Water Controlling Water

Emergency flood protection

M.J.J. Boon
Delft, August 2007

TU Delft

The Source

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Emergency flood protection

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Preface

This report contains my effort to complete my study at the faculty of Civil Engineering and Geosciences at TU Delft. The report is titled: "Water Controlling Water – emergency flood protection." It discusses emergency measures against overtopping and piping at river dikes and the systems that could be used for this purpose.

I would like to thank all the people who helped me during my graduation project, in particular the members of my graduation committee: Prof. drs. ir. J.K. Vrijling, Dr. ir. S van Baars, Ir. W.L.A. ter Horst, Drs. M. van Oranje, Prof. ir. H.J. Overbeek and Ir. H.J. Verhagen for their supervision and support. And last but not least I would like to thank Gemma for her continuous support.

Delft, August 2007

M.J.J. Boon

Summary

Introduction
This report discusses the feasibility and effectiveness of emergency measures against river floods. The problem definition for this project is:

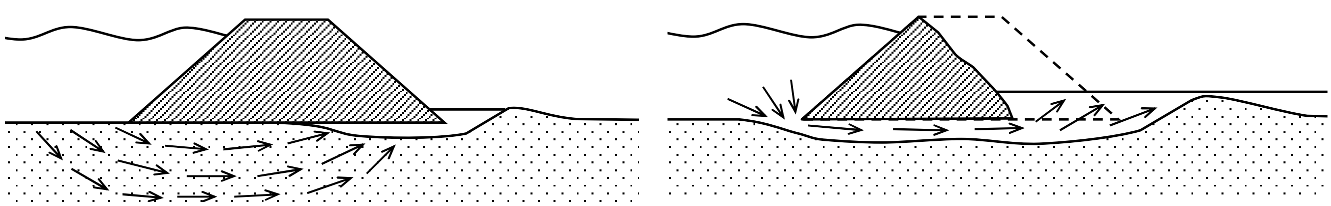
> *The strength of river dikes during high water is insufficient.*

Objective
The Objective of this project is:

To develop a method or system to temporarily increase the dike strength during flood wave conditions.

Here, the dike strength in its broadest sense is meant. This means that besides the crest height also other properties are considered.

Failure mechanisms
Generally, the dike height is designed with respect to a certain normative water level. The protection level is herewith based on the overtopping and overflowing mechanism. Besides overtopping and overflowing other mechanisms, especially piping, could contribute to dike failure. Piping is a mechanism where sand erodes from underneath the dike due to a groundwater flow that is the result of a water level difference. This erosion creates a “pipe” underneath the dike that will eventually undermine the dike and will cause its collapse.



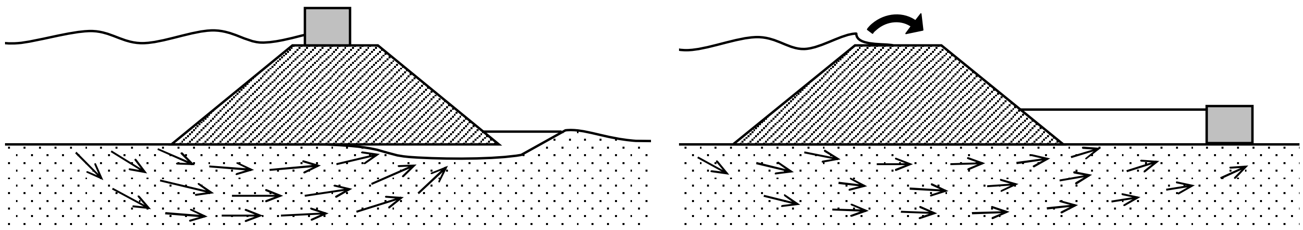
Two stages in the piping process: the formation of a pipe and the failure of the dike

Emergency measures
Piping and overtopping at river dikes both have their specific characteristics. Overtopping can occur at any dike while piping is restricted to dikes with specific geological characteristics. Piping has two other important properties that determine the usefulness of a method or system to prevent it.

On the one hand, there are only few emergency systems against it. On the other, it can occur for water levels below the protection level of a dike, while overtopping only occurs for water levels above this protection level. In other words, a measure against piping will be used on a more frequent basis.

The proposed emergency measures are the traditional temporary crest heightening against overtopping by means of the deployment of a Temporary Flood Defense (TFD) system and the creation of a water berm to prevent piping.

The latter decreases the head difference between the river and land side of the dike, which is the driving force behind the mechanism.

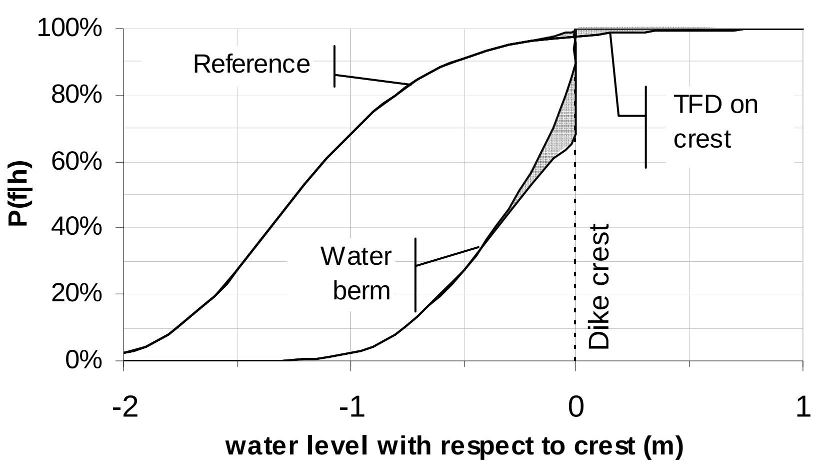


The proposed emergency measure against overtopping (left) and piping (right)

The chart below displays the conditional probability of failure related to the river water level. The chart incorporates the calculation results of three cases, namely a reference situation, a case where a water berm is applied and a temporary crest heightening.

The results show that a water berm can be very effective with respect to decreasing the probability of failure through piping. The curve shifts to the right, meaning that the probability of failure through piping decreases. For example, in the reference situation the probability of failure was 80% for a water level of 1m below the dike crest. When applying a water berm, the probability of failure drops to around 2%

On the other hand measures against overtopping have little effect when piping is also a problem. Though the probability of failure through overtopping decreases, the probability of failure through piping remains the same and results in a failure probability of the same magnitude. The line for the temporary crest heightening case is therefore practically the same as that of the reference situation. When other mechanisms such as piping are of smaller importance, temporary crest heightening can however be effective.



The effect of measures against overtopping and piping with respect to a reference situation

Temporary flood defense (TFD) systems

Both the temporary crest heightening and the water berm measure make use of some sort of water retaining system. Since there are numerous existing water retaining or TFD systems, the next step is to choose the most suitable system for the intended purpose.

The Twin Flex Barrier ("Mobiele Waterdam" in Dutch) water-filled tube type of system is determined to have the best performance on the aspects of costs, deployment, stability and applicability. This system consists of two parallel tubes connected at one point. It has relatively low costs and a very high deployment rate and makes very few demands on the location of application.

Attempts to improve the stability by a modified tube shape showed that the shape has little effect on stability. The standard circular twin tube shape will therefore remain the most obvious. Because piping could in some cases lead to failure of the system, the design was adapted by adding a sealing sheet in front of the tubes to increase the piping length.

Reliability and scale of application

The reliability of a system is assessed by examining the different failure modes. Especially the logistic feasibility is a point of concern. The maximum length that can be applied with reasonable reliability lies in the order of 4km in case of a warning time of 3 days. For the water berm application this is not so much of a problem since this mechanism is in general restricted to locations of often less than one kilometer. As a measure against overtopping, the scale limits the application to e.g. river cities where dikes can not be heightened due to a lack of space.

Recommendations

It must be stressed that the calculation of the logistic reliability incorporates a lot of uncertainties and assumptions. A next step in the research could be an inventory of possible sites of application both against piping and overtopping. For a specific site the logistic feasibility of the emergency measure can then be more accurately determined.

Besides the logistic reliability, there is still some uncertainty on the reliability of the effectiveness of a sealing sheet. It is therefore recommended to further investigate this by testing the system.

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Appendices

1 Introduction

This report discusses the feasibility and effectiveness of emergency measures against river floods. The literature study that preceded this report, evaluated several fields of application of emergency measures. Eventually the scope was directed towards emergency protection against river floods as being the most interesting field for further research. The problem definition for this project is therefore:

> *The strength of river dikes during high water is insufficient.*

The Objective of this report is then:

> *To develop a method or system to temporarily increase the dike strength during flood wave conditions.*

Here, the dike strength in its broadest sense is meant. This means that besides the crest height also other properties are considered. After a short description of the background of protection levels of river dikes world wide, the mechanisms that contribute to dike failure are investigated. This is done by means of a theoretical and historical analysis of dike failure (chapter 3) concluding with a choice for the most important failure mechanism(s). Based on this, the possible measures are identified and their effectiveness is determined through probabilistic calculation (chapter 4).

Next, the possible systems that can be applied to realize these measures are discussed in chapter 5. These systems will be compared on the aspects costs, deployment, stability and applicability in the chapters 6 to 8 respectively. On basis of these different aspects the system with the best properties will be chosen in chapter 9. Based on this system an preliminary design will be made in chapter 10. Chapter 11 discusses the reliability of the system mainly with respect to logistics, also giving an indication of the possible scale of application. Finally chapter 12 draws a conclusion and gives recommendations for further research.

2 Background on flood defenses

2.1 River floods

2.1.1 General

Floods currently account for half of all deaths caused by natural disasters.[41] Many floods are caused by rivers bursting their banks. The consequences of a river flood are often severe, ranging from damage to property, loss of crops and of course loss of life. The consequences of a number of historical floods are described next.

2.1.2 Impact

In the USA the Mississippi Flood of 1927 killed 246 people in seven states while flooding over 70,000km^2 of land. The Great Flood of 1993 along the Mississippi and Missouri rivers had a lower death toll (about 50 people) but flooded an area of 840,000km^2 (twenty times The Netherlands) and triggered the evacuation of 74,000 people.

In 2002 Central Europe was stricken by extensive flooding. In Germany floods along the Elbe River caused economic damage of approximately €15 billion. In the Czech Republic the Moldau River flooded, killing 17 and forcing the evacuation of 200,000 people. The total damage in the Czech Republic amounted to €3.3 billion.

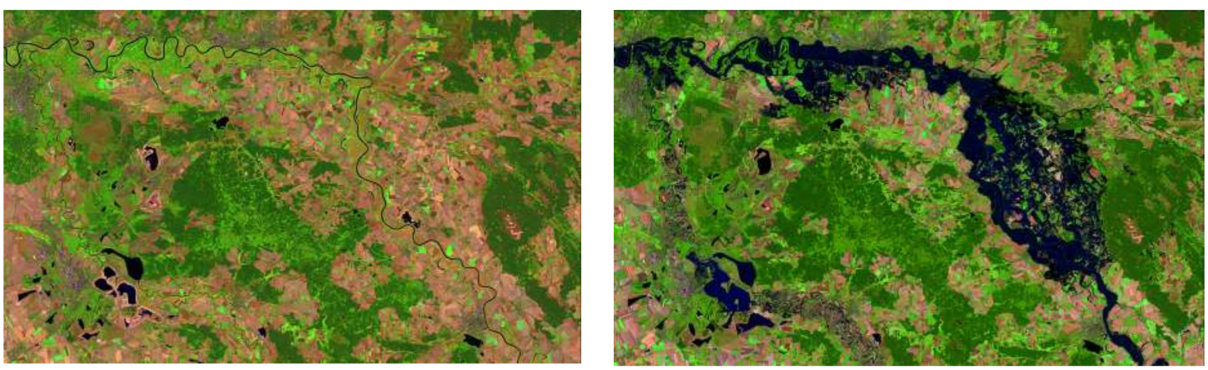


fig. 1 The Elbe River, 2002: before the flood (left) and during the flood (right)

The Far East is also regularly plagued by river floods. One of the largest river floods of the last decade is the Yangtze River flood in China in 1998 left 14 million people homeless and killed 3,004. Presently (summer 2007) the south of China is again suffering extensive flooding. Already half a million people have been evacuated and hundreds have perished.

2.1.3 Warning time and emergency measures

River floods are characterized primarily by the fact that they have a relatively long warning time compared to a storm surge at sea. In The Netherlands the warning time for high water ranges between 2 to 3 days for the rivers and only 6 to 12 hours for the sea.[16] Therefore, the application of emergency measures is more suitable for river floods than for storm surges.

The need for emergency measures is primarily dependent on the protection level of the existing flood defenses and the value of the protected area. For dikes with a low protection level this need will be higher than for those with a relatively high protection level.

2.2 Protection level

All over the world river dikes are designed to be able to control a certain normative water level. This water level has a probability of occurrence that is a measure for the protection level of the area behind the dike. This protection level differs per river and even more important: per country. For the sake of completeness it must be added that the crest height of a dike often incorporates an extra margin called the freeboard. This additional height above the normative high water level is primarily intended to prevent wave overtopping. In the next paragraphs the situation in several countries is described.

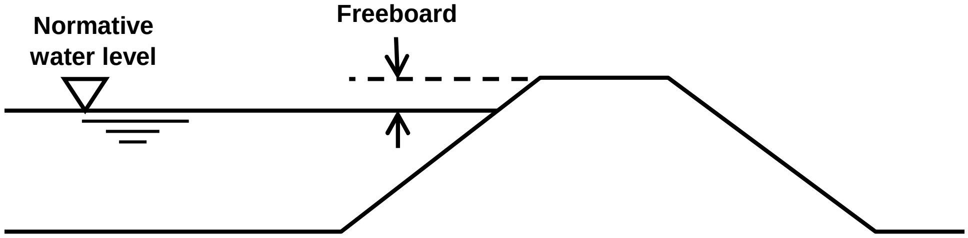


fig. 2 Protection level and freeboard

2.2.1 The Netherlands

The Netherlands is located in the delta of the rivers Rhine and Meuse. The fight against floods has existed for a millennium. Many floods where caused by storm surges on the North Sea but also as a result of high waters on the rivers. Between 1750 and 1800 alone, the river dikes breached for 152 times.[48] After the establishment of the *Bureau voor den Waterstaat* (now known as Rijkswaterstaat) that executed the much-needed dike reinforcements, the frequency of river floods decreased.

The Dutch river dikes are designed to resist the 1/1,250 per year high water level and have a crest freeboard of at least 0.5m. Dikes in the inter-tidal area have a slightly higher protection level of 1/2,000 per year because the threat comes from both the sea and the rivers. This high protection level stems from the severe consequences a river flood would have. Failure of a dike would cause the dike ring area to fill up like a bathtub, leading to serious economical damage, not to mention the potential loss of life.

In the Dutch province Limburg the areas around the river Meuse have a lower safety level. The dikes along the Meuse in Limburg have been designed to withstand the 1/250 per year water level. This is five times larger than the design frequency for the dikes in the lower delta area, but the consequences of a flood in Limburg are generally less serious.

2.2.2 Germany

In Germany, there are no national laws on the level of protection against river floods. The protection level differs between the federal states (Bundesländer) but also within these states.

For example the protection level for the Rhine dikes in Nordrhein-Westfalen is 1/200 per year upstream of Duisburg and 1/500 per year between Duisburg and the Dutch border.[17] The 1/500 per year protection level corresponds with a design discharge of 14,500 to 14,800m^3/s at Emmerich. These dikes have a freeboard above design water level of 1m. Compared to the 0.5m margin in The Netherlands the safety level of the Dutch and these German Rhine dikes is actually quite similar.[18]

2.2.3 USA

Many rivers cross the USA with probably the most well-known being the Mississippi River. The protection level in the USA differs for various rivers and various cities. In table 1 the protection level for different cities is shown to vary from 1/85 to 1/500 per year.

City	**River**	**Return period (years)** [47]
Saint Louis	Missouri and Mississippi Rivers	500
Dallas	Trinity River	500
Kansas City	Missouri and Kansas River	500
New Orleans	Mississippi River	250
Sacramento	Sacramento River	85

table 1 Protection level for some of America's Major River Cities

2.2.4 Vietnam and China

Part of Vietnam lies in the deltas of both the Red River and the Mekong River. During the 20th century, the Red River has caused 26 major floods, the biggest one in 1971 which had a return period of one hundred years. The dike system in the Red River delta protects some 16 million people. The dike heights average between 6 to 8m metres with maxima up to 11m. Design frequencies are in the order of 1/10 to 1/50 per year.

One of China's major rivers is the Yellow River. A large part of the Yellow River dikes can withstand a flood level that occurs as frequent as once every 10 to 20 years. As a consequence flooding often occurs. The dikes along the Yellow River on average have crest heights of around 10m, incorporating a freeboard of 2 to 3m with respect to the normative high water level.[3]

2.3 No guarantee for absolute safety

Protection levels of river dikes around the world differ in the order of magnitude from 1/10 (Asia) to 1/1,000 (Europe) per year. But even when protection levels are high, there is always a possibility that dikes are breached and a flood occurs. The next chapter discusses the different failure mechanisms that contribute to dike failure.

3 Dike failure mechanisms

3.1 General

The protection level of a dike is often linked to a certain normative water level. With this, the protection level is actually based on the mechanism of wave overtopping or overflowing. Traditionally, the focus of emergency measures is primarily on preventing this mechanism.

There are nevertheless several other mechanisms that could lead to dike failure, three of which are macro-instability, micro-instability and piping. This chapter discusses these mechanisms both theoretically by making some calculations and practically by looking at the historical causes of dike failure and flooding in several countries around the world.

3.2 Stability and failure mechanisms

3.2.1 General

This paragraph contains several calculations of the failure mechanisms overtopping, macro-instability, micro-instability and piping. The purpose of these calculations is to determine the potential impact and the sensitivity of the of the mechanisms to the input parameters. The used equations and parameters are incorporated in Appendix I.

For these calculations a case dike with a height of 6m, crest width of 5m (common for a dike with a road on its crest) and a slope of 1:3 will be used. A river dike with these dimensions is not uncommon in the Dutch river area. The impact of these mechanisms will be assessed for a water level that varies from 0 to up to the dike crest.

The results will be presented in graphs with factors of safety (FS). The factor of safety is the ratio between the resisting and the driving force of the failure mechanism.

$$FS = \frac{resisting\ \ force}{driving\ \ force}$$

When the factor of safety is 1, the driving and resisting force are exactly equal that the system is on the brink of instability. When the safety factor drops below 1, the driving force becomes larger than the resisting force. In other words: the dike becomes unstable.

3.2.2 Overflow and Overtopping

Overflow and wave overtopping cause a flow of water over the crest of a dike leading to erosion of the inner slope. Eventually this erosion will lead to failure of the inner slope and collapse of the dike. Overflow and overtopping are the most obvious failure mechanisms and traditionally the focus of emergency measures has been on preventing these mechanisms. The mechanisms cause a flow of water over the dike which could lead to erosion of the inner slope eventually resulting in the collapse of the dike.

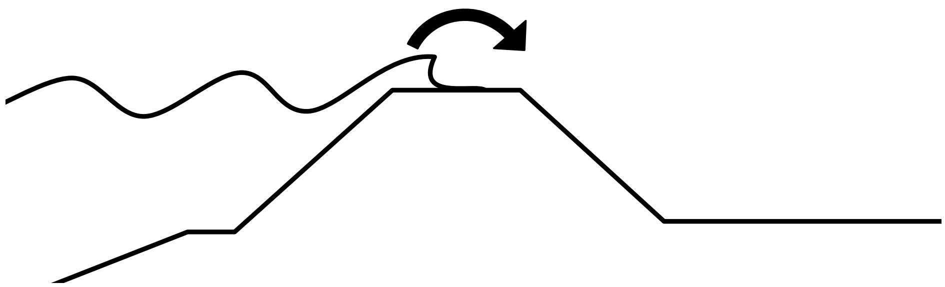


fig. 3 Overtopping of a dike

Overtopping does not become a problem before the so-called critical discharge is exceeded. The critical discharge depends on the state of the inner slope of the dike. This relation between inner slope condition and critical discharge is displayed in table 2.

Condition of inner slope	**Critical average discharge, q_c (l/s per m)**
Sandy soil with poor grass cover	0.1
Clayey soil with reasonably good grass cover	1.0
Clay covering and grass cover according to the requirements for the outer slope or for a armored inner slope	10

table 2 Critical average discharge for different inner slope conditions [10]

Recently, tests have been carried out by Infram and Rijkswaterstaat in The Netherlands in which the resistance of the inner dike slope against erosion was tested on a reinforced grass cover. Results show that higher overtopping discharges than 10 l/s per m can be allowed for slopes that are in good condition.

The most important parameters in determining the overflow and overtopping discharge are the height of the dike crest above the still water level and the wave height. The calculations were made for wave heights ranging from 0.25m to 1.00m.

The results are displayed in fig. 4. It shows the increase of the overtopping discharge related to the free water level below the crest. What can be seen in the chart is that for waves of 1m the critical discharge of 10l/s per running meter is already exceeded for a still water level of 1.25m under the crest. For lower waves the discharge becomes critical at a much higher water level. Waves of 0.5m only become dangerous when the water level is less than 0.5m below the crest.

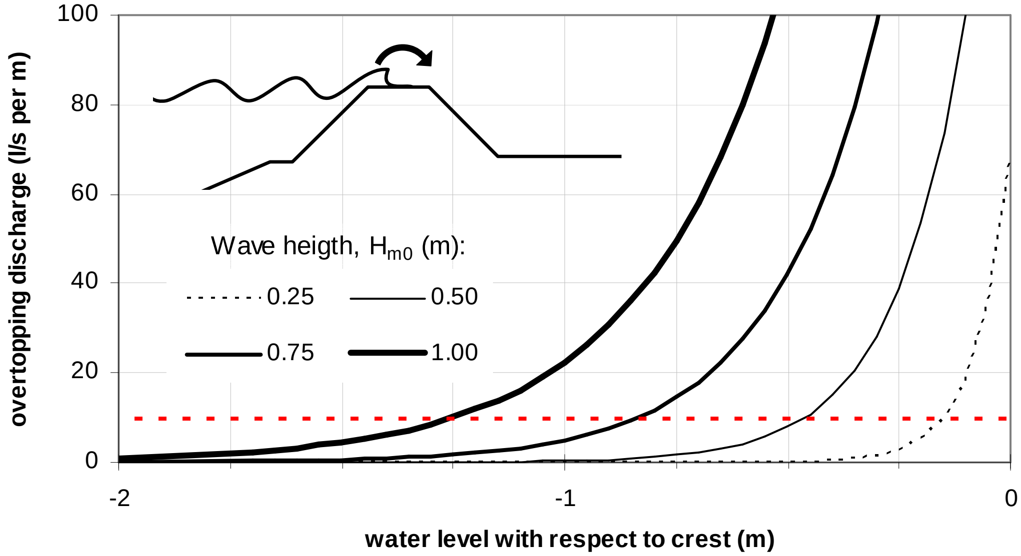


fig. 4 Wave overtopping discharge relative to the distance of the still water level below the crest

The wave height that can occur on a river depends mainly on the wind velocity and the fetch length. The latter is the distance over which the wind generates waves on the water surface and is in fact related to the width of the river. The depth of the water is also of some influence but not to the extent of fetch and wind velocity.

For rivers, the fetch is generally limited to several hundred meters for smaller rivers up to a few kilometers for bigger rivers. High waves will then only occur when the weather conditions are extremely unfavorable. For a fetch of 1km and for waves to reach a height of over 0.75m the Beaufort wind force has to be at least 10 (see the Bretschneider calculations of Appendix I). To put this in perspective: in The Netherlands wind force 10 or higher has only occurred 58 times in the last century.[40]

For rivers this means that overtopping will generally only occur for extremely high water levels. The probability that a high water level occurs simultaneously with extreme weather conditions is very small because these two situations are independent.

Intermezzo
For a storm surge at sea this is different since high water level and extreme weather conditions are correlated. An extremely high water level is then always the consequence of high winds creating a wind set up. That is why this report only discusses the use of emergency measures for river dikes. The weather conditions during a storm surge at sea would most likely undo any effort to apply the intended emergency measures.

3.2.3 Macro-instability

Besides overtopping, a possible failure mechanism is macro-instability. This describes the stability of a slope as a whole and is defined by failure of the dike through sliding of the slope. Generally, this is approached with a slip-circle analysis according to the Bishop-method. This method considers the ratio between load and strength (the Factor of Safety, see paragraph 3.2.1).

The mechanism is illustrated in fig. 5. It is influenced by the slope steepness (a steep slope is more prone to sliding), the soil type and the height of the freatic line in the dike. The latter is the groundwater level inside the dike that fluctuates with the river water level and is dependent on the permeability of the dike. A clay dike is less permeable as a result of which the freatic line will be lower than that in a sand dike. A high freatic line has a negative influence on macro-stability.

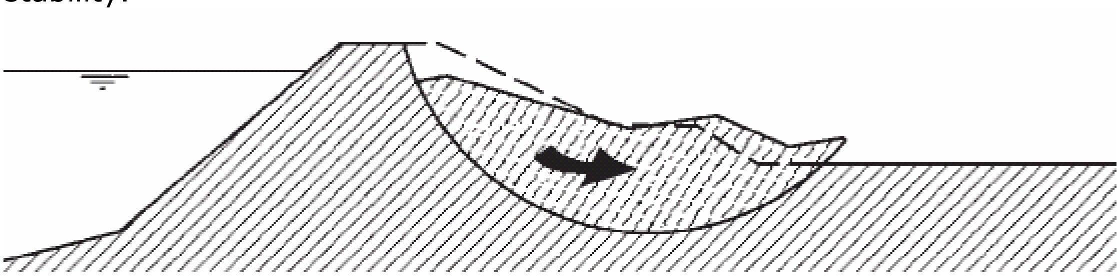


fig. 5 Failure of the inner slope of a dike through a circular slide plain

Two different dikes are calculated, a clay and a sand dike. For each dike the sensitivity of macro-stability to a variation in the slope is analyzed by varying the slope from 1/2 to 1/3 and 1/4. The results are shown in fig. 6.

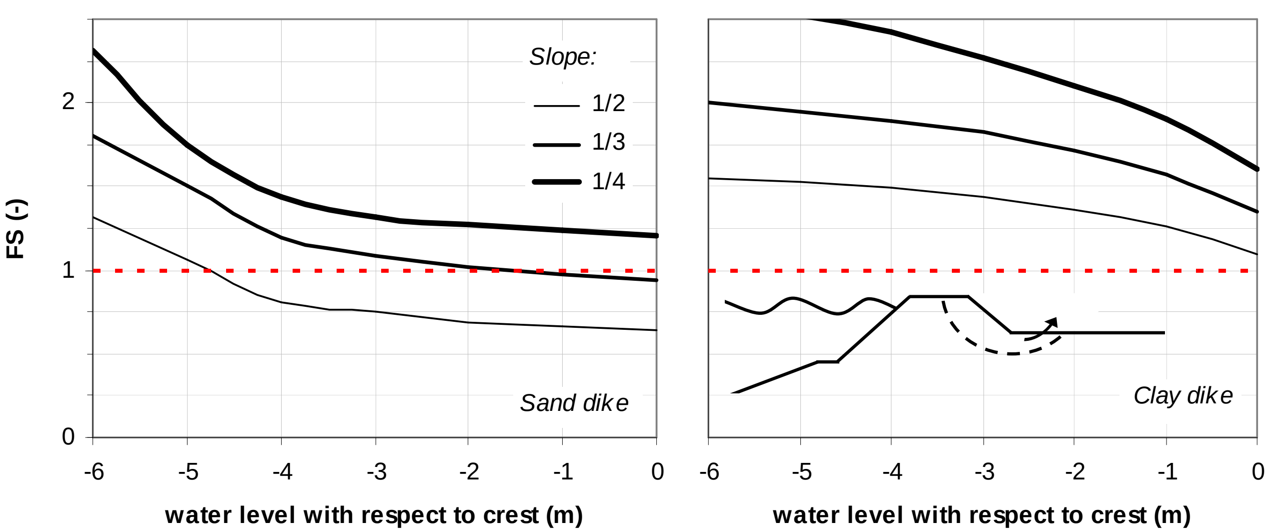


fig. 6 Macro-stability of a sand (left) and a clay dike (right)

Two important conclusions can be made on basis of these results. The first is that the macro-stability of a clay dike is good even for slopes as steep as 1/2. The second is that in the considered range of water levels, the stability of the sand dike is only sufficient for slopes less steep than 1/3. Depending on both dike geometry and material, macro-stability can become a problem for relatively low water levels.

3.2.4 Micro-instability

Micro-instability treats the stability of the grains on a slope, as opposed to the stability of the slope as a whole (macro-stability). This mechanism is a phenomenon in which the lower part of a dike becomes instable due to water seeping out of the inner slope.

The extent to which this is a problem depends on the dike material (see table 3). Clay dikes are insensitive for micro-instability since clay is a both impermeable and cohesive material. Sand lacks these two properties and is vulnerable to micro-instability. There are two different mechanisms: washing out of grains and heave and shear of the clay top layer.

For sand dikes it is a problem particularly when the water level on a river is high and when due to a failing drainage facility the freatic line in the dike is sufficiently high to reach the inner slope. The freatic line intersects the inner slope of a sand dike at approximately ¼ of the water level in front of the dike.[11]

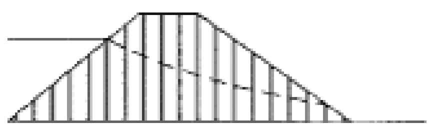
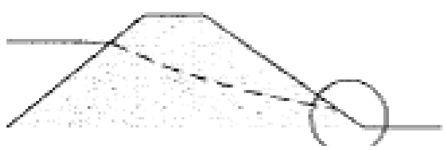
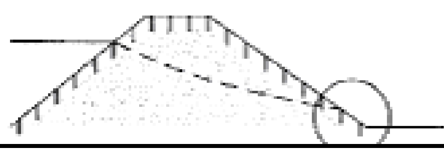
Dike type	**Effect high water level on micro-stability**	
Clay Dike		*No problem*
Sand Dike		*Washing out of grains*
Sand Dike with top layer of clay		*Heave and shear of clay top layer*

table 3 *Effects of high water on micro-stability for different dike types*

Whether or not erosion through micro-stability occurs, depends on the ratio between driving and resisting force, where the driving force is the result of the water flow through the dike and the resisting force a result of the weight of the grains on the slope and friction with other grains along the slope.

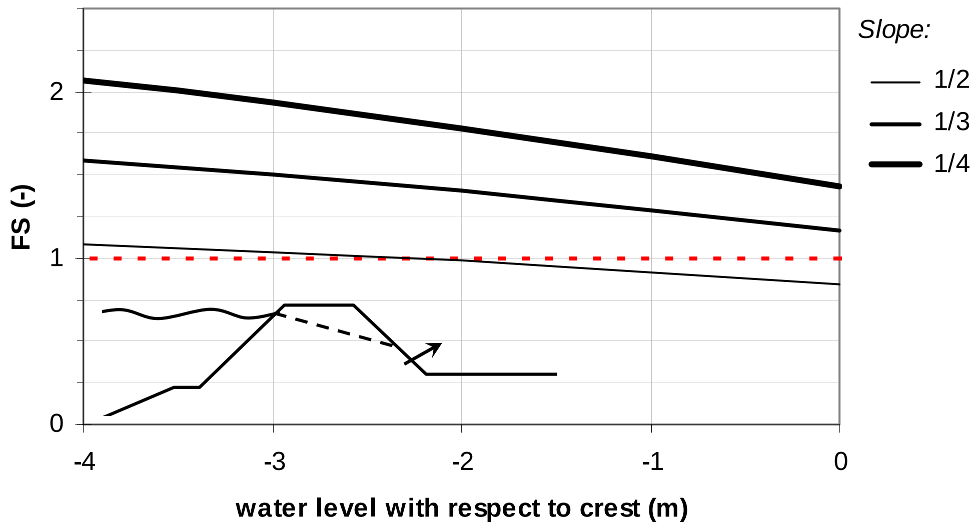


fig. 7 Safety against micro-instability

What is displayed in fig. 7 is that slope steepness has a significant effect on micro-stability. For a slope of 1/2 problems with micro-stability will occur for water levels of 3 to 4m while a slope of 1/3 is stable for the whole range of water levels. This means that micro-instability will only lead to dike failure for relatively steep slopes.

3.2.5 Piping

Piping is a mechanism similar to micro-instability. The difference is that micro-instability considers a flow of water through the dike while piping is caused by a flow of water underneath the dike. This flow of water can cause erosion of grains from the subsoil creating a pipe underneath the dike. This pipe undermines the dike which will eventually collapse. The development of a pipe underneath a dike and the eventual collapse of that dike are illustrated in fig. 8.

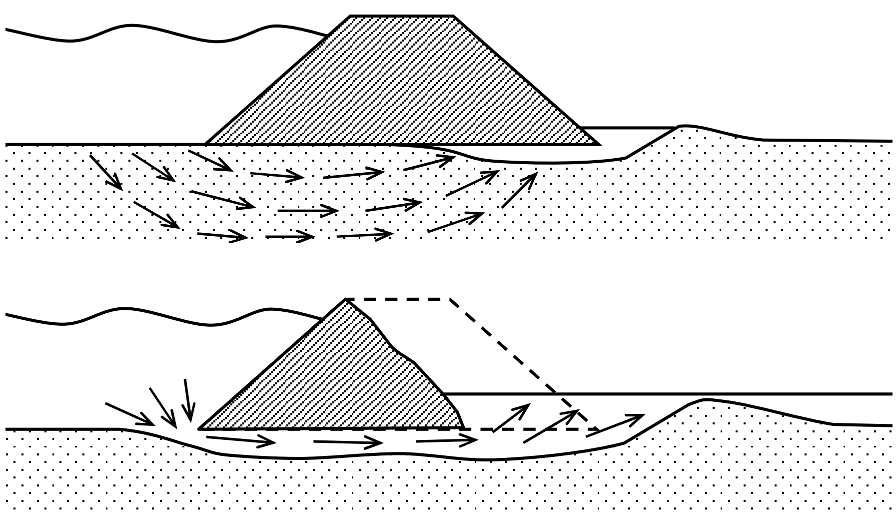


fig. 8 Two stages in the piping process

The piping process amplifies itself because the increasing pipe-length decreases the resistance and consequently increases the flow of water. Another important property is that piping often manifests at the same location for successive periods of high water, even when years may pass in between. This indicates that the created pipes somehow stay intact and may not "heal" in the meantime.[43]

There are three important conditions for piping to occur:

- an impermeable dike on a permeable subsoil;
- a water level difference between the river and the land side of the dike;
- a relatively short piping length;

In this case the piping length is in fact the base width of the dike which depends on the steepness of the inner and outer slope. Varying the dike slope from 1:2 to 1:4, results in a piping length that ranges from 29 to 53m. Two other important parameters in the calculation are the permeability of the sand underneath the dike and the grain diameter of this sand. To be able to visualize this sensitivity, both the results for fine (200μm) and coarse (300μm) sand are displayed in fig. 9.

The shown safety factor against piping is the ratio between the critical head difference (Δh_c) and the occurring head difference (in this case the water level on the river). This critical head difference can be calculated with the method of Sellmeijer (see Appendix I).

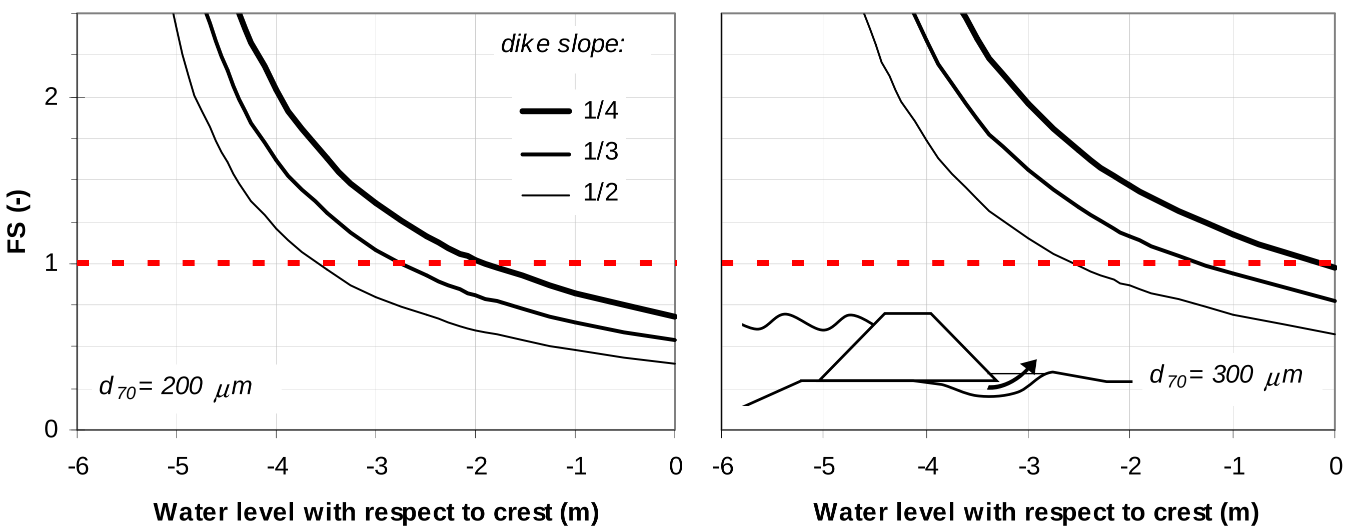


fig. 9 Safety against piping for different water levels

It is clearly visible that the safety against piping drops rapidly when the water level increases. The figures also show that finer sand is much more vulnerable to piping than coarse sand. For sand as fine as 200μm and the dike with slopes 1/3, which is not uncommon in the Dutch river area, piping occurs for a water level difference of little over 3m. In this case the water level is almost 3m below the crest.

Intermezzo
The piping mechanism can be examined by means of the Sellmeijer model. Sellmeijer incorporates many parameters and is therefore probably one of the more accurate methods presently available. However, the model is still a schematization of reality. There have been situations where piping occurred though calculations according to Sellmeijer did not indicate this as being possible.

A top clay layer is a common feature along river dikes. For piping to occur this clay layer must be discontinuous at some point to allow the outflow of water. Often discontinuities are present like man made works such as a ditch behind the dike or geological features like natural sand-ridges crossing underneath the levee (fig. 10).

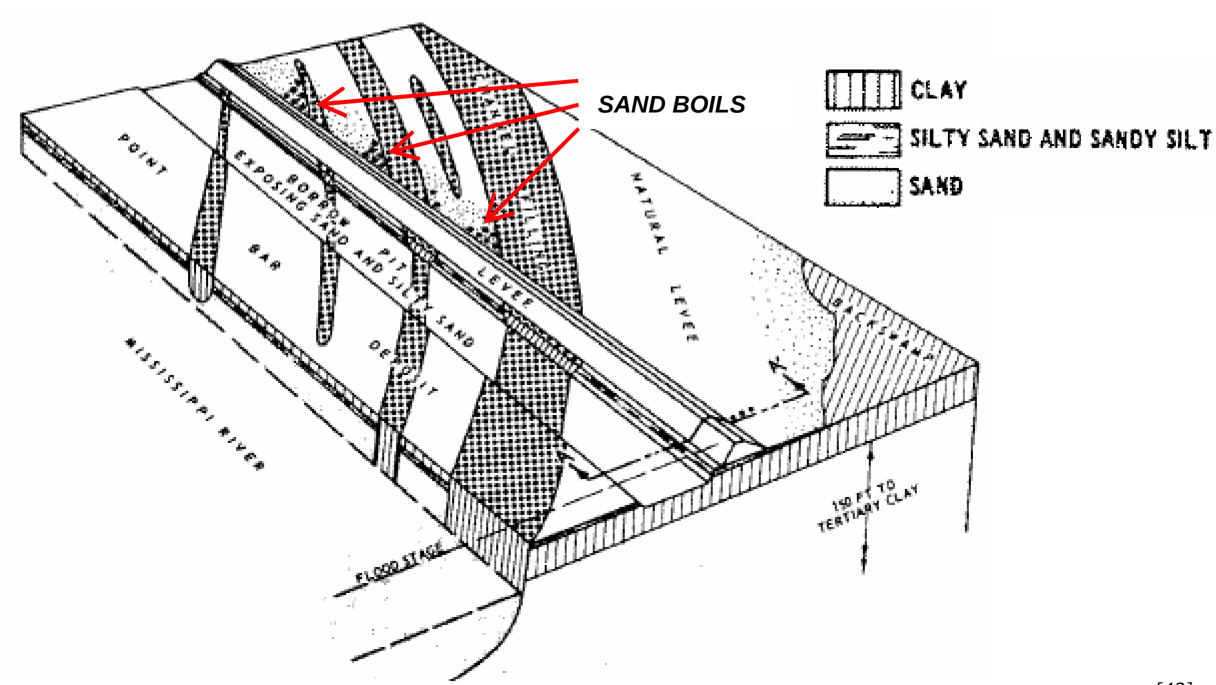


fig. 10   Sand deposits crossing a Mississippi river dike and sand boils behind the dike[43]

If a clay layer is continuous, the only possible way for piping to occur is by lift up of the clay layer by the water pressure in the aquifer. When the pressure becomes too great, the uplift force can cause the top layer to crack which will lead to an outflow of water and possibly piping. The uplift or "heave" of a clay layer is illustrated in fig. 11.

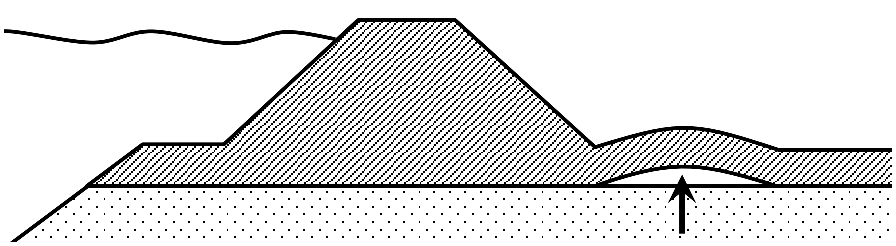


fig. 11   Heave of the clay top layer behind a river dike

The resistance against heave consists of the weight and thus the thickness of the impervious top layer. The uplift force is determined by the difference between potential head in the aquifer and the freatic water level. The factor of safety is the ratio between these two forces. The calculation results are displayed in fig. 12. The water level on the polder side is assumed to be at the surface.

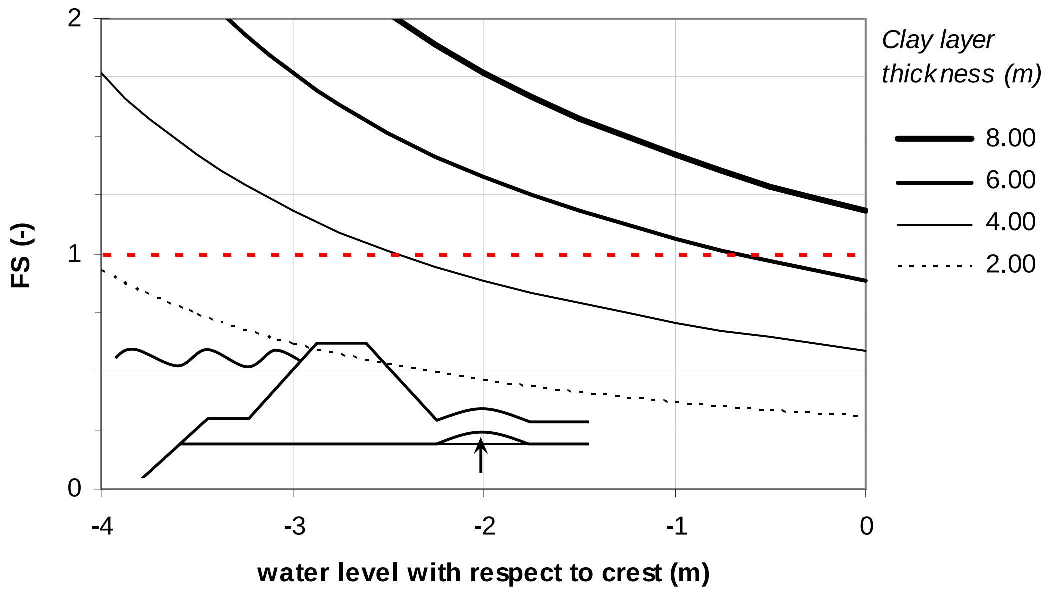


fig. 12 Safety against heave related to the water level difference

The results show that thin clay layers of 2m or less have a very low safety against heave and in fact will lift up for a water level of only 2m (in this case 4m below the dike crest). It is clear that an increasing water level exacerbates the heave mechanism and a thicker clay layer gives a larger resistance. To prevent heave, the clay layer should have a thickness of at least 6 to 8m.

3.3 Causes of dike breaches and flooding

3.3.1 General

After the general description and deterministic calculations of failure mechanisms in paragraph 3.2, It is still unclear to what extent the mechanisms and properties discussed, occur in practice. Therefore, this paragraph will give a brief overview of historical causes of dike breaches and flooding for river areas in different countries around the world.

3.3.2 The Netherlands

The last flooding by the River Rhine in The Netherlands occurred in 1926 when the discharge reached its historical maximum of 12,800 m^3/s. In 1993 and 1995, high water on the rivers caused a lot of trouble although no dikes where breached. In 1995 the situation was that precarious that the decision was made to evacuate 200,000 people. At several locations sand boils occurred, caused by the indicating the outflow of sand due to piping.

fig. 13 A sand boil

Historically, two third of the dike failures were caused by erosion of the inner slope as a result of overflow and wave overtopping, other mechanisms where of a lesser importance. Erosion of the outer slope and instability of the inner slope accounted for respectively 6 and 5% of the dike failures. In only 1% of the cases piping was the cause of failure. It must be added that in the past the more complex failure mechanisms were probably not recognized. Dike failure would then have been attributed to overtopping while in fact other mechanisms may have had a larger contribution to the eventual collapse of the dike.[42]

fig. 14 Overflow (left) and macro-instability: slope slide at Streefkerk, 1984 (right)

Besides this, the current dikes are built more robust. In the past, dikes where constructed with slopes as steep as 1:2 for which only a small overtopping discharge could do serious damage. Nowadays slopes are less steep and a small overtopping discharge does not immediately lead to failure.

Another explanation for the large contribution of overtopping to dike failure is that for centuries dikes were heightened only after a flooding took place. The height of a dike was thus determined on basis of the highest known water level. This has been the course of action until the 1950's. After the 1953 North Sea Flood, the policy changed to a semi-probabilistic approach, meaning that a normative water level was determined on basis of extrapolation of statistical data. This data consisted of water heights measured over a period of only two centuries. The determined normative water level is then actually a water level that has never occurred before. Dikes are much higher nowadays so the probability of failure due to overtopping is much smaller. The importance of other failure mechanisms has therefore increased.

Flood-risks and safety in The Netherlands

The project "Flood-risks and safety in The Netherlands" (VNK) investigates the safety against flooding of the different dike rings in The Netherlands. In the main report of November 2005[19] and the different sub-reports,[20] the flood risk and the importance of different failure mechanisms are compared for several dike ring areas. The Dutch dike ring areas are displayed in Appendix IV.

From this comparison (fig. 15) it can be derived that overtopping and overflowing are seldom the most important failure mechanisms. The chart of fig. 15 shows that for eight dike ring areas, heave and piping is the most important failure mechanism. The contribution of overtopping and overflowing of dikes is in some cases almost negligible. For the dike rings 10, 36a and 52 the flooding probability is for more than 90% dependent on heave and piping.

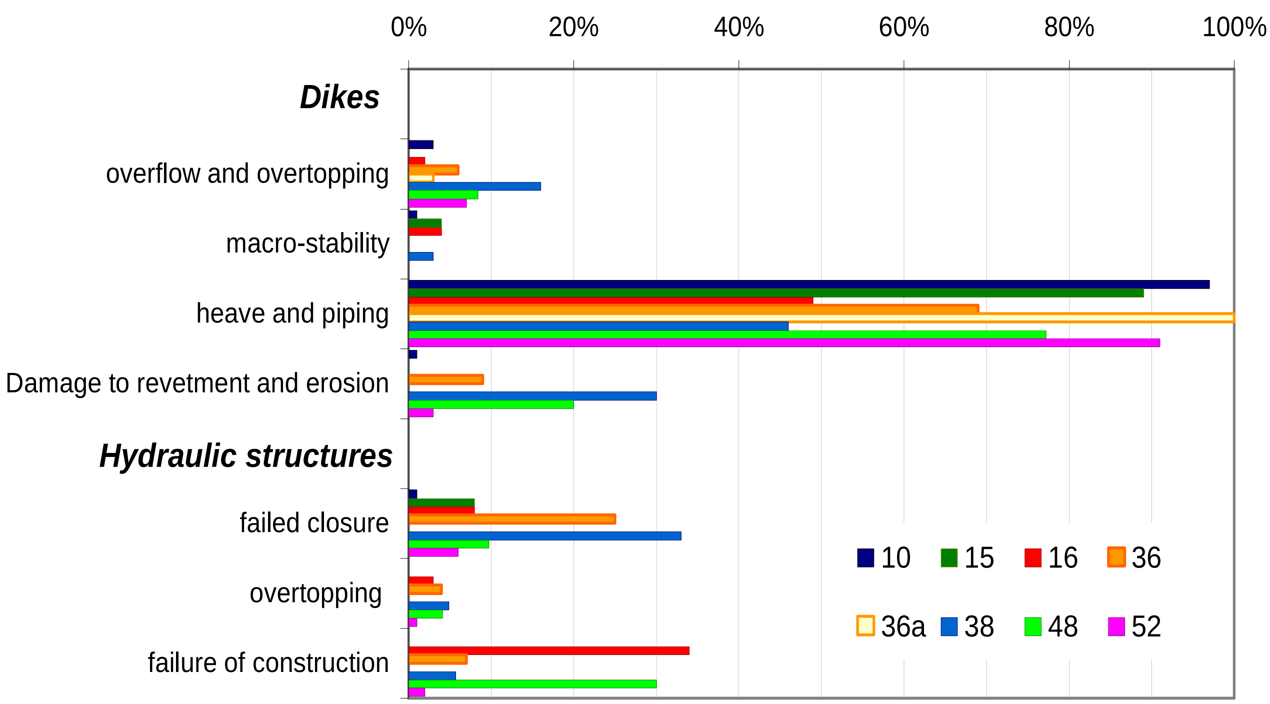


fig. 15 Relative contribution of different mechanisms to flooding probability of dike ring areas[20]

The contribution of a failure mechanism to the probability of flooding does not say anything about the actual probability of flooding; therefore the probability of flooding through piping for some dike rings is displayed in table 4.

Dike ring area	**P_{flood} (per year) through piping**
10 Mastenbroek	1/42
15 Lopiker and Krimpenerwaard	1/3.7
16 Alblasserwaard en Vijfheerenlanden	1/422
36 Land van Heusden / De Maaskant	1/110
36a Keent	1/280
38 Bommelerwaard	1/562
43 Betuwe, Tieler and Culemborgerwaard	1/441
48 Rijn en IJssel	1/214
52 Oost-Veluwe	1/48

table 4 *Probability of flooding through piping and contribution of piping to total flood probability for different dike ring areas*[20]

These probabilities should not be taken too literally. The once per four years flooding probability of dike ring 15 does not correspond with the fact that the last flooding of dike ring 15 dates back to the 1953 flood disaster. This can be partially attributed to the fact that there is a lot of uncertainty in the input parameters of the piping calculation. Although the data in table 4 may not fully correspond with reality, they may however indicate that piping could have a greater impact on dike stability than was acknowledged in the past.

These failure probabilities per dike ring area are composed of the different failure probabilities of individual dike sections. That piping is a local phenomenon can be seen in table 5 for the dike ring area Mastenbroek in The Netherlands. The different dike sections that are vulnerable to piping have lengths in the order of 1km or less and together represent only 20% of total length of the dike ring, but contribute for 90% to the flooding probability.

Dike section number	**Return period (years)**	**Length (m)**
04	67	600
11	240	1,000
08	294	1,000
41	334	800
36	380	500
06	416	1,300
03	871	1,150
42	1,131	900

table 5 *Weak dike sections due to piping for dike ring area 10* [20]

Another indication that piping is an important failure mechanism is presented by Ter Horst (2005).[1] For dike ring area 43 the conditional probability of failure given a high water level was determined. The mechanisms piping, instability of the inner slope, erosion of the outer slope and overtopping where considered.

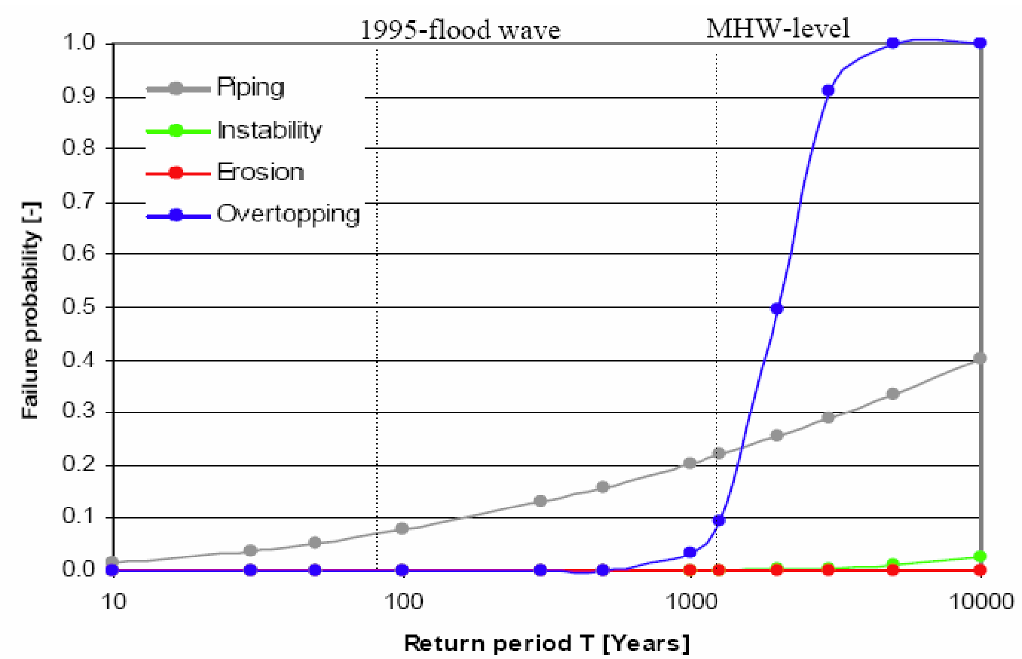


fig. 16 The development of the probability of flooding of several mechanisms with respect to the occurring water level

From the calculation results (fig. 16) it can be concluded that the influence of erosion and instability are negligible, while piping and overtopping are not. It can be seen that piping and overtopping are of equal importance for the 1/1,500 per year water level (roughly the design water level).

However, piping is already important for lower water levels. For example, the probability of failure through piping was 7 to 8% for the 1995 flood wave, while overtopping was of little concern. The high water situation with a return period of 50 years has a probability of failure through piping of approximately 5%.

That piping is more than a theoretical mechanism but can actually lead to the failure of dikes was demonstrated in Zalk, The Netherlands on January 8, 1926.[2] Here, the water level of the River IJssel was at a historically high level but still 50cm below the crest. The water level difference between the river and polder was only 2.5m but this was enough to cause a sand boil to develop. The continuous sand transport eventually undermined the dike and led to its collapse.

3.3.3 Outside The Netherlands

The Netherlands is a country where the dikes have a very high protection level. But as was described in paragraph 2.2, there are other countries that have much lower protection levels and possibly the causes of flooding are also different. This paragraph describes the situation in those countries.

USA

The Mississippi recently flooded in 1993, the flood was larger than the 1/100 per year event. Hundreds of levees failed along the Mississippi and Missouri Rivers. As a result, fifty people were killed and damages approached US$ 22 billion.

fig. 17 A breached river dike during the Great Flood of 1993

During the Great Flood of 1993 sand boils were reported along the Mississippi north of Cairo, Illinois, where a water level difference of 4m between the river and the landside occurred. Sand boils were abundant within 5m of the levee toe, only small pin boils were observed at a distance of 100m from the levee, and beyond 100m, there was no significant evidence of piping. The sand boils had diameters between 0.5m to 10m, and they commonly extended 0.3m above the ground surface.[43] Several dikes eventually collapsed due to sand boils.

In the USA a lot of dikes have been built on permeable subsoil and piping is a mechanism that occurs regularly. Dike breaches in the past have been reported to be preceded by the occurrence of sand boils, a strong indication of dike failure due to piping.

Besides the Mississippi, also the rivers in California are familiar with piping during high water. The picture of fig. 18 shows the emergency measures against piping that have been constructed to stop the erosion of sand from underneath the dike. It can be seen that the sand boils all occur near the toe of the dike.

fig. 18 Sand boils and emergency measures in California (left) and a levee breach caused by piping during the 1972 Mississippi flood (right)

China

The Yangtze River flood of 1998 killed over 3,000 and left 14 million people homeless while the economic losses amounted to 24 billion US Dollar. A total of 495 dikes breached of which 64% caused by overtopping and 9% by piping.[45] Though overtopping is still the major contributor to failure, piping is also of importance.

When in the future dike crests are heightened to decrease the overtopping probability, piping will gain importance as a cause for dike failure. And the dikes will have to be heightened because the river beds in China are rising due to continuous sedimentation of up to 10cm per year. To keep up with this rising river bed, Yellow River dikes presently have heights of up to 30m.

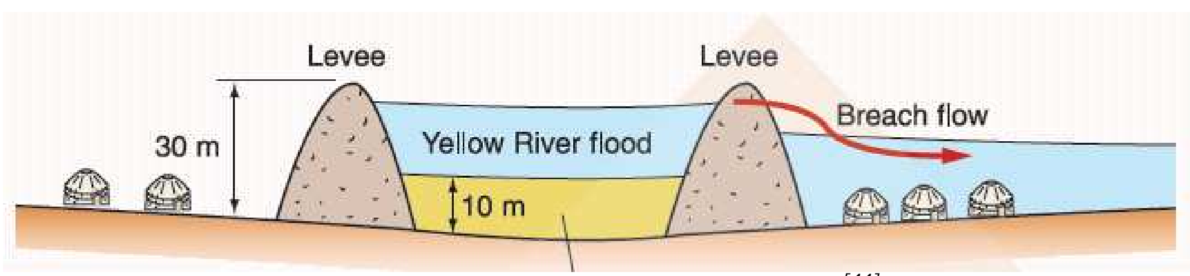


fig. 19 The raising of the Yellow River bed due to silt deposit [44]

Vietnam

The two primary causes of dike breaches in Vietnam are overtopping and piping through the foundation, resulting in sand boils near the landside toe. Seepage through the dyke body (micro-instability) also occurs, resulting in softening in the landside shoulder of the dyke and the risk of slope failures.[46]

3.4 Conclusion on failure mechanisms

In this chapter, four different failure mechanisms where considered. Their impact on dike stability during high water was discussed both theoretically by deterministic calculations and practically by explicating the historical causes of dike failure.

The deterministic calculations of paragraph 3.2 showed that a dike can fail due to piping, macro-stability or micro-stability even before overtopping becomes a problem. Especially piping proved to have the potential of causing dike failure for water levels far below the crest.

This image was confirmed by the overview of dike failure in paragraph 3.3. While overtopping occurs irrespective of geological properties, piping is a mechanism that occurs typically for a (relatively) impermeable dike on permeable subsoil which is a feature that can be found along many rivers in the world. The next chapter will elaborate on the possible emergency measures against both overtopping and piping.

4 Emergency measures

4.1 General

The previous chapter described four important failure mechanisms that contribute to dike failure of which two were concluded to be the most threatening to dike stability. These are the universally known overtopping and overflowing mechanism and the much less well-known but insidious piping mechanism. This chapter first discusses several emergency measures that can be applied to prevent each mechanism, after which the effectiveness of the two chosen measures will be assessed.

4.2 Emergency measures

4.2.1 Measures against overtopping

Traditionally, emergency measures focus at overtopping as being the failure mechanism to prevent. The emergency measure then consists of temporarily heightening the dike crest with a temporary flood defense (TFD) system. A well-known system that has been in use for a long time all over de world is a sandbag dam. When a high water level is forecasted a TFD can be applied to heighten the dike crest.

fig. 20 Placing sandbags

The required height of the measure is difficult to determine, since it is supposed to be used for emergencies that lie beyond the range of the protection level. It is not known beforehand how high a water level will rise beyond the dike crest. An indication of this can be made by the so-called decimation-height. This is height difference between the normative water level and a water level with an exceedence frequency that is smaller by a factor ten (e.g. 1/10,000 as opposed to 1/1,000 per year).

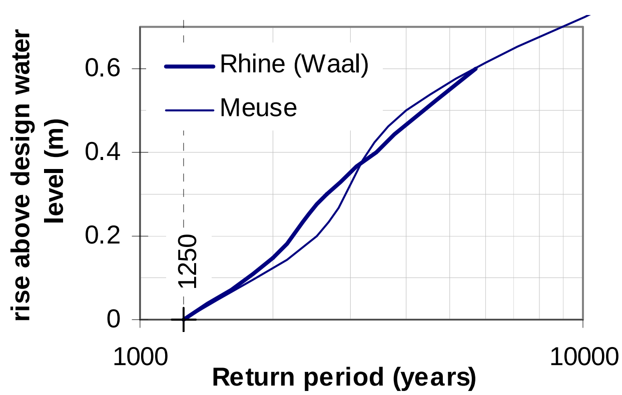


fig. 21 Rise above design water level for the Meuse and a Rhine river branch

In The Netherlands this is approximately 70cm for the upper river area (see fig. 21, data obtained from RBSO[14]). This would mean that an emergency measure against overtopping should be able to control a water level up to roughly 1m.

4.2.2 Measures against piping

While crest heightening as a measure against overtopping is quite obvious, measures against piping are not, therefore an explication of the possible measures is made here.

Sand berm

The first measure is actually meant to prevent heave and is therefore indirectly solving the problem of piping. The measure consists of the application of a sand berm that provides extra resistance against heave. The main disadvantages are that the measure requires a lot of sand and is time consuming.

fig. 22 A sand berm along a dike

Sand boil containment

When piping occurs during a high water level on the river, this becomes visible by so-called sand boils behind the dike. When sand boils are discovered in time, the pressure can be raised on the location by a wall of sand bags surrounding the boil. In this way, the water head difference (which is the driving force) is decreased and the sand boil will choke.

fig. 23 Containment of a sand boil

The above mentioned emergency measure is disputable. The method is in principle reactive, meaning that a sand boil is dammed only when it is discovered. It is however not unthinkable that a sand boil is not discovered in time. Another problem is that, if the whole area behind the dike is weak (which is often the case as was described in paragraph 3.3), closing of one boil will only move the piping process a few meters further away. The undermining process will then continue.

Water berm

A combination of the two previously described measures is the creation of a so-called water berm the construction of a temporary dam around the weak area behind the dike and filling the area with water (see fig. 24). The head difference decreases proportionally with the water height of the water berm. This means a move to the left along the curves of fig. 9, resulting in an increase of the factor of safety.

fig. 24 A large scale emergency measure to stop piping

Preventively damming the entire weak area is a much more reliable measure compared to reactively damming occurring sand boils. The measure can be used preventively because the weak areas are often known through historic data of sand boils or through information obtained from geotechnical survey.

4.2.3 Usefulness of emergency measures

Looking at the usefulness of prevention of piping and overtopping, each mechanism has certain interesting properties. Piping is limited to small locations with permeable subsoil while overtopping and overflowing can occur everywhere, which means that the market for measures against overtopping is much larger and in a way more interesting.

Nonetheless, an emergency measure against piping has two important properties. The first being that there are yet no commercial emergency measures against piping, possibly because the mechanism is only acknowledged to be a threat for the last few decades. And secondly, while overtopping can only occur for exceptionally high water levels, piping can become problematic for water levels that are much lower and occur more frequent.

The described measures against overtopping and piping are characterized by the fact that they require some sort of temporary dam or wall to control the water. The pictures in the previous paragraphs showed the use of sandbags for this purpose. There are however numerous alternatives for sandbags, which will be described in chapter 5.

To comply with the objective of this graduation project (the development of a method or system to temporarily increase the dike strength), the focus will be on the prevention of both piping and overtopping. The next paragraph will elaborate on the described emergency methods and will discuss their effectiveness.

4.3 Effectiveness of the emergency measures

In this paragraph a number of probabilistic calculations are made to give some more insight into the effectiveness of the two proposed emergency measures. The effect on the safety against flooding of these applications is evaluated by determining the conditional probability of failure given a high water level. In other words: the probability of failure through both mechanisms will be assessed for a range of water levels. The cases are (fig. 25):

- A reference situation where no emergency measures are used;
- Temporary crest heightening;
- The creation of a water berm at the inner side of the dike.

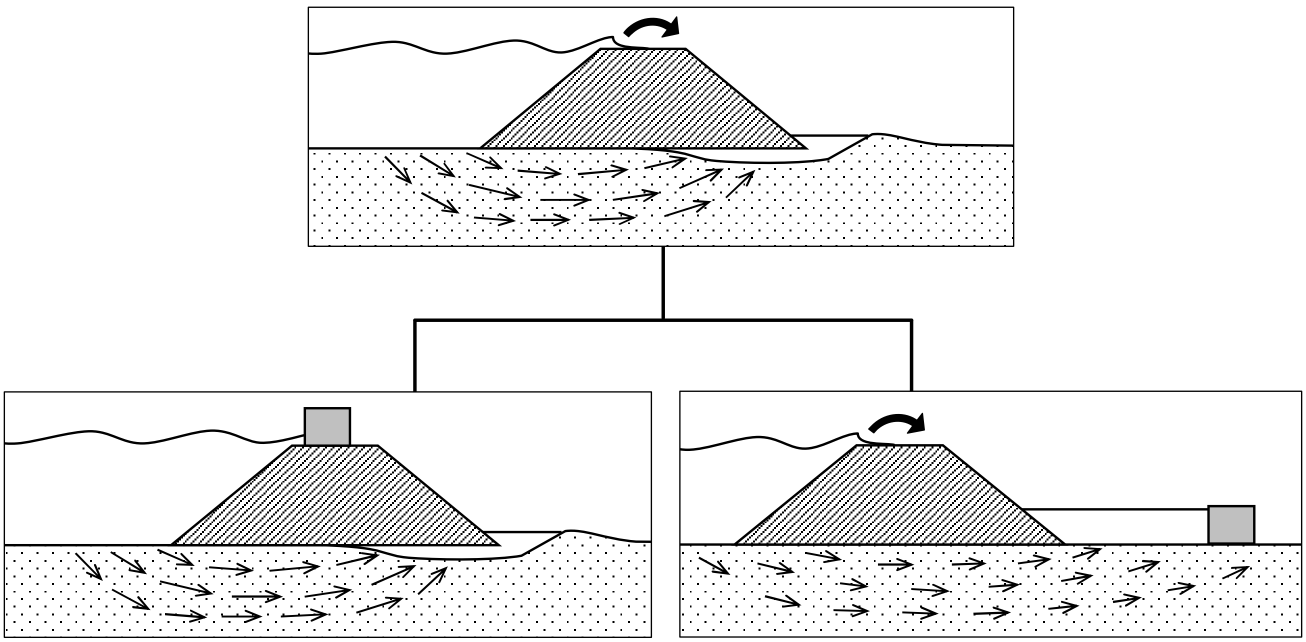


fig. 25 Schematized cases: reference situation (top), crest heightening (left) and water berm (right)

The calculations are made using a uniform schematized cross-section based on a Dutch river dike. It features a clay dike with a crest height of 6m, a crest width of 5m, an inner and outer slope of 1/3 and consequently a base width of 41m. On the land side of the dike a thin layer of clay (in the order of 1m thick) is present. The dike has a grass cover and is founded directly on a permeable subsoil of sand. The dike is situated perpendicular to the prevailing wind direction. The width of the river at the location is 1km.

The calculation is made by means of a Monte Carlo Analysis of 10,000 simulations. A conditional probability of failure is the failure probability given a certain situation, in this case a certain water level. The water level is therefore not a random variable, nor is the dike height. They are both deterministic variables.

4.3.1 Overtopping

Though waves on rivers are generally low because of the limited fetch, it remains possible that a high water level coincides with stormy weather conditions. The overtopping mechanism was earlier described in paragraph 3.2.2. For the probabilistic calculations the same equations are used. The equations and input parameters are incorporated in Appendix II.

The reliability function is: $Z = q_c - q$, where q_c is the critical overtopping discharge and q is the actual discharge. The dike fails for Z<0. The critical discharge depends on the condition of the inner slope. It was discussed earlier that the critical discharge varies from 0.1 to 10 l/s per running meter (see table 2). The considered dike consists of clay, and it is assumed that the inner slope has a grass covering of a quality that can resist a critical average discharge of 10 l/s per running meter. In the calculation it is assumed that the dike will fail ($P(o|h) \approx 100\%$) for a water level that rises above the crest.

4.3.2 Piping

The piping mechanism was earlier discussed in paragraph 3.2.5 and is again approached by means of the Sellmeijer model. The technical aspects concerning this method are further explicated in Appendix III, as are the parameters used in the calculation.

The reliability function for the piping mechanism is: $Z = \Delta h_c - \Delta h$. Here Δh_c is the critical head difference and Δh the actual head difference. When Z<0, the dike fails. The Δh value is a deterministic value because the calculations will be done to determine the conditional probability of failure.

Though there is a thin layer of clay present behind the dike, it is assumed that this layer will crack due to heave. Looking back at the results of the deterministic calculations of paragraph 3.2.5 this assumption is valid in the range of water levels for which piping occurs.

4.4 Case calculations

4.4.1 Reference - no emergency measures

The first case contains the results of the dike without emergency measures. Failure can be caused either by overtopping or by piping. The fault-tree of the system is displayed in fig. 26.

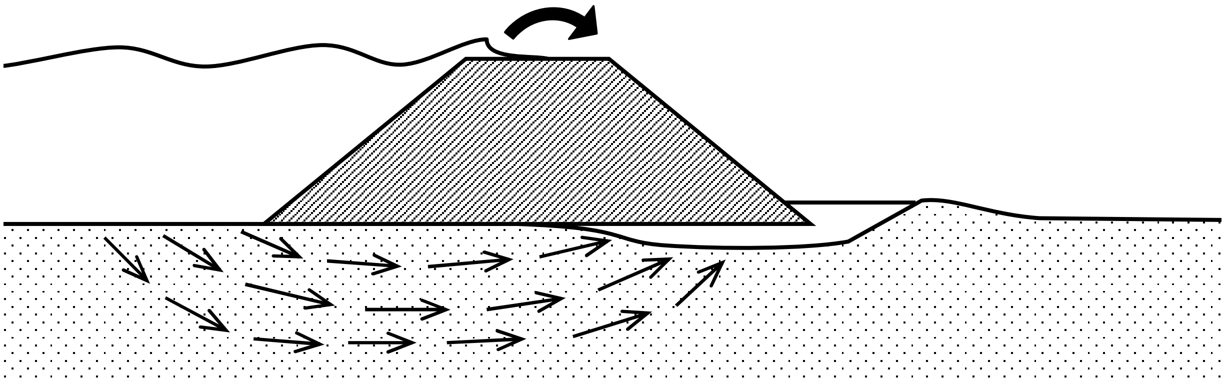


fig. 26 Reference situation

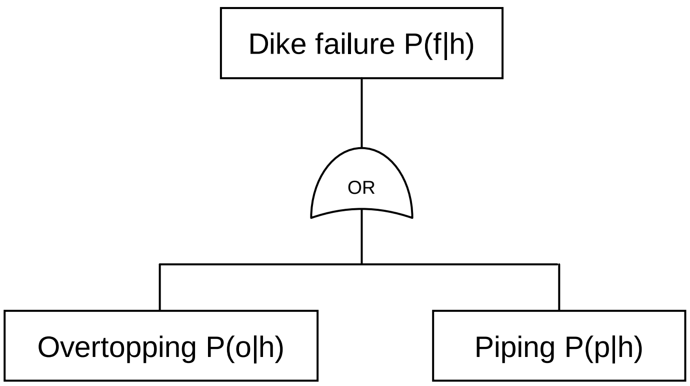


fig. 27 Fault tree for the reference situation

The total probability of failure depends on the extent of correlation between the two mechanisms. An upper and lower limit for the probability of failure can be determined based on the two extremes in the degree of correlation:

Upper limit: $P(f\,|\,h) = P(o\,|\,h) + P(p\,|\,h) - P(o\,|\,h)\cdot P(p\,|\,h)$ (no correlation);

Lower limit: $P(f\,|\,h) = \max\{P(o\,|\,h), P(p\,|\,h)\}$ (total correlation).

The results (fig. 28) clearly display the impact of piping for permeable subsoil. Though the sand is relatively coarse (d_{70}=300μm), the probability of failure is already over 70% for a water level of 1m below the crest. For this situation, overtopping is still of little concern. Overtopping only becomes relevant when the water level rises above approximately 0.5m below the crest.

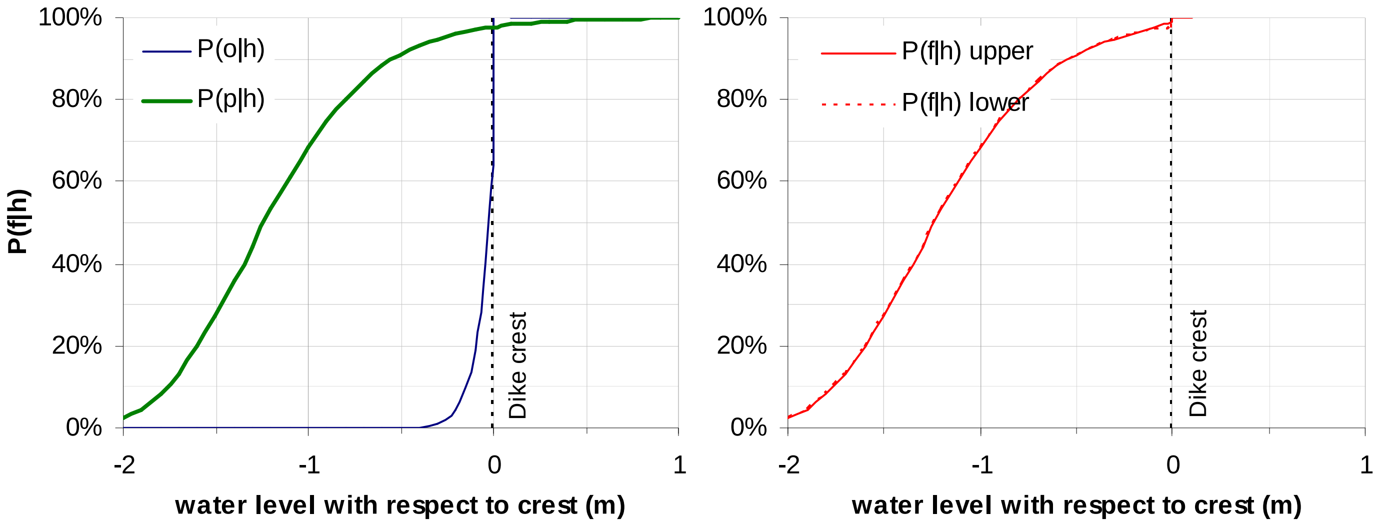


fig. 28 Development of the conditional probability of failure for the reference situation

4.4.2 Case 1 - Temporary crest heightening

With a measure against overtopping, the probability of failure through overtopping decreases. The fault tree is displayed in fig. 30. It shows that the dike can fail in three ways, the first being overtopping. The probability of overtopping with a temporary flood defense (TFD) on the dike crest shifts to the right with the water height the system can control. The piping mechanism is unaffected and $P(p\,|\,h)$ remains the same.

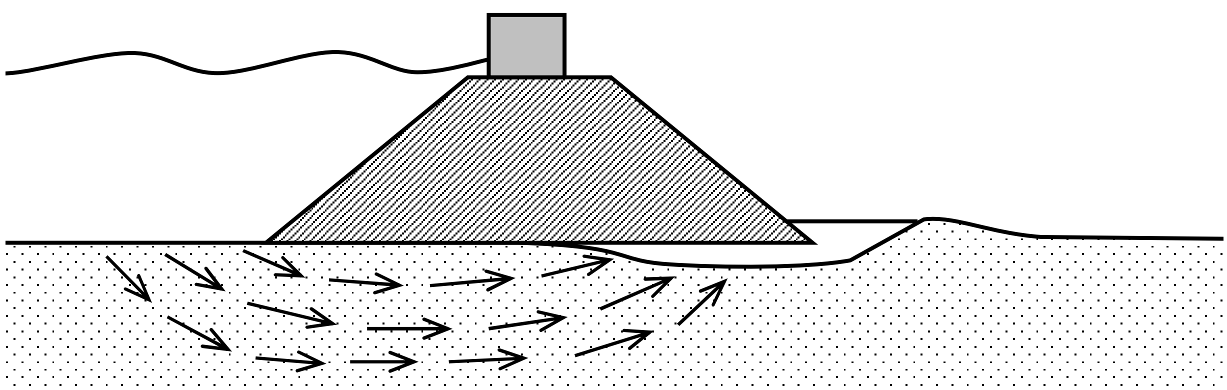


fig. 29 Case 1: temporary crest heightening

There is a third scenario that could occur, namely failure of the TFD system accompanied by overtopping causing dike failure. The overtopping probability without TFD (see fault tree) is then the original overtopping probability ($P(o\,|\,h)$) for the original crest height. These two failure modes can be assumed to be independent. This means that the probability of failure of the dike through overtopping after the TFD has failed, is the product of the two:

$$P(t \cap o\,|\,h) = P(t\,|\,h) \cdot P(o\,|\,h)$$

It is most likely that failure of the TFD system is immediately followed by failure through overtopping because (constructive) TFD failure only occurs for a water level well above the dike crest. In other words it can be assumed that: $P(o\,|\,h) \approx 1$

which means that: $P(t \cap o\,|\,h) \approx P(t\,|\,h)$.

The probability of TFD failure is discussed in chapter 11 after determining the preliminary TFD design. Here the focus is only on the effectiveness of the method, the reliability of the TFD system is discussed later.

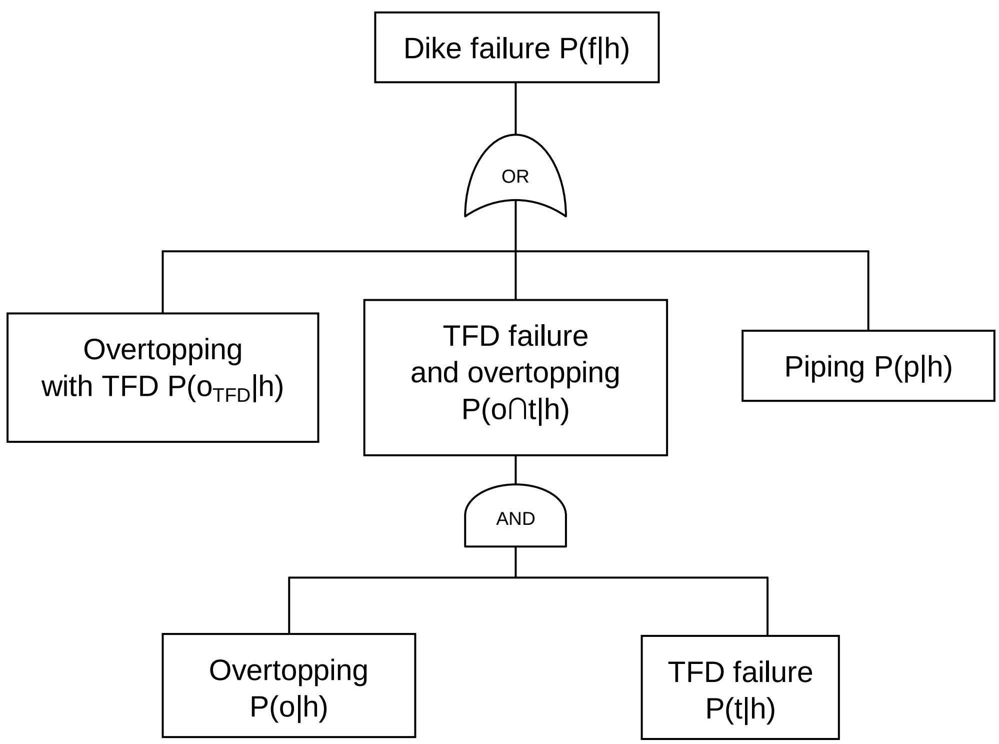


fig. 30 Fault tree for case 1: temporary crest heightening

The effectiveness of a crest heightening of 1.0m is displayed in fig. 31. The obvious result is that the $P(o\,|\,h)$-curve shifts to 1.0m above the crest. The total probability of failure only changes a little since piping still is of great importance. The water berm that is supposed to prevent piping is discussed in the next paragraph.

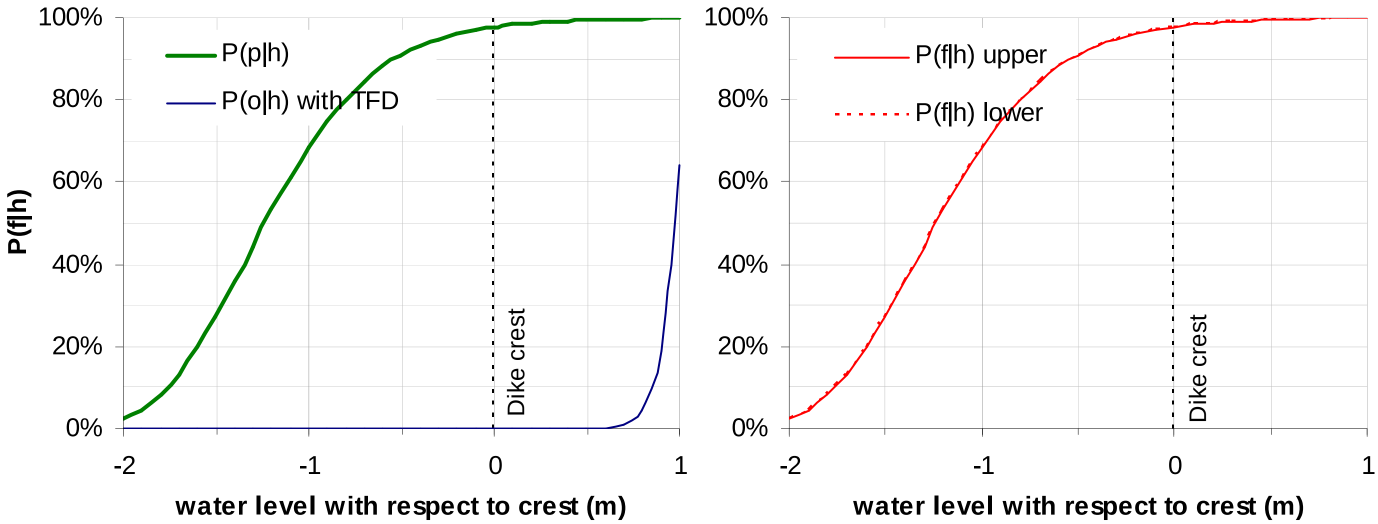


fig. 31 Development of the conditional probability of failure for case 1 - temporary crest heightening

4.4.3 Case 2 - Water berm

The application of a temporary water berm is meant to decrease the head difference that drives the piping mechanism. It shifts the $P(p|h)$ curve to the right similar to the effect of temporary crest heightening on overtopping. This is not entirely correct because piping could also occur behind the water berm. This is displayed in the picture of figure fig. 32 and incorporated in the fault tree.

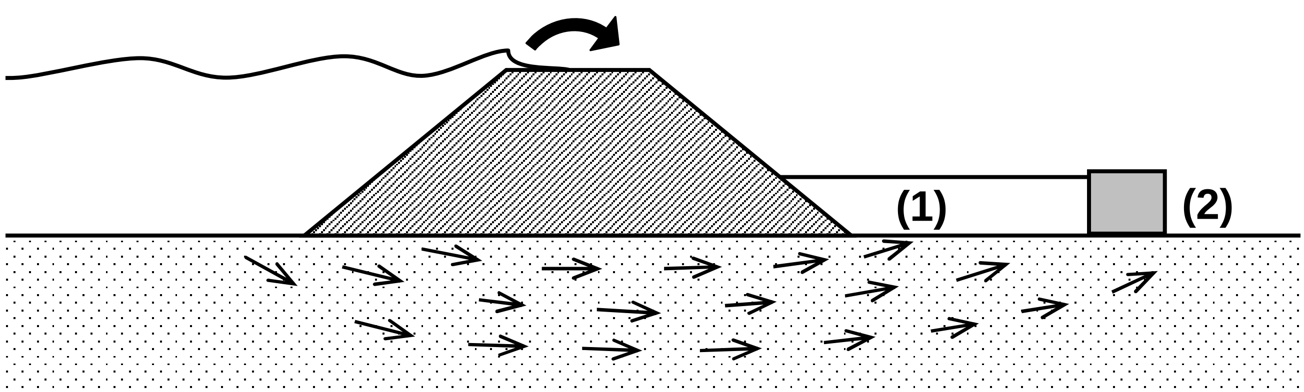


fig. 32 Schematic cross-section of case 2: water berm

The differences between piping at location (1) and (2) are the piping length and the head difference. For location (1) the piping length is the same as in the reference situation while the head difference is the original head difference minus the water level in the water berm. For location (2) the piping length is increased with the width of the water berm while the hydraulic head is the same as in the reference situation.

The fault tree also incorporates the water berm itself as a possible failure mode. As discussed in the previous paragraph, failure of a TFD on the dike crest implies overtopping. Opposed to this, water berm failure does not immediately result in piping, so: $P(b \cap p \mid h) = P(b \mid h) \cdot P(p \mid h)$, since berm failure and piping can be assumed to be independent. Water berm failure is not further discussed here. The reliability of the emergency measure is assessed in chapter 11.

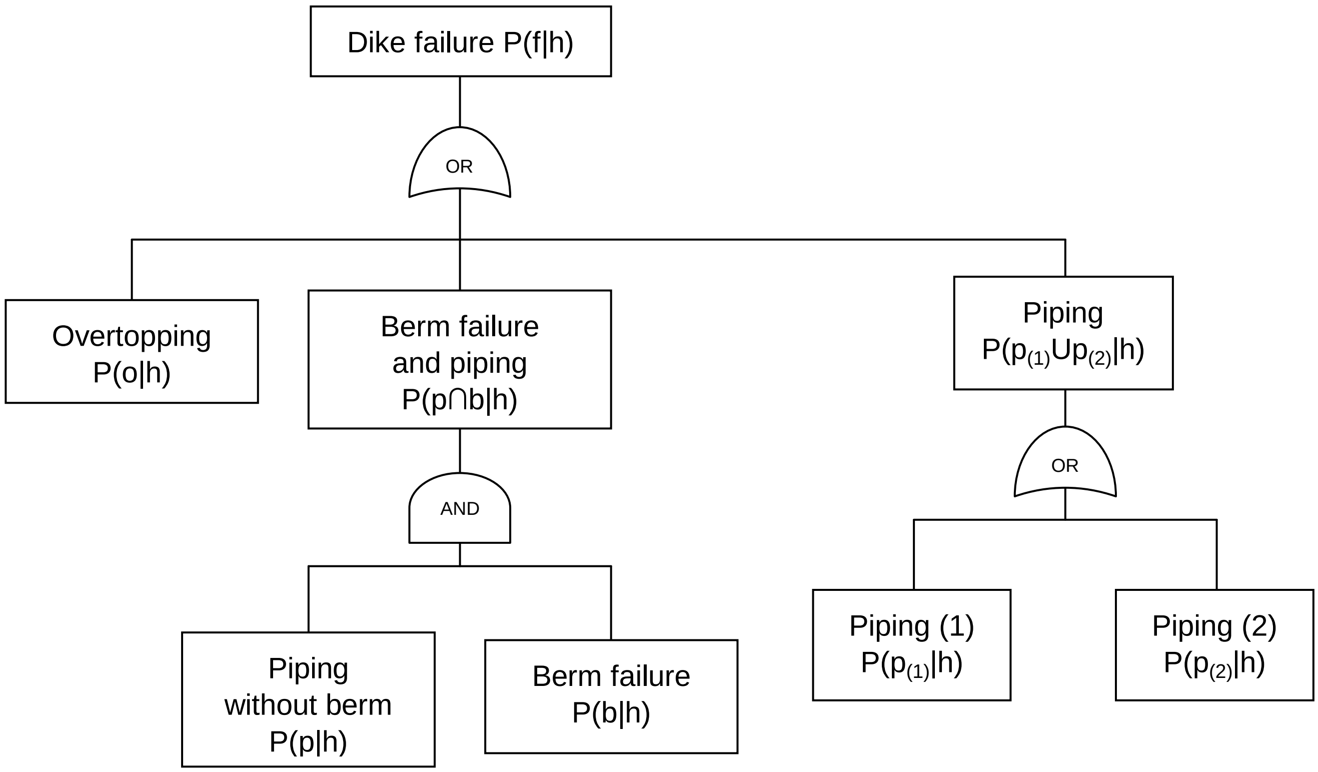


fig. 33   Fault tree for case 2: water berm

The probability of piping behind the water berm (2) depends on the original head difference and on the width of the water berm that increases the piping length for this location. The graph of fig. 34 shows that a wider berm will decrease the probability of piping behind the berm. The thick curve is the probability of piping in the berm $P(p_{(1)} \mid h)$, while the other curves display the probability of piping behind the berm $P(p_{(2)} \mid h)$ for berm widths of 10, 20 and 30m.

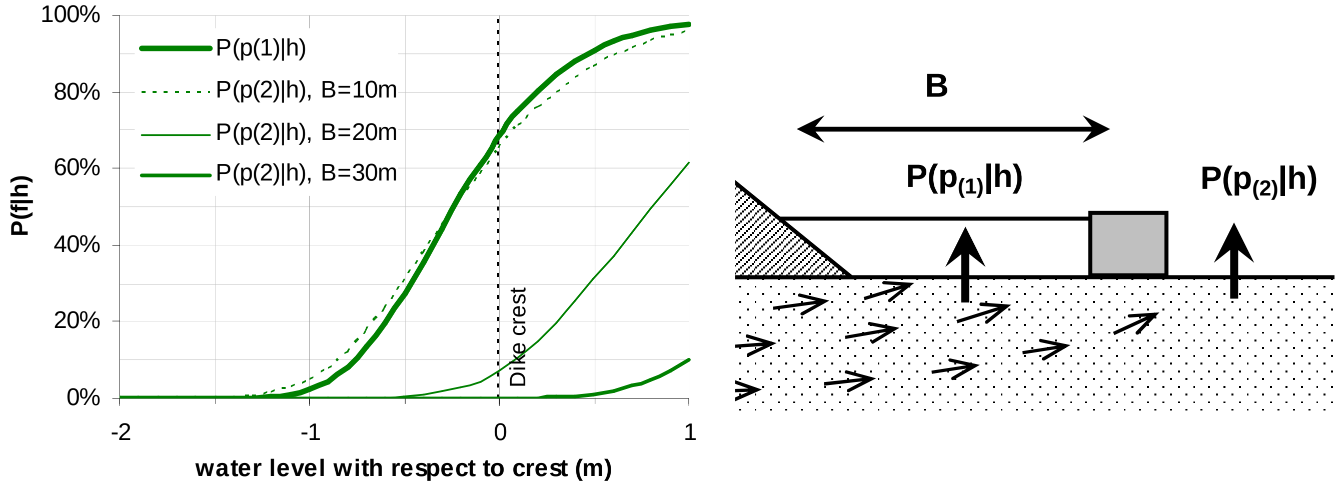


fig. 34   Effect of water berm width on probability of failure through piping

The total probability of piping depends again on the correlation between the two piping probabilities. The upper and lower limit can be described by:
Upper limit: $P(p\,|\,h) = P(p_{(1)}\,|\,h) + P(p_{(2)}\,|\,h) - P(p_{(1)}\,|\,h) \cdot P(p_{(2)}\,|\,h)$ (no correlation);
Lower limit: $P(p\,|\,h) = \max\left\{P(p_{(1)}\,|\,h), P(p_{(2)}\,|\,h)\right\}$ (total correlation).

It would seem that the lower limit approach neglects the influence of the water berm width since $P(p_{(2)}\,|\,h)$ is bound to drop out. This is not correct because for a small berm width piping behind the berm becomes larger and $P(p_{(1)}\,|\,h)$ drops out. This is displayed in fig. 34 . Another theory in favor of the lower limit approach is the fact that an occurring sand boil acts as a sort of exhaust valve reducing the pressure in the aquifer and thus preventing other sand boils to occur.

In this calculation the true piping probability is likely to be closer to the lower limit. In practice however, heterogeneities in the aquifer and the impermeable top layer (as discussed in paragraph 3.2.5) can have a large influence on the location where the water flows out and the sand boils occur. This will not further be discussed here but if heterogeneities are present the position of the berm should be adapted. In this schematized example a width of 20m seems acceptable. For water levels below the crest $P(p_{(2)}\,|\,h)$ is still small relative to $P(p_{(1)}\,|\,h)$ which means upper and lower limit deviate only little.

When these results are compared with the probability of failure through overtopping, it becomes clear that the upper and lower limit are now much further apart. This is because overtopping and piping are now of the same order of magnitude, which means the upper limit is approximately twice the lower limit. As expected, the total failure probability has gone down.

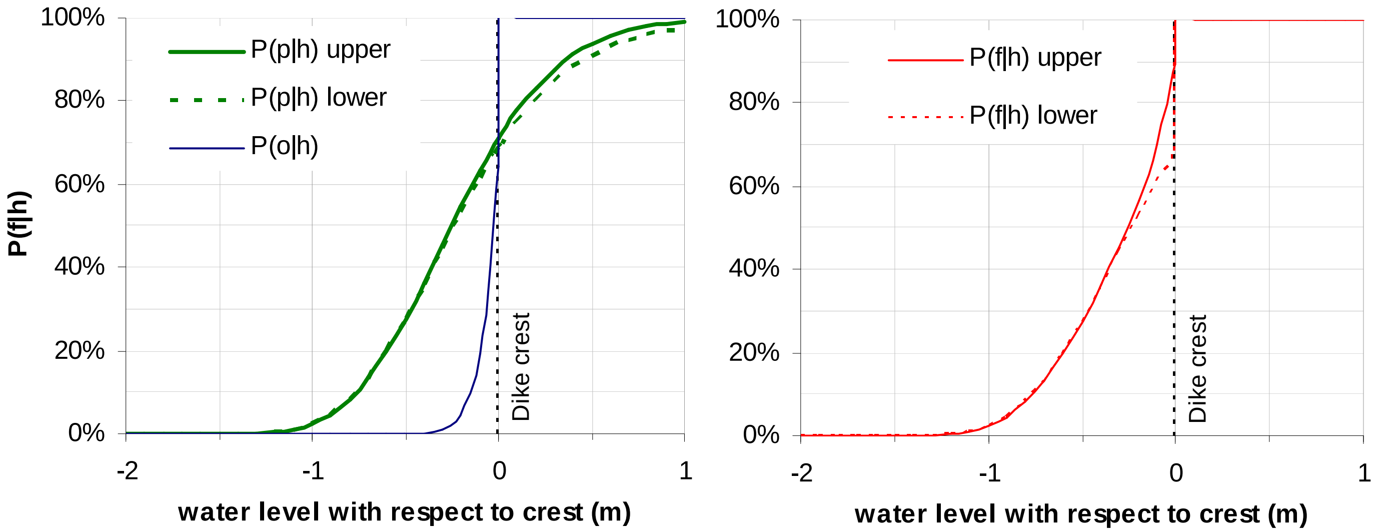


fig. 35 Development of the conditional probability of failure for case 2 - water berm

4.4.4 Case comparison

Comparison of the cases in one figure gives a clear view of the effectiveness of each measure. The chart of fig. 35 shows the upper and lower limit for the total probability of failure for each case. For the sake of completeness: the temporary crest heightening featured a TFD able to control 1m of water and the water berm results apply for a berm width of 20m and a water berm height of 1m.

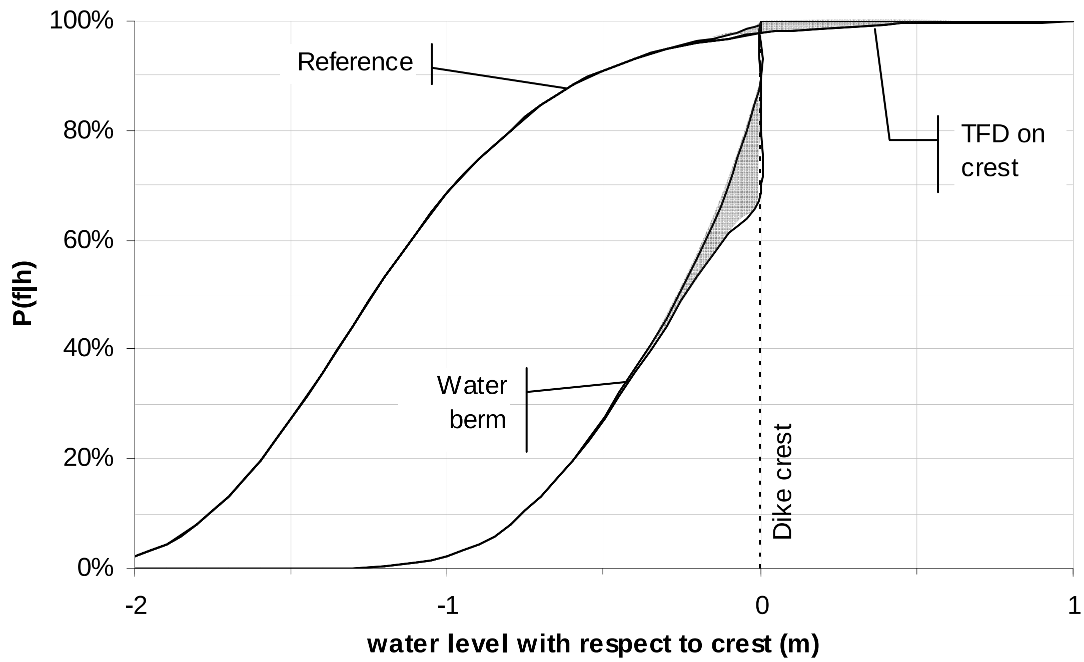


fig. 36 Comparison of the total probability of failure for the three cases

The figure shows that a water berm has a significant positive effect on the total probability of failure of the dike. For a water level of 1m below the crest the original dike already a encountered a probability of failure of almost 70% while application of a water berm reduces this to 2%.

The curve that represents the effect of a TFD on the crest is practically the same as that of the reference situation. Only the lower boundary is a little lower for water levels above the crest. Obviously a TFD on the crest has no real contribution since it does not solve the problem of piping.

4.5 Final remark

This chapter described the effectiveness of the water berm as a measure to prevent piping and temporary crest heightening as a measure against overtopping. The calculations have two important results:

- emergency measures that temporarily heighten the dike crest are only effective when overtopping is the primary failure mechanism and other mechanisms are of a lesser importance.
- the water berm is an effective measure against piping.

The calculations where made for a specific case incorporating features sensitive to piping. The result that a measure against overtopping is quite ineffective is therefore no surprise. There are however numerous other situations where piping is not a problem and measures against it unnecessary, while overtopping could be a threat.

Besides the effectiveness, another remark must be made about the reliability of emergency measures. Though the fault trees for both cases incorporated the failure mode of the emergency system, no calculation on the reliability was made. The next chapters will describe different temporary flood defenses and ultimately the best type of system will be determined. A reliability analysis will be made for this specific system.

5 Temporary flood defense systems

5.1 General

The previous chapter discussed two possible methods to increase the strength of a river dike during high water. These methods focus on overtopping and on piping. It was mentioned that both methods require some sort of water retaining device. Systems of this kind are numerous and will henceforth be referred to as temporary flood defense (TFD) systems.

This chapter gives a brief and general description of several existing temporary flood defense systems. In this chapter the numerous systems are subdivided in five categories, namely:

- immobile systems;
- traditional systems;
- concrete systems;
- composite structures;
- water-filled systems

Appendix V contains three flap-out pages that can be consulted for more specific information on the systems.

5.2 Immobile systems

5.2.1 System wall

Immobile systems are systems that are fixed to one location and can not be applied elsewhere. An example of an immobile TFD system for structural problems is a system wall. In a period of high water it can be deployed by adding beams between the columns (fig. 37, left). It is applied in Germany but also in UK, USA, Austria and Thailand, over lengths of generally several 100m up to 9km (Cologne, Germany).

fig. 37 System wall (left) and failure of sheet pile foundation in New Orleans (right)

It is a system that should be well founded to secure its stability. That a lot of care should be taken in designing this (sheet pile wall) foundation was demonstrated in New Orleans during hurricane Katrina where concrete flood walls founded on sheet piles collapsed under the water pressure. This foundation makes the solution very expensive.

5.2.2 Dutchdam

Another example of an immobile system is the Dutchdam. The Dutchdam is quite similar to the System Wall but the deployment is faster because it can be unfolded relatively quick to heights up to 2.4m. A big advantage of this system is that it can be more or less hidden while not in use. This improves the esthetic properties of the system.

The Dutchdam has already been installed on several locations in Dublin and there are plans for use in four other European countries. The system will also be installed at the Waal quay in Nijmegen. The system was tested by WL Delft Hydraulics on waves and ship collision.

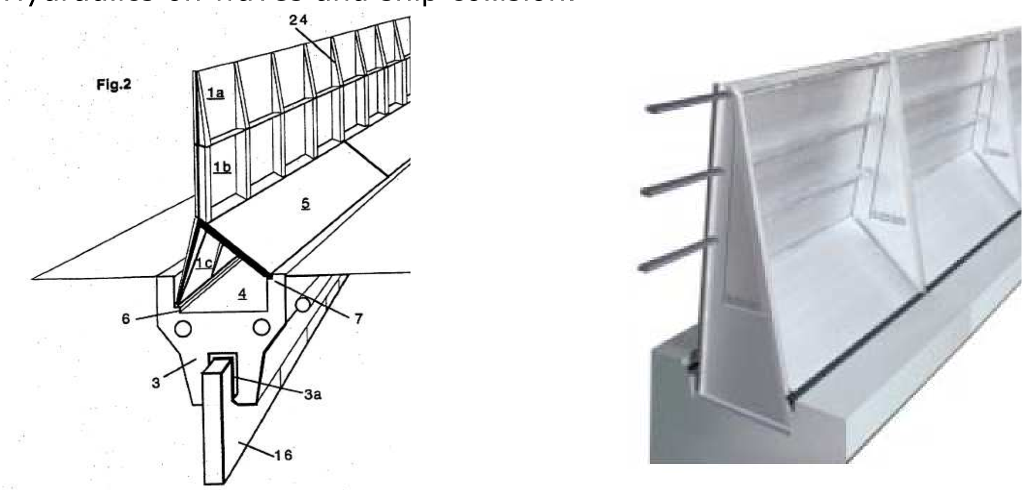


fig. 38 Dutchdam

The Dutchdam has similar stability considerations as the system wall; a strong foundation (e.g. a sheet pile wall) is indispensable for the system. The producer promotes the Dutchdam as being suitable for height increases to a certain limit. This does mean that the foundation must be strong enough to cope with the consequent load increase.

With respect to costs, more or less the same counts for the Dutchdam as for the system wall, which means that the system is relatively expensive. It is only attractive in a situation where there is not much room available and the esthetic properties of the solution are an important requirement, for instance in a historic city center along a river.

5.3 Traditional systems

5.3.1 Sandbags

The most well known traditional system is the sandbag dam. Sand bags have been in use for centuries and in The Netherlands still quite recently in 1995. The most important characteristic of a sandbag dam is that the deployment requires a lot of labor and therefore it has a relatively long deployment time despite the availability of countless types of filling equipment.

The advantage is that the materials (sand and bags) are easily obtainable and relatively inexpensive. Getting volunteers should not be a big problem either and deployment does not require specialist knowledge. Still, this alternative is only suitable for small scale application, mainly because the speed of deployment is too slow. Finally, it is not uncommon that the sand gets contaminated which will pose a problem when trying to dispose of it after use.


fig. 39 Sandbags

5.3.2 Big-bags

A modern variation on sandbags is the use of big-bags. These are bags of with a volume up to $2m^3$ that can be placed in a row as a wall of about 1m high. The bags are quite flexible and can be placed on rough terrain. A modification of the big-bag system is to attach five big-bags to one another with wooden frames. The system can be unfolded and filled on location.


fig. 40 Big-bags

Similar to traditional sandbags the costs of the used materials are relatively low. The advantage of big-bags compared to sandbags is the faster deployment, and the small team of personnel. It does however require some heavy equipment for deployment.

5.4 Concrete barrier

An example of a concrete barrier is the Waterfront-Block which looks like looks like the so-called "lego" blocks. It relies on the weight of the concrete for stability. The Waterfront-Block has rubber seals to prevent leakage. The structure is characterized by its inflexibility to connect to the surface it is placed on. It also requires a (paved) surface with a high bearing capacity. The blocks can be stacked to obtain greater heights but deployment does require some heavy equipment.


fig. 41 Waterfront-Block

5.5 Composite structures

5.5.1 Portadam and pallet barrier

There are several different composite structures on the market; an example is the Portadam that is generally used to create cofferdams. It consists of metal frames clad with impermeable fabric. The metal frames have to be placed on a paved surface or on a concrete foundation. Seepage can be decreased by increasing the sealing sheet length as shown in fig. 42. The extent to which this sheet is really capable of sealing to the subsoil merely by water pressure will be discussed in paragraph 10.3.

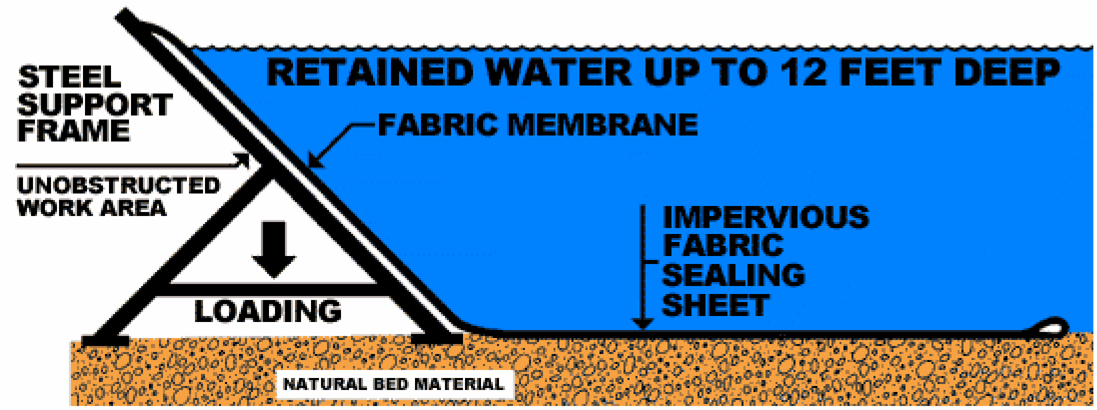


fig. 42 Cross-section of the Portadam system

A system similar to the Portadam is a barrier constructed of pallets with a synthetic sheet for impermeability. The pallets used are standard type that can be obtained anywhere. Financially, this is a major advantage compared to the Portadam. The pallet barrier is anchored through friction by a flange collar on bottom section for silt, sand, gravel, moraine and asphalt. It requires additional anchor pins for grass, clay and muddy subsoil.


fig. 43 Pallet barrier

The picture of fig. 43 shows sandbags on top of the structure, possibly to provide it with some extra weight in order to keep it stable. The Pallet Barrier uses an iron chain or a small sand-filled tube for the initial weight on the sealing sheet. A sealing sheet needs some means of initial loading to prevent it from floating when the water rises. Another option is to temporarily anchor the sheet to the pavement.

5.5.2 Rapidam and Ducoda

Two systems that function in the opposite direction are the Dutch Convertible Dam (Ducoda) and the Rapidam. These systems withstand the water with the hollow side and exists for the larger part of sealing sheets.

For both systems different elements can be zipped together to increase the length (see right picture of fig. 44). The seams between the sealing sheets are supposed to be pressed shut through the water pressure. Ducoda needs a shallow trench to be dug first (by tractor and a special type of equipment), where a piece of sealing sheet is installed. The other systems require some means of initial loading to prevent the sheet from floating when the water rises. Another option is to temporarily anchor the sheet to the pavement.

fig. 44 Dutch Convertible Dam (left), Rapidam and the watertight zip of the Rapidam (right)

5.6 Water filled systems

5.6.1 General

The water-filled systems (both tubes and blocks) have many forms but one basic principle. They are weight structures like big bags or concrete blocks that rely on their weight for stability. Systems of this type resist the horizontal water pressure by their weight and the consequent friction force with the foundation. For the tubes the problem of rolling away is solved by subtle adaptations of the design.

Tubes are flexible and can be applied on almost any terrain. Their height can be adapted flexibly by adding water. Apart from removing sharp objects, there are almost no preparations needed. Leakage of water through the construction itself is impossible.

5.6.2 Aqua Levee

The Aqua Levee is a system consisting of foldable triangular synthetic elements with a synthetic fabric bag that can be filled with water. The system is really a combination between a water-filled tube and blocks. It can be stacked like some block-type TFD. It can on the other hand also be deflated and will then use up very little space during storage. Unlike tube it has a hard shell on the outside that protects the tube from any punctures. This does however compromise the sealing to the subsoil that a tube has.

fig. 45 *Aqua Levee*

The units have sides are about 0.7m high. It can be heightened to be able to withstand up to 1m, but this will use up four times as much elements. The levee can be anchored to the subsurface. The advantage of this system is that it can be filled with water, which is available at the location. A disadvantage is that all elements have to be filled separately, which will cost a lot of time since they are quite small.

5.6.3 Aqua-Barrier and Aqua Dam

Two very similar water-filled tube systems are the Aqua-barrier and Aqua Dam. Both have already been extensively used as cofferdam and occasionally as emergency flood defense protecting private property. Both systems are available in different varieties to control over 3m of water and are available in lengths up to 60m.

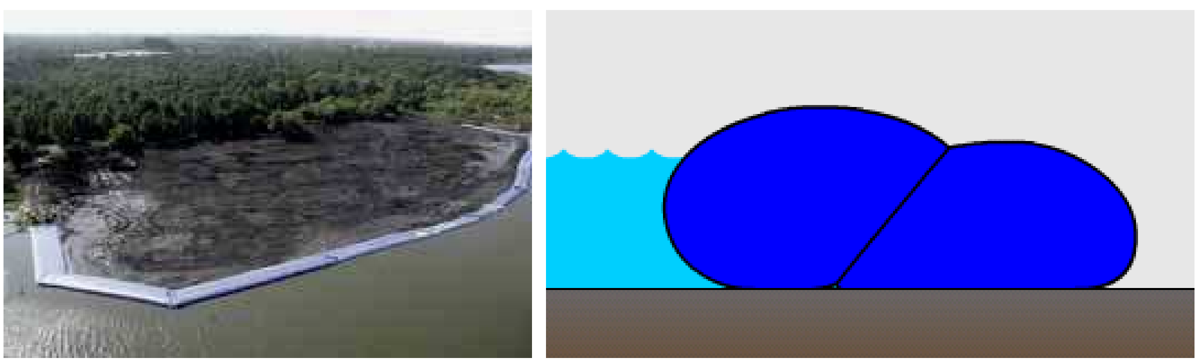


fig. 46 Aqua-Barrier in action (left) and cross-section with internal baffle system (right)

The Aqua-Barrier remains stable against rolling through its internal baffle system (fig. 46). The Aqua Dam consists of two basic parts: an outer tube holding the two inner tubes (fig. 47). When filled with water these inner tubes are in contact and provide the friction that keeps the system from rolling.

To achieve any necessary length each system uses a different technique. For the Aqua-Barrier, the different segments are connected simply by overlapping of the ends of the tubes. The Aqua Dam uses a coupling sleeve that is pulled over the ends of two adjacent tubes.

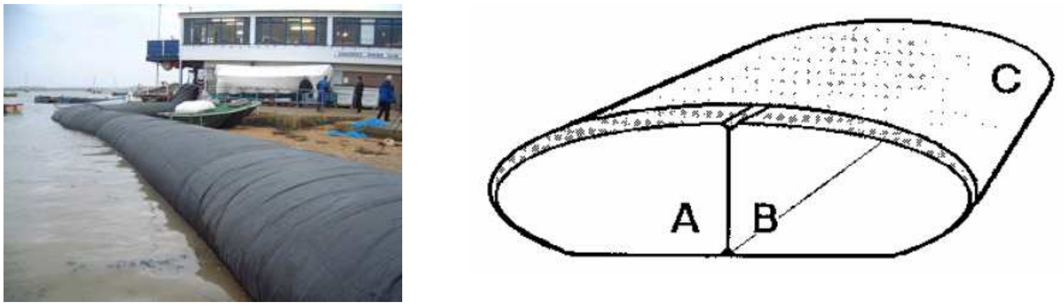


fig. 47 Aqua Dam in action (left) and cross-section with two inner tubes inside outer tube

5.6.4 Twin Flex Barrier

The Twin Flex Barrier ("Mobiele Waterdam" in Dutch) is a water-filled system very much like the Aqua Dam and Aqua Barrier but consists of two parallel tubes that are connected at one point. The Twin Flex Barrier has a coupling system that discerns it from the Aqua Barrier and Aqua Dam.

The system is available in lengths up to 250m and was invented particularly to be used as a temporary flood defense. In the design and method of application this has been taken into account to ensure a fast deployment. This will be further described in chapter 7. The barrier has been tested and demonstrated extensively in the field but has not yet had the opportunity to prove itself in an actual emergency situation.

fig. 48 Twin Flex Barrier and the coupling system

5.7 Aspects of comparison

As shown in this chapter, many types of temporary flood defense systems and all have different properties. This chapter described the general properties of number of systems. The next chapters will compare these systems on the following aspects:

1) Costs of purchase;
2) Deployment;
3) Stability;
4) Applicability;

Ad. 1) Costs of purchase
This first point needs only a brief explanation: the choice between different products always incorporates a financial consideration. A system is attractive when it offers the required safety for a relatively low price.

Ad. 2) Deployment
The nature of an emergency situation implies that the time available is limited. The deployment rate should therefore be fast, even during unfavorable conditions. Besides this, the system also needs to be transported to the location. Here the weight of the system is an important property.

Ad. 3 and 4) Stability and applicability
The stability of a TFD system determines the controlled water level. This depends also on the location the system is deployed (paved or unpaved, smooth or uneven, rough or slippery surface). The extent to which a certain system demands specific subsoil properties, determines its applicability. When a system is intended to be used for emergencies, the situation and location are partially uncertain. A system for this purpose should be flexible and have few specific demands.

A comparison on the purchase costs and deployment properties will be made in the chapters 6 and 7. In chapter 8 the stability and applicability of the TFD systems is discussed. The final conclusion will be drawn in chapter 9. The systems that are compared in these chapters are:

Traditional:
- Sandbags;
- Big bags;

Concrete:
- Waterfront-Block;

Composite:
- Pallet Barrier;
- Rapidam;

Water-filled:
- Aqua Levee;
- Twin Flex Barrier;
- AquaDam;
- Aqua Barrier.

The immobile systems are not further assessed because of their inflexibility to be applied at different locations. Only their costs are incorporated in the next chapter for the sake of comparison.

6 Costs

6.1 General

The costs are amongst the most important considerations in choosing between different products. The costs of a system can be distinguished in the constant and variable costs. The constant costs are mainly determined by the purchase costs. The variable costs are, among others, the costs of deployment and the costs of storage. This chapter deals with a comparison on the purchase costs followed by a comparison of the variable or "additional" costs.

6.2 Purchase

A complete cost comparison is difficult to make because available prices often lack specification. Still, a comparison of purchase costs was made and is displayed in fig. 49.

There is a clear difference between the immobile systems (System wall and Dutchdam) and the mobile systems. The costs of both immobile systems do not include the costs of a proper foundation, which means that in reality the difference would be even larger. The water-filled tube type systems are among the least costly systems. The systems can control different water heights (determined in chapter 8) so the final comparison has yet to be made. This will be done in chapter 9.

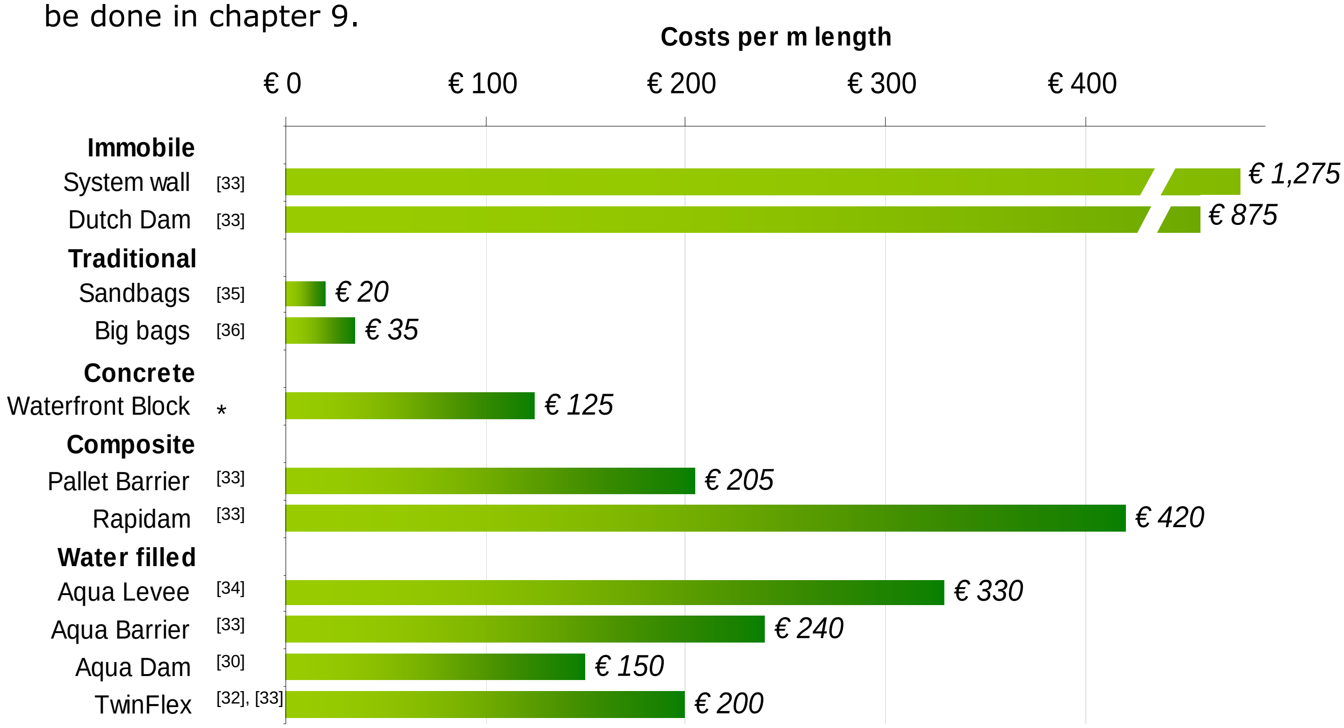


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fig. 49 Cost comparison for several systems (price level January 2007 ex. VAT or transport costs)

It is clear that the immobile systems are by far the most and the traditional systems are the least expensive. The water filled systems are in the same price range, with a price between approximately €100,- and €300,- per meter length. The pallet barrier that (according to the supplier) should have the advantage of being cost-efficient since it uses low-cost materials (like pallets), in fact does not seem to have a significant cost advantage over the water filled-tube systems. This is probably due to the fact that it requires a lot of additional supporting requisites ranging from steel support frames to plastic membranes and connection pieces.

The traditional systems are by far the least expensive. The difference between the Sandbags and Big bags can be explained by the fact that the considered sand bag dam has a height of only 0.4m. To construct a sandbag dam with the same height of a Big Bag dam, about twice the amount of sand is needed. This is the reason that for water levels larger than 0.4m, it is financially more attractive to use Big Bags instead of sandbags.

6.3 Additional Costs

Besides the costs of purchase, there are other costs that will have to be made that vary for each system. These are the costs of construction and deconstruction and the costs of storage of the system.

R.F.R van Dillen [4] made a cost estimate of different systems where he determined the costs, depending on the frequency of use. Among the systems compared were sandbags, big-bags and a water-filled tube type of TFD. The figure is shown below. The vertical axis has no scale because costs have merely a qualitative meaning. Especially when applied more frequently, a water-filled tube becomes relatively cheap, while a sandbag dam tends to be the most costly on the long term.

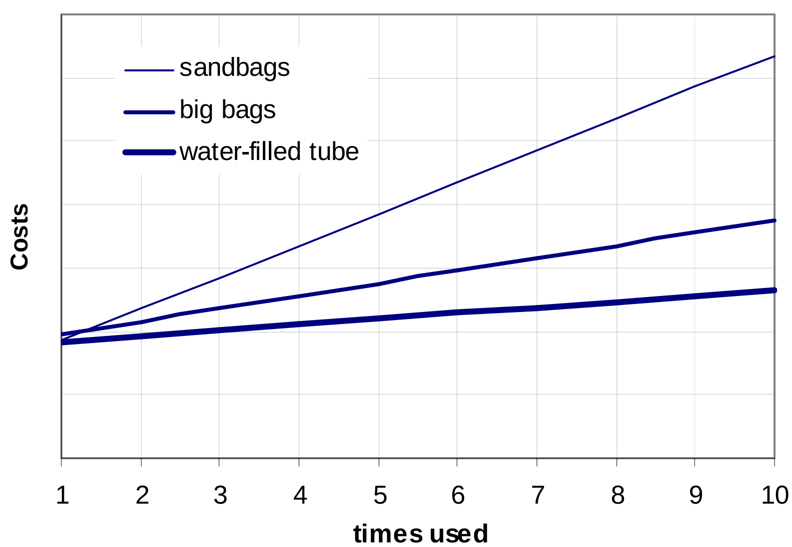


fig. 50 Relation between costs and frequency of use

It is however unclear whether the frequent use of a TFD that is intended for emergency situations with a low probability of occurrence, is realistic. It is very well possible that a TFD is used only once or not at all during its technical life-time. This means that the fixed costs (e.g. costs of acquisition) are of a greater significance than variable costs (costs made in case of emergency).

Finally, the lifetime of a system is also of importance when the long term costs are considered. There is little known about the lifetime of the different systems. Some producers have a certain warranty period for their products. For the Twin Flex Barrier this period is five years.

7 Deployment

7.1 General

The nature of an emergency situation demands for a system with properties that secure a quick deployment. The two most important indicators are deployment rate (i.e. the deployed length per hour) and system weight. The latter has a significant impact on the transport of the system from the storage to the location of deployment.

7.2 Deployment rate

The deployment rate of the different systems is displayed in table 6. Because the deployment rate for some part depends on the team size, this is also incorporated in the table.

The data was obtained from different sources. Information provided by suppliers is compared with data from more objective sources. Little is known about the actual circumstances in which test-deployment of the systems took place and how sensitive this placement is to extreme conditions.

System		team size	Deployment rate (m/hour)
Traditional			
Sandbags	[14]	30	17
Big-bags	[14]	4	25
Concrete			
Waterfront-Block	[14]	4 to 5	21
Composite			
Pallet Barrier	[14]	4	50
Pallet Barrier	[24]	2	113
Pallet Barrier	[33]	18	100
Rapidam	[33]	2	33
Water-filled			
Aqua Levee	[28]	3	120
Aqua Dam	[33]	2	33
Aqua Barrier	[33]	4	50
Twin Flex Barrier	[14], [32]	4 to 5	600 (to 1200)
Twin Flex Barrier	[33]	4	1000*

** Based on a test*

table 6 *Deployment speed for different systems*

The table shows that the sandbag dam has by far the smallest deployment rate and requires the largest team for deployment. Also striking is the big difference between the three different sources of information for the Pallet Barrier. It is assumed that the supplier data is too optimistic; this information is therefore dismissed from the comparison.

The image sketched shows a clear advantage of the Twin Flex Barrier. The three different sources show comparable results. Though the Aqua Barrier and Aqua Dam are in many ways similar to the Twin Flex Barrier, their deployment rate is more than a factor 10 smaller. This strange difference does raise questions about the validity of the data of the Twin Flex Barrier. Three arguments for the validity of the higher deployment rate are:

- The Twin Flex Barrier is supplied in lengths up to 250m while the Aqua Barrier and Aqua Dam systems are only supplied in lengths up to 60m. Adding shorter segments together is time-consuming.
- The Twin Flex Barrier uses a specialized coupling system connecting two tube segments together, while the other two systems use overlapping (Aqua Barrier) or a more labor-intensive coupling sleeve (Aqua Dam).
- The Twin Flex Barrier effectively uses equipment like a tractor with a spool on a wagon for deployment, while the deployment of other two systems is done mostly by hand.

fig. 51 Coupling system

These differences can be explained by the fact that the Twin Flex Barrier is designed specifically for the application as emergency flood defense, while the other two systems are primarily used to create cofferdams for which a short deployment time is of a lesser importance.

fig. 52 Deployment of the Twin Flex Barrier compared to the Aqua Dam

An important part of the deployment is the filling of the tubes. A twin tube system (with 125cm tubes) has a cross-sectional water volume of approximately $1.2m^3/m^1$ per tube. Each tube can be filled separately. Assuming the deployment rate of 600m/hour, each pump will need to have a capacity of $12m^3$ per minute, assuming that pumping can commence simultaneously with the unrolling of the tube. This is a capacity that is high but not unrealistic. This figure will be used in the final comparison of chapter 9.

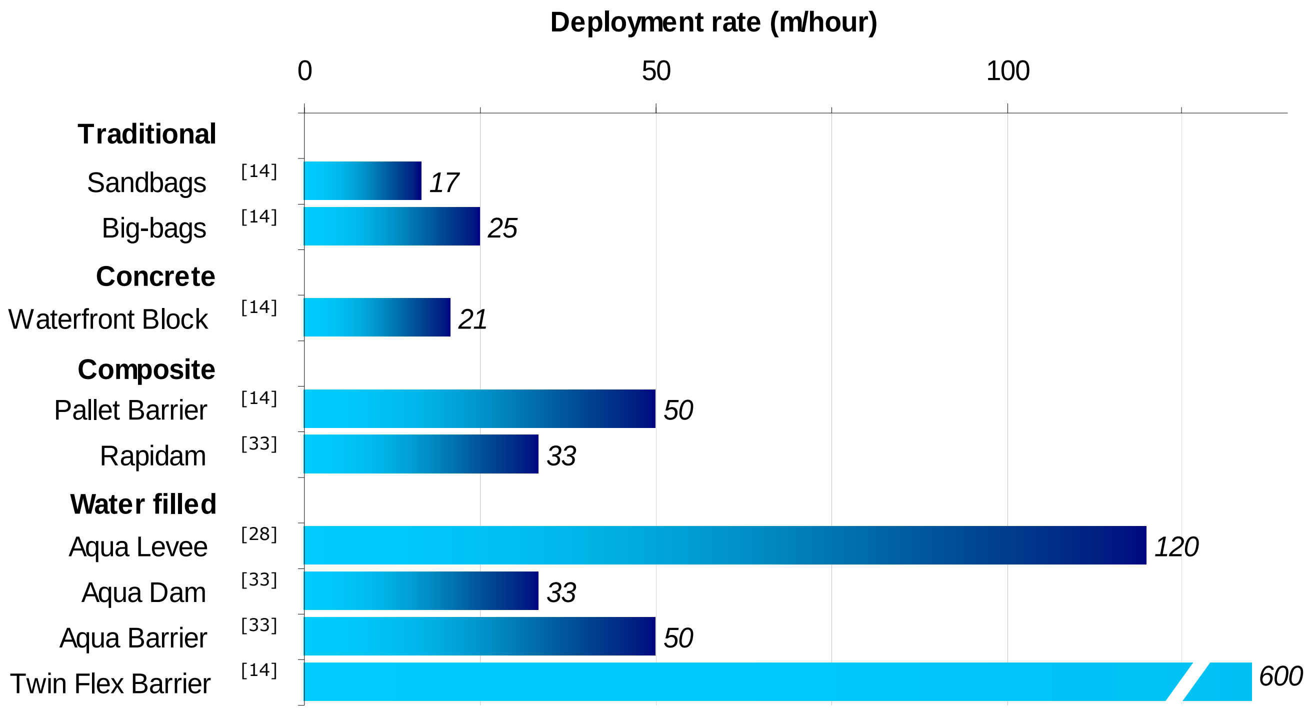


fig. 53 Deployment speed for different systems

7.3 System weight

Besides a high deployment rate a system will also require transport to the location. In important indicator for a fast transport is the weight of the system. There is a vast different in weight between the different systems. The chart of fig. 54 shows the weight per running meter on a logarithmic scale. It can be seen that there are roughly three weight classes based on the order of magnitude. The water filled systems all weigh about 10kg per running meter while for the big bags and water front block this is larger by two orders of magnitude. This will be taken into account when the final judgment on deployment is made in chapter 9.

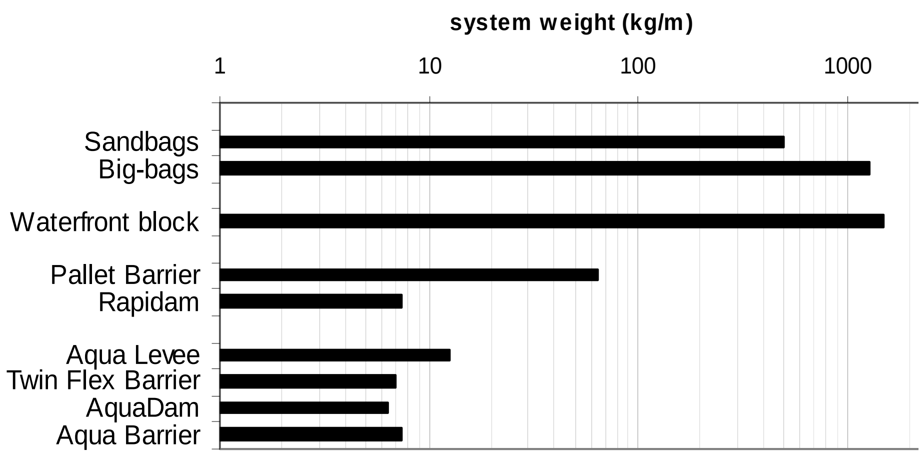


fig. 54 weight for different temporary flood defense systems

8 Stability

8.1 Failure mechanisms

This chapter looks at the stability of the systems as described in chapter 5. The problem of TFD stability is threefold (see fig. 55). The first problem is shear displacement which means the construction is horizontally displaced due to a horizontal force. The second stability problem is tipping over. The third is piping underneath the system caused by a flow water through the soil directly underneath the TFD. The more technical aspects of the calculations are further explicated in Appendix VI.

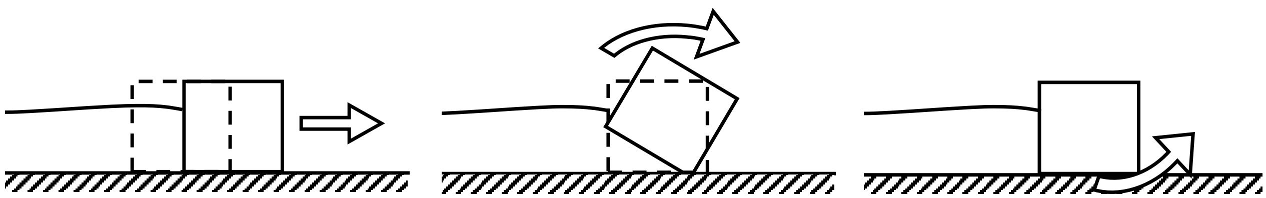


fig. 55 Three failure mechanisms: shear displacement, rotation and piping

Most structures rely on their own weight for stability but not the composite structures. For these systems, the presence of a sealing sheet or additional anchoring is of great importance for the stability since the weight of these structures is very low. The stability of the composite structures will therefore be separately determined.

8.2 Weight structures: shear and rotation

8.2.1 General

For both the rotation and the shear calculations it is important to distinguish the different forces acting on the mechanism. These forces are (see fig. 56):

- the weight of the system, W ;
- the horizontal hydrostatic force, $F_{w;h}$;
- and the vertical hydrostatic force, $F_{w;v}$.

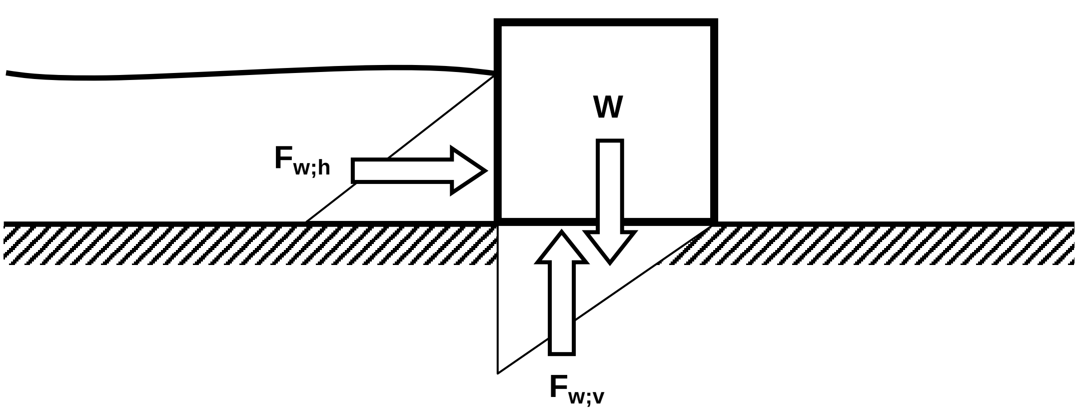


fig. 56 Pressure and acting forces on a temporary flood defense system

For the vertical hydrostatic force a linear water pressure distribution is assumed as shown in fig. 56. The validity of this assumption can be challenged since the horizontal hydrostatic force influences the stress distribution in the subsoil which results in a smaller contact width between system and subsoil. This effective contact width is displayed in fig. 57 (left), as well as the effect on the upward water pressure. It is obvious that in this case the upward water force is larger than for the assumed linearly distributed upward water pressure. This would seem to have a significant negative effect on the system stability. From calculations (Appendix VI) it can however be concluded that this effect is practically negligible.

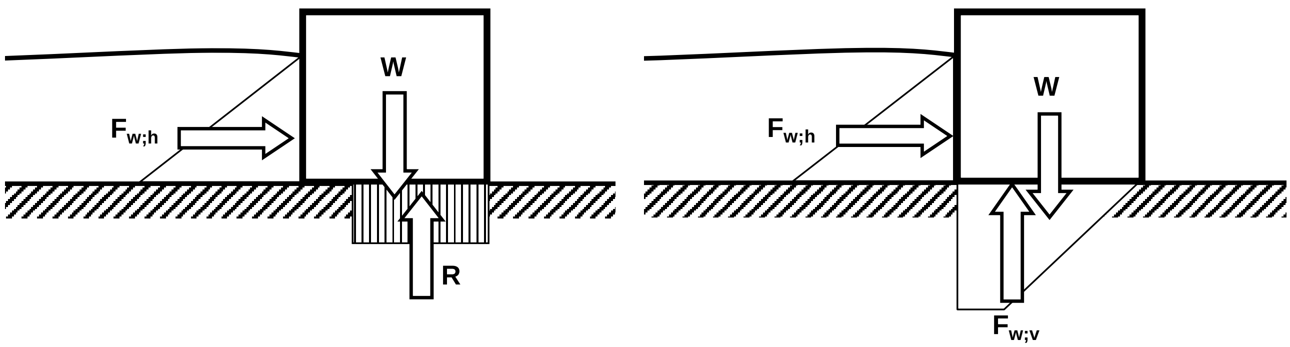


fig. 57 Effective contact width (left) and resulting upward water pressure diagram (right)

8.2.2 Shear

The factor of safety against shear is the ratio between the (resisting) friction force along the bottom of the system and the (driving) horizontal hydrostatic force:

$$FS = \frac{T}{F_{w;h}}$$

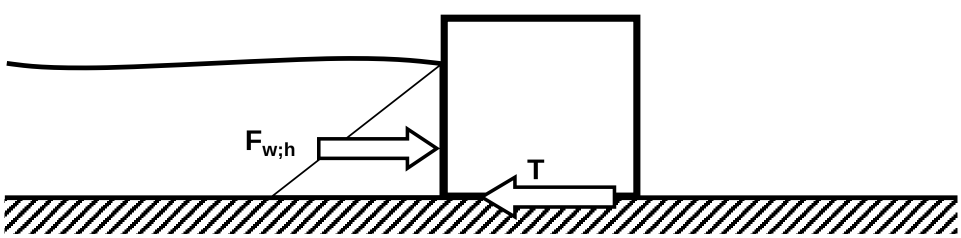


fig. 58 Shear: driving and resisting force

When the safety factor drops below FS=1, the structure becomes unstable. The friction force depends on the resultant of the system weight (W) and upward water force ($F_{w;v}$) and the shear coefficient. This shear coefficient depends on the material of the system and the foundation. For the calculations a shear coefficient of 0.25 is assumed for all systems.

The development of the factor of safety for shear of each system is displayed in fig. 59. The water-filled tube systems are among the more stable structures being able to withstand more than 80cm of water. This is a consequence of the weight of the filled tubes. The sandbag dam is not displayed in the figure but will remain stable even when water is at its crest at 0.4m. The Aqua Levee is the weakest structure since it can only withstand 33cm of water (less than 50% of its height).

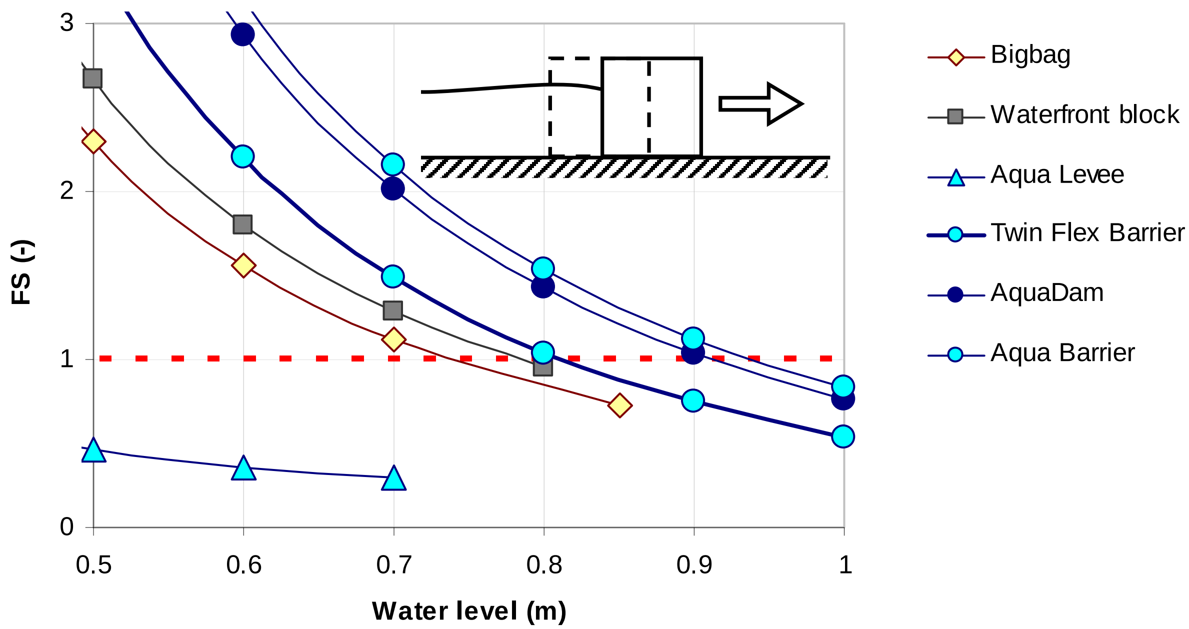


fig. 59 Safety against shear for different TFD systems

8.2.3 Rotation

In the calculation of the safety against rotation the rotation point is very important. Rigid structures (Waterfront-Block, big bag and Aqua Levee) can be assumed to rotate around a point at a distance of $^2/_3$ of their width.[8] Because the water filled tube systems are flexible the rotation point is a little harder to determine. It will probably lie somewhere between $^2/_3$ and 1 time its width. For a first impression of the rotational stability, $^2/_3$ (which is the most unfavorable situation) will be assumed. The rotational stability of a sandbag dam is not assessed since it consists of elements and tilting over is therefore not applicable.

The factor of safety against rotation is calculated by the ratio between the moment resisting and the moment driving rotational instability:

$$FS = \frac{driving\ moment}{resisting\ moment}$$

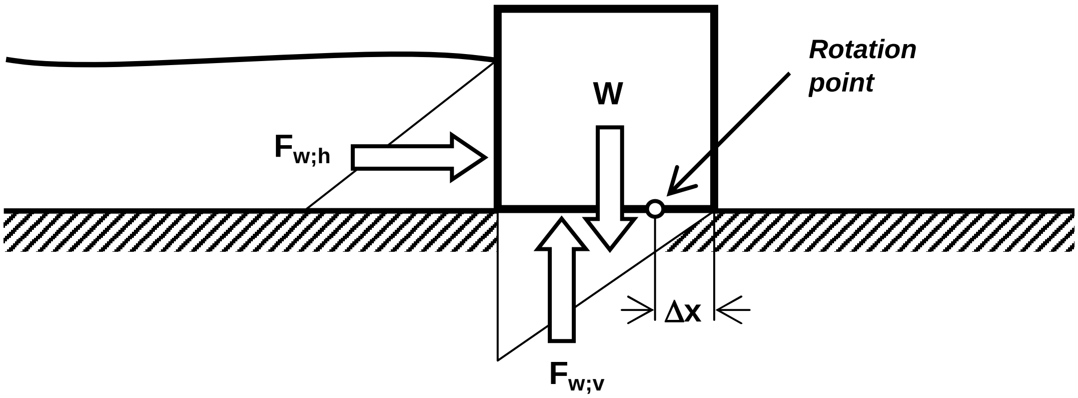


fig. 60 The rotation point in a of a temporary flood defense system

The stability of all systems with respect to rotation is somewhat better than their shear stability, which means that the structures will fail through shear and not rotation. The validity of the assumption of the rotation point for the water-filled tube systems is therefore not further examined.

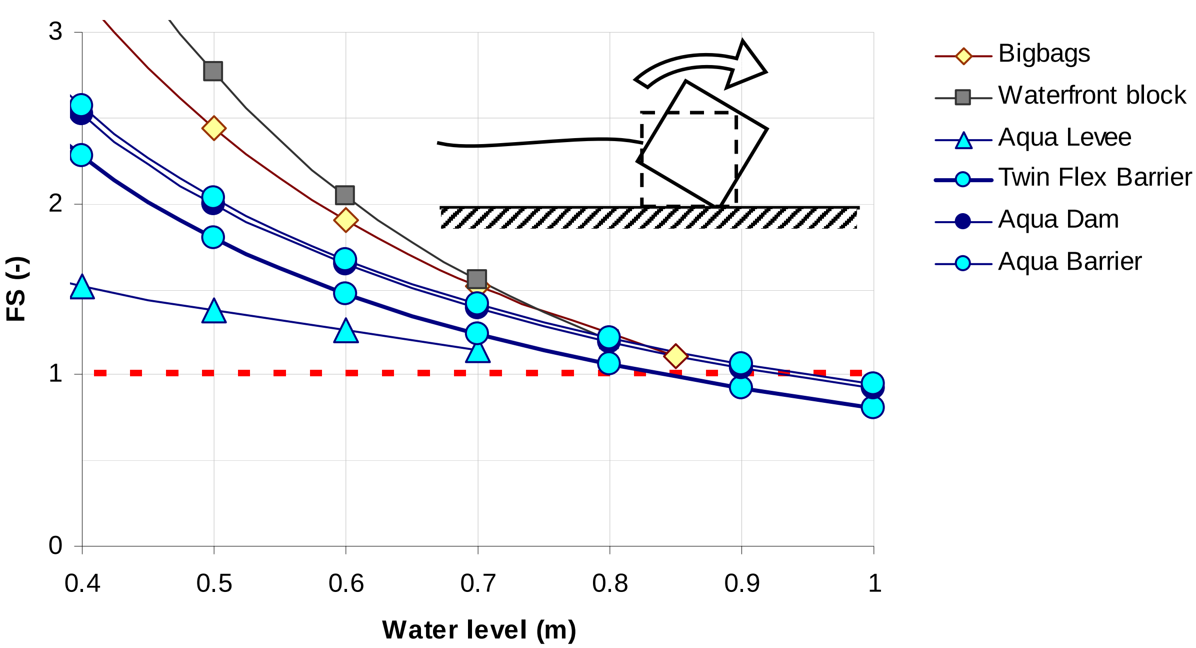


fig. 61 Safety against rotation for different TFD systems

In the previous calculation one aspect has been disregarded. This is the possibility of failure of the subsoil as a result of the stress caused by the system weight and the tilting of the system. To get an indication of the importance of this failure mode, the stability of the Waterfront-Block is considered. This system has the largest weight per running meter which is focused on a relatively small surface.

Failure of the subsoil can be discussed by means of the Brinch Hanssen method (see Appendix VI) which is generally used for horizontally and vertically loaded foundations. The calculations are made for sand, clayey sand, stiff clay, organic clay and peat. The results (fig. 62) show that even for a soil as soft as peat, the safety remains above 1.5. It can therefore be concluded that failure of the subsoil is of no concern for the discussed weight structures.

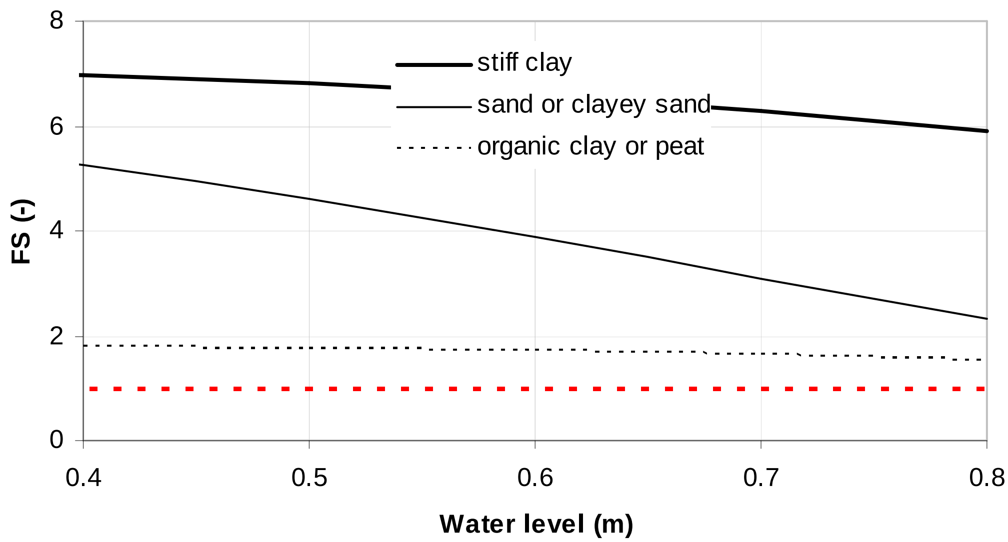


fig. 62 Safety against failure of the subsoil for the Waterfront-Block

8.3 Composite structures: shear and rotation

8.3.1 General

As described before, the composite systems differ from the other systems because they do not rely on their own weight for stability. Therefore, the calculation method in this paragraph also differs from the one used in the previous paragraph.

8.3.2 Pallet barrier

The pallet barrier system consists of pallets of 0.8 wide by 1.20m length supported by metal frames with a surface less than $0.25m^2$. These frames concentrate the force of the water on a small surface, leading to a relatively high stress in the subsoil.

Depending on the soil type, the subsoil can give way under the pressure as described in the previous paragraph. Though for the Waterfront-Block subsoil stability was determined to be sufficient, the small surface of the support frames is a concern. Therefore the subsoil stability will also be analyzed for the Pallet barrier.

The stability is again assessed with the method of Brinch Hansen (Appendix VI). The importance of the soil type for rotational stability of the barrier is displayed in fig. 63. For stiff soils like clean clay there seems to be no problem, but for softer clay, peat and also for sand the structure is highly instable.

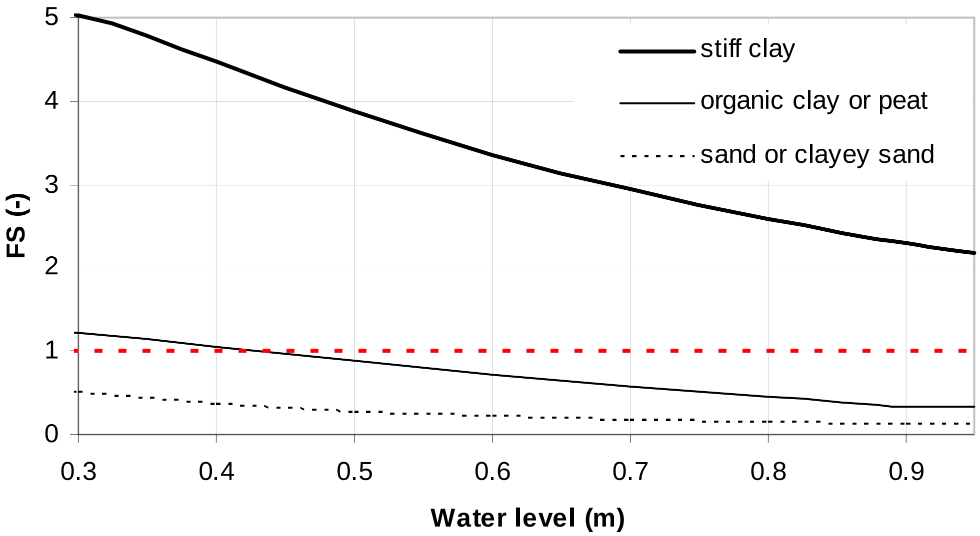


fig. 63 Pallet barrier: safety against rotation for different soil types

Compared to the Brinch-Hansen calculation made for the Waterfront-Block the results for the Pallet Barrier are much more pessimistic. This can be explained by the fact that the weight of the Waterfront-Block is distributed on a relatively large surface. This causes only a relatively small subsoil stress while the metal supports of the pallet barrier focus the force on a small surface leading to a relatively high subsoil stress.

For the shear stability calculation of the Pallet Barrier, the presence of the sealing sheet is of great importance. The calculation results in fig. 64 are based on different sheet lengths.

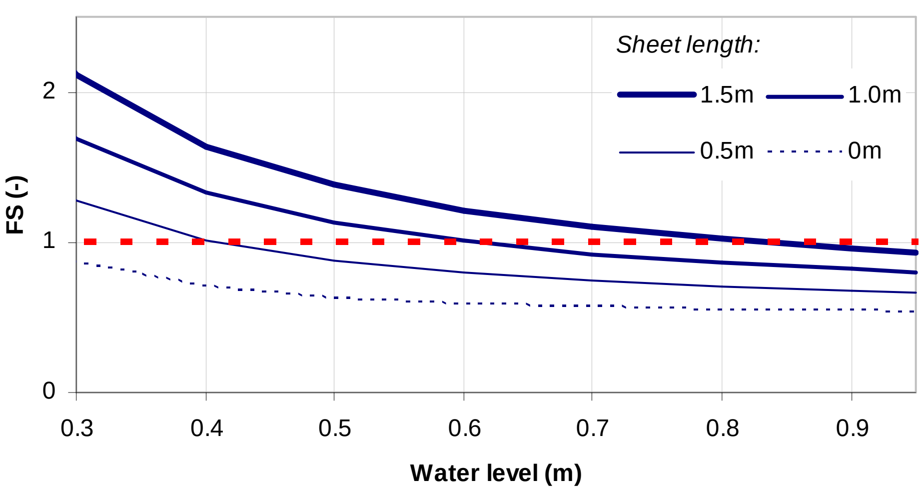


fig. 64 Pallet barrier: safety against shear for different sealing sheet lengths

Besides the point where each line crosses the FS=1 boundary, another interesting characteristic is visible. The slope of the graph is very flat. This means that the reliability of the structure is low. For instance, a pallet barrier with 1.0m of sealing sheet is supposed to fail at approximately a controlled water level of 0.6m. However, the safety factor for a water level of 0.5m is only slightly higher. In other words: a small negative deviation in the strength of the system (i.e. the sheet length) could lead to failure for much lower water levels.

It can be concluded that the stability of the Pallet Barrier is very poor on unstable subsoil and uncertain on a more stable (paved) surface. A solution could be to use obstacles at the location (e.g. the curb of a sidewalk) to prevent displacement. This however limits the applicability of the system to urban environments.

8.3.3 Rapidam

The Rapidam system (like Ducoda) consists almost entirely of a sealing sheet. It entirely relies on the shear strength resulting from the weight of the water. The system is sometimes anchored to the asphalt of an underlying road, a first indication that the shear strength is not always sufficient.

The calculation of the shear safety is different from that of the pallet barrier. The failure mechanism is actually a combination of shear and rotation. The horizontal and vertical force on the upright part of the barrier is transported to the bottom sheet. The force will locally pull up part of the bottom sheet and decrease the length along which a shear force can develop. The mechanism is displayed in fig. 65.

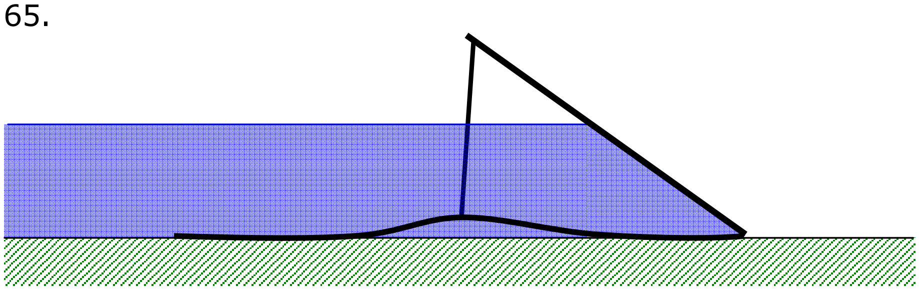


fig. 65 Pulling up of the bottom sheet of the Rapidam system

The described mechanism is hypothetical so the result should be considered as a rough indication of the stability of the system. The calculation depends partially on material properties like elasticity. A rough indicative calculation has been made in which material properties have however been neglected. The results are displayed in fig. 66, showing that the system can withstand even less than 0.5m of water.

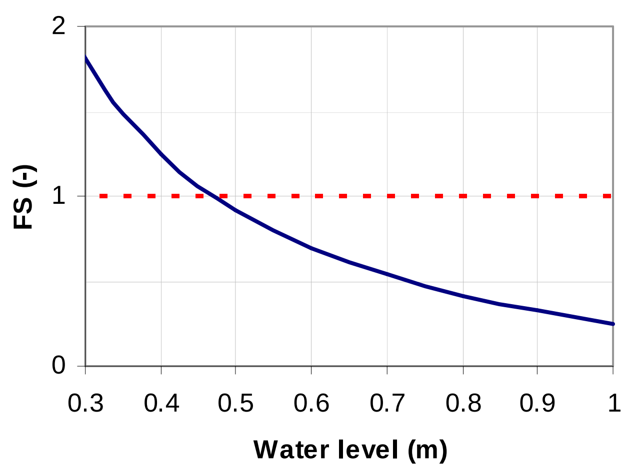


fig. 66 Rapidam: safety against shear

8.4 Piping

8.4.1 General

The third failure mechanism for temporary flood defense systems is piping. Seepage of water underneath a structure could lead to erosion similar to piping underneath a dike. Especially TFD systems with a relatively small width are vulnerable to piping because they have a small piping length.

The type of subsoil is of great importance. A TFD can be placed on a great range of surfaces ranging from clay to sand to a paved asphalt road. Clay has a very low permeability and piping will not be a problem if the TFD is placed on clay. While clay is very erosion resistant, sand is not. Fine grained soil types (e.g. fine sand) are more sensitive to erosion than coarse grained soil types like gravel.

8.4.2 Bligh and Sellmeijer

The mechanism can be assessed by both the model of Sellmeijer and the method of Bligh. Both determine the required piping length, depending on the subsoil. The difference between the two methods is that Bligh is based on the historical field data while Sellmeijer is based on numerical calculations and experimental data.

Another difference is that Sellmeijer incorporates many specific parameters (e.g. grain diameter, permeability, aquifer thickness, etc.), while Bligh uses a single coefficient, the creep coefficient (C_{creep}) which depends on the grain diameter.

Since the Sellmeijer equation was already discussed (see Appendix III), it will not be further explained here. Bligh was not discussed earlier so a brief explanation is appropriate. Bligh's criterion determines the required piping length (L_c) for a certain water level (ΔH_c):

$$\Delta H \leq \Delta H_c = \frac{L}{C_{creep}} \quad \text{or} \quad L_c \geq \Delta H_c \cdot C_{creep}$$

The influence of the subsoil is incorporated in the creep coefficient, which depends on the soil type (see table 7). Here, the creep coefficient actually represents the minimally required width of a TFD per m of controlled water height. For example: a system placed on very fine sand (C_{creep}=18), retaining 0.5m of water must have a minimal width of 0.5×18=9m.

Soil type	**Median diameter, d_{50} (μm)**	**C_{creep}**
very fine sand	105 – 150	18
moderately fine sand	150 – 210	15
very coarse sand	300 – 2000	12
fine gravel	2000 – 5600	9

table 7 Creep coefficient for different soil types

The results of the Bligh and Sellmeijer model differ in the sense that Sellmeijer is generally more optimistic than Bligh. There is also another difference which becomes visible when the ratio between L_c and Δh (the ratio between system width and controlled water level) is observed. For Bligh this ratio is actually the aforementioned creep coefficient. While for Bligh the width/height ratio is independent of the water level (it only depends on the soil type), for Sellmeijer it is not.

The development of the width/height ratio ($L_c/\Delta h$) with respect to the water level difference (Δh) is shown in fig. 67. The displayed range for both models is that between the results for very coarse (lower line) and very fine sand (upper line). For Bligh the width/height ratio varies between 12 and 18. As said earlier, the primary difference between the models is that Sellmeijer is more optimistic than Bligh.

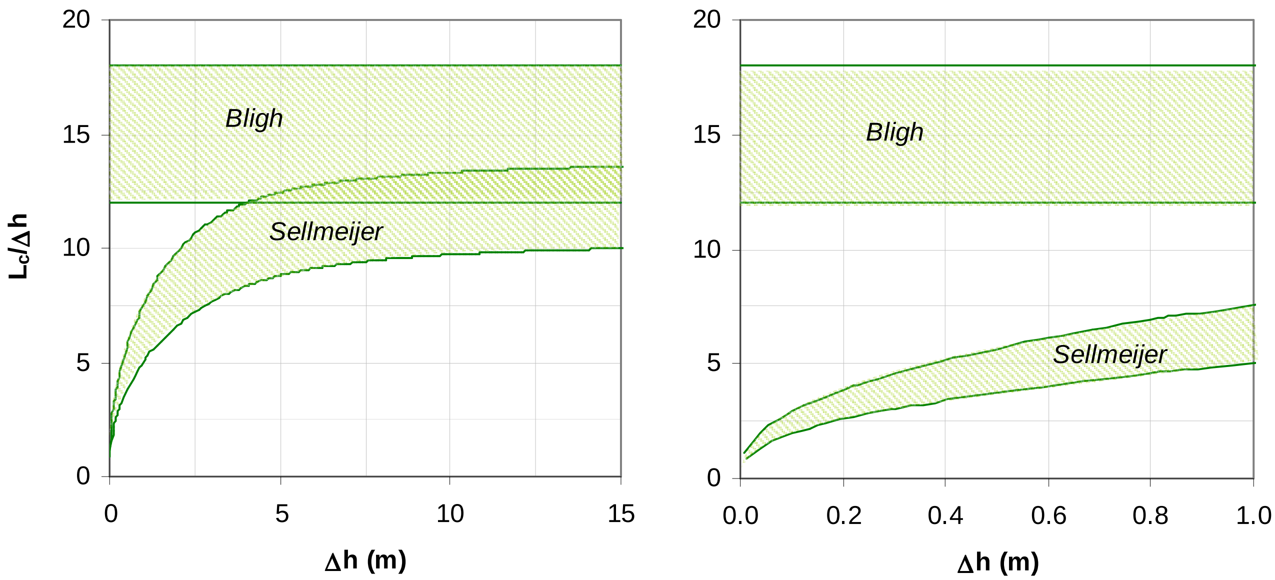


fig. 67 Development of $L_c/\Delta h$ with respect to Δh for Bligh and Sellmeijer

What is more important is that while the width/height ratio is constant for the Bligh method, for Sellmeijer it is positively dependent on the water level. For very fine sand and a water level of about 15m, the difference is still reasonably small with a ratio of 13.5 for Sellmeijer and 18 for Bligh.

However, when focusing on the area of interest (water levels up to 1m, displayed in fig. 67 right), the difference is impressive. While for Bligh the required ratio remains 18, for Sellmeijer it is much lower with a maximum of approximately 7.5. This will be further discussed in chapter 10, when the preliminary design will be made.

8.4.3 Performance of the TFD systems

The results of the discussion of the methods of Bligh and Sellmeijer show that it is difficult to make an unambiguous judgment on the performance of the systems with respect to piping. A comparison of the system geometry with the criteria set by both models can help to give an indication of this performance.

The four diagonal lines in fig. 68 display the minimally necessary system width (L_c) with respect controlled water height (Δh), or the other way around: the maximally controlled water height for a system of a certain width. This has been displayed for both Bligh and Selmeijer and for very coarse as well as very fine sand.

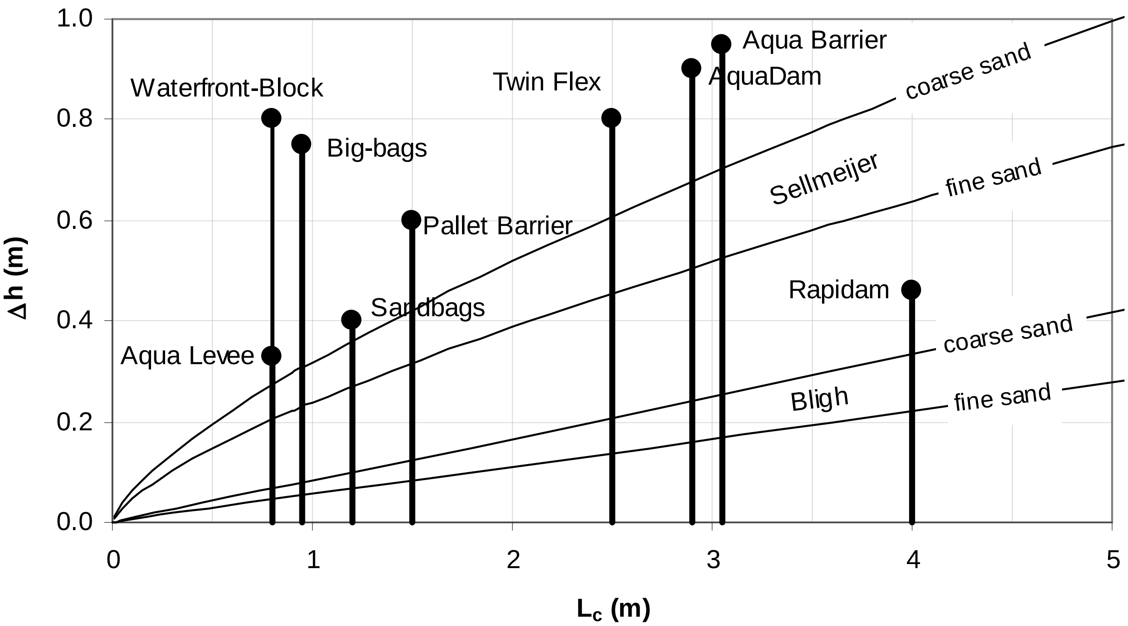


fig. 68 Performance of several TFD systems with respect to piping

The TFD systems are incorporated in the figure by thick vertical lines that display both the system width and controlled water level. The performance of each TFD system can be determined by its position with respect to these lines. To fulfill the requirements of the piping criteria (of Bligh or Sellmeijer), the line representing a system should be below the diagonal lines. As an example the performance of the Twin Flex Barrier is explained below.

Example
A Twin Flex Barrier has a width of 2.5m and can control approximately 0.8m of water. Even when placed on very coarse sand, the width of the barrier is insufficient. When Sellmeijer is considered, the thick line representing the system should be displaced horizontally or vertically until it is below the Sellmeijer line for coarse sand. When the line is shifted to the right (keeping the controlled water level Δh constant), it appears that the system width should be at least 3.7m. When the line is shifted downward (keeping the system width L_c constant), the maximally controlled water height appears to be 0.6m. For fine sand the shift should be even larger. When the Bligh criterion is considered even more so.

Though the difference between Bligh and Sellmeijer is quite large, they both are pessimistic about the problem of piping for the discussed systems. From the figure it can be concluded that all systems fail to satisfy Bligh's criterion. The performance with respect to Sellmeijer is better but still insufficient.

For the water-filled tubes and the sandbag dam the ratio between width and controlled water height is about 3. For a big bag or concrete barrier this ratio is approximately 1. The Rapidam has the best result due to its relatively large ratio (almost 9). This ratio is the consequence of the 4m of sealing sheet, incorporated in the design. Since the effect and reliability of a sealing sheet is uncertain, more about this is discussed in the next paragraph.

It must finally be added that the calculations are based on the assumption the system is placed on permeable sandy subsoil which is actually the most unfavorable conditions. But even when the system is placed on sand, there will often be a top layer of humus present. This layer reduces the permeability of the soil and thus the piping problem. The extent to which this influences the results is not further discussed here.

9 TFD Comparison on Aspects

9.1 General

In this chapter a final comparison is made on the performance of the different TFD systems. The immobile systems have been omitted from this comparison since they are too different in nature and lack the flexibility that is required for the intended purpose. The specific system properties are incorporated in Appendix V.

The systems are compared on the four aspects:

- stability;
- costs of purchase;
- deployment;
- applicability.

Of these aspects stability and applicability are strongly related as are stability and costs. The score on stability is therefore incorporated in the scores for both costs and applicability meaning that it will be valued indirectly (see paragraph 9.2, 9.3 and 9.5). The ways in which scores are allocated to the different properties are incorporated in Appendix VI.

9.2 Stability

The controlled water level results from the calculations of chapter 8. The controlled water level differs for each system, but this does not mean that the system that can withstand the highest water level is per definition the best. The performance of the systems with respect to stability is therefore linked to the costs of purchase (see paragraph 9.3).

Systems	**Controlled water level** ΔH_w *(m)*	**Seepage** $B/\Delta H_w$ *(-)*
Traditional		
Sandbags	0.40	3.0
Big-bags	0.75	1.3
Concrete		
Waterfront block	0.80	1.0
Composite		
Pallet Barrier	0.60	2.5
Rapidam	0.46	8.7
Water-filled		
Aqua Levee	0.33	3.0
Twin Flex Barrier	0.80	3.1
AquaDam	0.90	3.2
Aqua Barrier	0.95	3.2

table 8 Score on stability for different TFD systems

The maximum water level for which each system loses its stability is displayed in table 8. The ratio between controlled water level and system width (B) is also incorporated in the table and will be valued under *applicability* (paragraph 9.5).

9.3 Costs of purchase

For the sake of comparison the costs of purchase are linked to the controlled water level. This is done by dividing the costs of each system by this water level. In this way, an expensive system that can control a relatively high water level can be compared with a less expensive system that can withstand only a low water level.

The equivalent costs for the Aqua Levee and Rapidam are now much higher, while those of the traditional systems remain low.

Systems	**Purchase costs** *(per m length)*	**Equivalent costs purch. costs/ΔH_w** *(per m length)*
Traditional		
Sandbags	€ 20	€ 50
Big-bags	€ 35	€ 47
Concrete		
Waterfront block	€ 125	€ 156
Composite		
Pallet Barrier	€ 205	€ 342
Rapidam	€ 420	€ 913
Water-filled		
Aqua Levee	€ 330	€ 1 000
Twin Flex Barrier	€ 200	€ 250
AquaDam	€ 150	€ 167
Aqua Barrier	€ 240	€ 253

table 9 Purchase costs and equivalent costs for different TFD systems

9.4 Deployment

9.4.1 General

Judging the deployment properties of a system depends on more than just the speed of deployment. The aspects and scores that are considered to be of importance for a complete comparison are incorporated in table 10. To be able to make a comparison, the performances of the systems are valued according to different categories.

Systems	Deployment rate DR$_t$ (m/hour)	Team characteristics Team size	Profes-sionality of team	Use of Special Equipment	Logistic feasibility transported weight $W_t / \Delta H_w$ (kg/m)	Relative score
	50%	20%			30%	
Traditional						
Sandbags	17	30	N	N	1250	13%
Big-bags	25	4	N	Y	1733	13%
Concrete						
Waterfront block	21	4	N	Y	1875	13%
Composite						
Pallet Barrier	50	4	Y	N	108	26%
Rapidam	33	2	Y	N	16	56%
Water-filled						
Aqua Levee	120	3	Y	Y	38	59%
Twin Flex Barrier	600	4	Y	Y	9	87%
AquaDam	33	2	Y	Y	7	49%
Aqua Barrier	50	4	Y	Y	8	49%

table 10 Score on deployment for different TFD systems

The relative score is the ratio between obtained points and the maximum obtainable points. The points per aspect have been distributed in such a way that the total score is determined for 50% by the deployment rate, for 30% by the logistic feasibility (system weight) and for 20% by the team characteristics.

9.4.2 Deployment rate

The deployment rate is a very important aspect if emergency situations are to be dealt with effectively. The scores on deployment speed are allocated accordingly by highly valuing systems with high deployment rates. Though optimal team sizes differ per system, the differences are not that large that this requires some sort of compensation (e.g. an individual deployment rate). Only the team size for the sandbag dam differs significantly, but the deployment rate for this system is already small so additional compensation is deemed unnecessary. Furthermore, the team size is valued in the next paragraph.

9.4.3 Team characteristics

The team characteristics are represented by three aspects. The first being the average team size required for deployment. Getting enough personnel for deployment could become a problem when the system requires a team with a size of several tens of laborers (sandbag dam).

On the other hand, if this workforce requires no specific professional knowledge about the system, obtaining the necessary manpower is less of a problem. Examples are again the sandbag dam but also the big bag and Waterfront-Block systems, since any local contractor can be mobilized to do the job.

Finally, for some systems the use of special equipment is essential. This varies from the use of lift-trucks for the placement of big bags or concrete blocks to use of pumps for the water-filled systems. Here, systems that can be placed without the use of additional equipment have a higher score.

9.4.4 Logistic feasibility

This last aspect to characterize deployment is based on the weight that needs to be transported to be able to control the specific water level. The weight of each system per m length is therefore divided by the controlled water level as was done before to determine the equivalent costs.

Here, the important advantage of the water-filled structures with respect to the other weight structures becomes visible. The weight of sandbags, big bags or concrete blocks will put a lot of strain on the logistic process, since they need to be transported from the storage facility to the site of application. The water filled systems have the advantage that they are in fact already partially at location since the water that is needed to give them their weight is abundantly available at site.

9.5 Applicability

This aspect was not earlier described in a specific chapter but the applicability of the systems is roughly defined by the ability or inability to apply a system at any random location and is therefore closely linked to the stability issues described in chapter 8.

The aspects that are considered here are dependent on the subsoil requirements. These can be discerned in the extent to which a system:

- is sensitive to elevations in the surface it is placed on;
- needs a subsoil with a high bearing capacity;
- is susceptible to piping.

For example, the Waterfront-Block is a system that requires a smooth surface for seamless placement. Elevations will cause leakage through and under the system. Though it was discussed earlier that the stability of the Waterfront-Block on softer soils is good (chapter 8), settlement of the system is inevitable. While settlement itself does not have to be a problem when stability is concerned, uneven settlement will cause elements to shift with respect to each other resulting in leakage.

Therefore, the Waterfront-Block is deemed unsuitable for placement on softer soils. In addition to this the performance of the Waterfront-Block on seepage is also quite poor since it has a small $B/\Delta H$ ratio. In fact this means that the Waterfront-Block can only be placed on little else than an impermeable paved (asphalt or concrete) surface.

In contrast to the Waterfront-block, water filled tube systems have only few specific subsoil requirements and have a relatively large width compared to many other systems.

Systems	Subsoil requirements *surface elevations*	*bearing capacity*	*Seepage B/DH $_w$ (-)*	**Relative score**
Traditional				
Sandbags	no specific requirements	no specific requirements	3.0	83%
Big-bags	no specific requirements	no specific requirements	1.3	67%
Concrete				
Waterfront block	smooth	only stiff subsoil (paved)	1.0	0%
Composite				
Pallet Barrier	smooth	only stiff subsoil (paved)	2.5	0%
Rapidam	smooth	no specific requirements	8.7	67%
Water-filled				
Aqua Levee	smooth	no specific requirements	3.0	50%
Twin Flex Barrier	no specific requirements	no specific requirements	3.1	83%
AquaDam	no specific requirements	no specific requirements	3.2	83%
Aqua Barrier	no specific requirements	no specific requirements	3.2	83%

table 11 Score on applicability for different TFD systems

9.6 Conclusion on temporary flood defense systems

The overall picture of the scores of the different system is displayed in fig. 69. With respect to applicability, especially the water-filled tube systems have a high score as does the traditional sandbag dam and to a lesser extent the Big-bag and Rapidam. Only two systems score very low, namely the Waterfront-Block and the Pallet Barrier, stemming from the fact that they have several specific subsoil requirements.

fig. 69 Total score

With respect to deployment the low score for the sandbag, Big-bag and Waterfront-Block is striking. Though the Big-bag has is the least expensive system, its score on deployment is too low to consider it to be an option for a professional approach of emergency situations. This is a consequence of the low deployment rate and the relatively large weight that has to be transported.

The Twin Flex Barrier does have the properties required to satisfy the demands. The Twin Flex Barrier is the only system that has a good score on deployment and applicability and has a relatively low price. The Twin Flex Barrier incorporates the properties that make it the most promising system. These properties are its superior deployment rate, low weight and its flexibility to be applied at practically any random location.

The Water-filled tube design based on the Twin Flex Barrier is further elaborated in the next chapter where some additional modifications are discussed to increase its performance on applicability and stability.

10 Water-filled TFD design

10.1 General

In chapter 9 the water filled Twin Flex Barrier was chosen as the most promising system to be applied as an emergency measure to temporarily increase the strength of a dike. In this chapter the design of the water-filled tube TFD will be refined to improve its performance.

10.2 Shape

The first possible adaptation of the water-filled tube design can be done by changing the shape of the system. To get an idea of the effect on stability of different shapes, three standard geometrical shapes other than the circular tube are compared to the Twin Flex Barrier (see fig. 70).

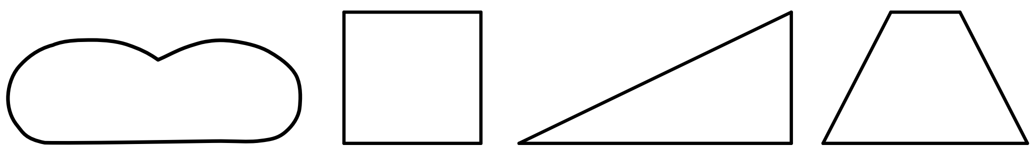


fig. 70 The Twin Flex Barrier, square, triangular and trapezoidal tube

The displayed tubes have the same volume per running meter as the Twin Flex Barrier. The square, triangle and trapezoid all have a height of 1.57m while the Twin Flex has a height of approximately 1.2m. The width of the Twin Flex barrier is 2.5m, for the square tube it is 1.57m and for the triangular tube 3.13. The width of the trapezoidal tube is 2.35 at its base and 0.78 at its top. For the calculations, the practical difference between the shapes is the variation between upward and downward water pressure as shown in the figure below.

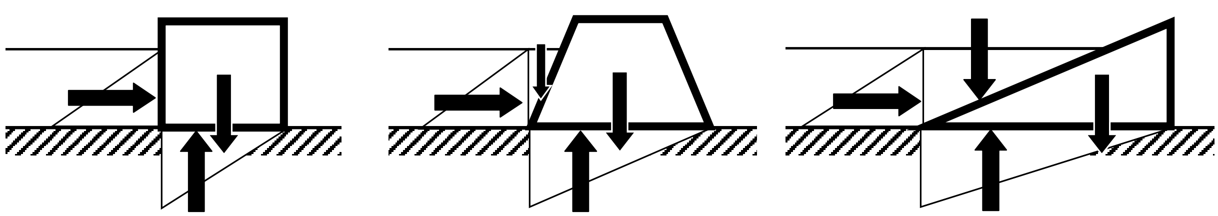


fig. 71 Different water-filled TFD shapes

Like before, the shear coefficient used in the calculations is 0.25 and the rotation point is assumed to be at 2/3 of the width. The safety against shear for the different types is displayed in fig. 72.

With respect to shear stability it is clearly visible that the differences are very small and none of the systems can safely withstand more than roughly 0.90m of water. Here, especially the point where FS=1 is of importance because the system is then about to become unstable. All tube shapes fail within a water level range of only 12cm (between 80cm and 92cm).

The shear stability of the square, triangular and trapezoidal “tubes” is somewhat better than the original Twin Flex Barrier. Though the stability of the triangular tube is the best, the expected positive effect of the extra downward water pressure is for a large part undone by the increased upward water pressure which is a consequence of the increased system width. This also explains why the stability of the trapezoid is somewhat worse than that of the square.

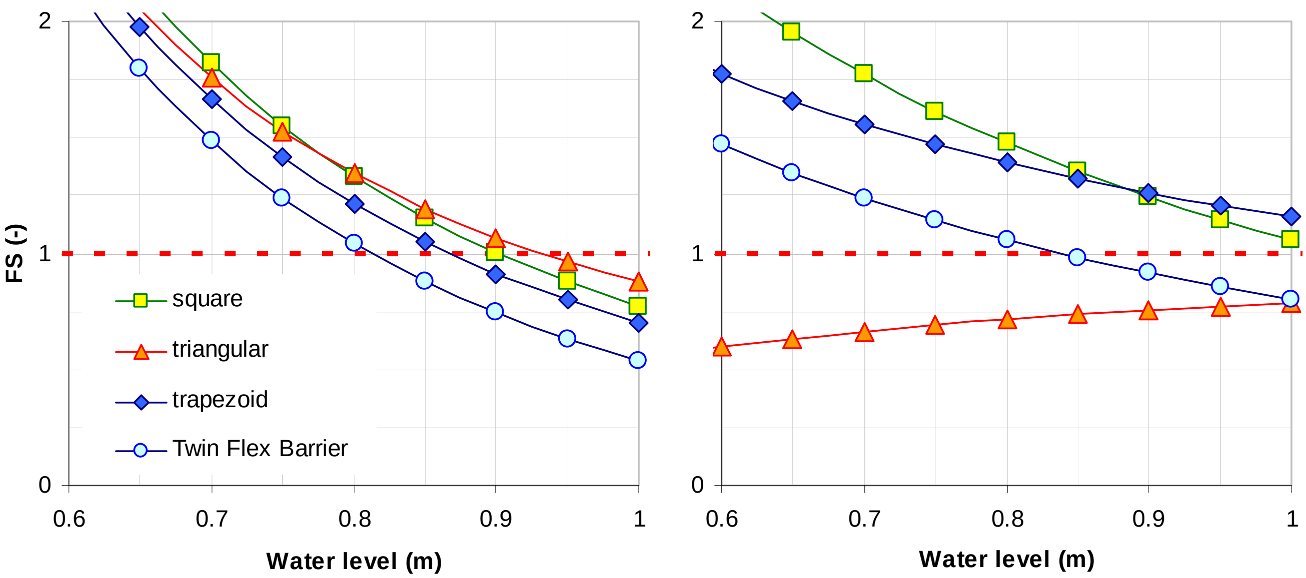


fig. 72 Safety against shear (left) and rotation (right) related to water height

The image of the rotational stability is less obvious. Generally, a system with a large width compared to its height would be more stable than a slender structure. In this situation however, the upward water pressure acts as an additional destabilizing force. This upward water force is larger for systems with a larger width and thus, the positive effect of a large width is partially undone.

This effect is especially visible for the triangular tube. The safety against rotation actually increases for an increasing water level. The reason for this is the fact that for a higher water level the stabilizing downward water force gains importance relative to the upward water force. Still, the triangular tube remains unstable with respect to rotation.

Another cause of the instability of the triangle is that a large part of its weight is located around the rotation point. In the calculation, the rotation point was assumed at 2/3 of the system width, which is actually the point where the force of the system weight takes effect. The moment arm of the system weight is then nil and the stabilizing influence of the system weight negligible.

When the overall stability of the systems is considered, it becomes clear that the square tube is the most stable, followed by the trapezoid and the Twin Flex Barrier. The differences are however small. While the Twin Flex can maximally withstand 80cm of water, for the square tube this is only 10cm more.

It can therefore be concluded that a variation in shape has only a marginal effect on stability. Therefore the aim should be to use the simplest (least costly) shape for the ultimate design. In the following paragraphs the original Twin Flex design will be used as a basis. If a higher water level has to be withstood, the TFD should either be larger or some special adaptations will have to be made.

10.3 Sealing sheet

10.3.1 Piping length

Though the water-filled tube systems do not score exceptionally low with respect to piping, the applicability would benefit from a modification of the design to further decrease the problem of piping. A possibility is the use of a sealing sheet comparable to the Rapidam or Ducoda system. When the Twin Flex Barrier is fitted with a sealing sheet, the piping length is increased with the length of the sheet.

The minimally required piping length depends on the type of subsoil but even more on the method (Bligh or Sellmeijer) used to analyze the mechanism, as was discussed in chapter 8. The difference between both of the methods is considerable. Bligh requires a system to have a width of at least 18 times the controlled water level while this ratio is only 7.5 for Sellmeijer.

In chapter 8 it was shown that the Twin Flex Barrier can maximally withstand a water level of 0.8m under unfavorable conditions. For this water level the required system width varies between 6m (Sellmeijer) and 14.4m (Bligh). Taking into account that the Barrier itself has a width of 2.5m, the sealing sheet length will need to be 3.5 to 12m to make the system "piping-proof."

This range is too large to be able too make a sound proposal for the ultimate design. It is however recommendable that the 12m sheet length is used for a reliable preliminary design.

10.3.2 Effect on shear stability

The sealing sheet also has a positive effect on shear stability because it increases the systems shear strength. In fig. 73 the stability-effect of a sealing sheet of different lengths is displayed with respect to the original Twin Flex Barrier.

The system fitted with the relatively short 3.5m sheet can withstand a water level of 0.8m with a safety of already more than 2, which implies that probability of failure of the system through shear is very small. The shear stability for the longer sheets is even better.

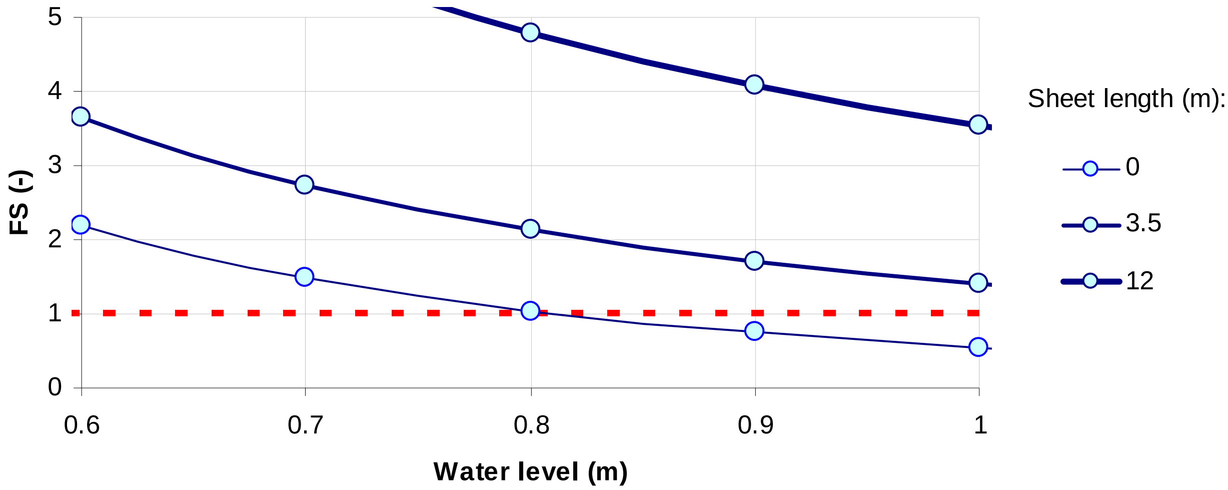


fig. 73 Effect of a sealing sheet on the safety against shear

10.3.3 Reliability

In the previous paragraphs the presence of a sealing sheet proved to have a significant positive effect on piping and shear stability. The calculations where made assuming perfect sealing to the subsoil through the water pressure, providing additional friction with the surface and an increased piping length.

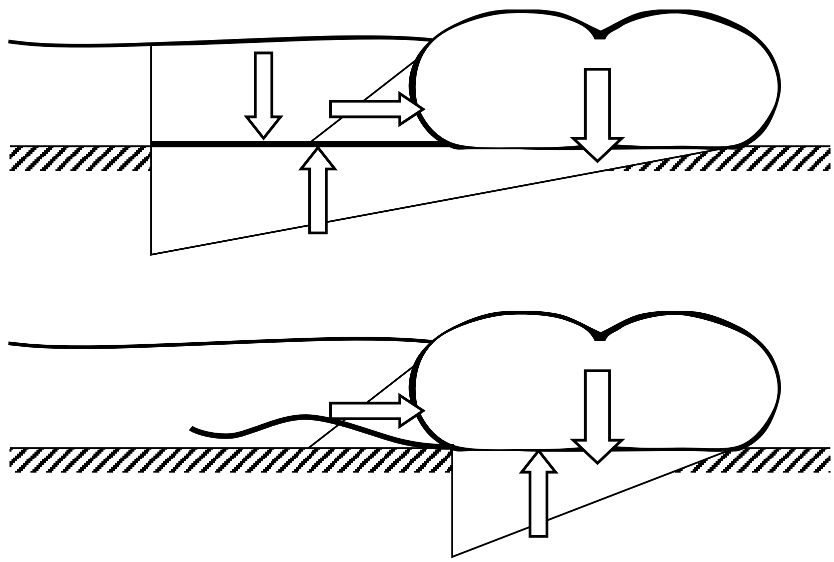


fig. 74 The (in)effectiveness of a sealing sheet

However, the assumption of perfect sealing is disputable. A synthetic sheet has a density that is not much more than that of water and therefore requires some initial loading to secure sealing to the surface. If the sealing is compromised, the sheet is likely to start floating in the water, which will mean that the effect of the sheet is completely undone. To overcome this problem, the various composite systems that use a sealing sheet all make use of methods to fix the end of the sheet to the surface, for instance by digging in or loading with sandbags.

Even with a method to fix the sheet to the surface, the water only needs a small opening to find a way underneath the sheet. When this occurs, the water pressure underneath will equal the downward pressure. The upward water pressure will eventually cover the full width of the sheet. The positive effect on shear stability and the increased piping length will then be undone.

Intermezzo
The NOAQ system is a system that consists of a single air-filled tube and a sealing sheet partially incorporating a drainage layer. The system was on the market until the manufacturer went bankrupt some five years ago. It relied entirely on the theoretical effectiveness of the sealing sheet as was proven by calculations and demonstrated through field tests executed by the supplier. Several Dutch Water Boards also tested the system but when the water level increased, it floated away. The theoretical effect of the sealing sheet proved not always to work in practice.

The sealing sheet can work but its effectiveness is very much dependent on its sealing to the subsoil. The apparent unreliability of this sealing is reason for concern. It is therefore recommended to further investigate the effect of the sealing sheet with respect to piping and sheer stability. This can be done by testing different types of sheet material and different types of initial loading of the sheet edge.

10.4 Preliminary TFD design

The comparison of the previous chapter led to the conclusion that the water-filled tube TFD is a promising system for use during emergency situations. In this chapter one modification was made to prevent piping. The preliminary design of the system is depicted in the figure below. This system is able to withstand at least 0.8m of water even under the most unfavorable conditions.

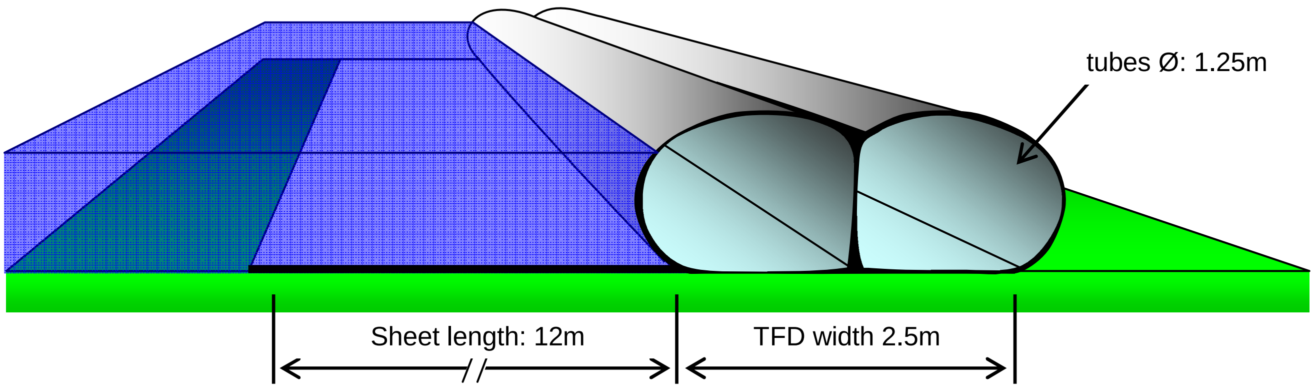


fig. 75 Impression of the preliminary twin tube TFD design

Because the sheet is quite long, it does have a negative effect on costs and workability. If there is less uncertainty in the location of deployment, the sealing sheet could turn out to be redundant. A buyer should then have the option to acquire the system without sealing sheet.

Besides these geometric and stability properties, the system should have the property of rapid deployment which is connected to the next three aspects (as described in paragraph 7.1):

- The system should have segments of considerable length to limit the time spent on connecting different segments;
- The system should have simple connection pieces that can link different segments together;
- The equipment used for deployment (tractor, spool and pumps) should be effective in supporting a quick deployment.

11 Reliability of the emergency measures

11.1 General

This chapter looks at the reliability of the temporary flood defense system, given the intended field of application. The most important part is the logistic feasibility. This is discussed in more detail since it also says something about the feasibility of the scale of application. The purpose of this chapter is therefore to also give a rough indication of the scale of application that is reasonably feasible.

The reliability of a system can be represented by the probability of failure of that system. The failure of a TFD system can be distinguished in three main failure modes of which logistic and constructive failure are the most obvious. The fault tree is depicted in fig. 76 and in the next paragraphs the failure modes are described in more detail.

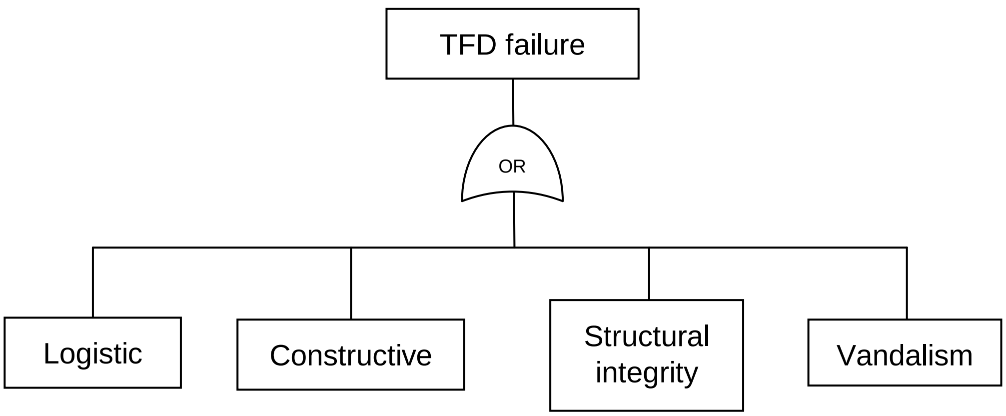


fig. 76 Fault tree for TFD system failure

The acceptable total failure probability of the system depends on the frequency of occurrence of the emergency and the desired safety level. When an area has a safety level of 1/10 per year and this is supposed to be increased to 1/1,000 per year, the failure of the system may be no higher than 1% for the range of water levels. In this chapter a maximum total failure probability of 1/50 is taken as a directive.

Intermezzo

The Maeslant Barrier (The Netherlands) was designed to provide the hinterland with a protection against the 1/10,000 per year storm surge. However, it was later determined that the safety level is only 1/7,000 per year. The following approach was made. The water level for which it has to be closed has a return period of 70 years while the system itself has a total probability of failure per closure of approximately 1%. Because failure of the barrier can be assumed independent of the water level, the safety level is then $1/70 \times 1/100 = 1/7{,}000$. *[39]*

11.2 Constructive failure

The constructive failure modes are shear, rotation and piping as described earlier in chapter 8. Since the system has been designed to withstand a certain maximum water level under the most unfavorable conditions and with a high safety factor, a probabilistic calculation is deemed unnecessary. The other failure modes are likely to have probabilities of failure that are at least one order of magnitude larger.

11.3 Logistic failure

11.3.1 General

The logistic failure of a system can be explained as the feasibility of a timely deployment. This depends on the time available and time required for deployment, both products of various location specific parameters. It is therefore very difficult to determine the general logistic reliability of a system. This paragraph provides some sensitivity analyses that give a rough indication of the logistic feasibility of the emergency measures. Additional to this the results will provide insight into the scale on which emergency measures are still reliable enough.

As said before the probability that the system is not deployed in time depends on the time available and the time required. The time available is in fact the time that expires between the moment that a situation is forecasted and the moment that this situation occurs, in other words, the warning time. The time required is the entire logistic process that has to be run through before the system has been placed. This process can be subdivided in three different activities:

- decision-making;
- transport;
- deployment.

These activities are assumed to be connected in series, meaning that the required time is the sum of the three. The reliability of the logistic process can then be represented by the following reliability function:

$$Z = T_{warning} - T_{decision} - T_{transport} - T_{deployment}$$

The logistic process will fail for any situation where $Z<0$. To make an assessment of the reliability, first the different parts of the process will be analyzed.

Though this report considers logistic failure only in terms of time, this is not entirely correct because logistic factors other than separate parts of the timeline could lead to failure. Especially for a shorter warning time, equipment breakdown or traffic jams could have a great influence. Still this is not in taken into account in this calculation since the objective of this paragraph is to give an indication not an exact failure probability.

11.3.2 Warning time

The warning time depends on the nature of the high water. A high water level on a river can be forecasted for several days while a storm surge along a sea coast has a warning time that is in the order of six hours. The Dutch FloRijn Rhine water level forecasting model is able to make a reasonably accurate third day prediction, resulting in a warning time of 72 hours.[6] It must be added that there are also river areas where the warning time is less because high water levels are unpredictable or because intricate warning systems are not present.

11.3.3 Decision making

During the 1995 high water in The Netherlands, the decision was made to evacuate several areas along the River Rhine. Each order for evacuation was preceded by approximately four hours of decision-making. Only the evacuation of Ochten needed less decision-time since the mayor did not want to wait for provincial approval. In this specific case, the decision was made in 1¼ hours.[6] The application of an emergency system has less negative consequences, that need to be pondered, than a large-scale evacuation. Therefore, the decision-making process is probably less time-consuming. A decision-time of two hours seems a reasonable estimate.

11.3.4 Transport

After the decision is made to deploy the emergency measure, the system will have to be transported to the location of deployment. The transportation time is very much dependent on the distance between the location of storage and the location of deployment. In this case a transportation time of 1 hour is assumed.

11.3.5 Deployment

Twin Flex Barrier
When the system is transported to the location, deployment can start. The deployment time depends on two aspects, namely the deployment rate and the length over which the system needs to be deployed. The first was earlier discussed in chapter 7 and was the main reason of the high performance of the Twin Flex Barrier in chapter 9.

When the deployment rate is based on that of the original Twin Flex Barrier the several sources of information provided rates varying from 600 to 1,200 m/hour per team of 4 to 5. Testing the system at a North Sea beach in Denmark resulted in a rate of 1km/hour. The test was done under the most favorable conditions (no wind or obstacles).

Extreme weather conditions
Deployment could be seriously hindered by external circumstances, the most obvious being extreme weather conditions. For instance the southwesterly storm of 18 January 2007 in The Netherlands caused a lot of damage. Several bridges and dikes were closed for traffic, since the situation got to dangerous or because trucks where blown down. Storms like these (wind force 10 or higher) have occurred 58 times over the last 100 years.[40]

For a storm surge at sea a high water level and extreme weather conditions are correlated. An extremely high water level is then always the result of high winds creating a wind set up. Application of a temporary flood defense system will then hardly be possible. Opposed to this, for a river area there is no correlation between a high water level and the weather condition are uncorrelated. The probability that a high water level coincides with extreme weather conditions is relatively small. Therefore the application of emergency measures in a river area is logistically better feasible.

It is difficult to determine a fixed threshold (e.g. Beaufort wind force) for workability. A storm like the one of January 18th will most likely make deployment of an emergency system impossible. Still, this does not mean that a lower wind force does not affect deployment. Wind force 8 will already have a significant negative effect on deployment progression. There is however no information available on the relation between deployment rate and weather conditions (or any other external influences for that matter), though the earlier figure of 1,000 m/hour is probably a sound estimation for the upper limit.

The lower limit for the deployment rate is of course 0 m/hour. The probability distribution between these limits is a point of concern since it is unknown but has a large influence on the results. A type of distribution that is often used for production rates is the beta distribution. This distribution is described by an upper and lower limit and two shape factors. Three examples of probability density functions of beta distributed deployment rates between the boundaries 0 and 1,000 are displayed in fig. 77.

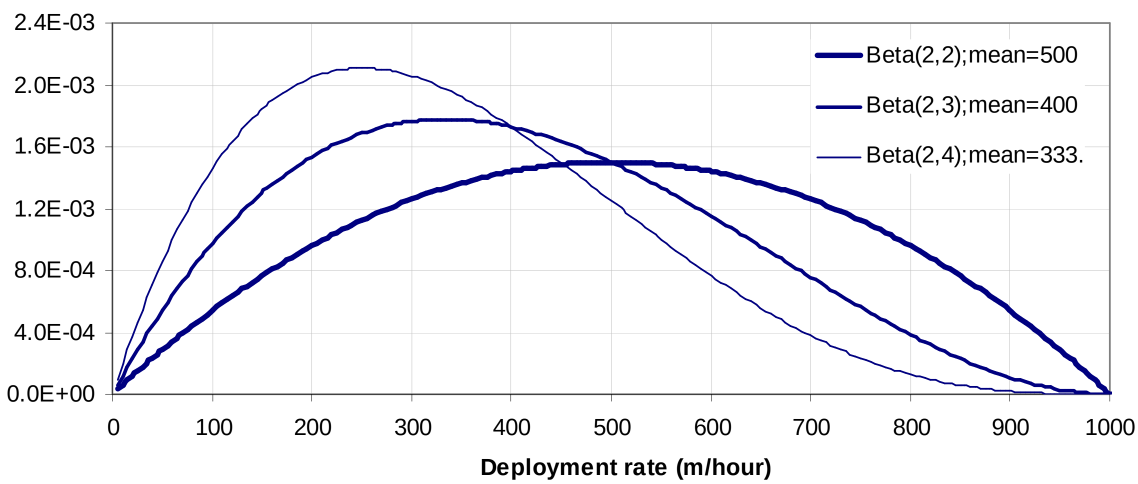


fig. 77 Probability density function of beta distributed deployment rates

The distributions have a clear distribution around an average value. These average values are chosen arbitrarily because they are actually unknown. The shown distributions can be discerned in an optimistic, an intermediate and a pessimistic approach.

11.3.6 Probabilistic calculations

For several parts of the reliability function a reasonable estimate has been made. The deployment rate is still the uncertain part in this calculation. The first calculation was made using the data proposed in the previous paragraphs, with the 72 hour warning time and for the three different beta-distributed deployment rates.

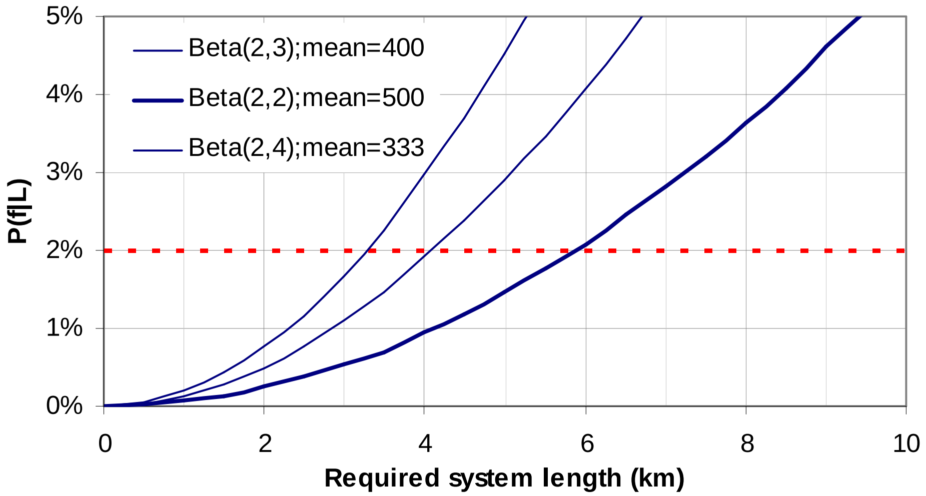


fig. 78 Conditional probability of failure of deployment for different deployment rate distributions

The graphs of fig. 78 show the development of the conditional probability of failure of the logistic process with respect to the required system length. The difference between the different distributions is large but in the same order of magnitude. For instance, for a required length of 4km, the failure probability varies between 1 and 3%. When the 2% boundary is respected the maximum system length varies between 3 and 6km for the three different distributions.

A parameter that has an even larger influence is the warning time. The sensitivity of the result is displayed in fig. 79 for the intermediate beta(2,3)-distributed deployment rate. For a warning time of only one day, the reliability of the logistic process drops alarmingly. The maximum length may then be little more than 1km.

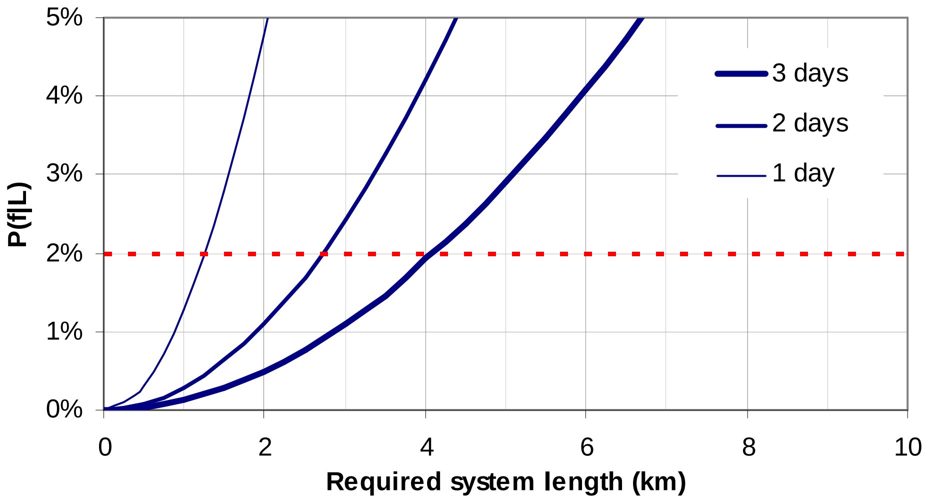


fig. 79 Conditional probability of failure deployment for varying warning time

It is of course also possible to assign more teams to the deployment job, but it is probably not realistic to assume that the responsible authority will keep a large workforce at hand to execute emergency measures with a low frequency of occurrence.

11.3.7 Water berm logistic feasibility

The water berm has an additional step in the process which is filling of the berm. For the deployment of the TFD system that walls the water berm, the same logistic considerations apply. The system will have to be deployed before the water level becomes critical. The filling of the berm should start no later than the moment the critical head difference (paragraph 3.2.5) is reached.

When assuming that the berm wall is constructed just before the head difference becomes critical, the pumping capacity should be large enough to guarantee the water level in the berm to rise at least at the rate the river water level rises. In 1995 the water level on the river Meuse rose with a rate of 5cm/hour.

Besides the rate of river water level rise, the pumping capacity is determined by size of the walled area that has to be filled. The size of the area depends on the width and length of the berm. For the case discussed in chapter 4, the width of the berm was set to be 20m. The length of the berm depends on the length of the weak dike section. The VNK flood risk studies show that dike sections vulnerable to piping have lengths that are often less than 1km. For example, the weakest dike section of dike ring area 10 has a length of 600m.[20]

Assuming an area of 1,000 by 20m (20,000m^2) and a minimum water level rise of 5cm per hour, the pumping discharge should be at least 1,000m^3/hour, which is less than 17m^3 per minute. This lies within the range of the pumps needed for deployment of the Twin Flex Barrier. So the application of this system as an emergency measure against piping is feasible.

11.3.8 Final remark on logistic failure

The analysis of this paragraph showed that the logistical process is broken down in various parts. The most important aspects in the process are the warning time and deployment rate, which both have a significant impact on logistic reliability.

It was determined that the length that can be deployed with the required reliability is in the order of magnitude of 4km. This does put a limitation on the possible fields of application.

For the expected small scale application of the water berm, this range is good enough. It is however relatively low for temporary crest heightening. For the purpose of temporary crest heightening only one type of application remains, characterized by the small scale of application.

This kind of application is the protection of areas with a low protection level but with a lot of potential damage like urban environments where dikes or quays cannot be heightened since there is too little space (e.g. Cologne in Germany, see fig. 80, left). Other examples are small areas of great value or importance such as hospitals and power stations like the Tewkesbury (Gloucestershire, UK) power substation which was nearly flooded during the high water of July 2007 (see fig. 80, right). At locations like these piping is of lesser concern.

fig. 80 High water in Cologne (left) and the Tewkesbury power substation (right)

11.4 Other failure modes: structural integrity and vandalism

Two other failure modes are structural integrity and vandalism. Structural integrity is the sum of possible failure modes through damage or design flaws. The points of interest with respect to structural integrity are the resistance of material to rupture and the reliability of joints. This is not further discussed here apart from stating that the probability of failure of the structural integrity should be at lower than that of logistic failure by at least one order of magnitude.

Besides structural integrity another source of failure is vandalism. That vandalism is a real threat was shown on January 2003 when the Water Board of Peel & Maasvallei used the Ducoda system to protect the village of Meerlo against the rising water of the Meuse (see fig. 81). The system was deployed over a length of in total 400m. Before the water level rose to the height of the TFD, it was discovered that the barrier had been cut with a knife. The system was repaired in time and eventually controlled about 20cm of water.

Verminkte Friese waterkering werkt

MEERLO – De demontabele waterkering van Ockels Geoproducten uit Hoornsterzwaag is zaterdag vlak na het opzetten langs de Veerweg bij het Noord-Limburgse Meerlo met een mes opengesneden. Het waterschap Peel en Maasvallei spreekt van een geluk dat het gat werd ontdekt voordat het Maaswater tegen de kunststofdam aan klotste. Woordvoerster Leonie Geelen: „Onbegrijpelijk dat iemand zo'n waterkering doelbewust vernielt." Er is aangifte gedaan.

De Ducoda (Dutch convertible dam) is een uitvinding van Ruurd Ockels. Hij reageert geschokt op de vernieling. „Er staan huizen pal achter die weg. Schandalig dat mensen zoiets doen. Tegen vandalisme is niets opgewassen." Doordat medewerkers van het waterschap het gat tijdig ontdekten, kon de scheur nog provisorisch worden gerepareerd.

De tijdelijke waterkering heeft de praktijktest goed doorstaan. De Friese vinding werd over 300 meter langs een fietspad opgesteld om een woonwijk in het achterland tegen het wassende water te beschermen. Nog eens 100 meter kering stond op een kade langs de Veerweg pal aan de Maas. Ockels: „Het is perfect gelopen. Er stond 20 centimeter water tegen de dam zonder dat er een drup doorheen lekte. Als het doek dat doorstaat, kan het ook een halve meter keren." Het waterschap toont zich enthousiast over het gemak waarmee de waterkering kon worden opgezet. Volgens woordvoerster Geelen heeft de Ducoda zich bewezen. „Waar anders vijftig man zandzakken zouden moeten vullen en sjouwen, konden nu een paar medewerkers de klus fluitend klaren", aldus Ockels. Een bijkomend voordeel is dat de dam ook snel op te breken is.

fig. 81 Article in the Leeuwarder Courant - 6 January 2003

It would seem that TFD systems made of synthetic material like water-filled tube systems are extremely vulnerable to vandalism. But this can be said for almost any mobile TFD system. A sandbag dam can also be demolished without much difficulty by displacing some bags from the dam and letting the resulting flow of water finish the job.

Intermezzo

During the high water on the Meuse in 2003, which was the first high water since that of 1995, a sandbag dam was constructed to protect the town of Arcen in Limburg. At night somebody kicked one of the bags out of the emergency dam. This did not lead to failure since the water was not yet at its highest point. *(Dagblad De Limburger, Tuesday 7 January 2003)*

Only the more robust systems like e.g. the concrete barriers have a considerable resistance against vandalism. Though vandalism could prove to be a significant problem, the data available is yet insufficient to be able to make a sound estimate of the failure probability.

11.5 Final remark on system reliability

This chapter described the reliability of the Twin Flex type of system in terms of a probability of failure. The main interest was the logistical process. Calculation showed that the length that can be deployed with reasonable reliability is quite limited. The calculations did however incorporate a lot of uncertainties. It is therefore recommendable that the logistic feasibility is further analyzed in the future.

12 Conclusion and recommendations

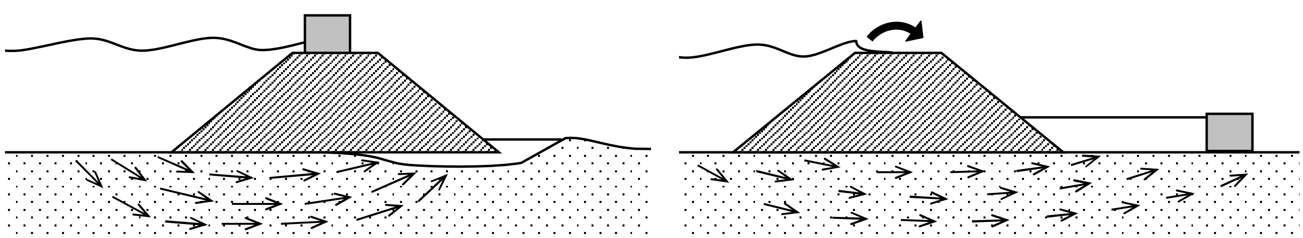
Objective

This report has the objective of designing a method or system to temporarily increase the strength of river dikes. To achieve this, the aim was first directed on the possible failure mechanisms rather than looking immediately at overtopping and systems against this. For this purpose a theoretical and practical inventory of failure mechanisms was made. This inventory led to the conclusion that besides overtopping, piping is also an important cause of dike failure in many river areas in the world.

Emergency measures

Piping and overtopping at river dikes both have their specific characteristics. Overtopping can occur at any dike or quay, while piping is restricted to dikes with specific geological characteristics. There are however two important characteristics that determine the usefulness for an emergency measure against piping. On the one hand, there are only few emergency systems against it. On the other, piping can occur for water levels below the protection level of a dike, while overtopping only occurs for water levels above this protection level. In other words, a measure against piping will be needed on a more frequent basis.

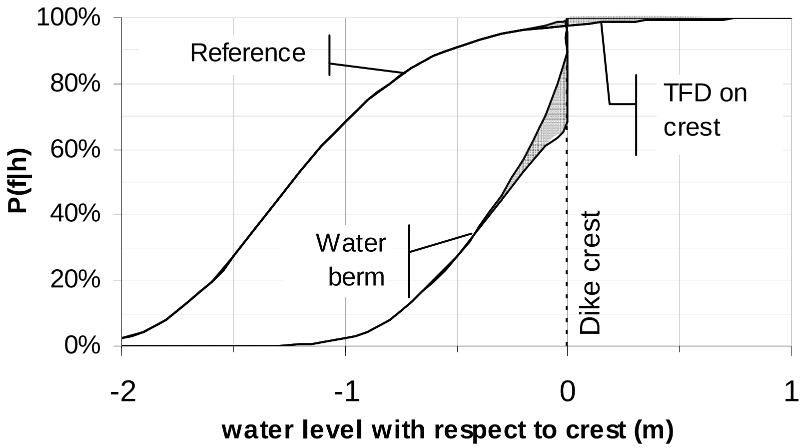
Emergency measures against both overtopping and piping where discussed. The considered emergency measure against overtopping is the traditional crest heightening by means of a water retaining system. The proposed emergency measure against piping is the preventively damming of a piping-sensitive area behind the dike and filling this area with water. This method decreases the head difference; the driving force in the piping mechanism.



The two proposed measures against overtopping (left) and piping (right)

By means of a probabilistic calculation it was determined that the water berm method can be very effective with respect to decreasing the probability of failure through piping. The measure increases the critical water level in front of the dike with the berm water height.

It was also determined that measures against overtopping have little effect when piping is also a problem for a certain dike section. But when other mechanisms are of less importance, temporary crest heightening can be very effective.



Effectiveness of the two measures with respect to a reference situation

Temporary flood defense systems

Because both measures make use of some sort of water retaining system, the next step was to design a temporary flood defense system for this purpose. Therefore, the existing systems have been examined and it was determined which system has the most promising properties. Ultimately the Twin Flex Barrier ("Mobiele Waterdam" in Dutch) water-filled tube type of system was chosen to have the best performance on the aspects of costs, deployment, stability and applicability. This system consists of two parallel tubes connected at one point. It has relatively low costs and a very high deployment rate and makes very few demands on the location of application.

Attempts to improve the stability by a modified tube shape showed that the shape has little effect on stability. The standard circular twin tube shape will therefore remain the most obvious. Because piping could in some cases lead to failure of the system, the design was adapted by adding a sealing sheet in front of the tubes to increase the piping length.

Reliability and scale of application

Finally the reliability of the measures was assessed. It turned out that especially the logistic feasibility is a point of concern. Even with a warning time of three days, the probability of logistic failure restricted the applied length to an order of magnitude of some 4km. For the water berm application this is not so much of a problem since this mechanism is in general restricted to small locations of often less than one kilometer. As a measure against overtopping, the scale limits the application to small areas with a lot of potential damage like river cities.

Recommendations

It must be stressed that the calculation of the logistic reliability did incorporate a lot of uncertainties and assumptions. A next step in the research could be an inventory of possible sites of application both against piping and overtopping. For a specific site the logistic feasibility of the emergency measure can then be more accurately determined.

Besides the logistic reliability, there is still some uncertainty on the reliability of the effectiveness of a sealing sheet. It is therefore recommended to further investigate this by testing the system.

13 List of figures

14 List of tables

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Appendix I. Deterministic calculations

Overtopping calculations

The actual overtopping discharge can be calculated with the equations below.[10]

$$q = \frac{0.067}{\sqrt{\tan(\alpha)}} \cdot \gamma_b \cdot \xi_0 \cdot \exp\left(-4.75 \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \right) \cdot \sqrt{g \cdot H_{m0}^3}$$

With a maximum for: $q = 0.2 \cdot \exp\left(-2.6 \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\gamma_f \cdot \gamma_\beta} \right) \cdot \sqrt{g \cdot H_{m0}^3}$

And with:

q	Average wave overtopping discharge	(m^3/s per m)
g	Acceleration due to gravity	(m/s^2)
H_{m0}	Significant wave height at toe of dike	(m)
ξ_0	Breaker parameter, $\xi_0 = \frac{\tan(\alpha)}{\sqrt{s_0}}$	(-)
s_0	Wave steepness, $s_0 = \frac{2\pi \cdot H_{m0}}{g \cdot T_{m-1;0}^2}$	(-)
$T_{m-1,0}$	Spectral wave period at toe of dike	(s)
tan(α)	Slope	(-)
R_c	Free crest height above still water line	(m)
γ	Influence factors for influence of berm, roughness elements, angle of wave attack and vertical wall on slope	(-)

The wave steepness was assumed to be 0.05 (typical for wind waves) with a wave direction perpendicular to the dike. The geometry of the dike is characterized by a slope of 1:3 and the γ-factors are all assumed to be 1.

Brettschneider method
The height of wind waves on a river can be determined using the Bretschneider method:[8]

Significant wave height: $H_s = \dfrac{u^2}{g} \cdot \tilde{H}$

where

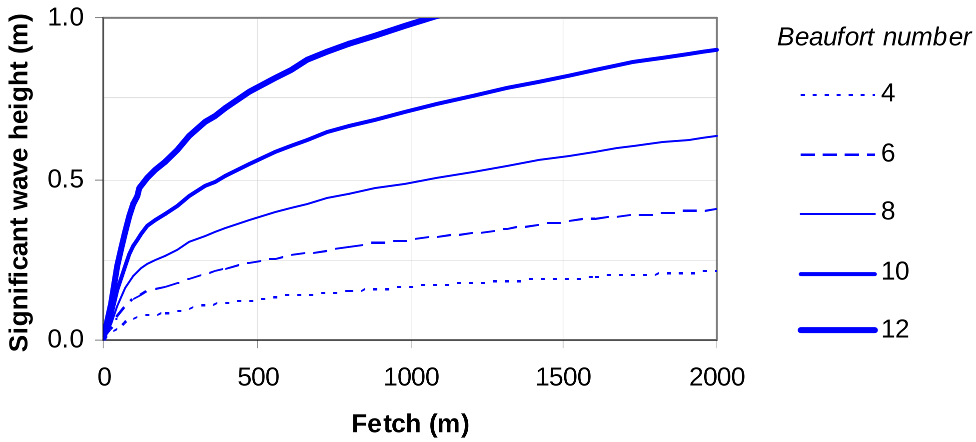
$$\tilde{H} = 0.283 \cdot \tanh\left(0.53 \cdot \tilde{d}^{0.75}\right) \cdot \tanh\left(\frac{0.0125 \cdot \tilde{F}^{0.42}}{\tanh\left(0.53 \cdot \tilde{d}^{0.75}\right)}\right)$$

with: $\tilde{F} = \dfrac{g \cdot F}{u^2}$ and $\tilde{d} = \dfrac{g \cdot d}{u^2}$

The input parameters are:

u	wind velocity	(m/s)
F	fetch length	(m)
d	water depth	(m)
g	acceleration of gravity	(m/s^2)

In the figure below some calculation results are shown for a water depth of 5m and various fetch lengths and wind conditions.



It is clear that only for high winds (Beaufort 8 or more) and a large fetch length (more than 1km), a wave height of 0.5m or more can occur.

Macro-stability calculations

The used parameters are:

Dike Geometry

crest height	6	m
crest width	5	m

slope is varying from 1/2 to 1/4

Soil parameters

	Subsoil	*Dike material*			
	sand	sand	clay		
γ_d=	18	18	17	kN/m^3	*(volumetric weight dry soil)*
γ_s=	20	20	17	kN/m^3	*(volumetric weight saturated soil)*
ϕ=	30	30	20	degrees	*(angle of repose)*
c=	0	0	10	kN/m^2	*(cohesion)*

Micro-instability calculations

The equation to calculate this ratio (the factor of safety) is displayed below, with the denominator being the driving force.

$$FS = \frac{\left(\left(\rho_s - \rho_w\right)\cdot g \cdot \cos(\alpha) - \rho_w \cdot g \cdot i \cdot \sin\left(\alpha - \theta\right)\right)\cdot \tan(\phi)}{\left(\rho_s - \rho_w\right)\cdot g \cdot \sin(\alpha) + \rho_w \cdot g \cdot i \cdot \cos\left(\alpha - \theta\right)}$$

with:

with:	α	slope angle
	ρ_w	volumetric weight of water
	ρ_s	volumetric weight of the sand grains
	g	acceleration of gravity
	i	gradient of the freatic line
	θ	angle of freatic line
	ϕ	angle of repose

Dike Geometry

crest height	6	m
crest width	5	m

slope is varying from 1/2 to 1/4

freatic line intersects at 25% of the river water level

Parameters

ρ_w=	1000	kg/m3	*(volumetric weight water)*
ρ_g=	2650	kg/m3	*(volumetric weight sand)*
ϕ=	30	degrees	*(angle of repose sand)*
g=	9.81	m/s2	*(acceleration of gravity)*

Piping calculations

The piping mechanism can be analyzed with the method of Sellmeijer. The equation is depicted below.

$$\Delta h_c = \alpha \cdot c \cdot \tan(\theta) \cdot \left(\frac{\rho_s - \rho_w}{\rho_w} \right) \cdot \left(0.68 - 0.10 \cdot \ln c \right) \cdot L$$

$$\text{with:} \quad \alpha = \left(\frac{D}{L} \right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8} - 1}} \quad \text{and:} \quad c = \eta \cdot d_{70} \cdot \left(\frac{g}{\upsilon \cdot k \cdot L} \right)^{\frac{1}{3}}$$

The used parameters are:

Dike Geometry

crest height	6	m
crest width	5	m

slope is varying from 1/2 to 1/4

Piping length (=dike base width) varies with slope

slope	1/2	1/3	1/4	
L=	29	41	53	m

D= 30 m *(aquifer thickness)*

Subsoil parameters

fine sand, d_{70}=	200	µm	*(70 percent value grain diameter)*
coarse sand, d_{70}=	300	µm	*(70 percent value grain diameter)*

k=	2.0E-04	m/s	*(permeability)*
ρ_g=	2650	kg/m^3	*(volumetric weight sand)*
ρ_w=	1000	kg/m^3	*(volumetric weight water)*
ν=	1.3E-06	m^2/s	*(kinematic viscosity)*
θ=	41	degrees	*(rolling resistance angle of sand grains)*
η=	0.25	-	*(coefficient of White)*

Heave calculations

The resistance against heave depends on the weight and thickness of the top clay layer and is determined by:

$$\Delta\varphi_c = d \cdot \frac{(\gamma_{sat} - \gamma_w)}{\gamma_w}$$

The safety against heave is then $FS = \frac{\Delta\varphi_c}{\Delta\varphi_x}$

with: $\Delta\varphi_x = \varphi_x - \varphi_0$ being the actual potential difference between the aquifer and polder.

The potential in the aquifer at a certain distance is determined by:
$\varphi_x = \varphi_0 + (\varphi_{HW} - \varphi_0) \cdot \exp(-x/\lambda)$, where: $\lambda = \sqrt{T \cdot C}$, $T = k_s \cdot D$ and $C = d / k_c$.

The used parameters:

Aquifer

D=	30	m	*(aquifer thickness)*
k_s=	2.00E-04	m/s	*(permeability of aquifer)*

clay layer

k_c=	1.00E-07	m/s	*(permeability of clay layer)*
γ_s=	1800	kg/m^3	*(volumetric weight of clay)*
γ_w=	1000	kg/m^3	*(volumetric weight of water)*

The clay layer thickness varies from 2 to 8m

Appendix II. Overtopping

Overtopping equations

The actual average overtopping discharge (q) can be determined with the following equations.[10]

$$q = \frac{0.067}{\sqrt{\tan(\alpha)}} \cdot \gamma_b \cdot \xi_0 \cdot \exp\left(-f_m \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \right) \cdot \sqrt{g \cdot H_{m0}^3}$$

With a maximum for: $$q = 0.2 \cdot \exp\left(-2.6 \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\gamma_f \cdot \gamma_\beta} \right) \cdot \sqrt{g \cdot H_{m0}^3}$$

in which:

q	average wave overtopping discharge	(m^3/s per m)
g	acceleration due to gravity	(m/s^2)
H_{m0}	significant wave height at toe of dike	(m)
ξ_0	breaker parameter, $\xi_0 = \frac{\tan(\alpha)}{\sqrt{s_0}}$	(-)
s_0	wave steepness, $s_0 = \frac{2\pi \cdot H_{m0}}{g \cdot T_{m-1;0}^2}$	(-)
$T_{m-1,0}$	spectral wave period at toe of dike	(s)
$\tan(\alpha)$	outer dike slope	(-)
R_c	free crest height above still water line	(m)
f_m	model factor	(-)
γ_b	influence factor for berm	(-)
γ_f	influence factor for roughness of slope	(-)
γ_β	influence factor for angle of wave attack	(-)
γ_v	influence factor for vertical wall on slope	(-)

In probabilistic calculations the parameter f_m is a normally distributed random variable with a mean of 4.75 and a standard deviation of 0.5.

Bretschneider method

The most important parameters in the overtopping formula are the significant wave height (H_{m0}), the spectral wave period ($T_{m-1,0}$) and the free crest height above the still water line (R_c). The free crest height depends on the chosen water level for which the calculations are made. The first two parameters can be determined with the Bretschneider method.

The Bretscheider method uses the wind velocity, fetch length (i.e. the distance over the water surface over which the wind can develop waves) and the water depth. The calculation is as follows:

Significant wave height: $H_s = \frac{u^2}{g} \cdot \tilde{H}$

Peak wave period: $T_p = \frac{u}{g} \cdot \tilde{T}$

where

$$\tilde{H} = 0.283 \cdot \tanh\left(0.53 \cdot \tilde{d}^{0.75}\right) \cdot \tanh\left(\frac{0.0125 \cdot \tilde{F}^{0.42}}{\tanh\left(0.53 \cdot \tilde{d}^{0.75}\right)}\right)$$

and

$$\tilde{T} = 7.54 \cdot \tanh\left(0.833 \cdot \tilde{d}^{0.375}\right) \cdot \tanh\left(\frac{0.077 \cdot \tilde{F}^{0.25}}{\tanh\left(0.833 \cdot \tilde{d}^{0.375}\right)}\right)$$

with: $\tilde{F} = \frac{g \cdot F}{u^2}$ and $\tilde{d} = \frac{g \cdot d}{u^2}$

The input parameters are:

u	wind velocity	(m/s)
F	fetch length	(m)
d	water depth	(m)
g	acceleration of gravity	(m/s^2)

The output values according to the Bretschneider equations deviate a little from the required input parameters for the overtopping discharge formula. The spectral wave period can be calculated from the peak period by: $T_{m-1,0} = \frac{T_p}{1.1}$.
The wave height deviates very little, so it is acceptable to assume: $H_{m0} \approx H_s$.

This is only true for relatively deep water at the toe for which $h/H_{m0}>3$ to 4 (where h is the water depth at the toe), because then it can be assumed that waves will not break at the foreshore. If breaking does occur, this influences the wave spectrum and the equations above are no longer valid. This might be a problem for the calculations with a low water level. Overtopping is however negligible for low water levels so this inconsistency is assumed to be acceptable.

Angle of wave attack
Besides the wind velocity, the wind direction is of great importance. Though the case dike is assumed to lie perpendicular to the prevailing wind direction there is still a large probability that the wind comes from another direction. This affects the overtopping discharge. Therefore the following relations will be used.

The relation between γ_β and the angle of wave attack is displayed below:

$$\gamma_\beta = 1 - 0.0033 \cdot |\beta| \qquad \left(0^o \leq |\beta| \leq 80^o\right)$$

$$\gamma_\beta = 1 - 0.0033 \cdot 80 \qquad \left(|\beta| > 80^o\right)$$

where β is the angle of wave attack which is 0 for the wave direction perpendicular to the dike.

In practice it is possible that the angle of wave attack is greater than 80° or that the wind is coming from the landside of the dike. For the situation that $80^o \leq |\beta| \leq 110^o$, the wave height and period should be adapted instead of the influence factor. This should be done by:

Multiplying H_{m0} with: $\frac{110 - |\beta|}{30}$

Multiplying $T_{m-1,0}$ with: $\sqrt{\frac{110 - |\beta|}{30}}$

When $110^o \leq |\beta| \leq 180^o$ the influence of waves is negligible so $H_{m0}=0$.

The deterministic parameters for the probabilistic calculations are summed op in the table below. The choice of the critical overtopping discharge has already been explained. The fetch length (F) is based on the river width. The river Rhine in The Netherlands has a width that roughly varies between 1 and 2km. Here, the width is taken to be 1,000m.

Parameter	***Value***	***Dimension***
q_c	10	l/s/m
F	1,500	m
d	5	m
tan(α)	1/3	-
γ_b	1.0	-
γ_f	1.0	-
γ_v	1.0	-
g	9.81	m/s^2

table 12 Deterministic variables

The depth (d) varies over the width of the river. At the summer bed of the river the river is much deeper than at the foreland. The depth also varies for different water levels. Since the depth is of little influence on the calculation results, it can be assumed to be constant at 5m. Finally, the dike slope and influence factors are determined by the case dike geometry. The influence factor for angle of incidence of waves (γ_β) depends on the wind direction.

Wind direction and velocity
The wind direction and wind velocity are the only random variables besides the model factor (f_m). The distribution of both wind velocity and direction is based on data obtained from the Royal Dutch Meteorological Institute (KNMI). The analyzed data exists of measurements of the maximum hourly average wind velocity per day over the period 1-1-1904 to 22-4-2007. The measurements were done at station De Bild, which lies land inward. The considered data can therefore be assumed to be representative for a river area.

The analysis exists of the division of wind direction in 8 segments of 45 degrees wide (north, north-west, west, etc.) and determining the probability of occurrence of each wind direction. In addition the wind velocity distribution per directional segment was determined.

Direction	**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**
Distribution	8.8%	11.3%	8.0%	9.2%	14.6%	23.2%	14.9%	10.0%
Wind velocity (Gumbel distributed)								
μ (m/s)	5.49	5.78	5.67	5.75	6.79	7.31	6.74	6.01
σ (m/s)	2.13	1.97	1.96	2.20	2.88	2.78	2.83	2.66

table 13 Distribution of wind direction and velocity

The table shows two important results. The first being that almost ¼ of the wind comes from a south-westerly direction. The second is that the wind direction and wind velocity distribution are correlated. The wind velocity is Gumbel-distributed for each direction. However, the table above shows that the wind velocity has a different mean and standard deviation for each direction. This correlation will be taken in account in the probabilistic calculations. Finally, it is assumed that there is no correlation between water level and wind velocity or direction, since a high water level is the result of a river discharge that itself is a consequence of rainfall hundreds of kilometers away.

The probabilistic calculation incorporates only three random variables. Besides the model-factor, the wind-direction and wind velocity are very important for the calculations results. Because these are both based on over a century of daily measurements, an additional sensitivity analysis is deemed unnecessary.

Appendix III. Piping

The Sellmeijer equation is used to assess the piping mechanism. The critical water level difference can be calculated by:

$$\Delta h_c = \alpha \cdot c \cdot \tan(\theta) \cdot \left(\frac{\rho_s - \rho_w}{\rho_w} \right) \cdot \left(0.68 - 0.10 \cdot \ln c \right) \cdot L$$

$$\text{with:} \quad \alpha = \left(\frac{D}{L} \right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8} - 1}} \quad \text{and:} \quad c = \eta \cdot d_{70} \cdot \left(\frac{g}{\upsilon \cdot k \cdot L} \right)^{\frac{1}{3}}$$

in which:

D	thickness of the sand layer	(m)
L	length of the piping line	(m)
d_{70}	70 percent value of the grain distribution	(m)
k	permeability of the sand layer	(m/s)
ρ_s	volumetric weight of sand	(kg/m^3)
ρ_w	volumetric weight of water	(kg/m^3)
ν	viscosity of water	(m^2/s)
g	acceleration of gravity	(m/s^2)
θ	rolling resistance angle of the sand grains	(°)
η	coefficient of White	(-)

The most important parameters are L, d_{70} and k and (to a smaller extent) D. In this case L is determined by the base width of the dike. D, d_{70} and k are all characteristic properties of the aquifer.

Correlation between permeability and grain diameter
In practice the d_{70} and permeability are correlated. Subsoil with a large grain diameter is relatively permeable, while finer grained soils have a much lower permeability. When assuming the two parameters to be uncorrelated, simulation could lead to the unlikely result that a small grain diameter is coupled to a high permeability, resulting in high failure probabilities. A method to correlate k to d_{70} is the Den Rooijen equation:

$$k = \left(c_0 - 1.83 \cdot 10^3 \cdot \ln(u) \right) \cdot d_{10}^2$$

where:

c_0	parameter for the packing of the sand	(m^{-1}s^{-1})
U	uniformity of the sand, $u=d_{60}/d_{10}$	(-)
d_{10}	10 percent value of the grain distribution	(m)
d_{60}	60 percent value of the grain distribution	(m)

The d_{10} can be estimated from d_{70} using $\frac{d_{70}}{d_{10}} \approx 1.1 \cdot U$ where the factor 1.1 in the formula is actually the ratio d_{70}/d_{60}.

The Den Rooijen formula, however, fails to incorporate the heterogeneity in the aquifer because the permeability is correlated to the d_{70} of the top layer of the aquifer. In practice it is often the case that the sand at greater depth is coarser than the sand directly underneath the dike. The Den Rooijen formula therefore underestimates the permeability. To compensate for this, a weighed k-value can be determined that also incorporates the influence of the coarser material at greater depth. This weighed permeability can be calculated by: $k^* = \alpha k_1 + (1-\alpha)k_2$

with: $\alpha(k_1, k_2) = 1 - 0.4\left(\frac{k_1}{k_2}\right)^{0.7}$

Here k_1 is the permeability correlated with the d_{70} of the top of the aquifer and k_2 is the permeability of the rest of the aquifer. This equation is based on the preliminary results of a calibration process in the framework of VNK2. The sensitivity of the chosen method of correlation is assessed at the end of this appendix.

The random and deterministic variables that are used in the Sellmeijer equation are summed up below. The piping length (L) is determined by the dike geometry (dike base width). The other subsoil and geometry parameters are common for the Dutch river area. The variation coefficients (vc) in the table are taken from the Technical Report on Piping at River Dikes.[13]

Random variables

Parameter	***Distribution***	μ	***Vc***	ε
D (m)	Lognormal	30	0.10	0
d_{70} (m)	Lognormal	300	0.20	150
U (-)	Lognormal	2.5	0.10	1.5
d_{70}/d_{60} (-)	Lognormal	1.1	0.05	1.0
k_2 (m/s)	Normal	$5*10^{-4}$	0.20	0

Deterministic variables

L	41	m
ρ_s	2,650	Kg/m^3
ρ_w	1,000	Kg/m^3
ν	$1.33*10^{-6}$	M^2/s
g	9.81	m/s^2
θ	41	^{o}
η	0.25	-

In contrast to the overtopping calculation, the piping calculation incorporates more uncertainties in the parameters. The most important uncertainty is, however, the method of correlation. As described earlier, three methods exist to describe the relation between grain diameter and permeability, namely:

- no correlation between k and d_{70};
- correlation according to Den Rooijen;
- weighed permeability.

For these three methods, the development of the conditional probability of failure with respect to the water level is displayed in fig. 82.

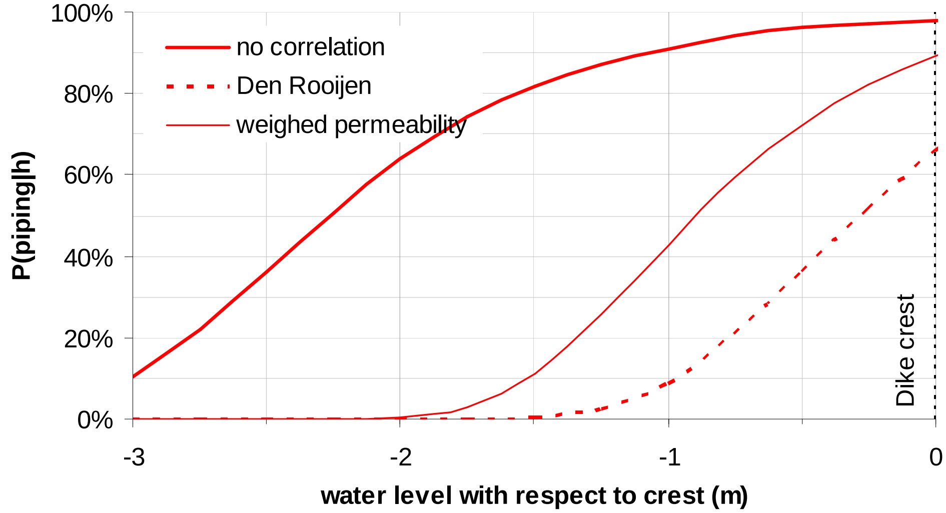


fig. 82 Sensitivity of correlation methods for piping calculation

The results show maximum failure probabilities for the uncorrelated calculation. This method is deemed to give a very conservative estimation of the conditional probability of failure. For the Den Rooijen method the results are much lower and probably show a too optimistic image. The weighed permeability method is assumed to produce the most realistic result. This method will be used further in the calculations.

Appendix IV. Dike ring areas

Appendix V. Temporary Flood Defense systems

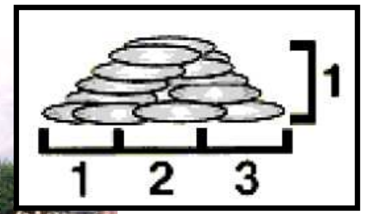
Sandbags

Characteristics

Height: 0.4m
Base width: 1.2m
Crest width: 0.4m

Volumetric weight sand
dry: 1,600kg/m^3
wet: 2,000kg/m^3

Costs of purchase: ± € 20,- per m length



Big Bags

Characteristics

Height: 0.85m
Base width: 0.95m
Filled volume: 0.8m^3/m

Volumetric weight sand
dry: 1,600kg/m^3
wet: 2,000kg/m^3

Costs of purchase: € 35,- per m length

Waterfront-Block

Characteristics

Height: 0.8m
Base width: 0.8m

Volumetric weight concrete: 2,400kg/m^3

Costs of purchase: € 125,- per m length

Characteristics

Pallet dimensions: 1.2 x 0.8 x 0.15m
(length x width x height)
Slope: 1:1
Height: 0.95m
Barrier weight: 60kg/m

Sealing sheet length: 2.7m
(1.5m in front of barrier)

Support dimensions: 0.95 x 0.245m
(length x width)

Costs of purchase: € 205,- per m length

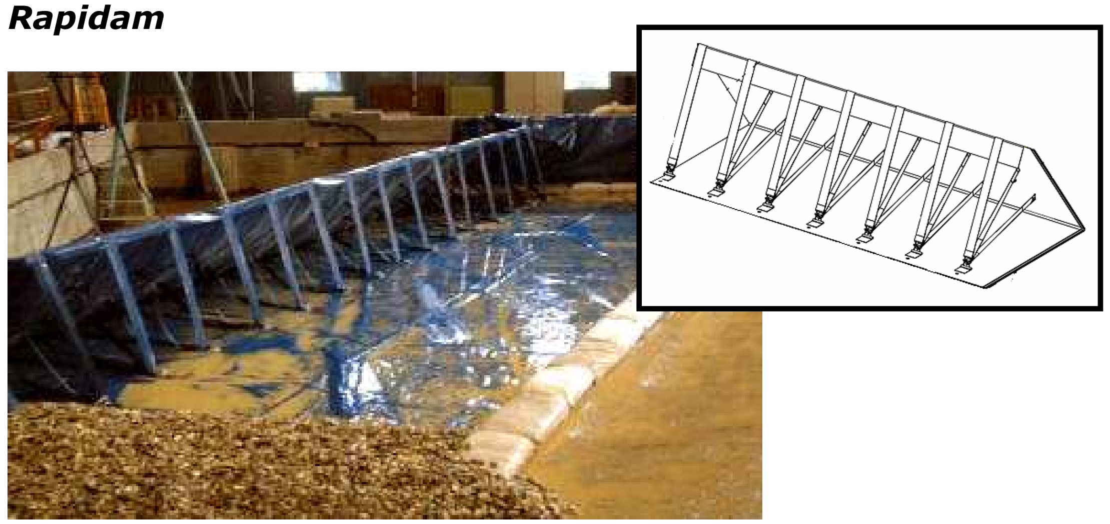
Pallet Barrier

Characteristics

Heigth: 1m
Slope: 1:1
Bottom width: 4m

Costs of purchase: € 420,- per m length

Rapidam



Characteristics

Sides: 0.8m
Heigth: 0.69m

Filled volume: 0.27m^3/m

Costs of purchase: € 330,- per m length

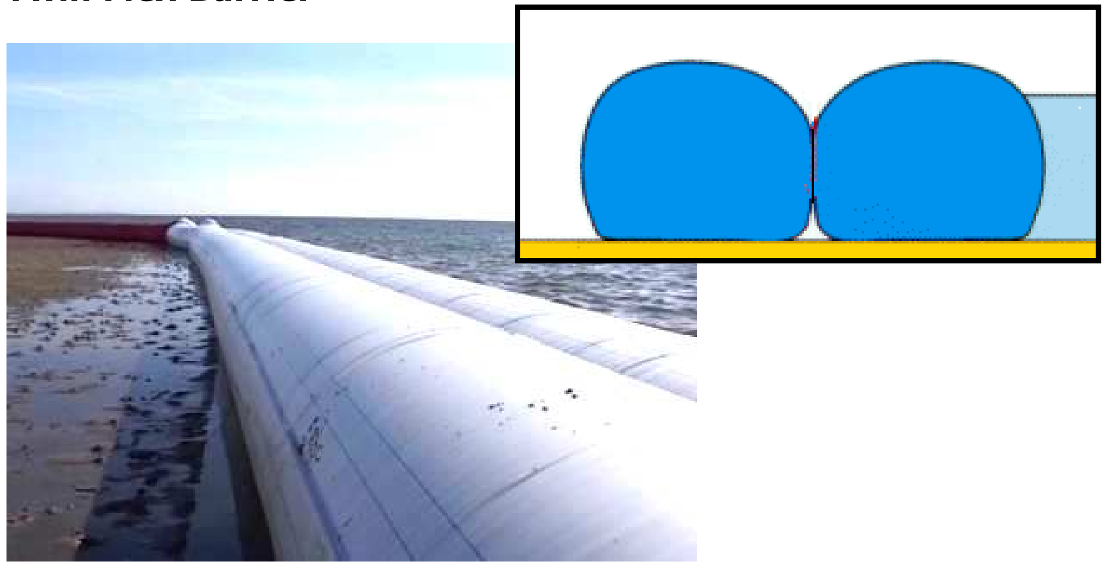
Aqua Levee

Characteristics

Barrier height: 1.20m
Base width: 2.5m
Inflated volume: 2.33m^3/m

Costs of purchase: € 200,- per m length

Twin Flex Barrier

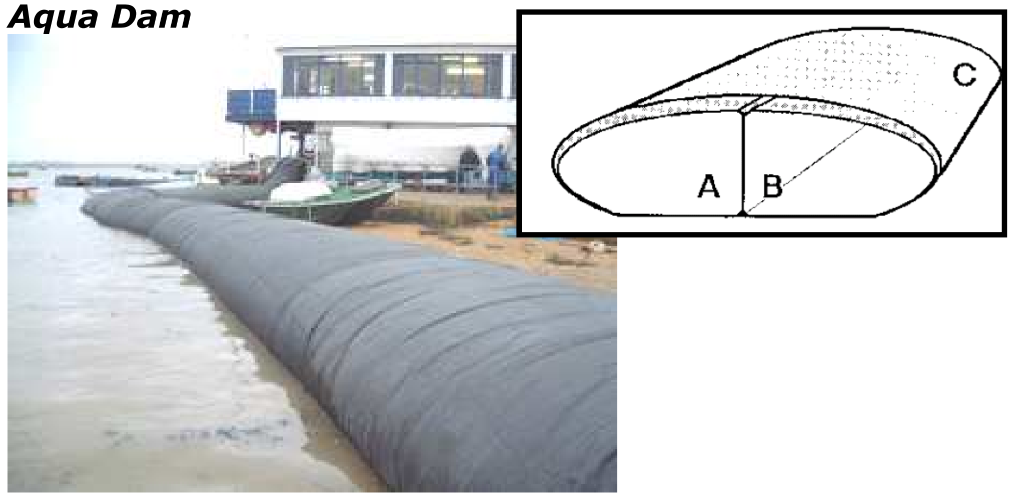


Characteristics

Barrier height: 1.22m
Base width: 2.9m
Inflated volume: 3.0m^3/m

Costs of purchase: € 150,- per m length

Aqua Dam

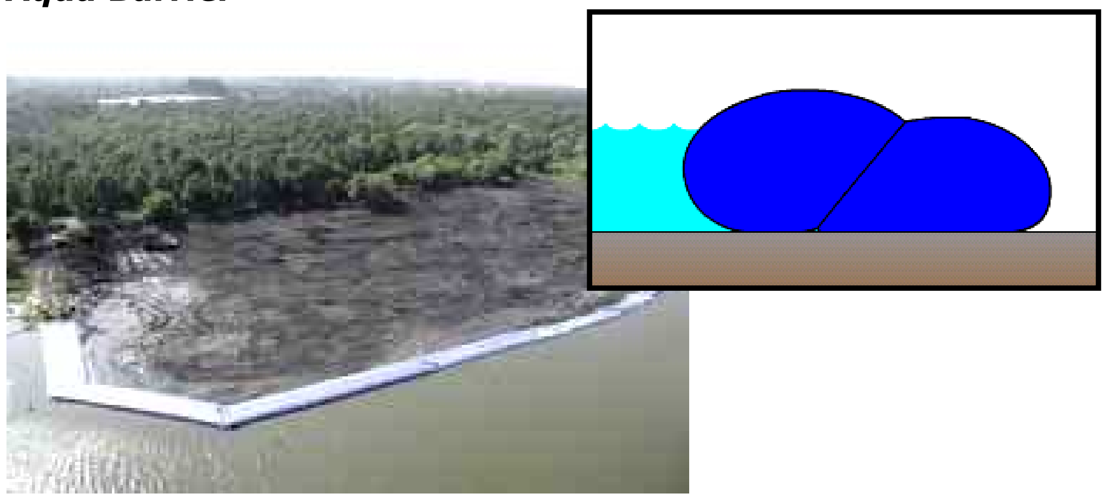


Characteristics

Barrier height: 1.22m
Base width: 3.05m
Inflated volume: 3.2m^3/m

Costs of purchase: € 240,- per m length

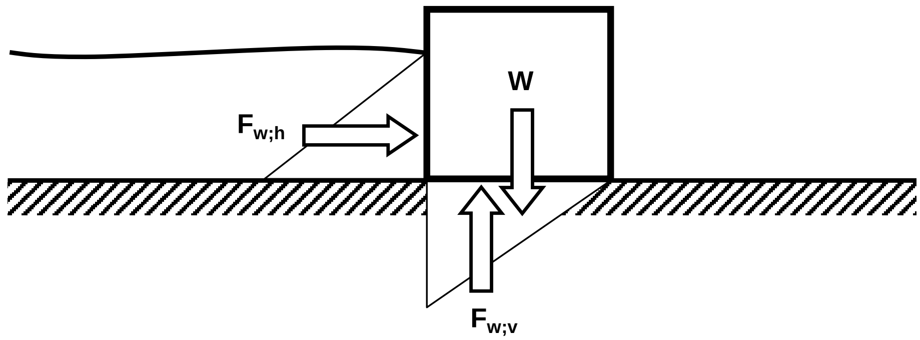
Aqua Barrier



Appendix VI. Temporary Flood Defense stability

Weight structures

The general schematization of a TFD system with the acting forces is displayed below:

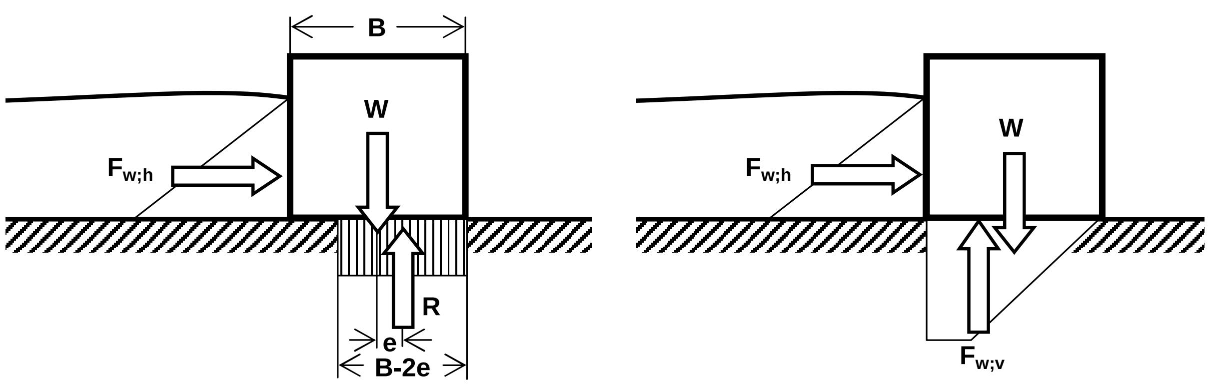


- System weight: W
- Horizontal hydrostatic force: $F_{w;h} = \frac{1}{2} \rho_w \cdot g \cdot h^2$
- Vertical hydrostatic force: $F_{w;v} = \frac{1}{2} \rho_w \cdot g \cdot B \cdot h$

Where h is the water level, B the system width and ρ_w the water density.

Validity of linear upward water pressure distribution

The horizontal hydrostatic force influences the stress distribution in the subsoil and results in a smaller contact width between system and subsoil. This effect can be examined by determining the effective foundation width for an eccentrically loaded foundation. This effective foundation width is displayed in the figure below, as well as the effect on the upward water pressure.



Effective contact with (left) and resulting upward water pressure diagram (right)

The resulting force (R) in the subsoil can be calculated by:

$$R = \sqrt{F_{w;h}^2 + W^2}$$

The eccentricity of this resulting force can be determined by summing the moments around the centre of the systems base, which results in:

$$e = \frac{F_{w;h} \cdot \frac{1}{3} H}{R}$$

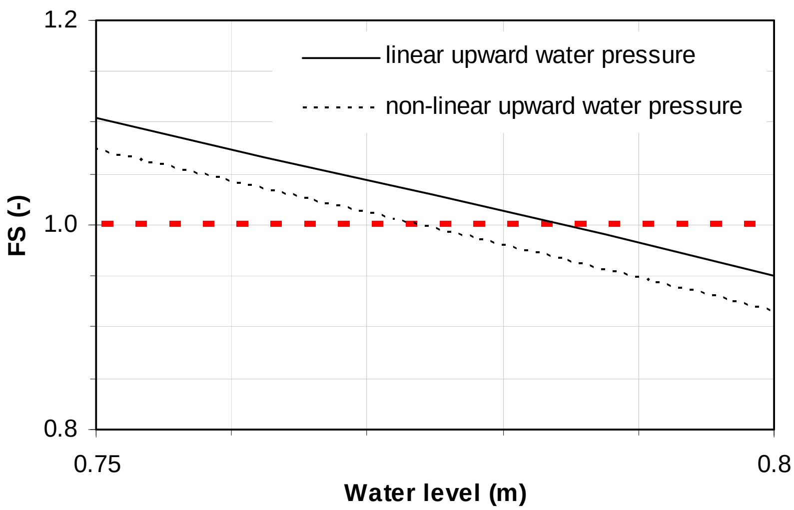
The width of the contact area is then: $B-2e$

To determine the difference between the original linear and non-linear upward water pressure distribution, an example calculation was made for the Waterfront-Block. The table shows the increase of the eccentricity and the decrease of the contact width for increasing water level.

H (m)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8
e (m)	0.00	0.00	0.00	0.01	0.01	0.02	0.04	0.05
B-2e (m)	0.80	0.80	0.79	0.79	0.77	0.75	0.73	0.69

Development of eccentricity and contact width with respect to water level

The graph shows the results of the calculation of the factor of safety, zoomed in at the water level for which the system becomes unstable. The results show that the difference between the two upward water pressure schematizations is marginal. The assumption of a linear upward water pressure distribution is therefore acceptable.



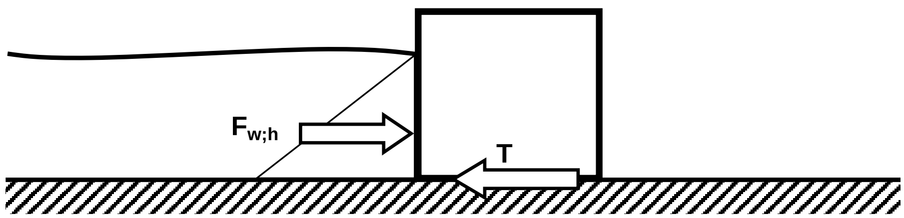
Influence of upward water pressure schematization

Shear

Factor of safety against shear: $FS = \dfrac{T}{F_{w;h}}$

With the friction force: $T = f \cdot N$

The normal force: $N = W - F_{w;v}$



Shear coefficient

The shear coefficient depends on the material of the system and the foundation. Shear coefficients for different materials are displayed in the table below. For any material the difference between a dry or wet surface is striking. The low shear coefficient of rubber and wet asphalt is a point of concern when applying a water-filled TFD on a road. The shear coefficient for synthetic material and grass is also low. For the calculations a shear coefficient of 0.25 for all systems is assumed.

Materials		**Shear coefficient**	
TFD	**Soil / foundation**	***min***	***max***
steel	*concrete*	0.45	
cement	*cement*	0.65	
cement concrete	*dry clay*	0.4	
cement concrete	*wet clay*	0.2	
cement concrete	*wet sand*	0.40	
cement concrete	*dry sand*	0.50	0.60
rubber	*asphalt (dry)*	0.5	0.8
rubber	*asphalt (wet)*	0.25	0.75
rubber	*concrete (dry)*	0.6	0.85
rubber	*concrete (wet)*	0.45	0.75
HDPE	*angular sand*	0.3	0.4
HDPE	*rounded sand*	0.2	0.3
synthetic material	*grass*	0.2	0.3

Shear coefficients for different materials *(Source: [4], [37] and [38])*

Composite systems

Pallet Barrier: Brinch Hanssen method [8]

The maximum bearing force can be approximated by:

$$F_{\max} = p'_{\max} \cdot A$$

whereby:

$$p'_{\max} = c' N_c s_c i_c + q' N_q s_q i_q + 0{,}5\gamma' B \cdot N_\gamma s_\gamma i_\gamma$$

In which the factors for the bearing force are:

$$N_c = \left(N_q - 1\right)\cot\phi' \qquad N_q = \frac{1+\sin\phi'}{1-\sin\phi'} e^{\pi \tan\phi'} \qquad N_\gamma = 2(N_q - 1)\tan\phi'$$

In which the factors for the shape of the foundation ($B \le L \le \infty$) are:

$$s_c = 1 + 0.2\frac{B}{L} \qquad s_q = 1 + \frac{B}{L}\sin\phi' \qquad s_\gamma = 1 - 0.3\frac{B}{L}$$

In which the factors for the horizontal load ($B \le L \le \infty$) are:

For *drained* soil:

For H parallel to L and $L/B \ge 2$:

$$i_c = \frac{i_q N_q - 1}{N_q - 1} \qquad i_q = i_\gamma = 1 - \frac{H}{F + Ac'\cot\phi'}$$

For H parallel to B :

$$i_c = \frac{i_q N_q - 1}{N_q - 1} \qquad i_q = \left(1 - \frac{0.70H}{F + Ac'\cot\phi'}\right)^3 \qquad i_\gamma = \left(1 - \frac{H}{F + Ac'\cot\phi'}\right)^3$$

For *undrained* soil:

$$i_c = 0.5\left(1 + \sqrt{1 - \frac{H}{Af_{undr}}}\right)$$ for the rest, see above.

With the used subsoil parameters:

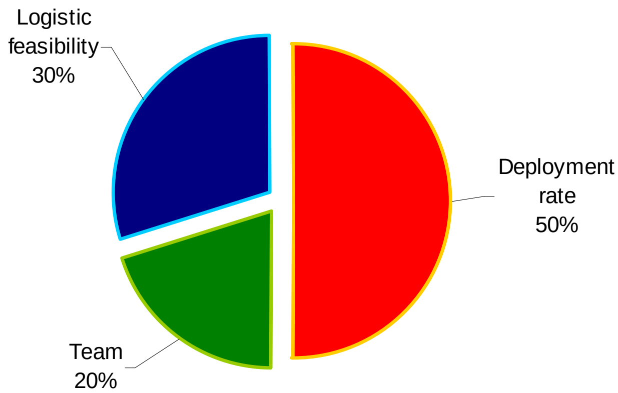
subsoil	**sand**	**clayey sand**	**Stiff clay**	**organic clay / peat**
F	32.5	27.5	17.5	15
c'	0	0	10	3
f_{undr}	0	0	50	20
g	20	21	17	13

The system geometry is incorporated in Appendix V

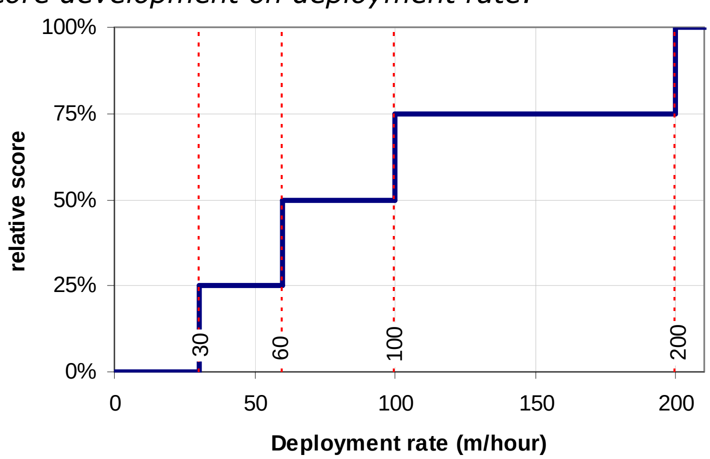
Appendix VII. TFD score table

Deployment

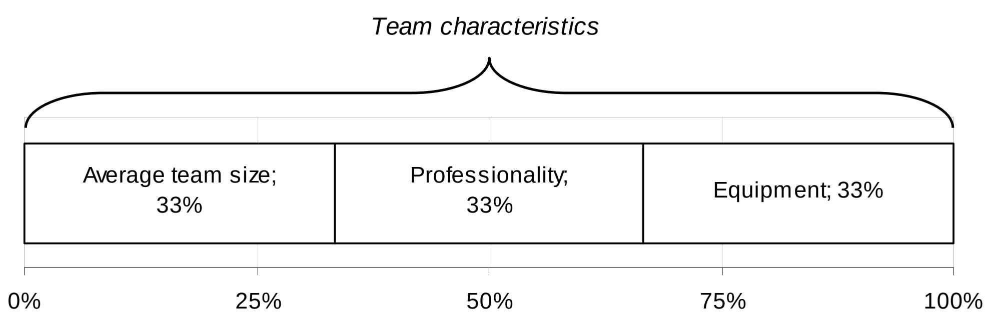
Importance of aspects:



Score development on deployment rate:

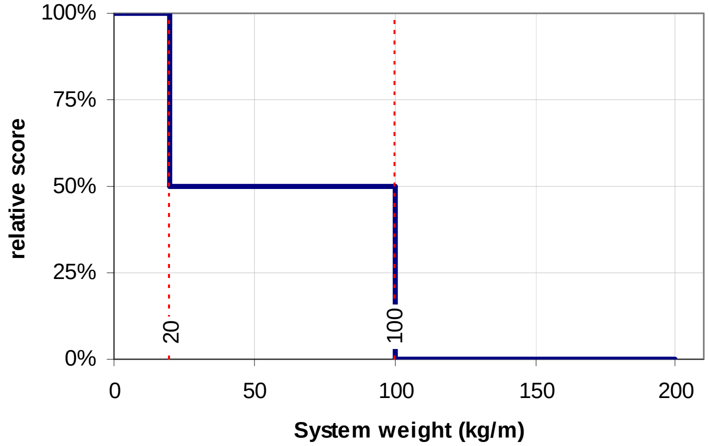


Score distribution on team characteristics:



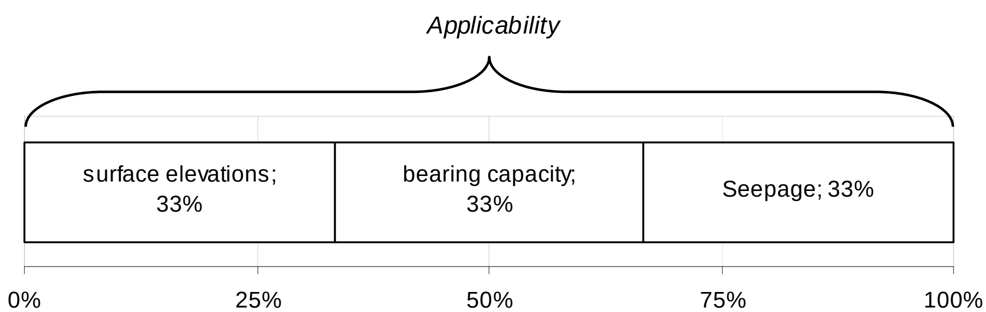
Score per aspect: Average team size >5: score=0
Professionalism required: score=0
Equipment required: score=0

Score development on logistic feasibility:



Applicability

Score distribution on applicability



For both surface elevations and bearing capacity the score is awarded to systems that have no specific requirements with respect to these aspects. The score on piping is related to the ratio between system width and controlled water level and is distributed as follows:

$\Delta H_w/B<3$: score=0

$3< \Delta H_w/B <6$: score=0.5

$\Delta H_w/B >6$: score=1