Flood risk analysis for the Bac Hung Hai polder, Vietnam

A preliminary assessment of the system approach, taking effects of flooding on the entire system into account

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Ferdinand Diermanse, Kees Sloff, Henk Ogink

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ABSTRACT:

This study deals with the application of the 'crude Monte Carlo' method to asses dike failure and flood damage with due account of system behaviour. The suitability of this probabilistic approach to determine the sequence of improvements of dike segments surrounding the polder to reduce the risk of flooding in a most cost effective manner has been investigated. The method has been applied to the Bac Hung Hai polder in the Red River delta in Vietnam.

The 'crude Monte Carlo' involves a large number of deterministic simulations of flood events, where for each individual simulation the random parameters and boundary conditions are sampled from their respective probability distribution functions. The result of each Monte Carlo simulation is flood damage, eliminating problems of double counting of damages when dike failure and flood damage are treated separately. Effects of dike breaches on the hydraulic conditions elsewhere (system behaviour) have been incorporated. The study showed that a sequence of improvements of dike segments, which follows the largest contribution to the overall Expected Annual Damage, does not lead to the best approach for the Bac Hung Hai polder. Due to system behaviour a different sequence proved to be more effective as improvement of one section may aggravate the flood risk at another.

The study demonstrated that this new approach improves the understanding of flood risk. The method is easy to apply and provides practical outputs for best investments for dike improvements. The method is very useful for optimization of the level of protection in river delta's.

REFERENCES:

VER	AUTHOR		DATE	REMARKS	REVIEW		APPROVED BY	
	Ferdinand Diermanse				Karin de Bruijn	KdB	A.G. Segeren	
	Kees Sloff							
	Henk Ogink		24/12/07					

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1 Introduction

1.1 General

Flood risk is determined by the probability of flooding and the damages due to flooding. This research report comprises the results of a flood risk analysis for a polder accounting for the consequences of multiple dike breaches with respect to damage in the polder and to the flows and stages in the surrounding river system. The latter effect is referred to as "system behaviour". A probabilistic approach, known as 'crude Monte Carlo', has been used to asses dike failure and flood damage with due account of system behaviour. This study has investigated the suitability of the approach to determine the sequence of improvements of dike segments surrounding the polder to reduce the risk of flooding in a most cost effective manner.

The method has been applied to the Bac Hung Hai polder in the Red River delta in Vietnam (see Figure 1.1).The downstream part of the Red River catchment is divided into a number of dike rings which consists of many polders. The dike ring selected for this research is threatened by flooding from different rivers. Land use is partly urban and partly agricultural. This area is selected as a pilot since it has similarities with the Dutch dike ring system and with other areas in Europe and because data and models are already available.

Recently, flood risk analyses have been carried out for this polder, but the applied procedure did neither properly account for the effect of multiple breaches on flood damage nor did it consider system behaviour. To overcome these shortcomings the procedure presented in this research report has been advocated. Note that in the application the Bac Hung Hai polder conditions were modified and only the dike failure mechanism wave overtopping was considered to focus on the suitability of the method rather than producing a solution for dike improvements of the polder at this stage.

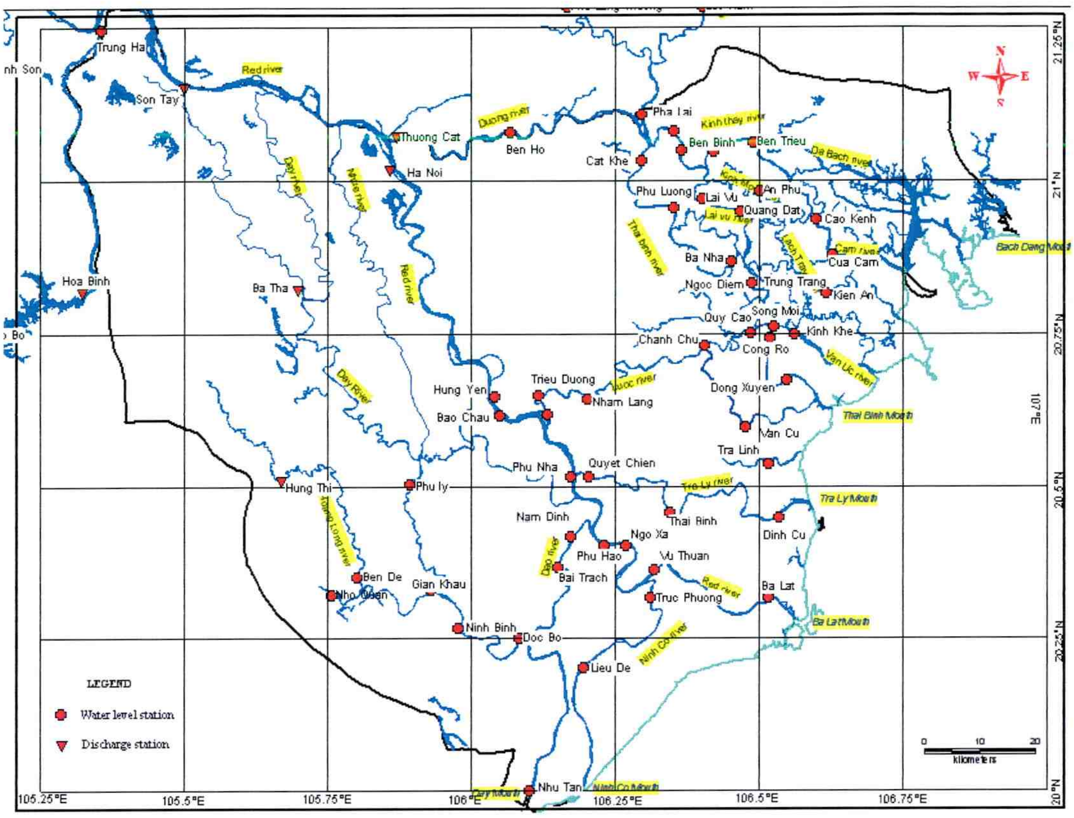

Figure 1.1 Layout of the Red River delta with the Bac Hung Hai polder (enclosed by Red, Duong, Thai Binh and Luoc Rivers)

1.2 Background and objectives

The 2nd Red River Basin Sector Project

In the frame of the 2nd Red River Basin Sector Project for Integrated Water Resources Management a Flood risk Assessment (FRA) is carried out for Bac Hung Hai Polder in the Red River delta in Vietnam, where floods pose a major threat to people, livestock and public and private property (Sweco-Groner *et al.*, 2006). Figure 1.2 shows the Bac Hung Hai polder and sub-polders. The Bac Hung Hai polder is enclosed by the Red, Duong, Tai Binh and Luoc rivers. The FRA activity in this project has the following objectives:

1. ranking of flood protection projects from a list of projects for the rehabilitation of dikes and irrigation structures, and
2. demonstrating the FRA methodology to the responsible agencies for flood management in Vietnam.

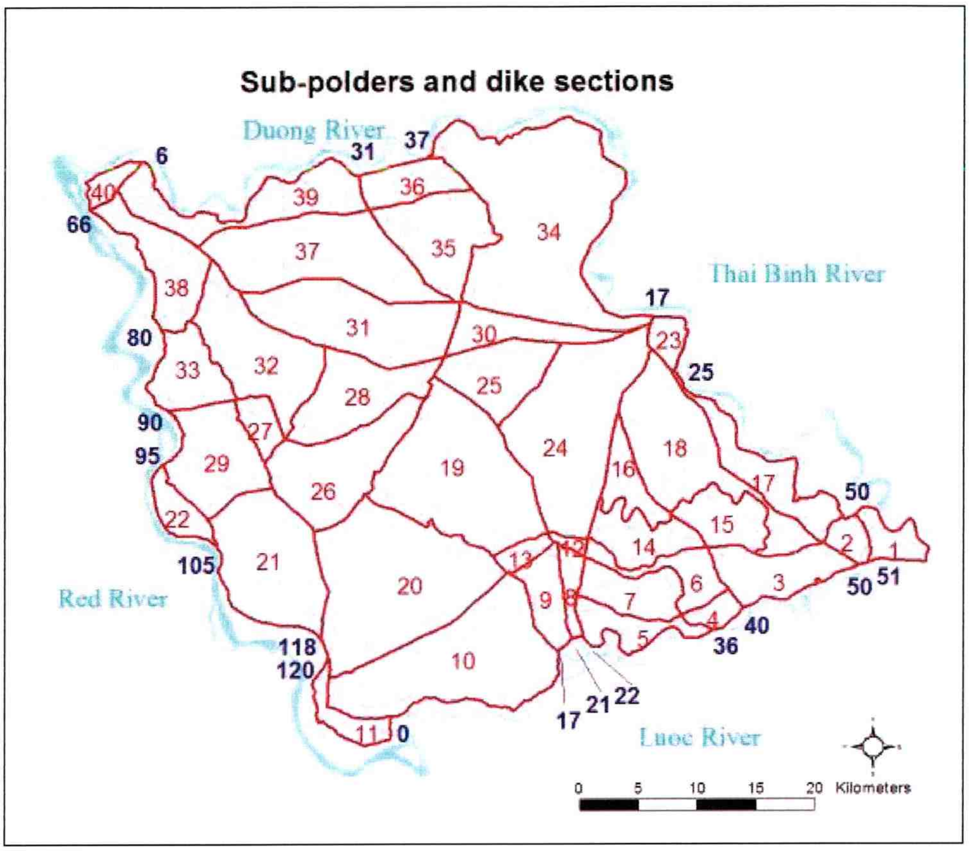

Figure 1.2 The Bac Hung Hai polder and sub polders (sub-polders and dike sections in red and river kilometres in blue)

In the first stages of the project the following procedure was used for Flood Risk Assessment:

- Identification of the failure probabilities due to the various failure mechanisms individually for each dike section including the stochastic nature of the constituting components using a Monte Carlo approach.
- Quantification of flood damages given the main failure mechanism and its location in the dike ring in relation to compartments.
- Determination of the expected or average annual damage cost (EAD) due to failure for each dike section.
- Assessment of the economic viability of upgrading of dike sections by cost-benefit analysis while a Flood Risk Index (FRI) method was used to include non-monetary / non-tangible values.
- Ranking of flood protection projects based on economic, social and environmental indicators, using a weighting proposed by the stakeholders.

A schematic layout of the used FRA procedure is presented in Figure 1.3.

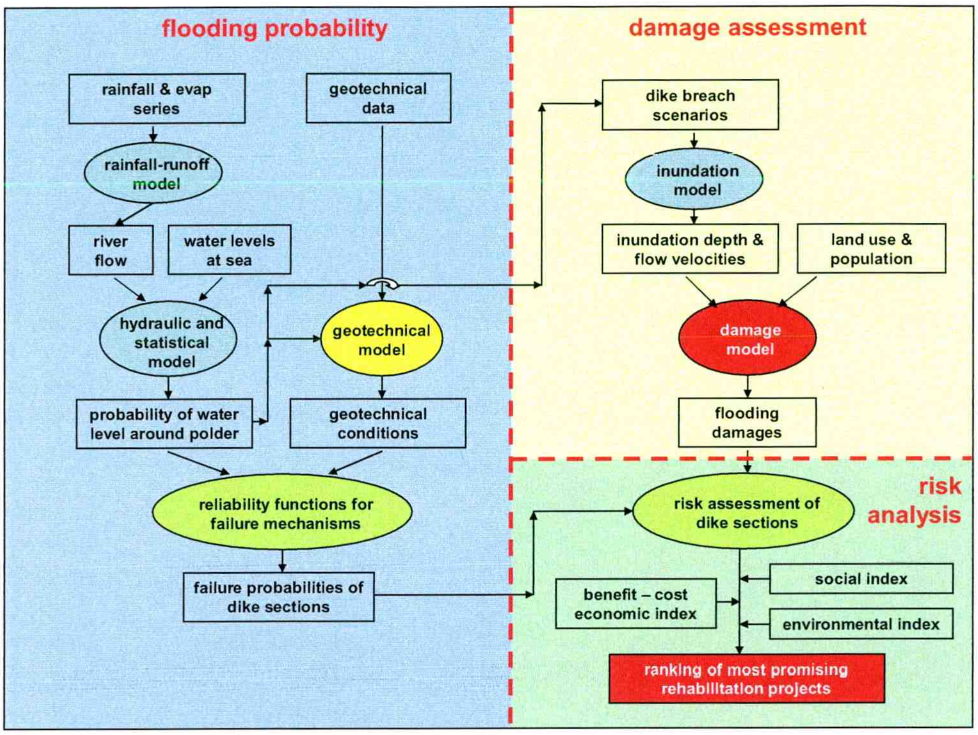

Figure 1.3 Scheme of Flood Risk Assessment methodology

An economic ranking of dike improvements was obtained starting off from the section with the highest flood risk and accruing to its improvement a benefit equal to the difference between the section EAD and the one with the second largest EAD, and so on. Shortcomings of this procedure are:

- The probabilistic model focused on determination of failure probabilities sec. These probabilities were subsequently combined with damages to arrive at expected annual damage costs. Since the analyses were carried out for dike sections, rather then for the whole ring-dike, the procedure leads to double counting of damages and biases in the EAD's of the sections.
- No account was made of system behavior, meaning that the effect of a breach at one location on the flows and water levels and hence on the failure probability at other locations was not considered.

Approach in this research

To eliminate these shortcomings the probabilistic model as used in this study, known as 'crude Monte Carlo', is extended with flood damage calculation and effects of system behavior. It implies that in the Monte Carlo simulation for each draw in case of failure also the damage due to breaches at one or more locations is calculated and the hydraulic conditions adjusted. The method then automatically produces besides improved failure probabilities also the expected annual damage cost for dike sections as well as for the whole dike ring. The probability of flooding is quantified for the current situation as well as for possible future situations in which (several) dike sections have been improved.

The main objective of this study is to develop a method that shows which dike sections along the Bac Hung Hai polder should successively be improved to realize a maximum reduction of the flood risk. This method should take the system behavior into account.

In this report first the concept of probabilistic computations and incorporation of system behavior are discussed in Chapter 2. In Chapter 3 the results of the application of the procedure to Bac Hung Hai polder is presented, followed by the Conclusions in Chapter 4.

2 Method

2.1 Concept of failure: reliability functions

There are various mechanisms that can lead to failure of dikes and other flood defence structures. Failure mechanisms are mathematically described by so-called *reliability functions*, *Z*, in which the resistance, *R*, is compared with the hydraulic load, *S*:

$$Z = R - S \tag{2.1}$$

This means the structure fails if $Z<0$. The reliability functions are different for different failure mechanisms. For the mechanism "wave overtopping" the reliability function is as follows:

$$Z_{wave\ overtopping} = h_c - (h + h_{runup}) \tag{2.2}$$

In which h_c is the crest level of the dike, h is the water level in the river and h_{runup} is the wave run-up. This equation simply states that failure occurs if the waves that run up against the dike exceed the crest level of the dike ('overtopping'). In this example the resistance, *R*, is equal to the crest height of the dike and the hydraulic load, *S*, is equal to the sum of the water level and the wave run-up.

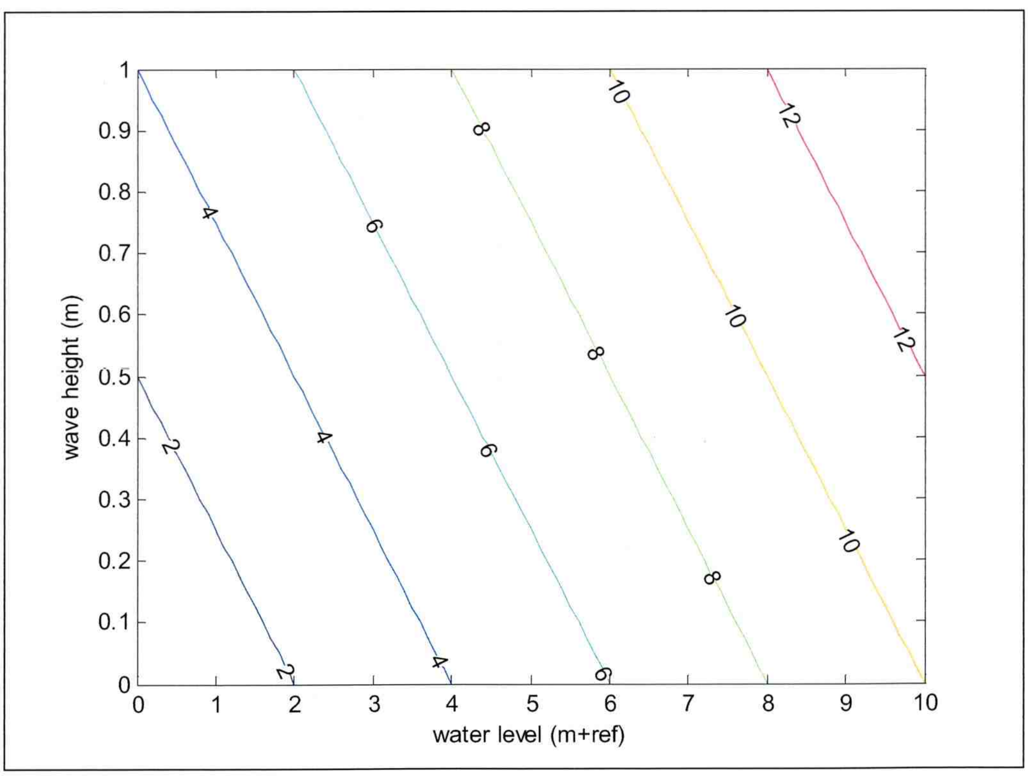

Figure 2.1 Example of hydraulic load as a function of water level and wave height.

The principle of the reliability function is illustrated with the (simplified) example of Figure 2.1. In this example the contour lines show the hydraulic load as a function of the water level and wave height during a storm event. Other relevant variables, such as wave period and wave direction are ignored for clarity reasons. Each point in the figure, i.e. each combination of water level and wave height, can be considered as the maximum values observed during a (synthetic) storm event. For a number of combinations of water level and wave height the resulting hydraulic load can be computed from a set of physical relations which is referred to as the *numerical simulation model.* The contour lines of Figure 2.1 are based on the outcome of the numerical simulation model.

Suppose the crest height of the dike is equal to 10 [m+ref]. This level is represented by the yellow line in Figure 2.1. The area to the upper right of this yellow line consists of all combinations of water level and wave height for which the resulting hydraulic load exceeds 10 [m+ref] and therefore will lead to failure of the dike. This area is referred to as the *failure domain*. The yellow contour line in this example is called the *limit state*, i.e. the threshold between failure and non-failure. This is also schematically depicted in Figure 2.2.

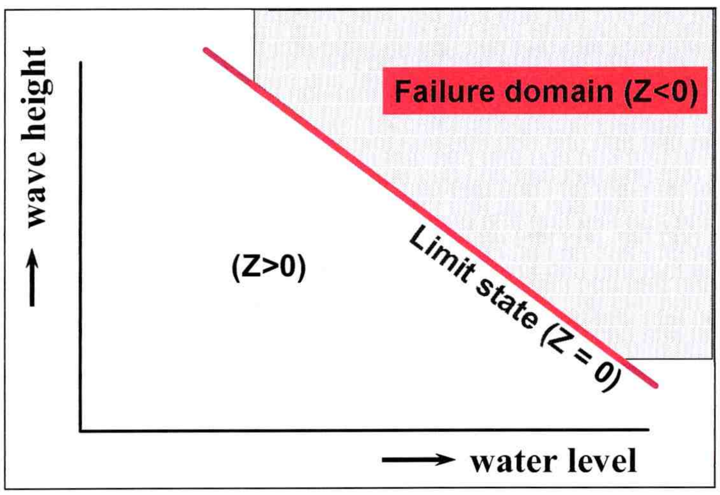

Figure 2.2 Schematic view of a failure domain

2.2 Computation of failure probabilities

The reliability function, Z, of the previous section describes for which (hydraulic) conditions the dike is expected to fail. For the safety of the land that is protected by the dike it is of course highly relevant to know the probability (or frequency) with which this will happen. Mathematically, this means that one needs to know the accumulated probability of occurrence of all combinations of variables in the failure domain. Hence, one requires a procedure that basically performs the following two tasks:

1. Find the failure domain. This involves exploration of the n-dimensional space of possible realisations, where n is the number of random variables (water level, wave height, etc.) involved.
2. Determine the cumulative probability of occurrence of all combinations in the failure domain.

The first task is done through application of a numerical simulation model, in this case a model that determines the wave run-up of the embankment from input variables like water level, wave height, wave direction etc. The model is executed for various combinations of realisations of the n random variables. The required number of model computations strongly depends on the selected probabilistic method.

For the second task one requires the combined (multivariate) probability distribution function of all random variables involved. If the random variables are statistically independent one can simply multiply their respective marginal (univariate) distribution functions. However, this will not work for the current example: for instance water level and wave height are not statistically independent. Therefore, a multivariate distribution function of water level and wave height is required, that correctly describes their mutual dependency. Generally, a simple correlation coefficient will suffice.

There are several methods that perform the two tasks as described above, such as:
1. Numerical integration;
2. Crude Monte Carlo;
3. Smart Monte Carlo techniques:
   a) Directional sampling:
   b) Importance sampling
4. FORM - First Order Reliability Method.

In the current studies a crude Monte Carlo technique is applied for reasons to be explained in the next section.

2.3 Crude Monte Carlo

2.3.1 Principle

The method we applied to derive failure rates for the various dike sections and failure mechanisms along the Bac Hung Hai polder is a crude Monte Carlo (MC) simulation technique. The main reason to apply this method is the fact that MC is relatively easy to apply and still, if enough samples are taken, very accurate. MC offers the possibility to take effects of system behaviour directly into account, whereas with other probabilistic methods this is much more complex.

In essence, MC involves a large number of deterministic simulations of flood events, where for each individual simulation the random parameters and boundary conditions are sampled from their respective probability distribution functions. Due to the random character of the input, the output of the deterministic simulations will vary and therefore have a random nature as well. The purpose of MC is to derive the statistical features of the output. In our case the random input variables are features such as the water level in the river or the seepage path length in the dike; the output consists of Z-values. A negative Z-value refers to a situation in which the dike is not strong enough to resist the (hydraulic) load, which means it "fails". The failure rate can simply be estimated by counting the total number of negative Z-values and divide them by the total number of samples.

2.3.2 Example

To clarify the above description of MC we give a simple example for the failure mechanism "overflow". The Z-function and variables of this mechanism is as follows:

$$Z_{overflowg} = h_c - h \quad (2.3)$$

In which h_c is the crest level of the dike and h is the water level in the river. Failure occurs (i.e. Z<0) when the water level, h, exceeds the crest level, h_c, of the dike. Suppose the probability distribution of the annual maximum water level is exponentially distributed:

$$F(h) = 1 - e^{-\frac{h-a}{b}} \quad ; a = 10, b = 0.86 \quad (2.4)$$

For the density function f(h) = F'(h) this means:

$$f(h) = \frac{1}{b} e^{-\frac{h-a}{b}} \quad ; a = 10, b = 0.86 \quad (2.5)$$

Both functions are shown in Figure 2.3.

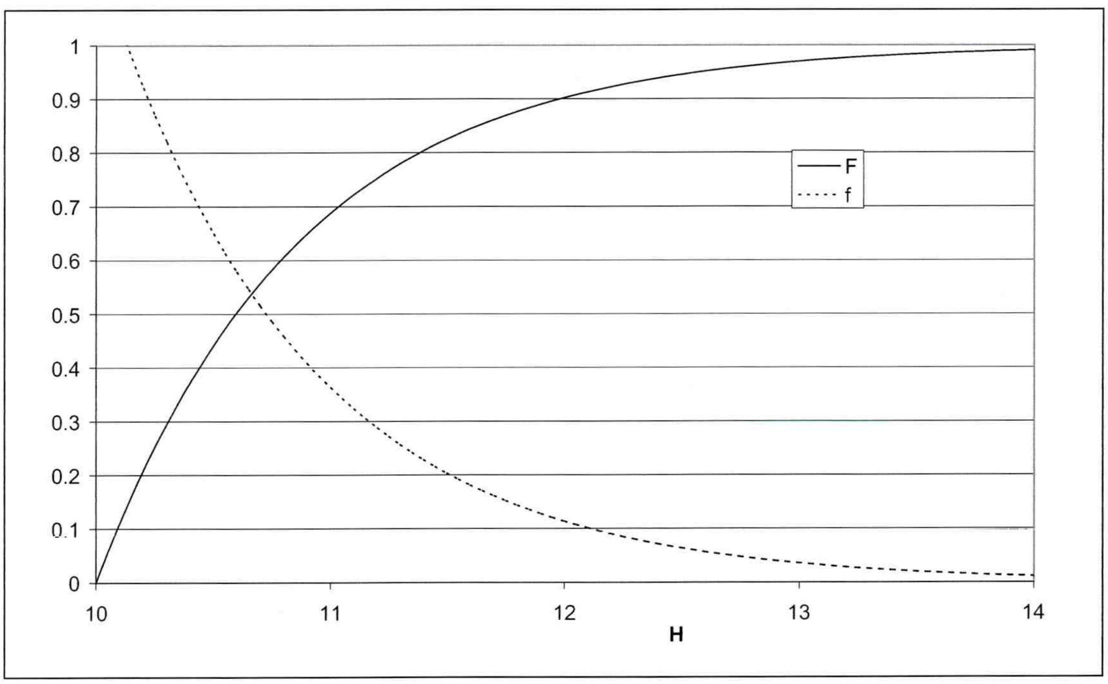

Figure 2.3 Assumed distribution function, F, and density function, f=F', for water level H.

The crest level, h_c, is assumed to be normally ('Gaussian') distributed with mean 11.5 and standard deviation 0.2. The standard deviation expresses the "uncertainty" in the available information on the crest level.

Table 2.1 shows 20 combined MC-realisations for both h and h_c, sampled from their respective distribution functions. In 4 cases the water level, h, exceeds the crest level, h_c. This means in 4 out of 20 cases failure occurs, which gives an estimated failure rate of 1/5 per year.

Note that we used a relatively small amount of (20) samples in this example. Generally, much more samples are needed to reduce the uncertainty in the resulting estimate of the failure rate. The next section will explain this aspect of MC in more detail.

Table 2.1 Monte Carlo realisations for the failure mechanism 'overflow'

realisation	h	h_c	Z	failure?
1	10.63	11.36	0.73	no
2	11.07	11.67	0.60	no
3	12.80	11.31	-1.49	**yes**
4	10.06	11.32	1.26	no
5	11.21	12.06	0.85	no
6	10.49	11.65	1.17	no
7	11.93	11.65	-0.28	**yes**
8	10.45	11.80	1.35	no
9	10.39	11.04	0.66	no
10	10.67	11.42	0.74	no
11	10.43	11.36	0.93	no
12	10.02	11.40	1.38	no
13	10.18	11.58	1.40	no
14	11.64	11.54	-0.10	**yes**
15	11.90	11.80	-0.10	**yes**
16	10.61	11.25	0.63	no
17	10.79	11.35	0.56	no
18	10.72	11.45	0.74	no
19	10.89	11.48	0.60	no
20	10.28	11.54	1.26	no

2.3.3 Uncertainties

In MC the uncertainty of the estimated outcome decreases as the number of samples increase. In the example of the previous session we used a relatively small number of samples. The resulting number of failures was equal to 4, but could easily have been 2. This would lead to an estimated failure rate of 1/10 per year, i.e. half of our initial estimate of 1/5 per year. An increase of the number of samples will reduce the effect of coincidence, according to the famous statistical law of large numbers. Note, however, that uncertainties in input parameters (such as parameter a of the distribution function of h are not reduced by an increase in the samples.

In reliability analyses, the failure probabilities that are analysed are generally relatively small. Underneath we will demonstrate that the errors introduced by Monte Carlo analysis are relatively large for small probabilities of failure. We will also show that these relative errors can be reduced by an increase in the number of Monte Carlo realisations.

Suppose a dike has a probability of failure, p:

$$p = P[Z < 0] \tag{2.6}$$

Now we have a set of physical equations, implemented in a numerical simulation model, to compute whether or not the dike fails for several combinations of water level, wave height etc. In this section we demonstrate the uncertainties of MC itself, so let us assume for the sake of clarity that this numerical simulation model is perfect. By this we mean that for each Monte Carlo realization we have:

$$P[Z_i < 0] = p \tag{2.7}$$

where Z_i is the resulting Z-value of the i^{th} MC-realisation. Let M be the total number of failures out of N MC realizations for this dike:

$$M = \sum_{i=1}^{N} 1_{[Z_i<0]} \overset{def}{=} \sum_{i=1}^{N} X_i \tag{2.8}$$

In this equation we introduced a new random variable X which equals 1 if the dike fails and equals 0 if the dike does not fail. This means:

$$\begin{aligned} P[X=1] &= p \\ P[X=0] &= 1-p \end{aligned} \tag{2.9}$$

After performing N MC realisation we estimate the probability of failure of the dike as follows:

$$\hat{p} = \frac{M}{N} \tag{2.10}$$

The expectancy of M equals:

$$E[M] = E\left[\sum_{i=1}^{N} X_i\right] = \sum_{i=1}^{N} E[X_i] = NE[X_i] = Np \tag{2.11}$$

For the expected value of the *estimated* probability of failure this means:

$$E[\hat{p}] = E\left[\frac{M}{N}\right] = E\left[\frac{Np}{N}\right] = p \tag{2.12}$$

So the expected outcome is equal to p, i.e. there is no bias in our MC analysis, which is good. Still, the outcome of our MC experiment is random, which means there is some uncertainty involved. This uncertainty can be expressed in terms of its standard deviation. A single MC realisation has two possible outcomes: X_i=1 or X_i=0 (the dike fails or it does not). For the standard deviation of a single realisation, this means:

$$VAR[X_i] = E\left[(X - EX)^2\right] = p(1-p) \tag{2.13}$$

This is a well-known result from basic statistics of the binominal distribution. For the combined outcome of multiple realisations this means:

$$VAR[M] = VAR\left[\sum_{i=1}^{N} X_i\right] = \sum_{i=1}^{N} VAR[X_i] = NVAR[X_i] = Np(1-p) \tag{2.14}$$

Or, expressed by its standard deviation:

$$\sigma[M] = \sqrt{Np(1-p)} \tag{2.15}$$

for $\hat{p}$ this means:

$$\sigma[\hat{p}] = \sigma\left[\frac{M}{N}\right] = \frac{\sigma[M]}{N} = \frac{\sqrt{Np(1-p)}}{N} = \sqrt{\frac{p(1-p)}{N}} \tag{2.16}$$

For small failure rates (p small) this can be approximated by:

$$\sigma[\hat{p}] \approx \sqrt{\frac{p}{N}} \tag{2.17}$$

This means the standard deviation *decreases* with *decreasing* value of p. However, if we consider this value *relative* to the value of p:

$$\frac{\sigma[\hat{p}]}{p} \approx \sqrt{\frac{1}{Np}} \tag{2.18}$$

It shows that relative to the value of p the standard deviation *increases* with *decreasing* value of p. This means for small values of p, the standard deviation of its estimate $\hat{p}$ is relatively large. As equation (2.18) shows, this can only be compensated for by an increase in the number of Monte Carlo realisations, N. So, the smaller the considered probability of failure, p, the larger the required number of Monte Carlo simulation runs, N.

The outcome above can be easily explained. Exceedance probabilities of extreme events are close to 0. Application of an unbiased estimation method as the one above will give an estimate of this probability that is also close to 0. So therefore, in *absolute sense*, differences between the estimated and actual probability of exceedance are small. The reason why in *relative sense* differences are large is because by definition only a small minority of the samples will be extremes. This means, there is less information available on the extremes than on the events that are 'moderate'. Of course, this is not only the case for Monte Carlo analysis, but also in for observed time series. At least with MC there is an opportunity to increase the amount of samples, albeit at the cost of additional computation time.

2.3.4 Combining failure rates

Different mechanisms

With MC it is relatively straightforward to combine failure rates of different mechanisms in order to obtain the failure rate of the dike section. In order to do so, for each of the N sample-rounds we take samples for all parameters of the various mechanisms involved. Subsequently, we derive Z-values for each mechanism. Then we determine whether or not the dike fails, according to the following rule:

> A dike "fails" if at *least* one failure mechanism occurs.

In numbers, this means that the dike fails if at least one of the Z-values is negative. In this way we determine for each sample-round if the dike fails. The failure probability of the dike can simply be estimated by counting the total number of failures and divide them by the total number of samples.

Multiple locations

In a similar manner, it is also possible to derive the failure probability of a series of dike sections, or a dike ring as a whole, as follows.

For each of the N sample-rounds we take samples for all parameters of the various mechanisms involved. For the water level in the river we sample one value that is representative for all dike sections involved. By this we mean if for location 1 the selected water level has a recurrence interval of 20 years, we assume the 20-year water level at each of the other locations as well. This is because water levels along the river are strongly related.

Subsequently we determine for each dike section whether or not failure occurs, according to the procedure of the previous section. Then we determine whether or not the dike ring fails, according to the following rule:

A dike ring "fails" if at *least* one of the dike sections fails.

In this way we determine for each sample-round if the dike ring fails. The failure probability of the dike ring can simply be estimated by counting the total number of failures and divide them by the total number of samples.

2.3.5 Computation of damage

Annual expected damage

In our approach N synthetic events are simulated where N is typically a large number in the order of $10^4 - 10^6$. Each synthetic event represents the annual maximum flow event. This basically means that a period of N years is simulated. Note, however, that over this synthetic period of N years the statistical and physical characteristics (like crest heights or expected maximum water levels) do not change. For each event we derive the damage in case of floods (see below). For the majority of events (or years) no floods occur and damage is zero.

Computation of damage of a single event

The main dominating factor that characterises a synthetic flood event is the discharge at Son Tay, the last gauging station on the Red river before the Duong bifurcates from the river, see Figure 1.1. This discharge is sampled from its probability distribution function, which is based on the observed annual maximum discharge over the past years. Each sampled discharge represents the annual maximum flow at Son Tay. Based on hydraulic model simulations the water levels in the Red river, Duong, Luoc and Thai Binh are obtained. Subsequently, in combination with the dike characteristics like the crest height it is determined whether or not failure of the dike system is expected to occur. This is done by evaluating Z-functions of one or more failure mechanisms.

In our approach we take into account the fact that multiple dike breaches can occur during a single flood event. Furthermore, we also take into account that a dike breach leads to a reduction of the water level in parts of the river system and, consequently, to a reduction in the probability of dike breaches at other locations. In computational sense this means that we apply the following iteration K times, where K stands for the number of dike breaches during a single event:

1. quantify Z-values for all dikes along the river system;
2. determine the location with the lowest Z-value;
3. if the Z-value of step 2 is negative: failure occurs and the dike will breach;
4. determine the total volume of water that flows into the Bac Hung Hai polder by integration of the breach flow (see Section 2.4);
5. determine the maximum discharge of water that flows into the Bac Hung Hai polder according to equation (2.21) when the river is at maximum stage;
6. determine the reduction of the water level in the river system, downstream of the breach, from the maximum reduction if river discharge using stage – discharge reduction formulae available for any location in the rivers surrounding the polder and flow distributions over bifurcations.

The reduction of the water level will lead to an increase in Z-values. We execute the above procedure until all z-values are positive. This means for some events we may have multiple dike breaches.

Based on the total volume, i.e. the sum of the volumes as derived in Step 4, the total damage in the Bac Hung Hai polder is estimated with the following equation:

$$D = 536*(1-e^{-4.44 \times 10^{-4} V}) \tag{2.19}$$

With: D = damage in million US Dollar
V = flood volume entering through the breaches in Mm^3

This equation has been derived from the damages calculated for flooding due to dike breaches at different locations. A total number of 5 breach locations has been considered. The inundations were calculated for flood conditions on the rivers of 100 and 25 year return period. A GIS was used to calculate the areas for different types of land use that are inundated, the amount of roads affected, and the population affected in case of a dike failure. The inundated areas (per land use type and depth) and kilometres of roads affected are the input of the economic model, see Figure 2.4.

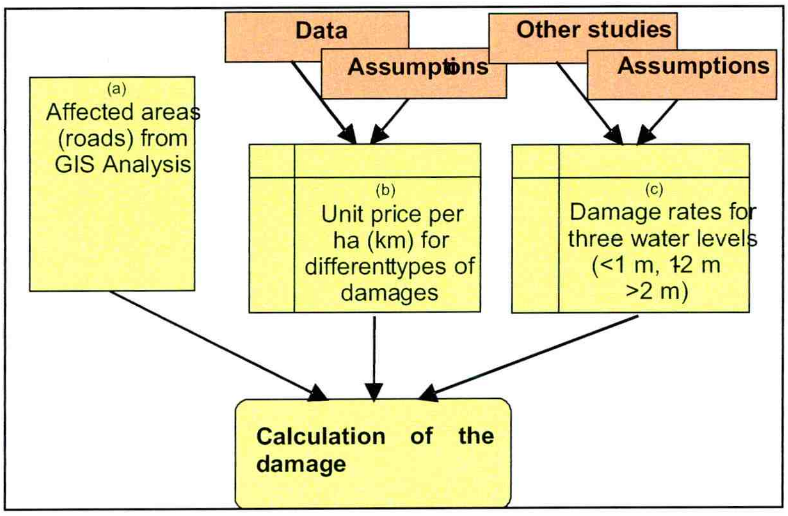

Figure 2.4 Economic model

In the model unit costs for the different types of damages are determined, expressed in a unit cost per ha and linked to one or more land use types. These unit costs have been combined with loss-rate tables to estimate the damage for different depths and durations. The types of damages included in the economic model were agricultural losses, damage to buildings and assets, infrastructural damages and GDP loss, i.e loss in production in flooded areas. It appeared that for this polder a fairly unique relationship existed between flood volume and associated damage as shown in Figure 2.5 and expressed by equation (2.19). It is noted that a larger spatial heterogeneity in the flood volume-flood damage relation can easily be accommodated for by extending the database holding the relevant relations for breach locations and sub-polders.

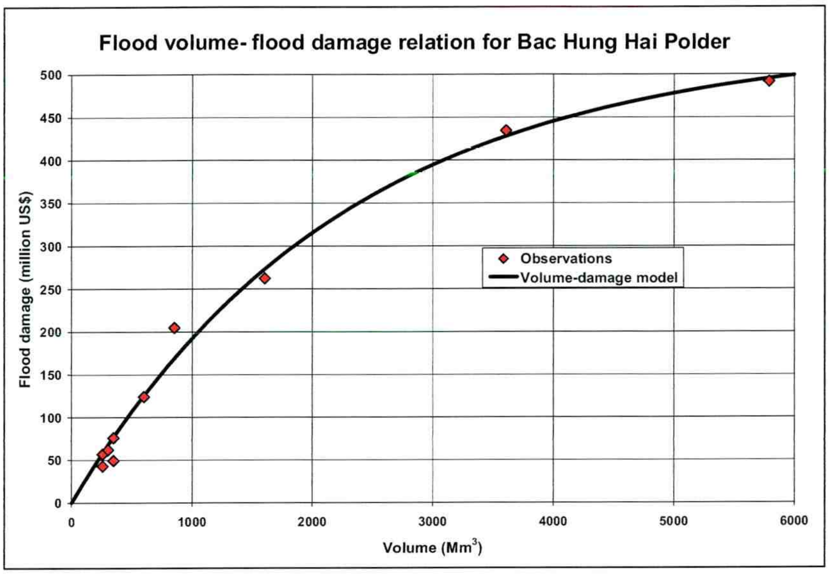

Figure 2.5 Flood volume-flood damage relation for Bac Hung Hai polder

2.4 Dike breach and system behaviour

System behaviour deals with the effects of an accidental or deliberate dike breach at a location in the delta on the hydraulic conditions elsewhere. The incorporation of this effect in the calculations is of importance to realistically determine the conditions for multiple breaches.

Dike failure is likely to occur at the moment of maximum water level or some time before the peak. If the dike breach occurs before the peak, the discharge through the breach will "cut-off" the peak and the downstream reach will experience lower levels. However, if the breach develops at the peak or later, the downstream reach will not have a reduction of water levels (they will only notice that the flood falls more rapidly after the peak has passed). Still it is useful to select the moment of breaching close to the peak because then:

- the pressure on the dikes is highest, and
- the largest damage is created in the polder because of the high water-level difference that drives the inflowing water and the growth of the breach.

Although dike failure may well occur during the fall of the flood, such a failure is expected to lead to lower damages as the inflow discharge to the polder will be less. In the procedure it is assumed that the dike failure takes place 24 hours prior to the passage of the peak at a location.

If a dike breaches, the breach erodes quickly (almost instantaneously) to its full depth. Subsequently, the width of the breach starts growing rapidly, but with a decreasing growth rate when the width becomes larger. The width of the breach is computed using the dike breach formula of Verheij and Van der Knaap (2002):

$$B = 1.3\sqrt{g}\left(\frac{\left[h_{up} - h_{down}\right]^{3/2}}{u_c}\right)\log\left(1 + \frac{0.04g\left[t - t_0\right]}{u_c}\right) \tag{2.20}$$

in which:
B = width of the breach (m)
g = gravity (m/s^2)
h_{up} = upstream water level (in river) (m)
h_{dow} = downstream water level (in polder) (m)
t = time (s)
t_0 = time of start of breach (s)
u_c = constant critical flow velocity sediment/soil (m/s), taken as 1.0 m/s

This equation requires the water level in the river (based on maximum flood level), and the level in the polder (approaching the polder elevation) as input. The maximum breach width has been limited to 200 m, based on experience from historic dike breaches, ranging from clay dikes to sand dikes, (Verheij, 2007, pers. comm.).

Taking the local water levels in the river as function of time from the simulation without breach, and comparing it with a fixed level inside of the polder, the water-level difference over the dike can be approximated. The discharge through the breach has been estimated from the following weir equation:

$$Q(t) = m \cdot B(t)\frac{2}{3}\sqrt{\frac{2}{3}g} \cdot H(t)^{3/2} \tag{2.21}$$

With:
$B(t)$ = width of the breach as function of time t (m)
$H(t)$ = water-level difference over dike breach (m)
m = energy-loss coefficient, equal to 0.5 (-)
$Q(t)$ = discharge through the breach as function of time t (m^3/s)

To arrive at a database with required inputs for the Monte Carlo simulations the following steps have been taken:

1. Frequency analysis of the annual maximum Red river flow at Son Tay incorporating the operation of upstream reservoirs.
2. Development of discharge hydrographs for Son Tay for different return periods with varying flood volume by scaling of the 20 largest historical floods.
3. Using the results of simulations of the hydrodynamic model of the Red river delta relation curves for all locations along the polder have been established, expressing the local maximum water level as function of the maximum water level at Son Tay for various return periods.
4. By scaling of the results for all grid points in the hydrodynamic model bordering the polder flood hydrographs ranging from 5 to 1,000 year return period were established. These have been used to determine the total flood volume through a breach at all locations as well to calculate the breach discharge at the time of the passage of the flood peak at the breach location from above formula's. The flood wave will be lowered in the reach downstream of the breach because of the discharge through the breach. The maximum reduction (ΔQ) is obtained when the peak passes the location of the breach, which is equal to the discharge through the breach at that very moment.

5. Relations have been established between the local discharge and the water level to compute the water level change for the reach downstream of the breach ΔH from ΔQ .
6. To determine the impact of a breach across bifurcations constant ratios appeared to be applicable for all discharges (ΔQ (Luoc)/ ΔQ (Red) = 0.475 and ΔQ (Thai Binh)/ ΔQ (Duong) = 0.3275).

3 Application of the method on the Bac Hung Hai polder

3.1 Base case

3.1.1 Failure mechanism

The base case represents the "current" situation of the Bac Hung Hai polder. This means no future measures of dike improvement are taken into account. We only consider the mechanism "wave overtopping". The fact that we only consider this failure mechanism is motivated by the fact that this is a preliminary analysis which has the purpose to demonstrate the applicability of the proposed probabilistic method.

The Z-function for wave overtopping is:

$$Z_{wave\ overtopping} = h_c - (h + h_{runup})$$

In which h_c is the crest level of the dike, h is the water level in the river and h_{runup} is the wave run-up. In other words: the dike fails if the waves that run up against the dike exceeds the crest level of the dike. In our studies, however we also consider a number of uncertainties. This means the z-function is extended to:

$$Z_{wave\ overtopping} = (h_c - h_s) - (h + h_{runup} + \Delta H_{model} + \Delta H_{fluctuations} + \Delta H_{trend})$$

where:

h	= water level
h_c	= crest level
h_{runup}	= wave runup
ΔH_{model}	= uncertainty in water level due to modelling errors
$\Delta H_{fluctuations}$	= uncertainty in water level due to fluctuations
ΔH_{trend}	= uncertainty in water level due to trends
h_s	= uncertainty in crest level due to settlements

In the probabilistic computations these are all random variables.

In the analyses the crest elevations of the dikes along the Luoc and Thai Binh have been increased with 1 m and hence do not represent the actual conditions. By doing so the suitability of the procedure can better be investigated as it eliminates largely breaches in areas with little system behavioural effects.

3.1.2 Resulting return periods of embankment failure for each location

The probabilistic procedure provides amongst others estimates of the annual probability of dike failure along the river system. For each location it counts the number of failures, i.e. the number of synthetic events for which the dike fails. Divided by the total number, N, of simulated years this gives the annual probability of failure. Figure 3.1 - Figure 3.4 show the results of this estimate (blue dots), expressed in terms of return periods of failure. These results were based on $N=10^6$ MC-samples.

Due to the relative simplicity of the considered failure mechanism (see previous section) we were also able to estimate the same return periods through numerical integration. The results of this additional computation are represented by the red lines in Figure 3.1 - Figure 3.4. A comparison of the results of the two methods (Monte Carlo and numerical integration) shows, that for locations with return periods $< 10^4$ years both methods provide the same results. For return periods $> 10^4$ years differences become increasingly larger with increasing return period. This is fully in line with the findings of Section 2.3.3: relative errors of the Monte Carlo procedure increase if failure probabilities decrease (i.e. return periods increase). For locations with return periods of, say, over 10^5 years we would thus require more MC samples than the current 10^6 to obtain a reliable estimate of the return period.

With respect to the expected annual damage we expect the number of 10^6 samples to be sufficient, because the locations with the relatively low return periods (high probability of failure) most strongly influence the expected damage. For these locations the estimated probabilities are very accurate as Figure 3.1 - Figure 3.4 demonstrate. In the remainder of this report we will constantly test if our applied number of MC-samples indeed suffices.

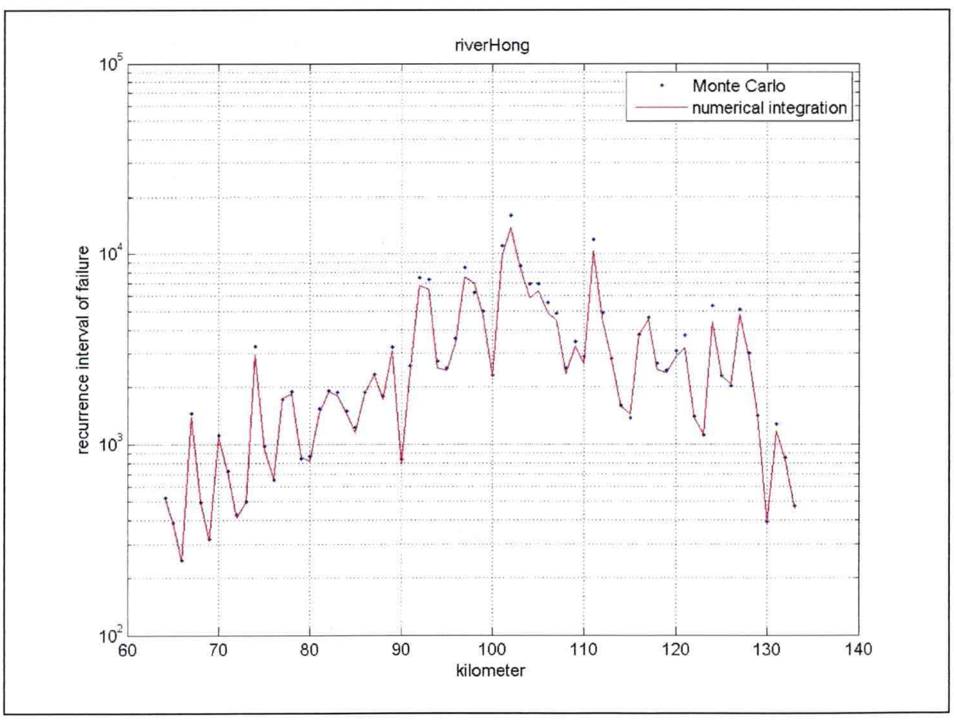

Figure 3.1 Estimated return periods of failure for locations along the Red river

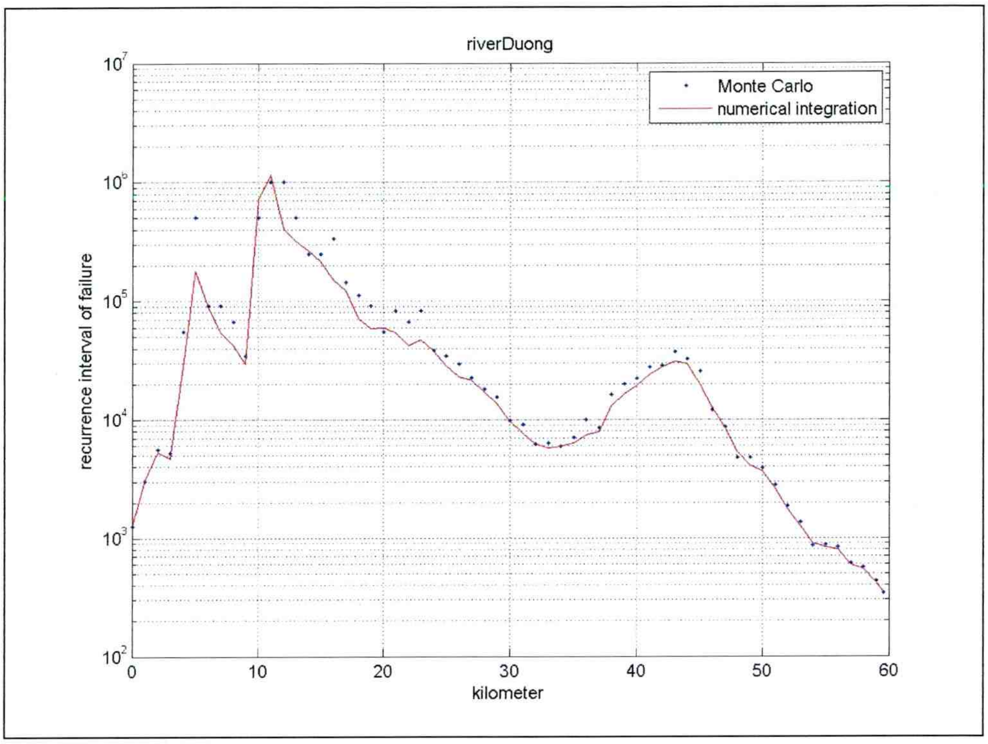

Figure 3.2 Estimated return periods of failure for locations along the river Duong

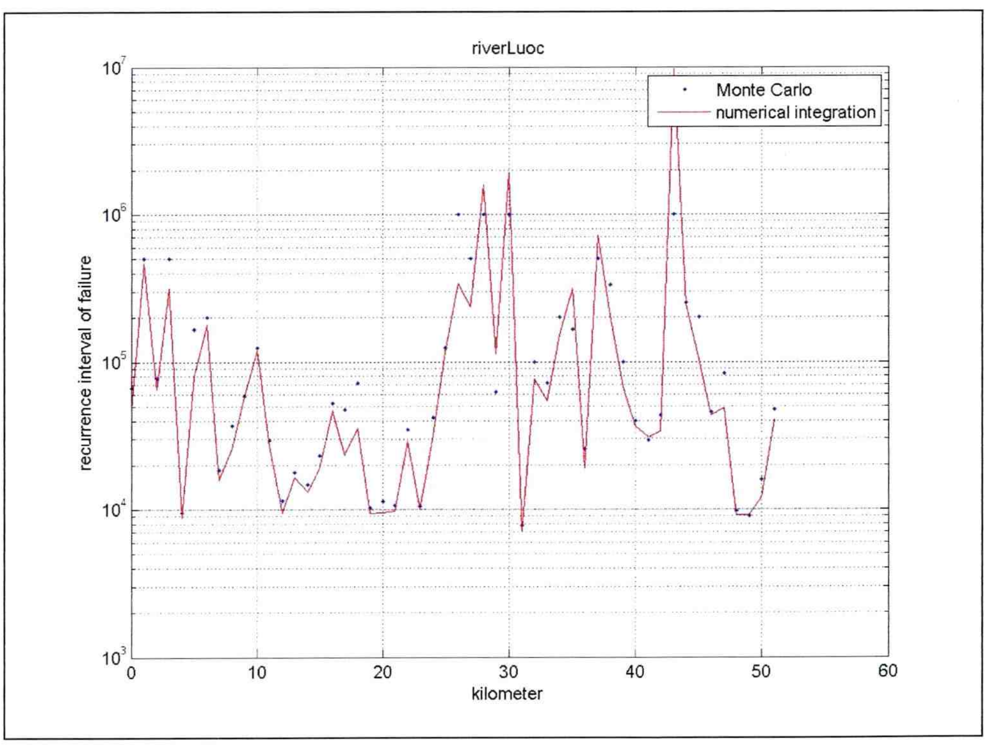

Figure 3.3 Estimated return periods of failure for locations along the river Luoc.

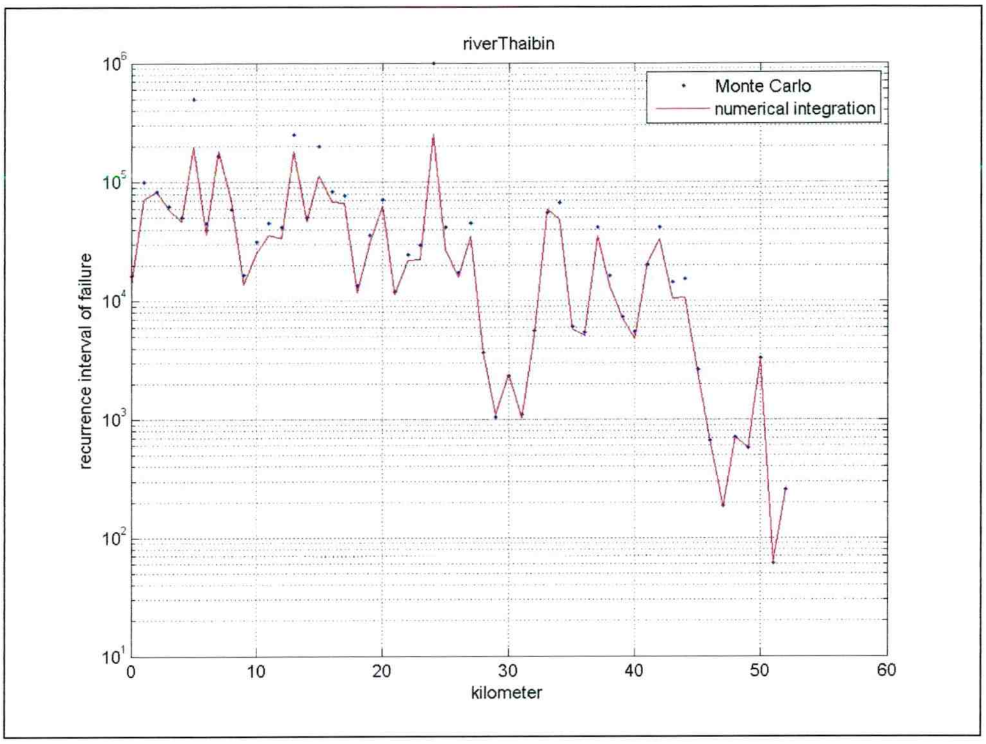

Figure 3.4 Estimated return periods of failure for locations along the river Thai Bin

3.1.3 Return periods of failure for each dike section

Figure 3.5 and Table 3-1 show the annual probability of failure for each dike section. Similar to Section 3.1.2 this is computed by counting the number of failures, i.e. the number of synthetic events for which one or more breaches in the dike section occur and dividing this number by the total number of simulated events. It shows that dike section 2 (see Figure 1.2 for an overview of the dike sections) has the highest probability of failure, followed by section 17 and 34. However, this does not necessarily mean that these are the dike sections that should get the highest priority with respect to dike improvement as we will show in the following sections.

The annual probability of failure of the Bac Hung Hay polder dikes as a whole is 0.035, which corresponds with a recurrence interval of 28 years.

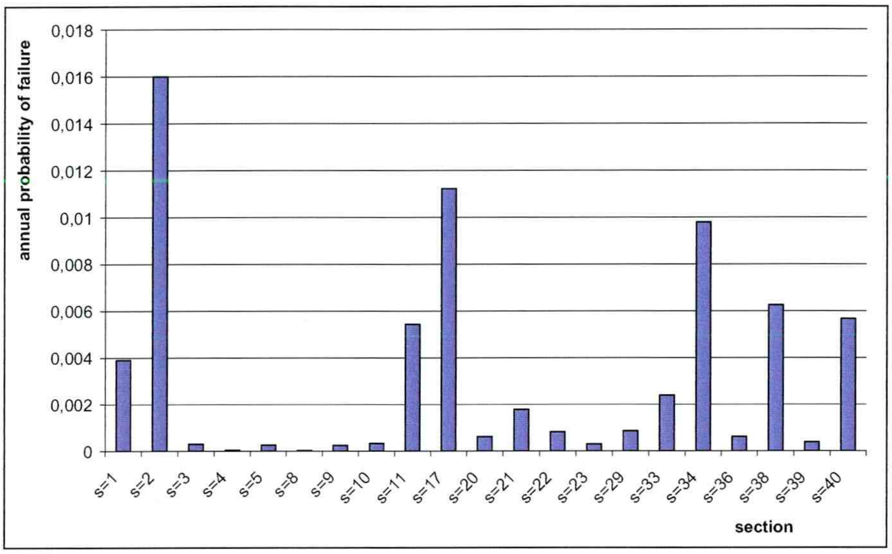

Figure 3.5 Annual probability of failure for each dike section.

Table 3-1 Annual probability of failure (P) and recurrence interval (T) for each dike section

dike section	P	T
1	3,89E-03	256
2	1,60E-02	62
3	3,14E-04	3,184
4	4,40E-05	22,727
5	2,71E-04	3,690
8	3,40E-05	29,411
9	2,50E-04	3,999
10	3,40E-04	2,941
11	5,43E-03	184
17	1,12E-02	89
20	6,27E-04	1,594
21	1,80E-03	557
22	8,20E-04	1,219
23	3,11E-04	3,215
29	8,67E-04	1,153
33	2,40E-03	417
34	9,80E-03	102
36	6,28E-04	1,592
38	6,26E-03	159
39	3,92E-04	2,551
40	5,66E-03	176

3.1.4 Expected Annual Damage

Figure 3.6 shows the results of 23 different MC-runs with 10^6 samples each. The average EAD of these runs is 12.1 million USD. The standard deviation is 0.1 million USD, which is considered to be accurate enough.

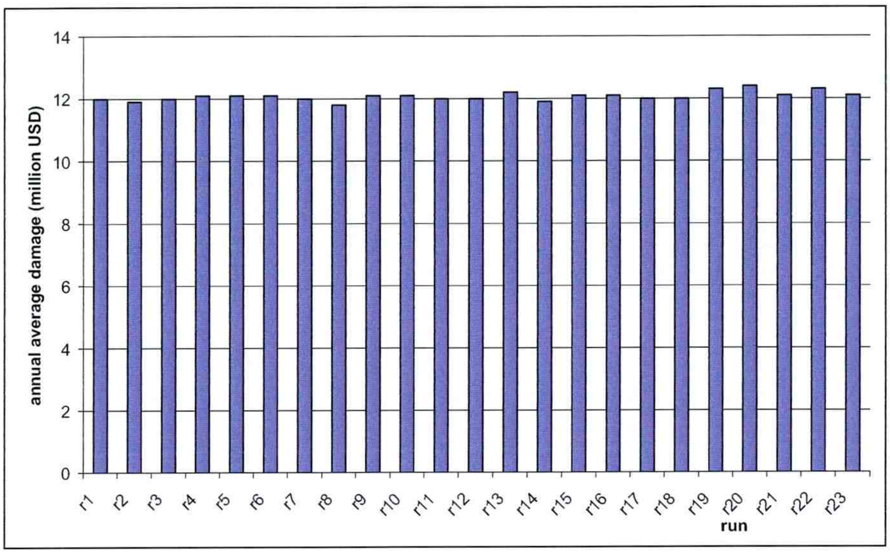

Figure 3.6 23 Monte Carlo estimates of the expected annual damage in million USD (**N = 10^6**).

3.2 Effects of dike improvement

The Expected Annual Damage (EAD) can be reduced by improvement of the dikes. With respect to the failure mechanism 'wave overtopping' this basically means an increase of the crest height. Naturally, it makes sense to (first) increase the dike sections that most influence the expected annual damage. In order to determine which dike sections should be improved first we applied and compared two different procedures, viz. maximum damage and maximum reduction of damage, to be discussed in the remainder of this section.

Procedure 1: maximum EAD contribution

In the first procedure we compute the EAD per section, i.e. the total damage in the Bac Hung Hai polder as a result of dike breaches along the various sections. Figure 3.7 and Table 3.2 show the EAD for each section. Note that the sum for all sections is equal to the total EAD (≈ 12 million USD/year) as derived in the previous section. Based on this figure one would advise to start improving dikes along section 40 since this section is associated with the largest damage. Second and third are sections 34 and 38.

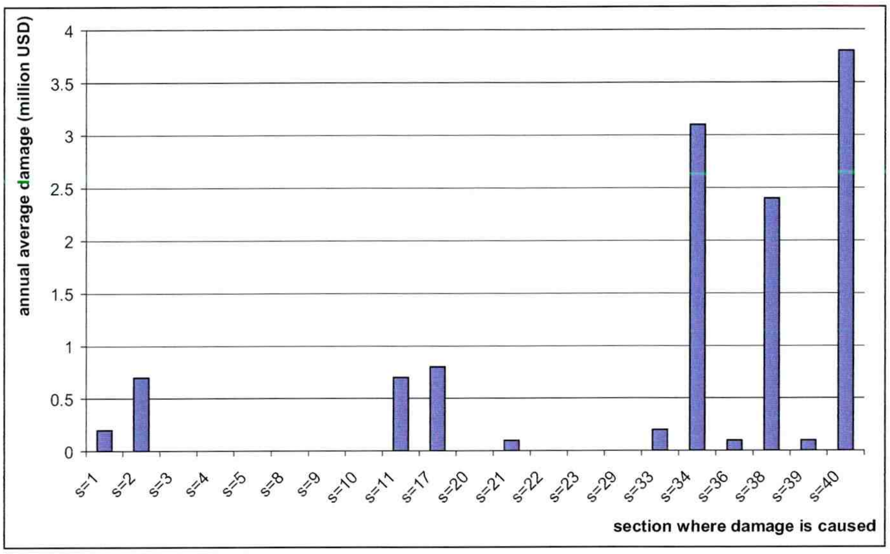

Figure 3.7 Expected Annual Damage in the Bac Hung Hai polder as a result of failure of dikes along the sections that are shown along the horizontal axis.

Table 3.2 Expected Annual Damage (in million USD) in the Bac Hung Hai polder as a result of failure of dikes along the sections that are shown in the first column.

Dike section	**EAD (million USD)**
1	0.2
2	0.7
3	0.0
4	0.0
5	0.0
8	0.0
9	0.0
10	0.0
11	0.7
17	0.8
20	0.0
21	0.1
22	0.0
23	0.0
29	0.0
33	0.2
34	3.1
36	0.1
38	2.4
39	0.1
40	3.8

Dike breaches along sections 34, 38 and 40 are responsible for about 75% of the total damage: approximately 9 million out of 12 million USD/year. However, if the dikes along these section are improved in such a way that hardly or no dike breaches occur, the EAD is not reduced by 75%! This is because the improvement of dikes along these three sections will result in an increase of the number of breaches along other sections. To demonstrate this, Figure 3.8 and Table 3.3 show the situation where the crest height of all dikes along sections 34, 38 and 40 have been increased by 1 m. Compared to the original situation (Figure 3.7), the total damage has decreased to approximately 6.5 million dollar, a reduction of almost 50%. The reduction of annual damages caused by breaches along sections 34, 38 and 40 is even more: over 8 million USD/year. This means the reduction of annual damages caused by breaches along the other sections have *increased* by more than 1.5 million USD/year.

This shows the importance of system behaviour. Because a dike breach leads to a reduction of the water level in parts of the river system and, consequently, to a reduction in the probability of dike breaches at other locations, a dike improvement at one location may *increase* the flood risk at other locations! In lay man's words: the repair of the weakest spot of the chain is only effective if there are no equally weak spots at other locations along the chain.

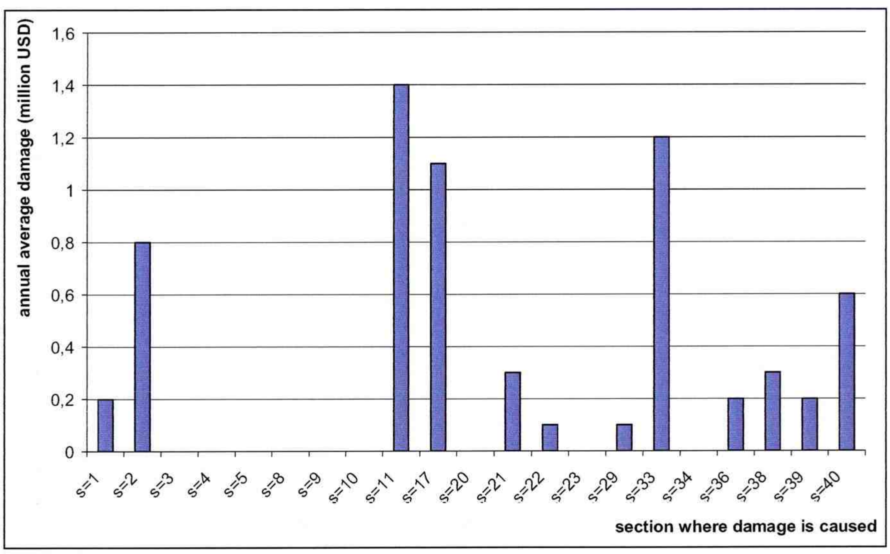

Figure 3.8 Expected annual damage in the Bac Hung Hai polder as a result of failure of dikes along the sections that are shown along the horizontal axis. In comparison with Figure 3.7 the crest height of dikes along sections 34, 38 and 40 have been increased with 1 m.

Table 3.3 Expected annual damage (in million USD) in the Bac Hung Hai polder as a result of failure of dikes along the sections that are shown in the column "after". In comparison with Table 3.2 (repeated in column "before") the crest height of dikes along sections 34, 38 and 40 have been increased with 1 m.

	EAD (million USD)		
Dike section	**after**	**before**	**diff.**
1	0.2	0.2	0.0
2	0.8	0.7	0.1
3	0.0	0.0	0.0
4	0.0	0.0	0.0
5	0.0	0.0	0.0
8	0.0	0.0	0.0
9	0.0	0.0	0.0
10	0.0	0.0	0.0
11	1.4	0.7	0.7
17	1.1	0.8	0.3
20	0.0	0.0	0.0
21	0.3	0.1	0.2
22	0.1	0.0	0.1
23	0.0	0.0	0.0
29	0.1	0.0	0.1
33	1.2	0.2	1.0
34	0.0	3.1	-3.1
36	0.2	0.1	0.1
38	0.3	2.4	-2.1
39	0.2	0.1	0.1
40	0.6	3.8	-3.2

Procedure 2: Maximum reduction of EAD

In the second procedure we increase the crest height of the dikes along a single section with 1 m and then (re-) compute the EAD. We repeat this step for each section. Note that in each computation only one dike section is 'improved' in comparison to the base case. Figure 3.9 and Table 3.4 show the outcome of this procedure for each section. We actually performed the whole procedure twice to test if the number of 10^6 samples is sufficient for this analysis. The relatively small differences between the two runs show that this is indeed the case.

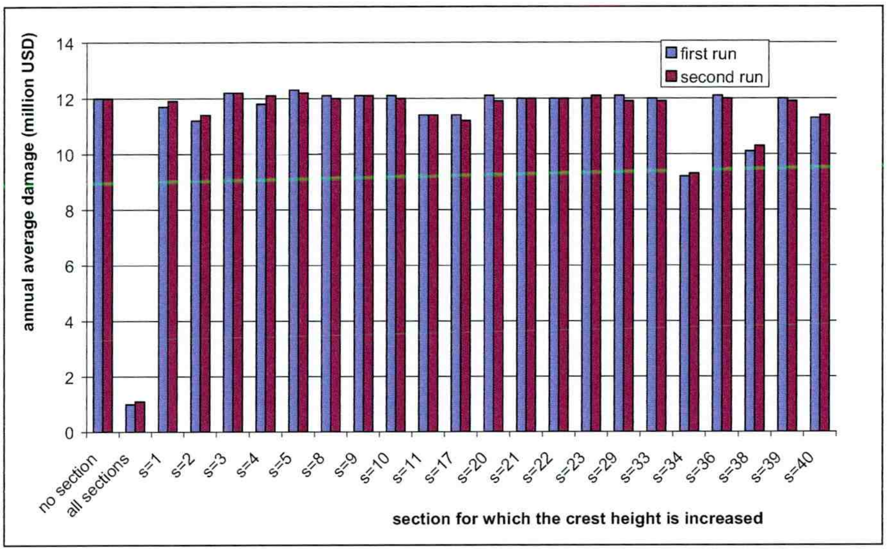

Figure 3.9 Expected Annual Damage in the Bac Hung Hai polder after increase of the crest height of dikes of a single section with 1 m. The horizontal axis shows the concerning section. The figure shows results of two different Monte Carlo runs.

Table 3.4 Expected Annual Damage in the Bac Hung Hai polder after increase of the crest height of dikes of a single section with 1 m. The first column shows the concerning section. The table shows results of two different Monte Carlo runs

	EAD per section (M$/yr)	
dike section	**first run**	**second run**
no section	12.0	12.0
all sections	1.0	1.1
1	11.7	11.9
2	11.2	11.4
3	12.2	12.2
4	11.8	12.1
5	12.3	12.2
8	12.1	12.0
9	12.1	12.1
10	12.1	12.0
11	11.4	11.4
17	11.4	11.2
20	12.1	11.9
21	12.0	12.0
22	12.0	12.0
23	12.0	12.1
29	12.1	11.9
33	12.0	11.9
34	9.2	9.3
36	12.1	12.0
38	10.1	10.3
39	12.0	11.9
40	11.3	11.4

Figure 3.9 shows that an increase in the crest height along section 34 leads to the largest reduction in the expected damage. So based on this approach one would advise to start improving dikes along section 34 first. This is different from the first procedure where dike section 40 was selected. Section 40 contributes most to the total EAD of the polder, but it is not the one that would result in the largest EAD reduction if strengthened. The fact that this is not the case is a typical consequence of system behaviour.

We will further clarify the differences in concept and results between procedure 1 and 2. Figure 3.10 - Figure 3.12 show graphs that are comparable with Figure 3.7 of procedure 1: the total expected annual damage per section. However, in these three figures the crest height of sections 34, 38 and 40 have been increased with 1 m. Figure 3.10 shows the case where the crest height of section 34 is increased, Figure 3.11 and Figure 3.12 show the cases for section 38 and 40 respectively. If we compare these figures with Figure 3.7 we see the effect of a dike improvement along a single section on the expected damages of *all* sections. Figure 3.13 summarises this comparison for sections 34, 38 and 40.

Figure 3.13 shows for example that in the current situation section 34 has an expected annual damage of 3.1 million USD. An increase of crest height along this section reduces this damage to 0. An increase of crest height along section 40 on the other hand leads to a slight increase in the expected damage of section 34: 3.3 million USD. This negative effect is even stronger for section 38: an increase of crest heights along section 40 leads to an increase in the expected damage of section 38 from 2.4 million USD tot 4.1 USD! The reason for this is quite straightforward: an increase of crest heights along section 40 makes that the directly downstream located section 38 becomes the weakest link along the Red river. This negative effect is the main reason why in the current procedure section 40 is not marked as the most effective dike section to be improved, whereas in procedure 1 it was.

This typically shows the benefit of a procedure that takes system behaviour into account.

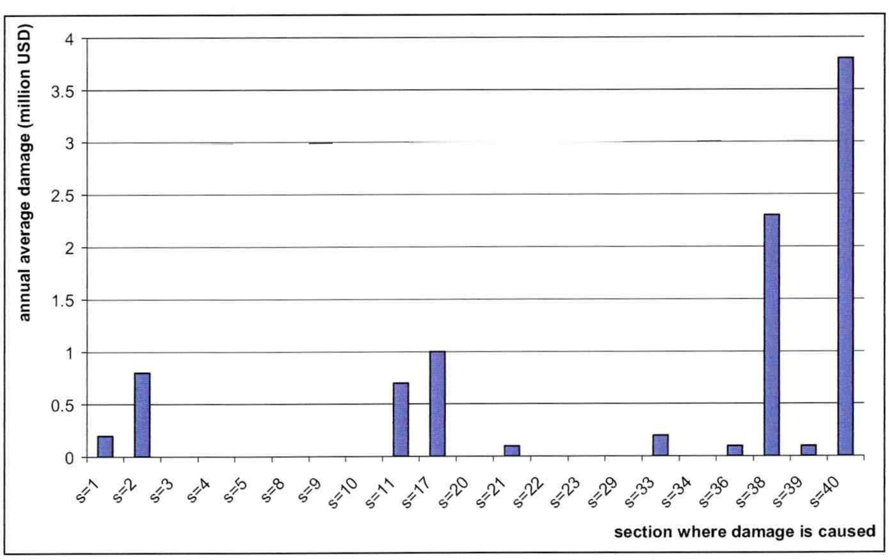

Figure 3.10 Expected annual damage in the Bac Hung Hai polder as a result of failure of dikes along the sections that are shown along the horizontal axis. Situation with increase of crest height (1 m) along **section 34**.

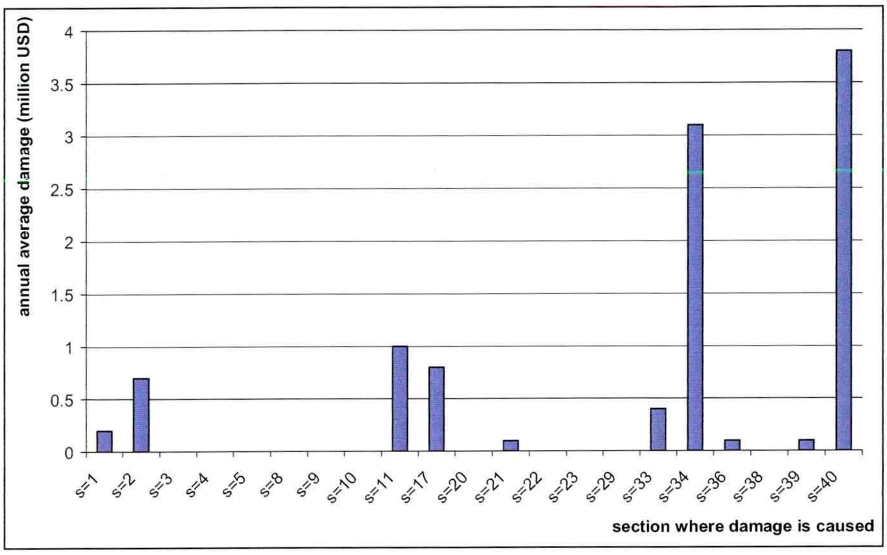

Figure 3.11 Expected annual damage in the Bac Hung Hai polder as a result of failure of dikes along the sections that are shown along the horizontal axis. Situation with increase of crest height (1 m) along **section 38**.

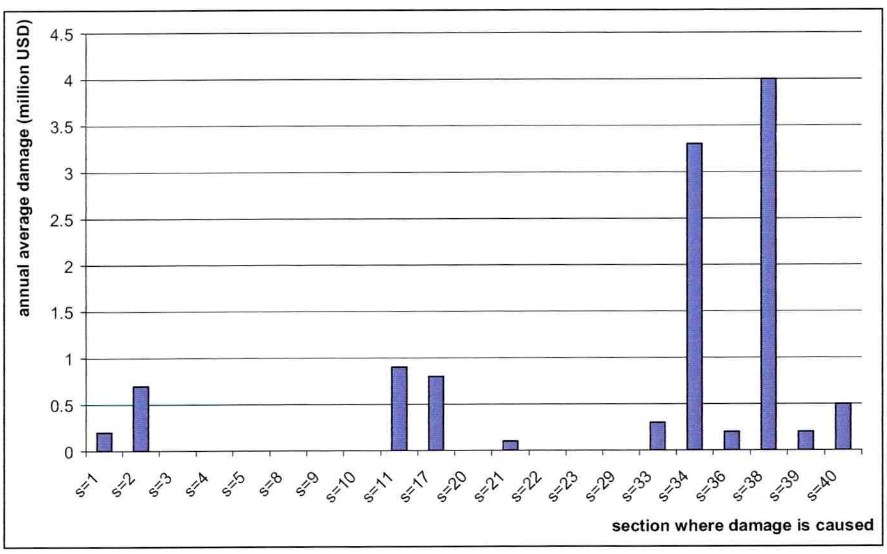

Figure 3.12 Expected annual damage in the Bac Hung Hai polder as a result of failure of dikes along the sections that are shown along the horizontal axis. Situation with increase of crest height (1 m) along **section 40**.

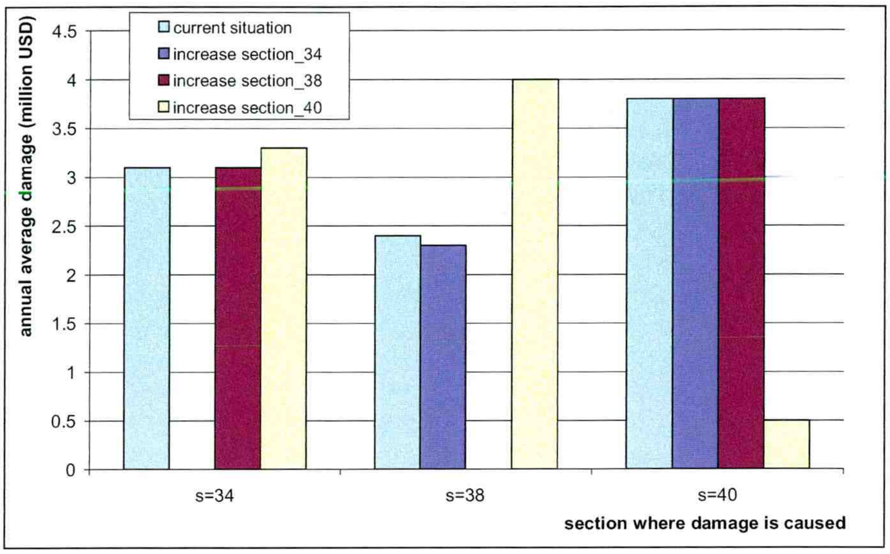

Figure 3.13 Expected annual damage in the Bac Hung Hai polder as a result of failure of dikes along the sections that are shown along the horizontal axis. Four different situations: current situation and situation with increase of crest height (1 m) along sections 34, 38, and 40.

4 Conclusions

A probabilistic approach, known as 'crude Monte Carlo', has been used to asses dike failure and flood damage with due account of system behaviour. This study has investigated the suitability of the approach to determine the sequence of improvements of dike segments surrounding the polder to reduce the risk of flooding in a most cost effective manner. The method has been applied to the Bac Hung Hai polder in the Red River delta in Vietnam.
The 'crude Monte Carlo' involves a large number of deterministic simulations of flood events, where for each individual simulation the random parameters and boundary conditions are sampled from their respective probability distribution functions. In the past the probabilistic model focused on determination of failure probabilities sec. These probabilities were subsequently combined with damages to arrive at average annual damage costs. Since the analyses were carried out for dike sections rather then for the whole ring-dike, the procedure led to double counting of damages and biases in the EAD's of the sections. Furthermore, no account was made of system behaviour, meaning that the effect of a breach at one location on the flows and water levels and hence on the failure probability at other locations was not considered.
In the approach presented in this study the result of each Monte Carlo simulation is flood damage, eliminating problems of double counting. Effects of dike breaches on the hydraulic conditions elsewhere (system behaviour) are incorporated. It leads to an improved assessment of dike failure at multiple locations in a Monte Carlo simulation. The study showed the importance of accounting for system behaviour to arrive at the best sequence for dike improvements. It was clearly shown that a sequence of improvements of dike segments, which follows the largest contribution to the overall EAD, does not lead to the best approach for the Bac Hung Hai polder. Due to system behaviour a different sequence proved to be more effective as improvement of one section may aggravate the flood risk at another.

The study demonstrated that this new approach clearly improves the understanding of flood risk. The method is easy to apply and provides practical outputs for best investments for dike improvements. The method will be very useful for optimization of the level of protection in river delta's like the Red River delta. Note that different protection levels around the polder may apply. To demonstrate this type of application, it is recommended to extend the investigation for Bac Hung Hai polder with cost-benefit analyses.

Where does the discussed method fit in the probabilistic computation methods for dike safety in the Netherlands? The following approaches can be distinguished, in increasing order of complexity:

1. Models that compute failure rates for single locations (Hydra-models);
2. Models that compute failure rates for single locations and dike rings (PC-Ring);
3. Models that compute failure rates for single locations and dike rings, using the system approach (Delft Cluster model, Bac Hung Hai model).

According to this ranking the Bac Hung Hai model is recognized to use a more complex approach than the Hydra-models and PC-Ring. However, the formulation of failure mechanisms in the Bac Hung Hai model is rather simple compared to the ones used in PC-Ring and the Hydra-models. On the other hand the Hydra-models only use a probabilistic description of the hydraulic loads, whereas PC-Ring, the Delft cluster model and the Bac Hung Hai model also use a probabilistic description of resistance parameters. It follows that the Bac Hung Hai model with the simple failure mechanisms provides an effective tool for a first assessment of dike safety and dike strengthening/rehabilitation planning.

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Rotterdamseweg 185
postbus 177
2600 MH Delft
telefoon 015 285 85 85
telefax 015 285 85 82
e-mail info@wldelft.nl
internet www.wldelft.nl

Rotterdamseweg 185
p.o. box 177
2600 MH Delft
The Netherlands
telephone +31 15 285 85 85
telefax +31 15 285 85 82
e-mail info@wldelft.nl
internet www.wldelft.nl