FAILURE MODE AND FAULT TREE ANALYSIS FOR SEA AND ESTUARY DIKES

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Abstract: A detailed analysis of failure modes for seadikes has been performed and was used to derive a complete set of limit state equations for the description of failure scenarios of seadikes. These models and scenarios were then used to calculate the respective failure probabilities and the overall failure probability by a fault tree approach. A simple example of a non-existing seadike was used to illustrate the procedure. Details of parameters, model uncertainties and their influences on the results as well as further information required for the aforementioned procedure are also discussed.

INTRODUCTION

Sea dikes are amongst the most important coastal structures along German coastlines and many other coasts in the world (Fig. 1). They usually protect low-lying coastal areas which are highly vulnerable to coastal and or river flooding. Generally, the design of such structures is still based on purely deterministic or quasi-deterministic approaches and is either based on the design water level superimposed by the maximum wave-run-up or on admissible wave overtopping rates under extreme storm surge conditions. Geotechnical failure modes of seadikes are often disregarded and not properly accounted for in the design process.

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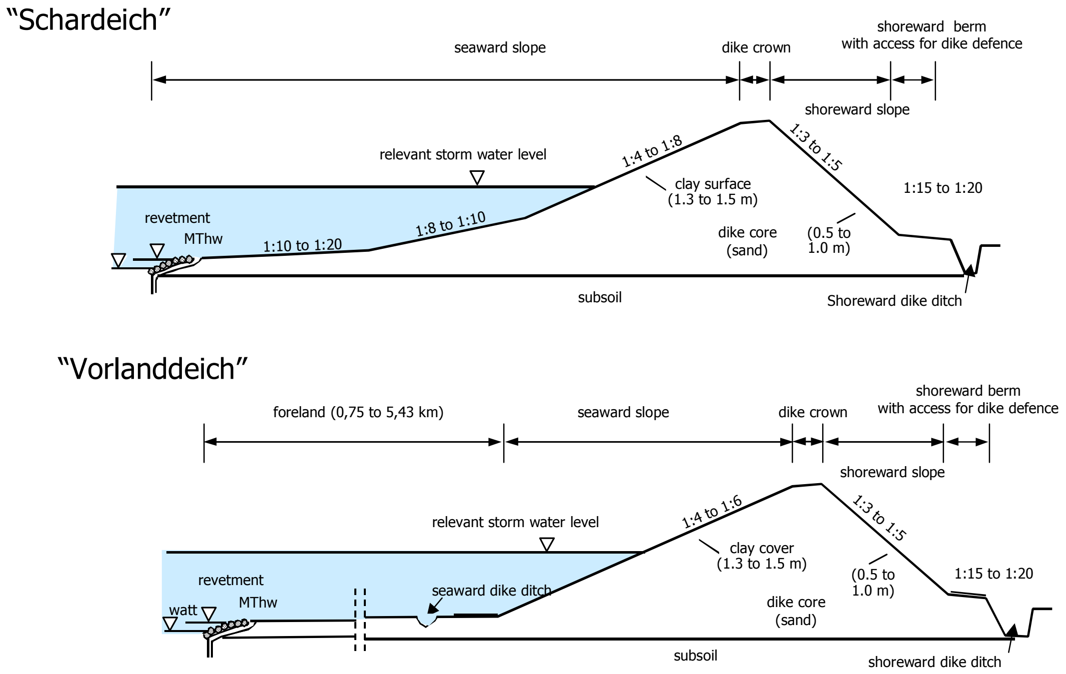


Fig. 1: Typical modern seadikes along the German coastline (simplified examples)

Reliability and risk based design concepts have been increasingly proposed during the last years (see e.g. the concept in Oumeraci and Kortenhaus, 2002). The probabilistic methods on which these concepts are based allow to account for the uncertainties in the input parameters and the models describing possible failure modes of various types of coastal structures. However, these methods are very often limited to simple cases or to just one or a couple of failure modes.

It was therefore found necessary to understand the hydrodynamic loading and the underlying physics of failures of dikes before embarking into detailed analyses on probabilistic methods and applying them to such structures. Therefore, a German research project was initiated by the authors in 2001 which focuses on the probabilistic design of seadikes based on a deeper understanding of the underlying hydromechanical and geotechnical processes. Some details of the results found in this project will be discussed in this paper.

CONCEPT

General procedure

The method used within this paper follows a risk-based design approach as for example discussed by Oumeraci and Kortenhaus (2002) in these proceedings. To predict the flooding risk R for existing coastal defence structures it is necessary to evaluate the probability of failure P_f of the respective sea defence and the potential damages E(D) resulting from failure. The risk can then be estimated as the product of the probability P_f and the damage E(D). This paper concentrates solely on the determination of P_f based on seadikes since this type of defence structure is dominant along the German coastlines. Furthermore, only one single cross section is considered so that 3-dimensional effects and separation of various sections along the defence line is not needed.

The concept of how to determine the overall probability of failure for a seadike is illustrated in Fig. 2. The figure shows a nine-step-procedure which starts with a deterministic design and the selection of the most relevant failure modes for which limit state equations have to be determined. These initial three steps are purely deterministic and are essentially based on failure analyses which will be described in the following section.

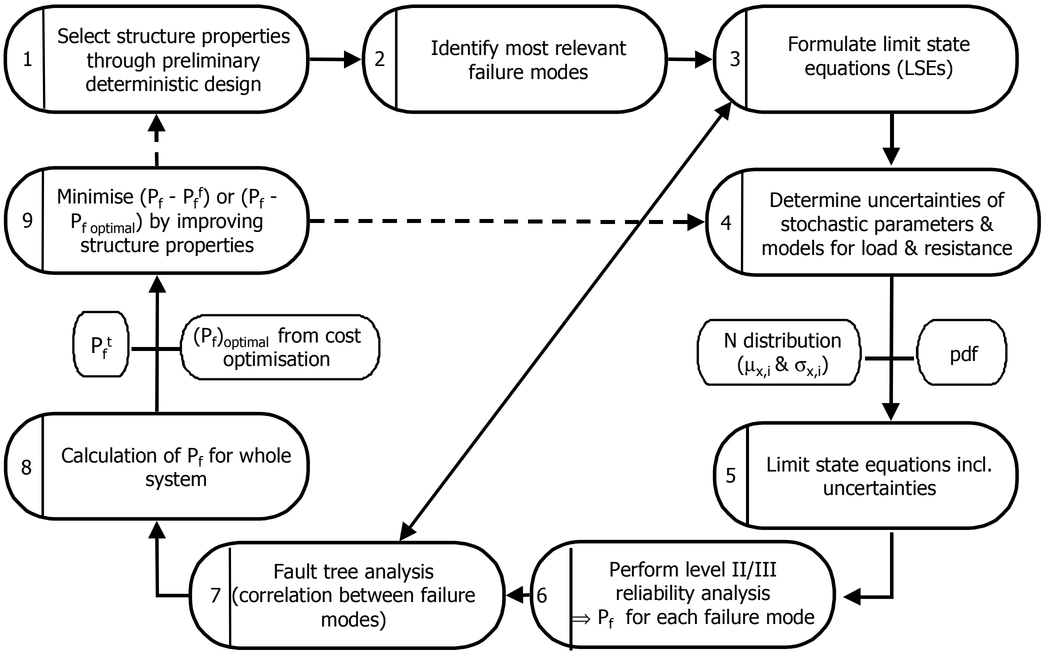


Fig. 2: Simplified flow chart illustrating the determination of a failure probability for a coastal defence structure

Step 3 already shows strong links with the set-up of a fault tree (step 7) which can be used to schematise the failures which have been observed at seadikes. Following this method a deterministic fault tree can be derived which can later be used for probabilistic design.

Steps 4 and 5 discuss the uncertainties of parameters and models which will be included in the probabilistic design. The description of the uncertainties as used in this study will be discussed in more detail in one of the following sections. In step 6 the probability of failure of all limit state equations is performed by means of Level II and III (Monte-Carlo) analyses. Steps 7 and 8 discuss the relation between the failure modes and the calculation of the overall probability of failure. In step 9 of the procedure some optimisation routines may be used to optimise the dimensions of the defence structure. This can be done by minimising costs or adopting target probabilities (for details on optimisation of defence systems see Voortman, 2002). However, this step will not be discussed here any further.

Definition of failure

Step 2 and 3 of the procedure described in Figure 2 require the analysis of failure modes and a suitable model to describe it. In comparison to deterministic approaches this needs a definition of "failure" which can later be used for probabilistic design, too. All limit state equations are therefore expressed as the difference between a resistance term R and the stress term S so that:

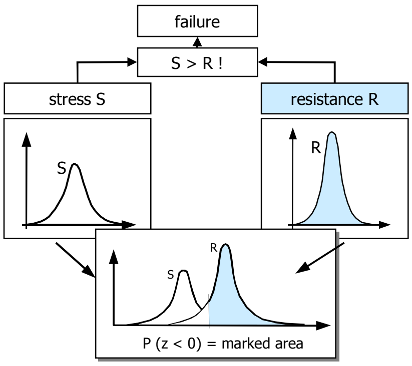
$$z = R - S \tag{1}$$

In this study failure is defined as negative z-values. The three examples in Figure 3 are used to illustrate the differences between the general deterministic understanding of failure in comparison to a probabilistic definition.

a) Wave overtopping event at a sea dike

b) Failure of the seaward slope of a seadike

c) Breaching of a seadike during storm surge



d) Probabilistic definition of failure

Fig. 3: Three example "failures" of a seadike

In Figure 3a overtopping over a dike crest can be seen. Depending on the allowable amount of overtopping this is usually acceptable and will not be regarded as failure. Figure 3b shows an erosion hole of the seaward slope of a seadike after a severe storm surge. In general this may be looked at as failure since the design of the cover layer of the dike has not been dimensioned properly. In probabilistic sense this is no failure since flooding did not occur (provided that the overall top event of a probabilistic fault tree is defined to be flooding of the hinterland of a dike). However, the probability of failure can be significantly higher than in the overtopping case. From these observations it is obvious that limit state equations are needed to describe e.g. the erosion process at the seaward side of the dike in a probabilistic sense. Figure 3c shows a complete failure of the dike (from both the deterministic and probabilistic "viewpoint") where flooding occurred.

ANALYSIS PROCEDURE

The steps as described in Figure 2 are presented in more details in the following. The deterministic steps of finding the failure modes and the appropriate limit state equations are based on an extensive failure analysis which will be discussed first.

Failure analysis

Based on the results of a detailed failure analysis of sea and estuary dikes (Oumeraci & Schüttrumpf, 1999) failure mechanisms have been identified which may eventually lead to breaching of the dike. These procedures are shown in Figure 4 for the seaward and the shoreward side of the dike (Fig. 4a) and 4b), respectively).

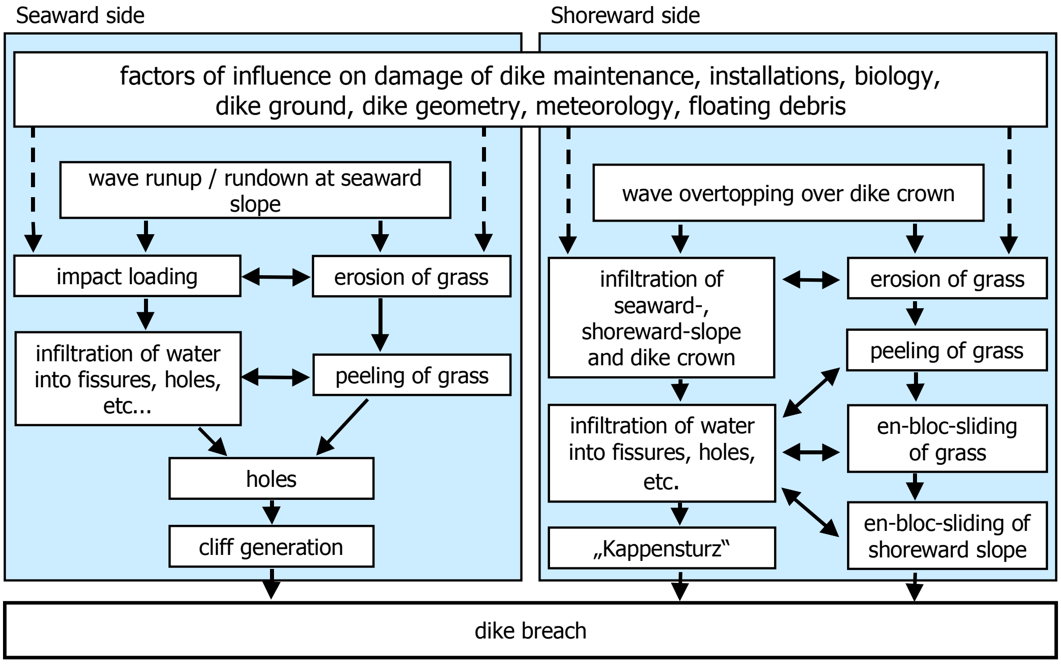


Fig. 4: Failure modes of seadikes eventually leading to dike breaching
(a) seaward side, b) shoreward side)

Principally, the dike can fail from the seaside, the shoreward side or from inner erosion (not shown in Fig. 4). All relevant processes as shown in Figure 4 need to be described and possibly simplified by limit state equations (LSE). An example how this has been done for the seaward slope is shown in Figure 5. The processes on the shoreward side of the dike are slightly more complex but can be simplified in the same way (Kortenhaus et al., 2002). The inner erosion of the dike may result from piping or matrix erosion, providing that the water level in front of the dike persists over a longer time. The latter has been accounted for by calculation of the seepage time through the dike body as an initial boundary condition.

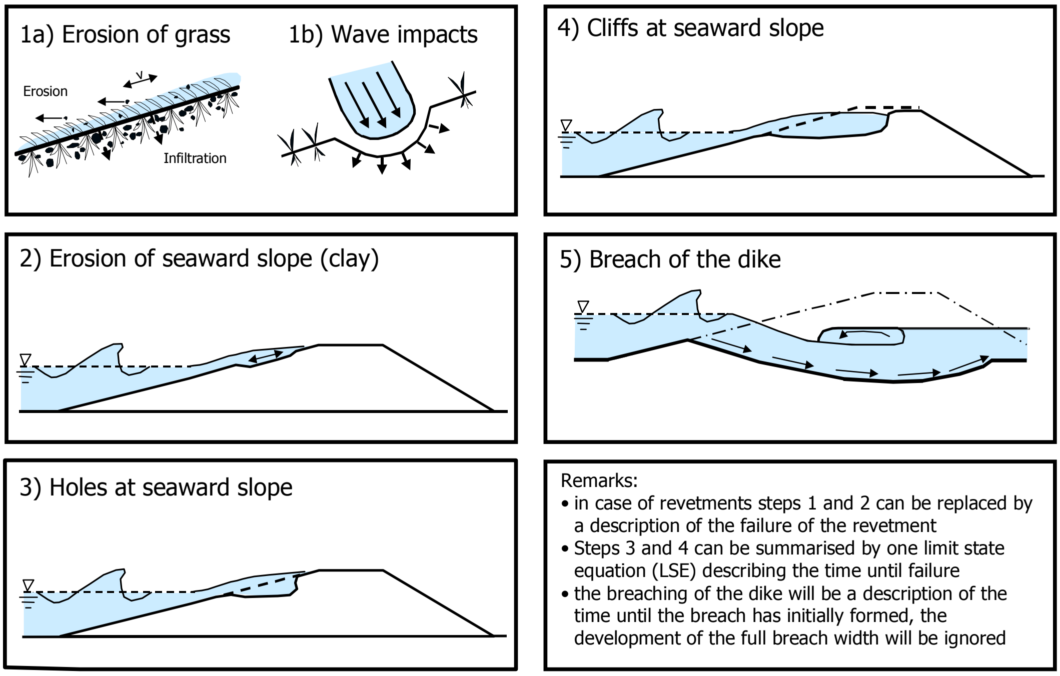


Fig. 5: Simplified description of processes at the seaward side of a seadike (after Oumeraci & Schüttrumpf, 1999)

Limit state equations

The simplified processes as discussed in the previous section need to be described by limit state equations. For this purpose a detailed review from methods available in literature was performed to arrive at suitable models (for an overview see Table 1). However, some of the processes cannot be described by analytical formulae yet so that simplifications or simple models based on physical understanding of the processes were derived. Indications for these models are also given in Table 1 together with a reference to literature, the physical parameter which are compared in the limit state equation and the units used.

Table 1 shows that nine limit state equations are based on comparison of storm duration with the resisting of the dike to withstand the respective loading. Therefore, (i) the correct assumption on the storm duration is very important; and (ii) models which are dependent on time need to be linked together to one model only to consider time appropriately. The latter will be explained in more detail in the subsequent sections of this paper.

Tab. 1: Overview of limit state equations used for sea and estuary dikes

no.	name	reference	comparison of	unit
1	overflow	Oumeraci et al. (1999)	freeboard	[m]
2	overtopping	Schüttrumpf & Oumeraci (2001)	freeboard	[m]
3	breaching	Visser (1995), extended and amended for overtopping	duration of storm	[h]
4	sliding	DIN (1996): DIN 1054	forces	[kN]
5	stability of revetment	Van der Meer (1998)	stone diameter	[m]
6	wave impacts	new model based on Führböter (1994)	forces	[kPa]
7	uplift of revetment	new model based on Klein Breteler et al. (1998) and Schüttrumpf (2001)	forces	[kPa]
8	run-up velocity	Schüttrumpf (2001)	velocity	[m/s]
9	erosion of grass at seaward slope	amended after TAW (2000)	duration of storm	[h]
10	erosion of clay at seaward slope	TAW (2000)	duration of storm	[h]
11	cliff erosion	extended after INFRAM (2000)	duration of storm	[h]
12	stability of seaward slope	DIN (1983): DIN 4084	forces	[kNm]
13	overflow velocity	newly derived after Schütt-rumpf (2001)	velocity	[m/s]
14	overtopping velocity	simplified after Schüttrumpf (2001)	velocity	[m/s]
15	erosion of grass at shore-ward slope	TAW, 2000	duration of storm	[h]
16	erosion of clay at shoreward slope	Rose et al. (1983)	duration of storm	[h]
17	infiltration	Weißmann (1999)	duration of storm	[h]
18	failure of cap (Kappensturz)	DIN (1983): DIN 4084	forces	[kNm]
19	phreatic line at shoreward slope	simple Darcy flow model modified after tests by Scheu-ermann & Brauns (2001)	duration of storm	[h]
20	clay uplift	Richwien & Weißmann (1999)	forces	[kN]
21	sliding of clay	Richwien & Weißmann (1999)	forces	[kN]
22	stability of shoreward slope	DIN (1983): DIN 4084	forces	[kNm]
23	partial breaching shoreward slope	Visser (1995)	duration of storm	[h]
24	piping	Weijers & Sellmeijer (1993)	hydraulic gradient	[-]
25	matrix erosion	De Mello (1975)	diameter	[m]

Fault tree

Based on the failure analysis described in the previous section and various descriptions of fault trees in literature (see e.g. Kuijper & Vrijling, 1998) a detailed fault tree was set up (Fig. 6) comprising about 30 different failure mechanisms. “Flooding” was defined to be the top-event of the fault tree.

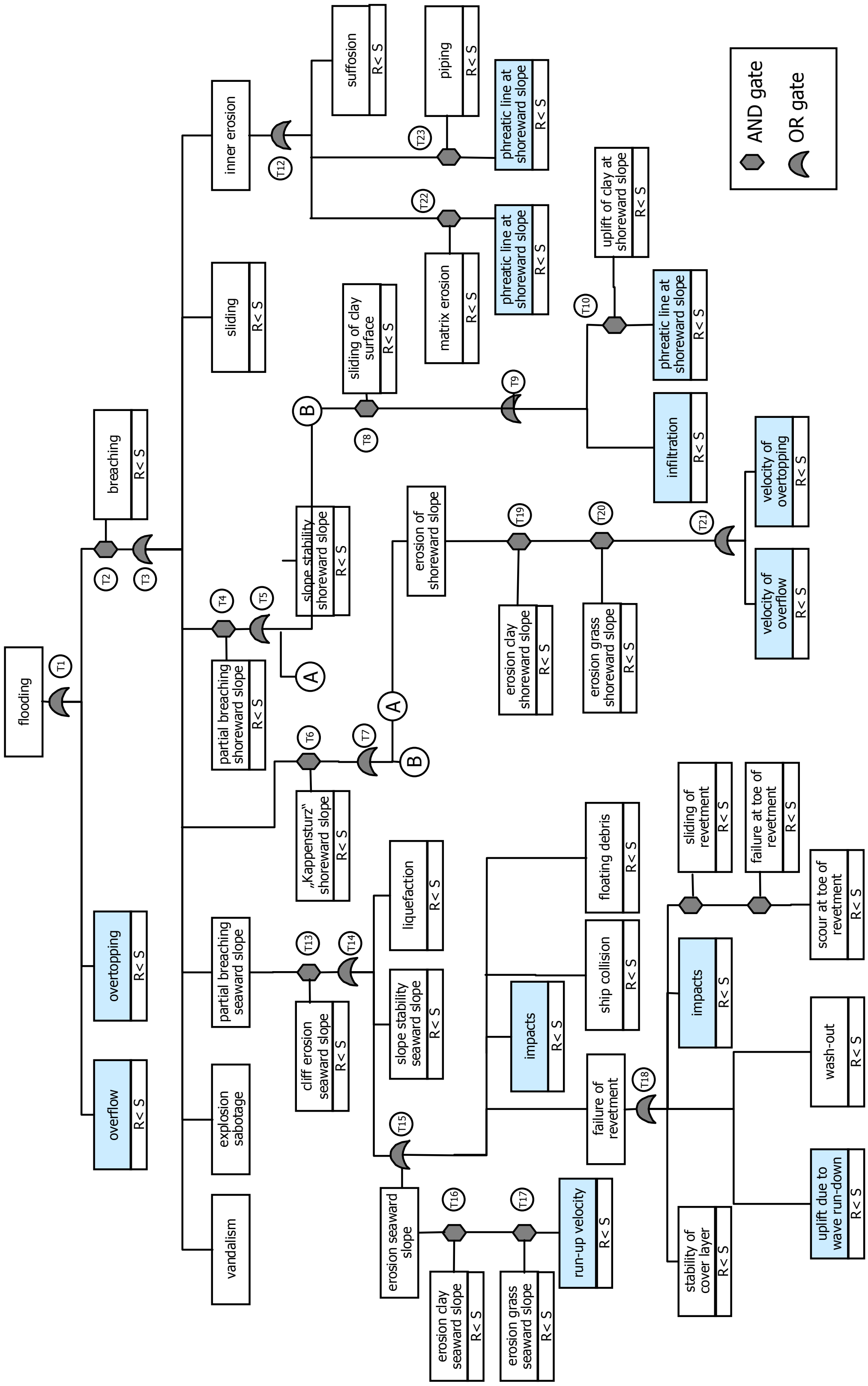


Fig. 6: Fault tree for sea and estuary dikes based on detailed failure analysis

The independence of failure modes from each other was carefully checked and adjusted if possible. All failure mechanisms discussed in the previous section have been considered in the fault tree. Firstly, spreadsheet calculations were performed to determine the deterministic safety coefficients $\eta = R / S$ in order to check the limit state equations and the critical failure modes. Furthermore, comparison to results taken from literature has been performed.

Results have shown that critical failure modes are the descriptions of erosion at the seaward and the shoreward side (given that the overtopping is high enough) of the dike. These calculations and comparisons have shown that the models work properly and that results are in the right range.

Application to a non-existing seadike

The fault tree and the limit state equations have been applied to a non-existing dike with an artificial cross sections with a berm on the seaward and the shoreward side of the dike (Fig. 7). Some geometrical parameters such as height, slopes and thickness of clay cover were taken from examples along the German coastline to generate a dike which is not too different from reality.

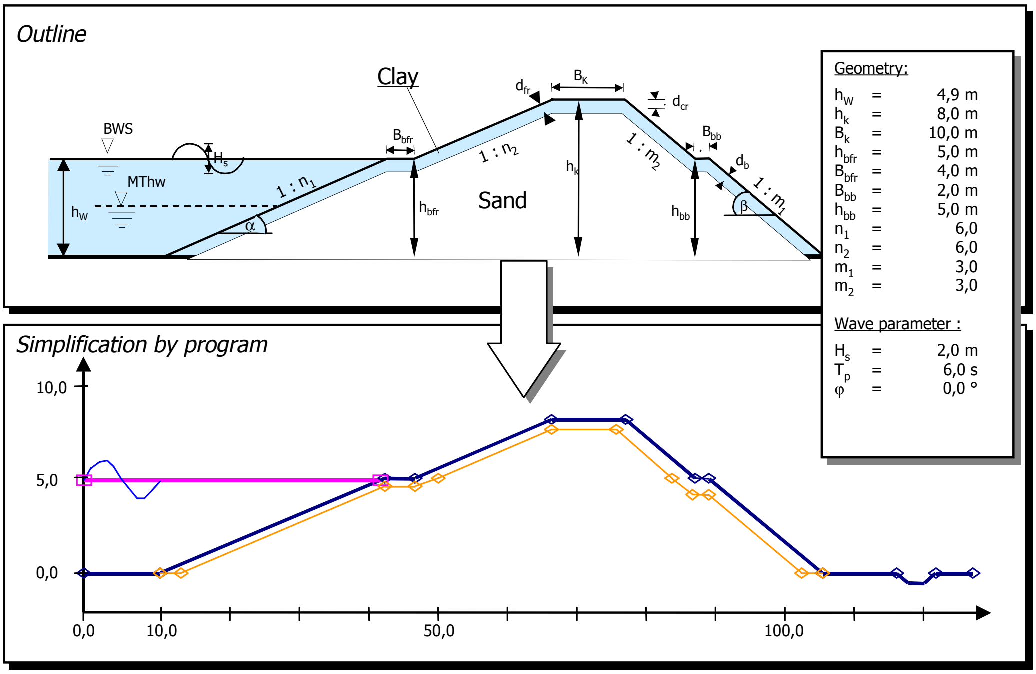


Fig. 7: Cross section and key parameters of an artificial example dike

The dike shown in Figure 7 was used for all probabilistic calculations which are discussed in the following.

Uncertainty evaluation

A total number of 87 input parameters are needed to describe (i) the geometry of the dike (27 parameters), (ii) the hydromechanic boundary conditions (13), and (iii) the geotechnical input parameters (47). A large number of uncertainties of the input parameters could be assessed from the analysis of published and non published documents, leaving only some geometric and geotechnical parameters where no information was available.

The information available for the uncertainties (standard deviation σ or coefficient of variation σ') and the distribution type of parameters is however very limited in literature. Therefore it was decided to take a normal (Gaussian) distribution and a standard deviation based on engineering experience (textbooks and communication with practitioners) if no data are available or described in literature. The magnitude and the type of distribution was then varied in a sensitivity analysis (see next section) to identify the importance of these parameters. A complete overview of uncertainties and how they were modelled is given in Kortenhaus and Oumeraci (2002).

Probabilistic calculations

The aforementioned procedure was applied to the example dike and compared to other calculations given in references. Due to the complexity of most of the models used almost all calculations were performed as Level III (Monte-Carlo) simulations. Whenever possible, simple LSE were calculated as Level II (FORM) calculations. A comparison of both methods (where possible) showed that generally the Level III calculations gave slightly higher probabilities of failure.

The calculation of gates in the fault tree was performed as shown in Table 2 depending on the dependency of failure modes.

Tab. 2: Calculation of gates in fault tree depending on the dependency of failure modes

	parallel systems	**series systems**
Gate	AND / IF	OR
maximum boundary	$P_f = \min(P_i)$	$P_f = \sum_{i=1}^{n} P_i$
fully dependent	$P_f = \min(P_i)$	$P_f = \max(P_i)$
fully independent	$P_f = \prod_{i=1}^{n} P_i$	$P_f = 1 - \prod_{i=1}^{n} (1 - P_i)$
minimum boundary	$P_f = 0$	$P_f = \max(P_i)$

Generally, all failure modes were assumed to be fully independent from each other. However, in the sensitivity analysis this assumption was varied and fully dependent failure modes were assumed (for more details see the next section).

The overall failure probability of the virtual seadike was $P_f = 8.4 \cdot 10^{-6}$. The failure modes which most strongly contributed to these results were the models describing the erosion of the seaward slope and the model for breaching of the dike. As already discussed in the previous section it was necessary to overcome the limitations when calculating time dependent failure modes. Therefore, a scenario fault tree was developed following the idea of minimal cut-sets. The overall probability of failure will then increase to $P_f = 5{,}1 \cdot 10^{-5}$ showing that ignoring the influence of time may lead to an order of magnitude difference between results. More details describing this approach can be found in Kortenhaus and Oumeraci (2002).

Comparisons were also performed to calculation of overall failure probabilities of seadikes in literature. The comparison was not always simple as in many times other models for the failure modes are used in the references. However, the results have generally shown that (i) the detailed results obtained from this study always result in lower probabilities of failure; and (ii) the differences between the results obtained here and other results may be up to two orders of magnitude. It may be concluded from these results that the failure analysis based approach and detailed consideration of the failure modes is very meaningful in order to achieve more accurate results than using methods which are based on design water levels.

Sensitivity analysis

A detailed sensitivity analysis of parameters, failure mechanisms, and uncertainties was performed to identify the following influences.

- model factors describing the uncertainties of the models themselves;
- uncertainties of parameters;
- type of distribution of input parameters (for simplicity reasons model factors were always assumed to be Gaussian distributed);
- dependency of failure modes to each other;
- dependency of failure modes from time

With respect to failure modes and fault trees the most important results from this sensitivity analysis were:

- influence of model factors on results is generally very low. Only the models for overtopping and piping are widely influenced by variations of model factor uncertainties. Since a lot of data are available for overtopping the model factor for this LSE is relatively well known and will not vary strongly. This means that only piping needs some further investigation on the accuracy of the model used (Weijers and Sellmeijer, 1993), improvements of other models used here will not result in much lower failure probability of the dike;
- analysis of the parameter uncertainties have shown that the design water level and the sea state parameters (storm duration, wave period, wave height) have the strongest influence on the overall failure probability. This is mainly due to two reasons (i) these parameters are used in many failure mode models, and (ii) for many failure modes they have the strongest influence on the individual failure probability

- the type of distribution was investigated by comparing normal to log-normal and Weibull 2 distributions. Generally, this has led to no significant changes in the individual failure probabilities, only showing some slight decreases in probabilities when Weibull distributions are used

Further details and more results from this sensitivity analysis can be obtained from Kortenhaus and Oumeraci (2002) and will be published in future papers on that subject.

CONCLUSIONS

The aforementioned analysis procedure is an essential starting point for a reliability based design and is based on a detailed analysis of the failure modes. In this study new models for describing failure modes of sea dikes have been developed or improved. Based on these models together with an extensive fault tree for sea dikes probabilistic calculations have been performed to arrive at overall failure probabilities of sea dikes. The key results of this study are:

- the overall failure probabilities of sea dikes may be significantly reduced if an approach is used which is based on the analysis of failure;
- the most important models contributing to the overall probability of failure are all models describing the erosion process of the outer slope, the inner slope is only relevant if the water level is much higher and overtopping volumes are increasing;
- the erosion process at the seaward and shoreward slope is not well understood up to now and future work should be concentrated on these aspects;
- the uncertainties of the models are much lower than initially expected (exceptions are wave overtopping and piping) so that further research in these areas would not result in lover failure probabilities;
- most relevant parameters for design of sea dikes are the water level and some sea state parameters (storm duration, wave period, wave height).

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REFERENCES

De Mello, V.F. (1975): Some lessons from unsuspected, real and fictitious problems in earth dam engineering in Brazil. *Proceedings 6th Regional Soil Mechanics and Foundation Engineering Conference for Afrika*, Rotterdam, The Netherlands, Balkema, no. 6, pp. 285-304.

DIN (1983): DIN 4084: Gelände- und Böschungsbruchberechnungen. Berlin, Germany: Beuth Bauverlag, S. 266-301. In German.

DIN (1996a): DIN 4084-100: Böschungs- und Geländebruchberechnungen. Teil 100: Berechnung nach dem Konzept mit Teilsicherheitsbeiwerten. Berlin, Germany: Beuth-Verlag, 1. Auflage, S. 215-234. In German.

DIN (1996b): DIN 1054: Sicherheitsnachweise im Erd- und Grundbau. Berlin, Germany: Beuth-Verlag, S. 109-191. In German.

Führböter, A. (1994): Wave loads on sea dikes and sea-walls. In: Abbott, M.B.; Price, W.A.: *Coastal, Estuarial and Harbour Engineers' Reference Book*, London/Glasgow, U.K., first edition, pp. 351-367.

INFRAM (2000): Rudimentaire opzet erosiemodel dijken. *Internal Report, INFRAM*, Zeewolde, The Netherlands, 8 pp. In Dutch.

Klein Breteler, M.; Pilarczyk, K.W.; Stoutjesdijk, T. (1998): Design of alternative revetments. *Proceedings International Conference Coastal Engineering (ICCE)*, ASCE, Copenhagen, Denmark, no. 26, Volume 2, pp. 1587-1600.

Kortenhaus, A.; Oumeraci, H. (2002): Probabilistische Bemessung für Seedeiche –Abschlussbericht. Berichte Leichtweiß-Institut für Wasserbau, Braunschweig, Germany, 200 pp, unpublished.

Kuijper, H.; Vrijling, J.K. (1998): Probabilistic approach and risk analysis. In: Pilarczyk, K.W. (ed.): *Dikes and revetments*, Rotterdam: Balkema, pp. 443-462.

Oumeraci, H.; Schüttrumpf, H. (1999): Review and analysis of failures experienced by sea and estuary dikes. Res. Report, Leichtweiss-Institut TU Braunschweig , 26pp. (in German).

Oumeraci, H.; Schüttrumpf, H.; Bleck, M. (1999): Untersuchungen zur Ermittlung der mittleren Wellenüberlaufrate ohne Freibord bei Stromdeichen. *Berichte Leichtweiß-Institut für Wasserbau, Technische Universität Braunschweig*, LWI, Nr. 842, Braunschweig, Germany, 43 S. In German.

Oumeraci, H.; Kortenhaus, A. (2002): Risk-based design of coastal flood defences: a suggestion for a conceptual framework. Proc. Inernational Conference on Coastal Engineering (ICCE), no. 28, Cardiff, Wales, UK, 13 pp., these Proceedings.

Richwien, W.; Weißmann, R. (1999): Quantifizierung der Sicherheitsreserven von Seedeichen bei Wellenüberlauf. *Abschlußbericht. Institut für Grundbau und Bodenmechanik, Universität und Gesamthochschule Essen*, Teil IV, Essen, Germany, 55 S. In German.

Rose, C.W.; Williams, J.R.; Barry, D.A. (1983): A mathematical model of soil erosion and deposition processes: I. Theory for a plane land element. *Journal Soil Science Society of America*, vol. 47, pp. 991-995.

Scheuermann, A.; Brauns, J. (2001): Die Durchströmung von Flussdeichen in Abhängigkeit von Vorbedingungen. *Zweites Forum Katatrophenvorsorge "Extreme Naturereignisse – Folgen, Vorsorge, Werkzeuge"*, DKKV, Leipzig, Germany, 8 S.

Schüttrumpf, H. (2001): Wellenüberlaufströmung bei Seedeichen – experimentelle und theoretische Untersuchungen. Fachbereich Bauingenieurwesen, Technische Universität Braunschweig. *Mitteilungen Leichtweiß-Institut für Wasserbau der Technischen Universität*, LWI, Braunschweig, Germany, Heft 149, S. 1-124. In German.

Schüttrumpf, H.; Oumeraci, H. (2001): Wellenauflauf und Wellenüberlauf. In: *Ausschuss für Küstenschutzwerke der DGGT und HTG, Arbeitsgruppe Wellenauflauf und Wellenüberlauf*, Braunschweig, Germany, 21 S. In German.

TAW (2000): Van overschijdingskans naar overstromingskans. *Adviesrapport met Achtergrondrapport, Technische Adviescommissie voor de Waterkeringen*, TAW, Den Haag, The Netherlands, 25 pp. In Dutch.

Van der Meer, J.W. (1998): Application and stability criteria for rock and artificial units. In: Pilarczyk, K.W. (ed.): *Dikes and Revetments*, Rotterdam/Brookfield: A.A. Balkema, Chapter 11, pp. 191-215.

Visser, P.J. (1995): A model for breach growth in sand-dikes. *Proceedings International Conference Coastal Engineering (ICCE)*, ASCE, Kobe, Japan, no. 24, Volume 3, Chapter 199, pp. 2755-2769.

Voortman, H.G. (2002): Risk-based design of large-scaled flood defence systems. Ph.D. thesis, Delft University of Technology, Delft, The Netherlands.

Weijers, J.B.A.; Sellmeijer, J.B. (1993): A new model to deal with the piping mechanism. *Filters in Geotechnical and Hydraulic Engineering, Proceedings of the First International Conference (Geo-Filters)*, Brauns, Heibaum & Schuler (eds.), Rotterdam, The Netherlands, Balkema, Karlsruhe 1992, pp. 349-355.

Weißmann, R. (1999): Infiltration von Wellenüberlauf in Deichbinnenböschungen. *Workshop "Strömungen in Böden", Universität Essen, 14.10.1999*, Essen, Germany. In German.