Efficiency of emergency retention areas along the river Rhine: Monte Carlo simulations of a 1D flow model

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ABSTRACT: A possible emergency retention area along the river Rhine (in the Netherlands) is investigated for its efficiency in increasing flood-protection levels. Towards this objective, Monte Carlo simulations of a 1D flow model of the Rhine are carried out. Random variables that are used to model uncertainties include the shape of the flood wave, wind conditions (strength and direction), and properties of the channel bed (geometry and bed friction). Among these, the dominant sources of uncertainty that affect safety levels at specific locations are identified. Furthermore, safety levels are compared to results from a previous study in which retention-effectiveness through uncertainties in stage-discharge relationships have been determined. The efficiency of current flood protection plans is discussed and recommendations are given for further studies.

1 INTRODUCTION

In recent years plans were made in the Netherlands to investigate the possibility of using an emergency retention area in the Netherlands to provide a preferred flooding area in case of extreme discharges on the river Rhine. Even though the entire river branches of the Rhine (in the Netherlands) should have a safety level such that on average flooding may only occur less than once every 1250 years (corresponding to a discharge of 16000 [m³/s] on the Rhine), an emergency retention area could provide the additional level of safety should an even larger discharge occur. In such an event, the designated area will be flooded in an attempt to avoid even larger damage elsewhere.

The present study investigates the efficiency of such a retention area by taking into account uncertainties that may be associated with the shape of the flood wave, uncertainties in river characteristics (geometry and floodplain roughness) and weather (wind) conditions. For this purpose, Monte Carlo simulations of the river's 1D hydraulic model (SOBEK) were run, in combination with two mechanisms that may cause flooding: flood wave overtopping (water level exceeds local crest level of dike) and surface wave overtopping. A method to describe the latter is adopted from van der Meer (1997), where surface waves are generated by wind and consequently overtop (or damage) the dike.

An earlier study by Stijnen et al. (2002) investigated the efficiency of a possible emergency retention area, by associating uncertainties with (meas-
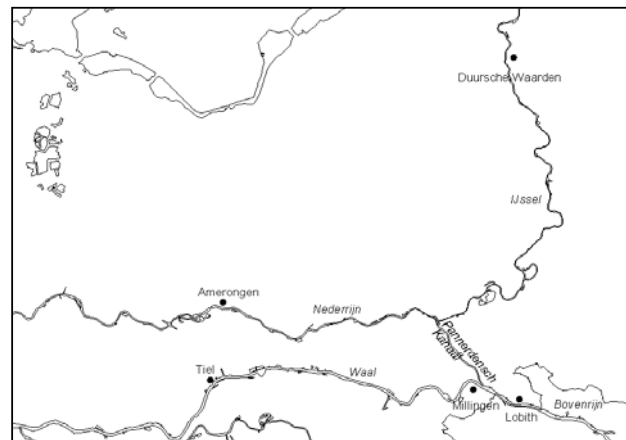

Figure 1: The sudy area in the dutch part of the river Rhine, reference locations are also shown.

Table 1. Study locations.

Name	River branch	Location (River km)
Lobith	Bovenrijn	862
Millingen	Waal	868
Tiel	Waal	915
Amerongen	Nederrijn	918
Duursche Waarden	IJssel	961

ured) stage–discharge relations at 5 locations along the Rhine-branches. Here we adopt the same reference locations for further investigation (see Figure 1 and Table 1), and also adopt the assumptions that were made by Stijnen et al. (2002) concerning retention location, storage capacity (250 M[m³]) and the manner of discharge-extraction from the flood wave.

2 THE HYDRAULIC SIMULATION MODEL

Analogous to Stijnen et. al (2002), the retention emergency area is simulated by correcting a passing flood wave for the available storage volume in the retention area. Figure 2 demonstrates how this wave correction takes place. In effect, the retention area instantaneously extracts all discharge above the critical value (i.e. the discharge above which deployment of emergency retention occurs, at $Q = 16000$ [m^3/s]). Next, for the progression of the flood wave through the study area, the SOBEK (1D) river flow model for the Dutch Rhine branches is used (version RT_2000 3, Duizendstra & Hartman 2002).

3 THE MONTE CARLO PROCEDURE

Monte Carlo Simulation is a method in which the uncertainty of an outcome is determined by evaluating the statistical properties of many experiments, with each experiment corresponding to a random set of input variables. In performing such a procedure, it is of great importance to (i) know the relevant input parameters of the process, (ii) know the probability distributions of these parameters and (iii) be aware of correlations between distributions of input variables. Together, they determine how many experiments (or, 'Monte Carlo runs') are necessary to reach a statistically reliable result.

A common problem in Monte Carlo Simulations is that the uncertainties of input parameters are not well known, let alone the correlations among input parameters. This is also the case in the present study. Therefore, we have chosen a probability distribution function for some of the input parameters, even though there is limited knowledge available. That way we still get uncertainty bounds in the results that reflect the various (assumed) uncertainties at the input side. The question of determining uncertainty bounds around a complicated model result is thereby transferred to the question of determining uncertainty bounds around (more) input parameters that are 'simple'.

3.1 *Spatially varying random variables*

In the present study, some input uncertainties are associated with spatial characteristics of the system, such as river geometry (bed levels across and along the river axis), bed roughness values and lateral discharges (in- and outflow). To treat each parameter at each location separately would be too extensive, and would also make it difficult to relate behavior of the outcome to processes at the input side. For that reason, some input parameters are divided into categories, of which each is treated as one random variable (Table 2). Take for example floodplain roughness,
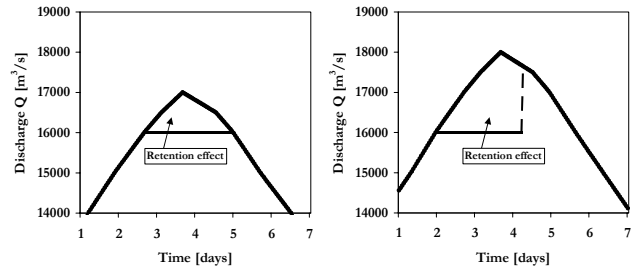

Figure 2: Correction on discharge waves to simulate the effect of the emergency retention area. For the wave on the left hand side (Q_{max}= 17000 [m^3/s]) the storage capacity of the retention area is large enough to extract all the discharge that exceeds the critical value of 16000 [m^3/s]. For the larger wave on the right, the emergency retention area is completely filled before the entire lood wave has passed.

where all roughness values are divided into one of three categories (Tables 2-3). Within each category a randomly drawn factor (according to an assumed probability distribution) is responsible for the variability of the whole category (all parameters from that category are multiplied by the random factor). Different categories have different random factors with different (chosen) probability distributions. One Monte Carlo experiment may result in the following three random factors for floodplain roughness: 1.07 for category 1, 1.01 for category 2 and 0.98 for category 3. As a consequence, all roughness values that belong to category 1 are multiplied by 1.07, category 2 by 1.01 and so on.

Table 2 gives an overview of all the random variables that have been included in the Monte Carlo Simulation. The following paragraphs give some more detail on their (chosen) characteristics.

3.1.1 *River geometry*

3.1.1.1 Reference height

In a SOBEK schematization of river geometry, bed elevation levels are defined relative to a reference height, which is typically set to 0 [m], and may be defined separately for each river branch. Changing this parameter thus elevates (or sinks) the whole branch by the chosen amount. The used model 'RT_2000 3' (Duizendstra & Hartman 2002) comprises 10 branches. For each of these, a random factor will be used to shift the branches independently. Since these vertical shifts occur over the whole branch they directly affect the water level and also influence discharge distributions at connecting nodes (branches meeting or diverting).

Assuming geometrical errors in the order of cm's, we chose normal distributions for all random factors (one for each branch) around a mean of 0 [m] with a standard deviation of $\sigma = 0.01$ [m].

Table 2. Overview of random variables.

Variable	Characteristics	Correlations	Remarks (probability distributions)
Wind	Conditional probabilities: 1. Wind speed 2. Wind direction 3. Storm duration	Wind speed & direction	Speed & direction taken from Geerse et al. (2002) Chosen distribution for storm duration (μ= 48, σ=8 [hours])
Roughness (floodplain)	Definition roughness categories 1. Low vegetation 2. Medium size vegetation 3. High vegetation	Complete dependence within 1 category, complete independence between categories	Chosen distributions (normal)
Flood wave	Wave generator (Klopstra & Duits 1999)	none	Range of wave shapes: 5%-95% (wave generator) Range of discharges: Q=13000-19500 [m³/s]
Geometry	Variation of transition – and reference height	Transition height: none Reference height: complete independence between river branches, complete dependence within 1 branch (10 branches total)	Chosen distributions (normal)
Lateral discharge	Definition discharge categories: 1. Outflow 2. Inflow 3. Twentsch Kanaal 4. Oude IJssel	Complete dependence within category, complete independence between categories	Chosen distributions (normal)

3.1.1.2 Transition height

When the water level in the main river section reaches the bank height, additional flow areas behind the summer dike may become available. The transition height is the height relative to the bank height where the available flow area behind the summer dike is completely filled; it therefore reflects the speed with which this area fills up. Typically, in SOBEK-models a transition height of 0.75 [m] is used. We introduce an uncertainty to this parameter by choosing a normal distribution of the transition height with a standard deviation of $\sigma = 0.10$ [m] around a mean of 0.75 [m].

3.1.2 *Channel roughness*

In most SOBEK river flow models a distinction is made between causes of energy loss in the flow field:

1 Main section roughness (bed bottom roughness)
2 Flood plain roughness (vegetation, roads, etc.)
3 Additional roughness (meandering, obstructions)
4 Additional roughness within structures

Flood plain roughness is quantified with a (Nikuradse) roughness height (k), which reflects the steepness of the vertical (logarithmic) velocity profile. One may associate this parameter with the local vegetation height. Three categories of flood plain roughness have been varied independently in the Monte Carlo Simulation through the introduction of three random variables (see Table 3). It was chosen to decrease the standard deviation for the higher vegetation categories, in order to avoid unrealistically large roughness heights.

The remaining types of roughness parameters will not be treated as random variables, because their probability distributions and correlations are unknown and difficult to estimate. Defined roughness-discharge relations in the main river section are the result of calibration without a solid physical explanation. Consequently, it is not clear on what these values depend, and how they should be treated as random variables. Moreover, in this study only global random variables will be considered (exceptions are lateral discharges), in order to avoid effects in the results that are no longer globally interpretable. For that reason the additional roughness due to structures and obstructions (which are defined locally) are not included as uncertain variables.

Table 3. Monte Carlo vegetation roughness categories.

Category	Roughness range [m]	σ/k [-]
Low vegetation	$k < 0.15$	7%
Medium size vegetation	$0.15 \leq k < 0.3$	6%
High vegetation	$k \geq 0.3$	5%

* σ: chosen standard deviation, k: roughness height (Nikuradse).

3.1.3 *Lateral discharges*

Along all branches in the used SOBEK model lateral discharges are defined, reflecting inflow due to rainfall, connecting streams or discharge-extraction activities (e.g. for irrigation). The river branch *IJssel* that passes the study location *Duursche Waarden*, is the only branch in the model with significant lateral discharges due to connecting streams (*Oude IJssel* and *Twentsch Kanaal*). The

other branches are only affected by runoff discharges.

As in the case of flood plain roughness, independent variables are defined as separate random variables (see Table 4). Inflow and outflow are treated separately, and so are the two larger streams *Oude IJssel* and *Twentsch Kanaal.*

Table 4. Monte Carlo lateral discharge categories.

Category	Q [m^3/s]	σ/Q [-]
Outflow	Q>0	10%
Inflow	Q<0	10%
Twentsch Kanaal	Q_{TK}	10%
Oude IJssel	Q_{OIJ}	10%

3.1.4 *Shape of the flood wave*

Klopstra & Duits (1999) developed a procedure that enables generation of flood waves on the river Rhine. For a given maximum discharge various shapes of the flood wave can be determined, which will be used in the treatment of wave shape as a random variable in the Monte Carlo Simulation. The median of possible flood-wave shapes is characterized by the 50%-percentile wave. Waves corresponding to a lower percentile value are more sharply peaked, and higher percentile values are more gradually peaked (broad waves). Figure 2 shows an example of the 50%-percentile wave with maximum discharges of 17000 and 18000 [m^3/s].

3.1.5 *Wind*

In SOBEK a storm is characterized by 3 parameters: speed, direction and duration. The wind direction determines the relative influence of the wind on the local water level for each river branch separately; if the wind blows parallel to the main flow direction of the river branch then the influence is larger than for transverse winds. In practice, storm direction and strength appear to be correlated. The values used here (including correlations) are taken from Geerse et al. (2002). Furthermore, the following assumptions were made regarding the storms:

1 After a (linear) built-up from 0 [m/s] to its maximum wind-strength, the storm holds on for 2 hours. Afterwards, the storm weakens again (linearly) to the base level of [0 m/s].
2 The average storm duration is 48 hours, with a standard deviation of 8 hours.
3 Storm maxima always occur when the crest of the flood wave passes *Lobith*.

3.2 *Effect of parameter variation on water level*

Figure 3 shows the effect of including an increasing number of random variables in the Monte Carlo
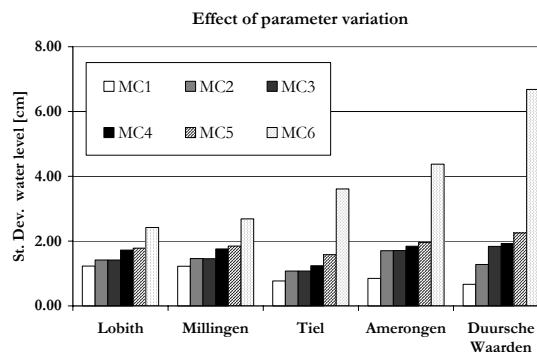

Figure 3: Effect of an increasing number of random variables (MC1-MC6, see text) on the water level uncertainty at the 5 reference locations (corresponding to a peak discharge of 16000 [m^3/s]).

simulation. Six Monte Carlo (MC1-MC6) procedures of each 2500 SOBEK simulations were carried out with the following random variables:

1 MC1: wind (direction, speed and duration).
2 MC2: as MC1 *plus* floodplain roughness.
3 MC3: as MC2 *plus* lateral discharges
4 MC4: as MC3 *plus* transition height
5 MC5: as MC4 *plus* reference height
6 MC6: as MC5 *plus* wave shape

The procedure MC6, containing all random variables, is used in the remainder of the study (see Table 2 for overview). Figure 3 shows that variations in wind characteristics (MC1) have the largest effect near *Lobith* and smaller effects further downstream (smallest effect at *Duursche Waarden*). This trend may be due to the fact that maximum storm strength always occurs when the crest of the floodwave is near *Lobith*. At *Amerongen* and *Duursche Waarden* variations in floodplain roughness (MC2) result in an increase in water level uncertainty that is about as large as the increase after MC1 (wind variations). At the other three locations the influence of floodplain roughness is notably smaller. The relative size of the floodplain is the dominant factor here (which is larger on the *Nederrijn* and the *IJssel*).

Only at location *Duursche Waarden* the variation in lateral discharges (MC3) has significant influence on water level uncertainties. This is due to the streams of *Oude IJssel* and *Twentsch Kanaal* that connect to the *IJssel*. In general, the effect of wave shape variation (MC6) has the largest overall effect on the spread of local water levels and increases in downstream direction. For *Tiel*, *Amerongen* and *Duursche Waarden*, the influence of uncertain wave shapes is responsible for more than half the resulting uncertainty in local water levels.
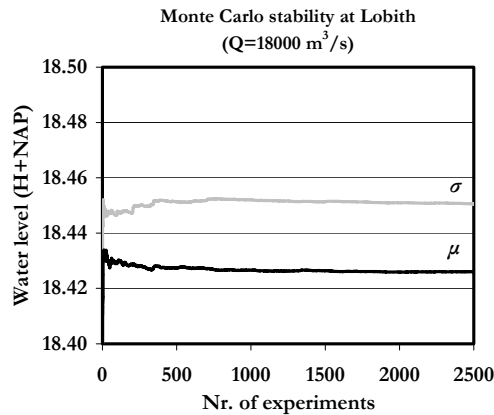
Figure 4: Progressive average water level (μ) and the corresponding standard deviation (σ) at *Lobith* with increasing number of experiments (in the Monte Carlo simulation). A flood wave with peak discharge of 18000 [m³/s] was considered.

3.3 *Size of the Monte Carlo procedur*

An important issue in Monte Carlo simulations is the amount of experiments needed to achieve a statistically reliable result. Here it was chosen to start out with 2500 SOBEK simulations and then test progressive values for stability. Figure 4 shows the progressive average water level (μ) and the corresponding standard deviation (σ) for *Lobith* at a discharge of 18000 [m³/s]. It can be seen that these values become relatively stable after 1500 SOBEK simulations. In case of retention deployment, the average water level is less stable because flood waves may be completely topped off (Fig. 2) or only partly, resulting in large water level differences for a constant maximum discharge. Nevertheless, 2500 SOBEK simulations seem sufficient to get reliable results (Fig. 5). For each of the 13 maximum discharge values (from 13500 to 19500 [m³/s] in steps of 500 [m³/s]) 2500 SOBEK runs were made.

4 FAILURE MECHANISMS

Two failure mechanisms (that cause flooding) are considered here in order to evaluate the safety level of the study locations:
1 Failure by flood wave overtopping: the maximum water level of the passing flood wave is higher than the local crest level of the dike.
2 Failure by surface wave overtopping: due to wind conditions, surface waves are generated that attack or overtop the dike. At a critical overtopping discharge the dike will fail.

The first failure mechanism is determined by the height of the flood wave and the local crest level. In this study, the local crest level is equal to the height that guarantees a safety level of flooding once every 1250 years (corresponding to a discharge of 16000
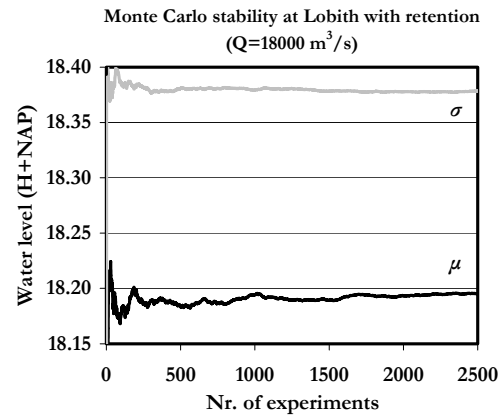

Figure 5: Same as Figure 4, but with deployment of the emergency retention area.

[m³/s] on the Rhine), *plus* an additional safety margin of 0.5 [m]. The second failure mechanism is based on the procedure as proposed by Van der Meer (1997), which determines a critical overtopping discharge due to surface waves based on meteorological conditions (i.e. wind speed and direction).

5 RESULTS

5.1 *Stage-discharge relations*

Stage-discharge relations for three of the five reference locations are shown in Figure 6. Note that the error bounds around the water levels are distributed fairly symmetrical around the mean value if no retention is deployed (16 and 84 percentile boundaries are shown). With retention, the uncertainties in the stage-discharge relation become more asymmetrical around the mean value. Furthermore, the overall magnitudes of estimated uncertainties are smaller than those estimated by Stijnen et al (2002) by more than a factor two (Table 5).

Figure 6 also at shows that, in general, retention increases the uncertainty bounds for predicted water levels. However, in some discharge regimes retention may also suppress uncertainty bounds around the stage-discharge relation: for *Lobith*, at a discharge of 16500 [m³/s] the stage-discharge relation is less uncertain than at surrounding (discharge)

Table 5. Estimated water level uncertainties by Stijnen et al. (2002)

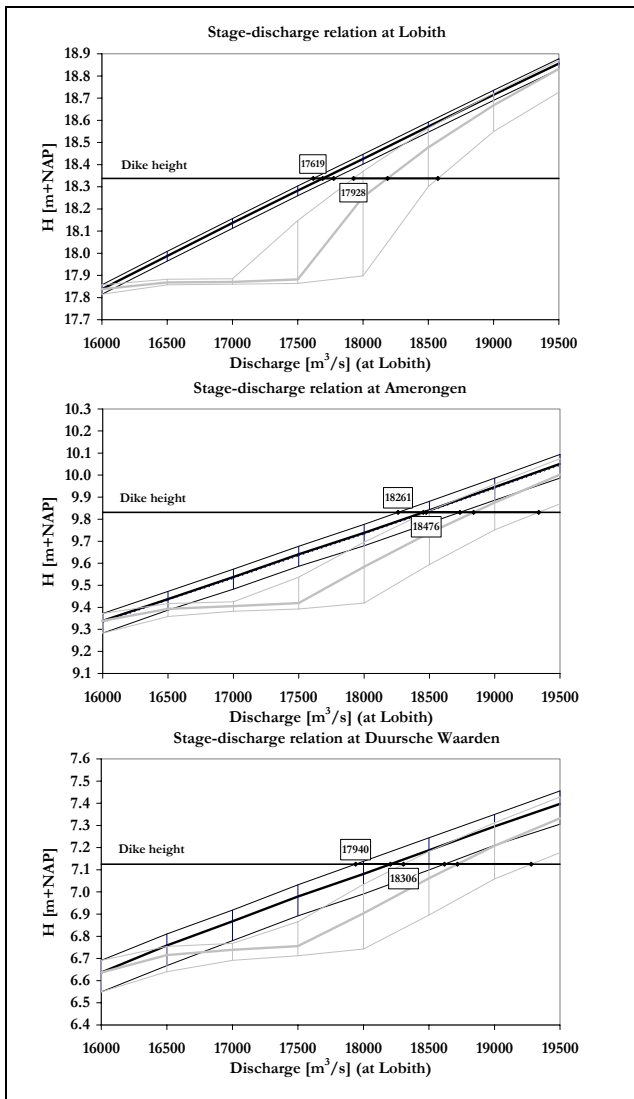| River branch (location)	Water level uncertainty σ [m]
Bovenrijn (Lobith)	0.11
Waal (Millingen, Tiel)	0.12
Nederrijn/Lek (Amerongen)	0.17
IJssel (Duursche Waarden)	0.25

* σ: standard deviation

Figure 6: Stage-discharge relationships of 3 of the 5 study locations (grey lines: with retention). Also the 16 and 84 percentile boundaries around the mean water levels are shown.

values, for *Duursche Waarden* the minimum in uncertainty bounds occurs at around 17000 [m³/s]. This may seem a surprising result, since, effectively, flood waves that are completely topped off (e.g. Fig. 2), are reduced to 16000 [m³/s] flood waves. How can these be less uncertain in behavior than flood waves that had a discharge of 16000 [m³/s] to begin with? The difference between these situations is that, in general, waves that are brought back to a discharge of 16000 [m³/s] by retention will be wider in shape (broad flood waves) than waves that remain unaffected by the retention area. Apparently, wider flood waves are less susceptible to external influences (wind, lateral discharges) than their more sharply peaked counterparts.

5.2 *Flooding probability density functions*

Having determined the stage-discharge relations with corresponding uncertainty bounds, a (flooding) *probability density function* (PDF) can be deter determined. In Figure 7 a PDF is shown for location
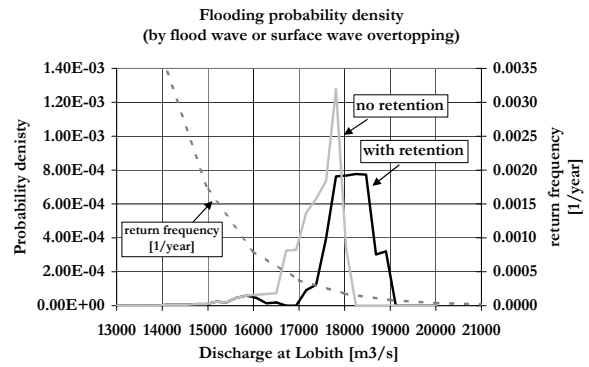

Figure 7: Flooding probability density function (PDF) at location *Lobith* with and without the deployment of retention (considering both flooding by flood wave and surface wave overtopping). The return frequency density function (RFF) of discharges is also plotted (corresponding to the axis on the right, function taken from Stijnen et al. 2002).
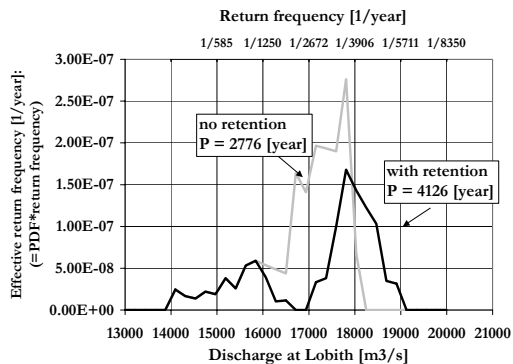

Figure 8: The effective probability density function (EPDF) is derived by multiplying the flooding probability density function with the return frequency density function (Fig. 8). The overall return periods (corresponding to the area under the graph) for the scenarios with and without retention are also given (P).

Lobith, taking into account both mechanisms. Next, the flood return period P (i.e. the average period between consecutive floods) for each location can be determined by weighing the PDF against the *return frequency density function* (RFF, Fig. 7) of given discharges, to give the *effective probability density function* (EPDF, Fig. 8). The RFF describes the yearly probability of discharge occurrence. Figure 8 shows the EPDF (=PDF*RFF) for location *Lobith* considering both failure mechanisms. The area below this function gives the overall return period (P) for flooding at that location.

This method of determining P is used for each of the study locations, considering (i) failure by flood wave overtopping and (ii) failure by flood *or* surface wave overtopping. Figure 9 and 10 show the corresponding results together with the values as determined by Stijnen et al. (2002). Furthermore the efficiency of the emergency retention area, defined as the fraction between return period with retention and without retention (P_{RET}/P) is shown in table 5. Again, the corresponding efficiencies as determined
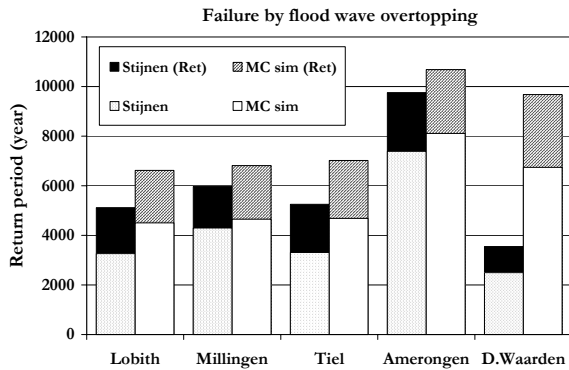
Figure 9: Return periods when considering only the failure mechanism overflow (no surface waves). The top part of each column in the histogram denotes the effect of the emergency retention area (Ret). Results from Stijnen et al (2002) are also shown.

by Stijnen are also shown. (see Figs 9 & 10 for return periods).

Note that when retention is deployed, the PDF (and the EPDF) goes to zero for discharges of around 16500 [m^3/s] (Figs. 7 & 8), while for some lower discharges the flooding probability is larger! This can be explained by the earlier addressed uncertainty-minimum in the stage-discharge relations at a discharge of 16500 [m^3/s]: at lower discharges (around 16000 [m^3/s]) the mean water level may be lower, but due to the larger uncertainty, a relatively larger number of events may lead to flooding by surface wave overtopping.

Figure 8 shows that a relatively large weight is attributed to flood events at low discharges because of the *return frequency function*: a small probability density at low discharges (below 16000 [m^3/s]) still results in a significant contribution to the overall failure return period (compare Figs 7 & 8). Therefore, if the failure density at low discharges increases only slightly, this can have a profound influence on the overall return period.

Extreme water levels corresponding to a low river discharge (i.e the low-discharge tail of the probability distribution) are important for the failure mechanism 'surface wave overtopping'. If, for example, at a discharge of 14000 [m^3/s] a water level occurs of 2σ above the average, then it becomes quite likely that failure due to surface wave overtopping occurs. Even though a 2σ deviation is unlikely in itself, due to the relatively low return period of a 14000 [m^3/s] flood wave, this still has a large impact on the overall flood return period..

Figure 8 also shows that in the low-discharge region the *effective return frequency function* is not very smooth, but shows fluctuations. This is due to under-sampling for the Monte Carlo simulations. Apparently, even though 2500 simulations were enough to determine average water levels satisfactorily, the overall return period cannot be determined very accurately, and more experiments are needed (when both failure mechanisms are considered).
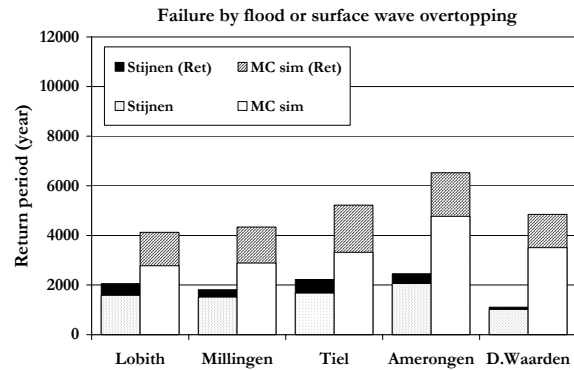

Figure 10: Same as Figure 7, only here both failure mechanisms flood wave and surface wave overtopping are considered.

The reliability of For Monte Carlo simulations increases with the square root of the number of experiments. Therefore, in order to double the reliability of surface wave overtopping events, the experiment set at low discharges should be at least 10000 SOBEK runs (four times 2500).

5.3 *Comparison to study by Stijnen et al. (2002)*

When only the failure mechanism overflow is taken into account, then results correspond well to the results by Stijnen et al. (2002) (Fig. 9). The difference here is mainly due to different stage-discharge relations at the various study locations. In the present study the discharge relations at extreme flood events are predominantly determined by the geometrical properties of the river branches in the SOBEK model, as opposed to relation-extrapolation in Stijnen's analysis. Consequently, having a stronger physical basis, the new results seem to be more reliable when only the failure mechanism overflow is taken into account. On the other hand, if also the failure mechanism 'surface wave overtopping' is considered, this need no longer be the case.

Taking into account both failure mechanisms not only results in smaller return periods, but also in a larger discrepancy between the present study and that by Stijnen et al. (2002) (Fig. 10). Partly this can be explained by differences in the used methodologies: Stijnen also included surface wave overtopping before or after passing of the flood wave crest, here, that effect is neglected. While significant in the study by Stijnen, the effect here would probably be small, because it was already assumed that maximum storm strength occurs when the crest of the flood wave passes (i.e. at the moment that failure through surface wave overtopping is dominant). More important is the fact that in the present study uncertainties in water levels are much smaller than in Stijnen's study (Table 5). Due to the large uncertainty of water levels at discharges below the critical discharge of 16000 [m^3/s], a relatively larger amount of failure by surface wave overtopping will occur in

Stijnen's study. This is reflected in the lower values of the relevant return periods and also in the corresponding lower efficiency of retention deployment (Table 6). Retention has hardly any effect on the failure by surface wave overtopping, and precisely this mechanism is much more dominant in Stijnen's analysis.

Table 6. Efficiency of the emergency retention area when considering both failure mechanisms.

Location	P [year]	P_{RET} [year]	E [-]	E_{ref} [-]
Lobith	2776	4126	1.49	1.30
Millingen	2886	4337	1.50	1.19
Tiel	3320	5222	1.57	1.32
Amerongen	4767	6524	1.37	1.19
Duursche Waarden	3507	4848	1.38	1.09

* P and P_{RET}: flood return period with and without retention respectively (present study). E: efficiency (P_{RET}/P). E_{ref}: efficiency of reference study (Stijnen et al. 2002).

6 CONCLUSIONS

In summary, the following conclusions were drawn in the present study:

1 The uncertainty in the shape of a flood wave is the most dominant factor in determining a local stage-discharge relationship with uncertainty bounds. Next, roughness conditions in the flood plain and wind conditions have an equally large impact on the distribution of possible water levels.

2 Without the use of a retention-emergency area, downstream water level probabilities (corresponding to a certain discharge at Lobith) are distributed fairly symmetrical around their mean value. Deployment of an emergency retention area heavily affects the shape of the probability distribution, resulting in a less symmetrical water level distribution with a wider spread.

3 The present study gives a more optimistic view of the efficiency of an emergency retention area near *Lobith* than the study by Stijnen et al. (2002). This difference is due to (i) in the present study, failure by surface wave overtopping is only considered at the peak of the flood wave and (ii) water level uncertainties at a given discharge are much smaller here as compared to Stijnen's study.

4 At low discharges, a larger set of experiments for the Monte Carlo simulation is needed. This would describe the effect of surface wave overtopping on the overall failure probability more adequately.

7 DISCUSSION

While the method in the current work has a stronger physical basis than the method by Stijnen et al (2002), a clear disadvantage is its large computational effort. The set of 2500 experiments (per discharge level) still seems to be too small for statistically stable retention efficiency prediction. Instead of expanding the current research by expanding the set of Monte Carlo experiments, it would require less effort to add more physical content to Stijnen's method. For example, flood waves that actually progressed through a hydrodynamic model could be used. That way, energy dissipation of the flood wave on its downstream progression could be taken into account. This effect becomes more important for sharply peaked flood waves, or similarly, for flood waves that were only partially topped off by retention deployment. For this purpose three representative wave shapes could be considered: a sharply peaked wave, the median and a broad wave. Eventually, retention efficiencies corresponding to each of the wave shapes have to be weighed to the relative probability of occurrence to find an overall efficiency value.

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