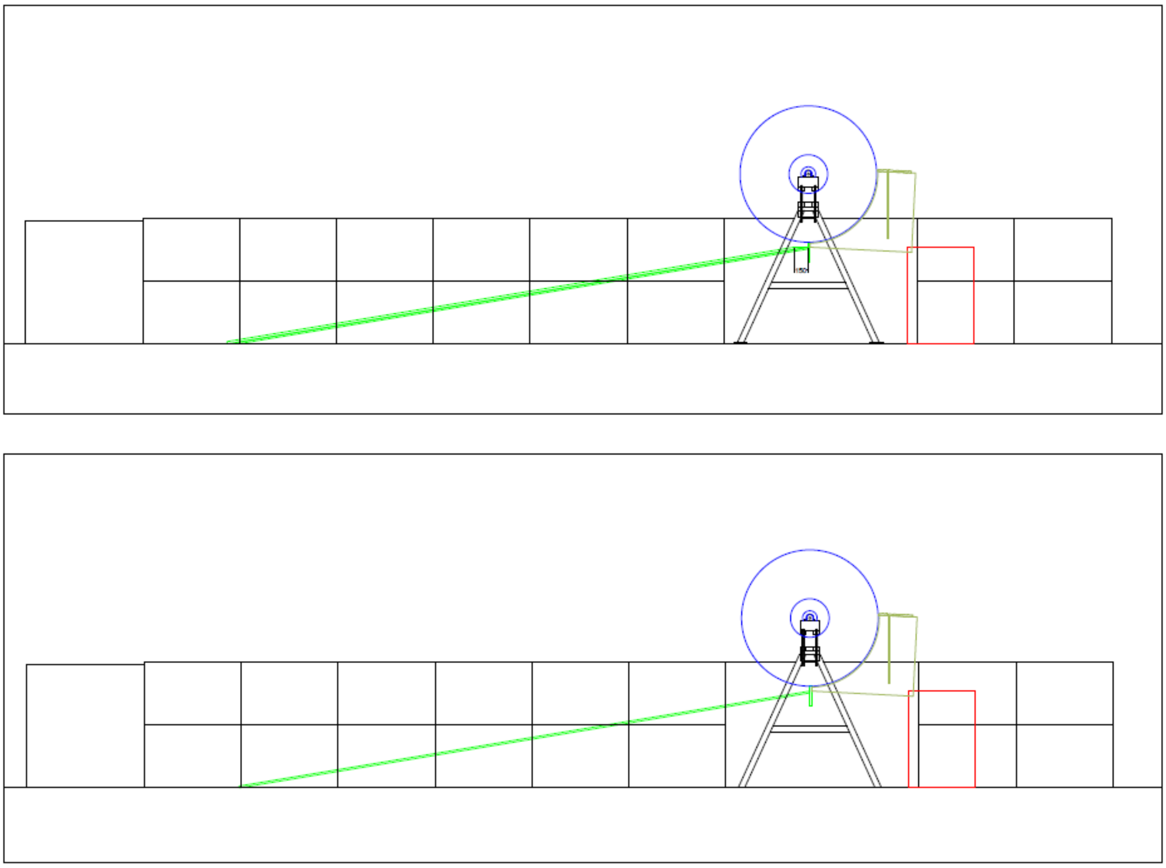

Figure A.2: Technical drawing of the model set-up: side view

In the third figure A.3 the colours indicate the same elements as figures A.1 and A.2. In this figure green also corresponds to the crest elements.

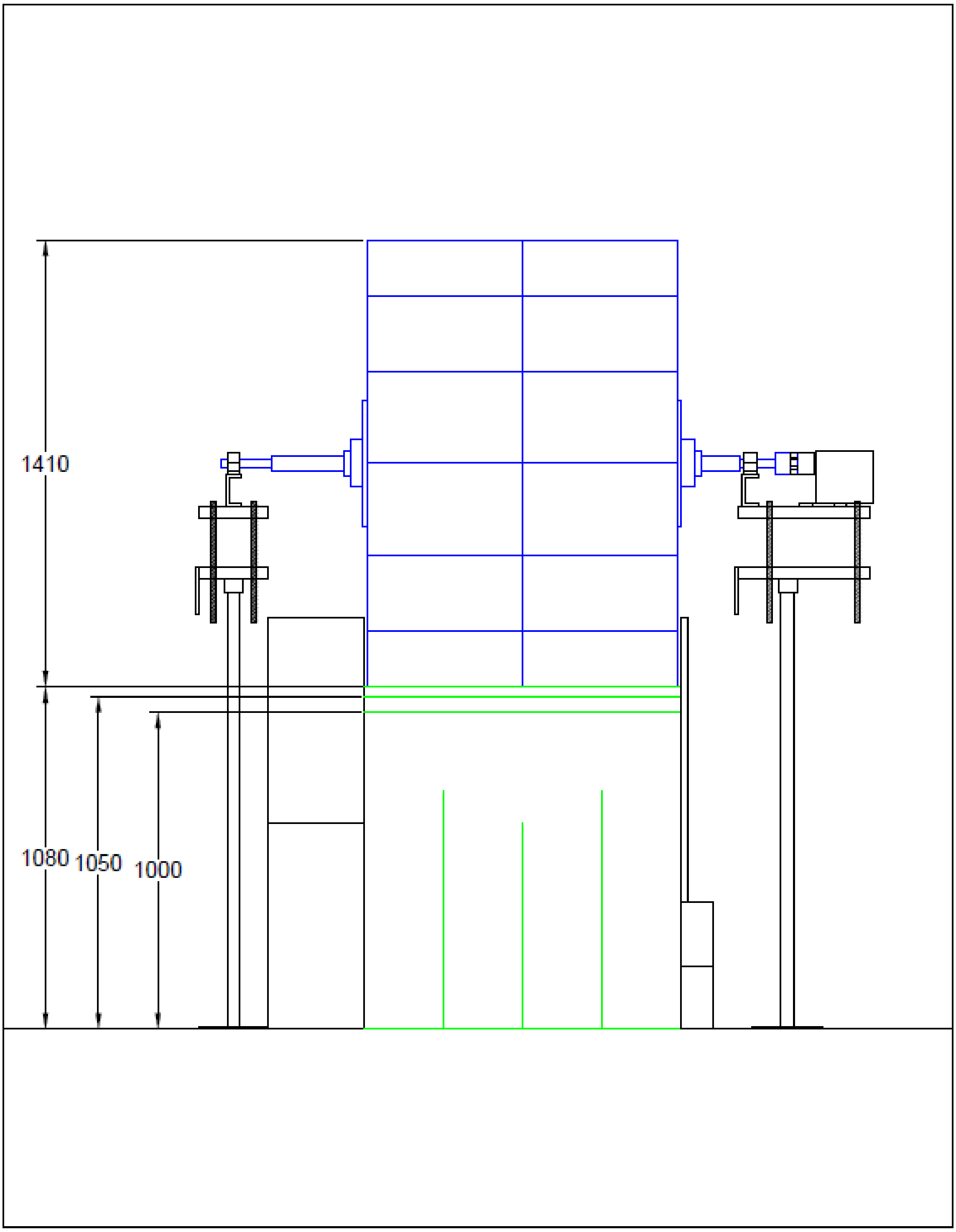

Figure A.3: Technical drawing of the model set-up: front view

A.2. Pictures model set-up

Below a collection of pictures made during the physical model tests are shown which can help interpret the results obtained in this research and help visualising the method used.

Figure A.4: Side view of the model set-up(maximum wind)

Figure A.5: Front view of the model set-up(maximum wind)

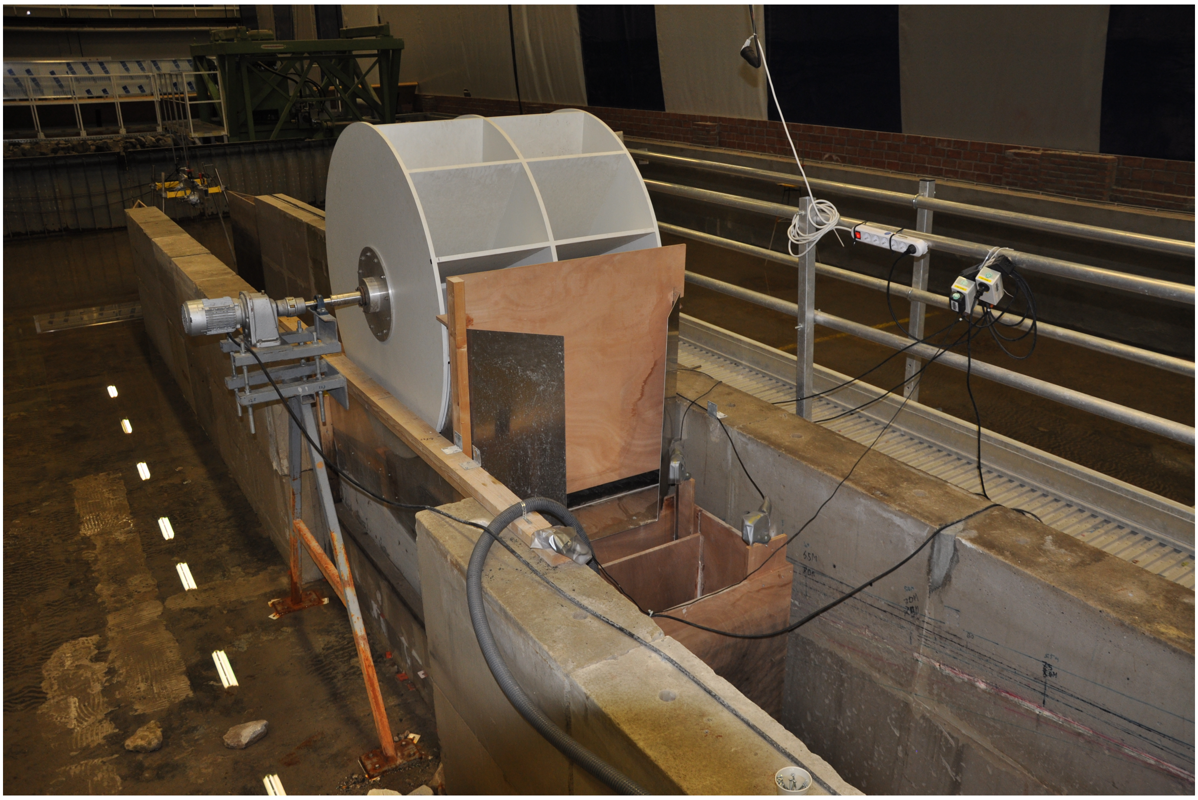

Figure A.6: Back view of the model set-up(maximum wind)

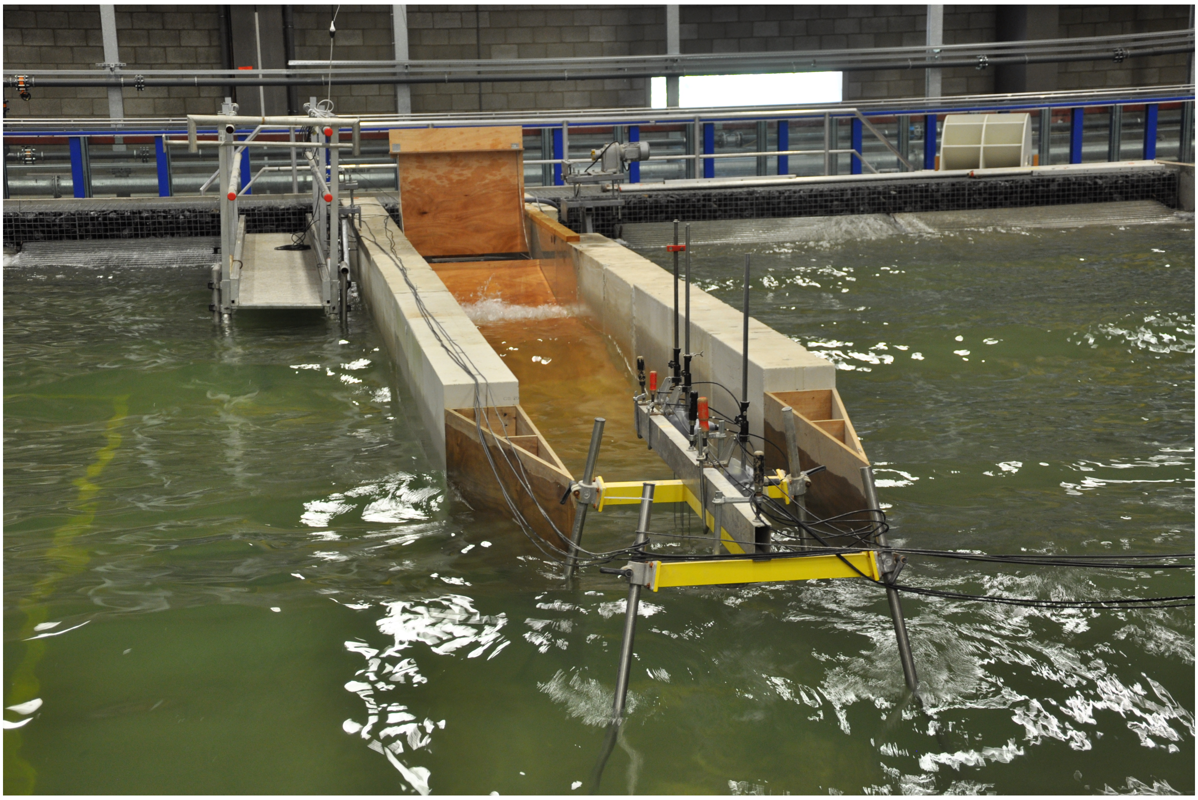

Figure A.7: Front view of the model set-up(no wind)

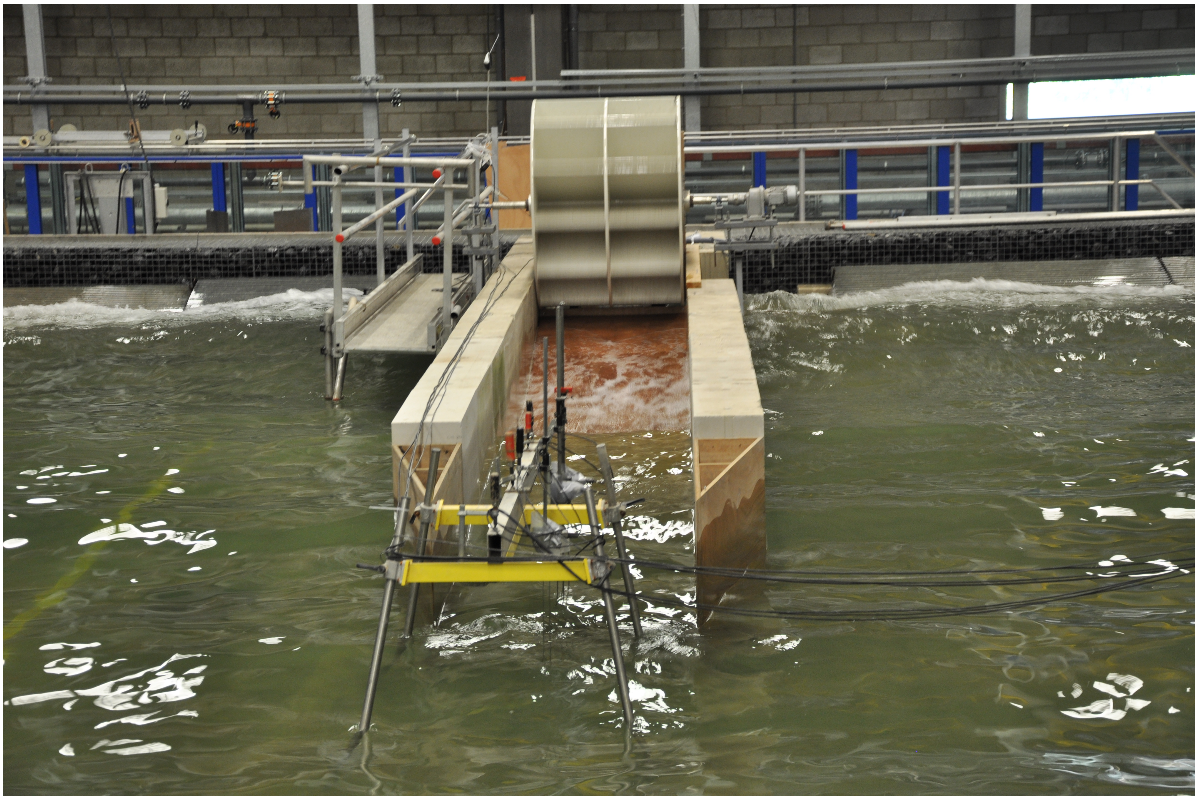

Figure A.8: Front view of the model(maximum wind)

Figure A.9: Connection between chute and paddle wheel, the losses are minimized

Figure A.10: Connection between crest element and paddle wheel, the losses are minimized