

FLOOD PROTECTION DUNES – AN APPROACH FOR RELIABILITY ASSESSMENT BY MEANS OF FRAGILITY CURVES AS PART OF A RISK AND DAMAGE ANALYSIS

A.Gruhn¹, D. Salecker¹, P. Fröhle¹, H. Schüttrumpf², F. Thorenz³

Low lying coastal areas have always been preferred settlement areas as well as trading and industrial areas. Unfortunately, those areas are strongly endangered by extreme storm surges. In the event of a flood defence failure, protected areas are flooded and damages have to be expected. For the assessment and management of flood risk, the European Union approved the “Directive of the European Parliament and the Council on the assessment and management of flood risk”.

As one part of a risk and damage analysis the risk of flooding - being the product of failure probability of a certain flood defence and the damages resulting from a failure of this flood defence - has to be determined. One possibility for the assessment of the failure probability is provided by fragility curves. A method for the derivation of fragility curves for flood defence dunes is described. Hence, the applied dune erosion model as well as the method for the derivation of the required input data is explained. Furthermore, first results of the calculation of failure probabilities and fragility curves are presented.

Keywords: flood defence dunes, failure probability, fragility curve, bivariate statistics, copula models

INTRODUCTION

Low lying coastal areas have always been preferred settlement as well as trading and industrial areas. Unfortunately, those areas are strongly endangered by extreme storm surges. In the event of a flood defence failure, protected areas are flooded and damages as well as losses have to be expected.

For the assessment of risk in coastal areas, the European Union approved the “Directive of the European Parliament and the Council on the assessment and management of flood risk” (2007/60/EC). Its objective is to establish a framework for the assessment and management of flood risk.

Existing methods and approaches for flood risk assessment and management that have been developed for inland areas cannot be transferred directly to coastal areas. Reasons for this can be seen in the different natural conditions, e.g. sea state, storm surges and salt water intrusion but also short warning periods and different protection strategies impede the transferability of such methods. Hence, it is necessary to develop methods for flood risk assessment and management adjusted to the different conditions at coastal areas.

Under this premise, the joint research project “HoRisK” was initiated. Several project partners are working together to develop application-oriented methods and approaches for the implementation of flood risk and damage analyses for coastal areas, as basis for the compilation of flood hazard maps, flood risk maps and flood risk management plans. The project objectives are i) the assessment of impacts (sea state, waves, currents etc.), ii) the determination of failure mechanisms and iii) the corresponding failure probabilities of flood defences, typical for the German North Sea and Baltic Sea Coast. Furthermore, iv) the effects of different flood defences regarding their protective effect and their effect on the inundation propagation itself shall be assessed. The project work also includes the detection of possibilities for risk mitigation. Beside the theoretical considerations, the developed methods shall be applied to selected areas at the German North Sea and Baltic Sea coast.

As one part of a risk and damage analysis, risk of flooding has to be determined. Flood risk, in general, can be regarded as the product of the failure probability of the considered flood defence and the damage resulting from the failure of this flood defence. One possibility for the assessment of failure probabilities is provided by the application of fragility curves.

In the following sections a method for the derivation of fragility curves for flood protection dunes is presented. The applied dune erosion model as well as the derivation of the required input data are described.

As the basis for the derivation of the methods, dunes as a measure of flood defence are introduced and the general concept of fragility curves as a possibility for reliability assessment is presented. In conclusion first results are shown and a summary is given.

¹ Hamburg University of Technology, Institute of River and Coastal Engineering

² Institute of Hydraulic Engineering and Water Resources Management, RWTH Aachen, Germany

³ Lower Saxony Water Management, Coastal Defence and Nature Conservation Agency (NLWKN), Germany

FLOOD DEFENCE DUNES AND FAILURE MECHANISMS

At the German Baltic Sea coast flood defence dunes are typically constructed with the aim to protect the hinterland without other additional flood protection structures (Fig. 1). That implies that those dunes have to withstand severe storm surges without additional measures. As single structures, flood defence dunes consist of three parts (Fig. 2). The seaward lying part is the so called wear part. This part contains the amount of sediment which is relocated during more frequent, comparatively low intensity storm surge events. By that, the eroding dune provides sediment for the foreshore and the overall sediment transport along the coast. The central part of the dune cross-section is the effective flood protection part. This part has to withstand severe storm surges (design conditions) and protect the hinterland from being flooded. The landward part of the dune is the so called safety part. It's function is to prevent the protected area from being flooded in the event of a complete erosion of the wear part and the effective flood protection part (StALU 2009).



Fig. 1: Flood defence dune

Within the scope of the HoRisK-project, methods and approaches for the implementation of risk and damage analyses for the determination of flood risk shall be developed. Therefore, it is necessary to identify dominating failure mechanisms for flood defence dunes and to determine corresponding failure probabilities. Based on a literature research the following possible failure mechanisms have been identified: i) erosion due to wave attack, ii) overflow, iii) overtopping (Allsop et al. 2007). Investigations presented in this paper focus on the failure mechanism i): erosion due to wave attack.

In the following sections an approach for the calculation of failure probabilities is described.

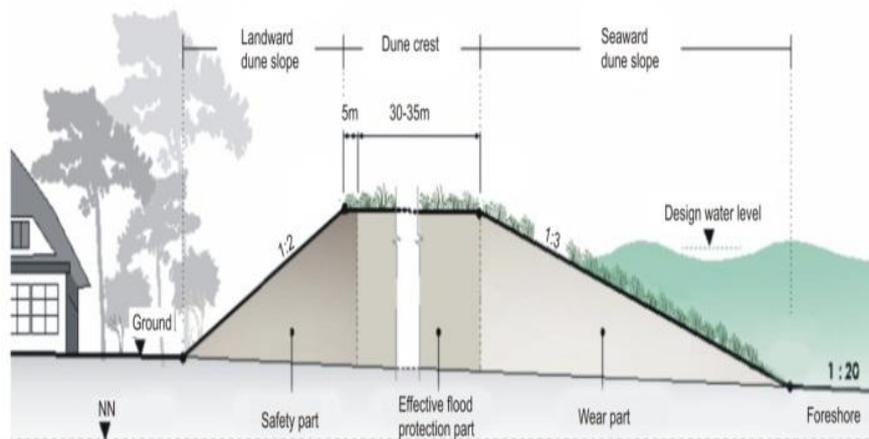


Fig. 2: Cross-section of a flood protection dune (StALU 2009, modified)

FRAGILITY CURVES AND THEIR CALCULATION

To assess the reliability of a certain flood defence, so called fragility curves can be used. These curves show the conditional probability of failure and non-failure in dependence of a certain stress acting on the flood defence, e.g. wave height, water level etc. (Fig. 3).

The ordinate indicates the applied stress and the abscissa shows the conditional probability of failure. The curve starts in the origin, which means a zero stress coincides with a zero probability of failure. On the contrary, a failure probability of one connotes that the flood defence will fail in any case. The gradient of the curve is always positive. An increase of the stress results in an increase of the probability of failure and a decrease in the probability of non-failure. The shape of the fragility curve is directly dependent on flood defence related parameters, e.g. the type of flood defence, the parameter characterizing the reliability of the structure, such as its geometry, design level as well as its maintenance level, and their uncertainties (Bachmann et.al. 2009).

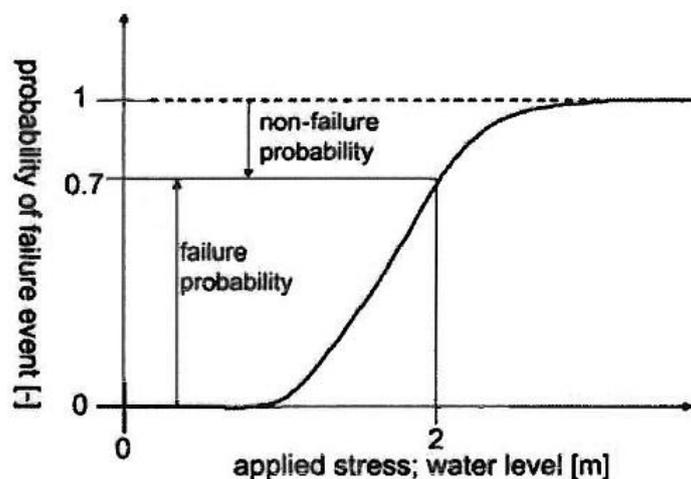


Fig. 3: General characteristics of a fragility curve
(In: Bachmann et. al. 2009)

Fragility curves are derived based on reliability analyses for discrete values of the applied stress. Within these reliability analyses a limit state equation has to be determined. This equation constitutes a comparison of the resistance of the flood defence and the stress acting on the structure. Failure of the flood defence dune is assumed to occur in the case of a negative result of the limit state equation. Monte Carlo simulations are used to calculate failure probabilities and to derive the fragility curves. Limit state equation for a multitude of combinations of water levels and wave heights are calculated repeatedly in order to take into account the uncertainties of the input data as well as the uncertainties of the applied calculation model. The probability of failure can be regarded as the relative frequency of negative results of the limit state equation.

HYDRODYNAMIC INPUT DATA

As mentioned before, Monte Carlo simulations are applied to derive failure probabilities and fragility curves. Storm surge water levels and wave heights with certain combined probabilities of occurrence are required as input data. This data is derived from measurements of water levels and sea state. Due to the fact that those measurements are rather short, wind-wave correlations are used to gain additional data. In order to obtain input data with a requested combined probability of occurrence bivariate statistics on the basis of Copula models are used.

In a first step, the marginal distribution function of the water levels and wave heights are determined on the basis of independent extreme event samples. The inverse of these functions is needed later on for back transformation of the results into real units of water levels and wave heights.

In a next step appropriate Copula models are fitted to water level and wave height samples. To assess the dependence of the measured variables the rank correlation coefficient Kendall's τ is used (1) (Salecker et.al. 2012):

$$\tau = \frac{P_n - Q_n}{\binom{n}{2}} \quad (1)$$

Where P_n and Q_n are the number of concordant & discordant pairs of variables [-] and n is the number of pairs of variables [-].

The parameter τ describes the dependence of both variables from the assigned ranks not from the values for the water levels and wave heights itself.

For each Copula within the generation of Archimedian Copulas a different generator function ($\varphi(t)$) is available. Every generator function contains a dependence parameter θ defining the copula function as soon as this parameter is determined (Genest and Favre 2007).

For example between the Kendall's τ and the generator function of the Frank Copula there is the following mathematical relation (2), (3):

$$\tau = 1 + 4 \int_0^1 \frac{\varphi(t)}{\varphi'(t)} dt \quad (2)$$

$$\varphi(t) = -\log \frac{e^{-\theta t} - 1}{e^{-\theta} - 1} \quad (3)$$

Where $\varphi(t)$ is the Frank Copula generator [-] and $\varphi'(t)$ the first derivative of $\varphi(t)$ [-].

By solving the integral (2) the dependence parameter θ can be estimated.

An arbitrary number of uniform distributed pairs (u, v^*) on the interval (0,1) is simulated.

If an explicit formula exists the inverse function of the derivative of the copula function with respect to u , can be used to model dependant data pairs (u, v) (4)

$$Q_u(v) = \frac{\partial}{\partial u} C(u, v) \quad (4)$$

$$v = Q_u^{-1}(u^*)$$

The goodness of fit is evaluated visually by comparing the simulated data pairs (u, v) with the transformed ranks of the observations ($R_i/(n+1), S_i/(n+1)$). Probabilities of a conjoint occurrence of u and v are estimated using the Frank Copulas cumulative distribution function $C(u, v)$ (5):

$$-\frac{1}{\theta} \log \left[1 + \frac{(e^{-\theta u} - 1)(e^{-\theta v} - 1)}{e^{-\theta} - 1} \right] \quad (5)$$

For the investigations isolines with the return periods of 50 years ($\cong 0,02$), 100 years ($\cong 0,01$), 150 years ($\cong 0,0067$) and 200 years ($\cong 0,005$) are calculated. From these isolines the required number of pairs of water levels and wave heights is chosen (Fig. 4) (Salecker et.al. 2007).

The simulated data pairs (u, v) as well as isolines and random data pairs are being transformed into real units of water level and wave heights. This is achieved by applying the inverse of the marginal distribution from the univariate case.

The peak period corresponding to the respective significant wave height is calculated based on measurements in the respective area using equation (6).

$$T_p = 1,21 * (1,11 * H_s + 2,5) \quad (6)$$

Where T_p is the Peak period [sec] and H_s the significant wave height [m]. These analyses are described in detail by Salecker et.al. 2012 and are based on the basic work on copulas by Sklar (1959, see Nelsen, 2006).

DUNE AND BEACH PROFILE

The calculation of the dune erosion is based on a comparison between an initial dune/beach profile and a profile resulting from the erosion process.

For the first investigations the crest height of the dune is assumed to be 5 m (Fig. 5) above MSL. The crest width is variable. Following the construction of the dunes, the inner slope and the outer slope are set to be 1:2 and 1:3, respectively (cf. Fig. 2).

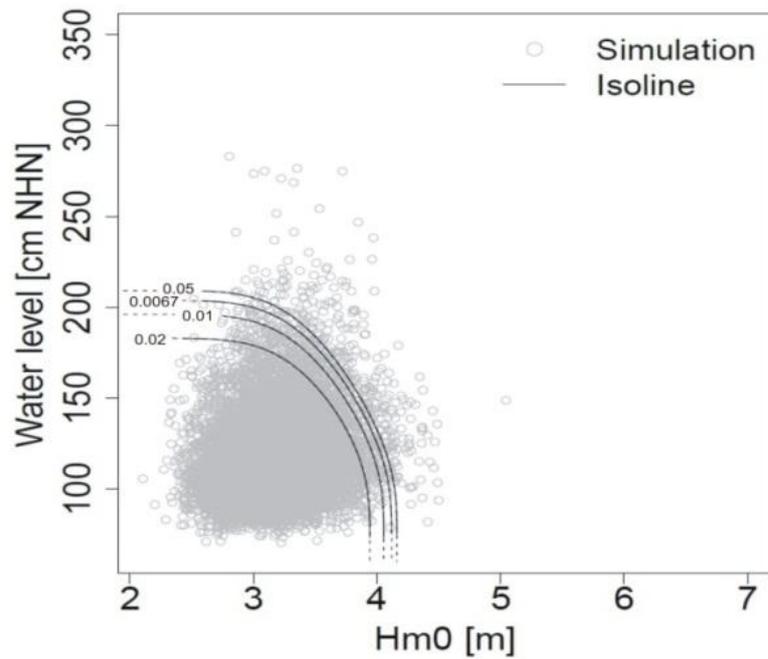


Fig. 4: Simulated data pairs and isolines with the requested probability of occurrence (Salecker et.al. 2012)

The profile of the adjacent beach is split into an upper part, the dry beach, with a sloping of 1:50 and a lower part, the wet beach, with a sloping of 1:100.

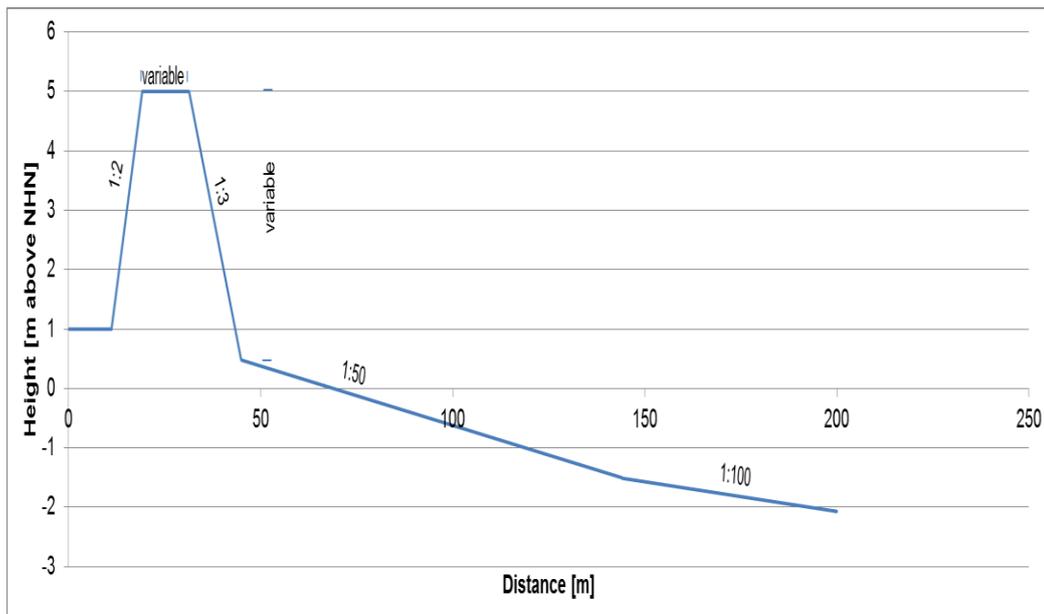


Fig. 5: Initial dune and beach profile

DUNE EROSION MODEL

Basis for the calculation of failure probabilities is the dune erosion model of Van Gent et.al. (2008).

This model contains an empirical approach under the assumption of an equilibrium profile. The initial profile of the dune and the adjacent beach are exposed to hydrodynamic forces of a storm surge and adapt to these forces. After a sufficient time, an equilibrium profile is reached and no further erosion of the dune and beach profile is assumed. In the model, the effects of storm durations are neglected, which means that the model is theoretically applicable for both, a comparatively short storm with duration of 5 hours with constant load and a comparatively long storm with a duration of e.g. 45 hours (Vellinga 1986, Brandenburg 2010).

The empirical model of Van Gent et.al (2008) considers a limited number of parameters, only. The wave height, the wave period and the fall velocity of the sediment (see equations (7) – (9)) are included in the dune erosion model directly whereas the storm surge water level is considered indirectly since the origin of the parabolic subaqueous erosion profile being located at storm surge water level.

Using equations (7) & (8) the seaward extent of the erosion profile is calculated. Equation (9) can be used to determine the parabolic subaqueous erosion profile. The connecting slope to the sea floor is assumed to be 1:12,5. A sloping of 1:1 is fixed for the eroded seaward dune face.

In order to obtain the equilibrium profile the erosion profile is shifted against the initial profile as long as the sum of the eroded areas (marked as V1 & V2 in Fig. 6) differs from the accumulated area (marked as V3 in Fig.6). The calculation of the areas is carried out by means of numerical integration.

Once this profile is attained the resulting crest width is determined. This parameter is needed for further calculations of failure probabilities and the derivation of the fragility curves.

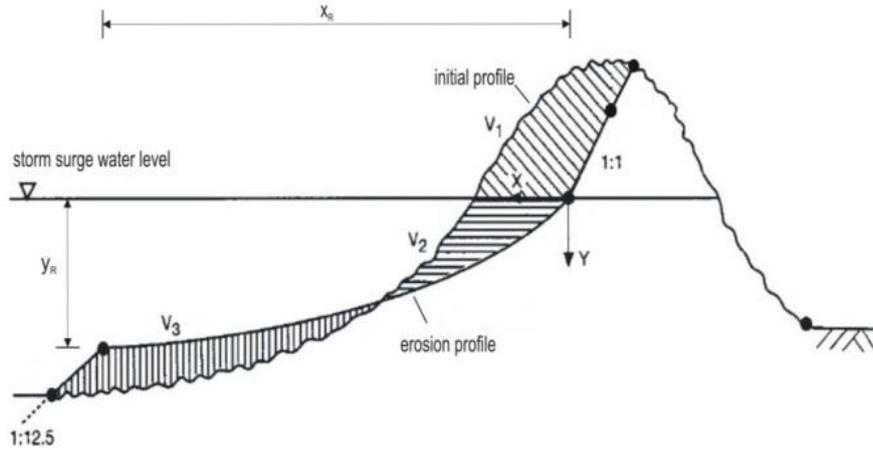


Fig. 6: Definition sketch for dune erosion model of Van Gent et.al. (2008)

$$x_R = 250 * \left(\frac{H_{0s}}{7,6}\right)^{1,28} * \left(\frac{0,0268}{w}\right)^{0,56} \quad (7)$$

$$y_R = \left(\frac{H_{0s}}{7,6}\right) * \left[0,4714 * \left(250 * \left(\frac{12}{T_p}\right)^{0,45} + 18 \right)^{0,5} - 2,0 \right] \quad (8)$$

$$\frac{7,6}{H_{0s}} = 0,4714 * \left[\left(\frac{7,6}{H_{0s}}\right)^{1,28} * \left(\frac{12}{T_p}\right)^{0,45} * \left(\frac{w}{0,0268}\right)^{0,56} * x + 18 \right]^{0,5} - 2,0 \quad (9)$$

Where H_{0s} is the significant wave height [m], T_p is the Peak period [sec], w is the fall velocity of the sediment [m/s], x and y is the distance from the origin [m], x_R and y_R is the seaward end of the erosion profile [m].

FAILURE FUNCTION

Failure probabilities are calculated using reliability analyses. The applied limit state equation compares the crest width resulting from the dune erosion with a minimum allowable crest width (see (10)).

$$Z = m * d_{crest_act} - d_{crest_crit} \quad (10)$$

Where m is a model-factor [-], d_{crest_act} is the crest width after erosion [m], d_{crest_crit} is the minimum allowable crest width [m]

The crest width resulting from the erosion is determined on the basis of the dune erosion model of Van Gent et.al. (2008) (see Dune Erosion Model). The minimum allowable crest width is set to 5 m corresponding to the width of the safety part of the flood protection dunes. The model factor 'm' describes uncertainties of the applied model.

The limit state equation is calculated for a multitude of combinations of storm surge water levels and wave heights with the same probability of occurrence. The failure probability can be regarded as the relative frequency of negative results of the limit state equation. Fig. 7 shows the failure probability in per cent as function of the crest width of the initial dune for different return periods. For combinations of water levels and wave heights with a return period of 200 years dune first failure of a respective dune has to be expected assuming a crest width of 14 m. For combinations with a higher probability of occurrence (return period of 150 years, 100 years and 50 years) dune failure starts at a crest width of 13 m, 12 m and 10 m, respectively.

As expected, with decreasing crest width of the initial dune the failure probability increases. Within these investigations a crest width of 8 m appears to be some kind of threshold of dune failure. Dunes with a crest width narrower than 8 m cannot even withstand storm surges with a return period of 50 years.

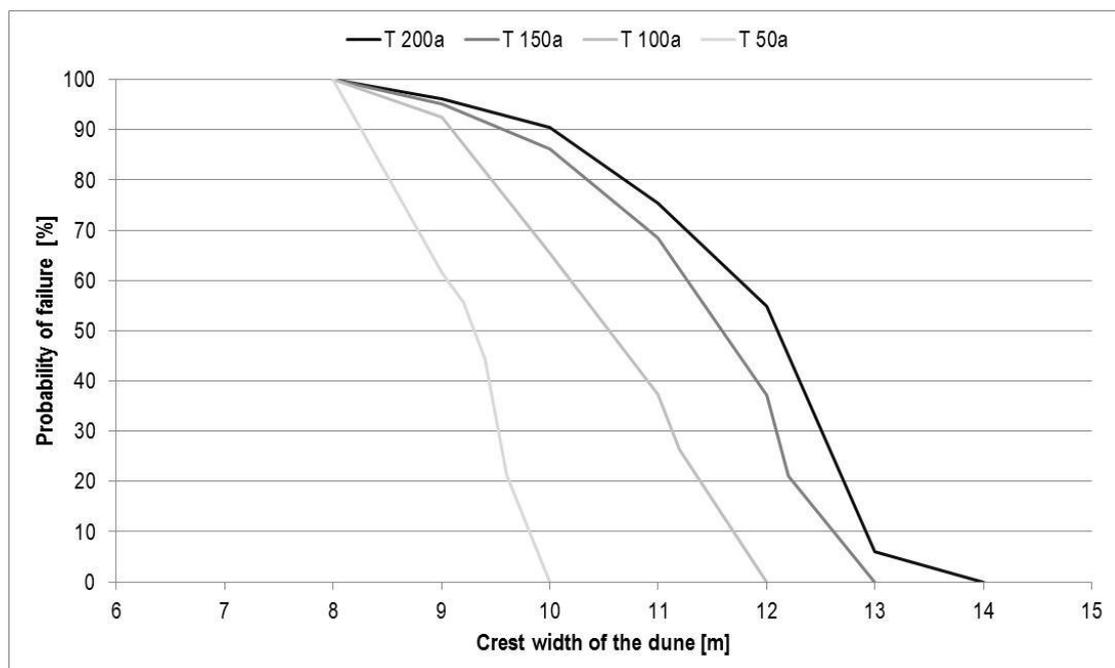


Fig. 7: Failure curves for various return periods of input data

FRAGILITY CURVES

Fragility curves are derived for the return periods of the input data of 200 years and 150 years, respectively. Probabilities of failure are calculated for discrete values of water levels in combination with all possible values of wave heights with the corresponding return period. Within the first investigations the calculation of the fragility curves is carried out for a fixed crest width of 12 m.

Fig. 8 shows the fragility curves for both return periods. It can be seen that, there is some kind of threshold with respect to the storm surge water levels. Water level below 1,9 m above NHN (German gauge datum, approx. mean sea level, MSL) will not cause dune failure. Once the water level exceeds the threshold of 1,9 m above NHN further increasing water levels also increase the probability of failure of the dune. Failure occurs within the range of 1,9 m and 2,1 m above NHN. It can also be seen, that the gradient of the fragility curve for input data with a return period of 150 years is smaller than for input data with a return period of 200 years. In the consequence, failure probabilities for data with the first mentioned return period are slightly lower than for data with the last mentioned return period.

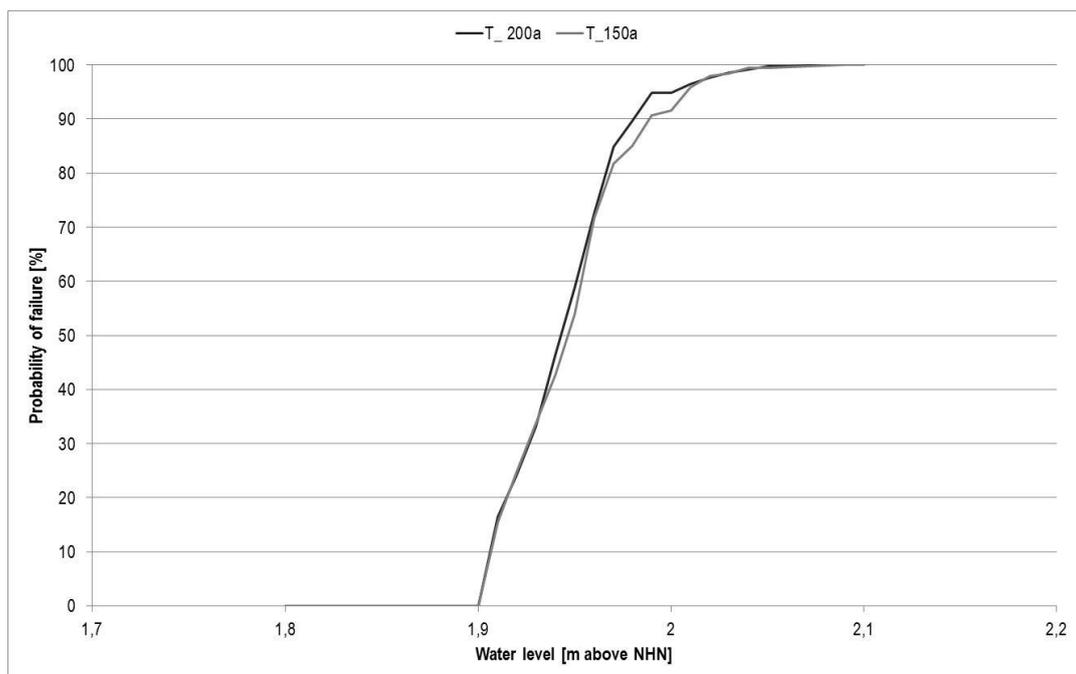


Fig. 8: Fragility curves for the stress water level

SUMMARY

In this paper a simple method to calculate failure probabilities and for the derivation of fragility curves for flood defence dunes are presented. Additionally, a method for the derivation of input data with a conjoint probability of occurrence is introduced.

To describe dune erosion the empirical model of Van Gent et. al. (2008) was used. With the help of this model reliability analyses on the basis of Monte Carlo simulations were carried out. Failure probabilities as function of the crest width of the initial dune were shown for different return periods of the input data. It can be concluded that an increase of the return period of the input data leads to an earlier failure initiation. For input data with the return period of 200 years failure starts at a crest level of 14 m, whereas dune failure starts at narrower crest widths for lower return periods of the input data.

Within the investigations the crest width of 8 m appears to be a kind of lower limit for the resistance of the flood defence dune against storms surges. Flood defence dunes with crest widths below this value cannot even withstand storm surges with a return period of 50 years.

Fragility curves as function of the storm surge water level were derived for return periods of 150 years and 200 years, respectively. These curves show some kind of threshold for the water level leading to a failure of the flood defence dune. Storm surge water levels below 1,9 m above NHN (German

gauge datum, approx. MSL) will not lead to dune failure. Once this threshold is exceeded further increasing water levels lead to increasing failure probabilities. Failure probabilities for input data with a return period of 150 years are slightly lower than for data with a return period of 200 years.

Further investigations on the basis of a numerical model will investigate the influence of the long shore sediment transport on the dune erosion, failure probabilities and fragility curves. Moreover, numerical model will be used to validate the investigation presented in this paper.

In addition, the influence of preceding damages of the dune due to foregone storm surges will be assessed.

ACKNOWLEDGEMENT

The project HoRisk is funded by the Federal Ministry for Education and Research and supported by KFKI (German Coastal Engineering Research Council).

REFERENCES

Allsop, W. et.al. ,2007: Failure Mechanisms for Flood Defence Structures, *FloodSite Report Nr. T04-06-01*, www.floodsite.net/html/publications2.asp?ALLdocs=on&Submit=View

Bachmann et.al. ,2009: Fragility Curve Calculation for Technical Flood Protection Measures by the Monte Carlo Analysis, In: *Flood Risk Management: Research and Practices*, Eds: Samuzels et.al., ISBN: 978-0-415-48507-4

Brandenburg, P. (2010): Scale dependency of dune erosion models - Performance assessment of the DUROS and XBeach model for various experiment scales, *Master Thesis*, Deltares, Rotterdam

Genest, C., Favre, A-C., 2007: Everything You always Wanted to Know about Copula Modeling but Were Afraid to Ask, In: *Journal of Hydraulic Engineering*, 12, pp. 347-368

Nelsen, Roger B., An Introduction to Copulas, Springer Series in Statistics, 2nd ed. 2006. Corr. 2nd. printing, 2006, XIV, 272 p.

Salecker et.al., 2012: Parameterisation of Storm Surge Hydrographs using Univariate and Bivariate Statistical Models, In: *Proceedings of the 8th International Conference on Coastal and Port Engineering in Developing Countries*, Copedec 2012, IIT Madras, Chennai, ISBN: 978-93-80689-06-7

Sklar, A. (1959). Fonctions de repartition a n dimensions e leurs marges. Publications de l'Institut de Statistique de l'Univiversite de Paris 8, 229 - 231

StALU ,2009: *Regelwerk Küstenschutz Mecklenburg-Vorpommern Übersichtsheft: Grundlagen, Grundsätze, Standortbestimmung und Ausblick*, Editor: Ministry for Agriculture, Environment and Consumer Protection Mecklenburg-Vorpommern

Van Gent, M.R.A. et.al. (2008): Large-scale dune erosion tests to study the influence of wave periods, *Coastal Engineering*, Vol. 55, S. 1041 – 1051, [doi:10.1016/j.coastaleng.2008.04.003](https://doi.org/10.1016/j.coastaleng.2008.04.003)

Vellinga, P. (1986): Beach and Dune Erosion during storm surges, *Dissertation*, <http://repository.tudelft.nl/view/ir/uuid%3Aeb7a4d20-86d2-469a-932a-dec0518274bb/>