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# Safety Standards of Flood Defenses

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ABSTRACT: Current design codes like the Eurocode use safety or reliability classes to assign target reliabilities to different types of structures or structural members according to the potential consequences of failure. That, in essence, is a risk-based criterion. A wide range of structures is designed with such codes, and distinction is made between reliability classes. These reliability classes are not necessarily well suited for flood defense systems, neither are the design rules and partial safety factors, which are calibrated for a wide range of standard applications. For a flood defense system protecting a large area from flooding, on the other hand, it is worthwhile to base the design and safety assessment standards on a risk assessment - a tailor-made solution. The investments can be considerable and the stakes are high, especially for low-lying delta areas, where the consequences of flooding can be devastating. In order to answer the question "How safe is safe enough?" a framework for acceptable risk is required. Subsequently, from acceptable risk we can deduce target reliabilities for the protection system as a whole as well as for its elements. For practical application, these target reliabilities can then be translated into design and assessment rules; for example, using LRFD (load and resistance factor design) to derive partial safety factors.

This paper describes how to define safety standards for flood defenses, in particular dikes, step-by-step. An important aspect in translating high-level requirements into specific (low-level) design rules that apply to specific failure modes for specific flood protection elements is the so-called "length-effect". This is especially relevant for long-linear structures like dikes, where usually the length is much larger than the scale of fluctuation of dominant load or resistance properties. The longer the structure, the higher the chance to encounter either and extreme load or a weak spot (i.e., low resistance) – hence the word "length-effect". The effect is that the probability of failure increases with the length of the dike. The implication for design and assessment rules is that the reliability requirements to a cross section ("zero length") need to be stricter (i.e., higher target reliability) than for the whole reach.

This paper attempts to demonstrate how tailor-made safety standards for large scale flood defense systems can be derived in a risk-based fashion. Since flood defenses differ from smaller scale geotechnical structures in many aspects and given the volume of investments in such large-scale engineering systems, it is very attractive to deviate from the standard design codes. That is not deviating conceptually, but rather deriving safety factors for the specific application to better account for the characteristics and uncertainties involved. The authors strive to show that safety levels and partial safety factors in the presented approach are far from arbitrary. They are part of an overall consistent flood risk framework, a framework that provides a link between geotechnical engineers and other disciplines involved in providing safety from flooding.

*Keywords: flood defenses, acceptable risk, uncertainties, probability of failure, length-effect, LRFD*

## 1 INTRODUCTION

Current design codes like Eurocode use safety or reliability classes to assign target reliabilities to different types of structures or structural members according to the potential consequences of failure. That, in essence, is a risk-based criterion. Also the design life plays a role in assigning target reliabilities. Due to

the wide range of structures design with such codes, a differentiation with, for example, three reliability classes makes sense. Because it is not (yet) realistic to design each structure using risk-assessment techniques. For a flood defense system protecting a large area from flooding, on the other hand, it is worthwhile to base the design and safety assessment standards on a risk assessment. The investments can be considerable and the stakes are high, especially for low-lying delta areas, where the consequences of flooding can be devastating. Therefore, tailor-made solutions become much more attractive.
The basic underlying question is "How safe is safe enough?". In order to answer that question a framework for acceptable risk is required. Having established acceptable risk we can deduce a target reliabilities for the protection system as well as for its elements. For practical application, these target reliabilities can then be translated into design and assessment rules; for example, using LRFD (load and resistance factor design) to derive partial safety factors.

This paper describes how to define safety standards for flood defenses, in particular dikes, step-by-step. The first step is to define what is socially acceptable. To this end, often is relied on fatality risk criteria, the risk of individuals of dying due to flooding or the number of expected fatalities. Next, economic considerations play a role, in which the cost of flood protection is weighed against the risk-reduction achieved by improved protection. These criteria allow decision makers to decide on protection standards in form of target reliabilities.

Such target reliabilities are high-level requirements in a sense that they are expressed in terms of the acceptable probability of failure of the flood protection (sub)system under consideration. In order to ensure the protection level of the (sub)system its elements need to be designed with higher target reliabilities. That is because typically flood defenses are linear defenses, in which failure of any element leads to system failure; a dike breach anywhere leads to flooding. From a system reliability point of view, flood defense system are serial systems where the probability of failure is dominated by the weakest links. In fact, the same holds for the different failure mechanisms; any mechanism may cause failure of an element (e.g., dike section).

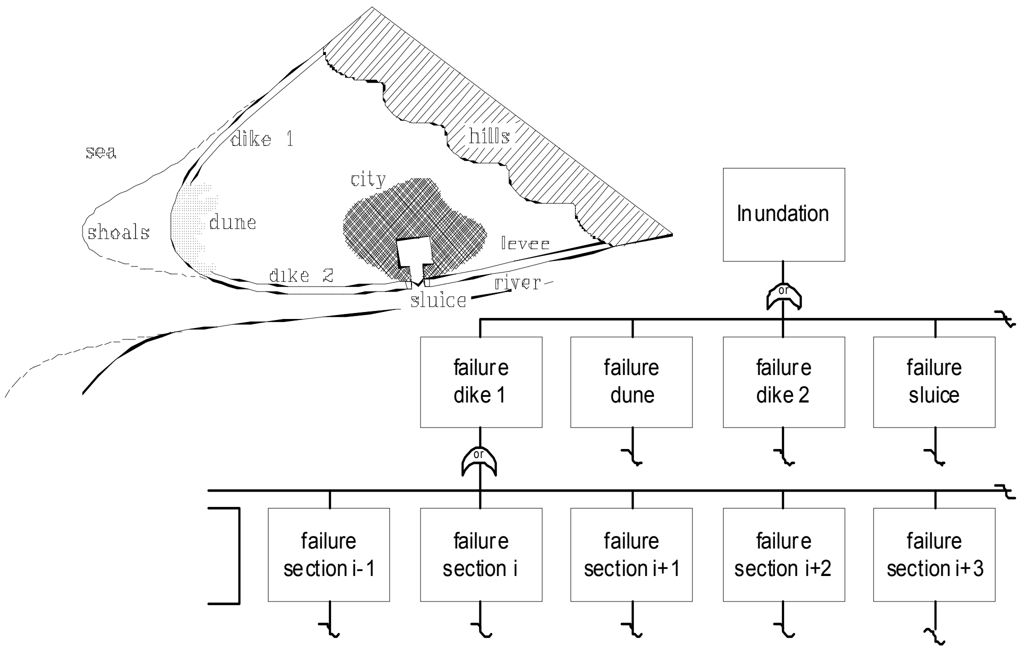


Figure 1. Schematic Overview of a Flood Defense System and its Elements

An important aspect in translating high-level requirements into specific (low-level) design rules that apply to specific failure modes for specific flood protection elements is the so-called "length-effect". This is especially relevant for long-linear structures like dikes, where usually the length is much larger than the scale of fluctuation of dominant load or resistance properties. The longer the structure, the higher the chance to encounter either and extreme load or a weak spot (i.e., low resistance) – hence the word "length-effect". The effect is that the probability of failure increases with the length of the dike. The implication for design and assessment rules is that the reliability requirements to a cross section ("zero length") need to be stricter (i.e., higher target reliability) than for the whole reach.

The organization of this paper follows the top-down structure as described above, from high-level to low level requirements. Section 2 addresses the acceptable risk criteria, followed by an inventory of the failure mechanisms considered in dike design in section 3, enriched by failure observations from New Orleans with Hurricane Katrina (2005). The length-effect is discussed and illustrated in section 4. Section 5 describes the steps from acceptable risk to design rules and partial safety factors. The paper finishes with a discussion in section 6.

## 2 ACCEPTABLE FLOOD RISK

### 2.1 *Acceptable Risk Framework*

Protection of individuals and groups against natural and man-made hazards is a task of human civilizations. Historically, most protection efforts were realized after major disasters, the consequences still being very much present in the collective memory. Modern risk-based approaches aim to enable preventive protection by identifying risks, before they manifest themselves as disasters. Risk is defined as the probability of an (unwanted) event times the consequences involved. Expressing them (amongst others) in monetary terms and fatalities is a means to enable weighing investments in prevention against the benefits of risk reduction.

The estimation of the consequences of flooding is a central element in flood risk analysis and management. The totality of flood damage comprises casualties, material and economic damage as well as the loss of or harm to immaterial values like works of art and amenity. However, for practical reasons the notion of risk in a societal context is often reduced to the total number of casualties using a definition as: "the relation between frequency and the number of people suffering from a specified level of harm in a given population from the realization of specified hazards". If the specified level of harm is limited to loss of life, the societal risk may be modeled by the frequency of exceedance curve of the number of deaths, also called the FN-curve (see 2.3).

The consequence part of a risk can also be limited to the material damage expressed in monetary terms. It should be noted however, that the reduction of the consequences either measure may not adequately model the public's perception of the potential loss. The simplification clarifies the reasoning at the cost of accuracy. Nevertheless, for practical tractability, three criteria are defined and used in the following:

1 individual risk
2 group risk
3 economical risk

The first two are belong to the category of “loss of life” criteria, which are often considered as boundary conditions providing minimum protection level. While individual risk refers to the probability of dying of an individual person in a specific location, group risk refers to large numbers of fatalities in one event. Economical risk refers to the direct and indirect economical consequences of a disaster, allowing for a direct comparison of investments in and effects of prevention in monetary terms. Both, group risk and economical risk are considered societal risk criteria, because they are usually applied (i.e., aggregated) on a national scale.

### 2.2 *Individual Risk*

Individual risk is defined as the probability of an individual residing in a given area to die as a consequence of flooding. This probability includes the nature of the hazard (i.e., probabilities of discharge, water level, wind, waves etc.), the effectiveness of the flood protection system (e.g., probability of a dike breach) and the conditional flood characteristics (e.g., water depth, flow velocity). Jonkman (2007) discusses loss of life related to flooding extensively. Individual risk is typically represented in risk maps; an example is given in Figure 2.

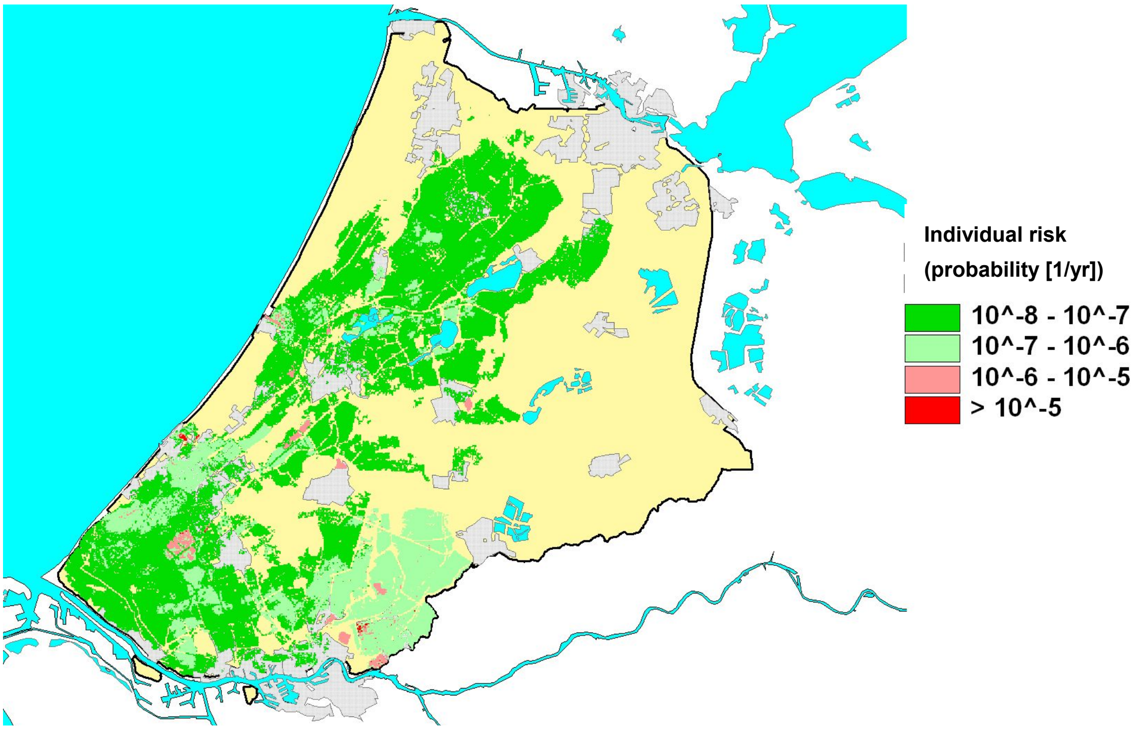

Figure 2. Individual Risk Central Holland, probability of dying to flooding [1/yr] (Jonkman, 2007)

Being able to determine individual risks with flood risk analysis, the question remains what is acceptable. The same question plays a role for many other hazards, especially in external safety (e.g., transport and storage of hazardous goods, chemical plants etc. An indicative figure for acceptable individual risk used in many applications is $10^{-6}$ per year (e.g. Lerche et. al, 2006).

One method to determine such acceptance limits to using revealed preferences (Vrijling et. al, 1993). That is done by analyzing accident statistics and differentiating between the activities during which persons lost their lives. The fact, that the actual personal risk levels connected to various activities show statistical stability over the years and are approximately equal for the Western countries, indicates a consistent pattern of preferences. The probability of losing one's life in normal daily activities such as driving a car or working in a factory appears to be one or two orders of magnitude lower than the overall probability of dying. Only a purely voluntary activity such as mountaineering entails a higher risk (Figure 3).

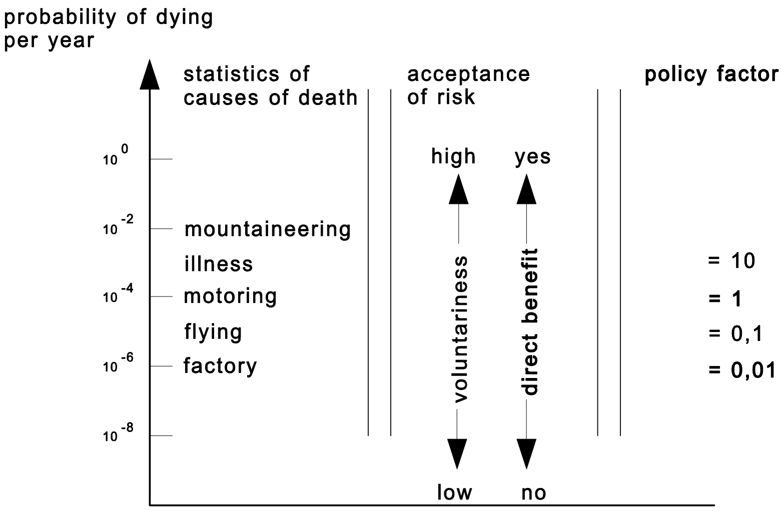

Figure 3. Personal risks in Western countries, deduced from the statistics of causes of death and the number of participants per activity (Vrijling et. al, 1993)

Apart from a slightly decreasing trend of the death risks presented, probably due to technical progress, it seems appropriate to use revealed preferences as a basis for decisions with regard to the personally acceptable probability of an accident (failure) $P_{fi}$ in the following way:

$$P_{fi} = \frac{\beta_i \cdot 10^{-4}}{P_{d|fi}} \qquad (1)$$

where $P_{d|fi}$ denotes the probability of being killed in the event of an accident. In this expression the policy factor $\beta_i$ varies with the degree of voluntariness with which an activity $i$ is undertaken and with the benefit perceived. It ranges from 100 in the case of complete freedom of choice like mountaineering to 0.01 in case of an imposed risk without any perceived direct benefit (such a large range was already noted in 1969 by Starr). The latter is also applied as individual risk criterion for hazardous installation nears housing areas without any direct benefit to the inhabitants. A proposal for the choice of the value of the policy factor $\beta_i$ as a function of voluntariness and benefit is given in the table below. For the flood defenses a $\beta_i$-value of 1.0 to 0.1 seems appropriate.

Table 1. The value of the policy factor $\beta_i$ as a function of voluntariness and benefit (Vrijling et. al, 1993)

| policy factor $\beta_i$ | voluntariness | direct benefit | example |
|---|---|---|---|
| 100 | voluntary | direct benefit | mountaineering |
| 10 | voluntary | direct benefit | motor biking |
| 1.0 | neutral | direct benefit | car driving |
| 0.1 | involuntary | some benefit | factory |
| 0.01 | involuntary | no benefit | LPG-station |

## 2.3 *Group Risk*

Another perspective on loss of life besides individual risk is the total number of people that would drown in one flood event. Considering impact on society, single events with large numbers of fatalities (e.g., a place crash with 200 casualties) are less acceptable than large numbers of accidents with small number of fatalities (e.g., 100 car accidents with 2 casualties each). Thus, with group risk the so called risk-averseness (Bernoulli, 1783) enters the assessment.

Since a flood-protected area can inundate due to breaches at various locations and in different scenarios, an FN-curve is an appropriate way to represent this type of risk. As an example the FN-curve of the Brielse polder area in the Netherlands is shown in Figure 4.

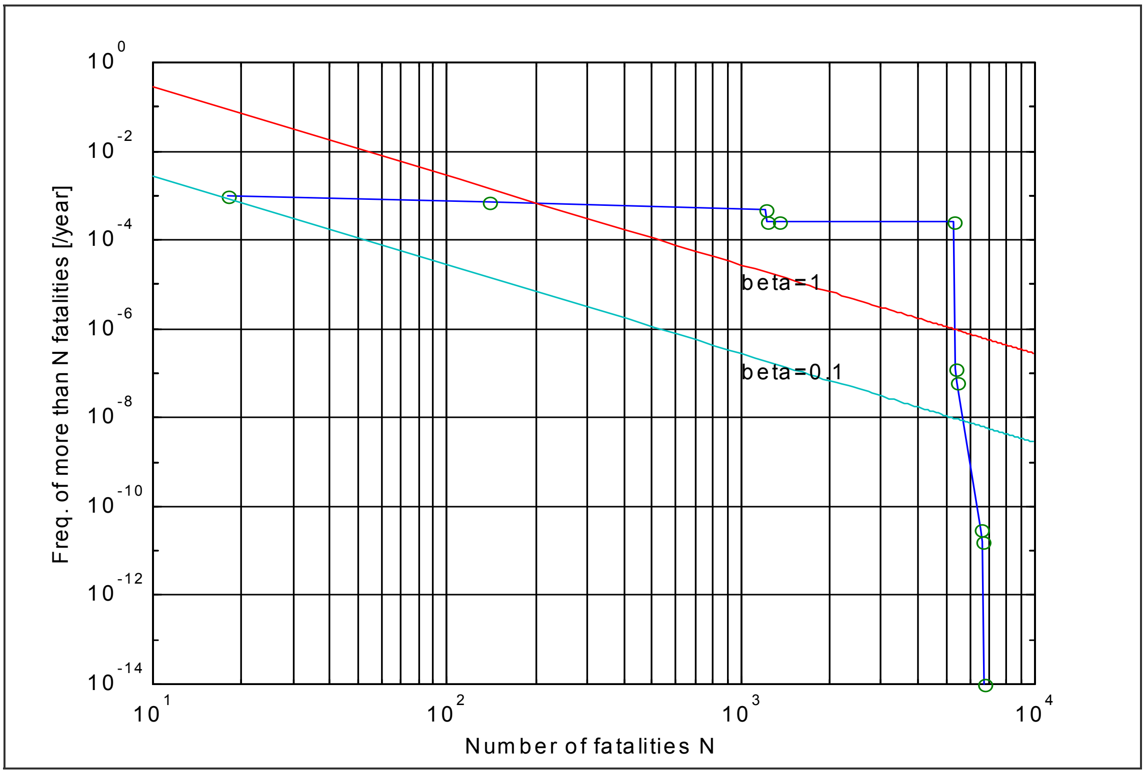


Figure 4. FN-curve for Flooding of the Brielse Polder (NL)

An FN-curve plots the number of expected fatalities per flood scenario over its corresponding occurrence probability. The FN-curve is the description of the current situation or a future scenario and, as for the individual risk, an acceptance criterion is needed. Jonkman (2007) discusses such criteria in detail.

## 2.4 *Economic Optimization*

While the loss of life-related acceptance criteria discussed above aim to define minimum safety criteria, from an economic point of view an optimal protection standard can be found by balancing the cost of protection against the benefit of risk reduction. In other words, the economically optimal probability of failure $P_{f,opt}$ is the one, for which the (marginal) investment $I$ in a safer flood defense system is equals the (marginal) benefit by the decreasing present value of the risk.

$$\min(Q) = \min(I(P_{f,opt}) + PV(P_{f,opt}\, D)) \qquad (2)$$

where $Q$ is the total cost, $PV$ the present value operator and $D$ the total damage in case of flood defense failure and subsequent flooding.

If (despite ethical objections) the value of a human life is rated at $d$, the amount of damage is increased by $P_{d|f}\, N_p\, d$ , where $N_p$ = number of casualties. A typical value chosen for $d$ is the present value of the net national product per inhabitant. The advantage of taking the possible loss of lives into account in economic terms is that the safety measures are affordable in the context of the national income (see also Vrijling and Van Gelder, 2000).

Omitting the value of human life, the decision problem as formulated by the Delta Committee (van Dantzig, 1953) is given below. The investment $I(h)$ in the protective dike system is given as a function of the crest level h by:

$$I(h) = I_0 + I_1(h - h_0) \qquad (3)$$

where $I_0$ is the initial cost (i.e., mobilization), $I_1$ is the marginal cost of raising the dike and $h_0$ is the current dike crest level. The probability of exceedance of the crest level of the dike is approximated by a shifted exponential distribution:

$$1 - F(h) = e^{-\frac{h-A}{B}} \qquad (4)$$

where in this example the location parameter is A=1.96m and the scale parameter B=0.33m. The risk of inundation in this simplified example is equal to the probability of exceedance of the dike crest times the damage $D$ in case of inundation.

$$Risk = e^{-\frac{(h-A)}{B}} \cdot D \qquad (5)$$

Because the risk is present every year the present value of the risk over an infinite period has is given by its present value.

$$PV(Risk) = e^{-\frac{(h-A)}{B}} \cdot \frac{D}{r} \qquad (6)$$

where $r$ is the rate of interest. The total cost is the sum of the investment and the present value of the remaining risk that is accepted.

$$Q(h) = I_0 + I_1(h - h_0) + e^{-\frac{(h-A)}{B}} \frac{D}{r} \qquad (7)$$

Differentiating the total cost with respect to the decision variable $h$ and equating the derivative to 0 gives a rather elegant result.

$$\frac{\partial Q(h)}{\partial h} = I_1 - \frac{1}{B} e^{-\frac{(h-A)}{B}} \cdot \frac{D}{r} = 0 \qquad (8)$$

$$P_{f,opt} = e^{-\frac{(h_{opt}-A)}{B}} = \frac{I_1 B r}{D} \qquad (9)$$

The last expression shows that the acceptable probability increases with the marginal cost of dike construction, with the standard deviation of the storm surge level B and the rate of interest. It decreases with the damage that will occur in case of an inundation.

The Delta Committee (van Dantzig, 1953) calculated an economically optimal probability of inundation for Central Holland in 1960 to be 8 10-6 per year (Figure 5).

Some approximate calculations performed by Dutch engineers in 2006 indicated a level of $10^{-3}$ per year for New Orleans. The city was protected against a hurricane category 3 with a return period of 30 to 100 years. The present system that was resurrected after Katrina has the same safety level.

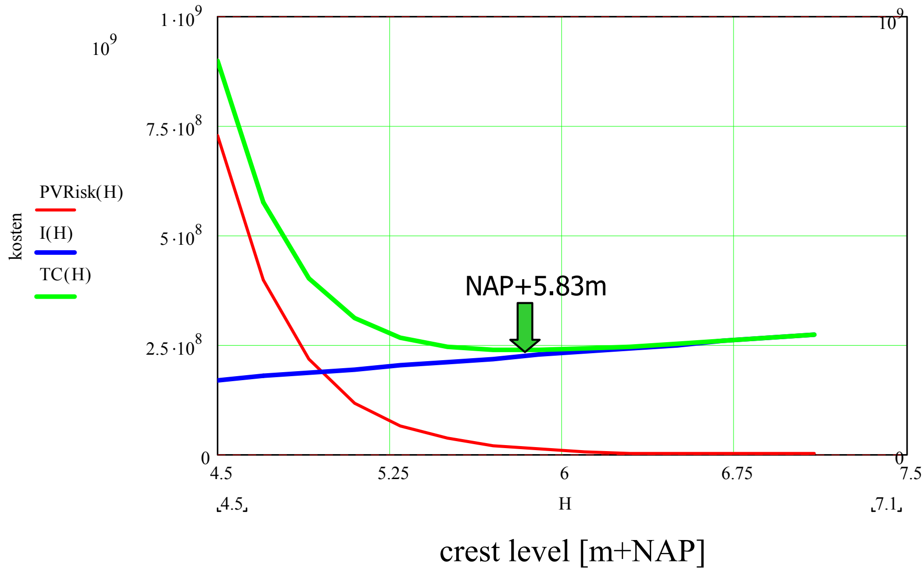


Figure 5. Example Economic Optimization: Optimal Crest Level

The economic criterion presented above should be adopted as a basis for the "technical" input to the political decision process. All information of the risk assessment should be available in the political process. It is emphasized that the decision remains a political one.

Another important remark is that in the historical approach the crest height of the dike was the main resistance parameter, as illustrated here for sake of illustration. Nowadays, such analyses are carried out, analyzing the cost to reach a certain protection level in terms of probability of failure; thus, including all kinds of failure mechanisms in addition to overtopping. The most dominant mechanisms being of geotechnical nature such as instability of the inner slope or piping.

### 2.5 *Summary*

For large engineering systems like flood defense systems it is worthwhile to determine taylor-made safety standards instead of relying on rather coarse consequence and reliability classes as in the Eurocode or other design codes. In order to establish appropriate target reliabilities, one needs to assess what risk is acceptable. A practical approach to this problem is to look at loss-of-life risk criteria on the one hand and at economical criteria on the other. For loss of life risks, individual risk is typically distinguished from group risk. Both give indications of desirable minimum protection standards. Economically optimal protection, on the contrary, seeks to balance investments and benefits in terms of reduced flood risk monetizing the damage. In principle, the most stringent criterion is to be applied. In other words, the highest target reliability derived from the three criteria should be adopted as target reliability of the flood protection system from a technical point of view.

The following two sections will deal with the failure mechanisms to be taken into account in designing flood defenses and how the target reliability on system level can be translated into practicable design rules for dikes.

## 3 FAILURE MODES AND LESSONS FROM NEW ORLEANS

While dikes in the field seem straight-forward engineered structures, their behavior can be complicated. This section deals with the physical behavior of flood defenses, especially in terms of failure mechanisms. The geotechnical aspects are very important due to the typically large uncertainties in ground conditions. Both theory and practical observations are discussed.

### 3.1 *Failure modes of flood defense systems*

Probabilistic design and safety assessment methods have raised the awareness, that the probability of exceedance of the design water level (or the reciprocal: the return period) is not an accurate predictor of the probability of flooding. Traditionally, the crest height is determined by such design water levels and the dike is designed according to design rules. However, other mechanisms, and certainly geotechnical ones like slope failure of piping can result in sudden failure and are poorly accounted for in design frequency approaches. More failure mechanisms than overtopping need to be accounted for, if the reliability target refers to the probability of flooding rather than the probability of a certain load condition (see Figure 6). As a single dike is only one element, the flood defense whole system should be considered (see Figure 1), which is only as strong as its weakest link; hence, the importance of the geotechnical mechanisms and the subsoil conditions.

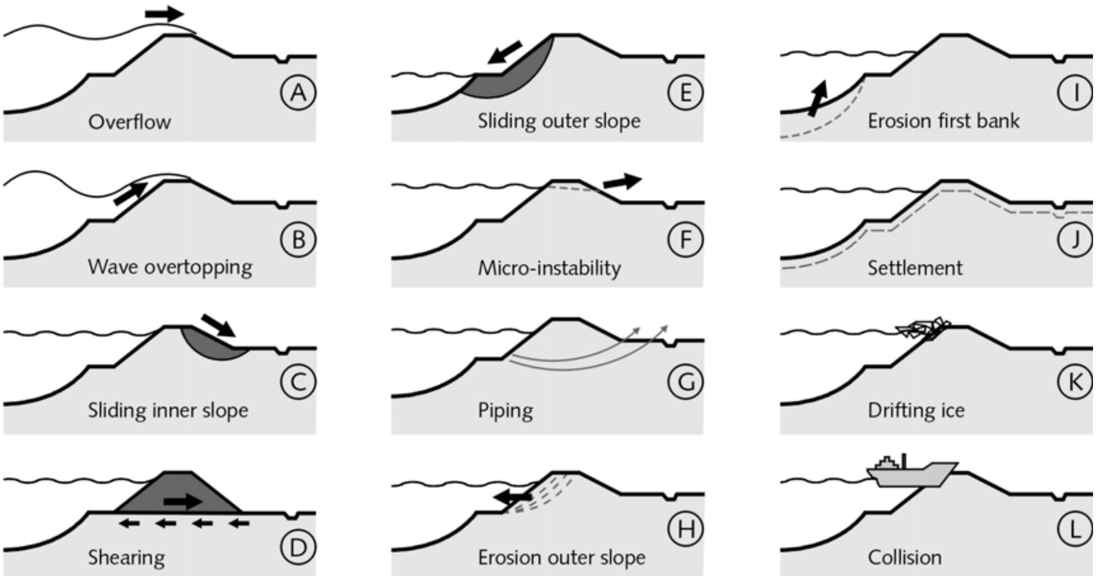


Figure 6. List of most important failure modes of dikes (TAW, 1998)

A similar list of failure mechanisms can be made for dunes and hydraulic structures, where other failure mechanisms should be added to the list, for instance structural failure of sluice doors or the failure to close movable elements.

### 3.2 *Relative contribution of the failure mechanisms*

The relative contribution of the different failure mechanism to the probability of system failure depends on different factors:

- The nature of the load: River dikes are generally more vulnerable for overflow, whereas sea dike are more vulnerable for overtopping. Piping and stability are time-dependent mechanisms that are more susceptible for long lasting high waters (on rivers).
- The local geology: Areas with a high occurrence of sand layers are more vulnerable for piping than areas consisting of mainly clays. Weak top layers increase the probability of sliding failure.
- The safety level: System designed for events with high safety standards tend to involve high crest levels. While the probability of overtopping may be low, the large potential head difference increases the vulnerability with respect to geotechnical mechanisms like piping.

In terms of observed failures, overtopping used to be dominant in the Netherlands in the past, together with ice-dams. Nowadays, strength related mechanisms are getting more attention and flood defense are explicitly assessed for these mechanisms. Besides, the warming of river due to excess heat from factories and power plants has minimized the risk of ice-dams.

### 3.3 *Experiences from New Orleans*

Hurricane Katrina caused one of the most catastrophic floods in recent history destroying large parts of the Mexican Gulf and New Orleans in August 2005. Many valuable lessons can be learned from the event regarding flood defenses. The large amount of breaches exhibited most of the well-known failure mechanism. For a more elaborate description of the breaches it is referred to Kanning et. al (2007). An overview of the breach locations is shown in Figure 7. Generally, the breaches can be distinguished in three groups. The first group (I) is on the east side of the city where the load on the system was much higher than the design resistance. The second group (II) of failure is around the navigation channels where overtopped flood walls failed. The third group of failure (III) occurred around dewatering channels where geotechnical failure caused the centre part of the city to flood. A few interesting breaches are discussed below.

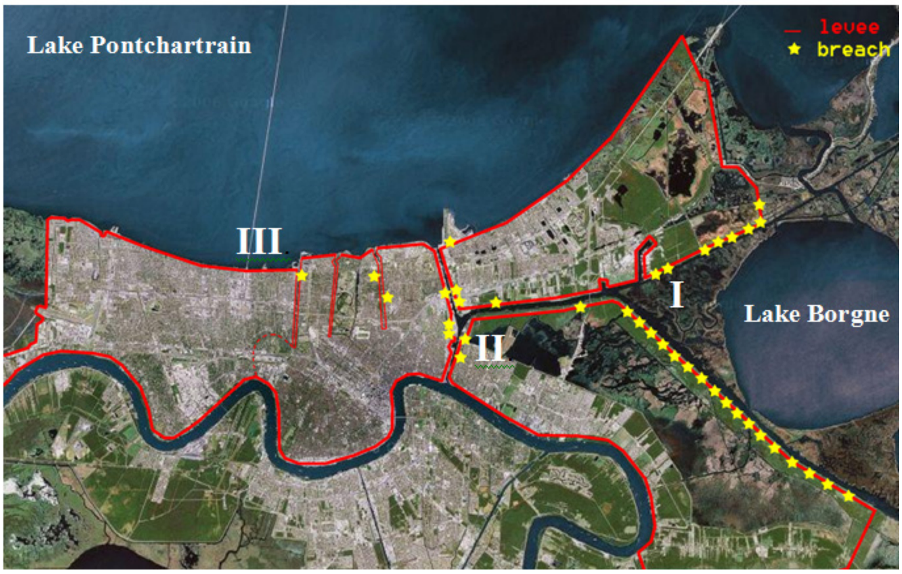

Figure 7. Overview failures in New Orleans

Figure 8 shows the failure of an earthen dike due to overtopping and overflow (area I). The water level in the whole area was much higher than the dikes. The unprotected dike eroded away for over many kilometers. Figure 9 show the geotechnical failure of a levee due to sliding in area III. The water level was below the crest of the floodwall and below design conditions. The subsoil slid horizontally over a weak layer, causing a large breach. Figure 10 shows the failure due to piping of a flood wall, again below the crest and below design conditions. For more information is referred to Kanning et. al (2008). Both failures (both in area III) emphasize the importance of geotechnical sound designs.

Figure 11 shows the failure at transition between a wall and an earthen dike in area II. Both adjacent dike and floodwall survived, only the transition failed. The vulnerability of transition could be observed all over New Orleans. Even small objects as staircases caused increased erosion of the dikes.


Figure 8. Overtopped levee in New Orleans (source: ILIT, 2006)

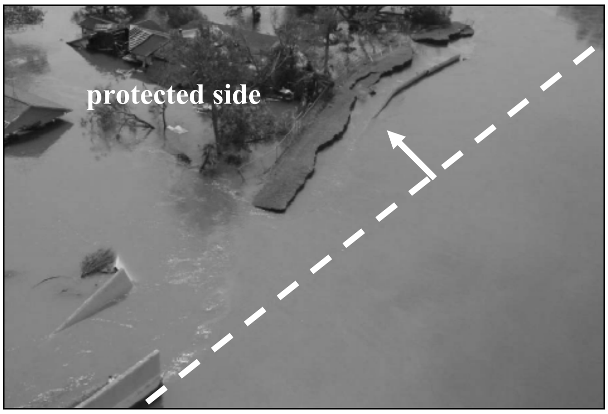

Figure 9. Stability failure in New Orleans due to hurricane Katrina (modified after ILIT, 2006)

Figure 10. Piping failure in New Orleans

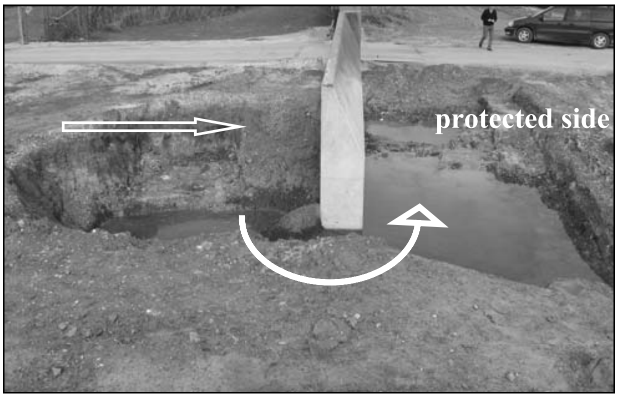


Figure 11. Failure between gate structure and dike

Perhaps more important than the individual failures was the system behavior. Or as stated by IPET (2006): "The System did not perform as a system: the hurricane protection in New Orleans and Southeast Louisiana was a system in name only." For many different reasons (e.g. funding structures, lack of funding etc.) the flood defense system could be regarded as a patchwork of defenses without clear coherence. Examples are missing levee parts, many different levee heights and abrupt changes in heights, the use of different reference datum. All these elements contributed to the total system performance. An interesting additional observation on system level is that most infrastructures (roads, pump pipes) penetrate the defense system (with gates to maintain the flood defense function). In contrast, in the Netherlands for example, infrastructure goes over the dike to reduce the amount of potential vulnerable spots.

### 3.4 *Summary*

System reliability considerations as well as observations during flood events like Katrina show that dike safety is much more than avoiding overtopping. Other failure mechanisms need to be considered, too. Geotechnical failure mechanisms play a crucial role and can dominate the probability of failure due to the large uncertainties associated with ground conditions. Hence, they need to be properly addressed in design and assessment rules of flood defenses.

## 4 SPATIAL VARIABILITY AND LENGTH EFFECT

### 4.1 *What is the Length-Effect?*

Section 2 on acceptable risk has provided a framework to derive an acceptable probability of system failure – the target reliability for the system as a whole. Section 3 has discussed the different mechanisms contributing to the probability of system failure. However, the different failure mechanisms are usually assessed at so-called representative cross sections, dike profiles of zero length. As mentioned, the so-called *length-effect* should not be neglected in deriving safety targets for dike cross sections. It is defined as the increase of the probability of failure with the increasing length of a dike reach. The two main factors determining the magnitude of the length effect are:

- The relative contribution of load and resistance: A high contribution of the resistance to the total variance increases the length effect. This is because for flood defenses, load parameters (e.g., river water level) tend to have much larger scales of fluctuation than resistance parameters (e.g. soil properties) do.
- The spatial variability in the subsoil: the higher the spatial variability in the subsoil (e.g., shorter auto-correlation distances of ground properties), the higher the length effect.

### 4.2 *Load vs. Resistance Uncertainty*

Usually the probability of failure of a flood defense system is determined by evaluating the following limit station function:

$$Z_i = R_i - S_i \qquad (10)$$

Where $R_i$ is the resistance vector consisting of all relevant dike sections and failure mechanisms contained in index $i$ and $S_i$ is the load vector. Loads usually exhibit large correlation distances (e.g. water levels in rivers). For load-dominated failure mechanisms (e.g. overflow), the probability of failure of a dike reach is close to the probability of failure for a cross section. On the other hand, resistance-dominated mechanisms exhibit significant length-effect, up to a ratio of 100 between the probability for a dike reach and the probability for a cross section. The breaches New Orleans (see section 2) underpin that large variability in ground conditions and resistance properties make the existence of weak spots likely.

### 4.3 *Heterogeneity in Ground Conditions*

The high resistance uncertainty of flood defenses is mainly caused by the high uncertainty in the subsoil caused by a high spatial variability (heterogeneity) in the subsoil combined with the limited availability of direct measurements. The spatial variability (of heterogeneity) can be subdivided into two classes (see Figure 12):

1. Continuous variability which is associated with continuous fluctuation of properties like layer thickness, hydraulic conductivity or shear strength.
2. Discrete elements like old river beds that are filled with less resistant or highly permeable materials. When undetected, these “anomalies” can represent weak spots.

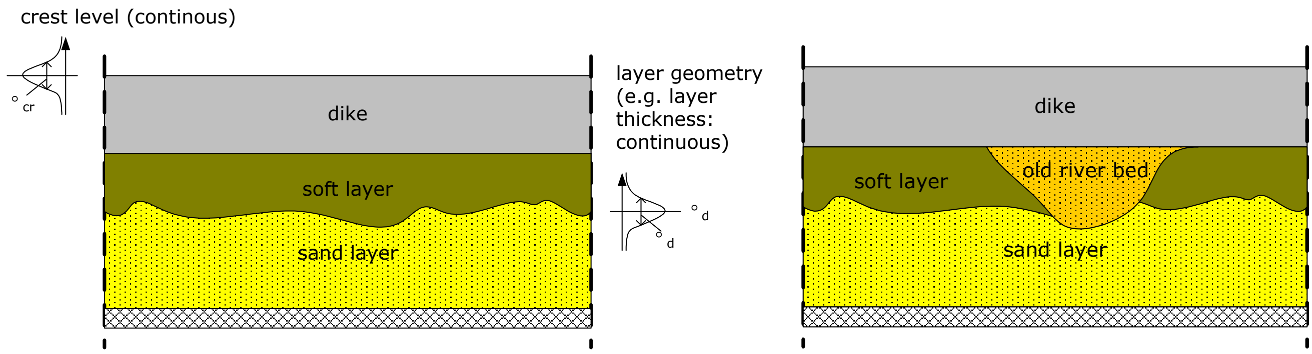


Figure 12. Continuous and discrete variability

### 4.4 *Modeling Heterogeneity*

*Continuous variability* can be modeled using random field theory (see e.g. Vanmarcke, 1977). Using this theory, the soil properties are modeled with mean, variance and autocorrelation function. The autocorrelation function describes how the correlation of a property between different locations decays with increasing lag (distance between two points). Vrouwenvelder (2006) uses the following auto-correlation function:

$$\rho(\Delta x) = \rho_x + (1-\rho_x)\cdot e^{-\frac{\Delta x^2}{d^2}} \qquad (11)$$

where $\rho(\Delta x)$ is the correlation between two point separated with distance $\Delta x$, $\rho_x$ is the lag-independent correlation and $d$ is the correlation distance of a parameter (see e.g. Vanmarcke, 1977)

*Discrete variability* is usually modeled using scenarios, see (Schweckendiek & Calle, 2010). Regional geological knowledge and experience can be used to determine (prior) probabilities of weak spots.

### 4.5 *Mathematical Treatment of the Length-Effect*

Vanmarcke (1977) and Vrouwenvelder (2006) use the outcrossing approach to determine the probability of exceedance of a threshold (here: the limit state Z=0) of length L using the mathematical properties of the autocorrelation function. Assuming full spatial correlation of the loads, for a *single* resistance variable *R,* this yields:

$$P_f = \Phi(-\beta_{\text{section}})\left(1+\alpha_R \frac{\beta_{\text{section}} L}{\sqrt{\pi} d}\right) \qquad (12)$$

Where $P_{f,system}$ is the probability of system failure, $\beta_{section}$ is the reliability index of a cross section, $\alpha_R$ is the importance factor of the resistance $R$, $L$ is the considered length and $d$ is again the scale of fluctuation Note that a low $\alpha_R$ corresponds to a low length effect, as mentioned in section 4.2

For a *multi-dimensional* problem (several resistance and/or load parameters), the length effect can be incorporated by using the equivalent mechanism length $l_{eq}$ (see Calle, 2010):

$$P_{f,system} = P_{f,\text{section}}\left(1+\frac{L}{l_{eq}}\right) \qquad (13)$$

$$l_{eq} \approx \frac{\sqrt{2\pi}}{\beta_{\text{section}}} \frac{1}{\sqrt{-\rho''(0)}} \qquad (14)$$

$$\rho''(0) = \sum_{i=1}^{N} \frac{2\alpha_i^2 (1-\rho_{x,i})}{d_i^2} \qquad (15)$$

Where the subscript $i$ refers to the different basic random variables. Finally the length effect factor for a mechanism, $n_{mech}$ is given by:

$$n_{mech} = \frac{P_{f,system}}{P_{f,\text{section}}} \qquad (16)$$

The influence of the probability of weak spots (discrete elements) can be incorporated by using conditional probability (Schweckendiek & Calle, 2010):

$$p_f = \sum p_{weak\,spot} \cdot p_{failure|weak\,spot} + \sum p_{f,\text{section i}} \qquad (17)$$

The different failure probabilities of different dike reaches cannot just be summed as they are correlated. For more information about combining correlated dike sections is referred to Vrouwenvelder (2006). It must be noted that these theories are an extension of a two dimensional analysis into the third dimension. Efforts are being done to extend these theories towards full heterogeneous and three dimensional models (e.g. Fenton & Griffiths, 2003 and Hicks, 2005).

## 4.6 *Example: Piping*

In this example we consider the failure mechanism piping. In Figure 13 the length-effect factor $n_{mech}$ is plotted over the length $L$ of the dike part of the flood defense system. Different combinations of the importance factor $\alpha_R$ and scale of fluctuation ($d$) are used. The case with $\alpha_R = 0.8$ and $d = 200m$ is representative for piping in typical Dutch ground conditions, see Lopez de la Cruz et. al (2010). It shows the length-effect factor can be larger than 100 in extreme cases. The other combinations of $\alpha_R$ and $d$ illustrate the sensitivity of the length effect.

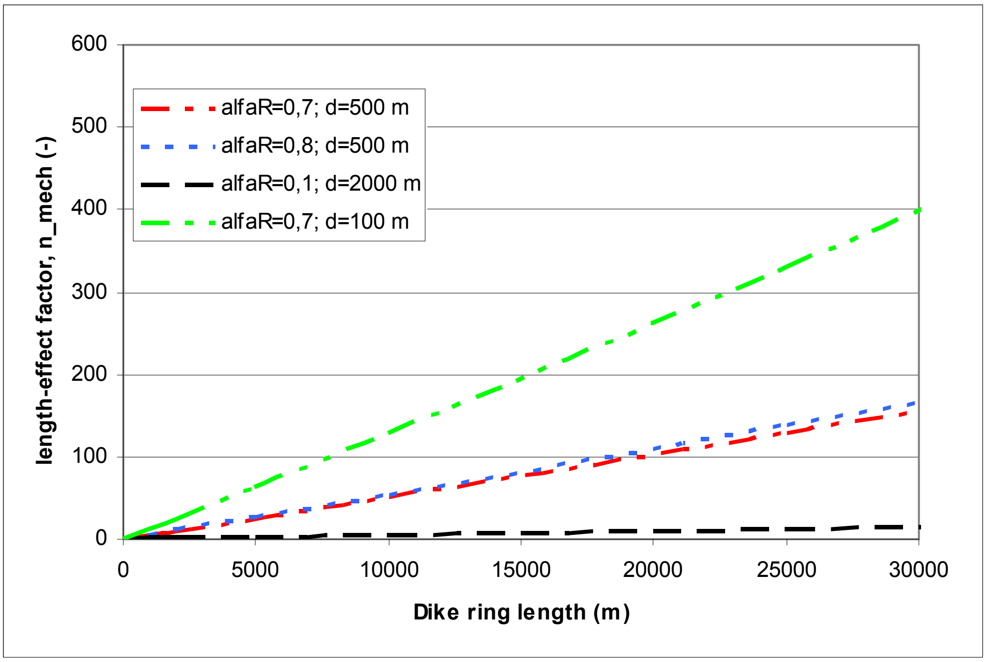


Figure 13. Length-effect factor of piping ring for different examples

## 5 FROM TARGET RELIABILITY TO PARTIAL SAFETY FACTORS

So far, we treated acceptable risk and made and inventory of what needs to be accounted for in terms of failure mechanisms and length-effect. This section shows how partial safety factors in design and assessment rules can be deduced from high-level requirements like acceptable probabilities of failure on system level.

### 5.1 *System Definition*

In section 2, the issue of acceptable flood risk was discussed without specifying the geographic extent or system that should be contemplated, when analyzing the likelihood and consequences of flooding. Purely in a theoretical sense, one would choose an independent system, for example a whole river basin, where there is no interaction with flood risk measures outside the chosen boundaries (Schweckendiek et. al, 2008). From a practical point of view, one rather works on a smaller scale. Hydraulic structures and dike sections with a similar flood pattern in case of failure and, therefore, similar consequences can be grouped and defined as the system to work with. N.B. legal aspects frequently impose boundary conditions, too, for example where flood defenses cross national or state boundaries. Regarding the acceptance criteria, the economical optimization only involves the risk contribution of a chosen (sub)system, while the criteria for social acceptability still need to be applied on the scale they were derived for. That means that for the location-specific individual risk one needs to consider the contributions of all sub-systems to the probability of dying in a certain spot. For group risk usually even all contributions on national or state scale need to be considered.

Figure 14. Fictitious Dike Ring in The Netherlands

For sake of illustration, in the remainder of this section, we will consider a fictitious example from the Netherlands. A so-called dike ring (polder surrounded by flood defenses and/or high grounds) exhibits very similar consequences regardless of the location of the failing dike or hydraulic structure. In fact, this is rather realistic for low-lying delta areas.

## 5.2 *Sub-System Requirements*

This section deals with the requirements in terms of reliability; their derivation from risk is out of the scope of this paper. The highest level requirement is the acceptable probability of failure of the (sub )system: $P_{f,adm,sys}$ .

## 5.3 *Dikes and Hydraulic Structures*

The first step is to distribute the acceptable probability of failure over the structures in the dike ring:

$$P_{f,adm,sys} = P_{f,adm,dike} + \Sigma P_{f,adm,other,i} \qquad (18)$$

where $P_{f,adm,dike}$ is the acceptable probability of failure of any dike section in the system and $P_{f,adm,other,i}$ is the acceptable probability of failure of any other structure in the system. Since the geotechnical aspects are of most interest here, the remainder of this section is restricted to the dikes.
Note that implicitly the assumption was made that the probabilities of failure of the elements are independent. In reality, there is often a positive correlation mainly due to the rather large spatial correlation of the loads. For example, long dike reaches in riverine areas experience very similar loadings, in coastal areas similar wave conditions. This assumption is conservative, which seems reasonable in this standardization procedure.

## 5.4 *Failure Mechanisms*

The second step is to establish acceptable probabilities of failure per mechanism $P_{f,adm,dike,mech,i}$ :

$$P_{f,adm,dike} = \Sigma P_{f,adm,dike,mech,i} = P_{f,adm,dike,over} + P_{f,adm,dike,inst} + P_{f,adm,dike,pip} + \ldots \qquad (19)$$

where $P_{f,adm,dike,over}$ is the acceptable probability of failure due to overtopping, $P_{f,adm,dike,inst}$ due to instability of the inner slope and $P_{f,adm,dike,pip}$ due to piping etc..

In principle, the $P_{f,adm,dike,mech,i}$ can be chosen in an economically optimal way. Mechanisms, for which high reliability target are inexpensive to realize should "occupy" less of the acceptable probability than mechanisms requiring relatively costly safety measures. In the Netherlands, such (historically rather qualitative) considerations suggest a (target) distribution as summarized in Table 2.

Table 2. Distribution of $P_{f,adm,dike}$ over mechanisms

| Failure mechanism | $\delta = P_{f,adm,dike,mech,i} \ / \ P_{f,adm,dike}$ |
|---|---|
| Overtopping | 90% |
| Piping | 3% |
| Instability | 3% |
| Other | ~1% |

Note that in the considerations regarding efficient distributions also interaction between design parameters for different failure mechanisms plays a role. For example, a dike that needs to be high for overtopping and that has a gentle slopes for slope stability automatically has quite some seepage length, which is the main resistance parameter for piping. Also, in deriving new standards, the current conditions of the flood defense system under consideration (i.e., starting point) have an influence on what is optimal.

## 5.5 *Dike Sections (Accounting for the Length-Effect)*

$P_{f,adm,dike,mech,i}$ is the acceptable probability of failure per mechanism for all the dike sections in the (sub)system. However, designs and safety assessments are usually made for dike sections a few hundred meters or several kilometers long, with rather homogeneous properties (both, loading and resistance). As discussed in section 4, the length-effect plays an important role in this step. The essence is that the longer the dike with respect to the scale of fluctuation, most importantly of the resistance, the higher the probability of failure. For deriving requirements this implies that the acceptable probability of failure per mechanism for a dike section (or a cross section that is representative for a dike section) $P_{f,adm,ds,mech,i}$ needs to be smaller than for all dikes in the considered reach:

$$P_{f,adm,ds,mech,i} = P_{f,adm,dike,mech} / n_{mech} \qquad (20)$$

The length-effect factor $n_{mech}$ is typically about 2 to 10 for load-dominated mechanisms like overtopping, and can be in the order of magnitude of 100 for resistance dominated mechanisms like piping.

## 5.6 *Partial Safety Factors*

$P_{f,adm,ds,mech,i}$ is the (low-level) target reliability $\beta_{req}$, to which design and safety assessment rules apply and for which partial safety factors are derived.

$$\beta_{req,mech} = -\Phi\ (P_{f,adm,ds,mech,i}) \qquad (21)$$

A common approach is to use level-I reliability theory with standardized importance factors (Table 3). For a lognormal-distributed resistance variable, the partial safety factor is determined by:

$$\gamma_R = \exp\left(-\left(1.65 - \alpha_R \beta\right)\sqrt{\ln\left(1 + V_R^2\right)}\right) \qquad (22)$$

where $V_R = \sigma_R / \mu_R$ , $\beta$ is the target reliability index and $\alpha_R$ is the importance factor of the resistance.

Table 3. Standardized Importance Factors for LRFD

| Parameter | $\alpha$ |
|---|---|
| dominant load parameter | 0.80 |
| other load parameters | 0.28 |
| dominant strength parameter | 0.70 |
| other strength parameters | 0.32 |

Figure 15 shows the dependence of the partial resistance factor of the target reliability index, the importance factor and the uncertainty in the (overall) resistance (here expressed in terms of the coefficient of variation $V_R$) with typical values for a resistance dominated failure mechanism like piping.

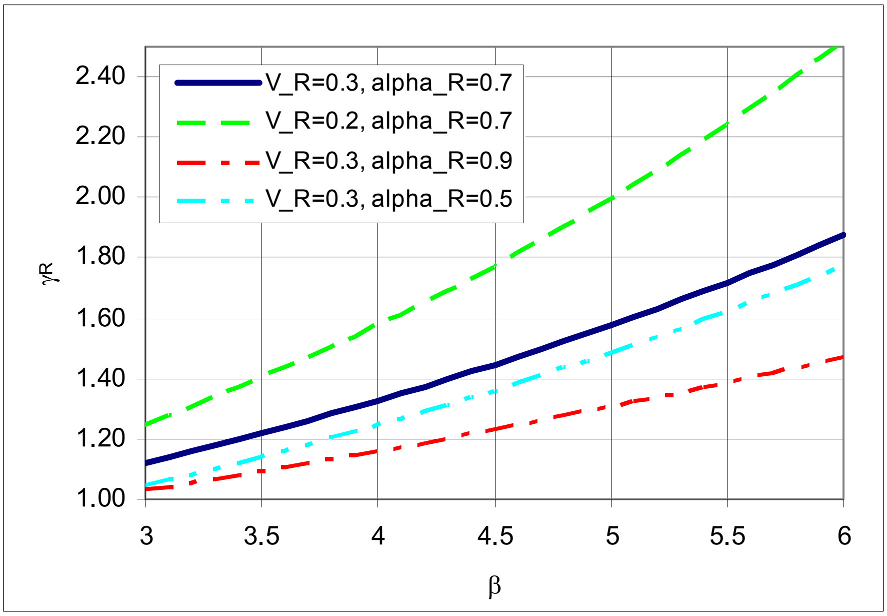


Figure 15. Relation of Partial resistance Factor and Target Reliability Index

Recently, Lopez de la Cruz et. al (2011) reported on a code calibration exercise for the failure mechanism piping in the Netherlands where a slightly different, more detailed approach was adopted. Instead of using standardized importance factors, the authors analyzed the performance for different values of the partial resistance factor in terms of resulting reliability indices. Furthermore, they established a reliability-dependent partial safety factor (see Figure 16) that can be used flexibly depending on the target reliability in a given area or (sub-system).

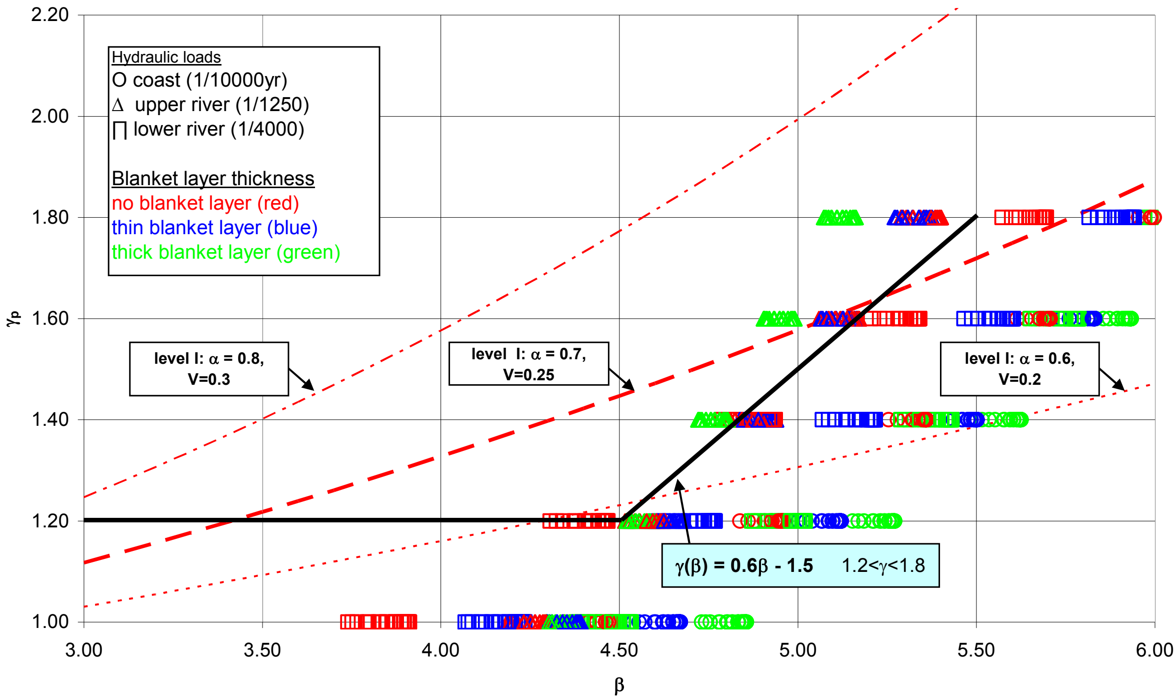


Figure 16. Partial Resistance Factor $\gamma_p$ vs. Reliability Index β for Piping according to Lopez de la Cruz et. al (2011)

## 5.7 *Overview / Example*

Figure 17 gives an overview of the steps described above for the different mechanisms, including a realistic numerical example. Note that, so far, only the treatment of resistance uncertainties has been treated. For the definition of design loads often an exceedance probability of the load combination is defined by:

$$F = P(S>S_D) = \Phi(1- \Phi(-\alpha_S \beta_{req})) \qquad (23)$$

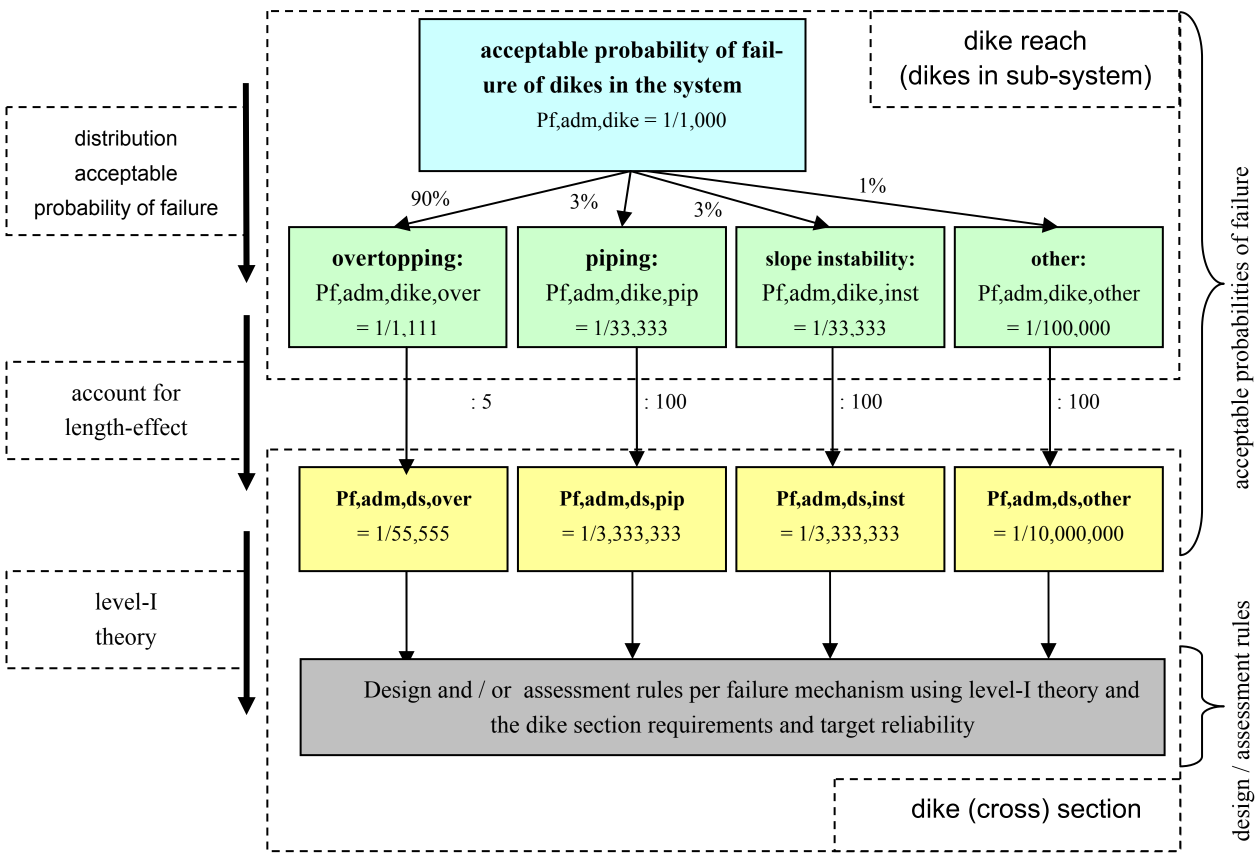


Figure 17. Overview of the Steps from high-level to low-level Requirements with a numerical example

Note that the difference between target the acceptable probability of failure on system and on mechanism level for a cross section can easily be a factor 1000 depending on the way it is dealt with different mechanisms and length effects.

### 5.8 *Alternative approaches*

The approach described so far is based on pragmatic choices. It is emphasized that there are numerous alternatives to the approach and also within the described framework. On example is to change the order of the distribution of the admissible probability of failure: go from system level to structures and dike sections first, then to the mechanisms. There are advantages and disadvantages that have mainly to do with the envisaged use of the rules in practice.

Furthermore, there are many possibilities for optimization. For example, correlations between mechanisms or structures can be taken into account in deriving the specific requirements from the high-level requirements.

Another obvious opportunity for improvement with large impact potential is the application of reliability analysis or reliability-based design (RBD), which can reduce the implicit conservatism in level-I approaches by better accounting for the uncertainties in specific conditions.

## 6 DISCUSSION

This paper attempts to demonstrate how tailor-made safety standards for large scale flood defense systems can be derived in a risk-based fashion. Since flood defenses differ from smaller scale geotechnical structures in many aspects and given the volume of investments in such large-scale engineering systems, it is very attractive to deviate from the standard design codes. That is not deviating conceptually, but rather deriving safety factors for the specific application to better account for the characteristics and uncertainties involved. The authors strive to show that safety levels and partial safety factors in the presented approach are far from arbitrary. They are part of an overall consistent flood risk framework, a framework that provides a link between geotechnical engineers and other disciplines involved in providing safety from flooding.

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