Assessment of flood risk accounting for river system behaviour

M.C.L.M. VAN MIERLO, *Senior Project Engineer, WL/Delft Hydraulic, PO Box 177, 2600 MH Delft, The Netherlands. Tel.: +31 15 2858793; Fax: +31 15 285 85 82; E-mail: thieu.vanmierlo@wldelft.nl; web:www.wldelft.nl*

A.C.W.M. VROUWENVELDER, *Professor of Civil Engineering, Delft University of Technology and Senior Project Leader, TNO Bouw, PO Box 49, 2600 AA Delft. E-mail: a.vrouwenvelder@bouw.tno.nl*

E.O.F. CALLE, *Research Manager, GeoDelft, PO Box 69, 2600 AB Delft. E-mail: e.o.f.calle@geodelft.nl; web:www.geodelft.nl*

J.K. VRIJLING, *Professor of Civil Enginering, Delft University of Technology, PO Box 5048, 2600 GA, Delft. E-mail: j.k.vrijling@citg.tudelft.nl*

S.N. JONKMAN, *DWW (Road and Hydraulic Engineering Institute) of the Directorate General of Public Works and Water Management (Rijkswaterstaat), The Netherlands*

K.M. DE BRUIJN, *Project Engineer, WL/Delft Hydraulic, PO Box 177, 2600 MH Delft, The Netherlands*

A.H. WEERTS, *Project Engineer, WL/Delft Hydraulic, PO Box 177, 2600 MH Delft, The Netherlands*

ABSTRACT
In this paper "river system behaviour" is defined as the complex interaction between river flow and the flooding of flood prone areas. A basic aspect of river system behaviour is that a local dike breach may affect hydraulic loads and hence dike failure probabilities at other locations. Important aspects in river system behaviour are discussed as well as the fact that effects of river system behaviour on flood risk may be both beneficial as well as adverse. This paper presents a conceptual approach to quantify effects of river system behaviour on probabilities of dike breach and flood risk. It was successfully applied to two example river configurations. The results of these examples are discussed. It is concluded that for proper flood risk assessment all relevant failure mechanisms, uncertainties as well as all proposed safety improvement measures are to be jointly taken into account. The conceptual approach enable all this. In the authors' views, there is a need for developing models that account for effects of river system behaviour on flood risk. Such models can serve as a tool for *policy makers* in evaluating the effects that (regional) safety improvement measures have on the flood risk in the entire river basin.

Keywords: Failure mechanisms; flood damage; flood modeling; flood risk; Monte Carlo Analyses; policy making; system behaviour; uncertainties.

1 Introduction

In this paper a methodology or *Conceptual Framework* for quantifying the effects of river system behaviour and interaction between dike ring areas on flood risk is discussed. Flood risk is defined as the product of probability of flooding and its consequences. River system behaviour is defined as the complex interaction between river flow and the flooding of flood prone areas. Due to the effects of system behaviour flood risk (or safety) of a particular area may depend on the safety of other areas. These effects can be considerable and can be beneficial as well as adverse, considering the flood risk in the entire river basin. Effects of river system behaviour are not yet being considered in the current Dutch flood protection strategy.

The conceptual framework was to be generic, meaning applicable for any hydrological system and capable of considering any kind of safety improvement measure. Hence, applicable to a complex hydrological system like the delta area of rivers Rhine, Meuse and Scheldt in the Netherlands. Furthermore, capable of considering safety improvement measures such as increasing river discharge capacity, temporarily storing flood water, heightening of embankments and so on. Moreover, the conceptual framework should serve as a tool for *policy makers* in making basin wide safety analysis, meaning that for a particular regional improvement measure not only its local safety consequences, but also its safety consequences for the entire river basin can be determined. As such it can contribute to better informed and more rational decision making regarding safety improvement measures taken in the river basin.

In developing the conceptual framework, an inventory was made of all relevant aspects for quantifying the effects of river system behaviour on flood risk. On basis of this inventory,

the conceptual framework was divided into an *Institutional Framework* and a *Computational Framework*. The institutional framework should deal with societal aspects as well as policies and decision making aspects. Only limited attention was paid to institutional aspects and the institutional framework as such was not yet elaborated. The computational framework should allow the assessment of flood risks. In the project a computational framework was developed and its suitability was evaluated for two example river configurations. The focus of this paper lies on the developed computational framework.

2   Current flood risk protection in the Netherlands

The Netherlands are located in the delta area of rivers Rhine, Meuse and Scheldt. The larger part of the Netherlands is prone to flooding from the North Sea, the major rivers and the IJssellake. The Flood Protection Act (Wet op de Waterkering) of 1996 forms the legal framework for the protection of flood prone areas in the Netherlands. The law designates in total 99 so called primary dike ring areas. A dike ring area is an area surrounded by dikes and high grounds, which provide protection against flooding. Flood protection is a shared responsibility of regional and national authorities (water boards, provinces and national government).

The current Dutch flood protection strategy is based on the analysis of single dike sections. Presently the possibility of flood risk analysis based on dike ring areas is being investigated (VNK, 2003). Interactions between various dike ring areas, being a basic aspect of river system behaviour, is not yet being considered. The dike sections are dimensioned based on a design water level and design wave height with certain probabilities of exceedance (DG RWS *et al.*, 2001). The protection level targets differ by dike ring area. Upstream dike ring areas close to the eastern border with Germany have a design water level with a return period of 1,250 yrs. Dike ring areas in the upper part of river Meuse, however, have a return period of 250 yrs only. Dike ring areas situated further downstream and in coastal regions have higher protection levels (i.e. return periods) varying from 2,000 to 10,000 yrs. These protection levels are based on the economic value of the areas and expected damage when flooded. The corresponding design water levels are determined on basis of the hydrodynamics of the river system and the probability density functions of sea levels and upstream river discharges. Therefore, in practice in the determination of design water levels, the considered effects of system behaviour are constrained to the propagation of tidal waves and flood waves in the river network. At Lobith upstream discharges of 18,000 m^3/s are, however, considered in the determination of downstream design water levels. Presently, such high discharge will lead to large flooding in the upstream areas. Such high discharge will, therefore, become greatly attenuated downstream leading to lower discharges. Hence, in this respect effects of river system behaviour are ignored.

Resuming, the current Dutch method in determining flood risk does not account for effects of river system behaviour as discussed in this paper. It is mentioned that in some studies different aspects of system behaviour were discussed. Studies like: VNK, 2003; Hao-Ming Zhou, 1995; Silva *et al.*, 2001; IRMA Living with floods, 2001; Commissie Luteijn, 2002; De Jager, 1998; PICASO, 2001; Spankracht Study, 2002; Tillie, 2001; and Vermeij, 2001.

3   Description of effects of river system behaviour on flood risk

River system behaviour is defined as the complex interaction between river flow and the flooding of flood prone areas. Flooding might occur due to the overtopping or failure of flood protection works, the manipulation or failure of flood regulating structures and so on. A basic aspect of river system behaviour is that local failure has consequences for the hydraulic loads and hence failure probabilities at other locations. Due to effects of system behaviour the flood risk (or safety) of a particular area may depend on the safety of other areas. These effects can be beneficial or adverse on the flood risk in the entire river basin. An example of a driving mechanism in system behaviour is the interaction between geotechnical/ structural failure of flood protection works and the response of the river system to these failures. An example of a *beneficial effect* of river system behaviour is the reduction of water levels in downstream areas due to upstream dike failures. As a result of these dike failures, a part of a flood wave will be (temporarily) stored in upstream flooded areas. This storage of flood water will result in reduced peak discharges and water levels in downstream located areas. So the safety of these downstream areas will be increased. An example of an *adverse effect* (Figure 1) is the undesired possible failure of the left Waal dike near Weurt. Except for a flooding of dike ring 41 (Land van Maas en Waal), this Waal dike failure may also result in an overtopping and hence failure of the right Meuse dike near Alphen.
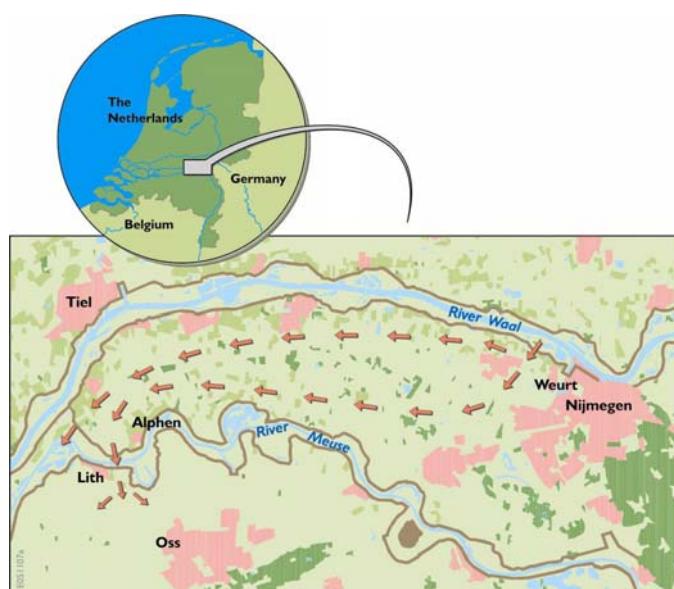

Figure 1   Example of *adverse effect* of river system behavior (unwanted divertion of flood water from river Waal to river Meuse in the Netherlands).

The latter dike failure will result in extremely large discharges and high water levels on river Meuse. This hydraulic phenomenon was analyzed in detail in the PoldEvac project (Van Mierlo *et al.*, 2001). Please note that usually flood waves on rivers Waal and Meuse coincide, while discharges on river Waal are much larger than on river Meuse. Peak discharges with a return period of 1,250 years on river Waal and Meuse, respectively amount to 10,000 and 3,600 m^3/s. Furthermore, corresponding flood levels on river Waal are about 2 m higher than the flood levels on river Meuse. Van der Wiel (2003) demonstrated that indeed adverse effects of river system behaviour may occur for an example river configuration, that is in line with the above described river Waal and Meuse situation.

Society either may or may not accept the current system behaviour within a river basin and its associated safety standards against flooding. The situation that society accepts the current system behaviour is defined as *passive interference*. Human activities in a passive interference strategy might refer to maintaining dikes and storm surge barriers only. In *active interference* a distinction can be made between interference during the actual occurrence of high river levels and structural flood prevention management. An example of the former is the diversion of flood water by so called green rivers (Vis *et al.*, 2003), aiming at reducing hydraulic loads for downstream located areas and hence increasing the safety of these downstream areas. An example of structural flood prevention management is the increase of river discharge capacity or the reinforcement of embankments. Both for active and passive interference measures yield that they may have beneficial or adverse effects on the safety of the entire river basin.

Due to river system behaviour, benefits of different safety improvement measures may not be superimposed on each other. Flood risk should be determined taking all safety improvement measures jointly into account. Moreover, due to system behaviour the probability of exerted hydraulic loads at a specific location may be correlated to the failure of embankments at other locations in the river basin. Hence, it is not straightforward to determine the probability of flooding due to various failure mechanisms at different locations on basis of the failure probability of each individual failure mechanism. Probability of flooding should, therefore, be determined by taking all those failure mechanism jointly into account.

4 Aspects in river system behaviour

Relevant aspects in analyzing the effects of system behaviour on flood risk are:

(a) *Hydraulic/hydrological aspects.* All relevant water related processes, which might result in a flooding should be considered. These include the propagation of tidal waves and flood waves in the river network, while accounting for structures, wind, local rainfall and possible flooding of adjacent areas. System boundaries should have autonomous boundary conditions. Hydraulic parameters are to be independent of the hydraulic state (e.g.

water levels and discharges) in the area of interest. The probability density functions of hydraulic parameters applied at system boundaries are to be free of effects of system behaviour in the area of interest. Furthermore, possible effects of upstream system behaviour are to be accounted for in the probability density functions of hydraulic parameters applied at system boundaries.

(b) *Geotechnical and structural aspects.* Failure of flood protection works as result of exerted hydraulic loads and actual geotechnical and structural strength is to be accounted for. The main potential failure mechanisms usually associated with dikes are overtopping, piping, inner and outer slope failure and erosion (Figure 2). The occurrence of a failure mechanism is considered likely to initiate a dike breach and subsequent breach growth, even though this is not always necessarily the case. A clear example of the latter is a sliding of the inner slope (Figure 2), which leaves a significant part of the dike crest intact. If no subsequent further erosion of the crest occurs, then a dike breach will be held off. This is called residual strength (Calle *et al.*, 2002 and Van Gent., 2002). In the examples discussed in Section 6, such residual strengths have been disregarded. When an initial dike breach is eroded by the flow, the extent of the final gap depends on the magnitude of the exerted hydraulic load and the geotechnical and structural properties of the dike (Visser, 1998; Van der Knaap, 2000; Verheij and Van der Knaap, 2002; Verheij, 2002). The erosion of an initial dike breach is an example of the driving mechanism in river system behaviour, e.g. the interaction between geotechnical failures of dikes and the hydrodynamic response of the river system to it.

(c) *Flood risk aspects.* Flood risk is defined as the sum of the products of all damages and associated probabilities. In other words flood risk is the result of all possible flooding scenarios their probability of occurrence and their associated impact on society. Floods might occur because hydraulic loads exceed design conditions, some structural elements are weaker than anticipated, or flood protection measures are not effected in time. There are many different types of uncertainties in flood protection. For proper flood risk assessment it is essential that all relevant uncertainties are taken into account. Vrijling (2002) provides an overview of uncertainties to be considered in the design of Dutch emergency storage areas. Floods may cause
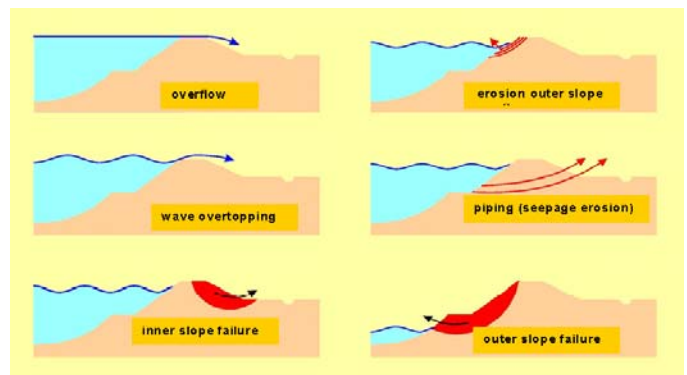

Figure 2 Main potential failure mechanisms usually associated with dikes.

loss of human lives and live stock, damage of buildings, industrial plants and infrastructure, loss of economic production and loss of cultural heritage. Environmental damage may occur due to the deposition of contaminated silt and toxic agents. The degree of flood damage depends on the number of inhabitants, the live stock, and the invested economic values as well as on the magnitude and forecast horizon of the floods. The forecast horizon determines how much time is available for evacuation, safeguarding economic goods and sealing off toxic depots.

(d) *Societal and institutional aspects.* The concept of river system behaviour must be embedded in decision making guidelines for policy making and crisis management. Consider the case that a particular dike ring area authority plans to strengthen a local dike, which may have consequences for the flood risk of neighboring dike ring areas as well. In this case the authorities of these other dike ring areas must be involved in the decision making process. Consider emergency storage areas, which are evacuated in a crisis situation for mitigating flood risk in downstream located areas. Such situation requires a specific organization and guidelines (Scholtes, 2002). The final decision is to be taken under time stress and immense pressure, while only limited information is available (Muller, 2003). In the Netherlands, decisions on evacuation are currently taken by local decision makers (e.g. the mayor). Active interference in system behaviour will probably shift this decision to a higher authority level. Communication on flood protection standards to society will become more complex due to interactions between different areas.

5   Outline of the computational framework

The applied computational framework for assessing the effects of river system behaviour on flood risk consists of three main components:

(1) *Accounting for driving mechanisms in river system behaviour.* The SOBEK software package was used for modeling the driving mechanisms in river system behaviour. Specifically SOBEK was used for modeling the interaction between river flow and the flooding of flood prone areas, dike failure mechanisms, and the growth of initial dike breaches. SOBEK includes 1D and 2D flow modeling (St Venant equations) and is capable of simulating the flooding of initial dry land (Dhondia and Stelling, 2004). Local rainfall can also be accounted for. Triggers and controllers can initiate a dike failure on basis of exerted hydraulic loads and a user defined failure mechanism. A failure mechanism concerns the evaluation of a failure algorithm on basis of exerted hydraulic loads and defined strength parameters. Triggers, controllers and failure mechanisms can be programmed in Matlab, allowing for any kind of failure mechanism to be incorporated. Dike breach development can be predefined as a relative function of time (Van der Knaap, 2000) or the Verheij and Van der Knaap (2002) breach formula can be used. The latter formula computes the dike breach development on basis of dike strength parameters and local exerted hydraulic loads.

(2) *Flood damage assessment.* A damage assessment model was developed using the GIS software package PC Raster (Van Deursen, 1995). Damage functions and potential maximum damages per land use were applied, that are in line with the so called HIS-SSM Module. Vrisou van Eck *et al.* (2000) and Vrisou van Eck *et al.* (2002) describe the HIS-SSM Module, which is commonly used in the Netherlands for assessing flood damage. The economic damage for each SOBEK 2D grid cell is determined as $ED_i = PED_i \cdot EDF_i \cdot A$, w*here*: ED_i = economic damage per grid cell; PED_i = potential economic damage per square metre, that depends on the actual land use in the grid cell; EDF_i = damage factor varies from 0 to 1, and is a function of the actual land use and the maximum flood depth in the grid cell; A = the area of the grid cell. The total damage in a particular area is the summation of the damage of all its grid cells.

(3) *Evaluation of flood risk, applying Crude Monte Carlo analysis.* A complete flood risk analysis should consider all flood prone areas within a river basin and reveals which flood scenario's can take place and evaluates their probabilities and impacts. The expression for a flood-risk calculation is given by:

$$R = E(D) = \int_{x \in P} D(x) f(x) \, dx \qquad (1)$$

Provided this integral exists. In equation (1), R denotes risk and E(D) denotes expectation of damage. Clearly, we adopt the definition of risk as the product of probability of an event and its adverse consequences as our basic concept. Further in equation (1), x denotes a vector of (realizations of) random variables, reflecting all of the uncertain parameters which play a role in flood risk analysis. This includes parameters which characterize and affect hydraulic loads, parameters which characterize resistance of the dikes against failure, parameters which affect the hydrodynamics of inundation after dike breach at one or more locations in the system, parameters which affect the consequential damage and, as far as relevant, parameters which characterize computation model uncertainties. The function $f(x)$ in equation (1) denotes the joint probability density function of the random parameter vector. Further, $D(x)$ denotes the damage, associated with the event that the random parameters x take on some specific realization within the parameter space P, for which the joint p.d.f. $f(x)$ has been defined. The main idea behind equation (1) is that once a specific realisation of x has been established, the process in time of river flow in the system area based on SOBEK calculations, the non occurrence or occurrence in time of dike breaches at one or more locations and, if so, the flooding and consequential damage, is completely deterministic. Consequently the system area wide flood damage associated with a specific realization of x is a deterministic quantity. It equals zero when no dike breach occurs, which is normally the case for most realisations of x. The influence of a dike breach at some location on the river levels at other locations within the system is, again given the realisation of x, also deterministic, since it is accounted for in the SOBEK calculation. Therefore, the influence of dike breach in one dike ring area within the system on the potential occurrence of dike breaches and consequential damage in other dike ring areas within the system is also deterministic for a given realisation

of x. Key components of x are the parameters which characterize the river discharge wave at the upstream boundary of the system area. The p.d.f. of these parameters are based on statistics of yearly extremes of river discharges. Hence, a realisation of the river discharge parameters refers to the extreme event in one year. Therefore the risk R reflects the flood risk per year, implicitly assuming that the probability of two or more timely separated flood events (e.g. in spring and in summer) in one year can be neglected.

A Crude Monte Carlo simulation technique was chosen for evaluating flood risk (Section 6). Such technique is simple and robust, but the computational burden involved is tremendous. However, for the try out of the two simple example river system configurations such brute force approach was considered acceptable. Nonetheless, we are aware that for more realistic systems, drastic improvement of the efficiency of the computational scheme is of prime interest. Some useful ideas, in this sense, have been suggested in literature (a.o. Dawson *et al.*, 2005 and Dawson and Hall 2006). Yet, efficiently establishing the correct influence of discharge through dike breaches, at one or more locations, on hydraulic head anywhere else in the system will be probably the most challenging issue.

The Monte Carlo approach proceeds as follows. For each dike ring area in the system fixed locations of potential breach were selected, based on heuristic reasoning. Standard deviations and probability density functions for the strength parameters were selected on basis of expert judgment. The same applies for the probability density function for the upstream peak discharge (Table 1). Sampling of these probability density functions resulted in a number of (Section 6) Monte Carlo scenarios (or runs) all representing a time span of one year. In principle each run has different values for the considered strength parameters as well as for the upstream peak discharge. For each run its magnitude of flooding was established using SOBEK. As mentioned previously, all possible physical interactions are automatically accounted for in the SOBEK computations. Furthermore, mutual correlation among different failure mechanisms is automatically accounted for as the evaluations of different failure mechanisms utilize the same realizations of flows and associated water levels. Suppose that two or more different failure mechanisms tend to become active at the same location and at the same time instance. Then, the sequence (defined in SOBEK) of evaluating possible

failure will not affect the occurrence of dike failure or the evaluation of flood risk. But, since the cause of dike failure will be attributed to the first mechanism in the computational sequence, it may affect to some extent the scores of relative contribution to dike failures attributed to the different failure mechanisms. All the possible physical interactions are accounted for in each computational time-step, which is automatically adjusted to ensure a proper simulation of the hydraulic phenomena involved. For Dutch river conditions, time-steps in 2D and 1D2D SOBEK computations usually amount to a few seconds only.

Thereafter, for each run, the possible flood damage was determined using the above described flood damage assessment model. The final result of the Monte Carlo analysis is a series of computed scenarios (or runs), each representing a yearly extreme event. For each Monte Carlo scenario it is known which areas were flooded as well as the associated flood damage. An estimator of the probability of dike failure at one of the fixed locations equals the number of Monte Carlo runs in which a dike failure occurred at that location, divided by the total number of Monte Carlo runs. Similarly, the flood risk of each area is estimated as the summation of the flood damage, computed in each of the Monte Carlo runs divided by the total number of runs, i.e.:

$$R_a = \frac{1}{N} \sum_{i=1}^{N} D_i \qquad (2)$$

Where: R_a is the annual flood risk; N the number of Monte Carlo runs and D_i the damage in Monte Carlo run i (if a run does not lead to a dike breach, D_i is equal to zero). The fact that equation (2) may be used for estimating flood risk can be justified as follows. In the case of estimating a probability of dike failure each of the Monte Carlo runs returns the value 0 or 1, zero denoting no failure and 1 denoting failure. Suppose the probability of failure is p. Then the mathematical expectation of the result of one Monte Carlo run equals p, since the probability of the outcome 1 is p and the probability of the outcome 0 is $(1-p)$. Adding up the outcomes of N Monte Carlo experiments yields, therefore, in expectation Np. So the sum of the actual outcomes of the experiments, divided by the number of experiments is an estimator for p. Now consider, similarly, an outcome of Monte Carlo experiments in terms of flood damage. Suppose the probability of flood damage D_k is p_k $(k = 1 \ldots n)$. Then the flood risk is: $\Sigma p_k D_k$ which we write as pD, where $p = \Sigma p_k$ and $D = (\Sigma p_k D_k)/p$, assuming

Table 1 Probability density functions for considered stochastic parameters

Stochastic parameter	Probability density function	Mean	V = Coef. of variation σ = Stand. deviation
Critical water level for overtopping (m)	Normal	12.12[1]	$\sigma = 0.3$
Critical head difference across the dike for piping (m)	Lognormal	4	$V = 0.125$
Critical duration for piping (hrs)	Lognormal	6	$V = 0.33$
Duration of breach growth (hrs)	Lognormal	40	$V = 0.5$
Final breach width (m)	Lognormal	100	$V = 0.5$
Peak discharge (m³/s)	Exponential	2640	$\sigma = 1810$

1. The critical water level for overtopping varies per dike breach location and corresponds to a discharge with a return period of 90 years (Peak discharge = 9.000 m³/s).
2. Coefficient of variation is equal to mean divided by the standard deviation.

$p > 0$. Then the expectation of the outcome of one Monte Carlo run equals pD. So the expection of the sum of outcomes of all Monte Carlo experiments equals NpD. Consequently, the sum of actual outcomes of the experiments divided by N is an estimator for pD, i.e. the flood risk.

6   Aim and scope of the examples

Two examples were elaborated in order to evaluate the suitability of the computational framework for estimating flood risk in a river basin where effects of system behaviour are of importance. Use was made of highly simplified river basin configurations in order to reduce computational burden. Furthermore, realistic parameters as well as parameter uncertainties were selected, considering the conditions along the river Rhine in the eastern part of the Netherlands. It is to be mentioned that among other things uncertainties in hydraulic model simulations (e.g. roughness, cross-sectional profiles) were not considered. Model uncertainties in the dike resistance are assumed to be incorporated in the scatter of the critical heads in Table 1.

6.1   Description of example 1

Model schematization: Example 1 concerns a basin comprising of one single river flowing towards the West (*Figure* 4, *except for the emergency storage area*). The river is bounded by a Northern and Southern dike ring area. River flow and possible flooding of dike ring areas were respectively computed as one and two dimensional flow. A downstream stage discharge relationship was imposed, ensuring uniform flow conditions all along the river for constant upstream discharges. Different flood waves were imposed at the upstream boundary. These flood waves are assumed to have one relevant peak discharge per year only. The duration of this peak wave is a few days only. Given the chosen schematisations, the duration of the peak wave is relevant only for the damage calculation, not for the evaluation of the piping or overtopping mechanism. The size, land use and potential flood damage for each area (*values for Southern area are, however, slightly different*) are given in Tables 3 and 4. All dikes have a crest level, which is 4.65 m above local surface level. This crest level coincides with a local river level, which corresponds to a discharge of 9,000 m^3/s. The river dikes are assumed to breach at predefined locations only (Figure 4). Breaches in the Northern area may occur at locations L1, L2 and L3. In the Southern area a breach may occur at location L4 only. The assumption of predefined locations was made for computational convenience. It is speculated that such simplification does not essentially disturb the true nature of river system behaviour characteristics in the considered highly simplified river basin configuration. Recent reseach (Van Mierlo, 2005), however, indicated that hydraulic consequences of river system behaviour in the Dutch part of the river Rhine delta are among other things strongly related to the actual location of a dike breach.

Stochastic parameters: An initial breach in a river dike can occur due to the overtopping and piping failure mechanism. Failure due to overtopping occurs instantaneously in case the water level in the river exceeds the *critical water level for overtopping* (i.e. the local crest level of the dike). Failure due to piping occurs when the *critical head difference across the dike for piping* is exceeded for at least a certain time lapse. This time lapse is defined as the *critical duration for piping*. An initial breach was assumed to develop, irrespective of the actual erosive flow passing through it. Two steps in breach development were discerned. In the first step, the dike is lowered to the local surface level for a constant width of 20m. In the second step, the dike breach grows linearly in width only over a time period T_{breach} (or *duration of breach growth*) till a *final breach width* is attained. The upstream flood wave represents a time span of one year and contains one *peak discharge* only. The definitions in italics are the six stochastic parameters considered. Their probability density functions are given in Table 1. The first five stochastic parameters are assumed to be valid for each dike breach location.

Computations: Different sets of Monte Carlo scenarios (or runs) were made for example 1 (Van Mierlo *et al.*, 2003). In this article only the results of three Monte Carlo sets are given for which both the overtopping and failure mechanisms were considered. The difference in the three reported Monte Carlo sets, respectively refers to considering the entire river basin, the Southern area only or the Northern area only. For these three reported Monte Carlo sets, the stochastic parameters (Table 1) were sampled 3,000 times. On basis hereof, 3,000 different Monte Carlo scenarios were made for each such Monte Carlo set.

6.2   Results of example 1

For the Monte Carlo set considering the entire river basin, the annual flood risk as function of successive Monte Carlo runs is depicted in Figure 3. *After each successive run n_i (= 1, 2, 3, . . . n_i, . . . N), the resulting annual flood risk was determined using Equation* (2). From this Figure, it can be observed that for this particular example, the flood risk stabilises after 1,500 Monte Carlo scenarios. Detailed analysis (Van Mierlo *et al*, 2003) showed that the set of 3,000 Monte Carlo scenarios provides a sufficient reliable flood risk estimate. The results of example 1 in

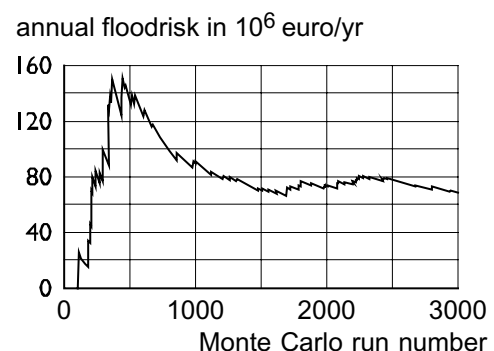annual floodrisk in 10^6 euro/yr

Figure 3   Example 1, Flood risk as function of successive Monte Carlo runs, Entire river basin, Overtopping and piping failure mechanisms.

Table 2 Example 1, No. of Failures and Flood risk, Stochastic parameters as given in Table 1

Overtopping and piping failure mechanisms; 3,000 scenarios per Monte Carlo set;
Number of failures and flood risk in 10^6 euro/yr

Monte Carlo set	Southern area		Northern area		Entire river basin	
	No. of failures	Flood risk	No. of failures	Flood risk	No. of failures	Flood risk
Entire river basin	28	8.9	59	60.9	60	69.8
Southern area only	42	20.2	–	–	–	–
Northern area only	–	–	59	61.4	–	–

which both the overtopping and failure mechanism were considered, are summarized in Table 2. Conclusions that were drawn from example 1 are:

(a) The number of failures (or flooding) for the Southern area reduces considerably when river system behaviour is taken into account. This can be explained as follows. The Southern area is flooded 42 times in case only the Southern area is considered and hence neglecting possible effects of river system behaviour. However, the Southern area is flooded only 28 times, when considering the entire river basin and hence accounting for possible effects of river system behaviour.

(b) The number of failures for the Northern area does not reduce when effects of river system behaviour are taken into account. This phenomen can be explained as follows. Dike breach locations L1 and L2 of the Northern area are situated upstream of dike breach location L4 of the Southern area (Figure 4). In case a breach occurs at location L1 or L2, the hydraulic load near breach location L4 will be reduced. As a result of this, the failure probability of the Southern area reduces. Apparently, for the *single* river of our example and flood waves dominated by upstream discharges only, downstream located areas benefit more from effects of river system behaviour than upstream located areas. Strictly speaking, this conclusion is valid for the configuration of the example and its specific parameter choices, i.e. equal safety against failure due to overtopping and piping in the potential dike breach locations. However, it is in line with expected results and likely a demonstration of a generic property of this type of river configuration.

(c) System behaviour has a *beneficial* effect on the overall flood risk in the river basin. This can be explained as follows. Considering the entire river basin, hence accounting for possible effects of river system behaviour, results in a flood risk of 69.8 M€/year. Considering the Southern area and Northern area separately, hence neglecting possible effects of river system behaviour, respectively results in a flood risk of 20.2 M€/year and 61.4 M€/year. Since a flood risk of 69.8 M€/year is less than the sum of 20.2 M€/year and 61.4 M€/year, river system behaviour has a beneficial effect. From a physical point of view, this beneficial effect can be explained as follows. In many scenarios the flooding of the Northern area resulted in such reduction of river
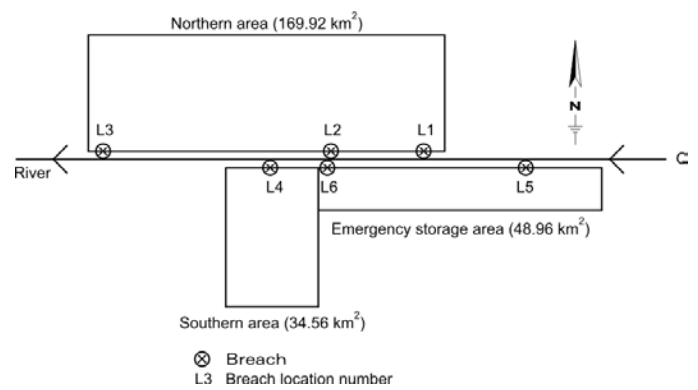

Figure 4 Example 2, Schematic layout of the river basin.

levels along the Southern area, that no breach occurred and hence the Southern area was not flooded.

(d) Additional sets of Monte Carlo scenarios (Van Mierlo *et al*, 2003) were carried out in which only the *piping* or only the *overtopping* failure mechanism was taken into account. From these computational results, it could be concluded that all possible failure mechanisms are to be considered *jointly* in order to determine flood risk correctly.

6.3 Description of example 2

Model schematization & stochastic parameters: Example 2 concerns a basin, depicted in Figure 4, comprising of one single river flowing towards the West. The river is bounded by an emergency storage area and a Northern and Southern dike ring area. The size, land use and potential flood damage for each area are given in Table 3 and Table 4. In order to optimize the use of the emergency storage area, its potential flood damage amounts to a negligible 77.9 M€only. The stochastic parameters were the same as used in example 1 (Table 1).

Deterministic design of the emergency storage area: The river dikes along the Southern and Northern area have a crest level, which is overtopped for discharges larger than 9,000 m^3/s. In accordance with the applied probability density function of upstream *peak discharges* (Table 1), this discharge of 9,000 m^3/s has a return period of 90 yrs. The use of the emergency storage area should ensure that river dikes along the Southern and

Table 3 Example 2, Land use and corresponding area

Area	Pastures 10^6 m^2	Forest 10^6 m^2	Residential areas 10^6m^2	Road 10^6m^2	Arable land 10^6 m^2	Total 10^6m^2
Northern area	94.88	38.72	21.92	3.36	11.04	169.92
Southern area	19.68	–	5.76	–	9.12	34.56
Emergency storage area	48.96	–	–	–	–	48.96

Table 4 Example 2, Potential flood damage in 10^6 Euro (i.e. damage factor $= 1$)

Area	Pastures 10^6 Euro	Forest 10^6 Euro	Residential 10^6 Euro	Road 10^6 Euro	Arable land 10^6 Euro	Total 10^6 Euro
Northern area	150.9	3.5	12,454.5	1.9	17.6	12,628.5
Southern area	31.3	–	3,272.7	–	14.5	3,318.5
Emergency storage area	77.9	–	–	–	–	77.9

Northern areas are not overtopped in case of a 260 years upstream design flood wave. This design flood wave has a peak discharge of 10,900 m^3/s. The emergency storage area was designed in a deterministic way. So no uncertainties in design parameters were considered. For the 260 years design flood wave sufficient inflow and storage capacity was provided for the timely storage of the necessary flood volume. More precisely, for storing the flood volume contained in the design flood wave by discharges larger than 9,000 m^3/s. By storing this upper part of the design flood wave, it was in a deterministic way ensured that discharges along the Southern and Northern areas should not exceed 9,000 m^3/s.

Operation of the emergency storage area: A fixed weir and an adjustable weir were applied for enabling the inflow of flood water into the emergency storage area. The fixed weir comprised the entire river dike along the emergency storage area. In this article only the use of the fixed weir is discussed. For information on the use of the adjustable weir, reference is made to Van Mierlo *et al.* (2003).

Water stored in the emergency storage area should not unexpectedly be released to the river as a result of the breaching of the dikes along this area. Breaching at locations L5 and L6 along the emergency storage area was, therefore, not considered (Figure 4). Of course, in the operation of the emergency storage area, breaching of the river dikes along the Southern and Northern area was considered. In the operation of the emergency storage area in total four breach locations (i.e. L1 up to L4) were considered. For all these four breach locations, the stochastic parameters as given in Table 1 were used.

A distinction was made in the number of failure mechanisms taken into account. Both the overtopping and piping failure mechanisms were considered in options a, while in options b only the overtopping failure mechanism was considered. For each option 3,000 Monte Carlo scenarios were made. In this article the results of two different Monte Carlo sets regarding the operation of the emergency storage area are given:

- *Monte Carlo set: Fixed weir a*: Inflow by means of a fixed weir. Considering both the overtopping and piping failure mechanism;

- *Monte Carlo set: Fixed weir b*: Inflow by means of a fixed weir. Considering the overtopping failure mechanism only.

Reference situation: The effectiveness of the operation of the emergency storage area was established by comparing flood risk in the river basin with and without controlled flooding. In the reference situation the emergency storage area is just used as an ordinary dike ring area (i.e. no controlled flooding). Hence, the river dikes along the emergency storage area may breach at locations L5 and L6 (Figure 4). So in the reference situation, in total six breach locations (i.e. L1 to L6) were considered. For all these six breach locations, the same stochastic parameters, as given in Table 1 were used. A distinction was made between considering both the overtopping and piping failure mechanisms (e.g. *option a*), and considering the overtopping failure mechanism only (e.g. *option b*). Resuming, two different Monte Carlo sets, each consisting of 3,000 scenarios (or runs), were made for the reference situation without controlled flooding:

- *Monte Carlo set: Reference a*: No controlled flooding, the emergency storage area just acted as an ordinary dike ring area. Considering both the overtopping and piping failure mechanism.
- *Monte Carlo set: Reference b*: No controlled flooding, the emergency storage area just acted as an ordinary dike ring area. Considering the overtopping failure mechanism only.

6.4 Results of example 2

As the emergency storage area was designed in a deterministic way, uncertainties were not considered in both dike strength parameters and the magnitude of upstream discharges. Results of example 2 are given in Table 5. An important conclusion that could be drawn from example 2 is that neglecting uncertainties in the design of an emergency storage area may yield incorrect indications of the flood risk. Neglecting uncertainties may, therefore, be misleading in evaluating such flood mitigating measure. This was demonstrated in our example as follows. *Firstly*, accounting for uncertainties makes the emergency storage area ineffective.

<div align="center">Table 5 Example 2, No. of Failures and Flood risk, Stochastic parameter as given in Table 1</div>

Monte Carlo set	Only Monte Carlo runs for the entire river basin were made							
	Southern area		Northern area		Emergency area		Entire river basin	
	No. of failures	Flood risk	No. of failures	Flood risk	No. of failures	Flood risk	No. of failures	Flood risk
Overtopping and piping failure mechanisms; 3,000 scenario's per Monte Carlo set; Number of failures and flood risk in 10^6 Euro/yr								
Reference a	40	14.5	57	48.2	57	1.0	73	63.7
Fixed weir a	39	14.1	58	48.3	26	0.4	66	62.8
Overtopping failure mechanism only; 3,000 scenarios per Monte Carlo set; Number of failures and flood risk in 10^6 Euro/yr								
Reference b	12	6.4	30	32.4	28	0.6	35	39.4
Fixed weir b	8	3.5	26	25.6	29	0.4	31	29.5

Secondly, by omitting one uncertainty (e.g. piping), the emergency storage area appears to become more effective. However, this phenomen concerns a spurious relationship. More details are given below:

(1) When considering both the overtopping and piping failure mechanism the controlled flooding of the emergency storage area is *ineffective*. Ineffective, because the operation of the emergency storage area does not reduce the number of failures of both the Southern area and the Northern area. For example, the Southern area fails 40 times (Table 5, Monte Carlo set: Reference a) in case of no controlled flooding. When the emergency storage area is operated by means of a fixed weir, the Southern area fails 39 times (Monte Carlo set: Fixed weir a). *Note that* the number of flooding of the emergency storage area reduces significantly as compared to the reference situation without controlled flooding. The reason is that in the reference situation breaches at location L5 and L6 occur (Figure 4), while in case of controlled flooding breaching at these locations is not considered. Also ineffective, since the operation of the emergency storage area does not reduce the flood risk in the entire river basin (i.e. the summation of the flood risk in the Southern area, the Northern area and the Emergency storage area). When the emergency storage area is operated by means of a fixed weir, the flood risk in the river basin amounts to 62.8 M€/year (Monte Carlo set: Fixed weir a). This flood risk is nearly the same as the flood risk of 63.7 M€/year for the reference situation without controlled flooding (Monte Carlo set: Reference a). The reasons why controlled flooding is ineffective is twofold. At first the fact that the uncertainty in upstream discharge was not taken into account in the deterministic design of the emergency storage area. In some scenarios, upstream discharges were larger than 10,900 m³/s, the assumed maximum upstream design peak discharge corresponding to a return period of 260 years. Secondly the fact that uncertainties in the strength of the dikes for the overtopping and piping failure mechanisms were not considered in the deterministic design of the emergency storage area. In some scenarios the strength of the dikes along the Southern and Northern areas was less than necessary to withstand a discharge of 9,000 m³/s. In the deterministic design of the emergency storage

area, this discharge was assumed to safely pass the Southern and Northern areas.

(2) When one uncertainty is omitted (e.g. the piping failure mechanism), the emergency storage area appears to become *more effective*. More effective, because the operation of the emergency storage area reduces the number of failures of both the Southern area and the Northern area. For example, the Southern area fails 12 times in case of no controlled flooding (Table 5, Monte Carlo set: Reference b). However, when the emergency area is operated by means of a fixed weir, the Southern area fails 8 times only (Monte Carlo set: fixed weir b). Furthermore it is more effective, because the flood risk in the entire river basin reduces due to the operation of the emergency storage area. A reduction from respectively 39.4 M€/year (Monte Carlo set: Reference b) to 29.5 M€/year (Monte Carlo set: Fixed weir b). Note that the fact that the emergency storage area becomes more effective concerns a *spurious relationship*. By omitting one uncertainty (i.e. the piping failure mechanism) the Monte Carlo solution becomes more deterministic of nature, and hence approaches towards its deterministic design conditions. In case all uncertainties would be omitted, only the emergency storage area will be flooded. The Monte Carlo solution would be in line with the deterministic design and the emergency storage area would be most effective.

From additional Monte Carlo scenarios results (Van Mierlo *et al.*, 2003), it was concluded that an emergency storage area may reduce the number of failures due to overtopping, but hardly limit the number of failures due to piping. Furthermore, in case the volume to be stored is much larger than the volume that can be stored, the exceedance of the critical head difference across the dike for piping will be elongated. This will result in an increase of the probability of a dike failure due to the piping failure mechanism.

7 Evaluation of the computational framework

Summarizing the examples: Both examples showed that the computational framework is suitable for determining flood risk in

river basins were effects of system behaviour are of importance. It was shown that failure mechanisms, proposed safety improvement measures and uncertainties should *jointly* be accounted for. Example 1 illustrated clearly that flood risk reduces (i.e. beneficial effect) considerably when system behaviour is accounted for. Furthermore, that for single rivers in general yields that downstream located areas benefit more from effects of system behaviour than upstream located areas. Note that an example illustrating adverse effects of system behaviour was not conducted in this study. Adverse effects of system behaviour were demonstrated by Van der Wiel (2003). Example 2, clearly demonstrated that leaving out uncertainties may yield incorrect indications of the flood risk and may, therefore, be misleading in evaluating flood mitigating measures.

Computational performance: The hydraulic simulations, including failure mechanisms and damage evaluations are by far the largest computational effort. Simple river basin configurations and for Dutch standards a small safety level of protections works (i.e. T = 90 yrs) were considered in both examples. This was done in order to limit the required number of Monte Carlo runs as well as the computational time per run. From a hydraulic point of view the flow was straightforward. Therefore, it was easy to select and omit beforehand those scenarios in which no flooding could occur. In this way the number of runs (for instance only 90 out of 3,000) could be reduced substantially. For complex river systems, eliminating scenarios beforehand will be much more difficult. Moreover, in both examples a straightforward Crude Monte Carlo analysis was performed. For complex river basin configurations, it will be worthwhile to adopt Importance Sampling Monte Carlo analysis (Dawson and Hall, 2006; Fishman, 1996) in order to reduce the number of required Monte Carlo scenarios. It is anticipated that in the near future, computational performance will become less restrictive due to improved computational techniques as well as improved computer performance.

Failure mechanisms: In both examples, simple formulations for the overtopping and piping failure mechanisms were used. More detailed failure analysis models can easily be incorporated into the computational framework. This, however, would further increase the computational burden. A tractable option to include (probabilities of) geotechnical and structural failure is the use of a response surface type approach (Waarts, 2000), which reduces computation time considerably. Inspired by this approach, suggestions for the refinement of failure mechanism were given (Van Mierlo *et al.*, 2003). Various different failure mechanisms depend on the same type of exerted hydraulic loads and the same soil parameters/process. Those failure mechanisms are, therefore, mutually correlated. Correlations due to the same exerted hydraulic loads are automatically accounted for as the evaluations of different failure modes utilize the same realizations (i.e. hydraulic computational results). Correlations, induced by soil parameters/processes, are more complex, either these correlations or the soil process (i.e. refinement of the failure algorithm) are to be included in the computational framework.

Breach development: In both examples it was assumed that after dike failure, the initial breach irrespective of the actual erosive flow, developed in accordance to a user defined time table. This implies that in case at two different dike sections at the same point in time a specific failure mechanism become active, both dike sections will fail and a breach will develop in accordance with the user defined time table. However, the development of a breach at one dike section may influence the development of a breach at another dike section. Nowadays, the Verheij and Van der Knaap (2002) formula can be used in SOBEK. In this formula the growth of a breach is a function of dike strength parameters and local hydraulic conditions. Hence, the interaction between breach growth processes at different dike locations can, nowadays, be accounted for. The Verheij and Van der Knaap formula is to be considered as a step forward towards improved formulations for breach growth. But, nevertheless it is anticipated that future research will lead to even more improved breach growth formulations.

Flood damage assessment: In the examples only economic damages were considered. The possibility of casualties was not considered, since it was assumed that there was ample time for the evacuation of local inhabitants. Furthermore the consequences of the spread of contaminated silt and toxic agents was not considered, since at the time of conducting both examples, the two dimensional water quality model in SOBEK as well as a tool for assessing damage due to contaminated silt and toxic agents was not available. Furthermore, other types of damage were neglected, such as: the indirect economic effects, damages to environmental and cultural values, and the societal impact.

Suitability of the Computational framework: It can be stated that the computational framework enables the assessment of the effects of system behaviour on the flood risk in a river basin due to both passive and active interference by mankind. In assessing flood risk several different type of safety improvement measures, different failure mechanisms as well as various type of uncertainties can *jointly* be taking into account. This ability is a *prerequisite* for the proper assessment of flood risks within any river basin. Furthermore the Computational framework was *generic* enough to conduct both examples. It is also considered generic enough to analyze the consequences of any kind of flood protection measure on flood risk. For example consider the construction of a local compartment dike that divides a dike ring area into two separate areas. This would mean the incorporation of this compartment dike in the existing hydraulic model schematization and the adding of relevant failure mechanisms and uncertainties with respect to the compartment dike. After running the model, the consequences of the proposed compartment dike on the flood risk in the concerned area as well as in the entire river basin will be available.

8   Concluding recommendations

The authors are very well aware that at present, there are computational constraints as well as the fact that additional computational,

geotechnical, structural and statistical research is needed for the further enhancement of the outlined computational framework. In addition the institutional framework is still to be elaborated. Once further developed, this conceptual framework is intended to be a tool for *policy makers* to evaluate the effects that (regional) safety improvement measures have on the flood risk in the entire river basin. Thus enabling a reliable cost benefit analysis for decision support. For more detailed recommendations see Van Mierlo *et al.* (2003).

Acknowledgement

References

1. CALLE, E.O.F and KNOEFF, J.G. (2002). Failure Mechanisms and Residual Strength, Delft Cluster project DC 02.03.01 (http:/www.library.tudelft.nl/delftcluster … Risk due to flooding) (in Dutch).

2. COMMISSIE LUTEIJN (2002). Gecontroleerd Overstromen: Advies van de Commisie Noodoverloopgebieden, Den Haag, The Netherlands (in Dutch).

3. DAWSON, R.J., HALL, J.W., SAYERS, P.D., BATES, P.D. and ROSU, C. (2005). "Sampling-Based Flood Risk Analysis for Fluvial dike Systems," *Stochastic Environmental Research and Risk Analysis*, 19(6), 388–402.

4. DAWSON, R.J. and HALL, J.W (2006). "Adaptive importance sampling for risk analysis of complex infrastructure systems". *Proc. Royal. Soc. A: Mathematical, Physical and Engineering Sciences* (http://www.journal.royalsoc.ac.uk).

5. DE JAGER, F.G.J. (1998). Beschermen Tegen Overstromen, Onderzoek Naar een Veiligheidsbenadering Op Basis van Risico's, Afstudeerwerk aan de Universiteit Twente (in Dutch).

6. DG RWS, RIKZ, DWW, RIZA (2001). Hydraulische Randvoorwaarden 2001 Voor het Toetsen van Primaire Waterkeringen (in Dutch).

7. DHONDIA, J.F. and STELLING, G.S. (2004). "Sobek 1D-2D integrated hydraulic model for flood simulation – its capabilities and features explained", in *sixth International Conference on Hydroinformatics*, Singapore, Liong Phoon&Babovic (eds), World Scientific Publishing Company, ISBN 981-238-787-0.

8. FISHMAN, G.S. (1996). Monte Carlo: Concepts, Algorithms, and Applications, Springer-Verlag, Berlin.

9. HAO-MING ZHOU (1995), Towards an Operational Risk Assessment in Flood Alleviation, Delft University Press.

10. IRMA Living with Floods (2001). Executive Summary, Alterra, IHE, RIZA, TUD, WL/Delft Hydraulics. Sponsored by IRMA (EU) in the context of the IRMA-Sponge research programme and sponsored by Delft Cluster.

11. MULLER, M. (2003). Het Hoofd Boven Water, een Onderzoek naar de Informatiebehoefte bij de besluitvorming over de inzet van Noodoverloopgebieden en de Ontwikkeling van een Spelsimulatie ten Behoeve Van Verder Onderzoek, Delft University of Technology (in Dutch).

12. PICASO (2001). Pilot Case Overstromingsrisico (Delen I t/m VI), Ministerie van Verkeer en Waterstaat, Directoraat-Generaal Rijkswaterstaat, Bouwdienst Rijkswaterstaat Waterbouw (in Dutch).

13. SCHOLTES, J. (2002). Wie Laat Het in Nood Overstromen, Delft University of Technology (in Dutch).

14. SILVA, W., KLIJN, F. and DIJKMAN, J.P.M., (2001). Room for the Rhine Branches in The Netherlands, What the Research Taught us, Ministry of Transport, Public Works and Water Management, Directorate-General for Public Works and Water management, Institute for Inland Water Mangement and Water Treatment/ RIZA, The Netherlands.

15. SPANKRACHT Study (2002). Eindrapport Projectgroep Spankrachtstudie, Ministeries VenW, VROM, LNV, Provincies Utrecht, Gelderland, Overijssel, Noord-Brabant, Zuid-Holland, Waterschappen, Vereiniging Nederlandse Riviergemeenten, The Netherlands (in Dutch)

16. TILLIE, J. (2001). Ontwerp van een Beslissingsmethodiek Voor de Inzet van Noodoverloopgebieden Langs de Waal, Afstudeerwerk aan de Technische Universiteit Delft, Faculteit Civiele Techniek en Geowetenschappen, Subfaculteit Civiele Techniek, Sectie Waterbouwkunde (in Dutch).

17. VAN DER KNAAP, F.C.M. (2000), Breach Growth as a Function of Time, WL/Delft Hydraulics Memo's 2 and 3 of May 21 and September 5 respectively, Q2655, Delft (in Dutch).

18. VAN DER WIEL, W.D. (2003). Probabilistic Risk Assessment of a System of Dike Ring Areas, Delft University of Technology, MSc Thesis.

19. VAN DEURSEN, W.P.A. (1995). Geographical Information Systems and Dynamic Models: Development and Application of a Prototype Spatial Modelling Language, Doctor's dissertation, Utrecht University, NGS 190 (http://pcraster. geog.uu.nl).

20. VAN GENT, M.R.A. (2002). Low Exceedance Wave Overtopping Events, measurements of Velocities and The Tickness of Water Layers on the Crest and Inner Slope of Dikes, Delft Cluster project DC 03.02.02 (http:/www.library. tudelft.nl/delftcluster).

21. VAN MIERLO, M.C.L.M., OVERMARS, J.M.S. and GUDDEN, J.J. (2001). DelftFLS Inundatie Simulaties van de Ooij-Duffelt Polder en het Land van Maas en Waal, Contributing to the PoldEvac (EU) Project, WL/Delft Hydraulic report R3268 (in Dutch).

22. VAN MIERLO, M.C.L.M., VROUWENVELDER, A.C.W.M., CALLE, E.O.F., VRIJLING, J.K., JONKMAN, S.N., De BRUIJN, K.M. and WEERTS, A.H. (2003). Effects of River System Behaviour on Flood Risk, Delft Cluster Project DC 02.01.01. (http://www.library.tudelft.nl/delftcluster …… Risk due to flooding).

23. VAN MIERLO, M.C.L.M. (2005). Verkenning van systeemwerking in het bovenrivierengebied van de Rijntakken, WL|Delft Hydraulics, Q4019, December 2005 (in Dutch).

24. VERHEIJ, H.J (2002). Bresgroei in Cohesieve Materialen – Voortgangsrapportage 2001, WL/Delft Hydraulics, Verslag Q2955 (in Dutch)

25. VERHEIJ, H.J. and VAN DER KNAAP, F.C.M. (2002). Modification Breach Growth Model in HIS-OM, WL|Delft Hydraulics, Q3299 (in Dutch).

26. VERMEIJ, M. (2001). Onderzoek Naar de Afmetingen van een Overlaat Langs de Waal, Afstudeerwerk Aan de Technische Universiteit Delft, Faculteit Civiele Techniek en Geowetenschappen, Sectie Waterbouwkunde (in Dutch).

27. VIS, M., KLIJN, F., DE BRUIJN, K.M. and VAN BUUREN, M. (2003). "Resilience Strategies for Flood Risk Management in The Netherlands", *Journal of River Basin Management*, 1, 33–40 (http://www.jrbm.net).

28. VISSER, P.J (1998). Breach Growth in Sand Dikes, (PhD thesis), Communications on Hydraulic and Geotechnical Engineering, Report nr. 98-1, Hydraulic and Geotechnical Engineering Division, Faculty of Civil Engineering and Geosciences, Delft University of Technology.

29. VNK (2003). Veiligheid Nederland in Kaart (Map of Safety against Flooding of the Netherlands), Research report to be Issued in 2006 (http:/www.projectVNK.nl).

30. VRIJLING, J.K. (2002). Uncertainties in Emergency Storage Areas, Working document, Delft University of Technology.

31. VRISOU VAN ECK, N., HUIZINGA N. and DIJKMAN, H.J. (2000). HIS-Schade en Slachtoffer Module (HIS-SSM), Gebruikershandleiding, Geodan Geodesie, HKV Lijn in Water (in Dutch).

32. VRISOU VAN ECK, N., KOK, M. and VROUWENVELDER A.C.W.M. (2002). Standaardmethode Schade en Slachtoffers als gevolg van overstromingen, Deel 1: Standaard methode (in Dutch).

33. WAARTS (2000). An Appraisal of DARS, P.A. Waarts, PhD Thesis, TU Delft.