H-wall

Research of a temporary flood barrier

Delft, April 2023

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(Kroonen, 2023)

Preface

The report you are holding is written for the bachelor thesis 'H-wall' of the faculty Civil Engineering at the Technical university of Delft. The main objective of the thesis is to research a temporary flood barrier known as the H-wall . This report is written to complete the bachelor programme of Civiele Techniek at the Technical University Delft.

I would like to thank my supervisors ir. J.R. Moll and dr. ir. D. Wüthrich. They guided me during this project. It's thanks to their knowledge, insight and contacts that I was able to write this report. I would also like to thank Tonnie Hospers for the help and enthusiasm. Finally I would like to thank ir. Lindsey Schwidder for her critical thinking, knowledge and guidance and Jean Paul de Garde for helping me with tests of the barrier at Flood Proof Holland.

I completed this bachelor thesis with pleasure!

Seppe Smeijers

TU Delft, April 2023

Summary

This thesis is focused on testing and demonstrating the H-wall and suggesting improvements for future versions of this temporary flood barrier. During the summer of 2021 regions in Belgium and Germany flooded and there were extreme water levels in the Dutch province Limburg. Sandbags were the temporary flood barrier of choice because they have proven their suitability through the centuries. They unfortunately have some pretty severe drawbacks. Placing the sandbags is a slow, expensive and labour-intensive process. To solve these limitations Altena came up with the H-wall as an alternative temporary flood barrier.

The H-wall is a simple structure consisting of grid panels, plastic foil and tension cables. Each section of H-wall consists of two panels. The first panel is placed flat on the ground and the second panel slides in and stands upright to form an L-shape. These sections can be placed next to each other to reach the length required to protect an area from flooding. When all the sections are in place the plastic foil is placed on top of the panels to make the structure watertight. The idea behind this light weight structure is that the weight of the water on top of the floor panel will always be bigger than the water pressure against the vertical panel. In order to prevent water from flowing under the foil initially, a line of sandbags is placed along the edge of the foil. For added safety the two panels of each section are connected using tension cables.

The flood barrier is made from lightweight and cheap materials and therefore is a promising alternative to sandbags. The focus of this project is to answer the question:

"What are the flood protection capabilities and limits of the H-wall and how can it be improved".

This question is answered by looking at the stability and failure mechanisms of the barrier and doing field tests.

The barrier hasn't been put through many tests yet and is the first iteration of the design. Altena asked the TU Delft to help them test, research and improve the design.

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1. Introduction

The TU Delft in collaboration with private and public partners such as Waterschap Limburg does research of temporary flood barriers. Rivers have to discharge a lot more water than what they are capable of due to heavy rain caused by climate change. Traditionally sandbags are used to temporarily increase the height of flood barriers during periods of extreme water levels, like at the end of the summer 2021 when regions in Belgium and Germany flooded and the water level rose significantly in Limburg. Placing the sandbags is a slow, expensive and labour-intensive process. This means there is a market for alternative solutions to combat high water levels.

The H-wall is designed to be such an alternative solution. It has been tested and will be subjected to more tests in the near future at Flood Proof Holland , a testing location of TU Delft. To test it under prototype conditions and demonstrate its capabilities it could also be tested at the field test location in Limburg. This has an added benefit of media coverage which can greatly raise the public support for adoption by the waterschappen or other institutions.

Altena, the developer, is especially interested in the performance of the barrier over longer periods of continuous use. Their goal with the barrier is for it to be usable in as many situations as possible. These situations range from private property protection during small floods all the way to stopping large floods by increasing the height of permanent barriers.

This bachelor thesis will answer the following research question:

“What are the flood protection capabilities and limits of the H-wall and how can it be improved?”

To facilitate the answering the research question is divided into the following sub questions:

1. What are the failure mechanisms of the H-wall and under which conditions do they occur?
2. Which water levels can be safely retained under what conditions (weather, topography & soil)?
3. How can the H-wall be improved?

In chapter 2 methods used are explained. Chapter 3 is about the loads and their computation . The failure mechanisms are covered in chapter 4. A description of the field tests is given in chapter 5. In chapter 6 the process and thesis are evaluated and discussed. In chapter 7 possible improvements are given and suggestions are made for future research. A conclusion is made in chapter 8.

2. Method

Each sub-question of the research question will be answered using a method. These methods will be covered in this chapter. For all sub- questions the following is true: the input of supervisors and the developer will be included.

Sub-question 1:

“What are the failure mechanisms of the H-wall and under which conditions do they occur?”

Simple calculations will be used to predict the behaviour of the temporary flood barrier under different conditions. The results will be verified using field tests at flood proof holland. Due to limited time (eight weeks) only a few tests can be performed. These will be on situations that are most critical according to the calculations or interest the supervisors and developer.

Sub-question 2:

“Which water levels can be safely retained under what conditions (weather, topography & soil)?”

This sub-question will be answered in a similar way to sub-question 1. It is therefore expected that most answers to this question will be obtained during the process of answering sub-question 1.

Sub-question 3:

“How can the H-wall be improved?”

This sub-question will be answered in different ways. The first one is by trying to solve problems or inconveniences discovered during the setup of the barrier for testing. The second way is to look at the design and see if improvements can be made. The third way is to look at other temporary flood barriers for inspiration to solve issues. Few or none of these improvements will be tested due to limited time.

3. Loads

In order to calculate possible failure mechanisms the loads acting on the temporary flood barrier should be computed first. This is done using simple computations to get a rough estimate. The loads covered in this chapter are wind load, hydrostatic load, under pressure, wave load and self-weight. Since the length of the barrier can vary the loads are calculated on a single segment of the barrier.

3.1 Wind loads

The wind direction in a governing load combination is perpendicular to the barrier and directed away from the water. In this way the load contributes to the hydrostatic and wave loads. Another direction to look at is towards the water since the foil could catch the wind like a sail and lift the barrier. To determine the magnitude of the load we first must calculate the wind load per m^2. This goes according to the following formula (Voorendt, c. 2022):

$$p_{rep} = C_{dim} * C_{index} * C_{eq} * \emptyset_1 * p_w \left[\frac{kN}{m^2}\right]$$

Equation 1: Wind load

Where:

P_{rep}	$\left[\frac{kN}{m^2}\right]$	: wind load as result of pressure, suction, friction and over or under pressure
C_{dim}	[-]	: factor for dimensions of the structure
C_{index}	[-]	: wind type factor
C_{eq}	[-]	: pressure dissipation factor
$\emptyset_1$	[-]	: magnification factor for the dynamic wind component
p_w	$\left[\frac{kN}{m^2}\right]$	: the peak velocity pressure

In the case of most hydraulic structures (h < 50 m and h/b < 5), the wind load equation can be simplified to (Voorendt, c. 2022):

$$p_{rep} = C_{dim} * C_{index} * p_w \left[\frac{kN}{m^2}\right]$$

Equation 2: Simplified wind load

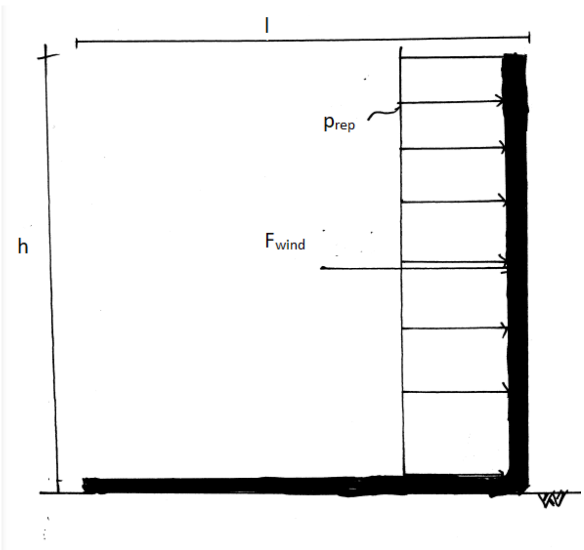


Figure 1: sketch wind load (own work)

As our barrier is less than 50 meters high and the width of the barrier is larger than its height, we can use this simplified equation.

The following parameter values are inserted in the equation for wind pressure after determining them in Appendix A1 :

P_w = 0,54 $\left[\frac{kN}{m^2}\right]$
C_{dim} = 0,96 [-]
C_{index} = 0,8 [-]

$$p_{rep} = 0{,}415 \left[\frac{kN}{m^2}\right]$$

If we multiply this value by the height of the barrier we can use it as a force per unit of width in our calculations. The height of the barrier is 0,9 meters. Therefore:

$$0{,}415 * 0{,}9 = 0{,}5063 \left[\frac{kN}{m}\right]$$

3.2 Hydrostatic loads

An assumption is made that the maximal water level is equal to 0,9 [m]. This is the same as the height of the H-wall

Pascals law is used to calculate the hydrostatic water pressure at any depth.

$$p = \rho_w * g * h$$

Equation 3: Pascals law

With:

ρ_w	$\left[\frac{\mathrm{kg}}{\mathrm{m}^3}\right]$	: density of water	= 1000 for fresh water
g	$\left[\frac{\mathrm{m}}{\mathrm{s}^2}\right]$	: gravitational constant = 9,81	
h	[m]	: water level	

This gives:

p = 1,000 * 9,81 * 0,9 = 8,83 $\left[\frac{\mathrm{kN}}{\mathrm{m}^2}\right]$

If we multiply the maximal pressure by the height of the barrier and divide by two to account for the pressure linearly increasing from zero at the water surface to the maximal pressure at the ground surface we get a force per unit of width:

0,5 * 8,83 * 0,9 = 3,9731 $\left[\frac{\mathrm{kN}}{\mathrm{m}}\right]$ = F_{hydro}

The vertical hydrostatic load or the weight of the water is twice as big as the horizontal one.

G_w = 8,83 * 0,9 = 7,9461 $\left[\frac{\mathrm{kN}}{\mathrm{m}}\right]$
The pressure is acting perpendicular to the barrier. A visualisation can be seen in Figure 2.

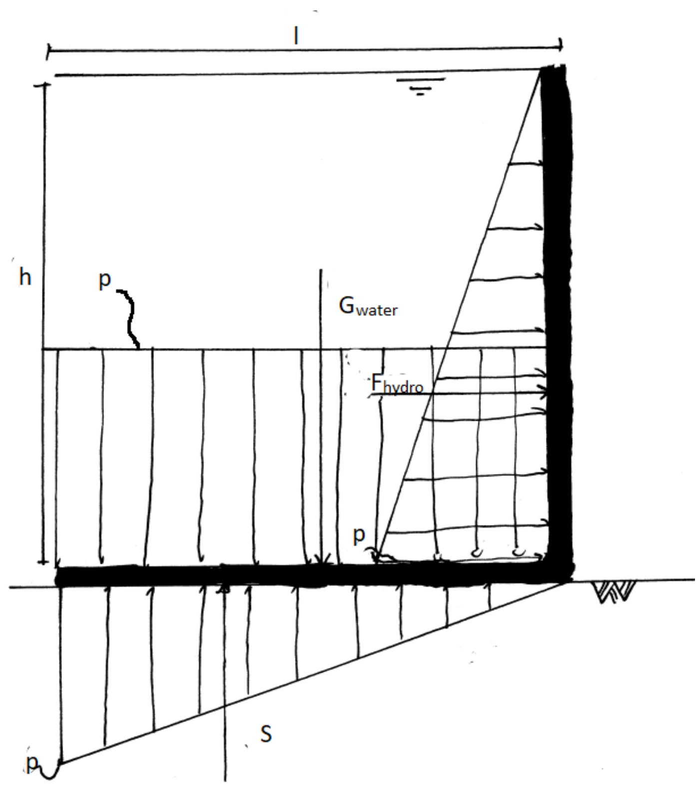


Figure 2: sketch water induced loads (own work)

3.3 Under pressure

If the barrier is located on a permeable soil there will be an upward pressure underneath. This pressure is called under pressure. On the end closest to the water this pressure has the same value as the maximal hydrostatic pressure and it decreases linearly until it reaches zero at the dry end of the barrier. The under pressure is also shown in Figure 2

The formula used to calculate the under pressure is the one also used to calculate the hydrostatic load:

$$p = \rho_w * g * h$$

When multiplied by the length and divide by two to account for the pressure linearly increasing from zero to p a force per unit of width is obtained:

$$S = p * l$$

3.4 Wave load

Depending on the location where the H-wall will be deployed waves could have an influence on the stability of the barrier. Waves will not have much influence when deployed next to smaller water bodies with a shorter fetch. When located next to water bodies with a long fetch however their influence has to be taken into account.

In order to determine the wave loads the wave type must be known because the calculation method depends on the wave type. Wave type is referring to whether the waves are breaking. The assumption is made that the waves are not breaking. The Sainflou method is used when calculating wave loads of non-breaking waves. A situation sketch is shown in Figure 3.

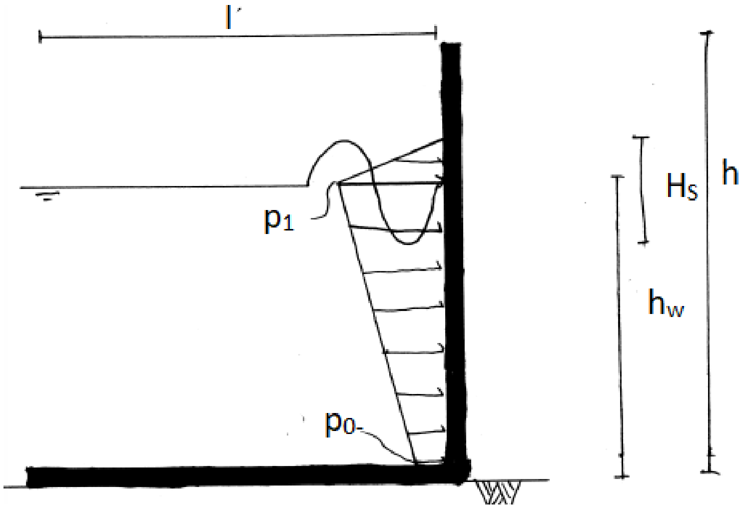


Figure 3: sketch wave load (own work)

Using the Sainflou method the maximal wave pressure can be calculated for two points, p_0 (near bed level) and p_1 (mean water level) (Voorendt, c. 2022).

$$p_0 = \frac{\rho * g * H_{in}}{cosh(k * d')}$$

Equation 4: wave pressure near bed level (Voorendt, c. 2022)

$$p_1 = \rho * g * (H_{in} + h_0)$$

Equation 5: wave pressure at mean water level (Voorendt, c. 2022)

All the parameters and the way to obtain them are clarified in Appendix A2.

Wave loads are not included in the other calculations since it is difficult to test with waves at flood proof holland.

3.5 Self-weight

The self-weight includes the weight of the grid structures, the tension cables, the foil and the sandbags used to keep the foil in place. The panels have a mass of 16,5 [kg] a piece and two pieces are used to construct one segment. The panel size is 1220 [mm] x 900 [mm] x 60 [mm] (w * h * t) according to Tonnie Hospers (personal communication, march 3 2023). The height of the standing panel is the same as the length (l) of the floor panel. Assuming the weight of the other components is negligible the weight of the barrier per unit of width is 0,2654 $\left[\frac{\mathrm{kN}}{m}\right]$ (=2 * 16,5 * 9,81 * 0,001 * 1,000 / 1,22).

During the first few minutes of testing sandbags will be used to keep the foil in place. This is needed because otherwise the water could start flowing under the foil and thus under the entire barrier. These sandbags have a mass of 18 [kg] on average. Their weight per unit of width is 0,1447 [kN/m]. An assumption is made that three of them are needed for every section (1,22 [m]) of the barrier. They are placed at a distance of 3 [m] from the vertical panel.

3.6 Loads overview

With all loads separately calculated we will combine them now to get an overall view of loads on the barrier. Since the wind load and hydrostatic load for the maximum water level can't act at the same time the two extreme situations, maximal water level and no water are calculated for our barrier In order for the structure to not fail under these loads the forces resisting the external forces should be larger than or equal to the external forces. This will be covered in the next chapter. In Figure 4 all loads are shown together.

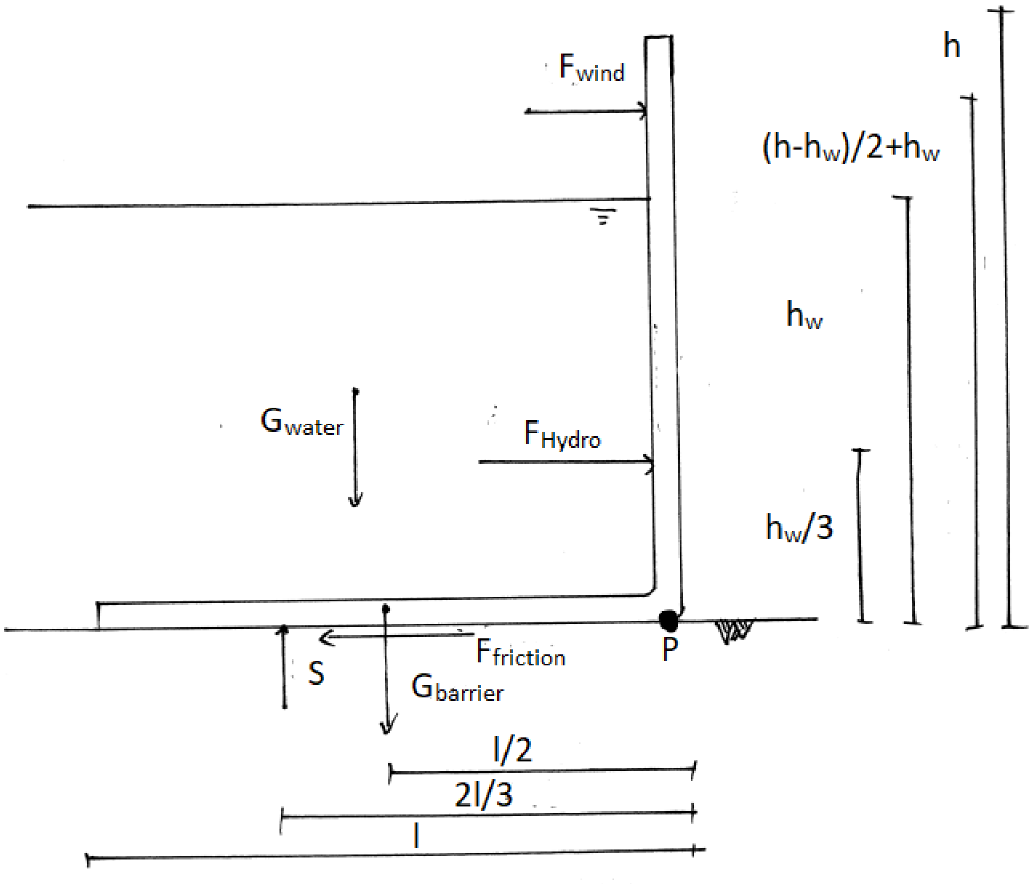


Figure 4: sketch loads overview (own work)

4. Failure mechanisms

Using the loads calculated in the previous chapter failure mechanisms of the H-wall can be calculated. The Failure mechanisms covered in this chapter are strength, stability, seepage, piping and overtopping. Since the report is about a temporary flood barrier scour will not be covered because the flood barrier should not be in use long enough for scour to become a problem.

4.1 Strength

The barrier and it components need to be strong enough to withstand the forces acting on them. If the H-wall fails because a panel breaks or the foil tears all other calculations are of little use until stronger materials are used. The assumption is made that failure because of stability or watertightness is likely to occur before failure due to strength. Unless testing shows this assumption is wrong no calculations will be made.

4.2 Stability

The (in)stability of the barrier will be computed by calculating the resistance required to maintain an equilibrium of the horizontal forces and of the force moments. Two load situations will be covered. Situation 1: These are the loads under maximal water level and Situation 2: the loads without any water. The hydrostatic load is larger but under those conditions the friction will also be higher due to the weight of the water making the barrier heavier. This is not the case for the conditions of the wind load for which the only weight of the barrier will come from the relatively light weight components.

Horizontal stability:

The following formula is used to calculate horizontal stability:

$$\sum F_{hor} < f * \sum F_{vert}$$

Equation 6 horizontal stability requirement (Voorendt, c. 2022)

Where:

ΣF_{hor}	[kN]	: horizontal force
ΣF_{vert}	[kN]	: vertical force
f	[-]	: friction coefficient

For an impermeable ground surface like asphalt or concrete the vertical forces are the weight of the two panels and the weight of the water. For permeable soils the under pressure has to be considered resulting in a smaller total vertical force with a smaller friction force as a consequence.

To ensure horizontal stability the friction force acting from the floor on the barrier has to be calculated. This will be done for grass, concrete and asphalt surfaces. The friction coefficient of the panel on concrete, grass and asphalt are 0,65; 1,52; 0,78 respectively. These had to be determined by doing some tests since there were no sources discussing the friction coefficient of temporary flood barriers. This test will be discussed in chapter 5. The measurements and calculations can be found in Appendix B. The Stability calculations can be found in Appendix A3 – A6.

Situation 1 and situation 2: These situations are not significant when using the barrier as intended. This means no overtopping and/or a water level on both sides of the flood barrier. The barrier will not fail under these conditions.

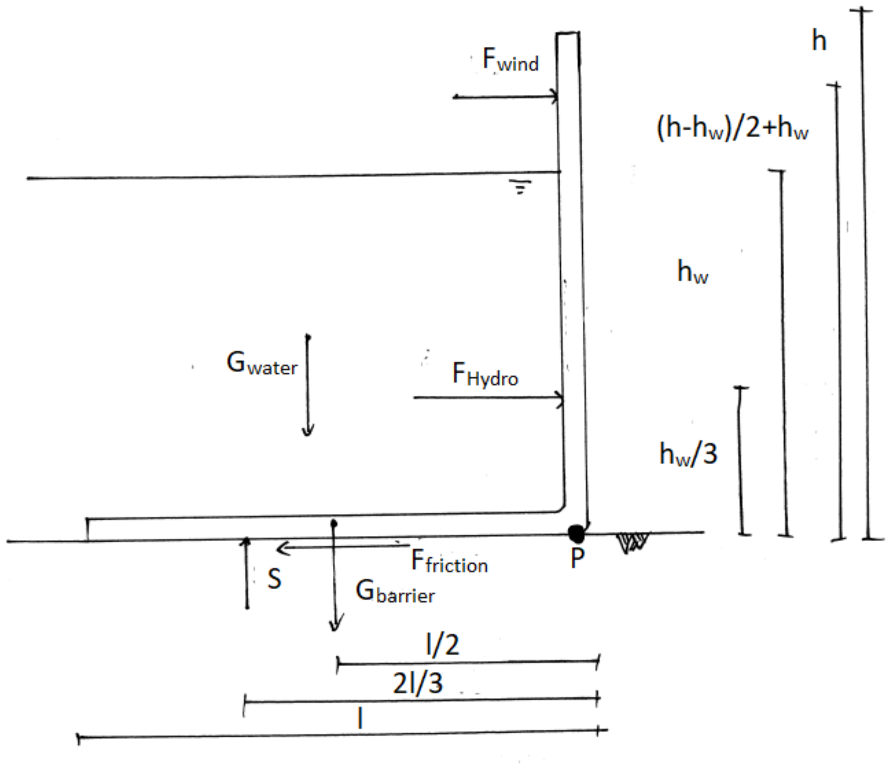
Rotational stability:

The following condition has to be met to determine rotational stability:

$$\sum M = 0$$

Equation 7: Rotational stability requirement

The moments of all forces are calculated around the point (P) where the two panels that make up a section of the barrier connect.



In order to calculate the total moment around P all forces and there perpendicular distance to P need to be known. The weight of the upright panel doesn't have a moment around P since it is in line with P. The same is true for the friction force acting on the floor panel. A list of all forces and there perpendicular distance to P can be found in Appendix A4 and a visual is given in Figure 4.

Situation 1 and situation 2: These situations are not significant when using the barrier as intended. This means no overtopping and/or a water level on both sides of the flood barrier. The barrier will not fail under these conditions.

4.3 Seepage & Piping

A flow of groundwater will likely occur underneath a flood barrier when the barrier creates a water level difference. This water flow is known as seepage and can erode the soil and compromise the stability of the structure. This last phenomenon is called piping. The length of the seepage path has to be approximated and its requirements have to be fulfilled based on the maximal hydraulic gradient value to find out if piping will influence the stability of the structure.

This failure mechanism depends heavily on the soil the structure is placed on. Because of this two soils will be considered. Those soils are normal clay and coarse sand. It is assumed no piping occurs when the flood barrier is placed on a concrete or asphalt surface.

A first impression is obtained using tools based on experiments of Bligh and Lane. These computations alone are not enough to base a decision on, but they can give a good insight of the piping mechanics. A vertical and horizontal part of the seepage path based on the type of foundation and its dimensions are considered in these methods. Only Bligh's method is used since it mainly considered horizontal seepage paths (Voorendt, c. 2022).

$$L = \sum L_{vert} + \sum L_{hor}$$

Equation 8: Bligh's method (Voorendt, c. 2022)

There is no vertical distance since the barrier is placed on top of the soil. This results in the following seepage length formula:

$$L = \sum L_{hor}$$

Equation 9: Simplified Bligh's method (Voorendt, c. 2022)

The estimated seepage length is based on the length of the foil and the length of the panels. The foil is 4 meters long and the panels are 0,9 meters long. The foil is folded over the top of the vertical panel the estimated length lost because of this is 0,1 [m]. another 0,9 [m] should be subtracted to account for the part of the foil covering the vertical panel. The resulting horizontal length is 3 [m]. This is also the seepage length.

The required distance is obtained using Equation 10 . For the average soil a soil based on coarse sand and normal clay is assumed.

$$L \geq \gamma * C_B * \Delta H$$

Equation 10: Bligh (Voorendt, c. 2022)

Where:

L	[m]	: total seepage distance
C_B	[-]	: Bligh's constant
γ	[-]	: safety factor = 1,5
ΔH	[m]	: head difference across the structure = 0,9 [m]

The C-values given in table 37.2 of the manual hydraulic structures are: 12 for coarse sand and no value for normal clay so 1 is assumed (Voorendt, c. 2022). These values lead to the following requirements. The total seepage distance is 16,2 [m] for coarse sand and 1,35 [m] for normal clay. This means that there will be seepage if the barrier is placed on coarse sand and piping could occur. When the flood barrier is placed on normal clay it's unlikely piping will occur. Since piping develops over a relatively longer period of time it is assumed it will not be important for the current research.

4.4 Overtopping

The barrier can overflow or be overtopped when the water level higher is than the height of the barrier. Or when waves push water over. The way the flood barrier functions and its stability can change when it is being overtopped. This can lead to failure. This failure can occur because of the water level rising above the design water level of 0,9 [m] causing rotational or horizontal instability. Another way overtopping could lead to failure is reducing the hydraulic head difference between the two sides of the barrier until it potentially reaches zero. This can possibly lead to the foil starting to float since the pressure is equal on both sides. A visualisation is given in Figure 5

The stability of the structure will be calculated again for the overtopping case to determine if and when it can lead to failure. Calculations are made for a water level up to 1,2 [m]. This is exceeds the height of the barrier by a quarter. According to these computations the structure is horizontally and rotationally stable on grass, concrete and asphalt.

Additional calculations are done where the water level on the other side of the H-wall rises up to 0,9 [m]. These have similar results to the calculations without a water level on the other side of the flood barrier.

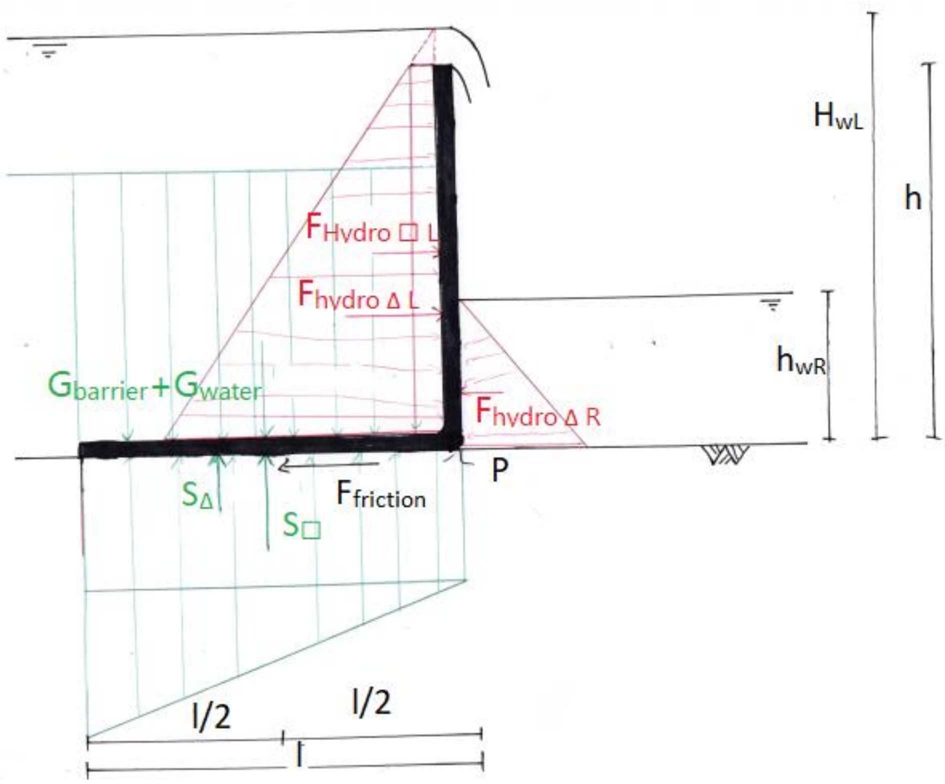


Figure 5: sketch overtopping (own work)

5. Field tests

In this chapter a summary and some remarks are given of the field tests conducted at Flood Proof Holland. Overtopping of the H-wall and friction coefficients where tested and/or measured. These subjects where considered the most interesting to investigate further.

5.1 Overtopping

The overtopping test and the preparation of the test where done on Wednesday the 8th of march. This was cold and windy day that highlighted some shortcomings regarding the ease of installation of the flood barrier. The installation of the plastic was severely hindered and was more time consuming because of the wind. This shows that some thought has to be put in to improve the installation process of the foil. The main problem was keeping the foil over the vertical panel. The barrier would benefit from a better way to keep the foil there in place. Hands and fingers lost a lot of dexterity and strength due to the cold temperatures and wind. This made installing the tension cables a more time consuming and difficult process than it should be. Another problem regarding the tension cables was finding the dark coloured screw holes through the muddy plastic. This could be made considerably easier by giving the screw holes a bright colour that clearly stands out compared to the dark colours of the mud. The test basin also had a groove between the concrete floor and the wooden retaining wall that allowed a small stream to bypass the barrier. The test setup can be seen in Figure 6 and Figure 7.

Figure 6: Back of the test setup (own work)

Figure 7: front of the test setup (own work)

Right after the test started one of the sandbags holding down the foil was moved by the fast flowing water enabling the water to flow underneath the foil and bypassing the barrier. After pausing the test and fixing the problem the test was continued. No further issues occurred and the barrier was successfully overtopped.. This can be seen in Figure 8. The H-wall didn't fail after continuously being overtopped for around 40 minutes. The water level reached +- 0,92 [m]

Figure 8: overtopped H-wall (own work)

The pipe allowing the overflowing water to drain was then closed to determine if rising water on the other side of the barrier would cause it to fail. When the water there was at +- 0,8[m] as can be seen in Figure 9 the supply ran out. It is fair to state the barrier wasn’t influenced much by the rising water. A final subject worth pointing out is that there where small leaks in the foil at the start of the experiment. They disappeared later and it was first noticed when the water started flowing over the barrier.

Figure 9: flooded H-wall (own work)

5.2 Friction coefficients

Due to a lack of sources covering the friction coefficients of temporary flood barriers the friction coefficients had to be determined with a test. The test setup consisted of a segment of the barrier (only the two plates), three sandbags a tension strap, a rope and a dynamometer. The setup is shown in Figure 10. The segment was set up on different ground surfaces. These where concrete, asphalt and grass. The tension strap was strapped around the upright panel as close to the ground as practical to avoid tilting. The rope was added to make attaching the dynamometer easier. The dynamometer is used to determine the force required to start moving the structure. The actual measurement is taken right before it starts moving since the stationary friction coefficient was required. Once the structure starts sliding the required force decreases a little. If that force would be used the dynamic friction coefficient would be determined instead.

Figure 10: Tilting of test setup of friction measurement (Hemmes, 2023)

The measuring was done for four different normal forces. This was accomplished by doing a measurement of just the two plates, one with a sandbag for extra weight and two more with two and three sandbags respectively. This was repeated for every ground surface. It was originally planned to measure both dry and wet ground surfaces but this was not possible due to rain making the surfaces wet. In the end only wet surfaces were measured. For the concrete and the asphalt everything went according to plan. When testing on grass however two problems arose. The measurement without sandbags couldn't be performed correctly since the structure started tilting before it could start sliding. This can be seen in Figure 10. The second problem occurred during the measurement with three added sandbags. The force required was too large and the tension strap failed before the required force was reached. This probably happened because the strap was sliding over the sharp edges of the grid panels. It also caused some back pain to pull with the required force so it was decided to not try again. Because of this these two measurements are not taken into account when calculating the friction coefficient of grass. All measurements can be found in Appendix B.

Once the measurements were taken the values where converted from kilogram to newton. Then they were each divided by the weight of the setup. Each panel has a weight of 161,87 [N] and each sandbag has a weight of 176,58 [N]. This resulted in four friction coefficients for each ground surface. They were plotted and a trendline was fitted. The slope of this trendline is the friction coefficient. The R^2 value is be used to determine the goodness of fit (GOF). The closer the value is to 1 the better the fit. This can however not be used when there are only two measurements since two points always make a perfect line. The result can be found in Figure 11.

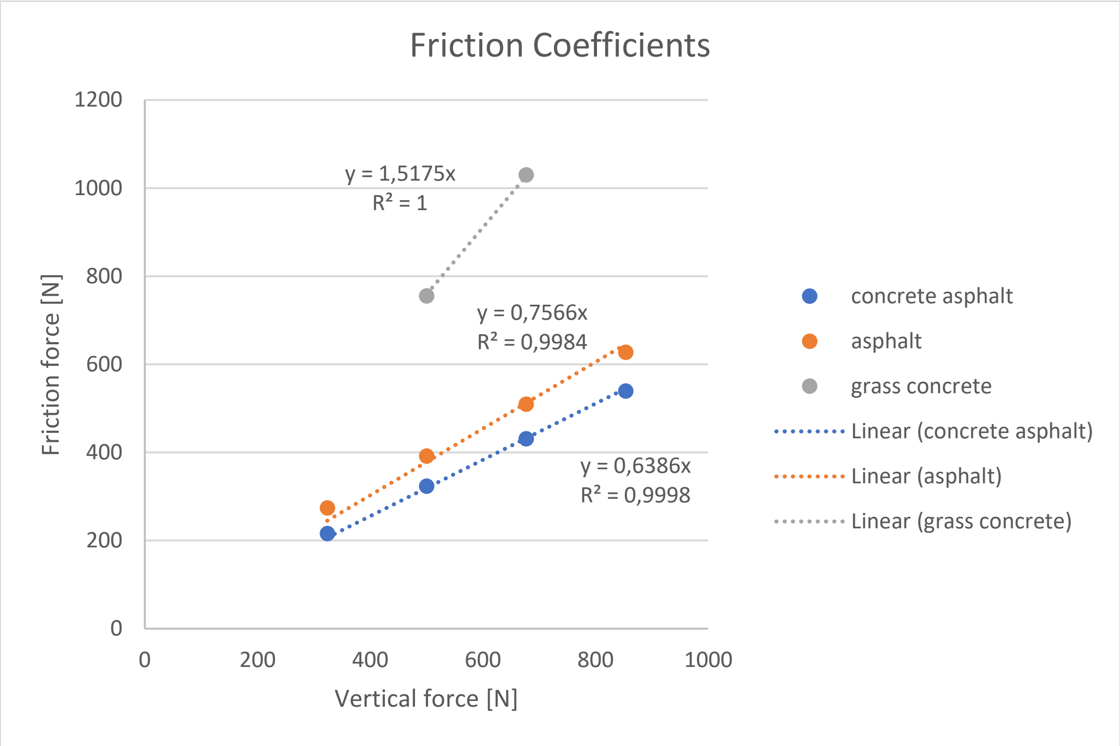


Figure 11: Trendlines friction coefficients (own work)

Another small note to make is that when carrying the grid panels with bare hands holding them by the larger opening that serves as a handle some superficial cuts were made in the hand.

6. discussion and evaluation

Based on the calculations and test at Flood Proof Holland backing them up the conclusion can be made that H-wall temporary flood barrier can retain water levels up to 0,9 [m]. The maximal water level was measured during the test. It reached the water level that was assumed in the calculations. The barrier could not be tested to failure at the testing location.

During the stability calculations the foil was assumed only long enough to cover the grid panels and the sandbags used to hold the foil down initially are not included. Because of this the weight of the water over a distance of +- 2 [m] is not included in the calculations. This weight would have a stabilising effect on the structure both horizontal and rotational. These assumptions are disadvantageous for the results of these calculations and the real world limits of the barrier could be better than what these calculations claim.

Wave loads were left out of the computation of the failure mechanisms since they could not be consistently tested for at Flood Proof Holland. If the H-wall temporary flood barrier is deployed in the field there is a high probability it has to deal with waves of some sort. The water retaining hight of the barrier is likely to be lower when waves are considered. Waves also generate wave pressure on the flood barrier resulting in more horizontal destabilising forces without any extra stabilizing forces. Because of this the results of this research might be too optimistic and further research is required.

Altena was interested in the performance over a longer period of time. This could not be tested because other students also required the testing basin that could reach the water level needed. Said basin also had a groove between the concrete floor and the wooden retaining wall that allowed a small stream to bypass the barrier.

The H-wall in its current form has excellent performance for its primary function of retaining water. During testing it was however discovered that the setup process and convenience can benefit from improvements. When it is windy installing the foil is nearly impossible for one person and needlessly difficult for two people. This might not be a problem for (local) governments adopting the H-wall but can cause regular people and small businesses to opt for an alternate solution that is easier to set up.

7. Recommendations & Suggestions

In this chapter recommendations for further research are given first and improvements to the H-wall are suggested after. These suggestions are made to solve problems discovered during testing of the H-wall. These suggestions are meant to improve the barrier as a whole or make its deployment more convenient.

Like stated in the previous chapter waves will likely lower the water retaining hight of the barrier. Their wave pressure on the flood barrier will also result in more horizontal destabilising forces without any extra stabilizing forces. It would be interesting to know the performance of the H-wall when waves are considered.

It would be interesting to see what the practical size limit of this concept is for when more water retaining height is required.

Another subject that will require more research is the performance of the barrier over long periods of time. It is highly likely that the length of the foil needs to be increased to prevent excess (ground)waterflow underneath the barrier for some soil types (Voorendt, c. 2022).

By using beams on the dry side and using a different floor-panel twice as long as the original floor-panel instead the tension cables could be replaced. Place the upright panel in the middle of said panel and use beams that click in place instead of tension cables. This way the beams are supported and will not sink in the ground and the foil will not be perforated. When doing this the foil will need to be kept in place differently. This can possibly be done by manufacturing the foil with premade holes designed not to tear using a metal ring for example. The foil will attach to hooks on the upright panel using these premade holes.

The importance of sustainability is growing in all areas of society so hydraulic engineering is no exception. The H-wall design would benefit from a more sustainable way to make the barrier water tight. One easy way this can be accomplished is by making the foil reusable in some way because the foil is the least sustainable part of the H-wall. A good first step is making sure the installation process of the foil doesn't easily make cuts in the foil and the foil is not perforated. This can be achieved by rounding the corners and sharp outer edges of the grid panels and an alternative for the tension cables.

A problem regarding the tension cables was finding the dark coloured screw holes through the muddy plastic. This could be made considerably more easy by giving the screw holes a bright colour that clearly stands out compared to the dark colours of the mud.

8. Conclusion

The H-wall is an innovative concept regarding flood protection. The design is able to retain water up to water levels of 0,9 meters. During the research of the H-wall the following research question was answered:

“What are the flood protection capabilities and limits of the H-wall and how can it be improved?”

The computation of the failure mechanisms and more specifically the stability calculations for the H-walls intended use as well as for overtopping proved the barrier would be able to fulfil its main purpose without any issues.

The flood barrier was tested to its water retaining limit at Flood Proof Holland. During testing a water level of +- 0,92 [m] was reached without any sliding or tilting of the barrier. After continuously being overtopped four circa 40 minutes the H-wall also remained in place. The experiment confirmed the predictions done by the calculations. The experiment also made clear improvements could be made in terms of ease of use and convenience.

Appendix A: Calculations, Formulae and Parameters

Appendix A1:Wind Loads

To determine the magnitude of the load we first must calculate the wind load per m^2. This goes according to the following formula (Voorendt, c. 2022):

$$p_{rep} = C_{dim} * C_{index} * C_{eq} * \emptyset_1 * p_w \left[\frac{kN}{m^2}\right]$$

Where:

P_{rep}	$\left[\frac{kN}{m^2}\right]$	: wind load as result of pressure, suction, friction and over or under pressure
C_{dim}	[-]	: factor for dimensions of the structure
C_{index}	[-]	: wind type factor
C_{eq}	[-]	: pressure dissipation factor
$\emptyset_1$	[-]	: magnification factor for the dynamic wind component
p_w	$\left[\frac{kN}{m^2}\right]$	: the peak velocity pressure

In the case of most hydraulic structures (h < 50 m and h/b < 5), the wind load equation can be simplified to (Voorendt, c. 2022):

$$p_{rep} = C_{dim} * C_{index} * p_w \left[\frac{kN}{m^2}\right]$$

As our barrier is less than 50 meters high and the width of the barrier is larger than its height, we can use this simplified equation.

We will now determine the three factors in the equation step by step, starting with the peak velocity pressure, p_w .

$$p_w = \left(1 + 7 * I(z)\right) * \frac{1}{2} * \rho * v^2{}_w(z) \left[\frac{kN}{m^2}\right]$$

Equation 11: (Voorendt, c. 2022)

$$I(z) = \frac{k}{ln\left(\frac{z-d}{z_0}\right)}$$

Where:

$I(z)$	[-]	: turbulence-intensity at height z above the surrounding plane
ρ	$\left[\frac{kg}{m^3}\right]$	: mass density of air = 1,25
z_0	[m]	: roughness length
d	[m]	: displacement height
u	$\left[\frac{m}{s}\right]$	: friction velocity
k	[-]	: factor
v_w	$\left[\frac{m}{s}\right]$	: wind velocity at height z $\left(2{,}5 * \mathrm{u} * \ ln\left(\frac{z-d}{z_0}\right)\right)$

However, the 'Technische Grondslagen voor Bouwconstructies' (TGB) method also provides a table in which values are given for certain areas (Voorendt, c. 2022).

We assume our barrier is in Delft, as testing will be conducted there. Therefore, we use the factors of open area II.

If we know z, we can choose the right value from the table. Z is in our cases the height of the barrier minus the height of the water level and just the height of the barrier. This gives the following z:

0,9 [m]

Reading the table then gives us the following value for the peak velocity pressure: 0,54 $\left[\frac{kN}{m^2}\right]$

Secondly we need to determine C_{dim} . This factor takes the dimensions of the structure into account and is calculated with the following formula (Voorendt, c. 2022):

$$C_{dim} = \frac{1 + 7 * I(h)\sqrt{B}}{1 + 7 * I(h)} \ (\leq 1)$$

Equation 12

Where:

I(h): $\frac{1}{\ln\left(\frac{h}{0,2)}\right)}$

B: $\frac{1}{0,4+0,021h^{\frac{2}{3}}+0,029b^{\frac{2}{3}}}$

This value can also be found in table 8.3 of the Manual Hydraulic Structures. In order to use the table the width of the barrier is required. This depends on the location and for now is assumed to be 12,2 meters. C_{dim}= 0,96

The last factors, C_{index}, determination can be done using a figure to determine the value. As the barrier is a closed structure we can use the value 0,8 which can be obtained from figure 8.4 of the manual hydraulic structures.

Concluding, we can insert the following values in the equation for wind pressure:

P_w : 0,54 $\left[\frac{kN}{m^2}\right]$
C_{dim} : 0,96 [-]
C_{index} : 0,8 [-]

$$p_{rep} = C_{dim} * C_{index} * p_w = 0{,}96 * 0{,}8 * 0{,}54 = 0{,}415 \left[\frac{kN}{m^2}\right]$$

If we multiply this value by the height of the barrier we can use it as a q-load in our calculations. The height of the barrier is 0,9 meters. Therefore:

$$0{,}415 * 0{,}9 = 0{,}5063 \left[\frac{kN}{m}\right]$$

Appendix A2: Wave Loads

Using the Sainflou method the maximal wave pressure can be calculated for two points, p_0 (near ground level) and p_1 (mean water level).

$$p_0 = \frac{\rho * g * H_{in}}{cosh(k * d')}$$

$$p_1 = \rho * g * (H_{in} + h_0)$$

H_{in} is needed in both calculations. This can be calculated using Equation 13.

$$h_0 = \frac{1}{2} * k * H_{in}^2 * coth\ (k * d)$$

Equation 13

Using:

H_{in} [m] : Incoming wave height = ½ H_s

H_s [m] : significant wave height

k $[m^{-1}] = \frac{2\pi}{L}$: wave number of the incoming wave.

L [m]: wave length

d [m]: water depth 2 or 3 wave lengths away from the barrier
d' [m]: water depth near structure
h_0 [m]: increase of the mean water level in front of the structure

$$L = L * tanh\left(\frac{2 \times \pi \times d}{L}\right)$$

Equation 14

Needs to be solved iteratively, assuming deep water starting with:

$$L = L_0 = \frac{g * T^2}{2\pi}$$

Equation 15

Where:

L_0 [m]: wavelength in deep water
T $[s^{-1}]$: wave period

Appendix A3: Force table

Using the formulae and methods clarified in the report and this appendix the following table of forces was constructed.

Table 1: forces (own work)

water depth [m]	G_barrier [kN/m]	G_water[kN/m]	F_hydro [kN/m]	F_wind [kN/m]	S [kN/m]
0	0,2654	0,0000	0,0000	0,3735	0,0000
0,1	0,2654	0,8829	0,0491	0,3320	0,4415
0,2	0,2654	1,7658	0,1962	0,2905	0,8829
0,3	0,2654	2,6487	0,4415	0,2490	1,3244
0,4	0,2654	3,5316	0,7848	0,2075	1,7658
0,5	0,2654	4,4145	1,2263	0,1660	2,2073
0,6	0,2654	5,2974	1,7658	0,1245	2,6487
0,7	0,2654	6,1803	2,4035	0,0830	3,0902
0,8	0,2654	7,0632	3,1392	0,0415	3,5316
0,9	0,2654	7,9461	3,9731	0,0000	3,9731

For some calculations the sum of the horizontal and vertical forces are required. These can be found in the following table:

Table 2: sum of forces (own work)

water depth [m]	ΣFH [kN/m]		ΣFV [kN/m]
		With S	Without S
0	0,3735	0,6996	0,6996
0,1	0,3811	1,1410	1,5825
0,2	0,4867	1,5825	2,4654
0,3	0,6905	2,0239	3,3483
0,4	0,9923	2,4654	4,2312
0,5	1,3923	2,9068	5,1141
0,6	1,8903	3,3483	5,9970
0,7	2,4865	3,7897	6,8799
0,8	3,1807	4,2312	7,7628
0,9	3,9731	4,6726	8,6457

Appendix A4: Moments and levers

An overview of all forces and their lever or the equation used to compute it is given in the following table

Table 3: levers (own work)

Force	Lever [m]
G_barrier	0,5*l = 0,45
G_water	0,5*l = 0,45
F_Wind	$h_w + (h-h_w)/2$
F_hydro	$h_w/3$
S	2/3*l = 0,6
G_sandbag	3

Where:

h_w [m]: water depth

h [m]: hight of the barrier

l [m]: length of the barrier

The moments around P can be found in the following table:

Table 4: moments around P (own work)

water depth [m]	ΣM [kNm/m]	
	without S	With S
0	1,1943	1,1943
0,1	1,5920	1,3271
0,2	1,9841	1,4544
0,3	2,3607	1,5661
0,4	2,7120	1,6526
0,5	3,0283	1,7039
0,6	3,2996	1,7104
0,7	3,5163	1,6622
0,8	3,6684	1,5494
0,9	3,7462	1,3623

Appendix A5: friction force

The friction force is calculated using the formula already described in the report.

$$\sum F_{hor} < f * \sum F_{vert}$$

The results can be found in the following table:

Table 5: friction (own work)

water depth [m]	concrete [kN]	asphalt [kN]	grass [kN]
0	0,4477	0,5317	1,0633
0,1	1,0128	1,2027	1,7343
0,2	1,5778	1,8737	2,4053
0,3	2,1429	2,5447	3,0764
0,4	2,7079	3,2157	3,7474
0,5	3,2730	3,8867	4,4184
0,6	3,8381	4,5577	5,0894
0,7	4,4031	5,2287	5,7604
0,8	4,9682	5,8997	6,4314
0,9	5,5332	6,5707	7,1024

Appendix A6: Safety factors

A safety factor is calculated both for the rotational and horizontal stability.

$$S_H = \frac{Fh_{stab}}{Fh_{destab}} = \frac{f * \sum FV}{\sum FH} \geq 1$$

$$S_T = \frac{M_{stab}}{M_{destab}} = \frac{G_{water} + G_{barrier}) * 0{,}5l + G_{sandbags} * 3}{FH * \frac{h}{3} + S * \frac{2 * l}{3}} \geq 1$$

The results can be found in the table below. For concrete and asphalt S is not included.

Table 6: safety factors (own work)

water depth [m]	Sh			St		
0	concrete	asphalt	grass	concrete	asphalt	grass
0,1	1,1987	1,4235	2,8470	8,1056	8,1056	8,1056
0,2	2,6579	3,1562	4,5515	10,4969	10,4969	4,0685
0,3	3,2419	3,8498	4,9422	12,4784	12,4784	3,0700
0,4	3,1036	3,6855	4,4556	13,1972	13,1972	2,5849
0,5	2,7290	3,2406	3,7764	12,3231	12,3231	2,2722
0,6	2,3509	2,7917	3,1735	10,4464	10,4464	2,0359
0,7	2,0304	2,4111	2,6924	8,3894	8,3894	1,8402
0,8	1,7708	2,1029	2,3167	6,6063	6,6063	1,6699
0,9	1,5620	1,8548	2,0220	5,2050	5,2050	1,5180
	1,3927	1,6538	1,7876	4,1430	4,1430	1,3810

The barrier complies with the requirements regarding the safety factors for al water levels.

Appendix B: Friction measurements

Once the measurements were taken the values where converted from kilogram to newton. Then they were each divided by the weight of the setup. Each panel has a weight of 161,87 [N] and each sandbag has a weight of 176,58 [N]. This resulted in four friction coefficients for each ground surface. They were plotted and a trendline was fitted. The slope of this trendline is the friction coefficient. The R^2 value is be used to determine the goodness of fit (GOF). The closer the value is to 1 the better the fit. This can however not be used when there are only two measurements since two points always make a perfect line.

Table 7: Friction measurements (own work)

			grass			concrete			asphalt		
sandbags	weight [kg]	weight [N]	force [kg]	force [N]	f [-]	force [kg]	force [N]	f [-]	force [kg]	force [N]	f [-]
0	33	323,73	40	/	/	22	215,82	0,6667	28	274,68	0,8485
1	51	500,31	77	755,37	1,5098	33	323,73	0,6471	40	392,4	0,7843
2	69	676,89	105	1030,05	1,5217	44	431,64	0,6377	52	510,12	0,7536
3	87	853,47	110	/	/	55	539,55	0,6322	64	627,84	0,7356
				Trendline	**1,52**		Trendline	**0,64**		Trendline	**0,76**

Appendix C: Overtopping calculations

Appendix C1: Force table

Using the formulae and methods clarified in the report and this appendix the following tables of forces was constructed for both the case with and without rising water on the other side.

Table 8: forces overtopping (own work)

water depth [m]	G_water[kN/m]	F_hydro_□ [kN/m]	F_hydro_Δ [kN/m]	S [kN/m]
0,92	8,1227	0,1766	3,9731	4,0613
0,93	8,2110	0,2649	3,9731	4,1055
1	8,8290	0,8829	3,9731	4,4145
1,1	9,7119	1,7658	3,9731	4,8560
1,2	10,5948	2,6487	3,9731	5,2974
1,3	11,4777	3,5316	3,9731	5,7389

Where:

F_hydro_□ [kN/m] = $\rho_w * g * (h_w - h) * h$

h_w [m] : water depth

F_hydro_Δ [kN/m] : is constant

Table 9: forces overtopping + flooding (own work)

water depth L [m]	G_water [kN/m]	water depth R [m]	F_hydro_□ L [kN/m]	F_hydro_Δ R [kN/m]	S_□ [kN/m]	S_Δ [kN/m]
0,91	8,0344	0	0,0883	0	0	4,0172
0,91	8,0344	0,1	0,0883	0,04905	0,8829	3,5757
0,91	8,0344	0,2	0,0883	0,1962	1,7658	3,1343
0,91	8,0344	0,3	0,0883	0,44145	2,6487	2,6928
0,91	8,0344	0,4	0,0883	0,7848	3,5316	2,2514
0,91	8,0344	0,5	0,0883	1,22625	4,4145	1,8099
0,91	8,0344	0,6	0,0883	1,7658	5,2974	1,3685
0,91	8,0344	0,7	0,0883	2,40345	6,1803	0,9270
0,91	8,0344	0,8	0,0883	3,1392	7,0632	0,4856
0,91	8,0344	0,9	0,0883	3,97305	7,9461	0,0441
1,2	10,5948	0	2,6487	0	0	5,2974
1,2	10,5948	0,1	2,6487	0,04905	0,8829	4,8560
1,2	10,5948	0,2	2,6487	0,1962	1,7658	4,4145
1,2	10,5948	0,3	2,6487	0,44145	2,6487	3,9731
1,2	10,5948	0,4	2,6487	0,7848	3,5316	3,5316
1,2	10,5948	0,5	2,6487	1,22625	4,4145	3,0902
1,2	10,5948	0,6	2,6487	1,7658	5,2974	2,6487
1,2	10,5948	0,7	2,6487	2,40345	6,1803	2,2073
1,2	10,5948	0,8	2,6487	3,1392	7,0632	1,7658
1,2	10,5948	0,9	2,6487	3,97305	7,9461	1,3244

Where:

S_□ [kN/m] = $\rho_w * g * h_{w_L} * l$

S_Δ [kN/m] = $0,5 * \rho_w * g * \left(h_{w_L} - h_{w_R}\right) * l$

h_{wL} [m]: water depth left

h_{wR} [m]: water depth right

F_hydro_Δ L [kN/m] : is constant

For some calculations the sum of the horizontal and vertical forces are required. These can be found in the following tables:

Table 10: sum of forces overtopping (own work)

water depth [m]	ΣFH [kN/m]	ΣFV [kN/m]	
		With S	Without S
0,92	4,1496	4,7609	8,8222
0,93	4,2379	4,8051	8,9105
1	4,8560	5,1141	9,5286
1,1	5,7389	5,5555	10,4115
1,2	6,6218	5,9970	11,2944
1,3	7,5047	6,4384	12,1773

Table 11: sum of forces overtopping + flooding (own work)

water depth L [m]	water depth R [m]	ΣFH [kN/m]	ΣFV [kN/m]	
			With S	Without S
0,91	0	4,0613	4,7168	8,7340
0,91	0,1	4,0123	4,2753	8,7340
0,91	0,2	3,8651	3,8339	8,7340
0,91	0,3	3,6199	3,3924	8,7340
0,91	0,4	3,2765	2,9510	8,7340
0,91	0,5	2,8351	2,5095	8,7340
0,91	0,6	2,2955	2,0681	8,7340
0,91	0,7	1,6579	1,6266	8,7340
0,91	0,8	0,9221	1,1852	8,7340
0,91	0,9	0,0883	0,7437	8,7340
1,2	0	6,6218	5,9970	11,2944
1,2	0,1	6,5727	5,5555	11,2944
1,2	0,2	6,4256	5,1141	11,2944
1,2	0,3	6,1803	4,6726	11,2944
1,2	0,4	5,8370	4,2312	11,2944
1,2	0,5	5,3955	3,7897	11,2944
1,2	0,6	4,8560	3,3483	11,2944
1,2	0,7	4,2183	2,9068	11,2944
1,2	0,8	3,4826	2,4654	11,2944
1,2	0,9	2,6487	2,0239	11,2944

Appendix C2: Moments and levers

An overview of all forces and their lever or the equation used to compute it is given in the following table:

Table 12: levers (own work)

Force	Lever [m]
G_barrier	0,5*l = 0,45
G_water	0,5*l = 0,45
F_hydro_□	0,5*h = 0,45
F_hydro _Δ	h/3
S	2/3*l = 0,6
G_sandbag	3
S_Δ	2/3*l
S_□	l/2 = 0,45

Where:

h [m]: hight of the barrier

l [m]: length of the barrier

The moments around P can be found in the following tables:

Table 13: moment overtopping (own work)

water depth [m]	ΣM [kNm/m]	
	without S	With S
0,92	3,7462	1,3094
0,93	3,7462	1,2829
1	3,7462	1,0975
1,1	3,7462	0,8326
1,2	3,7462	0,5677
1,3	3,7462	0,3029

Table 14: moment overtopping + flooding (own work)

water depth L [m]	water depth R [m]	ΣM [kNm/m]	
		without S	With S
0,91	0	3,7462	1,335857
0,91	0,1	3,7478	1,205057
0,91	0,2	3,7593	1,084067
0,91	0,3	3,7903	0,982697
0,91	0,4	3,8508	0,910757
0,91	0,5	3,9505	0,878057
0,91	0,6	4,0993	0,894407
0,91	0,7	4,3070	0,969617
0,91	0,8	4,5833	1,113497
0,91	0,9	4,9381	1,335857

1,2	0	3,7462	0,567734
1,2	0,1	3,7478	0,436934
1,2	0,2	3,7593	0,315944
1,2	0,3	3,7903	0,214574
1,2	0,4	3,8508	0,142634
1,2	0,5	3,9505	0,109934
1,2	0,6	4,0993	0,126284
1,2	0,7	4,3070	0,201494
1,2	0,8	4,5833	0,345374
1,2	0,9	4,9381	0,567734

Appendix C3: friction force

The friction force is calculated using the formula already described in the report.

$$\sum F_{hor} < f * \sum F_{vert}$$

The results can be found in the following tables:

Table 15: friction overtopping (own work)

water depth [m]	F_friction [kN/m]		
	concrete	asphalt	grass
0,92	5,6462	6,7049	7,2366
0,93	5,7027	6,7720	7,3037
1	6,0983	7,2417	7,7734
1,1	6,6633	7,9127	8,4444
1,2	7,2284	8,5837	9,1154
1,3	7,7934	9,2547	9,7864

Table 16: friction overtopping + flooding (own work)

water depth L [m]	water depth R [m]	F_friction [kN/m]		
		concrete	asphalt	grass
0,91	0	5,5897	6,6378	7,1695
0,91	0,1	5,5897	6,6378	6,4985
0,91	0,2	5,5897	6,6378	5,8275
0,91	0,3	5,5897	6,6378	5,1565
0,91	0,4	5,5897	6,6378	4,4855
0,91	0,5	5,5897	6,6378	3,8145
0,91	0,6	5,5897	6,6378	3,1435
0,91	0,7	5,5897	6,6378	2,4724
0,91	0,8	5,5897	6,6378	1,8014
0,91	0,9	5,5897	6,6378	1,1304
1,2	0	7,2284	8,5837	9,1154
1,2	0,1	7,2284	8,5837	8,4444
1,2	0,2	7,2284	8,5837	7,7734
1,2	0,3	7,2284	8,5837	7,1024
1,2	0,4	7,2284	8,5837	6,4314
1,2	0,5	7,2284	8,5837	5,7604
1,2	0,6	7,2284	8,5837	5,0894
1,2	0,7	7,2284	8,5837	4,4184
1,2	0,8	7,2284	8,5837	3,7474
1,2	0,9	7,2284	8,5837	3,0764

Appendix C4: Safety factors

A safety factor is calculated both for the rotational and horizontal stability.

$$S_H = \frac{Fh_{stab}}{Fh_{destab}} = \frac{f * \sum FV}{\sum FH} \geq 1$$

$$S_T = \frac{M_{stab}}{M_{destab}} = \frac{(G_{water} + G_{barrier}) * 0{,}5l + G_{sandbags} * 3}{F\_hydro\ \_\Delta * \frac{h}{3} + F\_hydro\_\square * \frac{h}{2} + S\_\Delta * \frac{2 * l}{3} + S\_\square * \frac{l}{2}} \geq 1$$

The results can be found in the tables below. For concrete and asphalt S is not included.

Table 17: safety factors overtopping (own work)

water depth [m]	Sh [-]			St [-]		
	concrete	asphalt	grass	concrete	asphalt	grass
0,92	1,3607	1,6158	1,7439	3,9466	3,9466	1,3531
0,93	1,3456	1,5980	1,7234	3,8573	3,8573	1,3399
1	1,2558	1,4913	1,6008	3,3572	3,3572	1,2590
1,1	1,1611	1,3788	1,4714	2,8858	2,8858	1,1699
1,2	1,0916	1,2963	1,3766	2,5715	2,5715	1,1021
1,3	1,0385	1,2332	1,3040	2,3470	2,3470	1,0487

Table 18: safety factors overtopping + flooding (own work)

water depth L [m]	water depth R [m]	Sh [-]			St [-]		
		concrete	asphalt	grass	concrete	asphalt	grass
0,91	0	1,3763	1,6344	1,7653	4,0416	4,0416	1,3668
0,91	0,1	1,3932	1,6544	1,6196	4,0429	4,0429	1,3193
0,91	0,2	1,4462	1,7174	1,5077	4,0522	4,0522	1,2775
0,91	0,3	1,5442	1,8337	1,4245	4,0774	4,0774	1,2433
0,91	0,4	1,7060	2,0259	1,3690	4,1266	4,1266	1,2183
0,91	0,5	1,9716	2,3413	1,3454	4,2075	4,2075	1,2040
0,91	0,6	2,4350	2,8916	1,3694	4,3283	4,3283	1,2016
0,91	0,7	3,3716	4,0038	1,4913	4,4969	4,4969	1,2122
0,91	0,8	6,0617	7,1983	1,9535	4,7213	4,7213	1,2368
0,91	0,9	63,3110	75,1819	12,8037	5,0093	5,0093	1,2764

1,2	0	1,0916	1,2963	1,3766	4,0416	4,0416	1,1287
1,2	0,1	1,0998	1,3060	1,2848	4,0429	4,0429	1,0962
1,2	0,2	1,1249	1,3359	1,2098	4,0522	4,0522	1,0676
1,2	0,3	1,1696	1,3889	1,1492	4,0774	4,0774	1,0446
1,2	0,4	1,2384	1,4706	1,1018	4,1266	4,1266	1,0289
1,2	0,5	1,3397	1,5909	1,0676	4,2075	4,2075	1,0217
1,2	0,6	1,4886	1,7677	1,0481	4,3283	4,3283	1,0243
1,2	0,7	1,7136	2,0349	1,0474	4,4969	4,4969	1,0378
1,2	0,8	2,0756	2,4648	1,0760	4,7213	4,7213	1,0631
1,2	0,9	2,7290	3,2407	1,1615	5,0093	5,0093	1,1013

The H-wall temporary flood barrier complies with the requirements for al water levels.

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