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Risk Assessment for the Ribe Area

COMRISK Subproject 7

THORSTEN PIONTKOWITZ, ANDREAS KORTENHAUS, HOCINE OUMERACI

Summary

Within COMRISK subproject SP7, a risk assessment of the coastal defence system in Ribe/Denmark has been performed based on the state-of-the-art of flood risk analysis methods. The flood risk has been defined as the product of the flooding hazard probability and the expected consequences of flooding. This paper describes the detailed hazard analysis by which the overall flooding probability of the Ribe defence system has been achieved. Results of sensitivity analyses regarding the assessment of uncertainties for input parameters and models used, the consideration of other special constructions such as one sluice and three outlets as well as an approach of dividing the defence system into 'homogenous' sections with respect to governing load and resistance parameters are also described.

Furthermore, the paper provides information about the vulnerability analysis, which has been performed to evaluate the consequences in case of flooding. Relevant damage categories, comprising vulnerable assets or non-material values, have been selected and valued together with the derivation of depth-damage functions for each damage category. Seven breach and inundation scenarios have been defined to assess the range of expected damage due to flooding in the flood prone area. Finally, risk values have been assessed.

Zusammenfassung

Innerhalb des siebten Teilprojekts von COMRISK wurde eine Risikoanalyse des Hochwasser- und Küstenschutzsystems (HuK-System) in Ribe/Dänemark durchgeführt. Das Überflutungsrisiko wurde hierbei definiert als das Produkt aus Versagenswahrscheinlichkeit des HuK-Systems und dem zu erwartenden potenziellen Schaden im Falle einer Überflutung. Dieser Beitrag beschreibt zunächst die detaillierte Gefahrenanalyse mit der die Gesamtversagenswahrscheinlichkeit des HuK-Systems in Ribe ermittelt wurde. Die Ergebnisse einer Sensitivitätsanalyse bezüglich der Abschätzung von Unsicherheiten der Eingangsparameter und Modelle, die Berücksichtigung von Sonderbauwerken (eine Schleuse und drei Auslässe), sowie die Einteilung des HuK-Systems in 'homogene' Abschnitte abhängig von maßgebenden Belastungs- und Widerstandsparametern werden ebenfalls beschrieben.

Zusätzlich wird die Vulnerabilitätsanalyse beschrieben, die durchgeführt wurde um die bedrohten Werte und die potenzielle Schädigung im Untersuchungsgebiet abzuschätzen. Hierzu wurden maßgebende Schadenskategorien, die Vermögensobjekte oder nicht materielle Werte umfassten, ausgewählt und Schadensfunktionen für jede Schadenskategorie hergeleitet. Sieben Versagens- und Überflutungsszenarien wurden definiert, um die Größenordnung des durch Überflutung verursachten potenziellen Schadens ermitteln zu können. Abschließend wurden szenarienabhängige Risiken bestimmt und beurteilt.

Keywords

Coast, risk management, flood defence, risk assessment, failure probabilities, vulnerability analyses, Ribe

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1. Introduction

More than 16 million people, corresponding to approx. 20 % of the total population in the North Sea region, live in coastal lowlands. Major economic activities, such as the seaports of Rotterdam, London and Hamburg or the tourist industry, have been increasingly concentrated in coastal regions over the last centuries. In addition, storm surges have increased over the last decades, both in frequency and intensity, increasing the hazard to the coastal regions. This has led to an increase of vulnerability to natural hazards within these regions.

Defence structures (e.g. sea dikes) to protect the flood prone areas have been designed by means of purely deterministic approaches or based on experience. Due to decreasing resources and increasing costs it is more and more desirable to optimise the cost-benefit-ratio for these structures (OUMERACI, 2004). Within the COMRISK subproject SP7, a risk assessment of the coastal defence system in Ribe/Denmark has been performed based on the state-of-the-art of flood risk analysis methods. The overall flooding risk has been defined as the product of the flooding probability P_f and the expected consequences of flooding $E(D)$.

The study has been performed in two major steps which comprises (I) the hazard analysis (calculation of the overall flooding probability) and the (II) the vulnerability analysis evaluating the expected consequences of flooding. Latter has been performed by the Danish Coastal Authority (DCA), being responsible for subproject SP7. The hazard analysis has been carried out together with the Leichtweiß-Institute for Hydraulic Engineering at the Technical University Braunschweig, Germany.

The overall aim of this paper is to describe the risk analysis of the Ribe area. For this purpose, the defence system and its components, the input parameters and their uncertainties as well as the limit state equations (LSE) and the probabilistic calculations will be described. Damage categories applied within the vulnerability analysis will be listed and their valuation will be explained together with the derivation of the depth-damage functions for each damage category. Seven breach and inundation scenarios are defined to assess the range of expected damage due to flooding in the flood prone area. Finally, the overall results will be critically discussed and remarks for future developments will be given.

2. Coastal Defence System in Ribe

The Ribe defence system is located about 50 km north of the German-Danish border at the Wadden Sea coast protecting approximately 95 km² low-lying flat marsh land surrounding Ribe town. Ribe is the oldest town in Denmark with about 9000 inhabitants. Three streams and a large river, Ribe Å, cross the flat marshland on their way towards their mouths

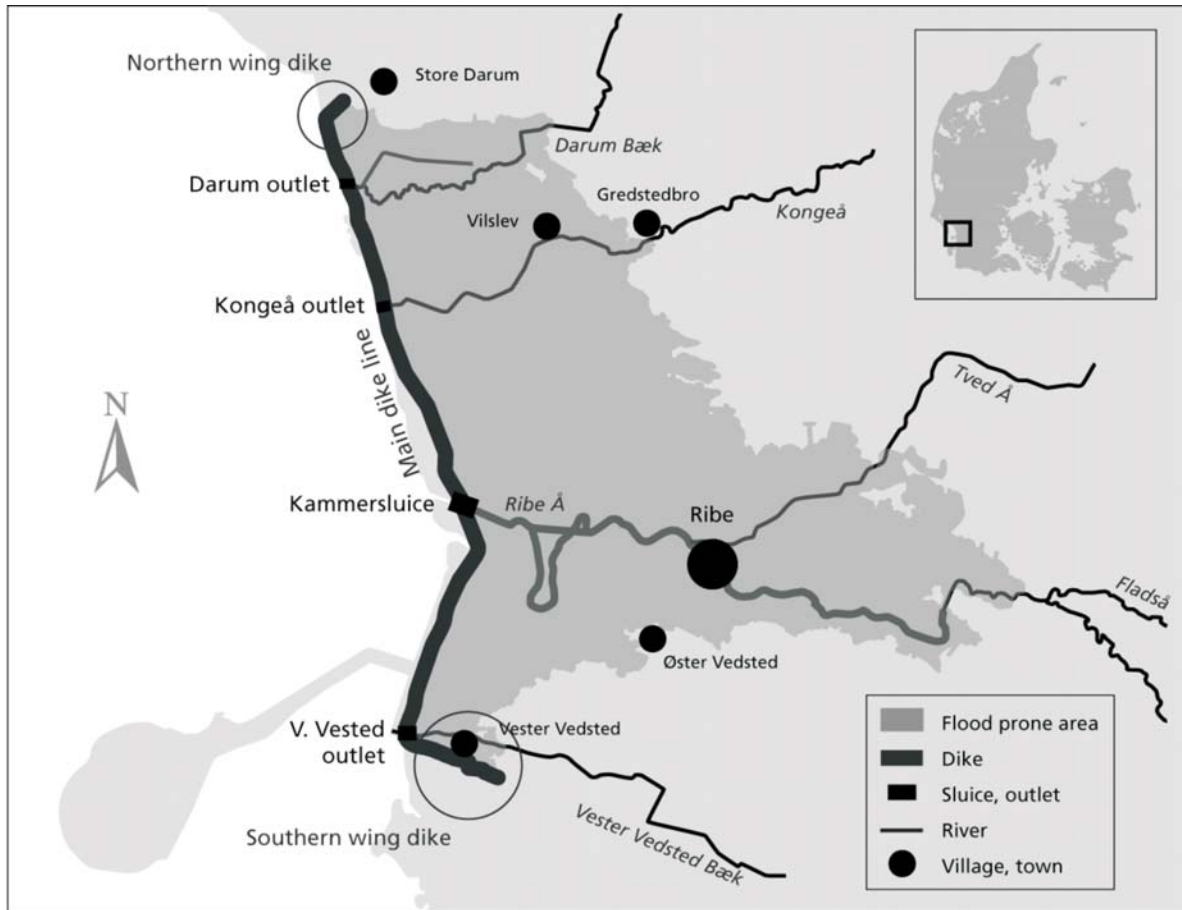


Fig. 1: Map of the Ribe flood defence system

(see Fig. 1). The river flows through Ribe town and passes a sluice shortly before reaching its mouth. The three streams pass the defence line through three outlets.

In this way, the 15.3 km long defence line consists of a main dike structure, a sluice and three outlets. The main dike is characterised by a sand core and a clay/grass cover. The standard profile shows a 1:10 seaward slope and a crown height of 6.88 m DVR90. The standard cross section and the key geometric parameters are shown in Fig. 2. The dike structure, the sluice and outlets are described in more detail in OUMERACI et al. (2004) and DANISH COASTAL AUTHORITY (2004).

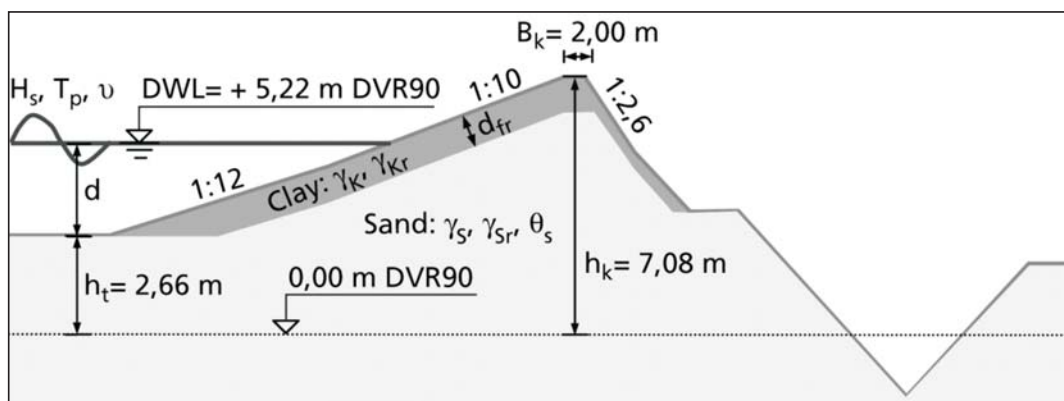


Fig. 2: Standard cross-section of the main dike in Ribe (km 6,644 as an example)

3. Hazard analysis

For the deterministic and probabilistic calculations within the hazard analysis, the model by Kortenhaus (2003) for sea dikes has been used. It comprises 25 failure mechanisms with a total number of 87 input parameters. The input parameters were grouped into parameters describing (I) the geometry of the structure, (II) the hydromechanic boundary conditions, and (III) the geotechnical properties of the structure.

3.1 Input parameters

I. Geometrical parameters

Topographic measurements were available for six cross sections along the main dike line which were regarded as the weakest points of the main dike. Thus, it was concluded that analysing these profiles would account for the potential weak spots of the dike.

The crest heights of the main dike were taken from available measurements performed in different years. The dimensions and constructional details of the sluice and the outlets were taken from technical drawings which were made available by the DCA.

II. Hydraulic boundary conditions

The design water level for all cross sections, the sluice and the outlets is a pre-described value which was determined from measurements at the Danish coast. It is defined as $h_w = 5.22$ m for a 200-year return period.

The input parameters for wave height and wave periods resulted from a study performed by Danish Hydraulic Institute (DHI). The offshore input parameters for this study were given by DCA so that 21 simulations with different input parameters were performed. The results of these runs are given as wave heights H_{m0} and wave periods T_m for specific points along the coastline (100 m distance) for water depths corresponding to 50 m and 300 m distance to the toe of the dike, respectively.

Angles of wave attack were based on instructions given by the DCA and corresponded to the most unfavourable conditions for the specific cross sections. The duration of design storm surge was assumed to be constant as $t_s = 6.5$ h.

III. Geotechnical parameters

All geotechnical parameters like the shear strength of the clay were predefined by the DCA. This information was based on geotechnical investigations which were performed close to the six cross sections during reinforcement works in 1980.

3.2 Deterministic calculations

3.2.1 Limit state equations

For deterministic calculations of the dike sections a total number of 23 failure mechanisms were considered. Due to the lack of stone revetments at the Ribe main dike all failure mechanisms associated with stone revetments were not considered for the calculations. Fig. 3 gives an overview of failure mechanisms of a sea dike considered in the method by KORTENHAUS (2003).

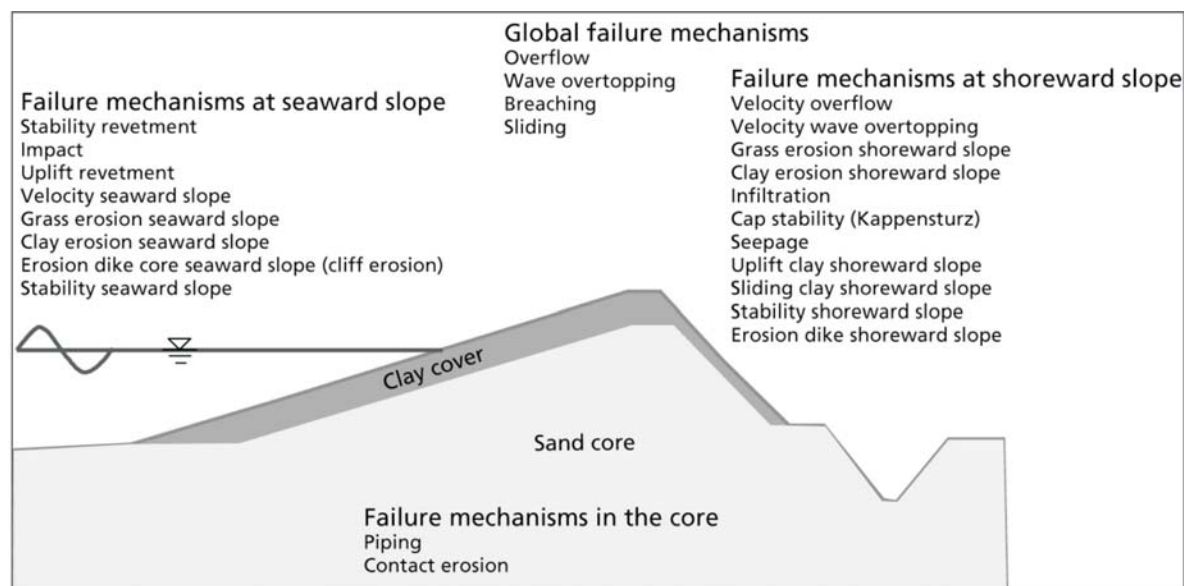


Fig. 3: Overview of failure mechanisms of a sea dike considered in the method after KORTENHAUS (2003)

For deterministic calculations of the sluice, the failure mechanisms described in KORTENHAUS (2003) could not be used. Therefore, OUMERACI et al. (2004) have formulated further limit state equations (LSE) describing e.g. the stability of the gates, piping underneath the sluice, wave overtopping over the gates and human error.

The failure mechanisms for the Ribe outlets are a combination of the failure mechanisms for the sluice and those for the dike sections. This combination was chosen since the outlets are partly sluices (walls, gates, etc.) but also show the characteristics of a dike (slope, grass cover, crest, etc.). Details of this approach are described in OUMERACI et al. (2004).

3.2.2 Calculation of dike sections

A deterministic calculation for all failure mechanisms described in section 3.2.1 was performed for all six dike sections showing that the failure mode “erosion of grass cover at the seaward side” will lead to failure for all cross sections. This means that for a design water level of $h_w = 5.22$ m and a storm surge duration of $t_s = 6.5$ h the grass cover both at the seaward and shoreward side of the dike will fail.

3.2.3 Calculations for sluice and outlets

The results of the deterministic calculation of the sluice and the outlets showed that there is failure for the LSE “wave overtopping” under design conditions. Consequently, there is also a total failure (inundation of flood prone areas) for the sluice and the outlets. The safety coefficients for wave overtopping and overflow are significantly lower as compared to the dike cross sections. A reason for this observation is that the seaward slope is much steeper in the case of the outlets. Furthermore, the water depth in front of the structures is significantly larger (up to 5.0 m) than in front of the dikes, thus allowing higher waves to occur at the structures.

3.3 Uncertainty of input parameters

Uncertainties indicate the variation of parameters around their mean values. They can be estimated using either a full statistical distribution or a mean value and a standard deviation assuming a Normal distribution. For some parameters, which were considered to be important for the failure mechanisms (a sensitivity analysis was performed beforehand), the uncertainties were evaluated in more detail (water level, dike height, wave height and wave period, geotechnical parameters, model uncertainties). Tab. 1 shows the results of the uncertainty evaluations used in this study.

Tab. 1: Overview of uncertainties of most relevant parameters

Parameter	Uncertainty	Restriction	Remarks
Crown heigh h_k	Sdev = 0,10 m	–	Uncertainty in measurements
Water level h_w	Sdev = 0,47 m	–	Extreme statistics from available measurements
Wave heigh H_s	CoV	$H_s = 0,55 \cdot d$	Breaker criterin, d = local water depth
Wave period T_p	CoV = 0,20	$T_p = (H_s/0,0938)^{0.5}$	Restriction by wave steepness

3.4 Probabilistic calculations

In a first step, the failure probability P_f of each mechanism was determined. Level II (FORM) or Level III (Monte Carlo Simulation) calculations were performed depending on the complexity of the limit state equations. The calculated failure probability is given as P_f/year . Failure probabilities smaller than $P_f = 1 \cdot 10^{-10}$ were taken as $P_f \cdot 0$ and were ignored for subsequent calculations.

To calculate the temporal dependencies of the failure mechanisms, a scenario approach proposed by KORTENHAUS (2003) was used. In the scenario approach, the chronology of time-dependent failure mechanisms was achieved by defining “scenario blocks”, which comprise several individual failure mechanisms in logical and temporal order.

3.4.1 Calculation of dike sections

In Tab. 2 an overview of all results for the probabilistic calculation of all dike cross sections and all individual failure mechanisms is given. The failure probability for breaching is extremely high since the LSE does not consider the temporal development of the erosion process. The same missing temporal dependency accounts for the failure mechanism ‘grass erosion seaward slope’. The very high failure probability does not have a significant importance for the overall failure probability since it only represents the start of the erosion process at the seaward slope.

Therefore, in the following the results of the calculation for a scenario approach (inc. temporal relations between failure mech.) and the related fault tree analysis will be discussed.

A simplified 'scenario fault tree' for dike cross section km 8,422, ignoring all branches in the fault tree with $P_f < 10^{-10}$, is shown in Fig. 4. The failure probabilities for the scenarios were calculated using the Level III approach (Monte-Carlo simulation). The results are summarised in Tab. 3.

Tab. 2: Overview of failure probabilities for all failure modes of all dike cross sections

No.	Failure mechanism	Dike section					
		3156	6644	8422	9400	10403	14499
Global failure mechanisms							
1	Overflow	1,0E-06	2,0E-07	2,3E-06	1,0E-0,6	3,4E-06	5,0E-07
2	Wave overtopping	3,0E-05	0,0E-06	4,1E-05	3,5E-05	6,6E-05	9,0E-06
3	Breaching	4,3E-02	1,8E-02	7,4E-02	4,2E-02	8,9E-02	3,6E-02
4	Sliding	3,4E-07	3,3E-07	4,1E-07	3,3E-07	3,5E-07	3,4E-07
Failure mechanisms at the seaward slope of the dike							
6	Impact	8,0E-06	5,0E-06	2,0E-06	4,0E-06	7,0E-06	8,0E-06
8	Velocity seaward slope	2,2E-02	1,8E-02	3,4E-02	1,9E-02	3,4E-02	3,2E-02
9	Crass erosion seaward slope	2,9E-01	2,4E-01	6,8E-01	2,6E-01	3,3E-01	3,0E-01
10	Clay erosion seaward slope	1,3E-05	5,6E-05	3,6E-05	1,3E-05	3,7E-05	4,6E-05
11	Erosion dike core seaward slope	1,7E-05	7,6E-05	4,8E-04	1,1E-04	3,2E-04	5,6E-04
12	Stability seaward slope	0,0	0,0	0,0	0,0	0,0	0,0
Failure mechanisms at the shoreward slope of the dike							
13	Velocity overflow	2,0E-06	3,0E-06	3,0E-06	2,0E-06	2,0E-06	0,0E+00
14	Velocity wave overtopping	2,6E-05	3,3E-05	1,4E-04	1,2E-05	1,9E-04	2,2E-05
15	Grass erosion shoreward slope	1,6E-04	1,0E-04	5,7E-04	8,5E-05	6,9E-04	1,2E-04
16	Clay erosion shoreward slope	6,3E-05	1,6E-05	6,6E-05	2,3E-05	7,6E-05	1,7E-05
17	Infiltration	0,0	8,0E-06	2,1E-04	1,0E-06	0,0E+00	1,6E-04
18	Kappensturz	1,4E-02	1,1E-02	7,6E-03	2,3E-02	4,1E-03	1,4E-02
19	Seepage	1,0E-06	1,0E-06	1,0E-06	2,0E-06	0,0E+00	1,0E-06
20	Uplift clay on shoreward slope	1,0E-06	2,0E-06	1,0E-06	1,0E-06	0,0E-06	0,0+00
21	Sliding clay shoreward slope	4,1E-04	5,4E-04	1,4E-04	1,3E-03	1,4E-03	2,4E-04
22	Stability shoreward slope	0,0	0,0	9,6E-05	0,0	0,0	0,0
23	Erosion dike shoreward slope	0,0	0,0	3,2E-05	3,0E-06	7,0E-06	0,0
Failure mechanisms in the dike							
24	Piping	9,6E-07	2,0E-06	2,0E-06	6,8E-07	1,1E-06	5,4E-7
25	Matrix erosion	2,5E-01	1,4E-01	2,7E-02	2,6E-02	3,8E-03	2,8E-04
Overall failure		3,1E-05	9,2E-06	4,3E-05	3,6E-05	6,9E-05	9,5E-06

Tab. 3: Failure probability of scenarios for all dike cross sections of the Ribe sea defence

No.	Individual mechanisms (see Tab. 4-7 for def.)	Dike cross section					
		3156	6644	8422	9400	10403	14499
Sc 1	9+10+11+3	3,0E-06	6,0E-06	3,5E-05	1,4E-05	3,4E-05	7,0E-06
Sc 2	11+3	3,0E-05	2,6E-04	1,1E-03	6,9E-05	4,9E-04	7,7E-04
Sc 3	15+16+18+3	0	0	0	0	0	0
Sc 4	17+21+18+3	0	0	0	0	0	0
Sc 5	19+20+21+18+3	0	0	0	0	0	0
Sc 6	15+16+23+3	0	0	1,5E-05	0	4,0E-06	0
Sc 7	17+21+23+3	0	0	0	0	0	0
Sc 8	19+20+21+23+3	0	0	0	0	0	0
Sc 9	23+3	0	0	3,8E-05	0	9,0E-06	0
Sc 10	19+24+23+3	0	0	0	0	0	0
Sc 11	19+25+23+3	0	0	0	0	0	0
Overall failure		3,10E-05	9,6E-06	4,50E-05	3,70E-05	7,10E-05	1,00E-05

For most of the scenarios the overall failure probability resulted in $P_f \cdot 0$. Scenario I comprises the grass erosion, the clay erosion, the cliff erosion and the breaching of the dike and Scenario I + II (cliff erosion and breaching) result in failure probabilities larger than $P_f = 1 \cdot 10^{-10}$. For cross section km 8,422, Scenario I gives $P_f = 3.5 \cdot 10^{-5}$. In comparison to the traditional fault tree approach where temporal relations of the failure modes are not considered sufficiently, the failure probability for the seaward side of the dike was $P_f \sim 5 \cdot 10^{-8}$, i.e. three orders of magnitude smaller.

The increased failure probability for the seaward slope including the grass erosion now comes much closer to the failure probability for wave overtopping so that the erosion process gets increasingly important for dikes investigated here.

Overall, the flooding probabilities using the scenario approach results in 3–5 % higher values for four cross sections (6644, 8422, 9400 and 10409). All other cross sections do not seem to be affected by the use of a scenario approach. This result is only valid for the Ribe case since the failure probability for wave overtopping is rather high in this case.

The influence of mean values of some key input parameters on the failure probability of individual mechanisms, scenarios and the overall flooding probability has been investigated. An increase, for example, of the mean water level hw by 0.50 m yields an increase of the failure probability for wave overtopping by a factor of 10 and a reduction by 0.50 m results in a lower failure probability by a factor of 10. The overall flooding probability is changed by the same order of magnitude. Moreover, the influence of mean values of further parameters on the failure probability for wave overtopping was investigated in detail (wave overtopping was selected since it has the largest influence on the overall flooding probability). In this connection, the key parameters, which affect the failure probability for wave overtopping and the overall flooding probability most, were determined as the water level and the wave period. Uncertainties for both parameters should therefore be evaluated very carefully.

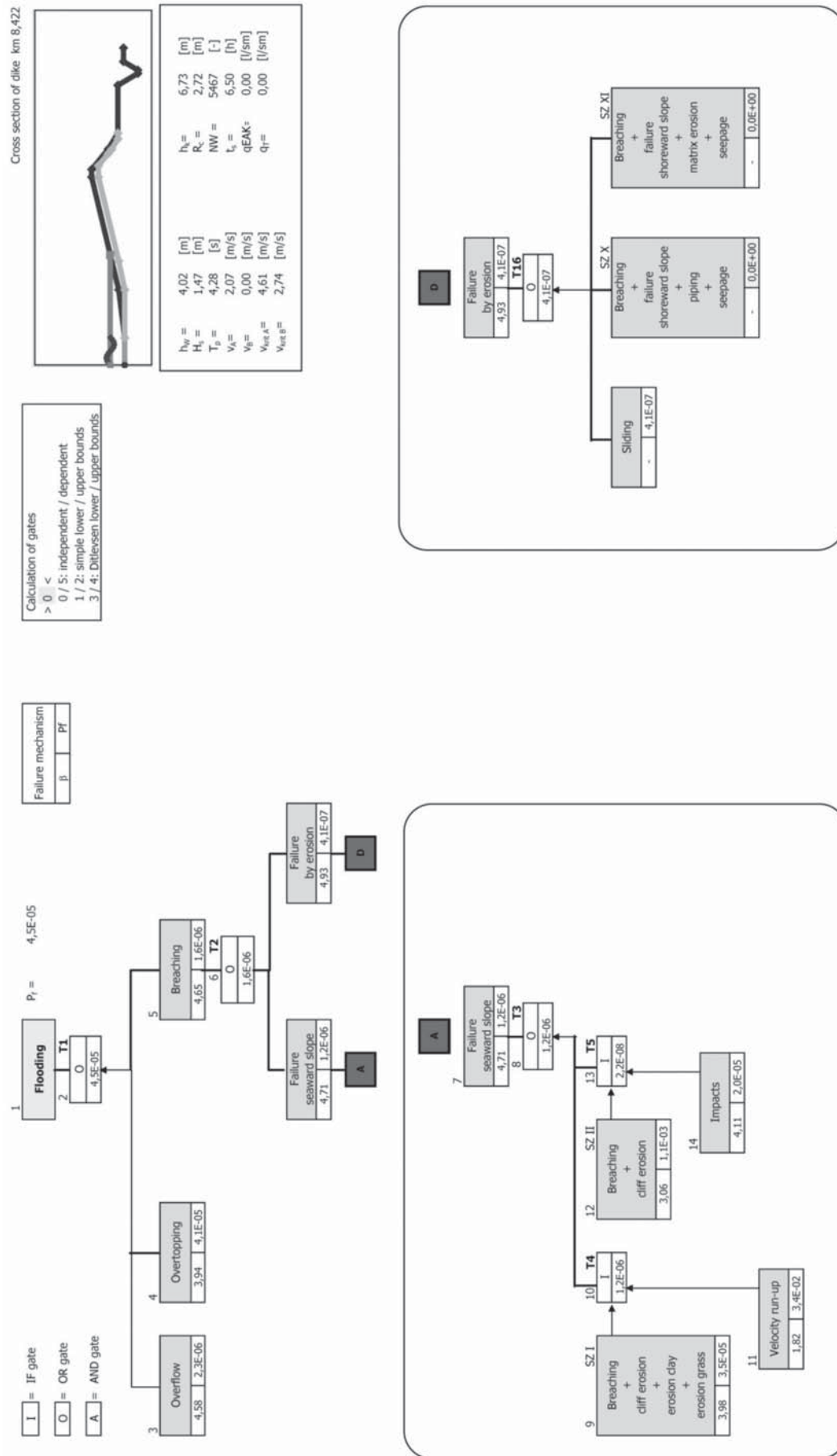


Fig. 4: Simplified scenario fault tree for dike cross section km 8,422

3.4.2 Calculation of the Ribe sluice and outlets

The failure probability of the Ribe sluice and the outlets was calculated using the Monte-Carlo method. The results are shown in Tab. 4.

The failure probabilities for the outlets are all in the range of $P_f \cdot 5 \cdot 10^{-1}$, which means that flooding occurs once every 2 years, approximately.

Calculations showed, that the key failure mechanism for all structures is wave overtopping where a tolerable wave overtopping rate of $q_{tol} = 20$ l/(sm) has been assumed. Variations of the tolerable overtopping rates ($q_{tol} = 100$ l/(sm), 200 l/(sm) and 515 l/(sm) where the latter corresponds to the overtopping rate for zero free-board as the maximum possible value were performed to study its influence on the results.

Increasing the tolerable wave overtopping rate to 100 l/(sm) and 200 l/(sm) results in a decrease of the failure probability for wave overtopping by a factor of about 10 and 100 for all outlets and the sluice, respectively. If the tolerable wave overtopping rate is set to 515 l/(sm), the failure probability for wave overtopping will be in the range of 10^{-4} for the sluice and 10^{-5} or 10^{-6} for the outlets.

Tab. 4: Results of probabilistic calculations for the sluice and the three outlets (selected failure mechanisms)

No.	Failure mechanisms	Kammer sluice	V.Vedsted	Outlets Konge Å	Darum
1	Overflow	5,3 E-02	1,5E-04	2,5E-04	7,9E-05
2	Wave overtopping	6,1E-01	5,6E-01	4,7E-01	4,9E-01
3	Hydraulic heave	1,0E-10	1,0E-10	1,0E-10	1,0E-10
4	Gates not closed	1,2E-03	1,0E-04	1,0E-04	1,0E-04
Overall failure		6,3E-01	5,6E-01	4,7E-01	4,9E-01

3.5 Failure probability of flood defence system

In order to determine the overall flooding probability of the Ribe defence system, a division into several dike sections was made. The division into several dike sections assumed for one section very small variation of the input parameters for either the stress or the resistance of the limit state equations. The crown height h_k of the dike and the peak wave period T_p were finally selected as the key criteria for the division of the defence system into 'homogenous' dike sections.

In all, 15 sections were defined as shown in Fig. 5. Four sections were defined to be close to section km 6,644, two sections close to km 14,499 whereas all other sections differ from each other. Section 4 in Fig. 5 could not be assigned to one of the investigated cross-sections since the crest height $h_k = 7.53$ m was significantly higher than for the other sections. This section was ignored for the subsequent probabilistic calculation. The sluice and the three outlets were defined as separate sections.

The failure probabilities of all sections (dike sections, sluice outlets) were linked to each other by means of a fault tree with just one OR gate, which was used to calculate the overall flooding probability of the hinterland.

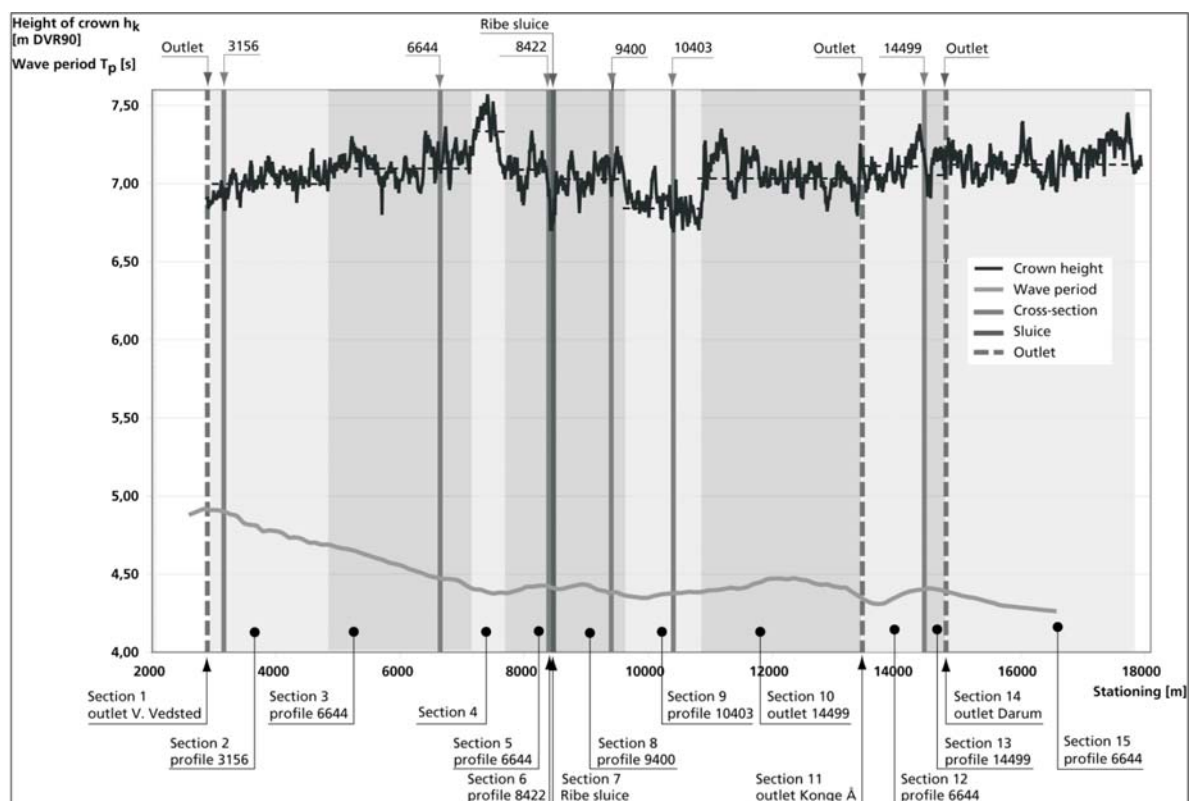


Fig. 5: Division of the Ribe flood defence system into representative sections

The fault tree calculations including all sections resulted in an overall flooding probability of $P_f = 9.5 \cdot 10^{-1}$. This result was considered to be much too high since it solely depends on the failure probabilities of the sluice and the outlets which are mainly governed by wave overtopping. Due to small inflow volumes during wave overtopping (limited stretch of sluice/outlets), a second calculation ignoring sluice and the outlets resulted in an overall flooding probability of $P_f = 2.5 \cdot 10^{-4}$.

Despite the fact that the sluice and the outlets are very narrow structures, calculations showed that the sluice and the outlets are the weakest elements in the defence line. An example calculation showed that flooding from wave overtopping over the sluice would result in a water level in the flood prone area of less than 1 mm only. However, structural failure of the sluice or one of the outlets would cause major flooding of the hinterland. It is therefore essential to investigate the sluice and the outlets in more detail to finally determine the overall flooding probability. For the time being it is recommended to use a flooding probability of $P_f = 2.5 \cdot 10^{-4}$ for the flood defence system in Ribe.

4. Vulnerability analysis

As cartographic basis for the vulnerability analysis, altitude data in a grid net of 25x25 metres was used to generate a topographical map of the flood-prone area, being delimited by the 5.0 m DVR90 altitude line. The altitude data was supplemented by altitude data from road surveys. Fig. 6 presents the topography of the Ribe flood-prone area, showing low-lying delta areas surrounding the watercourses far into the hinterland.

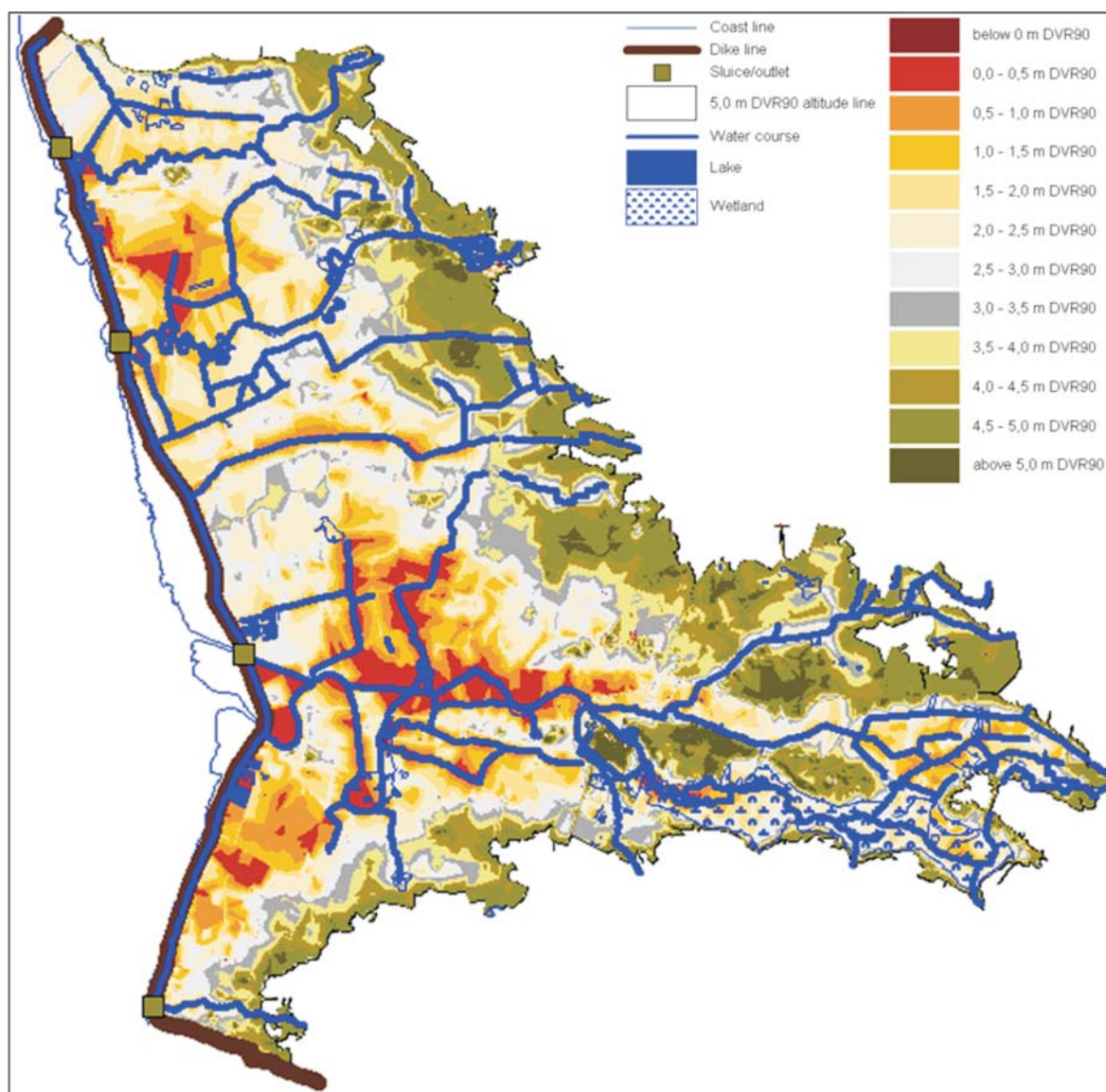


Fig. 6: Topography of the Ribe flood-prone area

4.1 Damage categories and data sources

Within the flood-prone area of Ribe six categories of direct, tangible damage were selected (buildings, movable property, agricultural acreage, livestock, electric installations, traffic system). Additionally, four damage categories (inhabitants, employees, vehicles, tourism) subject to intangible, direct/indirect damage were considered in a descriptive form. Typically, data was available at national registers, such as the Building and Housing register or the Central Livestock register. In other cases, data was provided by research centres or the responsible county. The request of data from national registers or public administrations about the damage categories showed however clear differences in data quality and format. This fact complicated the procedure of geocoding each risk element by means of a GIS software application.

4.2 Valuation analysis

The valuation analysis showed the location of most of the assets on high ground around the low-lying delta area of Ribe River. For example, only 7 % of the accumulated property value is located below 2.5 m DVR90. About 45 % of the accumulated property value is placed below 4.0 m DVR90 and about 30 % of the total property value is located between 4.5 and 5.0 m DVR90. Fig. 6 illustrates the total property value of buildings distributed over ten altitude intervals.

This distribution of assets over altitude has been characteristic for most of the damage categories. However, a differentiation of the total profit of all kinds of crop over altitude showed an almost linear distribution, which differed remarkably from the other damage categories.

4.3 Damage analysis and inundation scenarios

To determine the possible damage for each damage category, seven breach and overtopping scenarios were defined. In five scenarios inundation occurs due to one or more dike breaches, whereas two scenarios consider wave overtopping and the structural failure of both gates of the sluice, respectively. The seven scenarios are defined as follows (sections as referred to in Fig. 5):

Sc1: one dike breach in section 6

Sc2: one dike breach in section 2

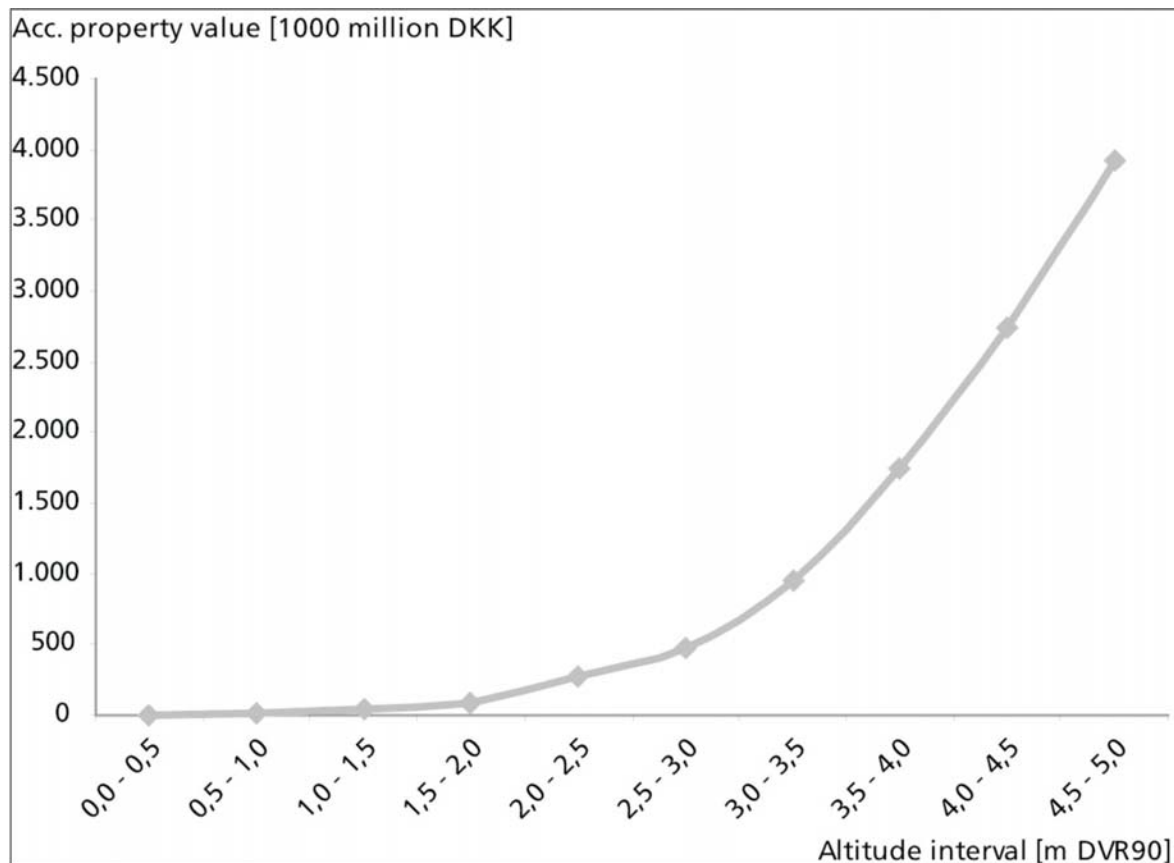


Fig. 7: Total property value distributed over altitude intervals

Sc3: one dike breach in section 9

Sc4: wave overtopping in section 9

Sc5: three dike breaches in sections 2, 6, and 9

Sc6: four dike breaches in sections 2, 6, 8, and 9

Sc7: failure of both gates at the sluice

Depth-damage functions were derived for each damage category where the damage depends on the inundation depth. In case of depth-independent damage, damage factors were derived to quantify the damage.

For buildings and movable property depth-damage functions could be derived from data about compensation payments regarding real flood damage to buildings and movable property in Denmark. The assessment of flood damage to agricultural acreage was performed by external experts. Their assessment comprised damage factors for different kinds of crop and inundation periods of 5, 14 and 28 days.

Based on the seven scenarios, inflow volumes between 0.5 and 127 million m³ were calculated. Input parameters, such as a standardised storm surge hydrograph, the failure probability of defence system sections, the time-dependent development of a dike breach as well as an assumed time of failure during storm surge were considered in the calculations of the inflow volumes. Based on these input parameters, the flood-prone area is differently inundated depending on the location and the number of failure events.

Due to differences in inundation behaviour, damage within each scenario varies between 1.15 and 424.5 million DKK (56.9 million €). Only the scenarios Sc5 and Sc6 resulted in damage exceeding 100 million DKK (13.4 million €). The scenarios Sc1, Sc2 and Sc7 showed comparable inundation behaviour and resulted in the same total damage for all three scenarios. Tab. 5 gives the final results of the calculated damage for the seven scenarios.

Tab. 5: The calculated damage for all inundation scenarios ('traffic system' and 'elec. installations' have been grouped together as 'infrastructure')

Risk element	Sc1/Sc2/Sc7	Sc3	Sc4	Sc5	Sc6
Buildings	DKK 4.937.000	DKK 0	DKK 0	DKK 54.179.000	DKK 203.555.00
	€ 662.685	€ 0	€ 0	€ 7.272.349	€ 27.322.819
Movable property	DKK 4.640.000	DKK 0	DKK 0	DKK 37.538.000	DKK 146.905.000
	€ 622.819	€ 0	€ 0	€ 5.038.658	€ 19.718.792
Agricultural acreages	DKK 2.208.000	DKK 933.000	DKK 211.000	DKK 7.098.000	DKK 9.489.000
	€ 296.376	€ 125.235	€ 28.322	€ 952.752	€ 1.273.691
Livestock	DKK 0	DKK 0	DKK 0	DKK 1.232.500	DKK 7.978.000
	€ 0	€ 0	€ 0	€ 165.436	€ 1.070.872
Infra-structure	DKK 7.226.000	DKK 844.000	DKK 942.500	DKK 22.862.000	DKK 56.554.000
	€ 969.933	€ 113.289	€ 126.510	€ 3.068.725	€ 7.591.141
Total	DKK 19.011.00	DKK 1.777.000	DKK 1.153.500	DKK 122.909.500	DKK 424.481.000
	€ 2.551.812	€ 238.523	€ 154.832	€ 16.497.919	€ 56.977.315

5. Risk assessment

Finally, risk values were calculated varying between 300 DKK/year and 110.000 DKK/year. In this connection, the risk values calculated for scenarios Sc3 and Sc4 represent the lower bound of risk values for the Ribe flood defence system. On the other hand, the upper bound is represented by the risk value based on scenario Sc6.

The risk assessment made clear that the range of risk values depends on the inundation scenarios and the damage, which was determined on the basis of the inundation extension and depth. The determination of these factors required several assumptions, such as the location and number of failure events, the time of failure, the water level at the time of failure and a standardised storm surge hydrograph.

The aforementioned assumptions were not analysed within the damage analysis, as, for example, the location and number of dike breaches was chosen mainly on the basis of the overall failure probabilities calculated for the dike sections of the defence system.

6. Concluding remarks

The aim of this study was to analyse the overall risk of the flood-prone area in Ribe/Denmark. A hazard analysis has been performed as part of determining the flooding probability. The probabilistic calculations resulted in a failure probability of $P_f = 1 \cdot 10^{-5}$ to $P_f = 1 \cdot 10^{-6}$ for the dike sections. Similar values were obtained when scenario fault trees were used because the overall flooding probability is primarily governed by the failure probability of wave overtopping. The failure probability of the sluice and the outlets were in the range of $P_f = 1 \cdot 10^{-1}$ which was mainly due to the high failure probability for wave overtopping.

In order to determine the overall flooding probability, the defence system was divided into 15 sections. The division was based on two selection criteria, the structure type and the input parameters. Regarding the latter the wave period T_p and the crest height h_k were most relevant. The fault tree calculations only considering the dike sections (without sluice and outlets) resulted in $P_f = 2.5 \cdot 10^{-4}$. However, this simple approach of dividing a defence system into sections has to be further developed in future, prompted by the following objectives:

- The variation of the relevant input parameters along the defence system (length effect) has to be considered. Wave attack on the seaward slope may vary locally because of changing the foreshore geometry. Furthermore, a varying crest height due to consolidation of different magnitude along the defence system may influence the probability of wave overtopping.
- The variation of input parameters along the defence system has to be considered in the probabilistic calculations in order to obtain a more accurate overall flooding probability of the defence system. In this respect, spatial and temporal correlations between different defence structures (dike, sluice, foreshore etc.) within one defence system have to be considered.
- Moreover, an improved approach of considering the parameter variation and dependencies (length effect) will give reliability-based indications of the location of failure (dike breach) along the defence line, which will be crucial in the process of defining inundation scenarios.

Due to the high failure probability of the sluice and the outlets, it was concluded that the sluice and the outlets represent the weak points of the Ribe flood defence system. Nevertheless, calculations showed that the limit-state equations and the uncertainty of the input parameters concerning sluices and outlets require further investigations. This goes

along with a more accurate estimation of the real wave height and wave period in front of sluices and outlets. Within the vulnerability analysis only few damage categories have been considered. For some damage categories the tangible property was difficult to assess. However, the vulnerability analysis showed that the total damage calculated within each scenario strongly depends on the definition of the scenarios, the considered damage categories, the determination of the inundation behaviour and the derived depth-damage functions. Therefore, further investigations on the following topics should be carried out:

- criteria for the definition of inundation scenarios;
- damage categories, which have not been considered in this study;
- determination of the inundation process, e.g. by using numerical modelling;
- understanding of the breaching process of a clay-covered dike and the flood inundation process;
- further development of the depth-damage functions and their verification by real data.

Despite these further investigations, the assessment of the inundation propagation and thus the dimension of the damage are only assessable to a certain degree of accuracy. However, to calculate the flood risk and to assess the importance of the flood defence system as a defence structure for the inhabitants and their assets, a vulnerability analysis is indispensable.

The presented risk analysis procedure has been considered as starting point of reliability-based design of flood defence systems. This study has shown that it is indeed possible to consider more stochastic parameters when analysing the safety of a flood defence system. Despite the fact that many questions are still open and problems regarding the feasibility remain unsolved, the risk analysis procedure has resulted in a considerable increase in information about the Ribe flood defence system and the protected hinterland, which certainly will contribute to improve the decision-making regarding future flood defence systems in the area.

7. References

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