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An advanced method for flood risk analysis in river deltas, applied to societal flood fatality risks in the Netherlands

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Abstract

This paper discusses the new method developed to analyse flood risks in river deltas. Risk analysis of river deltas is complex, because both storm surges and river discharges may cause flooding and since the effect of upstream breaches on downstream water levels and flood risks must be taken into account. A Monte Carlo based flood risk analysis framework for policy making was developed, which considers both storm surges and river flood waves and includes hydrodynamic interaction effects on flood risks. It was applied to analyse societal flood fatality risks (the probability of events with more than *N* fatalities) in the Rhine–Meuse delta.

1 Introduction

The National Water Plan in the Netherlands requires a reconsideration of flood risk management standards based on cost benefit analyses and fatality risk assessments (Min. V&W, 2009). The latter requirement induced research into flood fatality risks in the Netherlands. Fatality risks also get increasing attention in flood risk management and research in other countries, such as France (Lalande, 2012), USA (US Department of the Interior, 2011), the UK (Di Mauro et al., 2012), Belgium (IMDC, 2005), and Indonesia (Marchand et al., 2009).

Flood fatality risks can be assessed both from an individual and societal point of view. The individual flood fatality risk relates to the probability of a person to die as a result of a flood event. This perspective focuses on hazardous locations without taking into account the population density of those locations. Individual risks have been assessed for the Netherlands as part of the implementation of the National Water Plan (De Bruijn et al., 2010; Beckers et al., 2012). Currently, options for a tolerable level of individual flood risk are being discussed.

Societal flood fatality risk is related to the probability of events with many fatalities. It is expressed by an FN curve: a curve which gives the probability of an event with *N* or more fatalities (De Bruijn et al., 2010). It combines information on flood hazards, flood extents and population density in the flooded areas. In the Netherlands, the Rhine–Meuse delta (both the tidal- and non-tidal part) contributes most to the societal flood fatality risk (Beckers and De Bruijn, 2011; De Bruijn et al., 2010; Beckers et al., 2012). This paper focuses on the development and assessment of a method to analyse societal flood fatality risks in river deltas with flood protection infrastructure, such as the Rhine–Meuse delta. The method needs to facilitate an assessment of current societal flood fatality risks and risks corresponding with alternative flood risk management strategies.

To be applicable, the method should comply with three requirements. Firstly, it must be applicable to an area as large as the (Dutch part) of the Rhine–Meuse delta. Secondly, it should consider the most relevant processes which determine the number of flood fatalities per event. Since in the Netherlands floods are caused by dike breaches, the flood extent of an event is linked to the number and location of dike breaches. To obtain a realistic estimate of the number and location of breaches (and the corresponding consequences) an analysis is required which takes *hydrodynamic interaction* between locations into account. With *hydrodynamic interaction* we mean the decrease of water levels at potential breach locations due to a breach elsewhere in the river system. This hydrodynamic interaction can have a significant effect on the failure probabilities of (downstream) flood defences (Van Mierlo et al., 2007; Apel et al., 2009; Vorogushyn et al., 2012). Since upstream breaches, may also influence the flood probabilities in the tidal river part, an integrated analysis of both the river dominated and the tidal part of the delta must be carried out.

A third requirement is that the method should facilitate the analysis of societal flood risks corresponding with various potential flood risk management strategies.

The method should result in an accurate FN curve, potential numbers of fatalities for the whole system, insight in the contributions of different areas and insight in the most relevant flood events (set of breach locations, river discharges and sea water levels).

These results support the consideration of societal flood fatality risks in the decision on flood protection standards and flood risk management strategies.

Since we found no existing method which meets these criteria and is fast enough for a policy analysis, we developed a new method. This paper presents and discusses the method and its application on societal flood fatality risks in the Rhine–Meuse delta. The method may also be used to analyse economic risks and the approach is also applicable to other river deltas.

2 Existing flood fatality risk assessment methods

2.1 Deterministic and probabilistic approaches

Various methods have been developed to analyse flood risks in deltas protected by flood defences and the impact of flood risk management strategies on those flood risks. Those methods have been used for studying long term strategies in relation to climate change or socio-economic scenarios (Hall et al., 2005; Gouldby et al.,2008) and for national flood risk assessments (Klijn et al., 2012; Jongejan et al., 2011). Approaches usually include an analysis of possible loads or threats, analysis of the reliability of the protection infrastructure, analysis of breach sizes or breach growth given failure, modeling of the expected flood patterns and assessments of the associated consequences.

Some approaches result in a qualitative indication of the most risky areas (areas where many fatalities may occur) by analysing the most relevant factors which contribute to fatality risks (De Bruijn and Klijn, 2009). Others provide potential numbers of fatalities given flood hazard information, such as the "Risk to People method" of HR Wallingford et al. (2006), the analysis for the second sustainability outlook of the Netherlands (Klijn et al., 2012) and the long-term flood risk management strategy research for the Scheldt Estuary (De Bruijn et al., 2008). These deterministic approaches have the potential to deal with detailed information, but are generally based on indicative maps of the most important flood hazard and flood vulnerability indicators. Such a deterministic approach is useful in explorative analyses in which for example effects of climate change on flood risks are studied, when various long-term strategies are compared, or when the areas with highest flood risk must be identified. These studies generally do not take into account the various uncertainties in flood hazard and vulnerability parameters and may therefore grossly under- or overestimate the flood risk.

For issues that require more accurate flood hazard or flood risk estimates, probabilistic methods are emerging (Kalyanapu et al., 2012). Comprehensive flood risk analyses require the incorporation of uncertainties in both hazards and vulnerabilities (Vorogushyn et al., 2012). Probabilistic methods allow the incorporation of uncertainty in input data, model parameters, spatial and temporal variations (Di Baldassarre et al., 2010) and thus in both hazard and vulnerability parameters.

In the Netherlands, a national flood risk assessment project (VNK) is being carried out based on a probabilistic method (Jongejan et al., 2011; Den Heijer and Diermanse, 2012). In this project, risks of “dike rings” (areas surrounded by levees or higher grounds) in the Netherlands are being studied by first analyzing dike section failure probabilities, as a result of various failure mechanisms, followed by an integration of the failure probabilities of many dike sections to the failure probability of the dike ring. This procedure requires knowledge or assumptions on the correlations between failures at different locations. The consequences associated with dike failure at each location are taken into account to lead to a total flood risk per dike ring. The VNK project does not aim to provide risk estimates for the Rhine–Meuse delta or the Netherlands as a whole, but focuses on results per dike ring. The effect of breaches in other dike rings are not taken into account, which may cause a significant overestimation of the flood risk (De Bruijn, 2014).

Hall et al. (2005) developed an approach to enable a preliminary large-scale flood risk assessment for the whole of England and Wales. Gouldby et al. (2013) slightly adapted the approach and studied fatality risks. In these approaches, the load is assumed to be fully dependent within one flood area. Load reduction at downstream dike sections due to dike failures in the system was not taken into account, i.e. the response of different flood defence sections was studied independently from each other. The approach of Hall et al. (2005) and Gouldby et al. (2013) is suitable for coastal defence structures. However, for fluvial systems such as rivers and estuaries, where the effect of water retention may be significant, this effect on flood probabilities should be taken into account.

In the Dutch Rhine and Meuse River system, a breach in one of the upstream dike rings has an alleviating effect on the full downstream river branch. The hydraulic load thus varies along the course of the river and the probability of breaching in one of the downstream dike rings depends on what happens upstream in a way that cannot be approximated by an assumption of full dependency or independency. To assess the societal risk in such systems requires the consideration of *hydrodynamic interaction*.

In Beckers and De Bruijn (2011), a probabilistic approach was used to assess the societal flood fatality risk in the Netherlands. The correlation between breaches and the hydrodynamic interaction in the river area were included by using estimated correlation factors between dike rings and by expert assumptions of the retention effect of breaches on downstream breach probabilities. An event-tree of all combinations of dike ring flood scenarios was constructed to evaluate the probability of a given number of fatalities. In order to keep the computational effort feasible, the dike rings were used as basic elements in the event tree, instead of the far more numerous dike sections. Furthermore, the tidal area was considered fully independent from the river discharge dominated river area. The results of this study showed that the river area contributes most to the flood fatality risks in the Netherlands (De Bruijn et al., 2010; Beckers et al., 2012). Weak points in this study were the assumed correlation between breaching of dike rings, based on expert-judgment, and the use of dike rings as basic units in the event-tree, where individual dike sections would be more appropriate. The flood probabilities and flood consequences vary significantly between the different dike sections within one dike ring. This variation is lost when averages are used.

2.2 Effect of hydrodynamic interaction

Many flood risk analysis methods focus on failure probabilities per location, without taking into account hydrodynamic interaction. The reduction of the failure probability of downstream locations due to upstream breaches may, however, be very large. In risk estimates for single polders or of individual dike sections, neglecting this hydrodynamic interaction may result in an overestimation of risks. The assessment of societal flood fatality risks of floodplains along rivers is even impossible without assumptions or calculations on the hydrodynamic interaction, since one must assess the number of breaches and fatalities which may occur during a single flood event. To do this, one must find a way on how to assess the effect of breaches on downstream failure probabilities.

The effect of hydrodynamic interaction on flood probabilities and flood risk depends on the spatial variation of the hydraulic load and strength of the flood defenses, the moment at which a dike breaks (beginning/peak or after the peak of a flood wave), and the available storage volume of the flood-prone areas in relation to the discharge volume in the river. The effect can be very large as Olson and Morton (2012) and Apel et al. (2009) described. In the Mississippi river in 2011 the US Army made a breach and used the New Madrid Floodway to lower the water levels in the river with about 80 cm (Olson and Morton, 2012a).

Apel et al. (2009) studied the effect of upstream dike breaches on downstream flood frequencies for the Rhine River from Cologne to Rees. They used a probabilistic method to show that dike breaches have a significant effect on downstream discharges and on the probability of exceedance of design discharge levels. They sampled peak discharges and dike strength (from fragility curves for overtopping). By considering hydrodynamic interaction, the 1 : 5000 yr discharge at location Rees (at the Dutch–German border) reduces from 17 500 $m^3 s^{-1}$ to about 15 500 $m^3 s^{-1}$.

Several attempts to include hydrodynamic interactions into flood risk analysis methods have been made before. Van Mierlo et al. (2007) developed an approach to incorporate hydrodynamic interaction in risk analysis and illustrated it on a dike ring in the Rhine–Meuse delta in the Netherlands (Vrouwenvelder et al., 2010). They calculated flood probabilities at predefined potential breach locations without consideration of hydrodynamic interaction, then they sampled river flood waves and assessed whether the sampled flood waves could result in one or more breaches. If so, the flood waves were simulated with the 2-D Sobek overland model (Dhondia and Stelling, 2004; Hesselink et al., 2003) to determine the flood pattern. The consequences of the flooding were determined with the Standard Dutch Damage and Fatality Model (HIS-SSM, Kok et al., 2005). Finally, the risk was assessed as the sum of the damage divided by the number of samples. Their approach was computationally very time consuming. One Monte Carlo simulation took about 2 to 6 days per run (2 GHz Linux PC in 2010). For the analysis of the whole Rhine–Meuse delta instead of only one dike section, therefore, a computationally less demanding method is required. In the method of Vrouwenvelder et al. (2010) it is also not straightforward how to analyse proposed dike designs and their effect on flood risk.

Vorogusyn et al. (2012) showed that the effect of hydrodynamic interaction on flood probabilities for the Elbe flooding 2002 was limited due to the small storage volumes of the floodplains compared to the high flow volumes in that event. They calculated hazard maps, based on roughly the same approach as Vrouwenvelder et al. (2010). They did a Monte Carlo Analysis and coupled three models in a dynamic way: a 1-D unsteady hydrodynamic model for river routing, (2) a probabilistic dike breach model to determine possible dike breach locations, breach growth and the outflow of discharges and (3) a 2-D (storage cell) based inundation model for the protected parts of the flood plains. They considered overtopping, piping and slope instability failure mechanisms. They sampled the river discharge input and dike fragility curves. The method results in flood hazard maps for different flood hazard parameters.

Since Vorogusyn et al. (2012) focus on flood hazards, knowledge on the spatial variation of water depths in the flooded area was required. For our aim of obtaining an FN curve, no information is needed about the geographical variation of the hazard, but only a good estimate of the total number of fatalities for each flooding scenario is required. This number depends on the number and locations of the breaches, but it depends less on the exact inflow and flood pattern in the flood-prone areas. Pre-calculated inundation patterns and associated consequences in terms of number of fatalities can, therefore, be used. Moreover, the use of crude Monte Carlo analysis would take too much time in the Rhine–Meuse delta where some areas have very small flood probabilities, but where the consequences can be large. Therefore, a different setup of the Monte Carlo sampling will be used.

3 The probabilistic risk assessment method as applied to the Rhine–Meuse delta

3.1 Overview

As explained in the introduction, we need a method which takes into account hydrodynamic interaction, which facilitates the analysis of effects of alternative flood protection levels, and which includes both the tidal and non-tidal area to obtain insight in societal flood fatality risk in the Rhine–Meuse delta.

Our method is based on Monte Carlo Simulation (MCS) (similar to Vorogushyn et al., 2012 and Van Mierlo et al., 2007). However, to speed up the simulation an advanced importance sampling technique is used and the flood plain modeling is simplified. In the MCS, the hydraulic loads, strengths of flood protection and response variables are sampled and the hydrodynamic interaction is simulated with an efficient 1-D model, in which the flood plains are schematized as reservoirs. The water levels in the reservoirs are not used to assess flood impacts. Instead, for locations where breaches occur, existing 2-D model results are taken from a database with pre-simulated flood scenarios. The flood patterns were translated to fatality figures with the adapted version of the mortality functions of Jonkman (Jonkman, 2007; Maaskant et al., 2009a). We include hydrodynamic interaction and take into account uncertainties in loads, strengths, and evacuation success rates.

A schematic overview of the probabilistic risk assessment method is provided in Fig. 1. Besides pre- and post-processing of input and output data, it consists of three main steps, which are explained in the following sections:

1. sampling of the load, strength and evacuation response variables;
2. the hydrodynamic modeling of the sampled events to see where breaches occur;
3. the translation of the model outcomes to flood fatalities per breach location and per sample.

The most important outputs are FN curves, Potential Loss of Life (expected annual number of fatalities) for the area as a whole and the contributions of the three subzones (tidal, non-tidal and transition zone) to the total risk.

3.2 Schematisation and data requirements for the Rhine–Meuse delta

The case study of the Rhine–Meuse delta includes all main branches in the Rhine and Meuse delta between Lobith (along the Rhine at the German–Dutch boundary), Lith (along the Meuse), Maasmond and the IJssellake and the surrounding flood-prone areas (see Fig. 2).

To model potential dike breaches and their effects, 171 potential breach locations have been defined (see Fig. 2). Each of these locations is representative for a dike stretch with a length varying from 400 m to about 34 km. Fragility curves and river water level statistics are required for each potential breach location. The shape of the fragility curves determines the range of water levels at which failure is most likely to occur. Details on these are provided in the following sections.

The probabilistic framework is used to assess the societal flood risk for different candidate sets of safety levels. The safety levels can, therefore, be defined as input per dike section. A dike section can contain one or more potential breach locations.

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The fragility curves of the potential breach locations are shifted in an iterative procedure to ensure that the failure probabilities of each dike section correspond with the user-defined safety levels. In these user-defined failure probabilities the effect of hydrodynamic interaction is not taken into account. The shape of the fragility curves are not altered, only the mean is shifted. The shape mainly depends on the dominant failure mechanism (see De Bruijn and Diermanse, 2013a).

3.3 Sampling load, strength and evacuation response parameters

The probabilistic method is based on Monte Carlo Simulation (MCS) with importance sampling for load, strength and evacuation fraction. The MCS method was chosen because of the large number of breach locations for which dike strengths need to be sampled. Other probabilistic techniques, such as numerical integration or FORM, are less efficient for large numbers of random variables (De Bruijn and Diermanse, 2013a; Diermanse et al., 2014).

3.3.1 Sampling load parameters

Flooding in the Dutch Rhine and Meuse delta is caused by storm surges and by extreme discharges from the upstream catchments in Germany and Belgium. In the tidal river area, floods are not only linked to the storm surge severity, but also to the functioning of the storm surge barrier near the Maasmond, the Maeslant Barrier. The Rhine delta can be subdivided in three areas based on the event type which contributes most to the flood risk (see Fig. 2):

1. the river-dominated area, which may become flooded due to river flood waves in the Rhine or Meuse River;
2. the tidal area, where the influence of storm surges is dominant: in this area, flooding may occur when a storm surge raises the local water levels and the storm surge barrier fails to close;

3. the transition zone: the area where flooding may occur due to a combination of a storm surge and a high river flood wave at the same time.

To obtain accurate flood risk estimates, flood events in all three area types must be represented well. For each event, four variables are sampled in the MCS, which together determine the hydraulic load: the discharge of the Rhine River at Lobith, near the German border, the discharge of the Meuse River at the upstream location Lith, the sea water level at the Maasmond and the functioning of the Maeslant Barrier.

To increase the efficiency and accuracy of the Monte Carlo analysis, importance sampling is applied to each hydraulic load variable. For each simulated year two types of events are sampled:

- an extreme river discharge from the annual maxima distribution and a coincident sea water level sampled from the distribution of sea water level;
- an extreme sea water level from the distribution of annual maximum sea levels and a coincident maximum river discharge sampled from the daily river discharge statistics.

The general expression for the distribution function of annual maximum discharges of the Rhine and Meuse Rivers is provided in Eq. (1). The coefficients *a* and *b* are fitted to a series of 100 annual maxima that is corrected for anthropogenic and natural changes in the river bed over the years (De Bruijn and Diermanse, 2013a).

$$P(Q < q) = \exp\left(-\exp\left(-\frac{q-b}{a}\right)\right) \tag{1}$$

The fitted coefficients are $a = 1316.45\,\mathrm{m^3\,s^{-1}}$, $b = 6612.5\,\mathrm{m^3\,s^{-1}}$ for the Rhine River at Lobith and $a = 342.12\,\mathrm{m^3\,s^{-1}}$ and $b = 1190.34\,\mathrm{m^3\,s^{-1}}$ for the Meuse River at Lith. The daily discharges were sampled directly from an observational time series of approximately 100 yr. The river discharges of Rhine and Meuse are correlated. The correlation coefficient is equal to 0.6.

For the annual maximum water level, the conditional Weibull distribution is used to describe (non-) exceedance probabilities of annual maximum sea water levels at Maasmond (see Eq. 2).

$$P(M \leq m | m > \omega) = 1 - \exp[(\omega/\sigma)^{\xi} - (h/\sigma)^{\xi}] \qquad (2)$$

Where M is the annual maximum sea water level (in m above datum, in this case NAP), m is a potential realization of M and ω, σ, and ξ are the location, scale and shape parameter respectively. Exceedance *frequencies* of high sea water levels can be derived by multiplying the probabilities that follow from Eq. (2) with the frequency of exceedance, λ, of threshold ω. The value of λ is determined by counting the number of peaks above the threshold and dividing by the number of years of record. Their values are in this case: $\omega = 1.97\,\mathrm{m} + \mathrm{NAP}$, $\lambda = 7.237\,\mathrm{yr}^{-1}$, $\xi = 0.57\,\mathrm{m} + \mathrm{NAP}$, $\sigma = 0.0157\,\mathrm{m} + \mathrm{NAP}$.

The daily sea water level conditions which are used to sample the sea water level coinciding with an annual maximum discharge, are derived from histograms of observed tidal peak water levels (Diermanse et al., 2014). It is assumed that the high river discharge wave lasts 12 tidal periods (approximately 6 days). This duration is taken into account in the probability distribution from which the tidal peak sea water level is sampled. A detailed description on the sampling techniques are provided in Diermanse et al. (2014).

The functioning of the Maeslant Barrier is also sampled for each scenario. There are two possibilities: if the barrier functions, it closes upon request, that is if the water level at Rotterdam exceeds 3 m + NAP. If a failing storm surge barrier is sampled, the barrier will not close. The probability that a non-functioning barrier is sampled is 1 % per event.

The efficiency of the MCS is enhanced by importance sampling. In this sampling strategy, the original probability distribution for the maximum sea water levels, $f(x)$, is replaced by a uniform distribution function $h(x)$. This leads to all extreme water levels being equally sampled. The same is done for the distribution of annual and daily peak discharges. The distribution of daily sea water levels is replaced by a composite

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function in which both peak water levels, and intermediate water levels are represented well. Finally, the sampling failure probability of the Maeslant Barrier was increased from 1 to 10 %, since especially the situations with a failing storm surge barrier contribute significantly to the flood risk in the western part of the tidal river area. Because the sampling does not use the actual distribution function, the estimator of the failure probability as applied in Crude Monte Carlo needs to be adapted, according to Eq. (3).

$$\hat{P}_\mathrm{f} = \frac{\sum_{i=1}^{N} I(Z(x))\frac{f(x)}{h(x)}}{N} \tag{3}$$

Where $\hat{P}_\mathrm{f}$ is the estimate of P_f, I is the indicator function ($I = 1$ if $Z < 0$, and 0 otherwise. Z = a limit state function which represents failure. Failure occurs when $Z < 0$). $f(x)$ is the probability density function, and $h(x)$ the sampling function. In the translation of the outcomes, the weights of the samples are thus corrected by the factor equal to $f(x)/h(x)$. Diermanse et al. (2014) discuss the sampling strategy in detail.

In the MCS, a total of 2000 events were sampled (2 sampled events for 1000 representative years). In order to translate each sampled peak value into a time-depended wave, a standard hydrograph shape was used (De Bruijn and Diermanse, 2013a). A standard sea water level hydrograph was derived from a combination of a standard tidal pattern and a storm surge hydrograph (see De Bruijn and Diermanse, 2013a). The river flood wave shape used is equal to the standard flood wave applied to calculate the official Dutch design water levels for flood defenses (Min. V&W, 2007).

3.3.2 Sampling strength parameters

In the Rhine–Meuse delta area, 171 potential breach locations were identified (see Fig. 2) which provide a good representation of the flood risks in the area (De Bruijn and Van der Doef, 2011). These were selected, based on potential flood consequences. If the consequences of breaches were expected to differ between two nearby dike stretches, a new potential breach location was added.

The uncertainty in the strength of the dike stretches is described by fragility curves. These curves describe the probability of a breach as a function of the water level (H). This probability is equal to 0 for low water levels and 1 for extremely high water levels. For intermediate water levels, the probability of failure increases from 0 to 1 with increasing water levels. The shape of the fragility curve depends on the dominant failure mechanism. In this research, overtopping, macro-instability and piping were considered the main failure mechanisms. At locations where piping is relevant, the fragility curve is rather flat, which means that failure can occur due to a wide range of water levels. Other factors, such as the duration of the high water level are also relevant for piping, which explains the wide shape of the fragility curve for this mechanism. If overtopping is dominant, the fragility curve is steep and there is little uncertainty about the water level at which failure will occur. The fragility curves were derived from the Dike Analysis Module (DAM) (Van der Meij, 2012). Samples of fragility curves between different locations are assumed to be uncorrelated, because most of the potential breach locations are well-separated. As a further development of the method, the correlation between strengths of nearby dike sections can be included in the sampling in the future.

For each event, a water level for all potential breach locations is sampled from the fragility curves. This level is represented in the hydrodynamic model by a trigger at which the defence will fail and a breach will occur (see Sect. 3.4).

3.3.3 Sampling evacuation response parameters

The expected number of fatalities as a result of a flood depends not only on the characteristics of the flooding scenario itself, but also on the population density and on the number of people that have been evacuated before onset of the flooding. The fraction of the inhabitants who can be evacuated depends on the time available and the required time, which differ between the three subareas (tidal, non-tidal and transition zone). In the probabilistic framework, the success rate of the evacuation, which varies between 0 and 90 %, is sampled from a probability distribution. Different distributions were derived for the three different regions: the non-tidal river area, the tidal river area and the transition area, based on Maaskant et al. (2009b) and also used in Beckers and De Bruijn (2011). Evacuation in the non-tidal riverine area is on average 75 %, while in the tidal river area it is on average 15 %. In the river-dominated area, an accurate flood forecast can generally be made multiple days in advance and the road capacity is large in comparison to the population density. In the tidal area, the forecast lead times are typically less than a day, while the population density is very high. This explains the small values for the evacuation percentages in the tidal area compared to the non-tidal area. For the transition area, the evacuation success is considered correlated with the flood threat: if the flood is caused by a high river discharge, the forecast lead time is usually longer which increases the probability of a successful evacuation. Therefore, in the transition area the evacuation percentage is set equal to the sampled value for the river-dominated area in those events which have a Rhine peak discharge higher than 12 000 $m^3 s^{-1}$ at Lobith. For other events, the evacuation percentage is set equal to the value sampled for the tidal area. The sensitivity of the resulting FN curve for the precise level of this threshold was found to be low.

3.4 Hydrodynamic model simulation

The sampled load and strength parameters are input for a hydrodynamic model, which simulates the water levels in the system. These water levels determine the locations where dike breaches will occur. At each simulation time step, river water levels are compared with the threshold water levels for breach initiation. If a threshold is exceeded, a breach is initiated and water will flow through the breach out of the river. Water is thus abstracted from the river, which leads to a reduction of downstream water levels.

A dike breach is simulated by opening the structure which represents the breach in the model. The height of the outlet is equal to the level of the area directly behind the dike. The width of the structure opening grows in time from zero at the time of breaching to a maximum of 200 m. This growth rate is based on Verheij (2003). In order to save computing time in the MCS, the inundation of the area behind the breach is simulated in a simple way (the area behind the breach is represented by a simple reservoir). This serves only to obtain realistic estimates of the retention effect of breaches. The simulated water levels in the flooded areas are not used to assess flood consequences. Instead, the number of fatalities corresponding with each breach location is pre-calculated in an accurate 2-D flooding simulation.

The time step used in the hydrodynamic simulations is one hour. A full simulation period covers one month. The most important boundary conditions are the discharge as a function of time at Lobith and Lith (Rhine and Meuse Rivers), the sea water level as a function of time (at Maasmond and the Haringvliet) and a water level–discharge relationship at the downstream side of the IJssel River branch, which flows into the IJssel Lake (see Fig. 2).

Two model schematisations have been used: one for those samples in which the Maeslant Barrier functions and one for those samples in which the Storm surge barrier does not close upon request.

The hydrodynamic simulations result for each sample in: the dike stretches where a breach occurred, the maximum discharges through the breaches and the maximum water levels in the river at the breach locations.

3.5 Translation of model outcomes to societal flood fatality risk measures

For those locations where breaches were calculated, the corresponding number of fatalities is taken from a database with pre-calculated flood scenarios and consequences. These fatality figures are all determined for breaches at design conditions and simulated with a 1-D2-D or a 2-D model (De Bruijn and Van der Doef, 2011). Since in the current approach defences may fail at conditions higher or lower than design conditions, the expected number of fatalities may be slightly higher or lower than the number corresponding with a breach at design conditions. However, the sensitivity of the number of fatalities for small variations in breach inflow is small compared to other uncertainties, such as the sensitivity to the estimate of the number of breaches, the breach locations, and the evacuation success rates. In future extensions of the method, it may be possible to replace the fixed fatality figures by a relationship between breach inflow and fatality figures.

A final step of the method is to include evacuation. The number of potential fatalities per breach location is multiplied by the fraction of inhabitants who were not evacuated. A list is then generated which gives, for each of the 2000 events and for each potential breach location the number of fatalities and the total number of fatalities in the event. This result is then used to construct the FN curve.

In the case study area, the number of fatalities due to a single dike breach at design conditions varies between zero and more than 3000. The highest numbers are expected due to breaches along the Lek and Nieuwe Maas near Rotterdam and just east of Rotterdam. At those locations, breaches may result in extensive flooding, large water depths and steep water level rise rates. Furthermore, these dike sections protect densely populated areas and the evacuation possibilities are limited there due to the short forecast lead time of storm surges. In the non-tidal area, the evacuation possibilities are generally better and the population density is lower. Consequently, the number of fatalities here is lower.

4 Results for the Rhine–Meuse delta

The results presented here are carried out with a set of fragility curves that were tuned to match the current protection standards.

The efficiency of the MCS importance sampling was tested first on a simplified system and based on the accuracy of the water level statistics. The results of consecutive MCS runs showed very small variations in the water level (standard deviations of less than 6 cm for the 1 : 1000 yr water level) at all locations in the system (171 locations). Secondly, the risk convergence and convergence of the exceedance probabilities of N fatalities was analysed for $N = 10$, 100, 500 and 1000) and proved to be acceptable (see Fig. 3).

The contribution of the three subareas in the delta to the flood fatality risk is not equal: Fig. 4 shows that the upper river area contributes most to the FN curve up to 8000 fatalities. For events with more than about 8000 fatalities the tidal river area is contributes most. In the tidal river area floods are rare, but potentially catastrophic. In that area many breaches may occur during a single storm surge event, especially if the storm surge barrier fails to close.

4.1 Hydrodynamic interaction

Figure 4 shows the resulting FN curves for systems including and excluding outflow to inundated areas, thereby indicating the effect of hydrodynamic interaction on the FN curve. The effect is large, especially for high values of N. The probability of events with 1000 or more fatalities decreases from about $1/1000$ to $1/500$ if hydrodynamic interaction is taken into account. The number of fatalities which is exceeded with a probability of $1/10\,000$ a year decreases from 10 000 to 3000.

The effect of hydrodynamic interactions is also apparent from the number of breaches per event (Fig. 5). A dike breach reduces the water levels downstream and thereby the probability of additional breaches. If this alleviating effect is not considered, the number of breaches is much larger. In the upper and tidal areas, the reduction of the number of breaches due to hydrodynamic interaction is about 80 %. In the transition zone it is about 70 %. If hydrodynamic interaction is taken into account, events with more than 20 breaches are very rare. In the non-tidal area, a river flood wave with a peak value of about $16\,000\,\mathrm{m^3\,s^{-1}}$ at Lobith (the current "design discharge" for the flood defences in that area) causes 2 to 5 breaches.

4.2 Indicators derived from the FN curve

Several risk indicators can be derived from the FN curve, including the probability of an event with at least one fatality, the potential loss of life (PLL), the probability of more than 100 and 1000 fatalities and the number of fatalities associated with a probability of 10^{-6} (see Table 1).

The probability of a flooding with at least one fatality is about 1/70 per year. The highest flood probability of a single potential breach location is 1/320 (at Heerewaarden). The PLL, or annual expected number of fatalities, is about 3. The number of fatalities corresponding to a probability of exceedance of 10^{-6} is 8400 for the whole area. In the river dominated area, the probability of high numbers of fatalities is very small. No events with more than 10 000 fatalities were identified. However, the probability of events with 10 to 100 fatalities is higher than in the tidal and transition area (see Table 1).

In flood risk management in the Netherlands, the C value is often used as a measure for societal flood risk (Vrijling et al., 1995; Bedford and Cooke, 2001). This value is found by plotting a tangent of the FN curve with a slope of minus 2 on a logarithmic scale. The C value is then equal to the value where this line crosses the line $N = 1$. The slope of minus 2 implies a risk-averse policy: events with 1000 fatalities are considered 100 times more serious than events with 100 fatalities. In some other countries, a risk-neutral approach is used, reflected by a line with a slope of minus one. Although the use of the C value as a tolerable risk indicator is still under discussion, and no decision on a tolerable level has been made, 1100 is frequently mentioned as a reasonable value for the Netherlands. This value is based on a comparison of different types of risks (man-made or natural, voluntary taken or imposed on inhabitants without their consent) and tolerance levels for those risks (Vrijling, 1995). The discussion is still on-going.

In the case study of the Rhine Meuse delta, the C value of the FN curve is determined by flood scenarios with more than 1000 fatalities. To reduce the C value, the number of fatalities of scenarios with 1000 or more fatalities must be reduced. The scenarios which contribute most to the C value are scenarios with a high Rhine discharge and unsuccessful evacuation. These scenarios often include several breaches in the central river area along the Lek River. In general, these high discharge events will be forecasted days in advance. However, it may happen that levees fail before the evacuation is completed. A future extension of the method could include a correlated sampling of the evacuation success rate and dike strength and/or river discharge.

5 Conclusions

The method presented in this paper combines a hydrodynamic model with a probabilistic framework. It includes a Monte Carlo simulation with importance sampling of external hydraulic forcing, fragility curves of the levees and of evacuation scenarios. The sampled events are simulated by a 1-D hydrodynamic model and combined with pre-simulated 2-D model results to obtain fatality figures. Finally, an FN curve is derived, which quantifies exceedence frequencies of numbers of fatalities. The combination of probabilistic and hydrodynamic model-based risk analysis provides insight in the hydrodynamic interactions within the system. The method is not computationally demanding and thus allows for the analysis of the whole river delta and to analyse risks corresponding with various flood risk management strategies.

The method produced FN curves and various flood risk indicators as well as contributions of subareas or even individual dike sections to the overall flood risk. The results clearly show the relevance of taking hydrodynamic interaction into account in risk assessment. The flood risk including interactions is much lower than assessed when hydrodynamic interaction is not taken into account. Using the method helps to get a better understanding of the system and its spatial interdependencies.

The results show that the discharge dominated part of the river area contributes most to the societal flood risks. Only for events which cause 8000 fatalities or more, the tidal area contributes most.

The method enables to evaluate flood protection or mitigation measures in terms of societal flood fatality risk reduction. The method has already been used to analyse strategies involving dike strengthening, improving the failure probability of the Maeslant

Barrier and improving evacuation and other emergency response measures (De Bruijn and Diermanse, 2013b; Klijn et al., 2013).

6 Discussion and recommendations

6.1 Discussion

The FN curve presented in the previous section provides insight into the level of the societal flood risk if the flood protection would meet the current standards. It is not possible to draw conclusions on whether the societal risk is tolerable or too high, since no standards for societal flood fatality risks exist yet. It is, however, possible to determine which regions or dike sections contribute most to the FN-curve. The results in the previous section show that the discharge-dominated part of the river area contributes most to the societal flood risk. Only for events that cause 8000 fatalities or more, the contribution of the tidal river area is higher.

It is also possible to use the FN curve to target risk reducing measures. If, for example, the probability of more than 10 000 fatalities is considered too high, the measures should be targeted on functioning of the Maeslant Barrier or on the dike sections near Rotterdam, where the highest numbers of fatalities are found. On the other hand, if the probability of events with 1000 or more fatalities needs to be reduced, the central river area should get most attention. In all cases, improving the probability of a successful evacuation could be an alternative to raising or strengthening of flood protection. The flood fatality risk estimates are currently being used for developing flood risk management strategies (Van der Most et al., 2014).

The method that was developed has acceptable computation times (9 h for a full Monte Carlo analysis of 2000 samples on an Intel core i7 64 bit, 16 GB RAM). The Monte Carlo approach has the additional advantage that probabilistic results can be exemplified by flooding scenarios.

The method can be applied to flood risk assessment in other deltas. It is especially suitable for large river deltas with a developed infrastructure of flood defences where hydrodynamic interaction is important. For those areas the method presented here has advantages over other methods.

6.2 Further research

The probabilistic framework as described in this paper will be further developed in the near future. We identified three lines of research to further improve the method: (1) to better incorporate the distribution of the flood fatality figures given a breach at a certain location, (2) to further study the effect of hydrodynamic interaction on flood risks and (3) to take into account a correlation between, on the one hand, the evacuation rate and, on the other hand, the dike strength, peak river discharge, and the functioning of the Maeslant Barrier. These topics are discussed in more detail below.

1. In the current method, uncertainties in the number of fatalities corresponding with a breach are not considered. We tested the sensitivity of the FN curve to the uncertainty in the fatality figures given a breach and this sensitivity was small (see De Bruijn and Diermanse, 2013).

2. Since the effect of hydrodynamic interaction is significant, we will in future improve the study of the influence of several variables which affect hydrodynamic interaction. For example, we currently use a fixed "average" shape of the discharge wave for both the Rhine and Meuse Rivers. However, the shape of the flood hydrograph may vary and could be treated as an additional random variable. It is expected that the effect of hydrodynamic interaction on flood probabilities is larger for steeper river flood waves.

   Other possible random variables are the breach growth rate and the moment of breaching. In the current method, failure only depends on the instantaneous water level and not on the duration of a sustained high water level. In the current approach, dike failures will therefore, always occur if a critical water level is exceeded. This means that levees will always fail at or before the peak of the discharge wave. In reality, they may also fail when the peak of the discharge wave has already passed. The effect of a breach on the load of downstream locations is less when the breach occurs after the top of the discharge wave then if it would occur during the rise of the discharge wave.

3. Finally, it is recommended to investigate the effect of correlations between the evacuation success rate and the suddenness of the flooding. The evacuation fraction of the non-tidal area is currently sampled without any consideration of the discharge or the dike strength. Floods caused by extreme discharges are more likely to be forecasted well in advance. The probability of a successful evacuation is then larger. The correlation between evacuation in the tidal river area and the functioning of the Maeslant Barrier could also be added: if the Maeslant Barrier fails, water levels may unexpectedly rise fast and the available time for evacuation will be limited. A failure of the Maeslant Barrier to close will not be forecast and will only be known at maximum 6 h prior to the peak water level (at the change of the tides).

Finally, more research is needed on how to use FN curves in policy making: on what may be considered acceptable and for what reasons and on methods or guiding principles to translate risk outcomes to measures.

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Table 1. Values for Indicators derived from the FN curve: for the three subareas and the total area.

Indicators	Tidal	Upper river	Transition	Total area	Total (no hydrodynamic interaction)
$P\ (N \geq 1)$	1.36×10^{-3}	1.01×10^{-2}	3.03×10^{-3}	1.36×10^{-2}	1.4×10^{-2}
$P\ (N \geq 100)$	6.65×10^{-4}	3.67×10^{-3}	8.33×10^{-4}	4.87×10^{-3}	6.22×10^{-3}
$P\ (N \geq 1000)$	7.1×10^{-5}	8.71×10^{-4}	6.27×10^{-5}	1.05×10^{-3}	2.06×10^{-3}
$P\ (N > 10\,000)$	4.1×10^{-8}	–	–	2.89×10^{-7}	9.39×10^{-5}
$N\ (P \leq 10^{-6})$	7600	5800	3000	8400	39 000
C (slope −2)	160	1360	91	1574	14 003
C (slope −1)	1.0	0.2	0.2	1.1	2.2
PLL	0.4	2.5	0.4	3.3	8.4

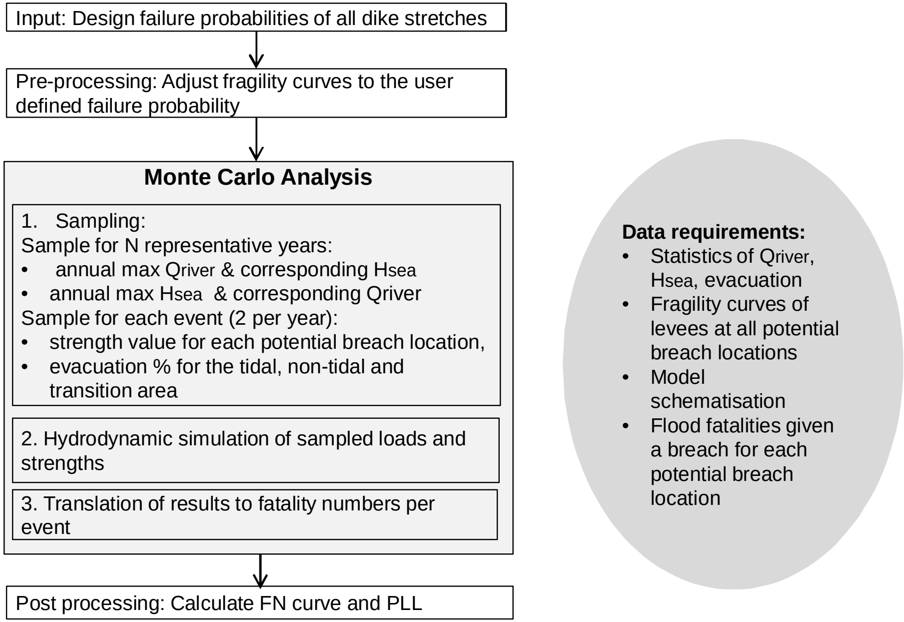

Fig. 1. Schematic overview of the probabilistic risk assessment method for river deltas.

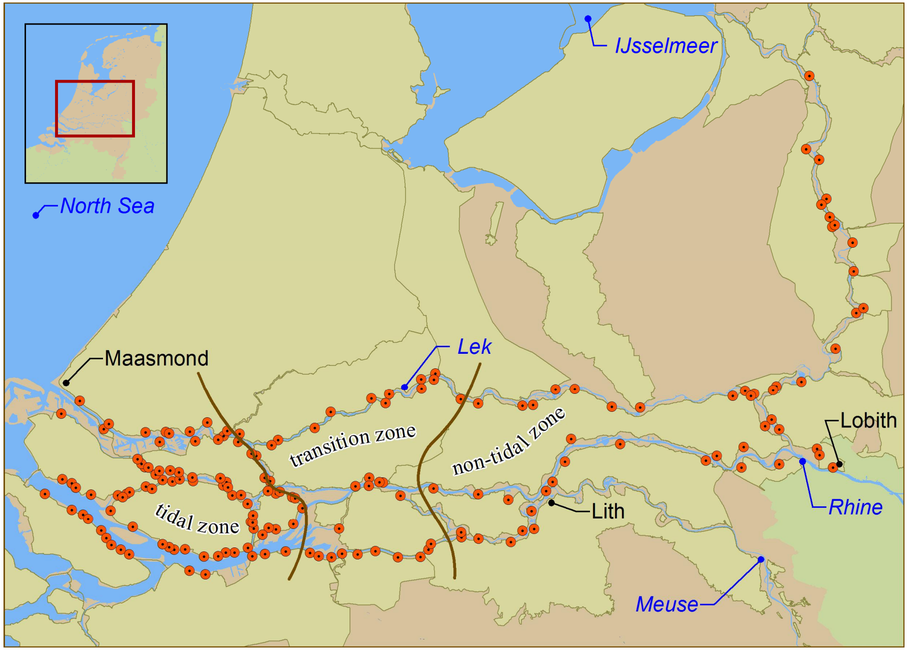

Fig. 2. Map of the Rhine–Meuse delta with the boundaries of the studied area (Lobith, LIth, IJssellake, North Sea), the three zones dominated by different flood types (tidal zone, non-tidal zone and transition zone), and the potential breach locations (red dots).

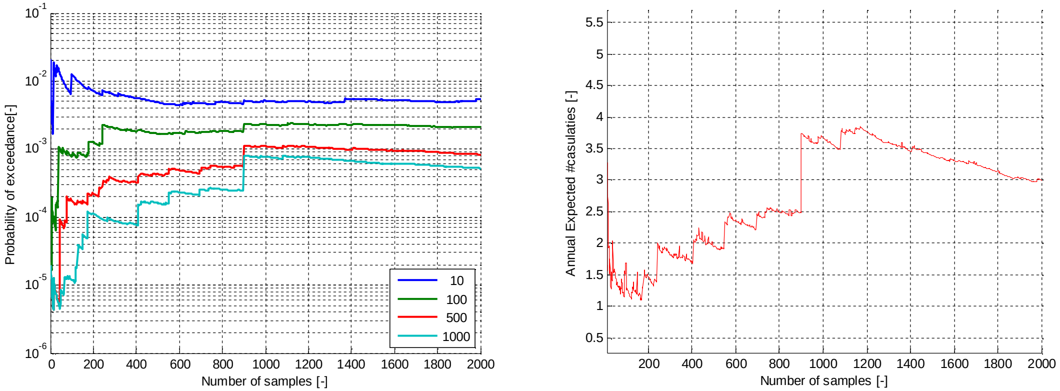

Fig. 3. Convergence of the probability of exceedance of events with more than 10, 100, 5000 and 1000 fatalities (left) and the annual number of fatilities (right).

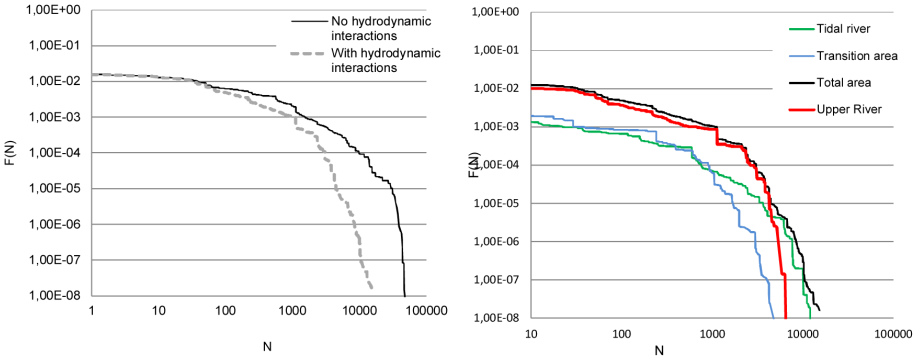

Fig. 4. Resulting FN curves. Left picture: FN curve of the delta determined in two ways: with and without considering the effect of hydrodynamic interaction. Right picture: contribution of the 3 zones to the total FN curve (with hydrodynamic interaction).

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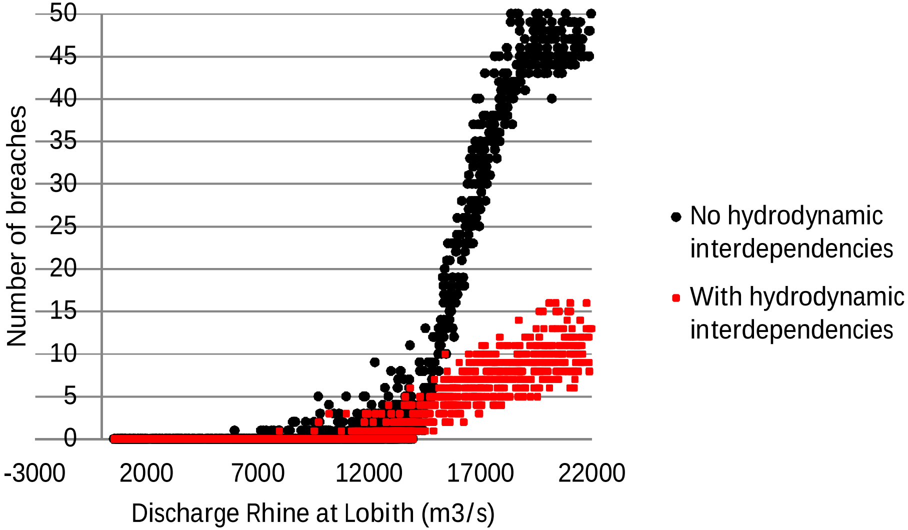

Fig. 5. Effect of hydrodynamic interaction on the number of breaches in the non-tidal part of the delta.