THE DELTADIKE CONCEPT AND THE EFFECTIVENESS OF VARIOUS DIKE REINFORCEMENT ALTERNATIVES
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'The ability to simplify means to eliminate the unnecessary so that the necessary may speak.'

Hans Hoffman (painter, 1880 – 1966)
Preface

The master’s thesis research has been performed to gain insight into the applicability of the deltadike concept in The Netherlands. Both the vagueness of the deltadike concept and the discussion on the concept involving professionals of different background motivated me to clarify what is actually of importance regarding the design of deltadikes. I considered simplification as an honourable duty throughout the research; the quote on the previous page expresses my feeling towards it. I have tried my very best to keep this report focused and structured towards the simple and short description of the relevant trade-offs. Hopefully you can reap the benefits from it!

Readers who are interested in the main line of thought of this report are advised to read the summary. For more (interesting) details please refer to the numbered chapters which comprise the core of the report.

Gratitude goes out to the people who contributed to this study. I thank Pieter van Gelder and Chris Zevenbergen for the initiation of this research project. Thankfulness also goes to colleagues at the Flood Resilience Group (UNESCO-IHE) and Dura Vermeer for the sociable and serious discussions. In special I thank Ellen Brandenburg for her support. I thank Wim Kanning for teaching me how to use MStab. I thank Kees den Heijer for explaining me how to use OpenEarthTools and the included script of the FORM routine. I thank Kathryn Roscoe for her interest in the project and her contributions to it. I thank Paul Visser and Thijs Robijns for providing and explaining the ‘breach growth model in sand-dikes’ to me. I thank Thomas den Hengst for teaching me how to use Adobe Illustrator and helping me to draw digital sketches. I thank Gerbrant van Vledder for his explanation and provided data regarding the wave load on dikes. I also thank prof. Vrijling, prof. Jonkman, prof. Phoon and dr. Goh for their reflections on the research. I thank Chris Zevenbergen, Edwin Blom, Kathryn Roscoe and my father for reviewing the report.

Special gratitude goes out to my direct supervisors Berry Gersonius and Wouter ter Horst for their supervision throughout the research. Berry, thank you for your enthusiasm during our discussions and for showing me how the deltadike is currently put into practice. Wouter, thank you for redirecting me a number of times and for affixing my attention to reporting the research outcomes in a structured way.

Lastly I thank teachers, friends and family for their indispensible help during the period of having confronting disabilities. In particular I thank Robert Hasselaar, Cecilia Shanti Dewi, Mary Josephine Lee, prof. Babovic, prof. Stelling, dr. Visser, Willem Jan den Hengst and my parents.

Simon den Hengst
Delft, December 2012
Summary

Recent identification of methodological, demographical, economical and physical changes in the flood protection capacity of dikes led to a rising awareness that the dikes in The Netherlands are not safe enough (Projectbureau VNK2, 2011a; Projectbureau VNK2, 2011b; Kind, 2011). The second Delta Committee responded to these developments by advising to increase the safety level of all dikes by at least a factor ten. The committee furthermore advised to investigate the concept of deltadikes which are ‘either so high or so wide and massive that the probability that these dikes will suddenly and uncontrollably fail is virtually zero’ (Delta Committee, 2008). The committee also advised to consider the deltadike concept for dikes which require a reduction in probability of failure by more than a factor ten.

The deltadike concept is currently associated with a variety of dike concepts such as the broad dike, the super levee, the overtopping-resistant dike and the climate dike. Sometimes the deltadike is affiliated with the breach-resistant sea dike introduced by Edelman (1954). In other cases a certain degree of robustness is awarded to the deltadike by increasing the dike width (Vellinga et al., 2009; Schreuder, 2008; Alterra, 2011) or increasing the strength of the revetment (Silva en Van Velzen, 2008). Several studies have been performed to quantify the costs and benefits of such dike reinforcement measures but this did not result in a uniform perspective.

In short, this study addresses the following problem: ‘The deltadike concept is still a vague concept; there is no consensus about the meaning of the concept and about the dike reinforcement measures which turn a dike into a deltadike’.

This problem is addressed by answering the main research question of this study: ‘In what way can a dike be transformed into a deltadike by various dike reinforcement alternatives under different types of loading conditions?’

This research question asks for a definition of the deltadike, subsequently it is possible to determine what reinforcement measures turn a dike into a deltadike.

Definition of the deltadike

The primary function of any dike, also a deltadike, is the prevention of damage and loss of life due to flooding of the polder. A dike fulfils this primary function by retaining water. Dike failure is the malfunctioning of a dike, in other words the passage of a volume of water that leads to damage or loss of life in (part of) the polder. This is caused by either structural or non-structural failure. Structural failure occurs when dike damage is followed by an initial breach (local lowering of the crest level) proceeded by an actual breach. The structural failure mechanisms considered in this research are ‘overflow and/or overtopping’ and ‘piping’. Non-structural failure occurs when overflow and/or overtopping without dike breaching results in a volume of water in the polder that it results in flood damage and/or loss of life.
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The previously cited description of the deltadike by the Delta Committee implicitly states that a deltadike should have a low probability of failure and either announced failure or controllable failure. The reason to put requirements on not only the probability of failure is that this may reduce the consequences of failure. This distinguishes deltadikes from dikes. A dike reduces flood risk by imposing requirements to the probability of failure, while a deltadike claims to reduce flood risk by imposing requirements aimed at reducing the probability of failure and the consequences of failure. The consequences of failure are low either when failure can be predicted with certainty or when the physical consequences (for example the flood depth) of failure are mild (not severe). Failure is predictable with certainty when both the load during a loading event and the strength of a dike can be predicted with certainty. Therefore a dike must meet the following deltadike requirements in order to be considered a deltadike:

1. Low probability of failure.
2. Low consequences of failure by having either:
   - Predictable failure (load and strength predictable with certainty) or
   - Mild failure (small physical consequences).

Approach to meet the deltadike requirements

In case a dike does not fulfil the deltadike requirements, dike reinforcement measures may (partly) fulfil these requirements by firstly significantly decreasing the probability of failure and secondly significantly increasing the predictability of failure or significantly increasing the mildness of failure. The considered reinforcement measures (strengthened dike elements) are:
- Increasing the height of the crest (increases the crest level).
- Increasing the width of the dike (increases the crest width and seepage length).
- Increasing the strength of the revetment (increases the critical discharge).
- Increasing the width of the piping berm (increases the seepage length).

Depending on the context, the text in this summary refers either to the actual reinforcement measure or to the increase of the strength of a certain dike element.

The effect of the reinforcement measures has been computed for three case studies in coastal, estuarine and riverine loading conditions. This has been done using two models. The Dike Reliability Model, developed and validated by the author, is used to calculate the probability of failure and certainty of the strength of both the initial dike and the reinforced dikes. The breach growth model in sand-dikes (Visser, 1998) is used to calculate the influence of the increase of the crest width. It turned out that reinforcement measures affect the probability of failure, certainty of the strength and severity of non-structural failure by strengthening dike elements. They do not affect the certainty of the load and hardly affect the severity of structural failure.
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The probability of failure

The probability of failure is only significantly reduced by reinforcement measures which both (1) tackle failure mechanisms which significantly contribute to the initial probability of failure and (2) significantly reduce the probability of failure of this (these) failure mechanism(s). In case the initial probability of failure is evenly distributed over the failure mechanisms, the probability of failure is only significantly reduced when the strength regarding all failure mechanisms is increased. If the initial probability of failure is unevenly distributed over the failure mechanisms, the probability of failure may be significantly reduced by increasing the strength regarding one specific failure mechanism.

The predictability of failure

The predictability of failure is determined by the certainty of the load in advance of a loading event and certainty of the strength of a dike. The certainty of the load is mainly dependent the type of loading conditions. The wave heights and water levels along the coast cannot be predicted with certainty as they are partly determined by the uncertain development of the wind speed and wind direction. The water levels in the rivers are determined by the discharge of the river, which can be predicted with certainty as it is determined by the volume of water which enters the rivers far upstream and the distribution of the discharge over the different river branches. The wave heights and water levels in estuaries are determined by the water level at sea and the discharge of the rivers. Therefore, the load on a river dike can be predicted with certainty and the load on a sea dike cannot be predicted with certainty in advance. The load on an estuary dike can be predicted with certainty in case it is dominated by the river discharge. The load on an estuary dike cannot be predicted with certainty in case it is dominated by the water level at sea. However, it is not known in advance if either the sea or river is going to dominate the load; therefore the load on an estuary dike cannot be predicted with certainty. The certainty of the strength is increased by reinforcement measures which increase the domination of relatively certain elements of the dike strength. The strength regarding the failure mechanism ‘piping’ is fairly uncertain. The strength regarding the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’ is fairly certain for dikes designed for and loaded by small waves (river dikes) and fairly uncertain for dikes designed for and loaded by large waves (sea dikes).

The severity of failure

The severity of failure generally depends on the type of failure and the loading conditions. Structural failure is generally severe (not mild) and the severity of structural failure is barely influenced by reinforcement measures. Non-structural failure differs in severity and the severity of non-structural failure depends on the loading conditions, the polder area and the strength of the dike. As the strength of a dike plays a role, the severity of non-structural failure is influenced by reinforcement measures (this is discussed further on in this summary). In case of estuary and sea dikes, a high discharge over the crest (for example 10 or 50 l/s/m) is unlikely to result in severe non-structural failure as the loading period is limited. Furthermore,
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At these overtopping rates, a small increase in the water level (a few decimetres) results in a small increase in the severity of non-structural failure. In case of river dikes a high discharge over the crest is more likely to result in severe non-structural failure as the loading period is prolonged. Furthermore, at these overtopping rates, a small increase in the water level (a decimetre) results in a large increase in the severity of non-structural failure. Therefore the severity of failure can be decreased by increasing the dominance of non-structural failure and decreasing the severity of non-structural failure.

Transforming a dike into deltadike

Several entrenched views on the deltadike concept are not supported by this study. First, non-structural failure may result in flood damage of the same order of magnitude as structural failure. The severity of non-structural failure mainly depends on the duration of overflow and/or overtopping and the maximum rate of overflow and/or overtopping, the length of the primary dike and the size of the flooded area. This study did however not investigate the effect of varying these characteristics. Second, the deltadike is not per definition a wider dike (which increases the crest width and the seepage length). It turned out that increasing the crest width does not make any dike breach-resistant. Increasing the dike width therefore has the same effect as widening the piping berm, which requires less soil and space. Third, the deltadike is not per definition a dike with stronger dike revetment (a higher critical discharge). This reinforcement measure contributes the first deltadike requirement, the significance of this contribution depends on the type of loading conditions (which is returned to further on in this summary). It furthermore only contributes significantly to the second deltadike requirement in case of a sea dike (which is discussed further on in this summary). Lastly, increasing the crest level of a dike is in most cases of great importance in transforming a dike into a deltadike. This is because it significantly contributes to the first deltadike requirement by increasing the strength regarding failure mechanisms which in most cases contribute significantly to the probability of failure. Increasing the crest level contributes to the second deltadike requirement by decreasing the severity of non-structural failure as it decreases both the period of time during which overflow and/or overtopping takes place and the rate at which it takes place.

River dikes

A river dike can be transformed into a deltadike. The first deltadike requirement is fulfilled by increasing the crest height and increasing the width of the piping berm. Strengthening the revetment does not significantly decrease the probability of failure; river dikes with a higher critical discharge are able to withstand water levels which are only slightly higher (a decimetre) as these dikes are designed for and loaded by low waves. The second deltadike requirement is fulfilled by increasing the certainty of the strength as the load on a river dike can be predicted with certainty while the severity of failure is generally large. The increase of the certainty of the strength is obtained by increasing the width of the piping berm (increasing the
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Seepage length) to such an extent that the uncertainty of the strength regarding the failure mechanism ‘piping’ does not dominate failure anymore. Increasing the critical discharge should also not be applied in view of the second requirement as it leads to unpredictability of the flood damage caused by non-structural failure; at discharges over the crest which are higher than 1 l/s/m only a small increase in water level within the margins of natural variations (for example several centimetres) results in a significant increase in flood damage caused by non-structural failure as the loading period is already prolonged and both the loading period and the discharge over the crest are very sensitive to a change in water level.

*Estuary dikes*

An estuary dike cannot be transformed into a deltadike. The first deltadike requirement is fulfilled by increasing the crest height, increasing the width of the piping berm and increasing the strength of the revetment. Increasing the critical discharge does significantly decrease the probability of failure; estuary dikes with a higher critical discharge are able to withstand water levels which are significantly higher (a few decimetres) as these dikes are designed for and loaded by moderate waves. The second deltadike requirement cannot be fulfilled. The load cannot be predicted with certainty as it is influenced by the coastal loading conditions; therefore the predictability of failure cannot be increased. The severity of non-structural failure is moderate as the loading time is moderate and both the loading period and the discharge over the crest are moderately sensitive to a change in water level. The dominance of structural failure cannot be significantly decreased because the dike is designed for and loaded by moderate waves.

*Sea dikes*

A sea dike can be transformed into a deltadike. The first deltadike requirement can be fulfilled by increasing the crest height and increasing the strength of the revetment. Increasing the strength of the revetment significantly decreases the probability of failure; sea dikes with a higher critical discharge are able to withstand water levels which are significantly higher (several meters) as these dikes are designed for and loaded by high waves. Increasing the seepage length does not affect the first deltadike requirement as the failure mechanism ‘piping’ does not play a significant role in the failure of sea dikes. The second requirement is fulfilled by increasing the dominance of mild non-structural failure over severe structural failure. This is done by significantly increasing the critical discharge. Non-structural failure is generally mild as the loading period is short and both the loading period and the discharge over the crest are insensitive to a change in water level.

Recommendations

In case the research is continued it is recommended include the influence of the predictability and severity of failure on the consequences; in other words to perform the study in a risk-based framework. A follow-up research project could investigate how cost-effective a deltadike actually is compared to a dike. This study could
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include the combination of reinforcement measures and multi-functional land use. In relation to the previous points, new research could be aimed at investigating which real-life river dikes could actually be cost-effectively transformed into a deltadike based on a risk-based design method.
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<td>$\alpha$</td>
<td>Factor to include the limited thickness of the sand layer [-]</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Bulk damage factor [-]</td>
</tr>
<tr>
<td>$a_i$</td>
<td>Damage factor of category i [-]</td>
</tr>
<tr>
<td>$Abrz$</td>
<td>Area of the breach zone [$m^2$]</td>
</tr>
<tr>
<td>$Ap$</td>
<td>Area of the polder [$m^2$]</td>
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<tr>
<td>$Arrz$</td>
<td>Area of the rapid rising zone [$m^2$]</td>
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<tr>
<td>$\beta$</td>
<td>Scale parameter of the Gumbel distribution [-]</td>
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<td>$bl$</td>
<td>Berm level [m+NAP]</td>
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<tr>
<td>$bw$</td>
<td>Berm width [m]</td>
</tr>
<tr>
<td>$c$</td>
<td>Parameter Sellmeijer formula [-]</td>
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<td>$cl$</td>
<td>Crest level [m+NAP]</td>
</tr>
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<td>$cw$</td>
<td>Crest width [m]</td>
</tr>
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<td>$D_0$</td>
<td>Thickness cohesive top layer [m]</td>
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<tr>
<td>$D_l$</td>
<td>Thickness sand layer [m]</td>
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<td>$d_{70}$</td>
<td>70-Percentile grain distribution sand layer [-]</td>
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<td>$dh$</td>
<td>Hydraulic (piezometric) head over the dike [m]</td>
</tr>
<tr>
<td>$dh_c$</td>
<td>Critical (piezometric) hydraulic head over the dike [m]</td>
</tr>
<tr>
<td>$dp$</td>
<td>Water depth in the polder [m]</td>
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<td>$dp_c$</td>
<td>Critical water depth in the polder [m]</td>
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<td>Final water depth in the polder [m]</td>
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<td>Maximum water depth in the polder (in view of the height of the secondary flood defences) [m]</td>
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<td>$dwl$</td>
<td>Water level difference over the dike [m]</td>
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<td>$F_d$</td>
<td>Mortality fraction [-]</td>
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<td>$F_{brz}$</td>
<td>Mortality fraction in the breach zone [-]</td>
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<tr>
<td>$g$</td>
<td>Constant of gravity [m$^2$/s]</td>
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<td>$H_{mo}$</td>
<td>Spectral wave height [m]</td>
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<td>$Hd$</td>
<td>Design wave height [m]</td>
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<td>$i$</td>
<td>Index of the damage categories</td>
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<td>$k_{zb}$</td>
<td>Permeability of the sand layer [m/s]</td>
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<td>$Lbr$</td>
<td>Length of the breach [m]</td>
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<td>$L_{br_{i=0}}$</td>
<td>Length of the initial breach [m]</td>
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<td>$Lds$</td>
<td>Length dike section [m]</td>
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<td>$L_{mo-1,a}$</td>
<td>Spectral deep water wave length [m]</td>
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<td>$Ls$</td>
<td>Seepage length [m]</td>
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<td>$MHW$</td>
<td>Mean High Water</td>
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<td>Model factor of the critical discharge over the crest [-]</td>
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<tr>
<td>$mq$</td>
<td>Model factor of the discharge over the crest [-]</td>
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<td>Model factor of the critical hydraulic head [-]</td>
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<td>$N$</td>
<td>Total number of damage units</td>
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<td>$NAP$</td>
<td>Amsterdam Ordnance Datum (reference level)</td>
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<td>$N_{exp}$</td>
<td>Exposed population to flooding [number of people]</td>
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<td>$N_{exp;brz}$</td>
<td>Exposed population to flooding in the breach zone [number of people]</td>
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<tr>
<td>$N$</td>
<td>Loss of life [people]</td>
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<tr>
<td>Symbol</td>
<td>Description</td>
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<td>-----------</td>
<td>------------------------------------------------------------------------------</td>
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<tr>
<td>$N_{sf}$</td>
<td>Loss of life due to breaching [number of people]</td>
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<tr>
<td>$N_{nssf}$</td>
<td>Loss of life due to non-structural failure [number of people]</td>
</tr>
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<td>$n$</td>
<td>Number of bulk damage units [-]</td>
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<td>Probability (annual) of exceedance of the maximum water level [-]</td>
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<td>$P_f$</td>
<td>Probability (annual) of failure of the dike section [-]</td>
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<tr>
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<td>Probability (annual) of structural failure of the dike section [-]</td>
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<td>$P_{nssf}$</td>
<td>Probability (annual) of non-structural failure of the dike section [-]</td>
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<td>$P_{sfofot}$</td>
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<td>$P_{sfpiping}$</td>
<td>Annual probability of failure of the dike section of the failure mechanism 'piping' [-]</td>
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<td>$P_{wl_{max}}$</td>
<td>Probability of occurrence of a certain maximum water level [-]</td>
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<td>Discharge over the crest (due to overtopping and/or overflow) [l/s/m]</td>
</tr>
<tr>
<td>$Q$</td>
<td>Discharge into the polder (due to breaching or overflow and/or overtopping) [m$^3$/s]</td>
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<tr>
<td>$q_c$</td>
<td>Critical discharge over the crest (Tolerable discharge regarding the erosion of the dike crest and dike inner slope) [l/s/m]</td>
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<tr>
<td>$q_{of}$</td>
<td>Discharge over the crest due to overflow [l/s/m or m$^3$/s/m]</td>
</tr>
<tr>
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<td>Discharge over the crest due to overtopping and overflow [l/s/m or m$^3$/s/m]</td>
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<tr>
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<td>Maximum discharge over the crest due to overtopping [l/s/m or m$^3$/s/m]</td>
</tr>
<tr>
<td>$R$</td>
<td>Resistance of the dike (or strength of a dike element)</td>
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<tr>
<td>$r_{brz}$</td>
<td>Radius of the breach zone [m]</td>
</tr>
<tr>
<td>$R_c$</td>
<td>Freeboard [m]</td>
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<tr>
<td>$R_{c_{min}}$</td>
<td>Freeboard at maximum water level [m]</td>
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<tr>
<td>$S$</td>
<td>Stress on the dike (or load imposed on a dike element)</td>
</tr>
<tr>
<td>$S$</td>
<td>Flood damage [€]</td>
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<tr>
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<td>Bulk maximum damage per damage unit [€ / unit]</td>
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<td>$s_i$</td>
<td>Maximum damage per damage unit in category i [€ / unit]</td>
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<tr>
<td>$t$</td>
<td>Time [h]</td>
</tr>
<tr>
<td>$T$</td>
<td>Period during which the discharge Q flows into the polder [s]</td>
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<td>$T_{m-1.0}$</td>
<td>Spectral wave period [s]</td>
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<tr>
<td>$T_{br}$</td>
<td>Period from the initial breach to an actual breach [h]</td>
</tr>
<tr>
<td>$tana$</td>
<td>Inclination of the front face of the structure [-]</td>
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<tr>
<td>$V_p$</td>
<td>Volume of water in the polder [m$^3$]</td>
</tr>
<tr>
<td>$wl$</td>
<td>Still water level water body [m+NAP]</td>
</tr>
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<td>$wld$</td>
<td>Design water level [m+NAP]</td>
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<td>$wl_{max}$</td>
<td>Maximum water level during a loading event [m+NAP]</td>
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<td>Water level in the polder [m+NAP]</td>
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<td>Rise rate in the breach zone [m/h]</td>
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<td>$w_{rrz}$</td>
<td>Rise rate in the rapid rising zone [m/h]</td>
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<td>$w_{p}$</td>
<td>Rise rate in the polder [m/h]</td>
</tr>
<tr>
<td>$Z_{sf_{ofot}}$</td>
<td>Limit state function of the failure mechanism 'overflow and/or overtopping’</td>
</tr>
</tbody>
</table>
List of symbols

\[ Z_{sf_{\text{piping}}} \] Limit state function of the failure mechanism ‘piping’
\[ Z_{nsf} \] Limit state function of the failure mechanism ‘non-structural failure’

\[ \alpha \] Factor to include the limited thickness of the sand layer [-]
\[ \beta_{\text{piping}} \] Reliability index of the failure mechanism ‘piping’ [-]
\[ \beta_{\text{ofot}} \] Reliability index of the failure mechanism ‘overflow and/or overtopping’ [-]
\[ \gamma_{b} \] Influence factor for the berm [-]
\[ \gamma_{\beta} \] Influence factor for the wave angle of incidence [-]
\[ \gamma_{cti} \] Apparent volumetric weight of the wet cohesive top layer [kN/m\(^3\)]
\[ \gamma_{r} \] Influence factor for the roughness of the outer slope [-]
\[ \gamma_{s} \] Volumetric weight of the sand grains [kN/m\(^3\)]
\[ \gamma_{v} \] Influence factor for the vertical wall [-]
\[ \gamma_{w} \] Volumetric weight of the water [kN/m\(^3\)]
\[ \eta \] Drag force factor (in other studies also referred to as the coefficient of White) [-]
\[ \theta \] Rolling resistance angle of the sand grains [\(^\circ\)]
\[ \kappa \] Intrinsic permeability of the sand layer [m\(^2\)]
\[ \mu \] Location parameter of the Gumbel distribution [m+NAP]
\[ \nu \] Kinematic viscosity of water at ten degrees Celsius [m\(^2\)/s]
\[ \xi_{m-1.0} \] Iribarren number (in other studies also referred to as ‘Surf similarity number’ or ‘Breaker parameter’) [-]
Chapter
Introduction

This chapter starts with an analysis of the current problems faced in deltadike design. It subsequently sets the definition and requirements of the deltadike, the objective of the study, the research questions, the research methodology and the research assumptions. This chapter finally presents an outline of the report.

I.I. Problem analysis

The aftermath of the flood in 1953 resulted in the first explicit attempt to compute and implement an optimum flood safety level by an economic optimization of the crest level (Van Dantzig, 1956) based on flood risk. As part of this optimization, the probability of failure was based on both the crest level and the exceedance frequencies of the water level (Van Dantzig and Kriens, 1960).

At that time, it was not yet possible to quantify the flood safety related to geotechnical failure mechanisms; the strategy was therefore to suppress the occurrence of all failure mechanisms except for overtopping. Dike reinforcements are still designed this way, which is referred to as the overload approach. In this approach overload is defined as the occurrence of more discharge over the dike due to overtopping (the passage of water over the dike crest due to the process of waves) than the critical discharge. The critical discharge is the discharge which is still resisted by the revetment of the crest and inner slope of the dike. The basis of the overload approach is that a dike suffices the standard when (TAW, 1998):

1. The probability of exceedance of the critical discharge by the overtopping discharge is equal or lower than the safety level stated in the Dutch law (Waterwet, 2009). The probability of exceedance of this overtopping discharge is equal to the probability of exceedance of the associated design water level for which the overtopping discharge is equal to the critical discharge. The overtopping discharge for a certain water level is calculated by determining the design wave height together with the characteristics of the outer slope.
2. The probability of failure of failure mechanisms not related to overtopping has to be smaller than 10% of the probability of exceedance mentioned under point 1 for the cases that the water level is equal or lower than the design water level.

Recent findings regarding dike design

Nowadays, it is possible to evaluate flood risks in greater detail due to the introduction of computer power and the increased availability of data (Voortman, 2003). Over time we are also increasing our understanding of the failure mechanisms. This has recently led to several findings regarding dike design in The Netherlands. Firstly, the project ‘Flood Risk and Safety in The Netherlands’ (Projectbureau VNK2, 2011b), abbreviated FLORIS, showed that the distribution of the probability of failure over the failure mechanisms is not in accordance with the overload approach (Projectbureau VNK2, 2011a). Secondly, it became apparent that
the consequences of flooding strongly depend on the flood pattern and the distribution of values in the flooded area. The flood pattern depends on the location of the breach, the physical-geographical properties of the dike ring area and the loading conditions during the breach (Projectbureau VNK2, 2011b). Thirdly, several studies have indicated that flood risks continue to grow in terms of both economic damage (Jonkman et al., 2008a) and loss of life (Jonkman et al., 2008b; Maaskant et al., 2009). The project ‘Flood Protection 21st Century’ investigated to what extent the level of flood protection agrees with the property and people protected by the flood defences. The project showed that The Netherlands is underinsured regarding the protection against flooding (Kind, 2011; Eijgenraam, 2011). The physical causes of this are a combination of subsiding land, rising sea levels and peak discharges of rivers (Parry et al., 2009) in addition to the increased damage potential.

The deltadike concept

In short, the identification of methodological, demographical, economical and physical changes in dike design led to a rising awareness that the dikes in The Netherlands are not safe enough. The second deltacommittee responded to these developments by advising to increase the safety level of all dikes with at least a factor ten and to implement a renewed risk approach. The committee further advised to investigate the concept of deltadikes which are ‘either so high or so wide and massive that the probability that these dikes will suddenly and uncontrollably fail is virtually zero’. Furthermore the committee advised to consider the deltadike concept for dikes with a probability of failure which has to be decreased by more than a factor ten (Delta Committee, 2008).

The interpreted deltadike concept

The different names associated with the deltadike depict the current situation of vagueness and ambiguity:

- The broad dike is a multifunctional dike of several hundred meters wide which can withstand a considerable amount of erosion during extreme conditions (Vellinga, 2008; Vellinga et al., 2009; Schreuder, 2008; Alterra, 2011).
- The super levee is a multifunctional dike which reduces flood damage by preventing a dike breach (Arakawa-Karyu River Office, 2007).
- The overtopping-resistant dike is a dike able to withstand overtopping rates of about 30 l/s/m (Silva and Van Velzen, 2008).
- The climate dike is a multifunctional, overflow-resistant dike which also provides safety when the climate changes in the future (Smolders, 2010).

In some studies (including Silva and Van Velzen, 2008) the deltadike is obtained by dike reinforcements which reduce the probability of failure significantly (by a factor ten or hundred), other studies still refer to a breach-resistant character (Arakawa-Karyu River Office, 2007; Vellinga, 2008; Van der Sande, 2009; De Bruijn & Klijn, 2011). Sometimes the deltadike is associated with the breach-resistant sea dike introduced by Edelman (1954) after the disastrous flood of 1953. This flood was
very disastrous due to the occurrence of dike breaches. In response to this flood Edelman stated that the safety of a dike should be independent of the crest level. Sometimes the deltadike definitions award a certain degree of robustness to the deltadike (Alterra, 2011) without clear definition of the property ‘robust’. Other reports state that deltadikes contribute to taking climate change into account (Planbureau voor de Leefomgeving, 2011) without explaining in what way this is done. Several studies included quantifications of the costs and benefits of dike reinforcement measures in view of the deltadike concept (Silva and Van Velzen, 2008; Smolders, 2010; Klijn and Bos, 2010; De Bruijn & Klijn, 2011; Knoeff and Ellen, 2011). These studies did however not lead to clear or uniform results.

**Problem definition**

The performed studies resulted in relevant insights but did not result in a uniform perspective on the deltadike concept. Furthermore, the studies do not agree about what reinforcement measures are necessary to turn a dike into a deltadike. In short, this study addresses the following problem: *The deltadike concept is still a vague concept; there is no consensus about the meaning of the concept and about the dike reinforcement measures which turn a dike into a deltadike.*

### 1.2. Definition and requirements of the deltadike

The primary function of any dike, also a deltadike, is the prevention of damage and loss of life due to flooding. Dike failure is the malfunctioning of a dike; in other words the passage of a volume of water that leads to damage or loss of life in (part of) the polder. A dike fulfills its primary function by retaining water. The original definition of the deltadike by the second Delta Committee (2008) refers both to a low probability of failure and to the ability to predict and control failure. The reason to put requirements on the failure behaviour is that such failure reduces the consequences of flooding. A deltadike thus not only takes the probability of failure into account, but also the consequences of failure. This is what distinguishes deltadikes from dikes. A dike reduces flood risk by imposing requirements on the probability of failure, while a deltadike claims to reduce flood risk by imposing requirements on both the probability of failure and the consequences of failure.

#### 1.2.1. Probability of failure

The second Delta Committee stated that the deltadike may be an appropriate solution when the flood protection level should be increased by more than a factor ten (Delta Committee, 2008). This statement is relative to the current flood protection level of dikes in The Netherlands. These dikes therefore do not meet the deltadike requirement of the probability of failure. The flood protection level of dikes should therefore be increased by at least a factor ten in order to obtain the title
‘deltadike’. The probability of failure is reduced by strengthening dike elements via dike reinforcement measures.

1.2.2. Consequences of failure

The consequences of failure are reduced by increasing the predictability of failure and decreasing the severity of failure. Predictable failure provides extra time to get property and people to safety. Mild failure reduces the impact of failure on the exposed property and people. The rapidity of the failure process itself is not included in the predictability of failure as it is difficult to quantify; the residual strength is difficult to quantify (see section 2.1.2), the speed of the breaching process is difficult to quantify (see section 4.2.2) and history shows that the failure mechanisms of a dike may all occur quickly. The failure mechanisms ‘Overflow and/or overtopping’ and ‘Piping’ may rapidly develop into failure (Rijkswaterstaat & KNMI, 1961 and section 2.1.2)

The predictability of failure

The predictability of failure is based on the predictability of the strength and the load (in this study represented by the maximum water level). The predictability is expressed into the certainty of the strength and the load in advance of a loading event. Figure 1 shows that the predictability of failure increases when either one of the two becomes more certain. The lower graph shows two situations; in one situation the load can be predicted with more certainty than in the second situation. The upper graph shows a dike with a strength which can be predicted quite certainly and a dike with a strength which cannot be predicted quite certainly. Both graphs together correspond to each other through the maximum water level. Together they show that dike failure is more predictable when the load is predictable with more certainty and/or when the strength of a dike is more certain.
Chapter Introduction

Figure 1 | The probability of failure of the dike section ($P_f$) conditional on the maximum water level ($w_{l_{\text{max}}}$) and the probability of exceedance of the maximum water level ($P_{\text{exc}}$) in advance of a loading event.

Dike reinforcements do not affect the certainty of the load in advance of a loading event; however, they do affect the certainty of the dike strength. The certainty of the dike strength is expressed by the fragility, which is the probability of failure conditional on a specific loading (Van der Meer et al., 2009b). The fragility is indicated by the fragility curve (see Figure 2). The steeper the fragility curve is, the higher the fragility of a dike and the more certain the dike strength.

Figure 2 | Fragility curves indicating more certain and less certain strength ($P_f$ refers to the probability of failure of the dike section, $w_{l_{\text{max}}}$ refers to maximum water level during a loading event).

The severity of failure

The severity of failure refers to the physical effect which failure has on the polder expressed in flood depth or flow velocities. In this study, the severity of failure is
indicated by the final water depth in the polder. Therefore a high upper bound of the curve shown in Figure 3 implies that the effect of failure is severe.

![Figure 3 | Graph indicating severe and mild failure (dp refers to water depth in the polder, t refers to time)](image)

1.2.3. Deltadike requirements

A dike must meet the following requirements to be appointed to deltadike:

1. Low probability of failure.
2. Low consequences of failure by having either:
   - Mild failure (small physical consequences) or
   - Predictable failure (load and strength predictable with certainty).

These requirements are referred to as the deltadike requirements. In case a dike does not meet these requirements, they may be (partly) fulfilled by dike reinforcement measures. The previous subsection showed which graphs are suitable indicators of these requirements (Figure 2 and Figure 3). These indicating graphs are used throughout this study to determine to what degree a dike reinforcement measure contributes to the achievement of the deltadike requirements.

1.3. Objective and research questions

The objective of this research is to define the deltadike concept and to determine which reinforcement measures can transform a dike into a deltadike under different loading conditions. The main research question is: ’In what way can a dike be transformed into a deltadike by various dike reinforcement alternatives under different types of loading conditions?’

This main research question is composed of the following sub-questions:

1. ’What is the deltadike concept and what are the requirements of a deltadike?’
2. ’What causes dikes to fail and how are dikes designed?’
3. ’In what way can the deltadike requirements be achieved?’
4. ’What is the effect of strengthening dike elements?’
5. ’What is the effect of dike reinforcement measures when performing actual dike reinforcements under different loading conditions?’
1.4. Methodology

The hypothesis and conclusions were composed, tested, revised and refined by literature review, studying (delta)dike design and performing (hypothetical) case studies. The (hypothetical) case studies were performed using the sequential case study approach (Verschuren & Doorewaard, 2005); in this approach the lessons learnt from foregoing case studies are applied in the subsequent case studies. When looking at this methodology in retrospect, the process was rather chaotic but it still contained the elements of the initial methodology (see Figure 4).

![Figure 4](image.png)

Figure 4 | Methodology in advance of the study (in blue) compared to the rather chaotic methodology in retrospect of the study (in orange)

1.5. Assumptions

This study considers anthropogenic dikes in The Netherlands. These dikes are generally made of traditional construction materials such as peat, sand and clay. Objects such as trees, houses, roads, cables and pipes are not taken into account. The use of modern techniques such as sensors in dikes is not included in this study as these techniques are currently not implemented in the design of dikes. The considered type of dike is the primary dike (the dikes along the major waterways, lakes and seas). Secondary dikes (the dikes along the minor waterways) are not considered. Multifunctional land use (in terms of houses or other facilities on the dike) is not taken into account as this study focuses on the flood protection aspects of the deltadike.

This study focuses on reinforcement measures only; it does not consider emergency or recovery measures. Reinforcement measures are actions taken in anticipation of an event (pre event) in order to prevent flooding. Emergency measures are actions
taken during an event (during event) in order to prevent flooding. Recovery measures are action taken after an event (post event) in order to restore the dike.

The reinforcement measures are considered individually as combining them would unnecessarily increase the complexity and extent of this study. Furthermore, the reinforcement measures are only considered on dike section level. This is because dike failure somewhere along the dike section results in about the same consequences. Interactions with other dike sections or areas within the dike ring are not considered as the focus of this study is on the effect of dike reinforcement measures on the dike section itself.

It is furthermore assumed that decreasing the severity of failure and increasing the predictability of failure (by increasing the certainty of the strength and load) actually reduces the consequences of failure. The relationship between the second deltagrive requirement and the consequences of failure is not studied.

1.6. Outline report

This report is subdivided into the introduction, the core of the report, the conclusions, the recommendations and lastly the appendix. Each chapter in the core of the report answers a sub-question; the answer is briefly summarized at the end of each core chapter in the concluding remarks. Chapter 2 discusses dike failure and dike design. Chapter 3 provides an overview of the way in which the deltagrive requirements can be achieved. Subsequently chapter 4 discusses the effect of strengthening dike elements on the probability, predictability and severity of failure. Chapter 5 provides an overview of the effectiveness of reinforcement measures in different types of loading conditions. The report finishes with the conclusions (chapter 6) and recommendations (chapter 7). Appendix A describes the dike reinforcement model (DRM), developed by the author, and the input of the model. Appendix B describes the excluded damage mechanisms.
Dike failure and dike design are of great importance for sound deltadike design. This chapter hence addresses the physical processes underlying dike failure (section 2.1) and dike design (section 2.2). By doing so, this chapter addresses the second sub-question: 'What causes dikes to fail and how are dikes designed?'

2.1. Dike failure

Dike failure is directly related to the primary function of a (delta)dike, which is the prevention of damage and loss of life due to flooding. A (delta)dike fulfils its primary function by retaining water. Dike failure is the malfunctioning of a dike, in other words the passage of a volume of water leading to damage or loss of life in (part of) the polder. The passage of water is caused by the failure mechanisms ‘piping’ or ‘overflow and/or overtopping’ followed by dike breaching (structural failure) or the process of overflow and/or overtopping itself without dike breaching (non-structural failure). Structural failure generally leads to failure while non-structural failure may lead to failure.

Non-structural failure is generally not considered in dike design. This is because it is avoided by constructing dikes such that the overtopping rate is limited to 0.1, 1 or 10 l/s/m during design conditions. Such an amount of water over the dike can be pumped away or stored in the polder. The deltadike concept is associated with increasing the strength of the revetment (increasing the critical discharge), which basically means allowing more water to flow over the dike without structural failure during an extreme event. This possibly results in a volume of water in the polder that results in flood damage or loss of life. Therefore non-structural failure is considered in this study.

This chapter first discusses the fault tree analysis and subsequently the two types of failure (structural and non-structural).

2.1.1. Fault tree analysis

The fault tree analysis is based on failure mechanisms, which are combinations of events leading to failure (see Figure 5). Each box represents an event. Some events include a box with ‘R<S’ or ‘R>S’ inside, which refers to Resistance (or strength) and Stress (or load) regarding an initiating damage mechanism. Only the events with these boxes are included in the computation of the probability of failure as the others cannot be modelled (accurately enough) or because the others cannot be based on a limit state function. As long as the strength is greater than the load, the dike element does not fail.

The naming of the failure mechanisms is based on the damage mechanisms or start events of the fault tree (lowest row of boxes), this is because the damage mechanisms or start events largely determine what the following events are:
1. Overflow and/or overtopping (for the boxes 'overtopping', 'overflow' and 'overtopping and overflow' below the event 'structural failure');
2. Piping (for the box 'formation pipe short circuit from polder to river' below the event 'structural failure');
3. Non-structural failure (for the boxes 'overtopping', 'overflow' and 'overtopping and overflow' below the event 'non-structural failure').

One of the assumptions is that structural failure is independent of non-structural failure; therefore the probabilities of these events together constitute the probability of failure:

\[ Pf = Psf + Pnsf \]

Where:
- \( Pf \) = Annual probability of failure of the dike section [-]
- \( Psf \) = Annual probability of structural failure of the dike section [-]
- \( Pnsf \) = Annual probability of non-structural failure of the dike section [-]

Another assumption is that the failure mechanisms 'overflow and/or overtopping' and 'piping' are independent; the probability of failure therefore becomes equal to:

\[ Pf = Psf + Pnsf = Psf_{ofat} + Psf_{piping} + Pnsf \]

Where:
- \( Pnsf_{ofat} \) = Annual probability of failure of the dike section due to the failure mechanism 'overflow and/or overtopping' [-]
- \( Pnsf_{piping} \) = Annual probability of failure of the dike section due to the failure mechanism 'piping' [-]
2.1.2. Structural failure

Structural failure is the result of the development of dike damage into a dike breach through which water flows into the polder. The structural failure trajectory describes this transition from a functioning dike to a malfunctioning dike (see Figure 6). It should be noted that Figure 6 represents an absolute view as dike failure is not bound to certain stages; all stages may even occur simultaneously. The reason for
presenting it like this is that the interpretation of the deltadike concept sometimes builds upon the notion of residual strength or the breaching process.

Figure 6 | Structural failure trajectory of a dike (after adjustments partly adopted from Van Gerven, 2004)

A damage mechanism, such as overtopping or piping, is the first step to failure (stage 1). A damage mechanism causes damage to a dike, which occurs only when the load on a dike element is higher than the strength of that element. The damage mechanisms are described further on in this subsection.

The follow-up damage mechanism may occur between the initiating damage mechanism and the breaching process. The follow-up mechanism is resisted by the residual strength. The residual strength refers to the ability of a dike to fulfil the water-retaining function after the occurrence of an initiating damage mechanism. A follow-up damage mechanism may be the same mechanism as an initiating damage mechanism; its occurrence is related to the damage and loading conditions of initiating damage mechanism. Therefore the residual strength of a dike is also related to both the strength and loading conditions of the initiating damage mechanism. The follow-up mechanisms are described further on in this subsection.

The breaching process starts when a follow-up mechanism results in a local decrease in crest level (an initial breach, the start of stage 3). This process is described further on in this subsection.

Damage mechanisms

This section describes damage mechanisms of dikes. Some damage mechanisms are excluded beforehand as they do not immediately result in danger of flooding. More
details regarding the exclusion of each damage mechanism are found in Appendix A. The beforehand excluded damage mechanisms discussed in this Appendix are:

- Attack outer slope by wave action.
- Horizontal displacement dike body.
- Erosion foreshore.
- Drifting ice.
- Objects and animals.
- Terrorist attacks.

The description of the remaining damage mechanisms is based on reports written by TAW (2001), Voortman (2003), Calle and Knoeff, (2002a, 2002b), Allsop (2007) and ENW (2010). The analyzed damage mechanisms are:

- Sliding inner slope.
- Overflow and/or overtopping.
- Micro-instability.
- Piping.
- Liquefaction foreshore.
- Sliding outer slope.

It turned out that many of the above-listed analyzed damage mechanisms do not result in direct danger of flooding. In short, several damage mechanisms were excluded from the fault tree analysis:

- Sliding inner slope.
- Uplift inner slope revetment followed by micro-instability.
- Liquefaction foreshore.
- Sliding outer slope.
- Attack outer slope by wave action.

Although sliding of the inner slope may directly result in failure, this damage mechanism has also been excluded. From the FLORIS project it became clear that the contribution of this damage mechanism to the total probability of failure is minor in comparison to the damage mechanisms ‘overflow and/or overtopping’ and ‘piping’. Dikes in The Netherlands generally have an inner slope which is milder than 1:3 (v:h). This makes the slope less vulnerable to sliding. It is also for this reason that overflow and/or overtopping followed by infiltration and sliding is not a governing damage mechanism anymore (Van der Meer, 2009). Liquefaction may also directly result in failure. This damage mechanism is excluded as it is not a very likely damage mechanism in the Netherlands anymore for two reasons. First, many vulnerable embankments have been fixed already. Second, steep slopes in rivers are not likely to occur, as they are regulated by groynes, river cut-offs and guide walls. Sliding of the outer slope is related to dropping water levels. Therefore there is no direct danger of flooding after this damage mechanism occurs. It is very likely that it is possible to restore the dike before the second extreme loading condition occurs.

In the end the analyzed damage mechanisms which are part of the fault tree are therefore:

- Overflow and/or overtopping.
Piping.

The remainder of this subsection describes these damage mechanisms. The sketches are distorted in scale. The undistorted scale is shown in Figure 7.

**Overflow and/or overtopping**

This section describes the damage mechanism ‘overflow and/or overtopping’ (see Figure 8).

Overtopping (which is the same as wave-overtopping) is the discharge of water over the crest of a dike due to the presence of waves. Overflow is the discharge of water over the crest of the dike due to a water level higher than the crest level. The combination of overflow and overtopping may also result in discharge over the crest.

Water over the crest of a dike possibly causes damage by infiltration or erosion of the inner slope. Infiltrating water increases the weight of the soil, which is the driving force of sliding. Erosion occurs when discharging water takes along soil particles. The strength of the grass cover depends on the quality of the grass and
the condition of the root system (Steendam et al., 2010). The erosion process usually starts with initial damage of weakened bonds (Hoffmans, 2008) which then develops downward by ripping of the grass cover. The erosion resistance of bare clay is considerably lower than the grassed cover (Van der Meer et al., 2006, Akkerman, 2007; Van der Meer et al., 2009a).

Due to either sliding or erosion, the sandy dike core is now exposed to surface erosion. As overtopping and overflow already take place, the residual strength is relatively low. Consequently there is also no distinct follow-up mechanism. In the remainder of this report sliding is not considered anymore as it is assumed that erosion is dominant over sliding based on the same reasons as the exclusion of the damage mechanism ‘sliding of the inner slope’.

**Piping**

This part of the subsection describes the damage mechanism ‘piping’ (see Figure 9). A more descriptive name would be ‘uplift and bursting of the cohesive top layer followed by the transport of grains’. The mechanism is extensively described by ENW (2010). There are several cases of the transport of sand through pipes underneath the dike occur regularly, sand boils have been observed along the river dikes in 1980 (COW, 1980), 1982 (COW, 1982), 1993 (TAW, 1994) and 1995 (TAW, 1995). In 1926 the process was spotted along The IJssel (translated from Calle & Knoeff, 2002a): ‘On 8 January 1926, at about half past seven a small well was discovered by the dike-guard near Zalk, it delivered clean water. On the river side of the dike there was a deep, poorly-coated gully present. In spite of the fact that the well was nothing to worry about, a group of officials from Public Works decided to cover the well with gravel. The companionship had just left the place when the remaining dike-guard came running with the message that the dike was collapsing. When the group turned around they saw a mud-fountain of about the size of a man at the location of the spotted well.’

![Figure 9 | Sketch of the damage mechanism ‘piping’](image)

This damage mechanism is initiated by uplift of the cohesive top layer. The next step is bursting of the cohesive top layer, which is breaking of top layer of the soil at the polder side of the dike. Bursting may directly follow uplift as cohesive soils swiftly crack when they are deformed. In some cases there is no cohesive top layer at the polder side; in these cases the occurrence of uplift considers the liquefaction of the top sand layer at the polder side through which water flows out vertically creating
quicksand (referred to as heave). Uplift and heave are grouped in one as both processes consider water which flows below the dike from the water body to the polder area.

In the cases that uplift or heave has occurred, water flows from the river through the permeable soil below the cohesive top layer to the polder area. In case the flow velocities are high enough, transport of sand takes place as well. The transport of grains creates small underground pipes, which start at the polder side and grow into the direction of the river. Eventually the pipes reach the river side creating a short circuit between the river and the polder (ENW, 2010). When this short circuit is formed, the pipes below the dike experience accelerated growth. The transport of sand grains stops when the flow velocities through the pipes decrease (as the pipes grow).

Over time, piping destabilizes the dike. This possibly locally decreases the crest level of the dike (an initial breach) which marks the start of the breaching process.

Breaching process

The breaching process has several stages. The initiation of the breaching process is marked by lowering of the crest due to the damage caused in stage 1 and 2 (see previous section). Due to the lowering of the crest, the amount of overflow and/or overtopping increases. A dike breach is generally modelled by a broad-crested weir (Visser, 1998). In the beginning the broad-crested weir is free-flowing and the discharge grows linearly with the width of the dike. At some point the flow turns into submerged flow; the discharge starts to depend on the hydraulic head as well as the water in the polder rises. When the hydraulic head decreases, the water velocity decreases and the breach growth decreases as well. At some point in time the breach growth stops. The discharge through the breach stops when the polder water level is equal to the water level of the water body. The discharge through the breach depends on the breach size in both vertical and horizontal sense.

This process has been modelled by Visser (1998) for sand dikes. Most primary dikes in the Netherlands have a sand core, which makes the model generally applicable to this study. Five stages are distinguished in the model (see Figure 10):
1. Local steepening of the inner slope at the eroding location;
2. Retrograde erosion of the inner slope through the dike;
3. Accelerated growth of the depth and the width of the breach and subsequently the discharge through the breach;
4. Further growth in the lateral directions;
5. Decreasing growth due to decelerating flow as the water level in the polder rises which finally results in equilibrium.
The model of Visser has been applied in this study to compute the period of time between an initial breach and an actual breach (see section 4.2) and to quantify the flood depth over time due to breaching (see chapter 5).

**Limit state functions**

This subsection provides an overview of the limit state functions of the structural failure mechanisms and their underlying models. The limit state functions are partly based on the theory manual of PC-RING (Vrouwenvelder and Steenbergen, 2003a; Vrouwenvelder and Steenbergen, 2003b; Steenbergen et al., 2008).

**Overflow and/or overtopping**

The failure mechanism ‘overflow and/or overtopping’ is assumed to occur when the limit state ‘erosion of the inner slope by overflowing or overtopping water’ is exceeded. The limit state function for erosion of the inner slope is defined as (Steenbergen et al., 2008; TAW, 2002):

\[ Zsf_{ofot} = R - S = m_{qc} \cdot q_c - m_q \cdot q \]

Where:

- \( Zsf_{ofot} \) = Limit state function of the failure mechanism ‘overflow and/or overtopping’
- \( R \) = Resistance of the dike
- \( S \) = Stress on the dike
- \( m_{qc} \) = Model factor of the critical discharge over the crest [-]
- \( q_c \) = Critical discharge over the crest (Tolerable discharge regarding the erosion of the dike crest and dike inner slope) \([m^3/s/m]\)

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Figure 10 | Five stages (stage 1 comprises the period between \( t_0 \) and \( t_1 \), stage 2 comprises the period between \( t_1 \) and \( t_2 \), etc.) in the breaching process of a sand-dike (copied from Visser, 1998)
Dike failure and dike design

\[ m_q = \text{Model factor of the discharge over the crest [-]} \]
\[ q = \text{Discharge over the crest [m}^3\text{/s/m]} \]

Several models are used to describe the processes of overflow and/or overtopping. The formula of Henderson (1966) describes the process of overflow for positive freeboard:

\[ q_{of} = \left(\frac{2}{3}\right)^{\frac{3}{2}} g \left(-R_c\right)^{\frac{3}{2}} \]

Where:
\[ q_{of} \quad = \text{Discharge over the crest due to overflow [m}^3\text{/s/m]} \]
\[ g \quad = \text{Constant of gravity [m}^3\text{/s]} \]
\[ R_c \quad = \text{Freeboard [m]} \]

The freeboard is the difference between the crest level and the water level:

\[ R_c = cl - wl \]

Where:
\[ wl \quad = \text{Still water level water body [m+NAP]} \]
\[ cl \quad = \text{Crest level [m+NAP]} \]

The formula of Van der Meer (Pullen et al., 2007, formula 5.9) describes the process of overtopping for positive freeboard:

\[ q_{ot} = \sqrt{g \cdot H_{m0}^3 \cdot \frac{0.067}{\tan \alpha} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot e^{-4.75 \cdot \left(\frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_f \cdot \gamma_b \cdot \gamma_v}\right)^2}} \]

Where:
\[ q_{ot} \quad = \text{Discharge over the crest due to overtopping [m}^3\text{/s/m]} \]
\[ H_{m0} \quad = \text{Spectral wave height [m]} \]
\[ \tan \alpha \quad = \text{Inclination of the front face of the structure [-]} \]
\[ \gamma_b \quad = \text{Influence factor for the berm [-]} \]
\[ \xi_{m-1,0} \quad = \text{Iribarren number [-]} \]
\[ \gamma_f \quad = \text{Influence factor for the roughness of the outer slope [-]} \]
\[ \gamma_b \quad = \text{Influence factor for the wave angle of incidence [-]} \]
\[ \gamma_v \quad = \text{Influence factor for the vertical wall [-]} \]

With a maximum of:

\[ q_{ot,max} = \sqrt{g \cdot H_{m0}^3 \cdot 0.2 \cdot e^{-2.6 \cdot \left(\frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_f \cdot \gamma_b \cdot \gamma_v}\right)^2}} \]

Where:
\[ q_{ot,max} \quad = \text{Maximum discharge over the crest due to overtopping [m}^3\text{/s/m]} \]
The Iribarren number is calculated using the following formula:

$$\xi_{m-1.0} = \frac{\tan \alpha}{(H_{mo}/L_{m-1.0})^{1/2}}$$

Where:
- $\xi_{m-1.0}$ = Iribarren number [-]
- $L_{m-1.0}$ = Spectral deep water wave length [m]

The spectral deep water wave length is calculated using the following formula:

$$L_{m-1.0} = \frac{g * T_{m-1.0}^2}{2 * \pi}$$

Where:
- $T_{m-1.0}$ = Spectral wave period [s]

The formula of Schüttrumpf (Pullen et al., 2007 formula 5.14) describes the process of overtopping for zero freeboard:

- For $\xi_{m-1.0} < 2.0$:
  $$q_{ot} = \left(\frac{g * H_{mo}^3}{m_0 * 0.0537 * \xi_{m-1.0}}\right)$$

- For $\xi_{m-1.0} \geq 2.0$:
  $$q_{ot} = \left(\frac{g * H_{mo}^3}{m_0 * \left(0.136 - \frac{0.226}{\xi_{m-1.0}}\right)}\right)$$

Finally the formula of Hughes and Nadal (2009, formula 17) describes the process of combined overflow and overtopping for negative freeboard:

$$q_{ofot} = \sqrt{g * H_{mo}^3 * \left(0.034 + 0.53 * \left(\frac{R_C}{H_{mo}}\right)^{1.58}\right)}$$

Where:
- $q_{ofot}$ = Discharge over the crest due to overtopping and overflow [m$^3$/s/m]

The formulas for overflow ($q_{of}$), overtopping ($q_{ot}$) or overflow and overtopping ($q_{ofot}$) are combined in order to obtain a formula for overflow and/or overtopping ($q$) for a specific setting (depending on the wave height, characteristics of the outer slope and freeboard). This combination is based on an exponential least-squares fit of the relevant formulas. In case such a fit is used, a graph is shown of the applicable abovementioned formulas and the fit.

**Piping**

The damage mechanism of piping occurs when the limit state of both bursting of the cohesive top layer and formation of a short circuit is exceeded. It is assumed that
the clay layer bursts when it is lifted up as clay layers are cohesive and usually crack when they are deformed. Another assumption is that the sub-damage mechanism uplift (uplifting of the land side clay layer) is fully correlated with the formation of a short circuit. In other words, the probability of failure of the failure mechanism ‘piping’ is equal to the probability of occurrence of a short circuit. The limit state function for the formation of piping is therefore defined as (TAW, 1999; Steenbergen et al., 2008):

$$Z_{sf_{piping}} = R - S = m_z \cdot d_{h_c} - (d_{h} - 0.3 \cdot D_0)$$

Where:

- $Z_{sf_{piping}}$ = Limit state function of the failure mechanism ‘piping’
- $m_d$ = Model factor of the critical hydraulic head [-]
- $d_{h_c}$ = Hydraulic (piezometric) head over the dike [m]
- $d_{h}$ = Critical hydraulic (piezometric) head over the dike [m]
- $D_0$ = Thickness cohesive top layer [m]

In which the hydraulic (piezometric) head over the dike is calculated using the following formula:

$$d_{h} = w_{l} - w_{lp}$$

Where:

- $w_{lp}$ = Water level in the polder [m+NAP]

The formula of Sellmeijer is used to calculate the critical piezometric head over the dike:

$$d_{h_c} = \alpha \cdot c \cdot L_s \cdot \left( \frac{\gamma_s - \gamma_w}{\gamma_w} \right) \cdot (0.68 - 0.1 \cdot \ln(c)) \cdot \tan(\theta)$$

Where:

- $\alpha$ = Factor to include the limited thickness of the sand layer [-]
- $c$ = Parameter Sellmeijer formula [-]
- $L_s$ = Seepage length [m]
- $\gamma_s$ = Volumetric weight of the sand grains [kN/m$^3$]
- $\gamma_w$ = Volumetric weight of the water [kN/m$^3$]
- $\theta$ = Rolling resistance angle of the sand grains [$^\circ$]

The seepage length is a characteristic of the flow of water beneath the dike; it is the distance between the entry point at the water body side and the exit point at the polder side as shown in Figure 11.
In this formula, the parameter $\alpha$ is calculated using the following formula:

$$\alpha = \left( \frac{D_1}{L_s} \right)^{\frac{0.28}{\left( \frac{D_1}{L_s} \right)^2}}$$

Where:

- $D_1$ = Thickness sand layer [m]

The parameter $c$ is calculated as follows:

$$c = \eta \cdot d_{70} \cdot \left( \frac{1}{\kappa \cdot L_s} \right)^{1/3}$$

Where:

- $\eta$ = Drag force factor [-]
- $d_{70}$ = 70-Percentile grain distribution sand layer [-]
- $\kappa$ = Intrinsic permeability of the sand layer [m$^2$]

The intrinsic permeability of the sand layer is calculated using the following formula:

$$k = \frac{\nu}{g} \cdot k_{z,b}$$

Where:

- $\nu$ = Kinematic viscosity of water at ten degrees Celsius [m$^2$/s]
- $k_{z,b}$ = Permeability of the sand layer [m/s]

2.1.3. Non-structural failure

Non-structural failure occurs when the processes overflow and/or overtopping result in a volume of water in the polder leading to flood damage or loss of life, see Figure 12. The three processes resulting in discharge over the crest are explained in 2.1.2 below the heading 'Overflow and/or overtopping'.

The volume of water that is allowed to enter the polder and the volume of water that actually results in failure of the dike depends on the characteristics of the polder...
itself, such as flood patterns, polder size, water storage in the polder, the use of pumps, the value and population of the land and the effect of the flood on the value and the population.

Figure 12 | Mechanisms resulting in non-structural failure

Non-structural failure occurs when the water depth in the polder is higher than the critical water depth. The critical water depth is the minimum water depth for which flood damage and/or loss of life occurs. The limit state function for non-structural failure is thus defined as:

\[
Z_{nsf} = d_p - d_{final}
\]

Where:
- \(Z_{nsf}\) = Limit state function of the failure mechanism ‘non-structural failure’
- \(d_p\) = Critical water depth in the polder [m]
- \(d_{final}\) = Final water depth in the polder [m]

It would have been better to include model factors in this limit state function as well, as the critical water depth is estimated based on logic and as the final water depth is calculated based on models of the processes of overflow and/or overtopping. As the computations were already performed when this was discovered this was not implemented.

It is assumed that non-structural failure takes place when the uniform flood depth in the polder is greater than one decimetre (the critical water depth). This depth is chosen because it is roughly equal to the average doorstep height. Furthermore, water depths lower than one decimetre correspond to heavy rainfall for half a day in
2.2. Dike design

Dikes are generally designed based on a combination of one design still water level and one design wave height near the dike (shown in Figure 13). The dike is designed such that the discharge over the crest is equal to the critical discharge during the normative loading conditions.

Figure 13 | Dike design parameters (the still water level is the water level in absence of wind-induced waves)

Discharge over the crest is caused by either one of the three processes shown in Figure 14. Altogether, these processes are referred to as overflow and/or overtopping.

Figure 14 | Processes which result in discharge over the crest

In any circumstance, overflow or overflow combined with overtopping is not desirable. This is because for discharges over the crest in the order of 1 to 10 l/s/m, a very small change in water level rapidly results in flooding of the polder. For example at a discharge over the crest of 1 l/s/m, an increase of the water level by 10 cm (for example due to natural variations) increases the discharge over the crest to 50 l/s/m; another 10 cm to 150 l/s/m. In section 3.3.2 it turns out that this rate may result in severe flooding. The relationship between the freeboard and the discharge over the crest for overflow is shown in Figure 15.
Therefore, dike design is based on a maximum overtopping rate (critical discharge), for example 1 l/s/m or 10 l/s/m. In case the dike is loaded by a high water level in combination with a small wave height, this requires only small freeboards. A high water level in combination with a large wave height requires large freeboards. This is visualized by Figure 16 and Figure 17 which consider the design of a dike with a small and large design wave height.

The relationship between the freeboard and the discharge over the crest is modelled in formulas described in subsection 2.1.2. This relationship for a wave height of 0.1 m and a grassed slope with an inclination of 1:3 (v:h) is shown in Figure 18. The relationship is a least-squares fit of the models which describe the processes of overtopping and combined overflow and overtopping. This fit is also performed for a
design wave height of 0.5 m and 5 m. The result is shown in Figure 19. Figure 19 shows that in case the design wave is smaller, the sensitivity of the discharge over the crest to an increase of the water level is larger.

Figure 18 | Least square exponential fit of the discharge over the crest ($q$) and the freeboard ($R_c$) on the formulas of Van der Meer (Pullen et al., 2007), Schüttrumph (Pullen et al, 2007) and Nadal and Hughes (2009) for a design wave height of 0.1 m

Figure 19 | Effect of the design wave height ($H_d$) on the relationship between the freeboard ($R_c$) and the discharge over the crest ($q$)
The analysis in this section showed that dike design is strongly related to the type of loading condition. In case a dike is loaded by and designed for high waves, the dike generally has a large freeboard during extreme loading conditions. In this case the discharge over the crest is quite insensitive to a change in water level. In case a dike is loaded by and designed for low waves, the dike generally has a small freeboard during extreme loading conditions. In this case the discharge over the crest is very sensitive to a change in water level.
Chapter
Approach to meet the deltadike requirements

This chapter provides an overview of the way in which the following deltadike requirements can be fulfilled by dike reinforcement measures:

1. Low probability of failure.
2. Low consequences of failure by having either:
   - Mild failure (small physical consequences) or
   - Predictable failure (load and strength predictable with certainty).

In case a dike does not possess the properties of a deltadike yet, dike reinforcement measures may (partly) fulfil these requirements by significantly both decreasing the probability of failure and either decreasing the severity of failure or increasing predictability of failure. By doing so, this chapter addresses the third sub-question of this research: ‘In what way can the deltadike requirements be achieved?’

3.1. The probability of failure

The probability of failure of a dike section can be decreased by reinforcement measures. To what extent of the decrease depends on:

1. The (diminishing) effectiveness of the reinforcement measure in reducing the probability of failure of the failure mechanism(s) it tackles.
2. The (diminishing) contribution of the failure mechanism(s) tackled by the reinforcement measure to the probability of failure.

The first point is specific for each reinforcement measure while the second point is generic for all reinforcement measures. The first point is discussed in the next chapter. The second point is discussed in this section; it refers to the impact of the reduction of the probability of failure of the failure mechanism(s) on the probability of failure. The important issues regarding this point are:

- The initial distribution of the probability of failure over the failure mechanisms.
- The law of diminishing returns.

3.1.1. The initial distribution of the probability of failure over the failure mechanisms

The initial distribution of the probability of failure over the failure mechanisms is the relative contribution of each failure mechanism to the probability of failure. The contribution of the probability of failure of the failure mechanism ‘overflow and/or overtopping’ to the probability of failure could for example be 70%. In that case it may be very efficient to increase the crest level or the critical discharge, as these reinforcement measures both increase the resistance against the failure mechanism ‘overflow and/or overtopping’ (see Figure 20).
3 Chapter
Approach to meet the deltadike requirements

Figure 20 | Distribution of the annual probability of failure of the dike section ($P_f$) which is favourable for reinforcement measures reducing this probability of failure of the failure mechanism ‘overflow and/or overtopping’ ($P_{sf_{ofot}}$) ($P_{nsf}$ refers to this probability of failure of the failure mechanism ‘non-structural failure’ and $P_{sf_{piping}}$ refers to this probability of failure of the failure mechanism ‘piping’)

On the other hand, measures which tackle the failure mechanism ‘overflow and overtopping’ are very inefficient when the contribution of failure mechanism ‘overflow and/or overtopping’ to the probability of failure is only minor (as shown in Figure 21).

Figure 21 | Distribution of the annual probability of failure of the dike section ($P_f$) which is unfavourable for reinforcement measures reducing this probability of failure of the failure mechanism ‘overflow and/or overtopping’ ($P_{sf_{ofot}}$) ($P_{nsf}$ refers to the probability of failure of this failure mechanism ‘non-structural failure’ and $P_{sf_{piping}}$ refers to this probability of failure of the failure mechanism ‘piping’)

3.1.2. The law of diminishing returns

The law of diminishing returns states that the marginal revenue of an investment decreases as the investment itself increases. The effectiveness of a reinforcement measure in decreasing the probability of failure is also subject to the law of diminishing returns. But only in a special way, as dike reinforcements are not about marginal revenues, but about the reduction of a probability of failure. Furthermore, in the case of Dutch dikes, the probability of failure should be reduced by a significant factor. The reduction is thus automatically linked to the initial probability of failure. The significant factor is for example 10 or 100.
In case the probability of failure is mainly due to one failure mechanism, a
significant reduction is obtained by addressing that single failure mechanism. This is
shown in Figure 22. The other failure mechanism should contribute less than 1% in
order to be able to reduce the probability of failure by a factor 100.

![Figure 22](image)

**Figure 22** | Significant reduction of the annual probability of failure of the dike section
\((P_f)\) with an uneven distributed initial probability of failure \((P_{\text{ofot}})\) refers to this
probability of failure of ‘overflow and/or overtopping’, \(P_{\text{nfs}}\) refers to this probability of
failure of the failure mechanism ‘non-structural failure’ and \(P_{\text{piping}}\) refers to this
probability of failure of the failure mechanism ‘piping’

In case the probability of failure is evenly distributed over the failure mechanisms, a
significant reduction is only obtained when encountering all failure mechanisms by
combining reinforcement measures. This is shown in Figure 23. In this study
combinations of reinforcement measures are however not considered as this would
unnecessarily complicate the report.

![Figure 23](image)

**Figure 23** | Significant reduction of the annual probability of failure of the dike section
\((P_f)\) with an even distributed initial probability of failure \((P_{\text{ofot}})\) refers to this probability of
failure of ‘overflow and/or overtopping’, \(P_{\text{nfs}}\) refers to this probability of failure of the
failure mechanism ‘non-structural failure’ and \(P_{\text{piping}}\) refers to this probability of failure of
the failure mechanism ‘piping’

### 3.2. The predictability of failure

The predictability of failure is determined by the certainty of the load in advance of a
loading event and certainty of the strength in advance of the loading event.
3.2.1. **The certainty of the load**

The certainty of the load is mainly dependent on the type of loading conditions. The wave heights and water levels along the coast cannot be predicted with certainty as they are partly determined by the uncertain development of the wind speed and wind direction. The water levels in the rivers are determined by the discharge of the river, which can be predicted with certainty as it is determined by the volume of water which enters the rivers far upstream and the distribution of the discharge over the different river branches. The wave heights and water levels in estuaries are determined by the water level at sea and the discharge of the rivers. Therefore, the load on a river dike can be predicted with certainty and the load on a sea dike cannot be predicted with certainty in advance. The load on an estuary dike can be predicted with certainty in case it is dominated by the river discharge. The load on an estuary dike cannot be predicted with certainty in case it is dominated by the water level at sea. However, it is not known in advance if either the sea or river is going to dominate the load. Therefore, the load on an estuary dike cannot be predicted with certainty.

3.2.2. **The certainty of the strength**

The certainty of the strength depends on the certainty of the strength regarding the dominant failure mechanisms. In case failure mechanisms dominate of which the strength is uncertain, the certainty of the dike strength is low. In case failure mechanisms dominate of which the strength is certain, the certainty of the dike strength is high. The certainty of the strength is expressed by the probability of dike failure given a certain load. This is also referred to as the fragility of a dike (Van der Meer et al., 2009b). The fragility of a dike is indicated by the aggregated fragility curve (see Figure 24). The aggregated fragility curve is the sum of the fragility curves of the failure mechanisms. The aggregated fragility curve is always left to the fragility curves as it is the sum of them. The steeper the aggregated fragility curve is, the more certain the strength becomes.

![Figure 24](image)

**Figure 24** | Composition of the aggregated fragility curve ($P_f$ refers to the probability of failure of the dike section, $wl_{max}$ refers to the maximum water level during a loading event)
The fragility of the dike is increased by reinforcement measures which increase the domination of the aggregated fragility curve by the failure mechanisms which have a more certain strength. Reinforcement measures do not influence the steepness of the fragility curves of the failure mechanisms as this steepness depends on the uncertainty of the strength of the dike and not on the greatness of the strength. The steepness of the aggregated fragility curve therefore mainly depends on the domain of each of the contributing fragility curves. The domain of these fragility curves is mainly influenced by dike reinforcement measures.

The remainder of this subsection discusses the effect of reinforcement measures on both the steepness of the (aggregated) fragility curve(s).

The domain of the fragility curves

Reinforcement measures change the domain of the fragility curves, whereby both the domain and the steepness of the aggregated fragility curve may change. As an example the possible effect of increasing the seepage length on the aggregated fragility curve in Figure 24 is shown in the resulting fragility curve in Figure 25.

![Figure 25](image)

**Figure 25** | Plausible effect of increasing the crest level on the fragility curves and aggregated fragility curve ($P_f$ refers to the probability of failure of the dike section, $w_{l_{max}}$ refers to the maximum water level during a loading event)

Only the domain of the fragility curve of the failure mechanism ‘piping’ is changed which increases the steepness of the aggregated fragility curve. This is desirable as it increases the fragility of the dike. The plausible effect of each reinforcement measure on the domain is discussed in chapter 4. The domain of the fragility curves mainly depends on the case-specific load and strength. Therefore several cases are studied in chapter 5.

The steepness of the fragility curves

The steepness of the fragility curve of the failure mechanism ‘overflow and/or overtopping’ depends on the type of loading condition. In case a dike is designed for and loaded by high waves, the sensitivity of the discharge over the crest (the load) to a change in water level is much smaller. Therefore the fragility curve of dikes designed for large waves is much milder (domain of several meters) than the
fragility curve of dikes designed for small waves (domain of several centimetres or decimetres) as shown in Figure 26.

![Figure 26](image)

**Figure 26** | Steep and mild fragility curves of the failure mechanism ‘overflow and/or overtopping’ (*P_f* refers to probability of failure of the dike section, *w_{l_{\text{max}})* refers to maximum water level during a loading event)

Regarding this failure mechanism, the load is represented by the water level difference between the water body and the polder (hydraulic head over the dike). The relationship between the freeboard and the load is therefore linear. Whether or not a load causes failure is determined by the critical hydraulic head. As the uncertainty of the critical hydraulic head is generally quite large, the fragility curve of the failure mechanism ‘piping’ is therefore in general quite mild (see Figure 27).

![Figure 27](image)

**Figure 27** | Fragility curve of the failure mechanism ‘piping’ (*P_f* refers to probability of failure of the dike section, *w_{l_{\text{max}})* refers to maximum water level during a loading event)

The reasoning on the steepness of the fragility curve of the failure mechanism ‘non-structural failure’ is the same as the reasoning on the steepness of the fragility curve of the failure mechanism ‘overflow and/or overtopping’. This is because the underlying process of overflow and/or overtopping is the same. This distinction is therefore also found in the height of the design wave as shown in Figure 28.
The analysis in this subsection showed that the failure mechanism ‘piping’ is fairly uncertain in general. The failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’ are both fairly uncertain for dikes designed for and loaded by high waves, while they are fairly certain for dikes designed for and loaded by small waves.

3.3. The severity of failure

The severity of failure generally depends on the type of failure and the loading condition. Structural failure is generally severe and the severity of structural failure is barely influenced by reinforcement measures. This is because the flood depth, rise rate and flow velocities in case of a dike breach are already very high. In case a dike would for example fail at a higher water level, the severity of the breach (which is already severe) only increases slightly. Non-structural failure differs in severity and the severity of non-structural failure depends on the loading conditions, the polder area and the strength of the dike. As the strength of a dike plays a role, the severity of non-structural failure is influenced by reinforcement measures; this effect is discussed in the next chapter. Therefore the severity of failure can be decreased by increasing the dominance of non-structural failure (which is indicated by fragility curves, see chapter 4) and decreasing the severity of non-structural failure. The objective of this section is thus to get hold of the severity of structural and non-structural failure is regarding the flood depth, flood damage and loss of life for different loading conditions.

This is done on the basis of an example of a polder protected by a primary dike, as shown in Figure 29 and Figure 30. The polder area is 10 km². The difference between the dike crest level and the polder level is 3.5 meter. Due to the height of the secondary dikes inside the polder, the physical maximum water depth in the polder is 3 meter. The slopes of the dike have an inclination of 1:3 (v:h). The
primary dike section length is 5 km. The distribution of the land use and the population density is discussed further on in the subsections 3.3.3 and 3.3.4.

Figure 29 | Top view polder area

Figure 30 | Cross-section dike (the house is meant to illustrate the dimensions, $d_{p\text{ max}}$ refers to the maximum water depth in the polder)

3.3.1. Structural failure

In case the dike undergoes structural failure, this generally results in large flood depths in the whole polder. The breaching process is described in section 2.1.2. Several assumptions are made in order to make a simple calculation at the same level of detail as the other calculations in this subsection. It is assumed that the breach can be modelled by the broad-crested weir formula of Henderson (1966), a constant hydraulic head of 2 meter and a constant breach width of 300 meter. The breach discharge is therefore:

$$Q = L_{br} \times \left(\frac{2}{3}\right)^{\frac{1}{2}} \times \sqrt{g} \times d_{wl}^{\frac{3}{2}} = 300 \times \left(\frac{2}{3}\right)^{\frac{1}{2}} \times \sqrt{9.81} \times 2^{\frac{3}{2}} \approx 1,500 \text{ m}^3/\text{s}$$

Where:

$L_{br}$ = Length of the breach [m]

$d_{wl}$ = Water level difference over the dike [m]
The water level difference over the dike is calculated as follows:

\[ dwl = wI - wlp \]

The volume of water in the polder is:

\[ V_p = Q \times T = 1,500 \times (6 \times 3,600) \approx 32 \times 10^6 \text{ m}^3 \]

The final water depth in the polder is:

\[ dp_{\text{final}} = \min \left( \frac{V_p}{A_p} \right) = \min \left( 3 \frac{32 \times 10^6}{10 \times 10^6} \right) = 3 \text{ m} \]

Where:
- \( dp_{\text{final}} \) = Final water depth in the polder [m]
- \( dp_{\text{max}} \) = Maximum water depth in the polder [m]

The rise rate in the polder is:

\[ wp = \frac{Q}{A_p} = \frac{1,500 \times 3,600}{10 \times 10^6} \approx 0.5 \text{ m/h} \]

The breach zone is the zone close to the breach where the flow velocities and rise rates are very high. It is assumed that in the breach zone the flow velocity is greater than 2 m/s and the product of the water depth and the flow velocity is greater than 7 m²/s. In this case the radius of the breach zone can be calculated using the analytical formula as proposed by Jonkman (2007):

\[ r_{brz} = \frac{1}{\pi \times 7} \times Q = 0.045 \times Q = 0.045 \times 1500 \approx 70 \text{ m} \]

Where:
- \( r_{brz} \) = Radius of the breach zone [m]

The area of the breach zone is:

\[ A_{brz} = L_{br} \times r_{brz} + \frac{1}{2} \pi r_{brz}^2 = 300 \times 70 + 0.5 \pi \times 70^2 \approx 29,000 \text{ m}^2 \]

Where:
- \( A_{brz} \) = Area of the breach zone [m²]
The rise rate in the breach zone is:

\[
\text{\(w_{brz}\)} = \frac{Q}{A_{brz}} = \frac{Q}{29,000} = \frac{1,500 \times 3600}{29,000} \gg \text{[m/h]}
\]

Where:

\text{\(w_{brz}\)} = \text{Rise rate in the breach zone [m/h]}

As shown in Figure 32, the erosion gap is located at the dike location itself; other possible locations are in front of the dike (in case of a foreshore) or behind the dike (for more information see Visser, 1998). The location affects the discharge through the breach. The discharge also depends on the course of the water level, the breach growth process, the location of the breach and the physical-geographical properties of the polder.

3.3.2. Non-structural failure

Suppose the primary dike undergoes non-structural failure. The amount of water which enters a polder area by overflow or overtopping depends mainly on the
maximum discharge over the crest, the development of the load over time and the length of the primary dike. It is independent of both the size and the depth of the polder, except for the case in which the water level in the polder becomes higher than the crest level. As the development of the load over time differs for different loading conditions (coastal, estuarine and riverine), the following analysis distinguishes a coastal dike and a river dike. The coastal dike is loaded for 2 hours while the river dike is loaded for 48 hours. It is assumed that the discharge over the crest is constant and equal to 10 l/s/m. It should be noted that this short investigation does not consider pumping excess water back into the water body.

**Sea dike**

First the coastal dike is considered. The discharge into the polder is:

\[ Q = q \times L_{ds} = 10 \times 5000 = 50,000 \text{ [l/s]} = 50 \text{ [m}^3/\text{s]} \]

Where:

- \( Q \) = Discharge into the polder [m\(^3\)/s]
- \( q \) = Discharge over the crest (per running meter) [m\(^3\)/s/m]
- \( L_{ds} \) = Length dike section [m]

The rise rate in the polder is:

\[ \frac{w_p}{A_p} = \frac{Q}{A_p} = \frac{50 \times 3600}{10 \times 10^6} = 0.018 \text{ [m/h]} \approx 0.02 \text{ [m/h]} \]

Where:

- \( w_p \) = Rise rate in the polder [m/h]

The volume of water in the polder protected by the sea dike is:

\[ V_p = Q \times T = 50 \times 3600 \times 2 = 0.36 \times 10^6 \text{ [m}^3\] \]

Where:

- \( V_p \) = Volume of water in the polder [m\(^3\)]
- \( T \) = Period during which the discharge \( Q \) flows into the polder [s]

The final water depth in the polder protected by the sea dike is:

\[ dp_{final} = \min \left( dp_{max}, \frac{V_p}{A_p} \right) = \min \left( 3, \frac{0.36 \times 10^6}{10 \times 10^6} \right) = \min (3, 0.036) = 0.036 \text{ [m]} \approx 0.04 \text{ [m]} \]

Where:

- \( dp_{final} \) = Final water depth in the polder [m]
- \( dp_{max} \) = Physical maximum water depth in the polder [m]
- \( A_p \) = Area of the polder [m\(^2\)]
The situation is illustrated by Figure 33 and Figure 34.

A small increase of the water level, for example due to natural variations, results in small increase of the final flood depth. When the water level increases by 0.1 m, the discharge over the crest hardly increases (see Figure 19).

**River dike**

The volume of water in the polder protected by the river dike is:

\[ V_p = Q \times T = 50 \times (48 \times 3,600) = 8.6 \times 10^6 \text{ [m}^3]\]

The final water depth in the polder protected by the river dike is:

\[ dp_{final} = \min \left( \frac{V_p}{A_p}, \frac{8.6 \times 10^6}{10 \times 10^6} \right) = \min (3, 0.86) \approx 1 \text{ [m]} \]
The situation is illustrated by Figure 35 and Figure 36.

![Figure 35](image1.png)  
**Figure 35 |** Top view polder area after non-structural failure of a river dike (*\(q\) refers to the discharge over the crest, *\(Q\) refers to the discharge into the polder, *\(T\) refers to the period during which the discharge *\(Q\) flows into the polder, *\(w_{p_{rz}}\) refers to the rise rate in the rapid rising zone, *\(w_{p}\) refers to the rise rate in the polder, *\(d_{p_{final}}\) refers to the final water depth in the polder)

![Figure 36](image2.png)  
**Figure 36 |** Cross-section polder after non-structural failure of a river dike (the house is only meant to illustrate the dimensions, *\(d_{p_{final}}\) refers to the final water depth in the polder)

A small increase of the water level, for example due to natural variations, results in large increase of the final flood depth. When the water level increases by 0.1 m, the discharge over the crest increases from 10 l/s/m to about 50 l/s/m (see Figure 18). In case the calculations above are performed for this discharge over the crest, the final flood depth is 3 meter. Therefore a situation with a high discharge over the crest is generally not desirable in the case of river dikes as only a small increase in flood depth results in a major increase in the flood depth of the polder.

### 3.3.3. Flood damage

The flood damage depends on both the value of the land and the effect of the flood on that value. This is expressed by the following formula (Kok et al., 2005) which sums the damage of several damage categories:
Where:

\[ S = \sum_{i=1}^{N} a_i n_i s_i \]

- \( S \) = Flood damage [€]
- \( i \) = Index of the damage categories
- \( N \) = Total number of damage units
- \( a_i \) = Damage factor of category \( i \) [-]
- \( n_i \) = Number of damage units in category \( i \) [-]
- \( s_i \) = Maximum damage per damage unit in category \( i \) [€ / unit]

The damage category represents a certain land use type which is associated with the flooded area such as recreation or agriculture. The maximum damage per unit in a category is based on averaged values derived from previous studies, see Table 1.

### Table 1 | Maximum damage values (Kok et al., 2005)

<table>
<thead>
<tr>
<th>Damage category</th>
<th>Unit</th>
<th>Maximum damage / unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Houses</td>
<td>piece</td>
<td>€ 172,000</td>
</tr>
<tr>
<td>Roads</td>
<td>m</td>
<td>€ 1,000</td>
</tr>
<tr>
<td>Industry direct damage</td>
<td>jobs</td>
<td>€ 279,000</td>
</tr>
<tr>
<td>Industry indirect damage</td>
<td>jobs</td>
<td>€ 70,000</td>
</tr>
<tr>
<td>Agriculture</td>
<td>m²</td>
<td>€ 2</td>
</tr>
<tr>
<td>Recreation</td>
<td>m²</td>
<td>€ 10.00</td>
</tr>
</tbody>
</table>

The damage factor depends on both the land use and the flood characteristics. The land use determines to what extent the value is affected by certain flood characteristics. Determining flood characteristics are the final water depth, the flow velocities, the presence of waves (the wind speed) and the type of water (salt or fresh and possibly polluted). There are furthermore several types of damage, such as direct material damage to capital, direct business failure, costs of health care, but also non-priced indirect costs such as travel time losses and emotional damage (Gauderis and Kind, 2011).

For example, houses which are not firmly built are affected by large flow velocities. Cars are generally not affected by flooding as long as the water depth is lower than the bottom of the chassis. However the use of roads, and thereby travel time losses, is affected by small water depths. A special case is agriculture which is severely affected by inundation by salt water. The critical salt load during these events depends on the local situation (Bakker, 2003), although there are some generalities. As most of the polders along the sea are brackish, chances are that the crops are salt-tolerant. On the other hand the critical overtopping events are very likely to occur in the storm season, during which the soil is not cultivated, the fresh water river discharges are high and the freshwater needs are small.
This leads to varying damage functions for different types of land use, see Figure 37. The damage functions only consider only one flood characteristic, which is the water depth. The other flood characteristics are implicitly included as the other flood characteristics are related to the water depth. This function is used in the remainder of the chapter.

![Figure 37 | Damage functions (Kok et al., 2005) showing the dependency of the damage factor \( a_i \) on the water depth in the polder \( dp \)](image)

The damage factors of Figure 37 are combined into one bulk damage factor function:

\[
\alpha = 1 - \exp(-0.5 \cdot dp)
\]

Where:

\[
\alpha \quad \text{= Bulk damage factor [-]}
\]

Evenhuis et al. (2007) indicated that not all costs are included in the method of Kok et al. (2005). Excluded or underestimated costs are the repair of water defences, the costs of evacuation, the costs of production loss and the costs of social disruption. The recent floods in New Orleans and Thailand also showed that these costs also affect the indirect damage (the damage outside the flooded area). This indirect damage also occurs for small water depths. Furthermore, small water depths may also result in serious damage (Syncera, 2007). For example, the breach of a secondary dike in Wilnis in 2003 flooded only 0.35 km\(^2\) with a water depth of 0.25 m (Gemeente De Ronde Venen, 2004), which still resulted in a total damage of about 16 million euro (Kok, 2006).
In order to show this consideration, another formula of the damage factor is introduced with two parts, a fixed and variable damage. The fixed damage is independent of the water depth in the polder (flood depth) while the variable flood damage increases with increasing flood damage. The fixed damage can be considerable, therefore a factor of 0.2 of the maximum damage is assumed to be a reasonable fixed damage. In order to keep the development for higher damages in accordance with the damage functions as defined by Kok et al. (2005), this constant factor diminishes to zero as the water depth increases. It is also assumed that damage only takes place when the uniform water depth is higher than 0.1 meter. This depth is chosen because it is equal to the average doorstep height. Furthermore, water depths smaller than one decimetre correspond approximately to heavy rainfall in the Netherlands for half a day which occurs regularly without having an actual flood. The damage factor function valid from a uniform water depth of 0.1 m onwards is (see Figure 38):

\[ \alpha = 0.2 \cdot e^{-0.5 \cdot dp} + (1 - e^{-0.5 \cdot dp}) \]

![Figure 38](image-url)  
*Assumed representative bulk damage factor (\( \alpha \)) function (\( dp \) refers to the water depth in the polder)*

The following example is now given to get an indication of the flood damage for varying flood depths. Suppose that the polder has about the same distribution in land use as the Netherlands (CBS, 2011a): about 70% is used for agriculture; water, recreation, housing, infrastructure, industry and others cover each 5%. In the considered polder this means that 7 km\(^2\) is used for agriculture and 0.5 km\(^2\) for water, recreation, housing, infrastructure, industry and others.
The average population density of a city is about 3000 people/km$^2$ in the Netherlands (CBS, 2012) and the average number of people per house is 2.2 (CBS, 2011b). Based on these numbers the city inhabits about 1500 persons in 700 houses. It is furthermore assumed that the industry area holds about 1000 jobs. The roads are all considered to be 10 meters wide, which makes 50 km of road in the polder.

The average damage per square meter is calculated based on the contribution of each damage category based on the number of units and the maximum damage per unit (see Table 2).

**Table 2 | Calculation of the weighted average maximum damage per square meter (maximum damage from Kok et al., 2005) also referred to as Bulk maximum damage per damage unit ($s$)**

<table>
<thead>
<tr>
<th>Damage category</th>
<th>Unit</th>
<th>Maximum damage / unit</th>
<th>Nr of units</th>
<th>Maximum damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Houses</td>
<td>piece</td>
<td>€ 172,000</td>
<td>700</td>
<td>€ 120,400,000</td>
</tr>
<tr>
<td>Roads</td>
<td>m</td>
<td>€ 1,000</td>
<td>50,000</td>
<td>€ 50,000,000</td>
</tr>
<tr>
<td>Industry direct damage</td>
<td>jobs</td>
<td>€ 279,000</td>
<td>1,000</td>
<td>€ 279,000,000</td>
</tr>
<tr>
<td>Industry indirect damage</td>
<td>jobs</td>
<td>€ 70,000</td>
<td>1,000</td>
<td>€ 70,000,000</td>
</tr>
<tr>
<td>Agriculture</td>
<td>m$^2$</td>
<td>€ 2</td>
<td>7,000,000</td>
<td>€ 10,500,000</td>
</tr>
<tr>
<td>Recreation</td>
<td>m$^2$</td>
<td>€ 10.00</td>
<td>500,000</td>
<td>€ 5,000,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>€ 534,900,000</td>
</tr>
</tbody>
</table>

When the damage function is applied to the example, this results in the following formula for the flood damage (as shown in Figure 39):

\[
S = a \times n \times s = (0.2 + (1 - e^{-0.5dP})) \times 10,000,000 \times 50 = 0.2 \times 10^9 + (10^9 - 10^9 \times e^{-0.5dP})
\]

Where:

- \(n\) = Number of bulk damage units [-]
- \(s\) = Bulk maximum damage per damage unit [€ / unit]
When considering the damage function with a fixed damage, the damage for a water depth of 1 meter is about €250 million, a water depth of 3 meter results in about €400 million in damage.

The analysis in this subsection shows that the flood damage for a flood depth of 1 meter is approximately 1.5 times smaller than in case of a flood depth of 3 meter. This means that in case non-structural failure results in flood depths in the order of 0.5 – 1.5 meter this still results in severe flood damage in the same order of magnitude as the flood damage in case of breaching.

3.3.4. Loss of life

J jonkman (2007) observes three steps in the estimation of the loss of life:

1. Assessment of the physical effects;
2. Determination of the number of people exposed;
3. Estimation of the mortality and loss of life among the exposed people.

With regard to the example of the polder, the first step is already taken (see Figure 33, Figure 34, Figure 31 and Figure 32). Flooding due to dike breaching and flooding due to overflow and overtopping have different physical effects. In both cases two zones are distinguished with differing rise rates. For dike breaching, both the rise rate and the final water depth are higher.
Chapter
Approach to meet the deltadike requirements

In order to take the second step, it is assumed that the event is caused by a sudden increase in wind speed (wave height) at night. The population at risk is therefore equal to the amount of people that are actually living (not working) in the polder; there is furthermore no time available for beforehand decision-making, warning, response and evacuation (Frieser, 2004; Jonkman, 2007). No time is available for preventive evacuation, thus the exposed population is equal to the number of people living in the polder area.

The mortality function is obtained from Jonkman et al. (2009), see Figure 40. The mortality function is based on the hurricane Katrina. Although the mortality is described as a function of the water depth only, the other flood characteristics are implicitly included as the other flood characteristics are related to the water depth. The mortality function averages is derived for the polder area which is not located in the breach zone. The mortality rate in the breach zone is 0.1 (constant). It is assumed that the breach is located in one of the villages of the polder. This means that about 90 persons are exposed to flooding in the breach zone (3000 people/km² and the area of the breach zone equal to 30,000 m²).

![Figure 40](image)

**Figure 40 | Assumed mortality function (obtained from Jonkman et al., 2009) showing the dependency of the mortality fraction ($F_D$ and $F_{D,brz}$) on the water depth in the polder ($dp$)**

When this mortality function is applied to the example, the loss of life for both events is therefore equal to:

$$N_{nsf} = N_{exp} \cdot F_D = 1000 \cdot 0.005 \approx 7 \text{ [number of people]}$$

Where:

$N_{nsf}$ = Loss of life due to non-structural failure [number of people]
3. Approach to meet the deltadike requirements

Exposed population to flooding \[N_{exp}\] [number of people]
Mortality fraction \[F_D\] [-]

\[N_{sf} = N_{exp,br} \times F_{D,br} + N_{exp} \times F_D = 90 \times 0.10 + 1410 \times 0.02 \approx 40\] [number of people]

Where:

Exposed population to flooding in the breach zone \[N_{exp,br}\] [number of people]
Mortality fraction in the breach zone \[F_{D,br}\] [-]

The analysis in this subsection showed that structural failure generally results in more loss of life than non-structural failure. However the loss of life in case of non-structural failure is certainly not negligible.

3.4. Concluding remarks

In case a dike does not fulfil the deltadike requirements, dike reinforcement measures may (partly) fulfil these requirements by significantly both decreasing the probability of failure and either decreasing the severity of failure or increasing predictability of failure. Reinforcement measures affect the probability of failure, the certainty of the strength and the severity of non-structural failure. They do not affect the certainty of the load and hardly affect the severity of structural failure.

The probability of failure

The probability of failure is only significantly reduced by reinforcement measures which both (1) tackle failure mechanisms which significantly contribute to the initial probability of failure and (2) significantly reduce the probability of failure of this (these) failure mechanism(s). In case the initial probability of failure is evenly distributed over the failure mechanisms, the probability of failure is only significantly reduced when the strength regarding all failure mechanisms is increased. If the initial probability of failure is unevenly distributed over the failure mechanisms, the probability of failure may be significantly reduced by increasing the strength regarding one specific failure mechanism.

The predictability of failure

The predictability of failure depends on the certainty of the load and the certainty of the strength.

The certainty of the load mainly depends on the loading conditions. The wave heights and water levels along the coast cannot be predicted with certainty as they are partly determined by the uncertain development of the wind speed and wind direction. The water levels in the rivers are determined by the discharge of the river, which can be predicted with certainty as it is determined by the volume of water which enters the rivers far upstream and the distribution of the discharge over the
different river branches. The wave heights and water levels in estuaries are determined by the water level at sea and the discharge of the rivers. Therefore, the load on a river dike can be predicted with certainty and the load on a sea dike cannot be predicted with certainty in advance. The load on an estuary dike can be predicted with certainty in case it is dominated by the river discharge. The load on an estuary dike cannot be predicted with certainty in case it is dominated by the water level at sea. However, it is not known in advance if either the sea or river is going to dominate the load. Therefore the load on an estuary dike cannot be predicted with certainty.

The certainty of the strength is increased by reinforcement measures which increase the domination of failure mechanisms which have relatively certain strength. The strength regarding the failure mechanism ‘piping’ is fairly uncertain. The strength regarding the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’ is fairly certain for dikes designed for and loaded by low waves (river dikes) and fairly uncertain for dikes designed for and loaded by high waves (sea dikes).

The severity of failure

The severity of failure can be decreased by increasing the dominance of non-structural failure in case of dikes loaded by and designed for high waves. Structural failure is generally severe and the severity of structural failure is barely influenced by reinforcement measures. Non-structural failure differs in severity and the severity of non-structural failure depends on the loading conditions, the polder area and the strength of the dike. As the strength of a dike plays a role, the severity of non-structural failure is influenced by reinforcement measures; this is one of the topics discussed in the next chapter. In case of estuary and sea dikes, a high discharge over the crest (for example 10 or 50 l/s/m) is unlikely to result in severe non-structural failure as the loading period is limited. Furthermore, at these overtopping rates, a small increase in the water level (a few decimetres) results in a small increase in the severity of non-structural failure. In case of river dikes a high discharge over the crest is more likely to result in severe non-structural failure as the loading period is prolonged. Furthermore, at these overtopping rates, a small increase in the water level (a decimetre) results in a large increase in the severity of non-structural failure. Therefore the severity of failure can be decreased by increasing the dominance of non-structural failure (which is indicated by fragility curves, see chapter 4) and decreasing the severity of non-structural failure.
This chapter discusses the effect of strengthening dike elements on the properties of dike failure regarding deltadike requirements. By doing so, this chapter addresses the fourth sub-question of this research: ‘What is the effect of strengthening dike elements?’

The considered strengthened dike elements are in accordance with the considered failure mechanisms in this study which are ‘overflow and/or overtopping’, ‘piping’ and ‘non-structural failure’:
- The crest level;
- The crest width;
- The critical discharge;
- The seepage length.

This chapter aims to clarify the effect of strengthening individual dike elements and not the effect of reinforcement measures (which are discussed in chapter 5). This leads to the two counterintuitive assumptions. First, the strengthening of a dike element does not refer to the implementation of a reinforcement measure. For example, increasing the seepage length is not the same as the construction of a piping berm, widening the dike body or extending the cohesive top layer into the polder area. An increase in seepage length could however be obtained by performing these reinforcement measures. Second, the strengthening of a dike element does therefore not affect the strength of another dike element. For example, increasing the crest width is assumed to be something different from constructing a wider dike, therefore increasing the crest width has no effect on the seepage length while constructing a wider dike would.

4.1. Increasing the crest level

In the 16th century, Andreas Vierlingh wrote a manuscript about the design and construction of dikes. He noted that ‘most salvation depends on the height of a dike’ (translated from Vierlingh, 1570). This statement contains truths which are still valid. This is mainly because the height is directly related to the primary function of dikes, which is retaining water. A dike is only retaining water if the elevation of the dike is increased relative to its surroundings. Regarding the deltadike requirements, increasing the crest level:
- Reduces the probabilities of failure of the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’.
- Increases the residual strength.
- Shifts the fragility curve of the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’.
- Increases the severity of structural failure.
- Decreases the severity of non-structural failure.

These effects are explained in the remainder of this section.
4.1.1. Reduction of the probabilities of failure

The difference between the crest level and the water level (the freeboard) is a determining factor in the failure mechanism ‘overflow and/or overtopping’. The exceedance line of the maximum water level in most water bodies can be approximated by a Gumbel distribution:

\[ P_{exc}(wl_{max}) = 1 - \exp\left(-\exp\left(\frac{wl_{max} - \mu}{\beta}\right)\right) \]

Where:
- \( P_{exc} \) = Annual probability of exceedance of the maximum water level [-]
- \( wl_{max} \) = Maximum water level during a loading event [m+NAP]
- \( \mu \) = Location parameter of the Gumbel distribution [m+NAP]
- \( \beta \) = Scale parameter of the Gumbel distribution [-]

This is an approximately linear relationship between the common logarithm of the probability of exceedance of a certain water level and the water level itself (see Figure 41). A rule of thumb is that the exceedance probability of the water levels in The Netherlands reduces with a factor 10 for an increase of about 0.3 – 1.0 m in water level (the decimation height). This rule of thumb is highly dependent on the water system and therefore not generally applicable.

![Figure 41](image.png)

**Figure 41 | Exceedance line of the maximum water level \((wl_{max})\) (\(P_{exc}\) refers to the probability of exceedance of the maximum water level)**

In case the relationship between the freeboard and the discharge over the crest is constant, increasing the crest also linearly decreases the common logarithms of the probabilities of failure due to the failure mechanisms which are very dependent on the freeboard. These failure mechanisms are ‘overflow and/or overtopping’ and ‘non-structural failure’ (see Figure 42).
Effects of strengthening dike elements

4.1.2. Increase of the residual strength

The residual strength of a dike is directly related to the overflow and overtopping rate after initial dike damage has occurred. The higher the crest level, the less water overtops and overflows the dike after initial damage and therefore the higher the residual strength.

4.1.3. Shift of the fragility curve of the failure mechanisms 'overflow and/or overtopping' and 'non-structural failure'

The fragility curves of the failure mechanisms 'overflow and/or overtopping' and 'non-structural failure' shift as the crest level increases as shown in Figure 43 and Figure 44. This is due to the direct dependence of the probability of failure of these failure mechanisms on the crest level.

Figure 42 | Effect of increasing the critical discharge on the common logarithm of both the probability of failure of the failure mechanism 'overflow and/or overtopping' ($P_{sf_{tot}}$) and the probability of non-structural failure ($P_{nsf}$)

The analysis in this subsection shows that increasing the crest level generally significantly decreases the probabilities of failure of the failure mechanisms 'overflow and/or overtopping' and 'non-structural failure'.
The analysis in this subsection shows that increasing the crest level has a negative effect on the certainty of the strength in case the failure mechanism ‘piping’ partly dominates the initial aggregated fragility curve.

4.1.4. Increase of the severity of structural failure

Increasing the crest level slightly increases the severity of structural failure as the difference between the water level and the water level in the polder during breaching may increase. In case this happens, this affects the flow velocities in the breach zone and the rise rate in the polder. This increase is however negligible in comparison to the total flood damage.

4.1.5. Decrease of the severity of non-structural failure

The severity of non-structural failure is decreased as for a certain water level, a higher crest level decreases the discharge over the crest and therefore decreases the volume of water in the polder.

4.2. Increasing the crest width

Regarding the deltidike requirements, increasing the crest width:
- Increases the residual strength;
- Increases the period from an initial breach process to an actual breach.
These effects are explained in the remainder of this section.

4.2.1. Increase of the residual strength

The residual strength depends partly on the amount of soil which has to be eroded by the overflowing and/or overtopping water in order to locally decrease the crest level. The residual strength therefore increases as the dike width increases.
4.2.2. Increase of the period from an initial breach process to an actual breach

The breach growth model in sand-dikes (1998) is a suitable model to quantify the contribution of the crest width to the period from an initial breach to an actual breach. This period is equal to the period from the start the breaching process to the end of stage 2 of the breaching process (see subsection 2.1.2 and Figure 45).

![Figure 45 | Transition between stages from an initial breach to an actual breach](image)

There are several advantages and disadvantages of the model. Most primary dikes in the Netherlands have a sand core, which makes the breaching process generally applicable to this study. Waves are not included in the model but do speed up the breaching process by approximately a factor two or three in the case of sea dikes (P.J. Visser, personal communication, 13 September 2012). On the other hand, sand erodes very rapidly compared to clay or mixtures of sand and clay. The model is based on pure sand; the slopes of river dikes however are still covered by a clay revetment. Furthermore, the size of the initial breach influences the speed of the breaching process, but it has to be set which is quite arbitrary. Due to these and other uncertainties, the result of the model only gives the order of magnitude of the size of the breach, the discharge through the breach and the flood depth in the polder. The results are very sensitive to the water level in the water body, the grain diameter and the size of the initial breach. It turned out that varying the values of these variables increased or decreased the results by about fifty percent.

Smolders (2010) already used the model of Visser to compute the effect of increasing the crest width on the period from an initial breach to an actual breach. However, these results were put in doubt based on the documentation on the input and the results (P.J. Visser, personal communication, 13 September 2012).

Based on the reasoning in the above two paragraphs the model is considered to be appropriate for this study. Use has been made of the latest version of the model developed by Robijns (2012) in Matlab. The analysis considers a dike with a crest height of 3.6 meter above the polder level. The dike has side slopes of 1:3 (h:w). The water level is 3.5 meter above the polder level. The initial breach has a size of 0.2 meter in height. The angle of repose of the sand is assumed to be 32° and the bed porosity 0.40. Furthermore the distribution of the sediment is: $D_{10}=100$ μm, $D_{50}=200$ μm and $D_{90}=400$ μm. It is assumed that there is a foreshore at the river side of the dike; therefore the breach type is type B (see Visser, 1998). The formula of Van Rhee (2010) has been selected to compute the sediment transport (Robijns, 2012).
The results of the computations are presented in Figure 47.

The result shows that the period from an initial breach to an actual breach of a dike with normal dimensions is very small (in the order of minutes). The result also
shows that increasing the crest width by 100 m increases this period by roughly 1 hour.

The analysis in this subsection showed that increasing the crest width has a very low (insignificant) yield regarding the increase of the period from an initial breach to an actual breach. As this effect is the only quantifiable effect of increasing the crest width in this study, increasing the crest width does not contribute to the achievement of any deltadike requirement; it also does not make a dike breach-resistant. Therefore the strengthening of this dike element is not considered in the remainder of the study.

4.3. Increasing the critical discharge

Regarding the deltadike requirements, increasing the critical discharge:
- Reduces the probability of failure of the failure mechanism ‘overflow and/or overtopping’;
- Decreases the residual strength;
- Shifts the fragility curve of the failure mechanism ‘overflow and/or overtopping’;

These effects are explained in the remainder of this section. It does not affect the probability of non-structural failure as neither the crest level nor the relationship between the freeboard and the overtopping discharge are affected.

4.3.1. Reduction of the probability of failure of the failure mechanism ‘overflow and/or overtopping’

Conventional dike design limits the discharge over the crest to 0.1, 1 or 10 l/s/m. Starting from these critical discharges, the sensitivity of the discharge over the crest to a change in freeboard is relatively large for dikes designed for and loaded by small waves; the higher the wave height, the smaller the sensitivity of the discharge over the crest to a change in freeboard.

For example, the design wave height of a dike with a 1:3 (v:h) grassed outer slope is 0.1 m and the critical discharge is 1 l/s/m. When the critical discharge is now increased to 30 l/s/m and the crest level is constant, the design water level increases by about 0.25 m (see Figure 18 on page 25). When the same situation is considered for a dike with a design wave height of 5 m, the design water level increases by about 6 m (see Figure 19 on page 25).

Therefore the increase in critical discharge is a suitable measure for dikes designed for and loaded by high waves while it is an unsuitable measure for dikes designed for and loaded by low waves. Following the relationships depicted by Figure 19 on page 25, the relationship between the critical discharge and the probability of structural failure due to the failure mechanism ‘overflow and/or overtopping’ is exponentially decreasing (see Figure 48).
Chapter
Effects of strengthening dike elements

The analysis in this subsection shows that increasing the critical discharge significantly reduces the probability of failure of ‘overflow and/or overtopping’ in case of a dike designed for and loaded by high waves. It does not significantly decrease this probability of failure for dikes designed for and loaded by small waves.

4.3.2. Shift of the fragility curve of the failure mechanism ‘overflow and/or overtopping’

While the probability of structural failure of the failure mechanism ‘overflow and/or overtopping’ decreases, the probability of the failure mechanism ‘non-structural failure’ stays constant. Therefore the contribution of this failure mechanism to the probability of failure increases as the dike will withstand higher discharges over the crest for a longer time. That cuts both ways regarding the volume of water which enters the polder area. At some point the volume which enters the polder area will result in flooding while the dike is not affected. Therefore, the higher the critical discharge the more non-structural failure will dominate over structural failure. This leads to the exposure of the fragility curve of non-structural failure (see Figure 49 and Figure 50). Generally the fragility curves of the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’ are quite close to each other. This means that increasing the critical discharge does not significantly affect the aggregated fragility curve.

![Figure 48 | The effect of increasing the critical discharge ($q_c$) on the probability of failure of the failure mechanism ‘overflow and/or overtopping’ ($Ps_{ofot}$) for a small and large design wave ($H_d$)](image)

![Figure 49 | Initial fragility curves ($Pf$ refers to probability of failure of the dike section, $w_{max}$ refers to maximum water level during a loading event)](image)
4.3.3. **Decrease of residual strength**

In case the dike with a higher critical discharge fails during extreme loading conditions, the amount of overtopping and overflowing water is larger. Therefore the residual strength decreases (see Figure 51).

4.4. **Increasing the seepage length**

Regarding the deltadike requirements, increasing the seepage length:

- Reduces the probability of failure of the failure mechanism ‘piping’;
- Shifts the fragility curve of the failure mechanism ‘piping’;

These effects are explained in the remainder of this section.

4.4.1. **Reduction of the probability of failure of the failure mechanism ‘piping’**

The difference between the water level in the water body and the water level in the polder (the hydraulic head over the dike) is a determining factor regarding the failure mechanism ‘piping’. As the water level in the polder is fairly constant, the exceedance line of the hydraulic head has the same distribution as the water level in the water body (a Gumbel distribution, see subsection 4.1.1). Whether the hydraulic head will lead to initiation of the failure trajectory is determined quantitatively by
the critical hydraulic head. An increase in critical hydraulic head therefore linearly decreases the common logarithm of the probabilities of failure due to the ‘piping’ as shown in Figure 52.

![Figure 52](image_url)

**Figure 52 | Effect of increasing the critical hydraulic head \((dh_c)\) on the common logarithm of the probability of failure of the failure mechanism ‘piping’ \((Psf_{piping})\)**

Therefore the increase of the seepage length results in a linear decrease of the common logarithm of the probability of failure of the failure mechanism ‘piping’ (see Figure 53).

![Figure 53](image_url)

**Figure 53 | Effect of increasing the seepage length \((L_s)\) on the critical hydraulic head \((dh_c)\)**

The analysis in this subsection shows that increasing the seepage length significantly reduces the probability of failure of the failure mechanism ‘piping’.

### 4.4.2. Shift of the fragility curve of the failure mechanism ‘piping’

As an increasing seepage length increases the critical hydraulic head and thus maximum water level before which failure due to the failure mechanism ‘piping’ takes place, the fragility curve of the failure mechanism ‘piping’ shifts for increasing seepage length (see Figure 54 and Figure 55).
Chapter Effects of strengthening dike elements

4.5. Concluding remarks

The strengthened dike elements in this study are increasing the crest level, crest width, critical discharge and seepage length.

Increasing the crest level decreases the probability of failure of the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’. It also shifts the fragility curves of these mechanisms whereby it has a negative effect on the certainty of the strength in case the failure mechanism ‘piping’ starts to dominate the initial aggregated fragility curve. The dominance of either structural or non-structural failure is hardly affected as the strength regarding both types of failure increases. The measure barely affects the severity of structural failure while the severity of non-structural failure decreases. By doing so it positively contributes to the achievement of first deltadike requirement while it does not contribute to the achievement of the second requirement.
Increasing the crest width does not affect the probability of failure. It contributes insignificantly to lengthening the period from the initiation of the breach to an actual breach. By doing so it does not contribute to the achievement of any of the deltadike requirements.

Increasing the critical discharge insignificantly reduces the probability of failure of the failure mechanism ‘overflow and/or overtopping’ of dikes designed for and loaded by small waves (river dikes). Increasing the critical discharge significantly reduces the probability of failure of the failure mechanism ‘overflow and/or overtopping’ of dikes loaded by and designed for moderate and high waves (estuary and sea dikes). Furthermore, it significantly increases the dominance of mild non-structural failure over severe structural failure for dikes designed for and loaded by high waves (sea dikes). Therefore, it does not contribute significantly to the achievement of any deltadike requirement of dikes designed for and loaded by low waves. It positively contributes to the achievement of the first deltadike requirements of dikes designed for and loaded by moderate waves. It contributes to both deltadike requirements of dikes designed for and loaded by high waves.

Increasing the seepage length decreases the probability of failure of the failure mechanism ‘piping’. Increasing the seepage length exposes the failure mechanisms with a more certain strength. Thus it contributes to the achievement of both deltadike requirements.
This chapter quantifies the effect of dike reinforcement measures on the deltadike requirements by considering three cases. By doing so, this chapter provides answer to the fifth sub-question of this research: *What is the effect of dike reinforcement measures when performing actual dike reinforcements under different loading conditions?*

Table 3 lists the characteristics of the case studies and Figure 56 shows the location of each case study. Each case views upon the effect of reinforcement measures on the probability of failure, certainty of the strength and severity of failure. This relates to the chapter 4 as reinforcements strengthen dike elements.

**Table 3 | Characteristics of the case studies**

<table>
<thead>
<tr>
<th></th>
<th>Case study I</th>
<th>Case study II</th>
<th>Case study III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Considered dike</td>
<td>Pettemer Zeewering</td>
<td>Kop van ’t Land dijk</td>
<td>Grebbedijk</td>
</tr>
<tr>
<td>Dike ring</td>
<td>Noord-Holland (13)</td>
<td>Eiland van Dordrecht (22)</td>
<td>Gelderse Vallei (45)</td>
</tr>
<tr>
<td>Adjacent water body</td>
<td>North Sea</td>
<td>Nieuwe Merwede</td>
<td>Nederrijn</td>
</tr>
<tr>
<td>Type of dike</td>
<td>Sea</td>
<td>Estuary</td>
<td>River</td>
</tr>
<tr>
<td>Loading condition</td>
<td>Coastal</td>
<td>Estuarine</td>
<td>Riverine</td>
</tr>
<tr>
<td>Wave height</td>
<td>Large</td>
<td>Moderate</td>
<td>Small</td>
</tr>
<tr>
<td>Loading period</td>
<td>Short</td>
<td>Moderate</td>
<td>Long</td>
</tr>
</tbody>
</table>

**Figure 56 | Location of the case studies (source background: Deltaprogramma, 2012)**
Chapter Case studies

The first case study considers a sea dike subject to coastal loading conditions. The load is composed of tides, storm-surges and waves; the wave characteristics and the water level are correlated as they are both increased by landward wind. The fetch length is very large and the storm duration is relatively short which generally results in large wave heights (several meters) in combination with high water levels for a limited period of time (a few hours).

The second case study considers a estuary dike subject to estuarine loading conditions. The load is composed of tides, storm-surges, waves and increased water levels due to an increased discharge from the river; the wave characteristics and the water level are correlated as they are both increased by landward wind. The fetch length is considerable and the storm duration is considerable which generally results in considerable wave heights (one meter) in combination with high water levels for a considerable period of time (several hours).

The third case study considers a river dike subject to riverine loading conditions. The load is composed of waves and increased water levels due to an increased discharge from the river; the wave characteristics and water level are not correlated as the wind speed and direction is not correlated with the discharge from the river. The fetch length is limited and the river flood wave is prolonged which generally results in small wave heights (one decimetre) in combination with high water levels for a prolonged period of time (several days).

Course of the water level

The course of the water level is based on historic extreme loading events. The course of the water level could have been based on the courses presented in the hydraulic boundary conditions 2006 (Ministerie van Verkeer en Waterstaat, 2007). This is however not the preferred choice as there is some discussion about these courses of the water level. Furthermore, the used approach is relatively simple which is in accordance with the other assumptions in this study.

Polder area

In each case study the polder is the same. This is because the focus of this study is on the dike reinforcement measures and not on the effects of a changing polder. The polder has the same characteristics as the polder introduced in chapter 4, shown in Figure 29 and Figure 30.

Reinforcement measures

Reinforcement measures are actual physical measures which strengthen dike elements. The considered reinforcement measures are in accordance with the considered failure mechanisms which are ‘overflow and/or overtopping’, ‘piping’ and ‘non-structural failure’.
Increasing the crest height (see Figure 57).
Increasing the strength of the revetment (see Figure 58).
Increasing the width of the dike (see Figure 59).
Increasing the width of the piping berm (see Figure 60).

These measures are visualized as reinforcement measures in Figure 57. The applied measures each affect the strength of certain dike elements. Increasing the crest height increases the crest level; the effect of increasing the crest height on the seepage length is minor and therefore not included in this measure. Increasing the strength of the revetment increases the critical discharge. Increasing the dike width increases the crest width and increases the seepage length. Increasing the width of the piping berm increases the seepage length.

---

Figure 57 | The reinforcement measure ‘increasing the crest height’

Figure 58 | The reinforcement measure ‘increasing the strength of the revetment’

Figure 59 | The reinforcement measure ‘increasing width of the dike’

Figure 60 | The reinforcement measure ‘increasing the width of the piping berm’

In the remainder of this chapter, increasing the crest width is not included as it has not got any significant and quantifiable effect on the flood protection level of dikes as turned out in chapter 4. The reinforcement measure ‘increasing the width of the dike’ therefore has the same effect as ‘increasing the width of the piping berm’ which requires less soil and space. Therefore the measure ‘increasing the width of the dike’ is excluded from further investigation in the case studies.

The greatness of the reinforcement measures depends on the effectiveness of the measure in decreasing the probability of failure of the failure mechanism it tackles. The maximum critical discharge is 30 l/s/m as recent full-scale tests show that inner slopes with a normal grass cover can resist even up to 50 or 75 l/s/m (Van der Meer et al., 2009a).
Models

Two models have been used throughout the case studies. The Dike Reliability Model (see Appendix B), which is developed by the author, has been used to compute the probabilities of failure. The model uses the First Order Reliability Method (FORM) routine to compute the probability of failure of a dike section. The code of the FORM routine applied in the model is found online using the open-source package OET (OpenEarthTools). The length effect is included by using the formulas given by Vrijling (2011). The input (the stochastic variables) of each case study is shown in section B.3. The breach growth model in sand-dikes (Visser, 1998) has been used to model the breaching process. Use has been made of the latest version of the model developed by Robijns (2012) in Matlab. The model contains several simplifications which are discussed in section 4.4.2. The residual strength of a dike is not included in either one of the models as there is both no limit state function and no model available to calculate the stress and the resistance of the residual strength. The residual strength may however be an important factor contributing to the flood protection level of dikes as discussed in chapter 4.

5.1. **Case study I: ‘Pettemer Zeewering’**

This case study considers the reinforcement of a dike section called ‘Pettemer Zeewering’ adjacent to the North Sea. The sea dike is part of dike ring 13 (called ‘Noord-Holland’). The local situation is shown in Figure 61. The length of the dike section is 1000 meter. Both the load on the dike and the strength of the dike are modelled by stochastic variables. Their names, distributions and correlation distances are listed in Table B.4 in Appendix B. The load and the strength data of this case study are obtained from Ter Horst (2007) and from drawings made by the Hoogheemraadschap Hollands Noorderkwartier (2004).

![Figure 61 | Overview local situation (background copied from Google Maps, 2012)](image)
The failure mechanism ‘piping’ and its affiliated reinforcement measure (increasing the width of the piping berm) are not considered as the failure mechanism is not known to have affected sea dikes in history.

Figure 62 shows the schematized representative cross-section of the dike section.

![Cross-section of the dike section](image)

**Figure 62 | Undistorted and distorted schematized representative cross-section (NAP refers to Amsterdam Ordnance Datum, MHW refers to Mean High Water)**

Table 4 lists the strength characteristics of the dike. These strength characteristics have been modelled as stochastic variables. The distributions, mean values and variances of these variables are listed in Table B.4 in Appendix B.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Mean value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crest level</td>
<td>12.9 [m+NAP]</td>
</tr>
<tr>
<td>Wall level</td>
<td>13.6 [m+NAP]</td>
</tr>
<tr>
<td>Critical discharge over the crest</td>
<td>10 [l/s/m]</td>
</tr>
<tr>
<td>Inclination front face of the structure</td>
<td>1/3 [-]</td>
</tr>
<tr>
<td>Influence factor for the berm</td>
<td>0.61 [-]</td>
</tr>
<tr>
<td>Influence factor for the wave angle of incidence</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Influence factor for the roughness of the outer slope</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Influence factor for the vertical wall</td>
<td>0.80 [-]</td>
</tr>
<tr>
<td>Berm level</td>
<td>5.5 [m+NAP]</td>
</tr>
<tr>
<td>Berm width</td>
<td>16.0 [m]</td>
</tr>
<tr>
<td>Inclination berm at the outer slope</td>
<td>1/20</td>
</tr>
</tbody>
</table>
The load is modelled by a combination of a varying water level and a varying wave height. The exceedance line of the maximum water level is described by the following formula (based on a least-square fit of the common logarithm of the exceedance probabilities provided by PC-RING for the considered location) as shown in Figure 63:

$$P_{exc}(w_{l_{max}}) = 1 - \exp\left(-\exp\left(-\frac{w_{l_{max}} - 2.2}{0.27}\right)\right)$$

Figure 63 | Relationship between the common logarithm of the probability of exceedance of the maximum water level ($P_{exc}$) and the maximum water level ($w_{l_{max}}$)

The course of the water level is based on the course of the water level during the storm surge at the North Sea near Hoek van Holland on 1 and 2 February 1953 shown in Figure 64. This course is scaled to the maximum water level.
The maximum wave height is the same for all maximum water levels, which is not correct as the maximum wave height is strongly correlated to the maximum water level. The maximum wave height is assumed to be equal to the spectral wave height. This wave height is assumed to be equal to the significant wave height of 4.45 meter as provided by the hydraulic boundary conditions 2006 (Ministerie van Verkeer en Waterstaat, 2007). The spectral wave period is assumed to be constant and equal to the significant wave period which is 13.2 seconds (Ministerie van Verkeer en Waterstaat, 2007).

The course of the wave height is assumed to be trapezoidal (shown in Figure 65). In reality the course of the wave height varies much more due to varying wind speeds, angle of incidence, bottom morphology and sheltering. Including these variations would unnecessarily complicate the calculations, as the objective is to show the effect of reinforcement measures and not to model reality as good as possible.

Since there is no question of overflow (a sea dike is a dike designed for and loaded by large wave heights and it has therefore large freeboards), the Van der Meer formula (Pullen et al., 2007; formula 5.9) is valid for the range of interest of the freeboard. In order to use the same methodology as in the other case studies, an exponential function is fitted through the overtopping formula (see Figure 66).
5.1.1. The probability of failure

This subsection describes and shows the influence of the initial probability of failure and the reinforcement measures on the probability of failure of the dike section. To
what degree reinforcement measures decrease the probability of failure is very
dependent on the initial distribution of the probability of failure. This distribution is
calculated using the DRM (see Appendix B); the result is shown in Table 5 and
Figure 67.

Table 5 | Distribution of the probability of failure over the failure mechanisms

<table>
<thead>
<tr>
<th>Failure mechanism</th>
<th>Probability of failure [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overflow and/or overtopping</td>
<td>5.7*10^(-4)</td>
</tr>
<tr>
<td>Piping</td>
<td>Not included</td>
</tr>
<tr>
<td>Non-structural failure</td>
<td>5.2*10^(-5)</td>
</tr>
<tr>
<td>All failure mechanisms</td>
<td>6.2*10^(-4)</td>
</tr>
</tbody>
</table>

Figure 67 | Initial distribution of the annual probability of failure of the dike section ($P_f$)
over the annual probability of failure of the failure mechanism ‘overflow and/or
ovetopping' ($P_{so/w}$) and the annual probability of failure of the failure mechanism ‘non-
structural failure' ($P_{nsf}$)

Figure 67 shows that the initial distribution of the probability of failure is uneven;
the probability of failure is almost fully comprised by the failure mechanism
‘overflow and/or overtopping’. This is firstly because the dike is loaded for a
relatively short period (storm duration in combination with the fluctuating tide)
which reduces the amount of water which enters the polder area during loading and
therefore reduces the stress regarding the failure mechanism ‘non-structural failure’.
Secondly the discharge over the crest is relatively insensitive to a change in water
level; this also reduces the stress regarding the failure mechanism ‘non-structural failure’.

Figure 68 shows the effect of increasing the crest height (which increases the crest
level) on the probability of failure. This result shows that the probabilities of failure
due to the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’ are decreasing linearly as the crest level increases. Furthermore the figure shows that the decimation height is about 0.7 meter. The effect of increasing the crest height on the probability of failure does also not diminish with the extent of the measure as the measure affects all failure mechanisms which comprise the probability of failure. Therefore the probability of failure can be reduced significantly by performing this measure. A decrease in probability of failure by a factor 10 or 100 is reached when the crest height is increased by respectively 0.7 or 1.4 meter.

Figure 69 shows the effect of increasing the strength of the revetment (which increases the critical discharge) on the probability of failure. The figure shows that the measure has a significant effect on the probability of failure of the failure mechanism ‘overflow and/or overtopping’. This is because this dike is designed for and loaded by high waves which causes the discharge over the crest to be quite insensitive to a change in water level. The effect diminishes as this sensitivity increases for higher critical discharges (see Figure 66). The measure also has a diminishing and significant effect on the probability of failure. This is because the tackled failure mechanism contributes significantly to the initial probability of failure in the beginning while this contribution decreases as the extent of the measure increases.

Figure 68 | Effect of increasing the crest level $\tilde{c}l$ on the annual probability of failure of the dike section of the failure mechanism ‘overflow and/or overtopping’ ($P_{sf_{overflow}}$), the annual probability of failure of the dike section of the failure mechanism ‘non-structural failure’ ($P_{nsf}$) and the annual probability of failure of the dike section ($P_f$)
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5.1.2. The certainty of the strength

This subsection describes and shows the effect of the reinforcement measures on the certainty of the strength. The fragility curves have been computed using the DRM (developed by the author, see Appendix B).

Figure 70 shows the initial fragility curves. This result shows that initial fragility of the dike is dominated by the two considered failure mechanisms. The strength regarding both mechanisms is fairly uncertain as the associated fragility curves have a fairly large domain of about 1 m and 2 m.

Figure 71 shows the fragility curves in case the dike is reinforced by increasing the crest height. This result shows that both failure mechanisms are affected equally by this reinforcement measure. Therefore this measure does not change the domination of the fragility curve by either one of the failure mechanisms. The aggregated...
fragility curve only shifts; it does not become significantly steeper or milder. This reinforcement measure does therefore not affect the fragility of the dike (or the certainty of the strength).

Figure 72 shows the fragility curves in case the dike is reinforced by increasing the strength of the revetment. This result shows that increasing the strength of the revetment only affects the failure mechanism ‘overflow and/or overtopping’. As the failure mechanism ‘non-structural failure’ is not affected, the domination of the initial aggregated fragility curve changes. The fragility of the dike after the reinforcement measure is fully dominated by the failure mechanism ‘non-structural failure’. By doing so, the measure increases the certainty of the strength although the strength regarding this failure mechanism is still fairly uncertain after applying the measure. Therefore this reinforcement measure has a limited and positive effect on the fragility of the dike and thus slightly increases the certainty of the strength.

Figure 73 shows the aggregated fragility curves of the initial dike, the dike after increasing the crest height and the dike after increasing the strength of the revetment. This result shows that the only reinforcement measure which affects the certainty of the strength is increasing the strength of the revetment. The effect of this measure on the certainty of the strength is still limited as the strength regarding the governing failure mechanisms of a sea dike is predominantly uncertain.

![Fragility curves of the initial dike](image)

**Figure 70** | Fragility curves of the initial dike (*P_f* refers to probability of failure of the dike section, *w_{max}* refers to maximum water level during a loading event)
Chapter Case studies

Figure 71 | Fragility curves of the dike reinforced by increasing the crest height ($P_f$ refers to probability of failure of the dike section, $w_l_{max}$ refers to maximum water level during a loading event)

Figure 72 | Fragility curves of the dike reinforced by increasing the strength of the revetment ($P_f$ refers to probability of failure of the dike section, $w_l_{max}$ refers to maximum water level during a loading event)
5.1.3. The severity of failure

Figure 73 shows the severity of structural and non-structural failure. This result shows that structural failure results in large water depths in comparison to non-structural failure. The severity of non-structural is mild. The severity of non-structural failure furthermore depends on the greatness of the load and therefore on the probability of exceedance of the maximum water level although this influence is limited. This is because both the discharge over the crest of the dike and the loading period do not increase significantly for increasing water levels in the order of a meter.
The water depths which are the result of structural and non-structural failure can be translated into estimations of the flood damage and the loss of life using the results of the computations in chapter 3. Structural failure results in a flood damage of about €420 million and the death of about 40 persons. Non-structural failure results in an average flood damage of about €120 million and the death of about 0 persons.

Figure 76 shows the effect of increasing the crest height on the severity of failure. This result shows that the final flood depth due to non-structural failure is in general small and furthermore barely affected by increasing the crest height. What is not shown in this graph is that increasing the crest height possibly increases the severity of structural failure by increasing the flow velocities in the breach zone. The final flood depth is however the same as it is limited to the maximum flood depth in the polder which is determined by the height of secondary flood defences.
5.4. Transforming the dike into a deltadike

The first deltadike requirement can be fulfilled. The dike has an initial of the probability of failure which is quite unevenly distributed over the failure mechanisms. The probability of failure is therefore mainly decreased by increasing the strength regarding the failure mechanism ‘overflow and/or overtopping’. This can be done by increasing the crest height and increasing the strength of the revetment. Increasing the crest height also increases the strength regarding the failure mechanism ‘non-structural failure’.

The second deltadike requirement can be fulfilled by increasing the dominance of non-structural failure which is relatively mild for sea dikes. This is done by strengthening the revetment (increasing the critical discharge). The fragility of the dike can only be increased slightly by increasing the strength of the revetment. Structural failure generally results in great flood damage and loss of life while non-structural failure results in moderate flood damage and no loss of life. The small severity of non-structural failure is barely decreased by increasing the crest height; the large severity of structural failure is not positively influenced by reinforcement measures.
5.2. **Case study II: ‘Kop van’t Land dijk’**

This case study considers the reinforcement of a dike section called ‘Kop van’t Land dijk’ adjacent to the estuarine canal ‘Nieuwe Merwede’. This estuary dike is part of dike ring 22 (called ‘Het eiland van Dordrecht’). The local situation is shown in Figure 76. The length of the dike section is 3500 m. Both the load on the dike and the strength of the dike are modelled by stochastic variables. Their names, distributions and correlation distances are listed in Table B.5 in Appendix B. The load and the strength data of this case study are obtained from Waterschap Hollandse Delta (2011) and the associated drawings and data-sheets.

![Figure 76 | Overview local situation (background copied from Google Maps, 2012)](image)

Figure 77 shows the schematized representative cross-section of the dike section.
Table 6 lists the strength characteristics of the dike. These strength characteristics have been modelled as stochastic variables. The distributions, mean values and variances of these variables are listed in Table B.5 in Appendix B.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Mean value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crest level</td>
<td>4.4 [m+NAP]</td>
</tr>
<tr>
<td>Critical discharge over the crest</td>
<td>1 [l/s/m]</td>
</tr>
<tr>
<td>Inclination front face of the structure</td>
<td>1/3 [-]</td>
</tr>
<tr>
<td>Influence factor for the berm</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Influence factor for the wave angle of incidence</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Influence factor for the roughness of the outer slope</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Influence factor for the vertical wall</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Seepage length</td>
<td>48 [m]</td>
</tr>
<tr>
<td>Thickness cohesive top layer</td>
<td>0 [m]</td>
</tr>
<tr>
<td>Thickness sand layer</td>
<td>2 [m]</td>
</tr>
<tr>
<td>70-Percentile grain distribution sand layer</td>
<td>6.5*10-5 [m]</td>
</tr>
<tr>
<td>Permeability sand layer</td>
<td>3.05*10-6 [m/s]</td>
</tr>
</tbody>
</table>

The load is modelled by a combination of a varying water level and a constant wave height. The exceedance line of the maximum water level (see Figure 78) is described by the following formula (based on a least-square fit of the common logarithm of the exceedance probabilities provided by PC-RING for the considered location):

\[
P_{exc}(wl_{max}) = 1 - \exp\left(-\exp\left(-\frac{wl_{max} - 1.6}{0.18}\right)\right)
\]
Figure 78 | Relationship between the common logarithm of the probability of exceedance of the maximum water level \((P_{exc})\) and the maximum water level \((w_{l_{max}})\)

The course of the water level (see Figure 79) is scaled to the maximum water level and is based on the course of the water level during the high water on the Nieuwe Merwede near Dordrecht in January 2012.

Figure 79 | Water level \((w_l)\) relative to the maximum water level \((w_{l_{max}})\) near Dordrecht with the time \((t)\) relative to the point in time of maximum water level \((t_{max})\) (data obtained from Rijkswaterstaat, 2012)
The wave height and the wave period are assumed to be constant. In reality the wave height varies due to varying wind speeds, angle of incidence, bottom morphology and sheltering. Including these variations would unnecessarily complicate the calculations, as the objective is to show the effect of reinforcement measures and not to model reality as good as possible. What is included in the estimation of the wave height and wave period is the orientation of the dike section to the wind direction. The wind which causes the most wind set-up at the North Sea is blowing from the northwest to the southeast. Therefore this wind direction is chosen as the governing direction. Using this direction, some parts of the dike section have a fetch length of about 2000 meter, while other parts are sheltered. The average fetch length is approximately 1200 meter. As the wave height is also correlated with the water level and therefore the wind speed, the assumed constant wind speed is 20 m/s. The spectral wave height and period are calculated using the formulas of Bretschneider (TAW, 1989; Steenbergen et al., 2008) assuming that the spectral wave height and wave period are equal to the significant wave height and wave period; the resulting spectral wave height is 0.5 meter and the spectral wave period is 2.5 seconds.

In order to combine the processes of overtopping and overflow a function has been fitted through the relevant models describing these processes (see Figure 80). The formulas of these models are found in subsection 2.1.2.

![Figure 80 | Least square exponential fit of the discharge over the crest (q) and the freeboard (Rc) on the formulas of Van der Meer (Pullen et al., 2007), Schüttrumph (Pullen et al, 2007) and Nadal and Hughes (2009)](image)

5.2.1. The probability of failure
This subsection describes and shows the influence of the initial probability of failure and the reinforcement measures on the probability of failure of the dike section. To what degree reinforcement measures decrease the probability of failure is very dependent on the initial distribution of the probability of failure. This distribution is calculated using the DRM (see Appendix B); the result is shown in Table 7 and Figure 81. This result shows that the probability of failure is relatively evenly distributed over the considered failure mechanisms.

Table 7 | Initial probabilities of failure

<table>
<thead>
<tr>
<th>Failure mechanism</th>
<th>Probability of failure [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overflow and/or overtopping</td>
<td>(5.1 \times 10^{-4})</td>
</tr>
<tr>
<td>Piping</td>
<td>(1.1 \times 10^{-3})</td>
</tr>
<tr>
<td>Non-structural failure</td>
<td>(1.0 \times 10^{-4})</td>
</tr>
<tr>
<td>All failure mechanisms</td>
<td>(1.7 \times 10^{-3})</td>
</tr>
</tbody>
</table>

Figure 81 | Initial distribution of the annual probability of failure of the dike section \(P_f\) over the probability of failure of the failure mechanism ‘overflow and/or overtopping’ \(P_{f,\text{of/\alpha}}\), the probability of failure of the failure mechanism ‘piping’ \(P_{f,\text{piping}}\) and the annual probability of failure of the failure mechanism ‘non-structural failure’ \(P_{f,\text{nsf}}\)

Figure 82 shows the effect of increasing the crest height (which increases the crest level) on the probability of failure. This result shows that the probabilities of failure of the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’ are decreasing linearly as the crest level increases. Furthermore the figure shows that the decimation height is about 0.7 meter. Increasing the crest height has a diminishing and insignificant effect on the probability of failure because the two tackled failure mechanisms together comprise only a part of the probability of failure. A decrease in probability of failure by a factor 10 or 100 is therefore not reached by performing this single reinforcement measure.
Figure 83 shows the effect of increasing the strength of the revetment (which increases the critical discharge) on the probability of failure. This result shows that the measure has a diminishing effect on the probability of failure of the failure mechanism ‘overflow and/or overtopping’. This reduction is less than in the case study 1 because this dike is designed for and loaded by moderate (less high) waves which causes the discharge over the crest to be more sensitive to a change in water level. The effect diminishes as this sensitivity increases for higher critical discharges (see Figure 80). Increasing the strength of the revetment has a diminishing and insignificant effect on the probability of failure because the tackled failure mechanism does not comprise the probability of failure on its own. A decrease in probability of failure by a factor 10 or 100 is therefore not reached by performing this single reinforcement measure.

Figure 84 shows the effect of increasing the width of the piping berm (which increases the seepage length) on the probability of failure. This result shows that the measure has a predominantly linearly reduces the probability of failure of the failure mechanism ‘piping’. Increasing the width of the piping berm has a diminishing and insignificant effect on the probability of failure because the tackled failure mechanism does not comprise the probability of failure on its own. A decrease in probability of failure by a factor 10 or 100 is therefore not reached by performing this single reinforcement measure.
Figure 83 | Effect of increasing the critical discharge \((q_c)\) on the annual probability of failure of the dike section \((P_f)\), this probability of failure of the failure mechanism ‘overflow and/or overtopping’ \((P_{sf\_of\_tot})\), this probability of failure of the failure mechanism ‘piping’ \((P_{sf\_piping})\) and this probability of failure of the failure mechanism ‘non-structural failure’ \((P_{nsf})\).

Figure 84 | Effect of increasing the seepage length \((L_s)\) on the annual probability of failure of the dike section \((P_f)\), this probability of failure of the failure mechanism ‘overflow and/or overtopping’ \((P_{sf\_of\_tot})\), this probability of failure of the failure mechanism ‘piping’ \((P_{sf\_piping})\) and this probability of failure of the failure mechanism ‘non-structural failure’ \((P_{nsf})\).
The results above indicate that the probability of failure is not reduced significantly by performing individual reinforcement measures. The following reinforcement measures are considered in the remainder of this case study:

- Increasing the crest height from 4.4 m+NAP to 5.2 m+NAP;
- Increasing the strength of the revetment from 1 l/s/m to 30 l/s/m;
- Increasing the width of the piping berm from 48.9 m to 80 m.

The magnitude of the reinforcement measures is chosen such that the measures significantly reduce the probability of failure of the failure mechanisms they tackle.

5.2.2. The certainty of the strength

This subsection describes and shows the effect of the reinforcement measures on the certainty of the strength. The fragility curves have been computed using the DRM (developed by the author, see Appendix B). Figure 85 shows the initial fragility curves. This result shows that the aggregated fragility curve is dominated by the failure mechanism ‘piping’ which results in a mild aggregated fragility curve. This is not desirable as the strength is therefore quite uncertain.

Figure 85 | Fragility curves of the initial dike (\(P_f\) refers to probability of failure of the dike section, \(w_{\text{max}}\) refers to maximum water level during a loading event)

Figure 86 shows the fragility curves in case the dike is reinforced by increasing the crest height. Although the measure makes the dike stronger (reduces the probability of failure), this result shows that it has an undesirable effect as it exposes the mild fragility curve of the failure mechanism ‘piping’.

Figure 87 shows the fragility curves in case the dike is reinforced by increasing the strength of the revetment. This result shows that this measure hardly has any effect
on the fragility of the dike. This is because the relatively steep fragility curves of the failure mechanisms 'overflow and/or overtopping' and 'non-structural failure' are quite close to each other while increasing the strength of the revetment only influences the fragility of the failure mechanism 'overflow and/or overtopping'.

Figure 88 shows the fragility curves in case the dike is reinforced by increasing the width of the piping berm. This result shows that this measure has a desirable effect as it exposes the steep fragility curve of the failure mechanism 'overflow and/or overtopping'.

The aggregated fragility curves of the initial dike and the reinforced dikes are shown in Figure 89. The aggregated fragility curve only becomes milder in case the width of the piping berm is increased. This is therefore also the only measure which significantly contributes to an increase in the certainty of the strength. Increasing the piping berm therefore significantly contributes to an increase in the certainty of the strength while increasing the crest height and increasing the strength of the revetment decrease the certainty of the strength. Therefore the certainty of the strength can be significantly increased by increasing the width of the piping berm to such an extent that the failure mechanism 'piping' does not dominate failure anymore.

![Figure 86 | Fragility curves of the reinforced dike by increasing the crest level (P_f refers to probability of failure of the dike section, w_l_max refers to maximum water level during a loading event)](image-url)
5 Chapter Case studies

Figure 87 | Fragility curves of the reinforced dike by increasing the critical discharge ($P_f$ refers to probability of failure of the dike section, $w_{l_{\text{max}}}$ refers to maximum water level during a loading event)

Figure 88 | Fragility curves of the reinforced dike by increasing the seepage length ($P_f$ refers to probability of failure of the dike section, $w_{l_{\text{max}}}$ refers to maximum water level during a loading event)
5.2.3. The severity of failure

Figure 90 shows the severity of structural and non-structural failure. This results indicates that structural failure results in large water depths in comparison to non-structural failure. The severity of non-structural is mild to moderately severe. The severity of non-structural failure furthermore depends on the greatness of the load and therefore on the probability of exceedance of the maximum water level. This influence is considerable because both the discharge over the crest of the dike and the loading period increase considerably for increasing water levels in the order of several decimetres.
The effect of structural and non-structural failure on the water depth in the polder ($d_p$) over time ($t$) relative to the point in time of maximum water level in the water body ($t_{\text{max}}$) depending on the annual probability of exceedance of the maximum water level ($P_{\text{exc}}$).

The water depths which are the result of structural and non-structural failure can be translated into estimations of the flood damage and the loss of life using the results of the computations in chapter 3. Structural failure results in a flood damage of about €420 million and the death of about 40 persons. Non-structural failure results in an average flood damage of about €200 million and the death of about 3 persons.

Figure 91 shows the effect of increasing the crest height on the severity of failure. This figure indicates that the measure hardly affects the severity of structural failure. The final flood depth is the same as this is limited to the maximum flood depth in the polder which is determined by the height of secondary flood defences. What is not shown in the graph is that the severity of structural failure slightly increases as the flow velocities in the breach zone increase. The severity of non-structural failure is significantly decreased by increasing the crest height. Thus the severity of non-structural failure can be decreased by increasing the crest height.
5.2.4. Transforming the dike into a deltadike

The first deltadike requirement can be fulfilled. The dike has an initial probability of failure which is quite evenly distributed over the failure mechanisms. This causes neither of the considered measures to reduce the probability of failure significantly (by a factor 10 or 100). The probability of failure is therefore mainly decreased by increasing the strength regarding all failure mechanisms by increasing the crest height, increasing the strength of the revetment and increasing the width of the piping berm.

The second deltadike requirement is cannot be fulfilled. The load on the dike cannot be predicted with certainty for sea-dominated loading conditions (see chapter 3). It is in advance not known if the load is going to be sea- or river-dominated. Structural failure generally results in great flood damage and loss of life while non-structural failure results in moderate flood damage and loss of life. The severity of non-structural failure is moderate and the severity of non-structural failure is decreased by increasing the crest height; The severity of structural failure is barely influenced.

5.3. Case study III: ‘Grebbedijk’

This case study considers the reinforcement of a dike section called ‘Grebbedijk’ adjacent to the river ‘Nederrijn’. This riverine dike is part of dike ring 45 (‘Gelderse
Vallei’). The local situation is shown in Figure 92. The length of the dike is 5350 meter. Both the load on the dike and the strength of the dike are modelled by stochastic variables. Their names, distributions and correlation distances are given in Table B.6 in Appendix B. The load and the strength data of this case study are obtained from the FLORIS project.

![Figure 92 | Overview local situation (background copied from Google Maps, 2012)](image1)

Figure 92 shows the schematized representative cross-section of the dike section.

![Figure 93 | Undistorted and distorted schematized representative cross-section (NAP refers to Amsterdam Ordnance Datum (reference level), MHW refers to Mean High Water)](image2)
Table 8 lists the strength characteristics of the dike. These strength characteristics have been modelled as stochastic variables. The distributions, mean values and variances of these variables are listed in Table B.6 in Appendix B.

**Table 8 | Strength characteristics**

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Mean value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crest level</td>
<td>12.4 [m+NAP]</td>
</tr>
<tr>
<td>Critical discharge over the crest</td>
<td>1 [l/s/m]</td>
</tr>
<tr>
<td>Inclination front face of the structure</td>
<td>1/3 [-]</td>
</tr>
<tr>
<td>Influence factor for the berm</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Influence factor for the wave angle of incidence</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Influence factor for the roughness of the outer slope</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Influence factor for the vertical wall</td>
<td>1.0 [-]</td>
</tr>
<tr>
<td>Seepage length</td>
<td>60.9 [m]</td>
</tr>
<tr>
<td>Thickness cohesive top layer</td>
<td>0.4 [m]</td>
</tr>
<tr>
<td>Thickness sand layer</td>
<td>15.8 [m]</td>
</tr>
<tr>
<td>70-Percentile grain distribution sand layer</td>
<td>30*10^(-4) [m]</td>
</tr>
<tr>
<td>Permeability sand layer</td>
<td>1.16*10^(-4) [m/s]</td>
</tr>
</tbody>
</table>

The load is modelled by a combination of a varying water level and a constant wave height. The exceedance line of the maximum water level is described by the following formula (based on a least-square fit of the common logarithm of the exceedance probabilities provided by PC-RING for the considered location) as shown in Figure 94:

\[ P_{exc}(wl_{max}) = 1 - \exp\left(-\exp\left(-\frac{wl_{max} - 9.3}{0.29}\right)\right) \]

The course of the water level (see Figure 95) is based on the course of the water level during the river flood wave at the Rhine near Lobith in January 1995. This course is scaled to the maximum water level.
Figure 94 | Relationship between the common logarithm of the probability of exceedance of the maximum water level \( P_{exc} \) and the maximum water level \( w_{l_{max}} \)

Figure 95 | Water level course during the 1995 river flood wave near Lobith with the time \( t \) relative to the point in time of maximum water level \( t_{max} \) (data obtained from Rijkswaterstaat, 2012)

The wave height and the wave period are assumed to be constant. In reality the wave height varies due to varying wind speeds, angle of incidence, bottom morphology and sheltering. Including these variations would unnecessarily complicate the calculations as the objective is to show the effect of reinforcement measures and not to model reality as good as possible. As the wave height is
uncorrelated with the water level and therefore the wind speed, the assumed constant wind speed is 4 m/s, which is relatively low. The dike section has a quite constant orientation with a quite constant fetch length of roughly 1500 meter. The spectral wave height and period are calculated using the formulas of Bretschneider (TAW, 1989; Steenbergen et al., 2008) assuming that the spectral wave height and wave period are equal to the significant wave height and wave period; the resulting spectral wave height is 0.1 meter with a spectral wave period of 1.3 seconds.

In order to combine the processes of overtopping and overflow, a function has been fitted through the relevant models describing these processes (see Figure 96). The formulas of these models are found in subsection 2.1.2.

![Figure 96](image1.png)

**Figure 96** | Least square exponential fit of the discharge over the crest ($q$) and the freeboard ($R_c$) on the formulas of Van der Meer (Pullen et al., 2007), Schüttrumph (Pullen et al., 2007) and Nadal and Hughes (2009); some lines overlay each other

### 5.3.1. The probability of failure

This subsection describes and shows the influence of the initial probability of failure and the reinforcement measures on the probability of failure of the dike section. To what degree reinforcement measures decrease the probability of failure is very dependent on the initial distribution of the probability of failure. This distribution is calculated using the DRM (see Appendix B); the result is shown in Table 9 and Figure 97. This result shows that the probability of failure is relatively unevenly distributed over the considered failure mechanisms. The failure mechanism ‘piping’ comprises almost the whole probability of failure.
### Table 9 | Initial probabilities of failure

<table>
<thead>
<tr>
<th>Failure mechanism</th>
<th>Probability of failure [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overflow and/or overtopping</td>
<td>$4.1 \times 10^{-5}$</td>
</tr>
<tr>
<td>Piping</td>
<td>$5.1 \times 10^{-3}$</td>
</tr>
<tr>
<td>Non-structural failure</td>
<td>$5.3 \times 10^{-5}$</td>
</tr>
<tr>
<td>All failure mechanisms</td>
<td>$5.2 \times 10^{-3}$</td>
</tr>
</tbody>
</table>

Figure 97 | Initial distribution of the annual probability of failure of the dike section ($P_f$) over the probability of failure of the failure mechanism ‘overflow and/or overtopping’ ($P_{sf_{or\alpha}}$), the probability of failure of the failure mechanism ‘piping’ ($P_{sf_{piping}}$) and the annual probability of failure of the failure mechanism ‘non-structural failure’ ($P_{nsf}$)

Figure 98 shows the effect of increasing the crest height (which increases the crest level) on the probability of failure. This result shows that the probabilities of failure of the failure mechanisms ‘overflow and/or overtopping’ and ‘non-structural failure’ are decreasing linearly as the crest level increases. Furthermore, the figure shows that the decimation height is about 0.6 meter. Increasing the crest height has a diminishing and insignificant effect on the probability of failure because the two tackled failure mechanisms together comprise only a very small part of the probability of failure. A decrease in probability of failure by a factor 10 or 100 is therefore not reached by performing this single reinforcement measure.

Figure 99 shows the effect of increasing the strength of the revetment (which increases the critical discharge) on the probability of failure. This result shows that the measure has an insignificant effect on the probability of failure of the failure mechanism ‘overflow and/or overtopping’. This is because this dike is designed for and loaded by low waves which causes the discharge over the crest to be very sensitive to a change in water level (see Figure 96). The effect diminishes as this sensitivity increases for higher critical discharges. Increasing the strength of the
revetment has also an insignificant effect on the probability of failure because the tackled failure mechanism does not contribute significantly to the probability of failure. A decrease in probability of failure by a factor 10 or 100 is therefore not reached by performing this reinforcement measure.

Figure 100 shows the effect of increasing the width of the piping berm (which increases the seepage length) on the probability of failure. This result shows that the measure has a predominantly linearly reduces the probability of failure of the failure mechanism ‘piping’. Increasing the width of the piping berm has a diminishing and significant effect on the probability of failure because the tackled failure mechanism does comprise almost the whole probability of failure. A decrease in probability of failure by a factor 10 is therefore reached by performing this single reinforcement measure. A decrease in probability of failure by a factor 100 is not reached by performing this single reinforcement measure.

![Figure 98](image)

**Figure 98** | Effect of increasing the crest level ($c_l$) on the annual probability of failure of the dike section of the failure mechanism ‘overflow and/or overtopping’ ($P_{sf,over}$), the annual probability of failure of the dike section of the failure mechanism ‘piping’ ($P_{sf,piping}$), the annual probability of failure of the dike section of the failure mechanism ‘non-structural failure’ ($P_{nsf}$) and the annual probability of failure of the dike section ($P_f$)
Figure 99 | Effect of increasing the critical discharge \( (q_c) \) on the annual probability of failure of the dike section \( (P_f) \), this probability of failure of the failure mechanism 'overflow and/or overtopping' \( (P_{sf, otfat}) \), this probability of failure of the failure mechanism 'piping' \( (P_{sf, piping}) \) and this probability of failure of the failure mechanism 'non-structural failure' \( (P_{nsf}) \).

Figure 100 | Effect of increasing the seepage length \( (L_s) \) on the annual probability of failure of the dike section \( (P_f) \), this probability of failure of the failure mechanism 'overflow and/or overtopping' \( (P_{sf, otfat}) \), this probability of failure of the failure mechanism 'piping' \( (P_{sf, piping}) \) and this probability of failure of the failure mechanism 'non-structural failure' \( (P_{nsf}) \).
The probability of failure can be decreased by a factor 10 by increasing the width of the piping berm. However, to decrease the probability of by a factor 100, the crest height should be increased as well. Although the increase of the strength of the revetment has no significant effect on the probability of failure it is considered in the remainder of this case study in order to be consistent with the previous case study. The following reinforcement measures are therefore considered in the remainder of this case study:

- Increasing the crest height from 12.4 m+NAP to 13.2 m+NAP;
- Increasing the strength of the revetment from 1 l/s/m to 30 l/s/m;
- Increasing the width of the piping berm from 60.9 m to 90 m.

The magnitude of the reinforcement measures is chosen such that the measures significantly reduce the probability of failure of the failure mechanisms they tackle.

5.3.2. The certainty of the strength

This subsection describes and shows the effect of the reinforcement measures on the certainty of the strength. The fragility curves have been computed using the DRM (developed by the author, see Appendix B). Figure 101 shows the initial fragility curves. This result shows that the aggregated fragility curve is partly dominated by the failure mechanism ‘piping’ and partly dominated by the failure mechanism ‘non-structural failure’ which results in a fairly mild aggregated fragility curve. This is not desirable as the strength is quite uncertain.

![Fragility curves of the initial dike](image)
Figure 102 shows the fragility curves in case the dike is reinforced by increasing the crest height. This result shows that this measure has an undesirable effect as it exposes the mild fragility curve of the failure mechanism ‘piping’.

Figure 103 shows the fragility curves in case the dike is reinforced by increasing the strength of the revetment. This result shows that this measure has no effect on the fragility of the dike. This is firstly because the strength of the failure mechanism ‘overflow and/or overtopping’ is hardly affected by the measure and secondly because the initial fragility curve of the failure mechanism ‘overflow and/or overtopping’ does not dominate the aggregated fragility curve. In case it would the effect would still be insignificant as increasing the strength of the revetment has only a very small influences on the domain of the fragility curve of ‘overflow and/or overtopping’.

Figure 104 shows the fragility curves in case the dike is reinforced by increasing the width of the piping berm. This result shows that this measure has a desirable effect as it exposes the steep fragility curve of the failure mechanism ‘non-structural failure’.

The aggregated fragility curves of the initial dike and the reinforced dikes are shown in Figure 105. The aggregated fragility curve only becomes milder in case the width of the piping berm is increased. This is therefore also the only measure which significantly contributes to an increase in the certainty of the strength. Increasing the piping berm therefore significantly contributes to an increase in the certainty of the strength while increasing the crest height decreases the certainty of the strength and increasing the strength of the revetment has not significant effect on the certainty of the strength.
Figure 102 | Fragility curves of the reinforced dike by increasing the crest level ($P_f$ refers to probability of failure of the dike section, $w_{l_{\text{max}}}$ refers to maximum water level during a loading event)

Figure 103 | Fragility curves of the reinforced dike by increasing the critical discharge ($P_f$ refers to probability of failure of the dike section, $w_{l_{\text{max}}}$ refers to maximum water level during a loading event)
Figure 104 | Fragility curves of the reinforced dike by increasing the seepage length (\( P_f \) refers to probability of failure of the dike section, \( w_{l_{\text{max}}} \) refers to maximum water level during a loading event)

Figure 105 | Aggregated fragility curves of the initial and reinforced dike (\( P_f \) refers to probability of failure of the dike section, \( w_{l_{\text{max}}} \) refers to maximum water level during a loading event)
5.3.3. The severity of failure

Figure 106 shows the severity of structural and non-structural failure. This figure indicates that structural failure results in large water depths in comparison to non-structural failure. The severity of non-structural is mild to moderately severe. The severity of non-structural failure furthermore depends on the greatness of the load and therefore on the probability of exceedance of the maximum water level. This influence is very large because both the discharge over the crest of the dike and the loading period increase from almost zero to very large values for an increase in water level in the order of only a few decimetres.

![Figure 106](image.png)

**Figure 106 | The effect of structural and non-structural failure on the water depth in the polder (dp) over time (t) relative to the point in time of maximum water level in the water body (t_max) depending on the annual probability of exceedance of the maximum water level (P_{exc})**

The water depths which are the result of structural and non-structural failure can be translated into estimations of the flood damage and the loss of life using the results of the computations in chapter 3. Structural failure results in a flood damage of about €420 million and the death of about 40 persons. Non-structural failure at high water levels results in a flood damage of the same order of magnitude as structural failure and the death of about 20 persons.

Figure 107 shows the effect of increasing the crest height on the severity of failure. This figure indicates that the measure hardly affects the severity of structural failure. The final flood depth is the same as this is limited to the maximum flood depth in the polder which is determined by the height of secondary flood defences. What is not shown in the graph is that the severity of structural failure slightly increases as the flow velocities in the breach zone increase. The severity of non-
structural failure is significantly decreased by increasing the crest height. Thus the severity of non-structural failure can be decreased by increasing the crest height.

![Diagram](image)

Figure 107 | The effect of structural failure ($sf$) and non-structural failure ($nsf$) of the dike reinforced by increasing the crest height on the water depth in the polder ($dp$) over time ($t$) relative to the point in time of maximum water level in the water body ($t_{max}$) depending on the annual probability of exceedance of the maximum water level ($P_{exc}$)

5.3.4. Transforming the dike into a deltadike

The first deltadike requirement can be fulfilled. The dike has an initial probability of failure which is quite unevenly distributed over the failure mechanisms. This causes neither of the individual considered measures to reduce the probability of failure significantly by a factor 100. There the probability of failure is reduced significantly by increasing the width of the piping berm (to obtain a decrease in probability of failure by a factor 10) and possibly additionally increasing the crest height (to obtain a decrease in probability of failure by a factor 100).

As the load can already be predicted with certainty, the second deltadike requirement can be fulfilled by increasing the width of the piping berm to such an extent that the failure mechanism ‘piping’ does not dominate failure anymore. Structural failure generally results in great flood damage and loss of life while non-structural failure may result in great flood damage and loss of life depending on the water level. The severity of non-structural failure is moderately decreased by increasing the crest height, the severity of structural failure is hardly affected by any reinforcement measure. The reduction of the severity of failure is however not required to fulfil the second requirement as the predictability can be significantly increased.
These conclusions aim to answer the main research question: *In what way can a dike be transformed into a deltadike by various dike reinforcement alternatives under different types of loading conditions?* This question asks firstly for a definition of the deltadike concept and secondly for insight into the way in which a dike can be transformed into a deltadike by various reinforcement measures in various loading conditions. This study distinguishes coastal, estuarine and riverine loading conditions and the following reinforcement measures (strengthened dike elements):

- Increasing the height of the crest (increases the crest level);
- Increasing the width of the dike (increases the crest width and seepage length).
- Increasing the strength of the revetment (increases the critical discharge).
- Increasing the width of the piping berm (increases the seepage length).

Depending on the context, the text in this conclusion refers either to the actual reinforcement measure or to the increase of the strength of a dike element.

**The definition of the deltadike concept**

The primary function of any dike, also a deltadike, is the prevention of flood damage and loss of life due to flooding. A dike fulfils this primary function by retaining water. Dike failure is defined as the passage of such a volume of water that leads to flood damage or loss of life in (part of) the polder. This is caused by either structural failure (dike breaching) or non-structural failure (overflow and/or overtopping without dike breaching). The distinctive character of a deltadike is that it claims to reduce flood risk by not only imposing requirements to the probability of failure but also to the consequences of failure. Therefore a dike must meet the following deltadike requirements in order to be considered a deltadike:

1. Low probability of failure.
2. Low consequences of failure:
   - Predictable failure (load and strength predictable with certainty) or
   - Mild failure (small physical consequences).

**Conclusion 1:** A deltadike claims to reduce flood risk by imposing requirements to both the probability and consequences of failure. Deltadike failure should therefore be low in probability and either high in predictability or low in severity.

**Transforming a dike into a deltadike**

In case a dike does not meet the deltadike requirements yet, dike reinforcements can contribute to the achievement of these requirements. Reinforcement measures do neither affect the predictability of the load nor decrease the great severity of structural failure. They can therefore contribute to the achievement of the deltadike requirements by:

1. Significantly reducing the probability of failure.
2. Significantly increasing the certainty of the strength of a dike and decreasing the severity of non-structural failure.

**Dikes in general**

The initial distribution of the probability over the failure mechanisms has a great effect on the effectiveness of reinforcement measures in reducing the probability of failure. In case of an even distribution, the strength regarding all failure mechanisms has to be increased to significantly decrease the probability of failure. In case of an uneven distribution, the strength regarding one specific failure mechanism has to be increased to significantly decrease the probability of failure.

Several entrenched views on the deltadike concept are not supported by this study. First, non-structural failure may result in flood damage of the same order of magnitude as structural failure. The severity of non-structural failure mainly depends on the duration of overflow and/or overtopping and the maximum rate of overflow and/or overtopping, the length of the primary dike and the size of the flooded area. This study did however not investigate the effect of varying these characteristics.

**Conclusion 2: Non-structural failure (overflow and/or overtopping without dike breaching) may result in flood damage of the same order or magnitude as structural failure (dike breaching).**

Second, the deltadike is not per definition a wider dike (which increases the crest width and the seepage length). It turned out that increasing the crest width does not make any dike breach-resistant. Increasing the dike width therefore has the same effect as widening the piping berm, which requires less soil and space.

**Conclusion 3: Increasing the crest width does not transform a dike into a deltadike**

Third, the deltadike is not per definition a dike with stronger dike revetment (a higher critical discharge). This reinforcement measure contributes the first deltadike requirement, the significance of this contribution depends on the type of loading conditions (which is returned to further on in this conclusion). It furthermore only contributes significantly to the second deltadike requirement in case of a sea dike (which is returned to further on in this conclusion).

**Conclusion 4: The deltadike is not per definition a dike with a stronger revetment.**
Lastly, increasing the crest level of a dike is in most cases of great importance in transforming a dike into a deltadike. This is because it significantly contributes to the first deltadike requirement by increasing the strength regarding failure mechanisms which in most cases contribute significantly to the probability of failure. Increasing the crest level contributes to the second deltadike requirement by decreasing the severity of non-structural failure as it decreases both the period of time during which overflow and/or overtopping takes place and the rate at which it takes place.

**Conclusion 5:** Increasing the crest level is of great importance in transforming a dike into a deltadike.

**River dikes**

A river dike can be transformed into a deltadike. The first deltadike requirement is fulfilled by increasing the crest height and increasing the width of the piping berm. Strengthening the revetment does not significantly decrease the probability of failure; river dikes with a higher critical discharge are able to withstand water levels which are only slightly higher (a decimetre) as these dikes are designed for and loaded by low waves. The second deltadike requirement is fulfilled by increasing the certainty of the strength as the load on a river dike can be predicted with certainty while the severity of failure is generally large. The increase of the certainty of the strength is obtained by increasing the width of the piping berm (increasing the seepage length) to such an extent that the uncertainty of the strength regarding the failure mechanism 'piping' does not dominate failure anymore. Increasing the critical discharge should also not be applied in view of the second requirement as it leads to unpredictability of the flood damage caused by non-structural failure; at discharges over the crest which are higher than 1 l/s/m only a small increase in water level within the margins of natural variations (for example several centimetres) results in a significant increase in flood damage caused by non-structural failure as the loading period is already prolonged and both the loading period and the discharge over the crest are very sensitive to a change in water level.

**Conclusion 6:** A river dike can be transformed into a deltadike as the probability of failure can be significantly decreased and the predictability of failure can be significantly increased.

**Estuary dikes**

An estuary dike cannot be transformed into a deltadike. The first deltadike requirement is fulfilled by increasing the crest height, increasing the width of the piping berm and increasing the strength of the revetment. Increasing the critical discharge does significantly decrease the probability of failure; estuary dikes with a higher critical discharge are able to withstand water levels which are significantly
higher (a few decimetres) as these dikes are designed for and loaded by moderate waves. The second deltadike requirement cannot be fulfilled. The load cannot be predicted with certainty as it is influenced by the coastal loading conditions; therefore the predictability of failure cannot be increased. The severity of non-structural failure is moderate as the loading time is moderate and both the loading period and the discharge over the crest are moderately sensitive to a change in water level. The dominance of structural failure cannot be significantly decreased because the dike is designed for and loaded by moderate waves.

**Conclusion 7: An estuary dike cannot be transformed into a deltadike because both the predictability of failure cannot be significantly increased and the severity of failure cannot be significantly decreased.**

**Sea dikes**

A sea dike can be transformed into a deltadike. The first deltadike requirement can be fulfilled by increasing the crest height and increasing the strength of the revetment. Increasing the strength of the revetment significantly decreases the probability of failure; sea dikes with a higher critical discharge are able to withstand water levels which are significantly higher (several meters) as these dikes are designed for and loaded by high waves. Increasing the seepage length does not affect the first deltadike requirement as the failure mechanism ‘piping’ does not play a significant role in the failure of sea dikes. The second requirement is fulfilled by increasing the dominance of mild non-structural failure over severe structural failure. This is done by significantly increasing the critical discharge. Non-structural failure is generally mild as the loading period is short and both the loading period and the discharge over the crest are insensitive to a change in water level.

**Conclusion 8: A sea dike can be transformed into a deltadike as the probability of failure can be significantly decreased and the severity of failure can be significantly decreased.**

The conclusions 6, 7 and 8 are summarized in Table 10.

**Table 10 | Overview conclusions 6, 7 and 8**

<table>
<thead>
<tr>
<th>Deltadike requirement</th>
<th>River</th>
<th>Estuary</th>
<th>Sea</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probability of failure</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Consequences of failure</td>
<td>Predictability of failure</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Mildness of failure</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Chapter 7
Recommendations

In case the research is continued it is recommended study the influence of the predictability and severity of failure on the resulting flood damage and loss of life by performing the study in a risk-based design framework. The focus of this study is on dike design only and the relationship between the second deltadike requirement and the consequences of failure in the polder is not studied.

Furthermore, a follow-up research project could investigate how cost-effective a deltadike actually is. This study did not consider costs, it may for example turn out that reducing both the probability of failure and the consequences of failure is not cost-effective in comparison to the traditional dike concept. It could also turn out that it is very cost-effective to decrease the severity of failure while it is very cost-ineffective to increase the predictability of failure.

Besides, this research considered only a few reinforcement measures. The construction of an erosion-resistant clay sill in the dike (Visser, 1998) or the use of modern construction materials such as asphalt or concrete is not considered, just as the use of modern techniques such as sensors. Furthermore, the combination of reinforcement measures including the realization of other functions such as accommodation or recreation (Stalenberg, 2010) is not considered.

In relation to the previous three points, new research could be aimed at investigating which real-life river dikes could actually be transformed into a deltadike based on a risk-based design method. This can be done by looking at which combination of reinforcement measures cost-effectively reduces the probability and consequences of failure of river dikes protecting case-specific polder areas including real-life land use, an uneven distribution of the population over the polder area, the use of pumps in the polder and the simulation of actual flood patterns.
References


References


References


References


References


References


Ter Horst, W.L.A. (2007, August). Effectiveness of ComCoast measures (Project number 06il02, final version).


References


This appendix reasons why the beforehand excluded damage mechanisms were excluded beforehand in section A.1 and presents an analysis of the excluded damage mechanisms in section A.2.

A.I. **Beforehand excluded damage mechanisms**

This section describes why certain damage mechanisms were excluded beforehand. The discussed damage mechanisms are:

- Attack outer slope by wave action;
- Horizontal displacement dike body;
- Erosion foreshore;
- Drifting ice;
- Objects and animals;
- Terrorist attacks.

**Attack outer slope by wave action**

Attack of the outer slope by wave action is the attack of the outer slope by waves. Grassed outer slopes are susceptible to aggregate and block erosion (Mous, 2010). Hard revetments may be damaged by tow erosion, the uplift of blocks, the formation of gullies below the revetment and/or the impact of waves. Along the coasts wave action can lead to severe damage of the outer slope. It is however not likely that damage by waves leads to breaching, as post-event restoration of the outer slope prepares the dike for the next extreme load.

**Horizontal displacement dike body**

Breaching due to the horizontal displacement of the dike core body damage mechanism has recently been identified as it occurred in Wilnis (the Netherlands) in August 2003. In principle the horizontal displacement of the dike body could directly result in breaching of the dike. Due to the drought the mass of the peat dike had been decreased such that the dike was lifted up and pushed away by the water. This phenomenon will not take place in primary dikes consisting of mixtures of sand and clay, as these types of soil are far less vulnerable to drying. Therefore, the fault tree also doesn’t include horizontal displacement of the dike body.

**Erosion foreshore**

Flow along the foreshore can cause erosion of the foreshore, possibly followed by sliding of the foreshore. Erosion of the foreshore is not likely to occur in the Netherlands. The estuaries of the South West delta many embankments have been stabilized and fixed using gabions. The regulation of the rivers prevents large flow velocities near the river embankments. Therefore this damage mechanism is disregarded in this study.
Drifting ice

Drifting ice can both increase the water level (by functioning as a kind of dam or weir) and damage the outer slope revetment (due to the forces of the ice on the revetment). Ice dams can cause flooding even at low discharges. This damage mechanism has endangered river dikes for a long time, but nowadays the rivers are regulated (refer to the previous section on erosion of the foreshore). Therefore this damage mechanism is currently only of importance for lake and sea dikes.

Objects and animals

Objects and animals located in the dike section increase the vulnerability to failure.

Objects include houses, cables, pipelines, trees, etc. The failure of an object can in some cases initiate failure of a dike as well. The broken water pipeline in a canal dike near Stein (the Netherlands) in 2010 contributed to failure of the inner slope. In other cases, a stiff object in combination with flexible soil results in problems such as cracks in the crest of the dike. Ship collision can damage the outer slope of a dike. Lastly, floating rubbish may also damage the outer slope.

Animals also increase the vulnerability to failure. Muskrats and moles construct extended underground connected corridors which increase the permeability of a dike. Cattle and rabbits create weak spots in the inner slope revetment of a dike.

Terrorist attacks

Terrorist attacks aimed at breaching a dike are no longer unrealistic. In May 2012, the Dutch judiciary requested to arrest a man who said he was planning to blow out Dutch dikes. This threat is however not included in this research, as the focus is on structural damage mechanisms which initiate failure.

A.2. Afterward excluded damage mechanisms

This section presents a short analysis of the damage mechanisms excluded after a short analysis. The reasons for excluding these damage mechanisms are mentioned in chapter 2. The afterward excluded damage mechanisms are:

- Sliding inner slope;
- Micro-instability;
- Liquefaction foreshore;
- Sliding outer slope;
- Attack outer slope by wave action.
Appendix
Excluded damage mechanisms

Sliding inner slope

This section describes the damage mechanism ‘sliding inner slope’ (see Figure 108). This damage mechanism is also referred to as ‘macro-instability’. There are several recent cases. Dike instability occurred during several river flood waves. In 1984 a slide of the inner slope took place along the Lek near Streefkerk (Bauduin et al., 1999). In 1995 a crack in the crest of a river dike along the Waal near Ochten indicated macro-instability of the inner slope (TAW, 1995). There were more cases of macro-instability during this high water, such as along the Waal near Wijnfort, the Nederrijn near Angeren and along the Maas near Wijk en Aalburg (TAW, 1995).

![Figure 108 | Sketch of the damage mechanism ‘sliding inner slope’](image)

Sliding of the inner slope is the occurrence of a slide of the inner slope along a slip plane. The driving force of the slide is part of the weight of the sliding soil dike body. The resisting force consists of two parts, which are the other part of the weight of the soil body and the shear stress along the slip plane. Two models of sliding are considered: the Bishop method and the method of Van. The Bishop method considers a circular slip plane (the lower right box of the initiating mechanisms in Figure 108). The method of Van considers a circular slip plane including the uplift of the cohesive top layer at the polder side of the dike (the upper right box of the initiating mechanisms in see Figure 108).

Sliding of the inner slope is more likely to occur when the soil is saturated, as the resisting shear stresses decrease and the driving soil weight increases. This inclines that the damage mechanism is related to high water levels. Therefore the occurrence of ‘overflow and/or overtopping’ is likely when a slide has occurred. At the same time the dike also becomes more susceptible to infiltration of overtopping
Appendix
Excluded damage mechanisms

water, as the slide possibly resulted in cracks in the dike revetment. Any slide may therefore accelerate the damage mechanism of 'overflow and/or overtopping'.

Micro-instability

This section describes the damage mechanism ‘micro-instability’ (see Figure 109). A more descriptive name would be ‘uplift inner slope revetment followed by micro-instability’. This damage mechanism recently occurred during the flood wave of 1995, which resulted in a gap of 1 m³ in a dike along the Ijssel near Hattem (Geodelft, 2002). After this flood wave micro-instability occurred along the Lek near Schoonhoven (TAW, 1995).

![Figure 109 | Sketch of the damage mechanism ‘micro-instability’](image)

Uplift of the inner slope is caused by a high phreatic water line inside the dike, which causes excess water pressure below the inner slope revetment. This is more likely to happen when the inner slope revetment on the slopes are less permeable than the core of the dike. Micro-instability is the transport of sand grains of the sandy dike core along with the water flowing out at the inner slope. Uplift of the inner slope revetment is a conditional event for the transport of grains. When the water, flowing through the dike, brings along sand grains, internal erosion of the dike takes place. Eventually, this leads to local instability of the inner slope.

As uplift and micro-instability are more likely in case of a saturated dike, the damage mechanism is related to high water levels. Therefore the occurrence of overflow and/or overtopping is likely when an uplift and micro-instability have occurred. At the same time the inner slope revetment is damaged and the dike core is vulnerable to infiltration and surface erosion. In course of time this may result in lowering of the crest which marks the start of the breaching process.

Liquefaction foreshore

This section describes the damage mechanism ‘liquefaction foreshore’ (see Figure 110).
Liquefaction occurs when a small increase in shear stress results in rearrangement of loosely packed sand grains. As the packing becomes denser, this creates water overpressure. This ultimately results in a flow slide of the sand body. The sand slides down the slope acting like heavy liquid slurry and settles again creating a very gentle slope. Excess water pressures and loose packing of the grains contribute to the occurrence of this phenomenon called liquefaction (TAW, 2001). Furthermore, the slope of the foreshore should be high and steep enough to generate a flow slide. Erosion for instance is a possible cause of these high and steep slopes. Lastly, the increase of the shear stress should be triggered by certain occasion, such as steepening of the slope, increase in top load, vibrations, wave action or decreasing water levels.

In case liquefaction actually affects the outer slope of the dike, it is not likely to result in failure. Liquefaction is not directly related to high water levels, which makes it possible to restore the embankment before the next high water level.

**Sliding outer slope**

This section describes the damage mechanism ‘sliding outer slope’ (see Figure 111). A recent case occurred during the flood wave of 1993; a slide of the outer slope took place along the Rijn near Ingen two days after the flood wave had passed (TAW, 1994).

The outer slope is vulnerable to sliding in the period after a flood wave has passed during which water levels sometimes drop rapidly. In combination with the low permeability of the outer slope, water pressure inside the dike body responses delayed. The saturated soil is relatively heave, the resisting shear stress is low while there is no stabilizing water pressure from the water in the water body anymore.
During these periods the outer slope therefore has an increased vulnerability to sliding.

A second flood wave may expose the dike core to flowing water and possibly waves. Sliding of the outer slope is directly related to dropping water levels.
Appendix

Dike Reliability Model

The Dike Reliability Model (DRM) estimates the probability of failure of a dike section using the First Order Reliability Method (FORM) procedure based on the fault tree analysis and limit state functions presented in Chapter 2. The content of the model can be viewed upon by making an appointment with the author of this report. The DRM is built in the programming language MATLAB within the open source environment of OpenEarth. The data, models and tools which are available through the OpenEarth environment can be accessed by signing-up through the website of OpenEarth (https://publicwiki.deltares.nl/x/ZwCHAg).

This chapter describes the dike reliability model, the validation of the model and the input of the model during the case studies.

B.1. Description

This section describes what programming language, programming environment, routine are used. Alongside of this it justifies why this has been done. The section furthermore provides an overview of the model using a flow chart.

B.1.1. Programming language and environment

The dike reinforcement cost-benefit model is built in MATLAB within the environment of OpenEarth. OpenEarth is an open source initiative by professionals in marine and coastal engineering. This initiative resulted in a package of open source data, models and tools related to coastal and marine engineering. OpenEarth was useful for this study as the FORM routine is included in the package (Heijer, 2012). Other reasons for using OpenEarth were the provided support and the rationale that the developed Model could contribute positively to the open source environment.

The alternative for this model would have been the use of an advanced reliability analysis package such as PC-RING. The main reasons for programming a new model to compute the probability of failure of dike section were:
- To obtain insight into the relevant processes and computations;
- To be able to easily adapt, iterate and rerun the model, for example in order to compute fragility curves;
- To be able to present the results in figures in the same language and environment.

B.1.2. FORM routine

The First Order Reliability Method (FORM) routine is used to calculate the probability of failure (given the random / stochastic variables and the parameters of the routine). An explanation of this method is given in Vrijling (2002).
This paragraph briefly elucidates on the computations followed by the FORM routine. The text is partly based on Heijer (2012). First order refers to the linearization of the limit state function. The linearization is applied at the design point, which is the point in the limit state or failure surface ($Z = 0$) where the probability density is maximal. This point is not known beforehand and is estimated by an iteration procedure. The reliability index $\beta$ is the quotient between the mean and the standard deviation of the limit state function. In the FORM analysis, this corresponds to the shortest distance between the origin and the failure surface in the normalised $u$-coordinate system. The relationship between the probability of failure and the reliability index is (taking the failure mechanism ‘overflow and/or overtopping’ as an example):

$$P_{sf_{ofot}} = \Phi(-\beta_{ofot})$$

Where:
- $P_{sf_{ofot}}$ = Annual probability of failure of the dike section of the failure mechanism ‘overflow and/or overtopping’ [-]
- $\beta_{ofot}$ = Reliability index of the failure mechanism ‘overflow and/or overtopping’ [-]

### B.1.3. Flow chart

Figure B.1 presents the flow chart of DRM. The probability of failure of a dike section is computed according to this chart. Other graphs shown in the report, such as the fragility curves, are constructed by iterating through this flow chart by changing the water level each time and settings it as deterministic value instead of using the Gumbel (extreme value) distribution. The figures which indicate the reduction in probability of failure for increasing crest level, critical discharge or seepage length are computed by iterating this flow chart for the initial dike profile and each reinforced dike profile.
B.2. Validation

The model was both calibrated and validated using PC-RING. This is a software package which currently used in the Netherlands to perform a reliability analysis for a dike ring system. This is in contrast to DRM, which is a model to calculate the probability of failure of one dike section based on only three failure mechanisms developed for the purpose of this study only.

The input into both DRM and PC-RING is same, see Table B.3. The validation is based on the reliability index of the considered failure mechanisms. The considered failure mechanisms are ‘piping’ and ‘overflow and/or overtopping’ as ‘non-structural failure’ is not directly included in PC-RING.

The differences between DRM and PC-RING are mainly found in the modelling of the failure mechanisms. This subsection discussed the major differences between DRM and PC-RING regarding the failure mechanisms ‘overflow and/or overtopping’ and ‘piping’.

With regard to the failure mechanisms ‘overflow and/or overtopping’ still some major differences are found between PC-RING and DRM. PC-RING considers multiple wind directions (fetch lengths and wind speeds) correlated with the water level.
(therefore varying wave heights for varying wind directions). Also, PC-RING includes closure regimes and set-up of the water level due to wind. Furthermore, the relationship between the freeboard and the discharge over the crest is different. In DRM this relationship is on multiple formulas for overflow and/or overtopping, using the formulas of Van der Meer (Pullen et al., 2007), Schüttrumph (Pullen et al., 2007) and Hughes and Nadal (2009). These formulas are shown in the main report. In PC-RING this relationship is based on the formula of Van der Meer only. The input of PC-RING and DRM regarding the wave height is therefore not the same. The used relationship in DRM is based on a constant wave height is 0.3 m in DRM while in PC-RING the wave height is not specified.

Also with regard to the failure mechanisms ‘piping’ some major differences are found between PC-RING and DRM. The DRM presumes that the occurrence of heave is fully correlated with the formation of a short circuit. Therefore failure due to ‘piping’ is purely based on the limit state function of the formation of a short circuit. In PC-RING these events are not fully correlated; failure due to ‘piping’ occurs only if both events occur.

The model has been validated based on the reliability index (\( \beta \)). The comparison of the \( \beta \)-values regarding the failure mechanism ‘overflow and/or overtopping’ is shown in. The calculation with higher critical discharges gives very deviating results regarding the \( \beta \)-values. It is very likely that this is due to the difference of calculating the governing wave height and subsequently also the relationship between the freeboard and the discharge over the crest. The higher the critical discharge, the more sensitive this relation becomes. When these results are disregarded, the maximum error is 0.2 with a maximum relative error of about 10%.
Table B.1 | Reliability index regarding the probability of failure of overflow and/or overtopping (\( \beta_{\text{ofot}} \) refers to the reliability index of the failure mechanism ‘overflow and/or overtopping’)

<table>
<thead>
<tr>
<th>Crest level [m+NAP]</th>
<th>Seepage Length [m]</th>
<th>Critical discharge [l/s/m]</th>
<th>( \beta_{\text{ofot}} ) DRM [-]</th>
<th>( \beta_{\text{ofot}} ) PC-RING [-]</th>
<th>Error [-]</th>
<th>Relative error [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>40</td>
<td>1</td>
<td>2.2</td>
<td>2.4</td>
<td>0.2</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>40</td>
<td>1</td>
<td>3.8</td>
<td>3.8</td>
<td>-0.1</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>40</td>
<td>1</td>
<td>5.0</td>
<td>5.0</td>
<td>0.0</td>
<td>0.3</td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>10</td>
<td>2.8</td>
<td>5.5</td>
<td>2.7</td>
<td>49</td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>30</td>
<td>8.6</td>
<td>5.6</td>
<td>-3.0</td>
<td>54</td>
</tr>
</tbody>
</table>

The comparison of the \( \beta \)-values regarding the failure mechanism ‘piping’ is shown in Table B.2. The maximum error of the common logarithm of the probability of failure is 0.6. The error is approximately constant, and is about 20%.

Table B.2 | Validation regarding the probability of failure of piping (\( \beta_{\text{piping}} \) refers to the reliability index of the failure mechanism ‘piping’)

<table>
<thead>
<tr>
<th>Crest level [m+NAP]</th>
<th>Seepage Length [m]</th>
<th>Critical discharge [l/s/m]</th>
<th>( \beta_{\text{piping}} ) DRM [-]</th>
<th>( \beta_{\text{piping}} ) PC-RING [-]</th>
<th>Error [-]</th>
<th>Relative error [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>40</td>
<td>1</td>
<td>3.5</td>
<td>2.9</td>
<td>-0.6</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>60</td>
<td>1</td>
<td>4.4</td>
<td>3.9</td>
<td>-0.5</td>
<td>13</td>
</tr>
<tr>
<td>3</td>
<td>80</td>
<td>1</td>
<td>5.1</td>
<td>4.6</td>
<td>-0.5</td>
<td>11</td>
</tr>
</tbody>
</table>
Table B.3 | Overview stochastic variables and their properties of the validation

<table>
<thead>
<tr>
<th>Name variable</th>
<th>Description variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Location parameter</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>g</td>
<td>Constant of gravity</td>
<td>[m²/s]</td>
<td>Deterministic</td>
<td>9.81</td>
<td>-</td>
</tr>
<tr>
<td>v</td>
<td>Kinematic viscosity of water</td>
<td>[m²/s]</td>
<td>Deterministic</td>
<td>1.33E-06</td>
<td>-</td>
</tr>
<tr>
<td>wl</td>
<td>Still water level water body</td>
<td>[m+NAP]</td>
<td>Gumbel for maxima</td>
<td>-1.62</td>
<td>0.18</td>
</tr>
<tr>
<td>cl</td>
<td>Crest level</td>
<td>[m+NAP]</td>
<td>Normal</td>
<td>Variable</td>
<td>0.1</td>
</tr>
<tr>
<td>qc</td>
<td>Critical discharge over the crest</td>
<td>[l/s/m]</td>
<td>Normal</td>
<td>Variable</td>
<td>0.1*1</td>
</tr>
<tr>
<td>mqc</td>
<td>Model factor critical discharge over the crest</td>
<td>[-]</td>
<td>Lognormal</td>
<td>1</td>
<td>0.5</td>
</tr>
<tr>
<td>mqofot</td>
<td>Model factor discharge over the crest</td>
<td>[-]</td>
<td>Lognormal</td>
<td>1</td>
<td>0.5</td>
</tr>
<tr>
<td>wlp</td>
<td>Water level polder</td>
<td>[m+NAP]</td>
<td>Normal</td>
<td>-1</td>
<td>0.1</td>
</tr>
<tr>
<td>gammaw</td>
<td>Volumetric weight water</td>
<td>[kN/m³]</td>
<td>Deterministic</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>gammas</td>
<td>Volumetric weight sand grains</td>
<td>[kN/m³]</td>
<td>Normal</td>
<td>27</td>
<td>0.01*27</td>
</tr>
<tr>
<td>gammactl</td>
<td>Apparent volumetric weight wet cohesive top layer</td>
<td>[kN/m³]</td>
<td>Normal</td>
<td>17</td>
<td>0.05*17</td>
</tr>
<tr>
<td>D0</td>
<td>Thickness cohesive top layer</td>
<td>[m]</td>
<td>Deterministic</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>D1</td>
<td>Thickness sand layer</td>
<td>[m]</td>
<td>Lognormal</td>
<td>2</td>
<td>0.1</td>
</tr>
<tr>
<td>eta</td>
<td>Drag force factor / The coefficient of White</td>
<td>[-]</td>
<td>Lognormal</td>
<td>0.3</td>
<td>0.15*0.3</td>
</tr>
<tr>
<td>theta</td>
<td>Rolling resistance angle sand grains</td>
<td>[°]</td>
<td>Lognormal</td>
<td>43</td>
<td>3</td>
</tr>
<tr>
<td>d70</td>
<td>70-percentile grain diameter sand layer</td>
<td>[m]</td>
<td>Lognormal</td>
<td>6.5*10^(-5)</td>
<td>0.25<em>6.5</em>10^(-5)</td>
</tr>
<tr>
<td>kzb</td>
<td>Permeability sand layer</td>
<td>[m/s]</td>
<td>Lognormal</td>
<td>3.05*10^(-6)</td>
<td>1.2<em>3.05</em>10^(-6)</td>
</tr>
<tr>
<td>Ls</td>
<td>Seepage length</td>
<td>[m]</td>
<td>Lognormal</td>
<td>Variable</td>
<td>0.05*48.9</td>
</tr>
<tr>
<td>mS</td>
<td>Model factor critical hydraulic head</td>
<td>[-]</td>
<td>Lognormal</td>
<td>1</td>
<td>0.08*1</td>
</tr>
<tr>
<td>Lds</td>
<td>Length dike section</td>
<td>[m]</td>
<td>Deterministic</td>
<td>3,500</td>
<td>-</td>
</tr>
</tbody>
</table>
### Table B.4 | Overview stochastic variables and their properties of Case study I ('Pettemer Zeewering ')

<table>
<thead>
<tr>
<th>Name variable</th>
<th>Description variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Location parameter</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>g</td>
<td>Constant of gravity</td>
<td>[m$^2$/s]</td>
<td>Deterministic</td>
<td>9.81</td>
<td>-</td>
</tr>
<tr>
<td>v</td>
<td>Kinematic viscosity of water</td>
<td>[m$^2$/s]</td>
<td>Deterministic</td>
<td>1.33E-06</td>
<td>-</td>
</tr>
<tr>
<td>wl</td>
<td>Still water level water body</td>
<td>[m+NAP]</td>
<td>Gumbel for maxima</td>
<td>-2.22</td>
<td>0.27</td>
</tr>
<tr>
<td>cl</td>
<td>Crest level</td>
<td>[m+NAP]</td>
<td>Normal</td>
<td>12.90</td>
<td>0.1</td>
</tr>
<tr>
<td>qc</td>
<td>Critical discharge over the crest</td>
<td>[l/s/m]</td>
<td>Normal</td>
<td>10</td>
<td>0.1*10</td>
</tr>
<tr>
<td>mqc</td>
<td>Model factor critical discharge over the crest</td>
<td>[-]</td>
<td>Lognormal</td>
<td>1</td>
<td>0.5</td>
</tr>
<tr>
<td>mqofot</td>
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<td>[-]</td>
<td>Lognormal</td>
<td>1</td>
<td>0.5</td>
</tr>
<tr>
<td>Lds</td>
<td>Length dike section</td>
<td>[m]</td>
<td>Deterministic</td>
<td>1,000</td>
<td>-</td>
</tr>
<tr>
<td>Ap</td>
<td>Polder area</td>
<td>[m$^2$]</td>
<td>Deterministic</td>
<td>1,000,000</td>
<td>-</td>
</tr>
<tr>
<td>dpc</td>
<td>Critical water depth polder</td>
<td>[m]</td>
<td>Normal</td>
<td>0.1</td>
<td>0.1*0.1</td>
</tr>
</tbody>
</table>
## Table B.5 | Overview stochastic variables and their properties of Case study II ('Kop van 't Land')

<table>
<thead>
<tr>
<th>Name variable</th>
<th>Description variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Location parameter</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>g</td>
<td>Constant of gravity</td>
<td>[m$^2$/s]</td>
<td>Deterministic</td>
<td>9.81</td>
<td>-</td>
</tr>
<tr>
<td>v</td>
<td>Kinematic viscosity of water</td>
<td>[m$^2$/s]</td>
<td>Deterministic</td>
<td>1.33E-06</td>
<td>-</td>
</tr>
<tr>
<td>wl</td>
<td>Still water level water body</td>
<td>[m+NAP]</td>
<td>Gumbel for maxima</td>
<td>-1.62</td>
<td>0.18</td>
</tr>
<tr>
<td>cl</td>
<td>Crest level</td>
<td>[m+NAP]</td>
<td>Normal</td>
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<td>0.1</td>
</tr>
<tr>
<td>qc</td>
<td>Critical discharge over the crest</td>
<td>[l/s/m]</td>
<td>Normal</td>
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<td>0.1*1</td>
</tr>
<tr>
<td>mqc</td>
<td>Model factor critical discharge over the crest</td>
<td>[-]</td>
<td>Lognormal</td>
<td>1</td>
<td>0.5</td>
</tr>
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<td>mqofot</td>
<td>Model factor discharge over the crest</td>
<td>[-]</td>
<td>Lognormal</td>
<td>1</td>
<td>0.5</td>
</tr>
<tr>
<td>wlp</td>
<td>Water level polder</td>
<td>[m+NAP]</td>
<td>Normal</td>
<td>-1</td>
<td>0.1</td>
</tr>
<tr>
<td>gammaw</td>
<td>Volumetric weight water</td>
<td>[kN/m$^3$]</td>
<td>Deterministic</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>gammas</td>
<td>Volumetric weight sand grains</td>
<td>[kN/m$^3$]</td>
<td>Normal</td>
<td>27</td>
<td>0.01*27</td>
</tr>
<tr>
<td>gammactl</td>
<td>Apparent volumetric weight wet cohesive top layer</td>
<td>[kN/m$^3$]</td>
<td>Normal</td>
<td>17</td>
<td>0.05*17</td>
</tr>
<tr>
<td>D0</td>
<td>Thickness cohesive top layer</td>
<td>[m]</td>
<td>Deterministic</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>D1</td>
<td>Thickness sand layer</td>
<td>[m]</td>
<td>Lognormal</td>
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<td>0.1</td>
</tr>
<tr>
<td>eta</td>
<td>Drag force factor / The coefficient of White</td>
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<td>Lognormal</td>
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<td>0.15*0.3</td>
</tr>
<tr>
<td>theta</td>
<td>Rolling resistance angle sand grains</td>
<td>[°]</td>
<td>Lognormal</td>
<td>43</td>
<td>3</td>
</tr>
<tr>
<td>d70</td>
<td>70-percentile grain diameter sand layer</td>
<td>[m]</td>
<td>Lognormal</td>
<td>6.5*10^(-5)</td>
<td>0.25<em>6.5</em>10^(-5)</td>
</tr>
<tr>
<td>kzb</td>
<td>Permeability sand layer</td>
<td>[m/s]</td>
<td>Lognormal</td>
<td>3.05*10^(-6)</td>
<td>1.2<em>3.05</em>10^(-6)</td>
</tr>
<tr>
<td>Ls</td>
<td>Seepage length</td>
<td>[m]</td>
<td>Lognormal</td>
<td>48.9</td>
<td>0.05*48.9</td>
</tr>
<tr>
<td>mS</td>
<td>Model factor critical hydraulic head</td>
<td>[-]</td>
<td>Lognormal</td>
<td>1</td>
<td>0.08*1</td>
</tr>
<tr>
<td>Lds</td>
<td>Length dike section</td>
<td>[m]</td>
<td>Deterministic</td>
<td>3,500</td>
<td>-</td>
</tr>
<tr>
<td>Ap</td>
<td>Polder area</td>
<td>[m$^2$]</td>
<td>Deterministic</td>
<td>1,000,000</td>
<td>-</td>
</tr>
<tr>
<td>dpc</td>
<td>Critical water depth polder</td>
<td>[m]</td>
<td>Normal</td>
<td>0.1</td>
<td>0.1*0.1</td>
</tr>
</tbody>
</table>
### Table B.6 | Overview stochastic variables and their properties of Case study III ('Grebbedijk')

<table>
<thead>
<tr>
<th>Name variable</th>
<th>Description variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Location parameter</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>g</td>
<td>Constant of gravity</td>
<td>[m^2/s]</td>
<td>Deterministic</td>
<td>9.81</td>
<td>-</td>
</tr>
<tr>
<td>v</td>
<td>Kinematic viscosity of water</td>
<td>[m^2/s]</td>
<td>Deterministic</td>
<td>1.33E-06</td>
<td>-</td>
</tr>
<tr>
<td>wl</td>
<td>Still water level water body</td>
<td>[m+NAP]</td>
<td>Gumbel for maxima</td>
<td>-9.34</td>
<td>0.29</td>
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<tr>
<td>cl</td>
<td>Crest level</td>
<td>[m+NAP]</td>
<td>Normal</td>
<td>12.43</td>
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<td>qc</td>
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<td>0.1*1</td>
</tr>
<tr>
<td>mqc</td>
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<td>Lognormal</td>
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<td>0.15</td>
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<tr>
<td>mqofot</td>
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<td>Lognormal</td>
<td>1</td>
<td>0.15</td>
</tr>
<tr>
<td>wlp</td>
<td>Water level polder</td>
<td>[m+NAP]</td>
<td>Normal</td>
<td>7.1</td>
<td>0.1</td>
</tr>
<tr>
<td>gammaw</td>
<td>Volumetric weight water</td>
<td>[kN/m^3]</td>
<td>Deterministic</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>gammas</td>
<td>Volumetric weight sand grains</td>
<td>[kN/m^3]</td>
<td>Normal</td>
<td>27</td>
<td>0.01*27</td>
</tr>
<tr>
<td>gammactl</td>
<td>Apparent volumetric weight wet cohesive top layer</td>
<td>[kN/m^3]</td>
<td>Normal</td>
<td>17</td>
<td>0.05*17</td>
</tr>
<tr>
<td>D0</td>
<td>Thickness cohesive top layer</td>
<td>[m]</td>
<td>Deterministic</td>
<td>0.4</td>
<td>-</td>
</tr>
<tr>
<td>D1</td>
<td>Thickness sand layer</td>
<td>[m]</td>
<td>Lognormal</td>
<td>15.8</td>
<td>0.2*15.8</td>
</tr>
<tr>
<td>eta</td>
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<td>[-]</td>
<td>Lognormal</td>
<td>0.3</td>
<td>0.1*0.3</td>
</tr>
<tr>
<td>theta</td>
<td>Rolling resistance angle sand grains</td>
<td>[°]</td>
<td>Lognormal</td>
<td>43</td>
<td>3</td>
</tr>
<tr>
<td>d70</td>
<td>70-percentile grain diameter sand layer</td>
<td>[m]</td>
<td>Lognormal</td>
<td>3.0*10^-4</td>
<td>0.304<em>3.0</em>10^-4</td>
</tr>
<tr>
<td>kzb</td>
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<td>[m/s]</td>
<td>Lognormal</td>
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<td>1.681<em>1.16</em>10^-4</td>
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</tr>
<tr>
<td>mS</td>
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<td>Lognormal</td>
<td>1</td>
<td>0.08*1</td>
</tr>
<tr>
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<td>Length dike section</td>
<td>[m]</td>
<td>Deterministic</td>
<td>5,350</td>
<td>-</td>
</tr>
<tr>
<td>Ap</td>
<td>Polder area</td>
<td>[m^2]</td>
<td>Deterministic</td>
<td>1,000,000</td>
<td>-</td>
</tr>
<tr>
<td>dpc</td>
<td>Critical water depth polder</td>
<td>[m]</td>
<td>Normal</td>
<td>0.1</td>
<td>0.1*0.1</td>
</tr>
</tbody>
</table>