Uncertainty analysis of river flood management in the Netherlands

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ABSTRACT:

The current flood defense design practice along the major rivers in the Netherlands is to include only the natural variability of water levels (or the discharge) in assessing the exceedance frequency. Other sources of uncertainty which could cause flooding (such as the roughness of the riverbed or the discharge distribution at the bifurcation points) are ignored.

In this paper we will show the influence of other uncertainties on the probability of flooding. Instead of the traditional design method (the exceedance frequency of water levels, using only the river discharge as random variable), we will consider the exceedance probability of (wave) overtopping of the flood defense. We have investigated the failure frequencies of dike sections and not the flood frequency of dike rings, which always consist of a number of failure mechanisms, dike sections and hydraulic structures. Therefore the number of random variables remains small enough so that numerical integration can be used to calculate the frequencies.

It is shown that other sources are a major contribution to the calculated safety against flooding. These uncertainties also influence the efficiency of measures which reduce the risks of flooding, such as the use of retention areas (for example in emergency situations). In the traditional approach this measure seems highly efficient, but if all uncertainties are taken into account this measure is less efficient. However, the attractiveness using retention areas depends on the costs and benefits of this measure, and the approach in this paper is an essential ingredient to assess the benefits. It is recommended to use the approach in this paper in a cost benefit analysis, and to investigate the influence of the assumptions.

1 INTRODUCTION

The Netherlands are situated in the delta of three of Europe's main rivers: the Rhine, the Meuse and the Scheldt. As a result of this, the country has been able to develop into an important, densely populated nation. But living in the Netherlands is not without risks. Large parts of the Netherlands are below mean sea and water levels which may occur on the rivers Rhine and Meuse. High water levels due to storm surges on the North Sea, or due to high discharges of these rivers are a serious threat to the low-lying part of the Netherlands. Proper construction, management and maintenance of flood defences are essential to the population and further development of the country.

Without flood defences much of the Netherlands would be flooded on a regular basis. The influence of the sea would mainly be felt in the West. The influence of the waters of the major rivers has a more (but limited) geographic impact. Along the coast, protection against flooding is predominantly provided by dunes. Where the dunes are absent or too narrow, or where the sea arms have been closed off, flood defences in such as sea dikes or storm surge barriers have been constructed. Along the full length of the Rhine and along parts of the Meuse protection against flooding is provided by dikes. For an overview of the current safety standards along the coast and major rivers, see Brinkhuis-Jak et al, 2003.

2 DESIGN METHODS

The current safety standard has been set after the big 1953 flood disaster in the Netherlands. After this flood the design method of flood protection was improved considerably because of the scientific approach. This approach was invented by the Delta Committee (Delta Committee 1960, Dantzig 1956). The default approach for designing flood protection structures that has been used until then, was based on the highest recorded water level. In relation to this water level a certain safety margin (varying from 0.5 to 1.0 meter) was maintained. The Delta Committee recommended that a certain desired "safe" water level be taken as a starting point. The safety standards should be based on weighting the costs of the construction of flood protection structures against the possible damage caused by floods. An econometric analysis was undertaken by the Delta Commission for Central Holland. Based on information from 1960 this led to an optimum policy of $8 \cdot 10^{-6}$ per year. For practical design this was converted into a design water level with a frequency of exceedance of 1/10000 per year. These design frequencies are used for the dike ring areas along the coast. For the major rivers, however, less strict design frequencies are demanded in the Act of Flood Defences, because the consequences of flooding in these riverine areas are less severe than a flood along the coast. The design frequency of flood defences along the major rivers has been set to 1/1250 per year.

The predominantly *deterministic* determination assumes the normative Design Water Level (DWL) that the dike must be able to retain (TAW, 1998). This water level may include wind effects on the local water level. In addition, the wave run-up is subsequently the most important parameter in the determination of the crest height. In the traditional deterministic design method the wave run-up is calculated based on certain wind characteristics at the design water level and the corresponding waves, and taking into account the geometry of the water defence system. Settlement of the soil body over a certain planning period is also taken into account to avoid repair actions, which may be needed if the height of the dike is below the required reference level.

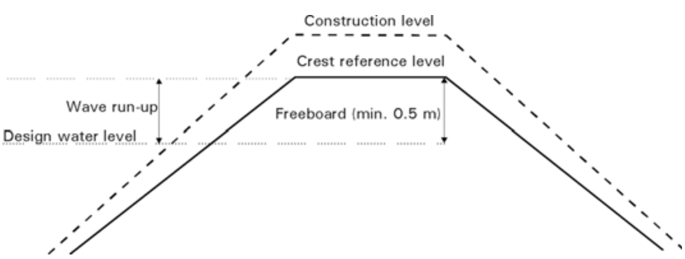

Figure 1. Design of a river dike (TAW, 1998)

In a *probabilistic* approach the results of the more deterministic method described above are still needed as input to the probabilistic calculations. In a probabilistic approach we are interested in the inundation probability of a dike ring area. Inundation is caused by failure of the flood defence system. Here, the whole range of water levels and waves are included in the analysis (TAW, 1998).

In this paper we will follow a probabilistic approach.

3 ASSUMPTIONS

The following assumptions have been made (see also Stijnen et al, 2002):

a. We investigated only one failure mechanism: overflow and wave overtopping. Other mechanisms (for example sliding of the inner slope, piping and micro instability, see TAW, 1998) are not included. These mechanisms may be important, but in the study TAW, 2001 it is concluded that overflow and wave overtopping is the dominant mechanism in the probability of flooding, assuming that the possible 'weak spots' are strengthened;
b. We investigated the following six locations along the major Rhine river branches (see for a map Figure 2):
   - Lobith, Upper Rhine river (km 862)
   - Millingen, Waal river (km 868)
   - Tiel, Waal river (km 915)
   - Opijnen, Waal river (km 929)
   - Amerongen, Lower Rhine river (km 918)
   - Duursche Waarden, IJssel river (km 961)

Figure 2. Overview of the Rhine Branches in the Netherlands

c. We did not consider the true dike heights, but the dikes as they should have been designed according to the design rules of the Technical Advisory Committee on Water Defences;
d. In the assessment of flood defences it is useful to distinguish between failure and collapse of a structure. Failure is defined as not fulfilling one or more water defence functions (the crest of a part of the flood defence is too low, for example). Collapse means the loss of cohesion or

large deformations in geometry. In this paper we only handle failure of the water defence.

e. The reliability function Z of the failure mechanism wave overtopping is: $Z = q_c - Q(H_s, h)$, where q_c stands for the critical overtopping discharge (which may be stochastic, but in this paper we will assume that it has a deterministic value of 0.001 m^3/s/m, which is equivalent to 1 l/s/m), H_s is the wave height and h is the water level.

4 UNCERTAINTY IN PARAMETERS AND THEIR DISTRIBUTIONS

When designing the height and strength of a water defense section, there are many (stochastic) factors which have to be taken into account. It is important to realize that we are not only dealing with uncertain parameters, but that each of these parameters has a distribution of its own that is unknown. Think about the natural processes such as the river discharge waves (height and shape), the resulting water levels, precipitation, wind (speed and direction), etc. On the other hand, there are also a number of uncertainties in the creation of models: a hydraulic or hydrological model is never perfect, and neither are the required parameters in these models. Finally, we often use measurement data. When we use these measurements to estimate parameters, or distributions, more uncertainties are introduced.

There are basically two categories in which we can place these uncertainties (Noortwijk et al., 2002): natural variability and epistemic uncertainties.

4.1 *Natural variability*

This is sometimes also called inherent uncertainty, and represents the unpredictability of physical processes. This concerns both uncertainties in time as well as in space. Uncertainties that are a direct consequence of the variability of natural processes fall into this category. Think about the direction or velocity of the wind, but also the local or downstream hydraulic roughness and the discharge distribution near a river bifurcation point. Uncertainties in the discharge itself (both the height of the peak and the shape) are part of this category as well.

4.2 *Epistemic uncertainty*

The category of epistemic uncertainty (also called knowledge uncertainty) is a large one, and can be further subdivided into statistical uncertainty, model uncertainty and planning uncertainty.

Uncertainties that play a role when determining the water level on the river, or the discharge into a retention area belong in the subcategory of model uncertainties. They arise from prediction models for the river and the retention area. Other examples of uncertainties that play a role within the hydraulic model, are the flow pattern near the inlet construction, and the slope across the inlet construction. These uncertainties maintain a certain amount of subjectivity (Cooke, 1991), because their size and relevance are hard to determine. It is also possible for these results to be influenced by new research results.

Statistical uncertainty arises when there are not enough data to estimate the parameters of a probability distribution of a random variable (Kok et al., 1996, Appendix B). The more data, the smaller becomes the statistical uncertainty. The uncertainties regarding the choice of the type of probability distribution also fall into this subcategory. Examples are the probability distributions for the discharge, and temporal and spatial correlations between the various random variables.

The decision to use a retention area brings with it a number of uncertainties from these different categories. There are, however, uncertainties that fall in yet another subcategory, dealing with the organisational side of a measure, especially in the case of retention. This is closely related to the ability to predict the duration of a discharge wave on a short term (in the order of days to a week). Of special interest in this case are questions that concern the actual use of a retention area. When should the retention area be flooded? Which retention area should be used? There are also social and economic aspects surrounding the decision whether to use a retention area or not, but these will not be discussed in this paper.

5 RESULTS OF DETERMINISTIC AND PROBABILISTIC CALCULATIONS

For each of the six locations mentioned in Section 3 the failure probabilities for the mechanism overflow and wave overtopping have been calculated. We also investigated what happens to these failure probabilities when retention is used as a measure to increase the safety of dike ring areas. In each case we investigated the resulting failure probability *with* and *without* the measure. This in turn enabled us to define the term “efficiency” of the measure retention as follows:

$$\text{Efficiency of retention} = \frac{\text{Failure probability without retention}}{\text{Failure probability with retention}}$$

With the aid of this definition it is possible to obtain insight in the actual safety benefit of a measure. In the computations we used the following random variables and distributions:

- The discharge, with actual exceedance probabilities of the discharge peak according to the working line (Parmet et al, 2002).
- The wind direction, with actual statistics for the measurement station of Schiphol Airport (Geerse et al, 2002).
- The wind speed, with actual exceedance probabilities for the measurement station of Schiphol Airport (Geerse et al, 2002).
- Water level, where a normal distribution is assumed. This is a result from uncertainties around the river bifurcation points, the geometry, hydraulic roughness and lateral inflow (Stijnen et al, 2002).

The results that are presented here are based on a recent study (Stijnen et al, 2002). We made the computations including the entire shape of the discharge wave. Given the peak of the discharge wave, the entire shape is assumed to be known. With the "peak" of the wave we mean in this case the highest discharge within a single wave that has a constant value for a period of 12 hours (an example wave is shown in Figure 3).

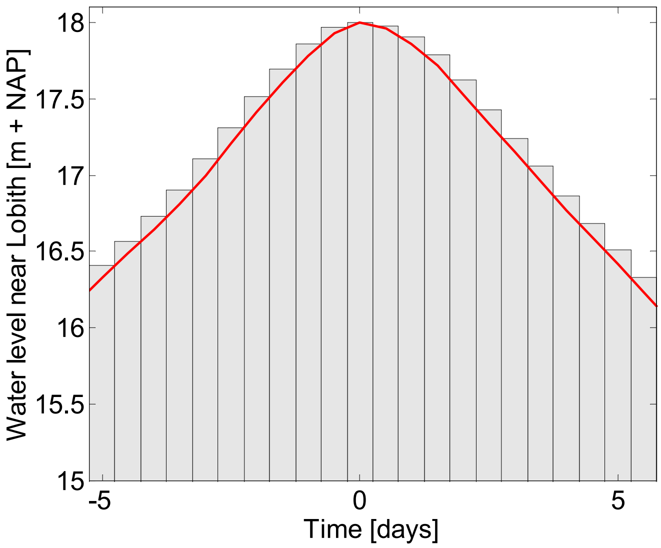

Figure 3. An example of a discretised water level wave for a discharge wave with a peak of 16000 m^3/s at Lobith, with independent blocks of 12 hours.

Because the number of random variables is relatively small, this enabled us to use numerical integration, instead of other approximation techniques.

The design discharge (the discharge with an annual probability equal to the safety standard of 1/1250) is equal to 16000 m^3/s for the Rhine river (Parmet et al, 2002). With the design discharge, the design water levels (DWLs) along the river branches are known. For every location along the Rhine branches, so-called *QH*-relations are available that couple the discharge (Q [m^3/s]) with the local water levels (H [m]). In order to obtain a consistent set of computations, the height of the dikes at the investigated locations are assumed equal to the design water level plus an additional minimum safety margin of 0.5 meter (see also Section 2).

In this paper we present the results with respect to one flood management measure: the use of retention areas in case of emergency situations. The efficiency of other measures (such as "Room for the River" and dike heightening) is studied in Stijnen et al, 2002, but are not presented in this paper. With regard to retention, we investigated a single area with a volume of 250 Mm^3, near the city of Lobith. This volume is inspired by the ideas in a recent advice of the committee Emergency Retention Areas. (Commissie Noodoverloopgebieden, 2002). The inlet construction is considered to be "ideal", meaning that no restrictions are posed on the amount of inflow, etc. The inlet sill is kept at a fixed level of 16000 m^3/s (the level it should be according to the current design practice).

In the case of retention the shape and peak of a discharge wave become important, because they determine the volume of water that needs to be withdrawn from the river. The peak of the wave when retention is used, can vary in height per location. It is also possible that the time at which the peak occurs shifts (Figure 4).

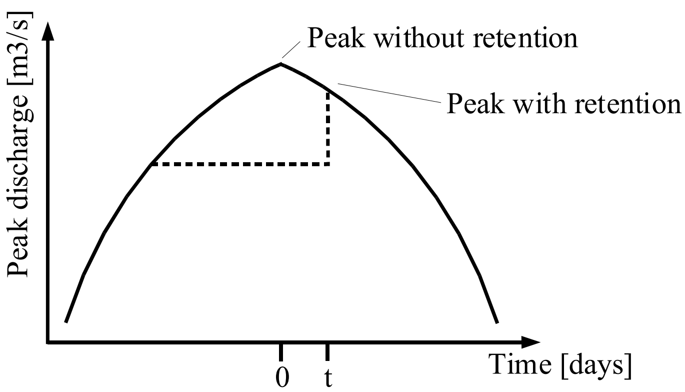

Figure 4. Illustration that shows the shift of the "peak" of a discharge wave in a situation with and without retention. The peak in the situation without retention occurs at time '0' days, while in the situation with retention the peak occurs at time 't' days.

Five sets of computations have been performed for each of the six locations. In each of these five sets another random variable has been added, in order to observe the impact that each additional ran dom variable has on the failure probabilities. In the next subsections we will more closely examine these five computations.

5.1 *Random variable: discharge (height of dike section: DWL [m+NAP])*

In this first calculation, we primarily wanted to establish a base for the rest of the computations. The results are straightforward, and can be found directly from discharge frequency function (commonly known as the working line). The annual failure probability that is found for each location is equal to the current, designated safety level of the dikes in the upper river branches region: 1 / 1250. This is indeed equal to the annual exceedance probability of the design discharge of 16000 m^3/s. In this situation, where we have only a single random variable, it is possible to select a single discharge for which the dike section fails for the first time. A discharge that is higher than the 16000 m^3/s level, will cause a dike section to fail immediately.

When we make use of the measure retention, the failure probability for each of the five locations decreases substantially, and is reduced to 1 / 4548 per year. In each case we also calculated the corresponding *critical discharge*, which is the lowest discharge that causes failure of the dike section. After retention the critical discharge is no longer equal to 16000 m^3/s but has increased to 17700 m^3/s (see also Figure 5). Even though the retention area starts to fill up, there is still a positive effect visible on the water levels downstream. The impact of retention is no longer noticeable for discharges above 18700 m^3/s, where the two working lines are equal again.

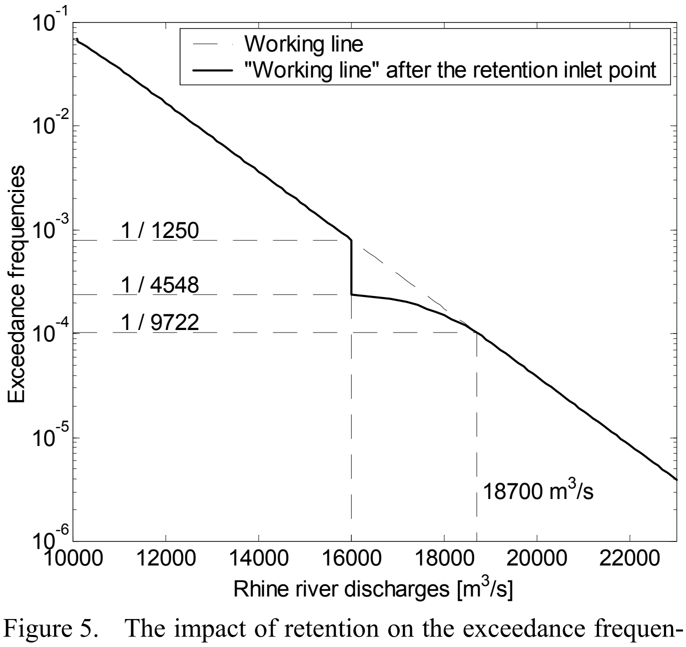

Figure 5. The impact of retention on the exceedance frequencies of the Rhine discharge downstream the inlet construction

Obviously, there is a very large, positive impact of retention for this calculation. The efficiency of the measure retention on the failure probability in this case is equal to E_{ret} = 4548 / 1250 = 3.6.

5.2 *Random variable: discharge (dike height: DWL + 0.5 [m+NAP])*

Again, we focus only on the discharge as a random variable, but this time the dike sections include an additional margin of 0.5 meter (which is needed for local wind waves). For each location, the discharge that is required to raise the water level to this additional height is different. Hence, the results are different for each location, because the relations between the discharge and the water level (the *QH*-relations) become important. The failure probabilities are collected in Table 1, and the corresponding critical discharges are shown in Table 2.

Table 1. Failure probabilities, caused by overflow of a dike section with height (DWL + 0.5 [m+NAP]).

Location	Failure probability [-]		Efficiency
	Without retention	With retention	
Lobith	1 / 3907	1 / 5712	1.46
Millingen	1 / 6163	1 / 6900	1.12
Tiel	1 / 4215	1 / 5712	1.36
Opijnen	1 / 3907	1 / 5712	1.46
Amerongen	1 / 28160	1 / 28160	1.00
D.W.	1 / 7441	1 / 8352	1.12

Table 2. Critical discharges in the situation of overflow of a dike section with height (DWL + 0.5 [m+NAP]).

Location	Critial discharge [m^3/s]	
	Without retention	With retention
Lobith	17500	18000
Millingen	18100	18300
Tiel	17600	18000
Opijnen	17500	18000
Amerongen	20100	20100
D.W.	18400	18500

The first thing we notice is that the failure probabilities without retention have become smaller (compared to the previous section). From the results it becomes clear that the *QH*-relation at Amerongen is very flat: a huge discharge is required to cause overflowing of the dike when it has been heightened by 0.5 meter. The steepest *QH*-relation is the one for Opijnen, and consequently this location also has one of the lowest critical discharges.

The results for retention vary somewhat. Clearly the effect of retention on the failure probability is positive, although not nearly as large as in the computation of Section 5.1. This can also be noticed in the limited efficiency. The reason for this diminishing effect is largely due to the fact the inlet sill is kept at a fixed level of 16000 m^3/s. For the location of Amerongen, the retention area has already been filled before the peak of the discharge wave arrives, and is therefore useless unless the inlet sill is raised.

Ideally, the discharge level for which water should be drawn-off into the retention area is equal to the critical discharge for that specific location (see Table 2).

5.3 *Random variables: discharge and water level (height of dike section: DWL + 0.5 [m+NAP])*

In addition to the discharge, we now also include the water level as a random variable. The height of the dike section is again equal to DWL + 0.5 [m+NAP]. The total discharge wave is now included. With two random variables it is possible that failure of a dike section can occur not only at the peak of the discharge waves (as was the case in Sections 5.1 and 5.2), but also at discharge levels *below* the peak.

It is assumed that the uncertainty surrounding the bifurcation points of the river can be modeled by a normal distribution with a standard deviation that is equal to 1% of the inflow at a bifurcation point. The mean of the distribution is assumed to be equal to the local water level.

Because of lateral inflow and differences in roughness and geometry, the uncertainty in the water level is different for specific branches of the river. For each of the river branches, the used standard deviations can be found in Table 3. The branches that start after the two bifurcation points in the Rhine (the Lower Rhine and the IJssel) have the largest standard deviation, which is caused by the fact that the uncertainties surrounding these bifurcation points add up.

When retention is introduced as a measure, an additional uncertainty is incorporated that is related to the uncertainties of retention. It is assumed that this uncertainty is different for each of the branches of the Rhine. The adapted standard deviations per branch in the case of retention are also shown in Table 3.

Table 3. The used standard deviations of the water level per river branch

River branch	Standard deviation [m]	
	Without retention	With retention
Upper Rhine	0.11	0.15
Waal	0.12	0.14
Lower Rhine	0.17	0.17
IJssel	0.25	0.25

The results of the calculations can be found in Table 4. The failure probabilities in the situation without and with retention are both shown, as well as the resulting efficiency factors.

Table 4. Failure probabilities, caused by overtopping of a dike section, including both discharge and water level as random variables (DWL + 0.5 [m+NAP]).

Location	Failure probability [-]		Efficiency
	Without retention	With retention	
Lobith	1 / 3265	1 / 5119	1.57
Millingen	1 / 4295	1 / 5965	1.39
Tiel	1 / 3311	1 / 5248	1.59
Opijnen	1 / 3151	1 / 5107	1.62
Amerongen	1 / 7387	1 / 9752	1.32
D.W.	1 / 2494	1 / 3547	1.42

We see that the impact of the additional uncertainty in the water level has a distinctly negative impact on the failure probability (compared to the previous sections). In particular for the locations with large standard deviations the effects are quite large (such as Amerongen and the Duursche Waarden).

The efficiency of retention has even increased compared to the previous section. The reason for this is the combination of uncertainties in the water level and the use of the shape of the discharge wave. In the case of overflowing of a dike section without uncertainties in the water level, it is possible to select a single discharge for which the dike section will overflow. A location such as Amerongen, for which the retention area has already been completely filled before the peak of the discharge wave passes, will never profit from retention. When the uncertainties in the water level are included there are multiple times within one discharge wave at which overflowing may occur. This increases both the failure probabilities and the efficiency, because other discharges (water levels) besides the peak of the wave are important.

5.4 *Random variables: discharge, wind direction and wind speed (height of dike section: DWL + 0.5 [m+NAP])*

In this calculation we did not only look at overflowing of a dike section, but at wave run-up due to the effect of the wind as well. So instead of uncertainties in the water level, we now added the speed and direction of the wind as random variables, besides the discharge. Again, the effect of the entire shape of the discharge wave is important, because failure of a dike section may occur not just at the peak. The results for both the failure probabilities and the efficiency can be found in Table 5.

Table 5. Failure probabilities caused by overtopping of a dike section, with discharge, wind direction and wind speed as random variables (DWL + 0.5 [m+NAP]).

Location	Failure probability [-]		Efficiency
	Without retention	With retention	
Lobith	1 / 1708	1 / 2398	1.40
Millingen	1 / 1741	1 / 2254	1.29
Tiel	1 / 1856	1 / 2691	1.45
Opijnen	1 / 1149	1 / 1293	1.13
Amerongen	1 / 5054	1 / 7357	1.46
D.W.	1 / 2091	1 / 2819	1.35

The failure probabilities have increased significantly in comparison to Table 3, even more so than in Table 4. A location that seems particularly vulnerable to effects of wind-induced waves is Opijnen, where the failure probability even drops below the safety standard of 1 / 1250. The effect of retention is also very poor for this location.

5.5 *Random variables: discharge, water level, wind direction and wind speed (height of dike section: DWL + 0.5 [m+NAP]).*

The computations in this section are a combination of the random variables in Sections 5.3 and 5.4. Again, we investigated only the failure mechanism of overtopping due to overflowing of a dike and due to wave run-up. The results for both the failure probabilities and the efficiency can be found in Table 6.

Table 6. Failure probabilities caused by overtopping of a dike section, with discharge, water level, wind direction and wind speed as random variables (DWL + 0.5 [m+NAP]).

Location	Failure probability [-]		Efficiency
	Without retention	With retention	
Lobith	1 / 1583	1 / 2055	1.20
Millingen	1 / 1514	1 / 1808	1.19
Tiel	1 / 1678	1 / 2219	1.32
Opijnen	1 / 1064	1 / 1169	1.10
Amerongen	1 / 2059	1 / 2458	1.19
D.W.	1 / 1015	1 / 1106	1.09

We see that besides the location of Opijnen, the location of Duursche Waarden now drops below the annual safety standard of 1 / 1250 as well. Both locations are sensitive to wind effects and uncertainties in the water level, and these properties translates itself into an unfavorable efficiency for retention. The impacts for the other locations can be less clearly distinguished, although clearly the location of Amerongen benefits from a ‘flat’ *QH*-relation.

5.6 *Random variables: discharge, water level, wind direction, wind speed and the parameters of the frequency-discharge relation (height of dike section: DWL + 0.5 [m+NAP]).*

As a further extension to the calculations of Section 5.5, the parameters in the discharge frequency function are considered to be uncertain. For details we refer to Stijnen et al, 2002). The results are shown in Table 7.

Table 7. Failure probabilities caused by overtopping of a dike section, with discharge, water level, wind direction, wind speed and the parameters of the frequency-discharge relation as random variables (DWL + 0.5 [m+NAP]).

Location	Failure probability [-]		Efficiency
	Without retention	With retention	
Lobith	1 / 1129	1 / 1422	1.26
Millingen	1 / 1101	1 / 1296	1.18
Tiel	1 / 1185	1 / 1515	1.28
Opijnen	1 / 824	1 / 903	1.10
Amerongen	1 / 1469	1 / 1737	1.18
D.W.	1 / 807	1 / 880	1.09

For each of the investigated locations the failure probability increases with approximately 30% to 40%. This is roughly consistent with an increase in the design discharge of 500 m^3/s.

Regarding the efficiency, we see that in this case not much has changed compared to the results of Table 6. This can be explained by realizing that the effect of the uncertainties in the frequency discharge relation is present in both the situation with and without the measure retention.

6 DISCUSSION

The results of Section 5 are perhaps somewhat surprising: the retention area seems to have a big influence on the exceedance frequency of water levels, (see section 5.1), but not on the failure probability of wave overtopping. It is important to realize that (many) people and cattle are living in the retention areas, and in order to actually use the areas they have to be evacuated. A flood forecast of 1-2 days is needed for evacuation of the area. The river discharge can be forecasted reasonably accurate, but the other sources of uncertainty cannot. In the calculations it is assumed that the retention area is used when the river discharge exceeds the level of the design discharge, 16000 m^3/s. However, since the dikes are higher than the water levels that correspond to the design discharge, the retention area is sometimes used unnecessary. On the other hand, the system may fail for discharges that remain lower than the design discharge (due to uncertainties in the water level).

In order to interpret more detailed the results in Section 5, we distinguish three different classes in which locations can differ: the slope of the *QH*-relation, the sensitivity to wind-induced waves and the uncertainty in the water levels. For each of the three classes we have made a rough indication (see Table 8).

Table 8. The influence of the *QH*-relation (1), the uncertainty on the water level (2), and wind-induced wave effects (3). The meaning of the symbols is given in the text.

Location	Impact on exceedance probabilities (small/large)		
	(1)	(2)	(3)
Lobith	ooo	o	o
Millingen	oo	oo	oo
Tiel	ooo	o	o
Opijnen	ooo	o	oo
Amerongen	o	ooo	ooo
D.W.	oo	ooo	ooo

An explanation of the symbols that are used in Table 8 above is given below:

(1) For the *QH*-relation a “o” indicates a relatively flat gradient, while a “ooo” stands for a relatively steep gradient. It is in fact a comparison of the failure probabilities of Table 1.

(2) The uncertainties in the water level are closely related to the uncertainties regarding the two bifurcation points. The Duursche Waarden and Amerongen are located after the two bifurcation points and the impacts are therefore the largest for these two locations. In this case we have compared Table 3 with Table 5.

(3) The effect of waves is largely influenced by different effective fetches and the orientation of the dike section. A “o” means that a location is not particularly sensitive to the influence of the wind (and waves). In contrast, a “ooo” indicates a location that is sensitive to wind effects. Examples of such locations are the Duursche Waarden en Opijnen. Now we compared the results of Table 3 and Table 6.

In comparison, the weight of the slope of the *QH*-relation is larger than that of the other two criteria. This follows for example from the results of Amerongen, which is a location that is influenced substantial by uncertainties in the water level as well as by waves, but still comes out as a relatively safe location.

7 CONCLUSIONS & RECOMMENDATIONS

The following conclusions and recommendations can be drawn:

a. The safety standard (as it is given in the Flood Protection Law) is an exceedance frequency of water levels, and along the river Rhine this is equal to 1/1250. In the actual design method only one random variable is included: the discharge at the boundary with Germany (Lobith). If we include uncertainties, the failure probability depends on the properties of the location, but is still in the same order of magnitude of the safety standard (range: 1/800 – 1/2000).

b. The efficiency of retention is defined by the failure probability without retention divided by the failure probability including retention as an option. This efficiency strongly depends on the inclusion of all the uncertainties which may cause failure or collapse of a dike. If we take only a single random variable into account (the peak of the river discharge) this factor may be equal to 3.6, whereas if we take *all relevant uncertainties* into account this factor falls in the range of 1.1 to 1.3. The efficiency reduces significantly if we take these uncertainties into account, but the retention areas still have a positive impact on the failure probability.

c. It is possible that the conclusions depend on the assumptions that were made in the study. We have used, for example, the heights of dikes as designed by the actual design rules (instead of the real heights of the dikes). Another assumption that has been made is that the retention area is used as soon as the river discharge rises above 16.000 m^3/s. It is also very well possible that, instead of such a simple control strategy, more advanced control strategies are desirable. Moreover, the different sources of uncertainty may be reduced with additional measures, which may increase the efficiency of retention. We recommend that the influence of these assumptions on the results of this study are investigated further.

d. In a decision-analysis framework, the efficiency factor as such is not important. However, this factor can be used to assess the benefits of flood management measures with greater accuracy. In a cost-benefit analysis (see for example Vrijling, 1990 or Brinkhuis-Jak et al, 2003) the costs and benefits of measures are optimized. We recommend to use the efficiency factor in a cost-benefit analysis of (different sorts of disaster management) measures to reduce the expected flood damage.

Disclaimer

Any opinions expressed in this paper are those of the authors and do not necessarily reflect the position of the Dutch Ministry of Transport, Public Works and Water Management.

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