Adaptation with small steps or a big step? A probabilistic approach of flood risk reduction in the Dutch Delta

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ABSTRACT: In view of expected socio-economic development of the Dutch Delta it has recently been decided to update the safety standards, in terms of a maximum allowable loss of life probability and more stringent acceptable flooding probabilities. In order to assess safety, a probabilistic approach is followed, where the uncertainties in hydraulic loads and strength of flood defences is described by probability distributions. Also, dependencies between various stochastic variables are taken into account. The approach will be described in the paper and applied to the Rotterdam area (South West Delta) in The Netherlands. Here, two divergent strategies are identified. The first strategy is to follow an adaptive approach with small temporal steps, with many strengthened inland flood defences as the main core. The second strategy is to continue with shortening the coastline by building a dam and pumping stations in the harbor of Rotterdam. The safety consequences of both strategies will be discussed.

1 INTRODUCTION

Large parts of The Netherlands are exposed to the threats of flooding due to the influence of high water coming from the North Sea region, due to high discharge of rivers like the Meuse and Rhine or due to a combination of North Sea water levels and river discharges. In order to cope these threats, The Netherlands is protected by flood defences. However, in 1953 a large flood occurred, where more than 1800 people lost their lives. As a reaction to this flood disaster, the Delta Works were constructed in the past decades. The main philosophy of the Delta Works was to shorten the coast line by building large dams and storm surge barriers along the south west delta.

From 2017, new safety standards become effective, stating the maximum allowable flood risk for the entire Netherlands. The decision context of the paper is how to implement the new safety standards. In the neighbourhood of Rotterdam the acceptable flooding probability is $10^{-4}$ per year. Every twelve years, flood defences are assessed to check whether they fulfill the requirements.

2 TWO STRATEGIES TO SAFEGUARD URBANIZED DELTAS

The Dutch government works with a strategy that describes the measures that will be undertaken the coming years in order to control the flood threats in an efficient manner, see e.g. (Deltaprogramma 2014). In the current situation, several river mouths are closed permanently by dams or storm surge barriers. This holds for the Haringvliet, Lake Grevelingen and Eastern Scheldt, as shown in Figure 1. The Western Scheldt is entirely open, providing free navigation to the port of Antwerpen, and the New Waterway is also open, but can be closed with the Maeslant barrier in case of storm surges from the North Sea. The Maeslant barrier is a closable storm surge barrier and is only closed when extreme high water levels are expected in the cities Rotterdam or Dordrecht. As the Maeslant barrier is only closed for extreme events, ships are able to navigate to the Port of Rotterdam and the hinterland freely most of the time.

2.1 Current strategy: ‘DP’ - An open system with closable barrier

The current strategy by the Dutch government, the strategy that is elaborated in the Delta Programme and further referred to as strategy ‘DP’, accounts for flood safety of the Delta by means of the functioning of the Maeslant barrier in combination with an extensive dike reinforcement program and room for the river measures, see also Figure 1. With a fully functioning Maeslant barrier, i.e. failure probability of 0, the barrier is capable of reducing hydraulic loads in
the system significantly. For the location of Rotterdam, the design water level of 4.0m + NAP, (Nieuw Amsterdams Peil or Amsterdam Ordnance Datum), is occurring with a return period of about 50,000 years (Zhong et al. 2012), while it would occur with a return period of 600 years without a Maeslant barrier. Due to climate change the return periods will reduce in the future. Besides, the Maeslant Barrier has a current failure probability of 1/100 per closure (Kallen et al. 2012), reducing the calculated return periods. This failure probability is not negligible and have to be taken into account when computing hydraulic load levels and design water levels in the system.

With respect to the functioning of the Maeslant barrier, an additional research will be undertaken in order to improve its failure probability from 1/100 to 1/200 per closure, taking partial functioning into account. On the long term, the Maeslant barrier will be replaced at its technical and economic life time after 2070. It is assumed that the new structure will have a failure probability of at least 1/1000 per closure (Vos 2014).

2.2 Alternative strategy: ‘Sluices’ - A permanent closed barrier

Opposed to the preferred strategy of the Delta Programme, an alternative strategy is proposed by six reputed Dutch engineers, led by Frank Spaargaren, a retired engineer who has been in charge of the construction works of the Eastern Scheldt barrier. According to the engineers, the ‘DP’ strategy is insufficient to cope the threats on climate change in combination with the waterway function for the Port of Rotterdam. The engineers have the hypothesis that a cheaper strategy will be realized when the Maeslant barrier is removed and replaced by a closed dam, with locks, sluices and pumping stations, further referred to as alternative ‘Sluices’. The dam prevents storm surges of the North Sea to penetrate into the system, whereas the sluices and pumping stations are implemented to discharge water from the rivers Lek, Waal and Maas. The second aspect of the alternative strategy is that the Eastern Scheldt is used as a retention basin by increasing the flow profile from the Maas to the Eastern Scheldt. This is done by realizing an open connection in the Volkerakdam, the construction of discharge sluices at the Philipsdam and reducing the leakage area of the Eastern Scheldt barrier. This strategy reduces the design water levels upstream of the complexes significantly. With the reduction in design water level, the engineers want to reduce necessary dike reinforcements, leading to a cheaper strategy overall. The complexes are located nearby the Beneluxtunnel on the Nieuwe Maas and nearby Het Scheur on the Oude Maas, which is more inland than the current location of the Maeslant barrier in order to let sea-going vessels moor the Botlek area without the intervention of locks, see also Figure 1.

2.3 Impact of strategies

The strategies have, amongst others, impact on hydraulics and strength of flood defences, investments in infrastructural measures, navigation, fresh water, development of old harbour area, water quality and ecology. For an indication on impact for these aspects, see Van Waveren et al. (2015).

This paper focuses on the aspects related to flood risk, specifically to the hydraulic loads and the strength of flood defences. Risk is defined as follow: “Risk is a function of the probabilities and consequences of a set of undesired events.” (Jonkman 2007). For the current ‘DP’ strategy - with an open situation and a closable barrier - a vast risk assessment is undertaken in which the current flood risks are determined (Vergouwe 2014). Insights of the risk assessment are used to determine the safety standards for dike segments. Dike segments have a length of 20-30km and are part of a dike ring, which is defined as an area enclosed by a system of flood protections or high ground to protect the area from flooding (Vermeer en Waterstaat 2008). For the determination of the safety standards, the following aspects are taken into account:

1. Individual Risk (IR) is related with the loss of life probability as consequence of a flood. Everywhere in The Netherlands, the maximum probability that causalities may occur as consequence of a flood disaster is $10^{-5}$ per person per year.
2. Economic Risk (ER) is the risk related with annual expected value of economic losses. In a cost-benefit analysis the economic optimum for flood defences is found.
3. Societal Risk (SR) considers the number of deaths as consequence of a flood disaster. It is based on the perspective of people on risks; an extreme event with many fatalities has a higher impact than many smaller events with few fatalities.

With above aspects, safety standards ($P_{max}$) for all dike segments in The Netherlands are derived. For the locations that are assessed in this paper, the safety standard of the governing segment is $10^{-4}$.

The failure probability of a flood defence is a function of hydraulic loads acting at the flood the defence and strength of the defence and is described by (CUR190 1997):

$$ P_f = P(Z \leq 0) = P(R \leq S) = \int \int f_{R,S}(R,S) dR dS $$

(1)

where $Z = R - S$, $P_f =$ failure probability, $Z =$ limit state function, $R =$ strength parameter, $S =$ load parameter and $f_{R,S} =$ joint probability density function of $R$ and $S$. 
3.1 Computation of design water levels and hydraulic loads

A large part of the Hydraulic Boundary Conditions are determined with the Hydra-Zoet model (Geerse 2011). This is a probabilistic model, implemented in a computer program. It is used to assess the primary flood defences along the major water systems in The Netherlands. Instead of considering a single safety level (norm frequency), the model determines the water levels and hydraulic load levels for a whole range of exceedance frequencies, yielding so-called frequency lines for water levels and hydraulic load levels. The model is been developed by Rijkswaterstaat Waterdienst (the executive arm of the Ministry of Infrastructure and Environment).

Evaluating the current situation can be done fairly easily with Hydra-Zoet. For the rivers, lakes and the sea, the model uses all kinds of statistical information, and a large number of calculated water levels and wave variables, calculated with physically-based models such as the hydraulic model SOBEK and the Bretschneider formula. When taking into account new enclosure dams or changes in the river bed, changes of the input of Hydra-Zoet are necessary, leading to different databases of water levels and waves. Considering alternative climate scenarios, changes to the statistical input is necessary.

In the part of the lower reaches of the Rhine and Meuse where storms on the North Sea have a significant effect on the water, the random variables of the model are:
- Sea level at Maasmond
- Discharge of the Rhine at Lobith (for locations along the Rhine or its branches)
- Discharge of the Meuse at Lith (for locations
• Wind speed
• Wind direction
• Barrier state of the Maeslant storm surge barrier
• Prediction of the water level at Maasmond

Recurrence levels, also called quantiles, for these random variables are found in Geerse (2011).

In case of storm surges the Maeslant Barrier closes off the area from the sea. In the operation of the barrier predicted water levels at Maasmond are used. These predictions contain uncertainties, which is why they affect the effectiveness of the closure procedure of the barrier, and why they have to be included in the model. The barrier might fail to close for two reasons: The predicted water levels might have been lower than in reality, so that the barrier has not been closed or was closed too late (wrong prediction). The barrier might also fail to close when it had to (operational failure). Besides the use of predicted water levels, the probability of failure to close is considered in the model as well.

As stated before the SOBEK model is used to generate input for Hydra-Zoet. We note that for each location where frequency lines are needed for evaluation, the water levels have to be calculated for various so-called combinations of realizations of the random variables. Actually, the SOBEK model is used to simulate several storm events in the Rhine-Meuse estuary. We will not comment on the equations and computational schemes used in SOBEK, since these are beyond the scope of this paper (more details about the model can be found in De Deugd (2002) and the references therein.

The result of a SOBEK simulation is a time series of water levels for a large number of calculation points on the SOBEK branches. Only the maximum water level from the series is used in the probabilistic calculation of Hydra-Zoet. Hydra-Zoet uses numerical integration to calculate the frequency lines for water levels and hydraulic load levels. The combinations used as input for SOBEK are found in Geerse (2011).

3.2 Calculation failure probabilities

The safety standards for dike sections are derived from the proposed safety standards for dike segments, which are enshrined in law. As the length of a dike section is smaller than the length of a segment, the safety standard for the section is generally more stringent than for an entire trajectory due to the length effect, see Kanning (2012). The length effect can best be compared by a chain; if there is a critical load acting on the chain, the chain will always break at the weakest link. A longer chain with more variation in the strength of the links will lead to a higher probability of a weak link.

Furthermore there are several failure mechanisms that could lead to the failure of a flood defence. The failure mechanisms are considered to occur independently of each other and each failure mechanism only may take a fraction (the so-called partial factor, \( \omega_{\text{mechanism}} \)) of the overall failure probability. The sum of all partial factors that contribute to the overall failure probability equal 1, in Infrastructuur en Milieu (2015a), the partial factors per mechanism are stated for the Dutch levees. The subdivision of the overall safety standard for a dike section to a safety standard per mechanism gives therefore a semi-probabilistic calculation instead of a full probabilistic calculation.

3.2.1 Overtopping

The first dominant failure mechanism in the system concerns overtopping. Overtopping occurs as a combination of high water levels with local setup of waves and wind. When there is a critical flow over the levee, the revetment of the levee at the inner side may fail, inducing a breach. As the flow accelerates, the breach increases as a consequence of this positive feedback mechanism. Due to the breach, structural failure of the levee occurs, leading to inundation of the area (CIRIA 2013). In the new design instrumentation (Infrastructuur en Milieu 2015a), maximum allowable critical flows are determined. For this study a maximum allowable critical flow of 0.005m/s is used, which is a conservative value according to the instrumentation. See Figure 2 for an overview of the mechanism (Schiereck 1998).

![Figure 2: Failure mechanism overtopping](image)

With the model Hydra-Zoet, hydraulic loads are computed on a probabilistic manner in which the orientation and the geometry of a dike are taken into account. In our approach the significant wave height, the peak period and the wave direction are calculated with the 1-d wave growth formulas of Bretschneider (Geerse 2011). Load levels with corresponding return periods are offset against the actual height of the flood defences, resulting in failure probabilities for the overtopping mechanism.

The standard specification per dike section, for overtopping, is given by (Infrastructuur en Milieu 2015a)

\[
P_{\text{dikesection, overtopping}} = \frac{P_{\text{max}} \cdot \omega_{\text{overtopping}}}{N_{\text{overtopping}}} \tag{2}
\]

where \( P_{\text{max}} \) is the safety standard for a dike segment as described in Deltaprogramma (2014) and Infrastructuur en Milieu (2015b), varying from 1/300 per year to 1/100,000 per year, \( \omega_{\text{overtopping}} \) the partial factor that is allowed for this mechanism (0.24) and \( N_{\text{overtopping}} \) the factor for the length effect (1 or 2 for the segments taken in this example).

The limit state function for overtopping is given by:

\[
Z = m_{qc}q_{c} - m_{q0}f_{0} \tag{3}
\]
where \( m_{qc} \) = model uncertainty factor for the critical overtopping, \( q_c \) = the critical average flow at which a dike collapses, set at 0.005m\(^3\)/s, \( m_{qd} \) = Model uncertainty factor for the appearing overtopping and \( q_d \) = occurring overtopping flow.

### 3.2.2 Piping

Failure of a levee due to piping occurs in case three related mechanisms to piping occur. The mechanisms related with piping are ‘uplift’, ‘heave’ and ‘piping’. Schweckendiek & Calle (2013) describe these as: ‘Excessive pressure in the aquifer causes the aquitard to lift up (Uplift), next groundwater flows towards the leak (Seepage) and the flow starts to erode granular material (Heave). A pipe starts to develop in upstream direction until a continuous pipe is formed and erosion accelerates, resulting in a structural collapse by undermining.’

In Equation 4 the safety standard for an arbitrary dike section for piping is given (Infrastructuur en Milieu 2013).

\[
P_{\text{dikesection, piping}} = \frac{P_{\text{max}} \cdot \omega_{\text{piping}}}{N_{\text{piping}}} \cdot L_{\text{dikesection}} \cdot \frac{b}{a} \tag{4}
\]

where: \( P_{\text{dikesection, piping}} \) = the derived safety standard for a dike section for piping, \( P_{\text{max}} \) = the safety standard for a dike trajectory, \( \omega_{\text{piping}} \) the partial factor that is allowed for this mechanism (0.24), \( N_{\text{piping}} \) the factor for the length effect, \( L_{\text{dikesection}} \) = the length of the dike section, \( a \) = part of length of trajectory that is sensitive to piping (0.4 in this study), \( b \) = length of independent, equivalent sections for the respective failure mechanism. \( N_{\text{piping}} \) is given by (Infrastructuur en Milieu 2015a):

\[
N_{\text{piping}} = 1 + \frac{a \cdot L_{\text{segment}}}{b} \tag{5}
\]

where: \( L_{\text{segment}} \) = length of the dike segment on which the standard specification is based on.

The computation of the failure probability takes place by the computation of fragility curves via the revised formula of Sellmeijer (Sellmeijer et al. 2011). The limit state function for piping is given by:

\[
Z_p = m_p H_c - (h - h_b - 0.3d) \tag{6}
\]

where \( m_p \) = model factor that states the uncertainty in the model that determines the critical water level [-], \( h \) = local (occuring) water level [m], \( h_b \) = water level at exit point [m], \( d \) = blanket layer thickness [m], \( H_c \) = critical difference in water level [m] which is given by:

\[
H_c = F_{\text{resistance}} \cdot F_{\text{scale}} \cdot F_{\text{geometry}} \cdot L \tag{7}
\]

\[
F_{\text{resistance}} = \eta \left( \frac{\gamma_s}{\gamma_w} - 1 \right) \tan(\theta) \tag{8}
\]

\[
F_{\text{scale}} = \frac{d_{70m}}{\sqrt{k \frac{\eta}{g} \frac{\eta}{\theta}}} \tag{9}
\]

\[
F_{\text{geometry}} = 0.91\left( \frac{D}{L} \right)^{0.28} \tag{10}
\]

where in Equations 7 to 10: \( L \) = leakage length [m], \( \eta \) = coefficient of White or drag factor [-], \( \gamma_s \) = dry volumetric weight of sand grains [kN/m\(^3\)], \( \gamma_w \) = volumetric weight water [kN/m\(^3\)], \( \theta \) = internal friction angle sand grains [\(^\circ\)], \( v \) = viscosity of water [m\(^2\)/s], \( k \) = specific conductivity of aquifer [m/s], \( g \) = gravitational constant [m/s\(^2\)], \( d_{70m} \) = 70% quantile of the grain size distribution [m], \( d_{70m} \) = mean \( d_{70m} \) in small scale laboratory tests [m] and \( D \) = thickness aquifer [m].

In some cases, the subsoil consists of two aquifers with different k-values, depending on the stratification. It is chosen to schematize the subsoil with one aquifer in this case study. The parameters regarding to the geometry of a levee are shown in Figure 3.

![Figure 3: Parameters regarding to the geometry of a levee for piping](image)

A function in which both the fragility curve as the probability density function of occurring water levels are combined, leads to the failure probability due to piping. It is given by (Van der Meer et al. 2009):

\[
P_f(Z < 0) = \int_{-\infty}^{0} P_f(Z < 0|h)f(h)dh \tag{11}
\]

where \( P_f(Z < 0|h) \) = the cumulative distribution function of the strength of the dike section given water level \( h \) (or the so-called fragility curve) and \( f(h) \) the probability density function of the water level.

### 4 CASE STUDY

#### 4.1 Area of study

The total length of category-A flood defences in the South-Western delta is over 500 km. These flood defences are defined as primary flood defences in The Netherlands that retain open water directly (Vergouwe 2014). For the case study, there are three dike sections assessed within the system; A, B and C, see Dokter (2015). Research has shown that these sections are sensitive to failure due to piping (Vergouwe et al. 2014). The sections are located within dike trajectories 16-2 and 16-3 in the so-called ‘transition-zone’, where extreme water levels are determined by both influence of high water levels on the North Sea as high river discharges. The applicable safety standard...
will be upgraded from an exceedance frequency of 1/2000 to a flood probability of 1/10,000 (Vergouwe et al. 2014, Deltaprogramma 2014). See Figure 4 for an overview. The length and height characteristics are stated in Table 1 and are modelled deterministic.

![Assessed dike sections in case study](image)

Table 1: Characteristics assessed dike sections

<table>
<thead>
<tr>
<th>Section</th>
<th>Segment</th>
<th>N</th>
<th>Length [m]</th>
<th>Height [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>16-2</td>
<td>1</td>
<td>1203</td>
<td>4.88</td>
</tr>
<tr>
<td>B</td>
<td>16-3</td>
<td>2</td>
<td>2318</td>
<td>5.92</td>
</tr>
<tr>
<td>C</td>
<td>16-2</td>
<td>1</td>
<td>1139</td>
<td>4.37</td>
</tr>
</tbody>
</table>

4.2 Overtopping

For the three dike sections, analyses have been performed to determine the hydraulic load levels that occur in extreme events. Both strategies have been evaluated for the reference years 2015 and 2100, where ‘Sluices’ 2015 is only hypothetical as this strategy is not implemented in the current situation.

Figures 5–7 show the calculated return periods for dike section A, B and C. The intersection between the water level lines and the safety standard, indicate the hydraulic load level which should be resisted by the dike sections. When the cross section is above the dike height, the safety standards are not met and dikes should be heightened. Note that the required safety standard for section B is a factor two lower than for sections A and C. This is the consequence of a different length-effect factor $N$, which is 2 for segment 16-3 and 1 for segment 16-2 (see also Table 1).

![Results for section A](image)

![Results for section B](image)

![Results for section C](image)

Sections A and C clearly benefit from the reductions in hydraulic loads and do not need to be heightened up to 2100 in strategy ‘Sluices’. The differences between the strategies is around 0.8m and ‘Sluices’ in 2100 still results in lower hydraulic load levels than ‘DP’ in 2015. The rather large differences are explained by the fact that in ‘Sluices’, high waters from the North Sea do not any longer form a threat to penetrate the system, as the system is permanently closed (opposed to strategy ‘DP’ where there is still a significant probability that the Maeslant Barrier fails to close during a storm). Secondly, the retention capacity in ‘Sluices’ is larger than in ‘DP’, because also the Eastern Scheldt is used as a retention basin. Thirdly, thanks to the pumping stations, part of the river can be discharged in the North Sea during storm conditions in the ‘Sluices’ strategy, whereas this is impossible in ‘DP’. For section B there exists only a slight difference between the strategies. This can be explained by the geographical location of section B. It is located far upstream from the location of the proposed dams, locks, sluices and pumping stations and the measures will not be effective at this distance any longer.

Above findings show that there is a clear benefit of strategy ‘Sluices’ on reducing risks of flooding due to overtopping.

4.3 Piping

The failure probabilities for piping are calculated via Equation 11, where the probability density functions
of water levels are combined with the fragility curves of the strength of the dike sections. The fragility curves are constructed by means of Monte Carlo simulation. The parameters are characterized either normal, lognormal or deterministic, see Dokter (2015).

In Figures 8-10, the probability density functions of the water levels in both strategies and reference years are displayed in combination with the corresponding fragility curve of the dike section, which is equal for each strategy and reference year, assuming that the dike sections are not reinforced in between and there is no deterioration.

In the figures, a clear shift in the probability density functions to more moderate water levels can be distinguished when strategy ‘Sluices’ is compared to ‘DP’. Note that for failure due to piping, all occurring water levels contribute to certain extend to the failure probability, whereas for failure due to overtopping, only extreme water levels play a role.

In Table 2 the failure probabilities are given for the three dike sections for both strategies and reference years. Despite the fact that strategy ‘Sluices’ leads to a significant reduction in failure probability, still the safety standard is not met for most of the dike sections. This is explained by the revised and more stringent safety standards that become effective from 2017. However, even when the section is rejected and needs to be strengthened, the amount of dike reinforcements in strategy ‘Sluices’ will be less than for strategy ‘DP’.

5 DISCUSSION

In this paper we have presented a probabilistic method to assess the safety impact of different strategies. Flood probabilities are used now as safety standards for the flood prone areas in The Netherlands. Limit state functions are available to assess the reliability of the flood defences. However, data to feed these functions is not widely available. More attention is needed for a more realistic safety assessment of these flood defences. There is a (political) debate in The Netherlands when to investigate in more depth the Sluices alternative. Is it an alternative for the far future, or does implementation of this strategy have also many advantages for the near future?

5.1 Costs of dike reinforcements

The results of this study are used for cost calculations. It is interesting to compare how the results are addressed with terms of costs and cost savings. In Labrujere and Van Waveren (2015), cost calculations for dike reinforcements have been performed for the entire South-Western Delta. The costs for dike reinforcements in the ‘DP’ strategy are estimated to be €5.2 billion against €4.5 billion in strategy ‘Sluices’, which is only a slight gain in costs compared to the realized reduction in flood risk. There are three distinctive aspects of influence:

- Firstly, in strategy ‘Sluices’ also dike reinforcements are necessary. The dam and the pumping stations do reduce the hydraulic loads, but still not all safety standards for the flood defences behind the dam are met due to the more stringent rules. In cost functions for dike reinforcements, initial costs are a major cost driver with about 70% of the costs (Dokter 2015).
- Secondly, in several areas, there is not only an increase in extreme water levels, but also land
standards. This is also the situation for the sections assessed in the case study, for which it is believed that the land subsidence could be up to 0.6m/century. This can be compared with the speed of sea level rise and thus a serious challenge as the height of dikes decrease with the same rate.

- Thirdly, the timing of measures is of influence on the decision. Strategy ‘Sluices’ becomes more attractive when the speed in sea level rise becomes rather large. In this situation, many dikes need to be heightened at an earlier stage in the ‘DP’ strategy. With respect to the timing of measures it is also interesting to mention that the interest rate for investments is of large influence on the preferred strategy. Strategy ‘Sluices’ is effective for a long timespan, but because of the interest rates these benefits on the long term have only little impact in the investment decision.

5.2 Impact on other aspects

The research in this paper is part of the evaluation of both strategies. In Labrujere and Van Waveren (2015) a full evaluation is presented. Among others, impact on the activities in the harbor, ecological impacts and new housing opportunities in the harbor area are described. As often is the case, there is not one strategy which scores best on all criteria. A major point of discussion is that DP strategy relies on a renewed Maeslant barrier, but with sea level rise the Maeslant barrier has to close once per month in the storm season so this will also be a barrier for the harbor activities.

6 CONCLUSION

The results in this paper show that the probabilistic assessment of flood defences can successfully be applied in assessing safety standards for the Dutch dikes. Two strategies to meet the new safety standards in the Dutch Delta are assessed and strategy ‘Sluices’ is a promising strategy in order to reduce flood risks behind the dams. Despite of the promising results for strategy 'Sluices', still additional dike reinforcement measures in combination with the dam and pumping stations will be necessary for several flood defences to meet the new safety standards.

In the near future, water authorities will start to use the probabilistic method in order to assess all flood defences in The Netherlands. It is however a challenge to use this method on large scale in order to assess all flood defences according the new safety standards.

REFERENCES