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Assessment of maintenance efforts and probabilities of failure at German inland waterways to advance the design of bank revetments

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Abstract

Revetments protect waterways or flood defenses against erosion from waves and currents. In Germany a high percentage of about 7235 km of waterways is secured by revetments. Like many stakeholders of various infrastructures, the German Federal Waterways and Shipping Administration increasingly aims for a more economic and ecological design and maintenance strategy. Thus, new methodologies must be introduced that relate the structural condition of the revetment to consequences such as required maintenance. In this paper, we investigate the correlation of maintenance and revetment stability. Using the example of German inland waterways, maintenance measures conducted over at least six years are correlated with a deterministic and a probabilistic stability assessment. To account for realistic traffic loads, the stability assessment employs field measurements with data on ship-induced waves. It was found that a positive correlation between revetment stability and maintenance must be assumed. A comparison between the deterministic and the probabilistic stability assessment shows that particularly for small sample sizes and low failure probabilities, the probabilistic approach should be favored over the deterministic approach. In the case that only maintenance is of relevance for design considerations, the results of the probabilistic analyses indicate that β = 1.3 ($p_f \approx 10^{-1}$) may be a suitable first estimate for an annual target reliability.

Keywords

Riprap revetments, probability of failure, maintenance, target reliability

1 Introduction

The German Federal Waterways and Shipping Administration (WSV) maintains about 7235 km of waterways whose shores are mainly secured by loose or grouted armor stones. To enable the WSV to make optimal use of its resources, new methodologies must be introduced that relate the structural condition of the revetment to the consequences of a condition assessment such as maintenance. While the limits of safety or a change in the hazard situation are not at issue, increasing economic and ecological demands require factors of safety or target reliabilities that are optimized with respect to potential risks associated with damage or failure.

In Germany, the design of armor stone revetments is conducted according to BAW Code of Practice: Principles for the Design of Bank and Bottom Protection for Inland Waterways (GBB, 2010), which comprises a hydraulic and a geotechnical design. While the hydraulic design defines the minimum armor stone diameter necessary to withstand (ship-induced) waves and currents, the geotechnical design evaluates the armor layer thickness required to ensure embankment stability. Expert interviews (Sorgatz et al., 2018) have shown that armor stone displacement is the most significant damage from the point of maintenance, but also in terms of possible failures as they are the most frequently observed. Thus, this paper focuses on armor stone displacements or, in other words, the hydraulic design.

In literature, a number of approaches for a risk-informed design of hydraulic structures are known. For bank protections, PIANC (1987) outlines an approach to find a suitable factor of safety or target reliability based on initial investment costs and costs resulting from unserviceability or failure over the lifetime of the structure. The Rock Manual (2007), which considers hydraulic structures in general, recommends to minimize the sum of investment, maintenance, monitoring and failure costs to obtain the optimal lifetime costs. Schweckendiek et al. (2012) describe an approach adopted in the Netherlands to define a risk-informed target reliability for flood defense structures that considers economic, individual and societal risk criteria. A practical guideline for a multivariate structural risk assessment for coastal and offshore structures is presented by Salvadori et al. (2015). PIANC (2016) provides a guideline specifically for breakwaters that defines the optimum safety level as a function of the costs resulting from construction, maintenance, repairs and demolition, depositing and reuse of materials over the lifetime of the structure. So far, the outlined approaches are either rather generic in nature, as often the required information on construction costs and costs over lifetime of a hydraulic structure are not available or they focus on coastal protection structures, which experience different loads due to storm surges and flooding. Yet, inland waterways are mainly subjected to shipping traffic and have a different significance which is predominantly based on their use as transport routes.

This paper investigates the relation between stability and maintenance for riprap revetments. Since maintenance of riprap revetments usually consists of (re-)placing armor stones, it is assumed that a correlation between the likelihood of armor stone displacement and the yearly placed tonnage of armor stones can be found. Using the example of German inland waterways, maintenance measures that were collected over several years are compared to the revetments' stability that is characterized by the probability of failure. As (non-Bayesian) probabilistic approaches are often rejected due to a limited data basis, additionally, it is investigated whether comparable correlations can be found for a deterministic stability assessment. The investigations aim at proving a basis for a more risk-oriented design approach for bank revetments at inland waterways. Moreover, in the case that only maintenance efforts are of relevance for design considerations, the presented investigations may provide insights about suitable target reliabilities.

The paper is organized as follows: Data collection, maintenance measures and field measurements, are introduced in Section 2. In addition, the methodology for the presented stability assessment is outlined. Section 3 presents the results of the analyses combining the elicited maintenance measures and the deterministic and probabilistic stability assessment. In Section 4, methodology and findings are discussed with regard to suitable target reliabilities and the applicability of the proposed methodology. Finally, this paper closes with a brief recap of results and an outlook regarding future research.

2 Methodology

2.1 Data and data collection

2.1.1 Waterway and revetment characteristics

The present study considers data from three different canals (see Figure 1). The Dortmund-Ems Canal (DEK) is located in the North-West of Germany. With a length of 269 km it connects the inland port of the city of Dortmund and the seaport of Emden. The Midland Canal separates the DEK into a Northern (DEK, North) and Southern part (DEK, South) and provides connections to rivers Weser and Elbe. The Rhine-Main-Danube Canal (MDK) is located in Southern Germany. It connects the Main near Bamberg and the Danube river near Kelheim (approx. 171 km). The third canal, the Wesel-Datteln Canal (WDK), is a 60-kilometre long canal in the west of Germany. It is a major European transport route connecting the lower Rhine with Northern and eastern Germany. All examined waterways but the DEK (North), which is classified as category B waterway, are categorized as category A waterways, which means that these are waterways carrying over 5 million tons of goods per year. In contrast to waterways of category B and C, larger investments for extension, but also maintenance measures can be budgeted for these waterways.

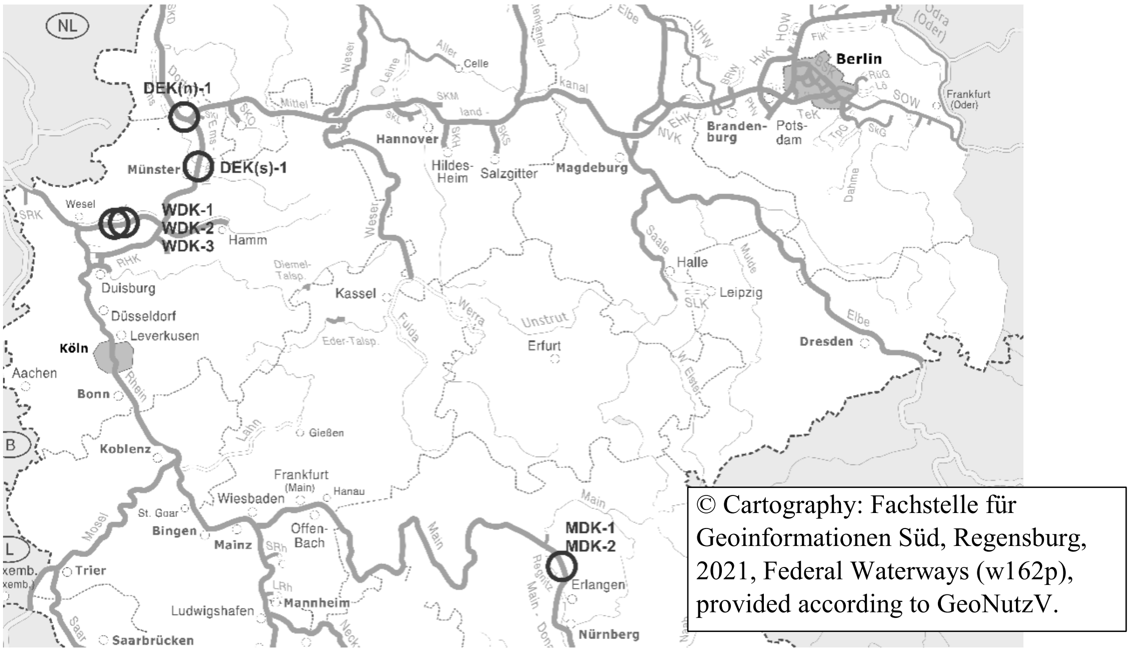

Figure 1: Map of Germany. The bold circles point to the observation locations. Please note, drawings are not in scale.

At the time of the measurements, DEK (North) belonged to the waterway class IV according to the Classification of European Inland Waterways (CEMT), which means that vessels of a length of 80.00–85.00 m, a width of 9.50 m and a draught of 2.50 m can pass through the canal. DEK (South) is classified as waterway class IV and class V allowing vessels of 85.00 m length, 9.50 m width and 2.50 m depth to pass. MDK and WDK belonged to the waterway class Vb allowing for the passage of vessels of a length of 172.00–185.00 m, a width of 11.40 m and a draught of 2.50–4.50 m. Table 1 summarizes main characteristics of the investigated waterways.

In fairways confined laterally and in depth, the hydraulic loads are significantly governed by the vessels' velocity in relation to the blockage ratio n that describes the ratio between the cross-sectional area of the waterway A_W and the cross-sectional area of the submerged part of a vessel A_S $(n = A_W / A_S)$. The smaller the blockage ratio, the slower the vessels can pass. For modern waterways in Germany, the blockage-ratio of $n \geq 5.3$ is to be considered as reasonable compromise between economic navigation and ease of navigation (BMVBS, 2011). Water level and blockage ratio of the investigated canals are nearly constant throughout the year. To ensure safe travels, the water level is regulated by weirs and locks in order to not fall below or exceed the lower or upper operating water level. As a result, the water level varies by a maximum of 0.65 m throughout the year. In contrast to waterways with significant water level fluctuations such as rivers and estuaries, where damage can spread over the entire slope, the ship waves will impact a smaller area and cause concentrated damage. The exposure of the filter layer is therefore more likely.

Table 1: Summary of waterway characteristics based on Kayser (2006, 2007a, 2007b, 2008).

Waterway	Year of construction	Waterway class	Armor stone diameter $D_{50,site}$	Slope inclination m	Blockage ratio n
	--	--	mm	--	--
DEK (North)	1950´s	IV	145 – 170	2.1 – 3.0	4.1 – 4.6
DEK (South)	1999 – 2002	IV / V	175	3.0	7.3
WDK	1984 – 1987	Vb	170 – 190	3.0	5.2 – 7.2
MDK	1966	Vb	200 – 230	3.0	6.8 – 8.2

2.1.2 Field observations

Hydraulic loads are measured in specific cross-sections. However, if the cross-section geometry and the vessel navigation do not change significantly, measurements can be regarded as representative for a larger canal section. Common design guidelines, e. g., Rock Manual (2007), GBB (2010), require to include ship-induced waves and currents when assessing or designing a revetment. In the case of this study, only the stern wave height H_{stern} was recorded, since the measurement of the ship-induced currents requires a more complex and expensive measurement set-up. Yet, since the ship-induced currents are directly correlated with the stern wave height, it can be assumed that wave measurements represent an acceptable approximation of the load situation at each canal.

The waves were recorded by means of absolute pressure probes (Driesen + Kern GmbH, type P-LOG125-B and P-LOG520-A, see Figure 2, left) which were positioned at two (approx. 0.5 m and 1.0 m below water level) or three different depths (approx. 0.5 m, 1.0 m and 1.5 m below water level) of the sloped embankment. The pressure probes have the following characteristics:

- high-precision piezoresistive sensors (0 to 400 kPa),
- measurement of air pressure (800 to 1200 kPa) and temperature in water and air (0.2 to +80.0°C) and
- measuring accuracy: 0.1 % of the measuring range.

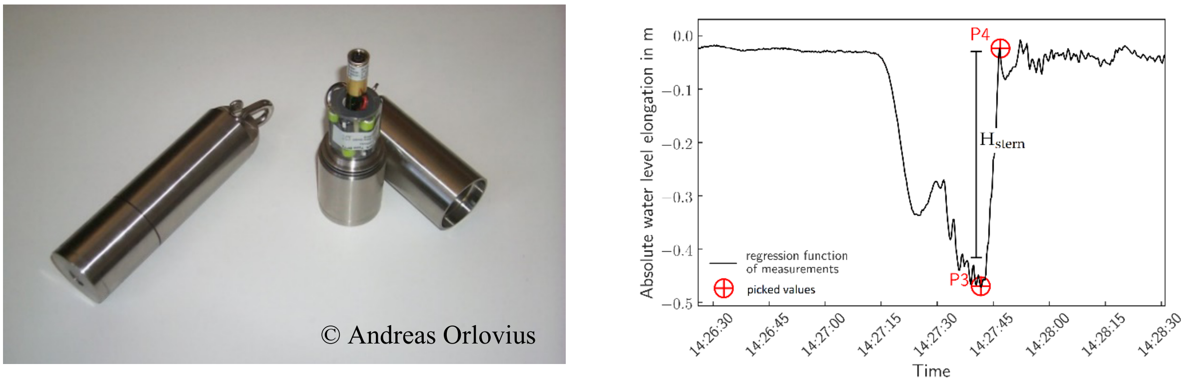

Figure 2: Absolute pressure probes, Driesen + Kern GmbH, type P-LOG125-B and P-LOG520-A (left) and post-processing of field observations (right).

As a result of the limited data storage capacity of the loggers 15 – 20 years ago, a recording frequency of 2 Hz was chosen. Fortunately, preliminary investigations at that time with a recording frequency of 4 Hz showed that a recording frequency of 2 Hz is sufficient to provide accurate maximum wave height readings. From the measured water level fluctuations, waves are determined for each vessel passage. This process is illustrated in Figure 2, right for H_{stern}. For each wave event the points marked in the graph are picked manually by an experienced consultant. The difference between P4 and P3 yields H_{stern}. A comprehensive summary of the measurement data is provided in Table 2. To compare the stability analyses of different datasets later on, a code for each dataset is introduced which is constructed as follows: The upper-case letters refer to the canal, the lower-case letter distinguishes in North (N) and South (S), followed by a consecutive number for each dataset per canal.

Table 2: Summary of data statistics of H_{stern} derived from field measurements. Values are provided in m.

	14 – 21 September 2002	11 – 25 July 2006	26 July – 09 August 2004		10 – 23 May 2005		
Canal Section	DEK(S)-1 / *Los 14*	DEK(N)-1 / *Rodde*	MDK-1 / *Strullendorf*	MDK-2 / *Strullendorf*	WDK-1 / *I*	WDK-2 / *I*	WDK-3 / *II*
km	81.600	111.200	23.800	23.900	36.615	36.710	41.000
count	198	253	207	202	509	521	319
mean	0.18	0.25	0.30	0.29	0.23	0.25	0.26
standard deviation (std)	0.07	0.10	0.11	0.10	0.09	0.09	0.08
coefficient of variance (cov)	0.38	0.41	0.38	0.33	0.37	0.36	0.32

2.1.3 Maintenance measures

The majority of damages at riprap embankment is observed along the shore close to still water level, e. g., Sorgatz (2021). The maintenance of riprap revetments commonly encompasses the (re-)placement of armor stones to fix minor to moderate discontinuities. It is therefore assumed that the annual amount of placed armor stones represents the maintenance effort required for the considered stretch of the waterway. As the maintenance of the riprap is executed periodically, the maintenance measures per year can differ largely. Thus, a time span of several years must be regarded in order to get a representative value for the average annual maintenance and maintenance costs.

During 2001 and 2014 an annual survey was conducted to elicit tons (and costs) of maintenance measures for several canal sections (see Table 3 and Figure 3). The data was acquired from the local agencies of the WSV which are responsible for the maintenance of the waterways. In order to get consistent data, every stretch was inspected together with the local authority at the start of the elicitations and the authors provided guidelines for the acquisition of the data. At the beginning of every year, the data of the preceding year were transmitted to the Federal Waterways Engineering and Research Institute (BAW). The following investigations focus on the maintenance efforts in terms of the annual mass of placed armor stones rather than on the maintenance costs, since the costs may be affected by factors such as the availability of personnel and machinery as well as by locally and over time differing costs for armor stones. However, in order to make the data available to other researchers, the elicited maintenance costs are presented in the Appendix, Table A.1.

Table 3 and Figure 3 show that complete data are not available for all waterways. In addition, even on waterways where major maintenance is required within the 13 years of elicitation (darker colored squares in Figure 3, there has been less maintenance in the years in-between (lighter colored squares in Figure 3) highlighting the need for a long-term data collection. For waterways that follow the current standard (MAR, 2008) with $n \geq 5.3$, the required mass of armor stones per year is relatively low (0 to 0.5 kg/m²·a), while at the less modern waterway DEK (North) more armor stones (3 – 15 kg/m²·a) are required for maintenance. Only DEK (South), Los 14 falls out of this scheme with 3.57 kg/m²·a. This may be a result of its unusual construction: the armor stones were covered with soil above the water level directly after placement and grass was seeded in order to improve the ecological value of the shore. Due to the fast-growing plants, the armor stones above the water level were fixed by the mix of soil and roots. Armor stone displacements which normally occur after construction and which lead to the stabilization of the system could not be compensated by the movement of armor stones above the waterline resulting in, e. g., a directly exposed filter layer (see Figure 4).

Although not discussed further in this paper, a significant improvement in the correlations between maintenance measures and failure probability (see Section 3.2) could be achieved with a conversion of replaced armor stones from t/km to kg/m². The revetment area considered for investigations results from the canals' depth and the surface area of bottom and bank. Sections that are secured by sheet pile walls, e. g., near constructions or ship berths, are omitted to obtain the total riprap revetment area (bottom and bank) that requires maintenance.

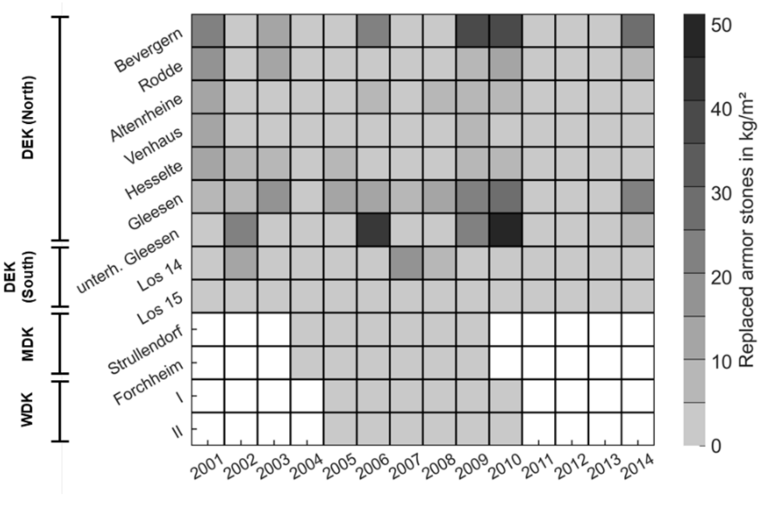

Figure 3: Visualization of annual maintenance measures between 2001 and 2014 for the different waterways. The darker the colored square, the more maintenance was conducted. White squares indicate missing data.

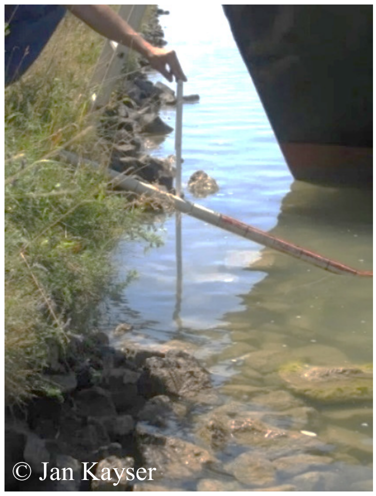

Figure 4: Damage in DEK (South).

Table 3: Elicited maintenance measures (replaced armor stones) in kg/m² for the period from 2001 to 2014. Gray highlighted rows indicate waterway sections that are not considered in the correlation analyses due to unavailable field observations (see also Section 2.2.4).

Canal	Section	from km	to km	Replaced armor stones 2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	Sum
				kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/m²	kg/(m²a)
DEK (North)	Bever-gern	108.4	109.3	24.40	0.00	12.58	1.41	2.44	22.99	0.00	1.03	39.98	39.32	0.00	0.00	0.00	30.69	14.57
	Rodde	109.3	112.5	16.89	0.00	10.92	0.22	3.07	4.17	0.00	4.08	8.60	12.98	0.00	0.00	0.00	6.49	5.62
	Alten-rheine	112.5	117.9	13.77	0.00	1.15	1.49	3.07	8.85	0.00	5.86	9.98	10.05	0.00	0.00	0.00	0.99	4.60
	Venhaus	117.9	126.6	11.45	4.63	3.88	0.56	3.41	5.10	0.64	0.62	6.14	4.56	0.00	0.00	0.00	3.01	3.67
	Hesselte	126.6	134.5	12.46	10.12	6.64	0.83	8.83	3.90	4.20	1.71	6.33	5.73	0.00	0.00	0.00	3.66	5.37
	Gleesen	134.5	137.8	9.92	9.13	15.67	3.97	14.88	15.04	8.61	13.17	21.87	27.58	0.00	0.00	0.00	25.12	13.75
	unterh. Gleesen	137.8	138.3	0.00	25.00	0.00	0.00	0.00	41.67	0.00	0.00	23.33	51.25	0.00	0.00	0.00	8.75	12.50
DEK (South)	Los 14	79.4	84.0	0.00	12.97	0.00	3.02	0.00	0.00	17.19	8.87	0.00	0.36	0.00	0.00	0.00	0.45	3.57
	Los 15	84.0	89.0	0.00	0.00	0.00	0.00	0.00	0.00	0.37	3.08	1.33	0.23	0.00	0.00	0.00	0.91	0.49
MDK	Strullen-dorf	21.8	25.4				0.00	3.21	1.71	0.00	0.00	0.00						0.70
	Forch-heim	26.4	32.0				0.14	0.18	0.00	0.00	0.00	0.00						0.05
WDK	I	35.5	37.9					0.00	0.00	0.00	0.00	0.00	0.00					0.00
	II	39.7	42.8					0.00	0.00	0.00	0.00	0.00	0.00					0.00

2.2 Stability analyses

2.2.1 Conceptional definitions

DIN 1990 (2010, p. 17) defines reliability as "ability of a structure or a structural member to fulfil the specified requirements, including the design working life, for which it has been designed. Reliability is usually expressed in probabilistic terms." Thus, the reliability of a structure or component applies to the structural integrity, the serviceability and the durability with regard to the intended lifetime. In the course of this paper, the displacement of armor stones is considered as serviceability limit state (SLS). It is assumed that the Ultimate Limit State (ULS) is only reached when the filter layer or soil is exposed locally. This condition is avoided by (re-)placing armor stones in time (PIANC, 1987; Oumeraci et al., 1999; Sorgatz, 2021). The main advantage of this methodology is that a complete loss of stability (slope failure) does not have to be considered.

For the purpose of a probabilistic design or assessment, the reliability of the structure is compared to the target reliability. The target reliability is a "specified average acceptable probability of failure that is to be reached as close as possible" (ISO 2394, 2015, p. 5). Target reliabilities allow to relate the condition of a structure to the consequence associated with a failure in terms of damage, cost or loss of life. They can be expressed via the probability of failure p_f or the reliability index β. The functional relationship between p_f and β is given via the standard Gaussian distribution Φ as:

$$p_\mathrm{f} = \Phi(-\beta) \tag{1}$$

Commonly a similarity between coastal structures and revetments is assumed which may allow to transfer target reliabilities proposed for coastal structures to the assessment and design of revetments in inland areas. Vrijling (1999) and PIANC (2003) categorized target reliabilities for vertical breakwaters with a design lifetime of 50 years by their limit state and safety class. The proposed values range between $p_\mathrm{f} = 0.05$ and $p_\mathrm{f} = 0.40$ (see Table 4). These values are confirmed by PIANC (2016) who investigated various failure mechanisms and conducted additional parameter studies in the context of a life cycle analysis. On a broader basis, the Joint Committee on Structural Safety (JCSS, 2001) defines annual target reliabilities for ULS and SLS which are independent of the type of structure (see Table 5). Consequences are incorporated by dividing the classification into three consequence classes.

Table 4: Indicative values of acceptable (maximum) probabilities of failure within structure lifetime (PIANC, 2003).

Limit State	Safety Class			
	Very low	Low	Normal	High
SLS	0.40	0.20	0.10	0.05
ULS	0.20	0.10	0.05	0.01

Table 5: Target reliability indices (and associated probabilities of failure) related to one-year reference period; ULS and SLS (JCSS, 2001).

Ultimate Limit State				Serviceability Limit State	
Relative cost of safety measure	Minor consequences of failure	Moderate consequences of failure	Large consequences of failure	Relative cost of safety measure	Target index (irreversible SLS)
Large (A)	$\beta = 3.1$ ($p_\mathrm{f} \approx 10^{-3}$)	$\beta = 3.3$ ($p_\mathrm{f} \approx 5\cdot10^{-3}$)	$\beta = 3.7$ ($p_\mathrm{f} \approx 10^{-4}$)	High	$\beta = 1.3$ ($p_\mathrm{f} \approx 10^{-1}$)
Normal (B)	$\beta = 3.7$ ($p_\mathrm{f} \approx 10^{-4}$)	$\beta = 4.2$ ($p_\mathrm{f} \approx 10^{-5}$)	$\beta = 4.4$ ($p_\mathrm{f} \approx 5\cdot10^{-6}$)	Normal	$\beta = 1.7$ ($p_\mathrm{f} \approx 5\cdot10^{-2}$)
Small (C)	$\beta = 4.2$ ($p_\mathrm{f} \approx 10^{-5}$)	$\beta = 4.4$ ($p_\mathrm{f} \approx 5\cdot10^{-6}$)	$\beta = 4.7$ ($p_\mathrm{f} \approx 10^{-6}$)	Low	$\beta = 2.3$ ($p_\mathrm{f} \approx 10^{-2}$)

2.2.2 Design equations

In 1959, Hudson (1959) proposed a semi-empirical equation to determine the required armor stone diameter against waves based on a large number of flume tests. The Hudson equation offers the advantage of simplicity and is valid for a wide range of armor units and conditions. In 1987, van der Meer (1987) proposed two design equations, one for plunging and one for surging waves which are nowadays also frequently used for the design of loose riprap structures. Neither the van der Meer nor the Hudson formula or any related extensions cover waves travelling parallel to the shore, thus, the design equation against armor stone displacement that is formulated in the German design standard "Principles for the Design of Bank and Bottom Protection for Inland Waterway" (GBB) only relies on the "basic features" of the van der Meer or the Hudson formula. The wave height at which damage occurs is proportional to the relative density, a characteristic grain size of the loose riprap, and a function of the slope inclination. By means of field tests (BAW, 2009) a derivative of the Hudson equation with a proportionality constant (B'_{B} or B^*_{B}) was calibrated that can be used for the design of loose armor stone revetments subjected to ship-induced waves in fairways confined laterally and in depth.

Following GBB (2010), the required mean armor stone diameter $D_{50,\mathrm{req}}$ can be determined as follows:

$$D_{50,\mathrm{req}} \geq \frac{H_{\mathrm{stern}}}{B'_{\mathrm{B}}\left(\frac{\rho_{\mathrm{S}}-\rho_{\mathrm{W}}}{\rho_{\mathrm{W}}}\right)m^{\frac{1}{3}}} \quad , \qquad B'_{\mathrm{B}} = [1.5,\ 2.3] \tag{2}$$

$$D_{50,\mathrm{req}} \geq \frac{H_{\mathrm{stern}}\, C_{\mathrm{slope}}}{B^*_{\mathrm{B}}\left(\frac{\rho_{\mathrm{S}}-\rho_{\mathrm{W}}}{\rho_{\mathrm{W}}}\right)m^{\frac{1}{3}}}, \qquad B^*_{\mathrm{B}} = [2.0,\ 3.0] \tag{3}$$

$$C_{\mathrm{slope}} = \frac{1}{\cos\delta\left[1-\left(\frac{\tan^2\delta}{\tan^2\phi'_{\mathrm{D,hydr}}}\right)\right]^{0.5}} \tag{4}$$

where H_{stern} is the stern wave height, B'_{B} and B^*_{B} are empirical factors considering the revetment stability, C_{slope} is an empirical factor considering the slope inclination, $\phi'_{\mathrm{D,hydr}}$ is the effective angle of repose of the armor stones, ρ_s is the density of the armor stones, ρ_{W} the density of water, $m = \cot\delta$ is the slope inclination with δ as slope angle.

2.2.3 Deterministic and probabilistic stability assessment

In this section, two calculation approaches for the stability assessment of revetments are presented. In the first approach, the hydraulic loads are described via their measured values. Characteristic values of slope geometry and material properties are used. In the second approach, the input variables are described via their probability density functions. In both cases, a "probability of failure" can be determined. For simplicity, we refer to calculation method and probability of failure of the former as "deterministic" and the latter as "probabilistic".

Deterministic stability assessment

In the case of the deterministic calculations, firstly, a utilization rate η is defined; $\eta > 1$ indicates that the required armor stone diameter exceeds the armor stone diameter in-situ which will result in armor stone displacements. Equation (5) features a set of deterministic variables $\mathbf{X}$ and is defined as the ratio of the maximum of the required armor stone diameter $D_{50,\mathrm{req}}$ and the armor stone size in-situ $D_{50,\mathrm{site}}$.

$$\eta(\mathbf{X}) = \frac{\max D_{50,\mathrm{req}}\,(\mathbf{X})}{D_{50,\mathrm{site}}} \tag{5}$$

For the purpose of stability assessment, the maximum of the observed wave heights and the mean values of the armor stone characteristics (see Table 6 and Table 7) are used. A conservative (damage and maintenance are not permitted) and a less strict (moderate damage and maintenance are allowed) design are considered by means of the B_{B}'* values. Failure is defined as follows:

$$I_d(\mathbf{X}) = \begin{cases} \text{stable } (0), & \text{if } \eta(\mathbf{X}) \leq 1 \\ \text{unstable } (1), & \text{if } \eta(\mathbf{X}) > 1 \end{cases} \tag{6}$$

Only if $\eta > 1$ is observed, the maintenance over one year can be extrapolated from the number of daily exceedances of η. For this purpose, the required armor stone diameter is computed for each of the observed wave events and, subsequently, compared to the armor stone diameter in-situ. The ratio of number of failures and total number of observations n_{total} results in a "deterministic" probability of failure $p_{\mathrm{f,d}}$, see eq. (7).

$$p_{f,d} = \frac{1}{n_{\mathrm{total}}}\sum_{i=1}^{n_{\mathrm{total}}} \mathbf{1}_{\{\eta(\mathbf{X})>1\}} \tag{7}$$

Probabilistic stability assessment

The probabilistic calculations are performed with the Python package OpenTURNS (Baudin et al., 2015). Due to a highly non-linear limit state function Monte-Carlo simulations were employed to determine the probability of failure. The ratio between the asymptotic standard deviation of the probability estimate and its mean value is used as convergence criteria. It is a relative measure of dispersion and must be lower than 0.05 in order to end a simulation.

The limit state function g for the reliability-based analysis is identical to the design equation of the deterministic (partial factor-based) analysis, see eq. (5). Instead of a set of deterministic variables, random variables are used. Failure is then defined by the indicator function I_{p}:

$$I_{\mathrm{p}}(\mathbf{X}) = \begin{cases} \text{stable } (0), & \text{if } g(\mathbf{X}) \leq 1 \\ \text{unstable } (1), & \text{if } g(\mathbf{X}) > 1 \end{cases} \tag{8}$$

The "probabilistic" probability of failure $p_{\mathrm{f,p}}$ is then defined as follows:

$$p_{\mathrm{f,p}} = \frac{1}{n_{\mathrm{total}}} \sum_{i=1}^{n_{\mathrm{total}}} \mathbf{1}_{\{g(\mathbf{X})>1\}} \tag{9}$$

To account for the correlation of the random variables, a joined probability density function is required. It is found via a Gaussian copula $C(\mathbf{X})$ for modeling multivariate dependence during the reliability analyses:

$$f\left(H_{\mathrm{stern}}, D_{50,\mathrm{site}}, \rho_{\mathrm{s}}, B_{\mathrm{B}}^{*}, B_{\mathrm{B}}'\right) = C\left(f_{H_{\mathrm{stern}}}, f_{D_{50,\mathrm{site}}}, f_{\rho_{\mathrm{s}}}, f_{B_{\mathrm{B}}^{*}}, f_{B_{\mathrm{B}}'}\right) \tag{10}$$

$$C\left(f_{H_{\mathrm{stern}}}, f_{D_{50,\mathrm{site}}}, f_{\rho_{\mathrm{s}}}, f_{B_{\mathrm{B}}^{*}}, f_{B_{\mathrm{B}}'}\right) = \Phi_{\mathbf{R}}^{5}\left(\Phi^{-1}\left(f_{H_{\mathrm{stern}}}\right), \Phi^{-1}\left(f_{D_{50,\mathrm{site}}}\right), \Phi^{-1}\left(f_{\rho_{\mathrm{s}}}\right), \Phi^{-1}\left(f_{B_{\mathrm{B}}^{*}}\right), \Phi^{-1}\left(f_{B_{\mathrm{B}}'}\right)\right) \tag{11}$$

with the marginal distributions $f_{H_{\mathrm{stern}}}, f_{D_{50,\mathrm{site}}}, f_{\rho_{\mathrm{s}}}, f_{B_{\mathrm{B}}^{*}}, f_{B_{\mathrm{B}}'}$ and the cumulative distribution function of the Gaussian distribution $\Phi_{\mathbf{R}}^{5}$ with zero mean, the marginal variances of the random variables, and the Spearman correlation matrix $\boldsymbol{R}$. Further information on the random variables and their correlations are outlined in Section 2.2.4.

Consideration of traffic

To account for different measurement duration and traffic volume at the investigated canal sections, the equation of annual probability of exceedance known from flood frequency analyses is applied to the calculated $p_{f,p}$. Firstly, the probability of failure per vessel $p_{\mathrm{v,p}}$ is determined by:

$$p_{\mathrm{v,p}} = 1 - \exp\left[\frac{\ln\left(1 - p_{\mathrm{f,p}}\right)}{n_{\mathrm{m}}}\right] \tag{12}$$

where n_{m} represents the number of vessels observed in each campaign. Subsequently, the annual probability of failure $p_{\mathrm{a,p}}$ is calculated. The number of vessels per year n_{y} is estimated based on the number of observed vessels and the duration of the campaign.

$$p_{\mathrm{a,p}} = 1 - \left(1 - p_{\mathrm{v,p}}\right)^{n_{\mathrm{y}}} \tag{13}$$

In the case of the deterministic approach, the traffic related probabilities of failure, $p_{\mathrm{v,d}}$ and $p_{\mathrm{a,d}}$, are calculated in the same way as their probabilistic equivalents, see eq. (12) and eq. (13). For a large number of samples, $p_{\mathrm{f,d}}$, should approach $p_{\mathrm{f,p}}$.

2.2.4 Case studies and input parameters

From the combination available field measurements, a number of case studies can be formulated. The case studies summarized in Table 6 are subsequently denoted as general case studies. In the case of DEK (North), detailed data on maintenance measures for each canal section is available (see Table 3). The wave heights measured at DEK (North) km 111.200 (Rodde) are transferred to the other sections allowing for a more site-specific analysis for DEK (North) with respect to geometry and revetment characteristics (see Table 7).

Methodologically, the two case study groups are analyzed in the same way. Failure probabilities are determined using measured wave events and information on the canal geometry and the revetment construction. These are related to the collected data on maintenance measures (see Section 3). The wave heights are spatially extrapolated from the measurement in one cross-sectional profile for an entire canal section. The only difference between the general case studies and the DEK (North) case studies is that in the former the extrapolation is carried out for a maximum of approximately 10 km. In the latter, the wave events are extrapolated for a distance of 30 km. The assumption that measured wave events are transferable to other canal sections is a common procedure in revetment design. It is justifiable under the constraint that geometry and fairway in the canal do not change significantly as it is the case for the considered sections of the DEK (North).

Due to the unusual construction, the elicited maintenance measures of DEK (South), *Los 14* will not be considered for further analyses. Instead, the elicited maintenance measures of DEK (South), *Los 15* will be used for the stability assessment of DEK (South). This section has a comparable geometry and traffic situation. Furthermore, the elicited maintenance measures for MDK, *Forchheim* are not considered as the canal geometry differs significantly compared to that of MDK, *Strullendorf,* which means that the wave measurements are not transferable. DEK (North), *unterh. Gleesen* and *Bevergen* are not included in subsequent analyses, as these sections are rather short canal sections, one at the confluence of the Mittelland Canal and one at the confluence of the river Ems. In these sections, the traffic behavior is not comparable with the traffic behavior in the other canal sections.

Table 6: Summary of revetment constructions in the different cross-sections where field measurements are available, subsequently referred to as general case studies.

Wave measurements	DEK(S)-1	DEK(N)-1	MDK-1	MDK-2	WDK-1	WDK-2	WDK-3
Slope inclination	1:3.0	1:2.1	1:3.0	1:3.0	1:3.0	1:3.0	1:3.0
$D_{50,\text{site}}$ in mm	175	170	230	230	170	170	190
Maintenance location	Los 15 [a)]	Rodde	Strullendorf	Strullendorf	I	I	II

[a)] Extrapolated data, the measurement location DEK(S)-1 was located at km 81.6, whereas the elicited maintenance measures refer to km 84.0–89.0.

Table 7: Summary of revetment constructions in the different cross-sections where field measurements are available, subsequently referred to as DEK (North) case studies. The wave heights originate from DEK (North), km 111.200 (Rodde).

	Altenrheine	Gleesen	Heeselte	Rodde	Venhaus
Slope inclination	1:2.1	1:2.1	1:2.1	1:2.1	1:2.1
$D_{50,\text{site}}$ in mm	165	155	160	170	145

For the probabilistic analyses the following distributions and correlations are employed: The analyses of various measurements at different canals by Sorgatz (2021) indicates that H_{stern} is lognormally distributed. The present data supports these findings. Figure 5 shows the fit of two different distribution types to H_{stern} for the examples MDK-1 and WDK-1. Particularly at the lower end and the body of the distribution, it can be noticed that the Lognormal distribution provides a slightly better fit to the data than the Gaussian distribution.

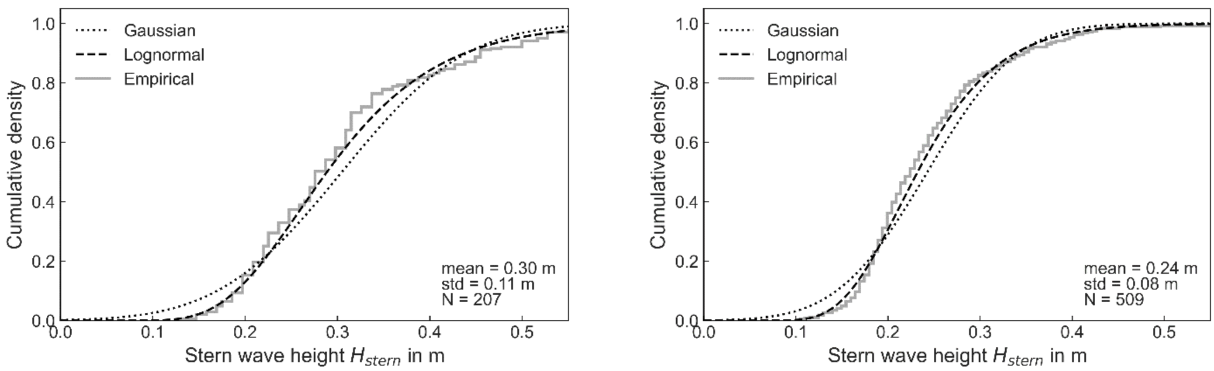

Figure 5: Gaussian and Lognormal distribution fitted to the measured stern wave height (empirical), MDK-1 (left) and WDK-1 (right). It can be observed that the Lognormal distribution fit the data best.

A sieving analysis with more than 1000 armor stones and two different armor stone classes indicated that the mean μ of D_{50} varies depending on the armor stone class and delivery batch, whereas the standard deviation σ is constant at 12 mm for larger armor stones ($D_{50} \geq 150$ mm) and 10 mm for smaller armor stones (Sorgatz, 2021). The armor stone density ρ_s is assumed to follow a Gaussian distribution with μ = 2650 kg/m³ independently of the investigated canal. The empirical factors B'_B and B^*_B are described by Bernoulli distributions that are positively correlated by the factor of 1.

The variables m and $\phi'_{D,hydr}$ are assumed the be deterministic variables. Firstly, this simplification is a result of sensitivity analyses presented in Sorgatz (2021) who found that these parameters have a minor effect on the probability of failure. Secondly, this simplification is a result of the validity range of eq. (2) and eq. (3); particularly for large slope inclinations as observed at DEK (North), a distribution would exceed the validity range of $2 \leq m \leq 5$ or result in a strongly truncated Gaussian distribution. A summary of distributions and parameters used throughout this paper is given in Table 8.

Table 8: Summary of random variables and their distributions.

Variable	H_{stern}	$D_{50,site}$	m	ρ_s	$\phi'_{D,hydr}$	B'_B and B^*_B
	m	mm	--	kg/m³	°	--
Distribution	Lognormal	Gaussian	Deterministic	Gaussian	Deterministic	Bernoulli
Mean μ	0.15 – 0.30	145 – 230	2.1, 3.0	2650	45	0.5
Standard deviation σ	0.07 – 0.11	10, 12	--	100	--	--

3 Results

3.1 Probabilities of failure from deterministic and probabilistic analyses

The results of the deterministic analyses (see Table 9) show that only in the case of a conservative design, $\eta > 1$ is reached frequently, whereas with the less strict design, $\eta \leq 1$ is found for only 6 out of 12 case studies. The results thus confirm the existing empirical parameter combinations (B'_B and B^*_B) that allow to account for maintenance requirements during design. Although the usage of B'_B and B^*_B may be less conducive when assessing the actual stability of the structure.

Table 9: Summary of the results of the deterministic assessment. Grey highlighted cells indicate values which are unlikely to represent the actual $p_{f,d}$.

Campaign	Less strict design	Conservative design				
	η	η	Vessel passages / year	$p_{f,d}$	$p_{v,d}$	$p_{a,d}$
General case studies						
DEK(N)-1	1.030	1.545	6596	4.74E-02	1.92E-04	7.18E-01
DEK(S)-1	0.662	0.993	10324	0.00E+00	0.00E+00	0.00E+00
MDK-1	0.678	1.016	5266	4.83E-03	2.40E-05	1.19E-01
MDK-2	0.648	0.972	5266	0.00E+00	0.00E+00	0.00E+00
WDK-1	0.797	1.196	14291	7.86E-03	1.55E-05	1.99E-01
WDK-2	0.870	1.305	14628	1.92E-02	3.72E-05	4.20E-01
WDK-3	0.915	1.373	8957	1.25E-02	3.96E-05	2.98E-01
DEK (North)						
Altenrheine	1.061	1.592	6596	6.72E-02	2.75E-04	8.37E-01
Gleesen	1.129	1.694	6596	9.88E-02	4.11E-04	9.34E-01
Hesselte	1.094	1.641	6596	8.70E-02	3.60E-04	9.07E-01
Rodde	1.030	1.545	6596	4.74E-02	1.92E-04	7.18E-01
Venhaus	1.207	1.811	6596	1.23E-01	5.17E-04	9.67E-01

Only if $\eta > 1$ is observed, the maintenance over one year can be extrapolated from the number of daily exceedances of η. Thus, only the conservative design is used in the following. For the results of these calculations ($p_{f,d}$, $p_{v,d}$, $p_{a,d}$ in Table 9) it must be noted that $p_{f,d}$ = 0.00 for DEK(S)-1 and MDK-2 are unlikely to represent the actual probability of failure. The deterministic analyses rely on the observed frequency of limit state exceedances, which, in turn, is affected by the total number of available observations. Thus, the $p_{f,d}$ = 0.00 values are most likely a result of the short observation period.

Table 10 presents the results of the probabilistic analyses. Since the probability estimate $p_{f,p}$ was determined by means Monte-Carlo simulations, the standard deviation σ of the probability estimate is given to evaluate the accuracy of the results. Considering the propagation of σ through eq. (12) and eq. (13), $p_{a,p}$ deviates by a maximum of +/−5 % as a consequence of the variability of the initial probability estimate $p_{f,p}$. To put this into numbers using the example of DEK(N)-1, if $p_{f,a}$ = 3.44E-02 with σ = 1.66E-03, then $p_{a,p}$ deviates by +/− 1.80E-02.

Overall results (Table 9 and Table 10) it can be noted that high η or p_f does not automatically result in high p_a. The p_a values are a result of the revetment stability and the traffic volume at the canal. For instance, considering the deterministic case studies, while $p_{f,d}$ is smaller for WDK-1 than for MDK-1, $p_{a,d}$ is larger for WDK-1 than for MDK-1 due to the heavier traffic at WDK-1. Moreover, for the six case studies at DEK (North) higher η, p_f and p_a are observed with the deterministic and probabilistic analyses. This may be due to the large slope inclination at DEK (North) confirming experiences on inland waterways that attribute a higher damage potential to steep slopes, e. g., Rock Manual (2007).

Table 10: Summary of the results of the probabilistic assessment.

Campaign	Number of vessel passages per year	$p_{f,p}$	σ of $p_{f,p}$	$p_{v,p}$	$p_{a,p}$
General case studies					
DEK(N)-1	6596	3.44E-02	1.66E-03	1.38E-04	5.99E-01
DEK(S)-1	10324	1.42E-04	7.10E-06	7.17E-07	7.38E-03
MDK-1	5397	8.17E-04	4.08E-05	3.95E-06	2.11E-02
MDK-2	5266	2.17E-04	1.08E-05	1.07E-06	5.64E-03
WDK-1	14291	2.93E-03	1.46E-04	5.76E-06	7.90E-02
WDK-2	14628	3.56E-03	1.77E-04	6.84E-06	9.52E-02
WDK-3	8957	9.98E-04	4.98E-05	3.13E-06	2.76E-02
DEK (North)					
Altenrheine	6596	3.67E-02	1.80E-03	1.48E-04	6.23E-01
Gleesen	6596	5.03E-02	2.44E-03	2.04E-04	7.39E-01
Hesselte	6596	4.56E-02	2.20E-03	1.84E-04	7.03E-01
Rodde	6596	3.25E-02	1.56E-03	1.30E-04	5.77E-01
Venhaus	6596	6.50E-02	3.18E-03	2.66E-04	8.27E-01

3.2 Regression analyses with deterministic and probabilistic analyses

Various regression analyses (linear, exponential and logarithmic) were conducted (see Table 11) to examine the correlation between maintenance measures and probabilities of failure to identify a suitable mathematical model. Each equation is supposed to predicted the required maintenance measures A in kg/m² as a function of the different probabilities of failure. To account for different assumptions during the data assessment, two different data combinations are investigated. Studies that are abbreviated with "-all" consider all available data points (general case studies + DEK (North)). To evaluate the effect of the extrapolation of the wave heights, regression analyses with a reduced data set were conducted ("-general"). For this purpose, only the general case studies are considered.

For the hypothesis tests, a significance level α = 0.05 is assumed. In the context of Table 11, this means that the null-hypothesis (model does not explain the data) is rejected if $p < 0.05$. In simple terms, if $p < 0.05$, we can assume that the investigated relation is significant. Comparing the p and R^2 values of functions that indicate an increasing relation between maintenance and failure probability (linear, exponential) with the logarithmic function, then, the majority of case studies indicates that an increase in maintenance is associated with a larger failure probability. Furthermore, for the majority of the linear and exponential regressions, an increase of R^2 is observed for the reduced data (-general) compared to the regression analyses with all data. The best fit can be achieved for the exponential function with the reduced data, but also with all data. Yet, for these regressions, it must be noted that the maintenance measures at WDK were omitted, since the exponential regression cannot handle values ≤ 0. As a consequence, EXP-general features only four data pairs. Despite this fact, site-specific information seems beneficial for a more accurate prediction of the required maintenance measures.

In view of all regression analyses, it can be noted that the regression analyses with the probabilistically determined failure probabilities show a better agreement with the elicited maintenance measures than the deterministically calculated values. The deterministic approach shows an improvement in the correlation between revetment stability and maintenance (smaller p-value, larger R^2) when using the annual frequency of limit state exceedances ($p_{a,d}$) rather than η. For both the deterministically and the probabilistically calculated failure probabilities, it can be noted that the regression analyses which use pa as regressor show a better fit than when p_f is used.

Table 11: Summary of p-values, coefficients of determination (R^2) and the regression parameters. The following abbreviations and equations are used: LIN – A [kg/m²] $= a \cdot p_{f,a} + b$, LN – A [kg/m²] $= a \cdot \ln(p_{f,a}) + b$, EXP – A [kg/m²] $= e^{a \cdot p_{f,a} + b}$. The grey shaded cells indicate that the investigated relation is not significant (for significance level α = 0.05).

Model	Data: η	$p_{f,d}$	$p_{a,d}$	$p_{f,p}$	$p_{a,p}$
LIN-all	p = 0.0209, R^2 = 0.4050, a = 9.42, b= −9.80 [a)]	p = 0.0010, R^2 = 0.4892, a = 68.76, b = 0.25 [a)]	p = 0.0090, R^2 = 0.5036, a = 7.89, b = −0.74 [a)]	p = 0.0080, R^2 = 0.5118, a = 126.48, b = 0.41 [a)]	p = 0.0047, R^2 = 0.5644, a = 9.31, b = 0.02 [a)]
LIN-general	p = 0.1732, R^2 = 0.2023, a = 5.36 [a)], b= –5.36 [a)]	p = 0.01629, R^2 = 0.6599, a = 103.52, b = −0.28	p = 0.0710, R^2 = 0.4134, a = 5.66 [a)], b= –0.34 [a)]	p = 0.0003, R^2 = 0.9271, a = 157.05, b = 0.11 [a)]	p = 0.0008, R^2 = 0.8925, a = 9.04, b = −0.01 [a)]
LN-all [b)]	p = 0.0263, R^2 = 0.3769, a = 12.14, b= −0.42 [a)]	p = 0.0293, R^2 = 0.4466, a = 2.67, b = 12.96	p = 0.0439, R^2 = 0.3856, a = 3.92, b = 6.59	p = 0.0159, R^2 = 0.4377, a = 1.27, b = 9.91	p = 0.0180, R^2 = 0.4233, a = 1.49, b = 6.45
LN-general [b)]	p = 0.2203, R^2 = 0.1381, a = 5.97 [a)], b = 0.07 [a)]	p = 0.1493, R^2 = 0.4050, a = 2.09 [a)], b= 10.27 [a)]	p = 0.2305, R^2 = 0.2382, a = 2.33 [a)], b = 4.14 [a)]	p = 0.0945, R^2 = 0.3507, a = 0.74 [a)], b= –0.29 [a)]	p = 0.1080, R^2 = 0.3198, a = 0.82 [a)], b = 3.78 [a)]
EXP-all	p = 0.0014, R^2 = 0.8132, a = 3.19, b = −3.56	p = 0.0085 [c)], R^2 = 0.6628, a = 21.41, b= −0.22 [a)]	p = 0.0006 [c)], R^2 = 0.8605, a = 2.62, b= −0.54 [a)]	p = 0.0048 [c)], R^2 = 0.7190, a = 41.90, b= −0.29 [a)]	p= 0.0009 [c)], R^2 = 0.8388, a = 3.15, b= −0.46 [a)]
EXP-general	p = 0.0131, R^2 = 0.9609, a = 3.97, b = −4.41	p = 0.0087 [c)], R^2 = 0.9740, a = 47.87, b = −0.55	p = 0.0123 [c)] R^2 = 0.9633, a = 3.19, b = −0.59	p = 0.0101 [c)], R^2 = 0.9700, a = 64.77, b = −0.50	p = 0.0103 [c)], R^2 = 0.9693, a = 3.75, b = −0.52

[a)] Parameter is not significant (for a significance level α = 0.05).

[b)] Analyses do not include MDK-2 and DEK(S)-1, since $p_{f,d} = 0.00$ or $p_{a,d}$ cannot be considered in a logarithmic regression.

[c)] Analyses do not include WDK-1, WDK-2 and WDK-3, since zero maintenance cannot be considered in an exponential regression.

The regression analyses are also subject to uncertainty. Exemplary, two suitable models, LIN-all and EXP-all, are shown in Figure 6. Once more, the figures suggest that the EXP-all model provides a more suitable fit to the data than the linear fit indicated by the different width of the confidence bands. In the area of most interest, moderate failure probabilities, the linear regression estimates a greater amount of replaced armor stones than the exponential model. For lower and higher probabilities of failure, the exponential model forecasts a larger amount of armor stones to replace. Comparable observations are made for the use of η, $p_{f,d}$ and $p_{a,d}$. However, greater uncertainty in the form of larger confidence bands are found with these regression models.

A major drawback of the linear model in combination with η or the "deterministic" failure probabilities is the negative intercept with the y-axis allowing for negative maintenance in the case of small failure probabilities. However, if we look at the confidence intervals in Figure 6, we can see that $b > 0$ is still in the range of likely outcomes. Moreover, the p-values indicating the significance of the individual model parameters, which are, for the sake of clarity, not shown in Table 11, but indicated by a small footnote, suggest that the intercept is not significant. This observation applies to all of the in Table 11 summarized linear regressions, even if this effect is less pronounced in few cases. Therefore, for further considerations with the linear regressions, one can assume b = 0. Furthermore, this assumption also applies to some of the logarithmic and exponential regressions, which is why in these cases one can assume b = 0, too. It is particularly noteworthy that for the exponential regressions with the reduced data set, both parameters (a, b) are significant.

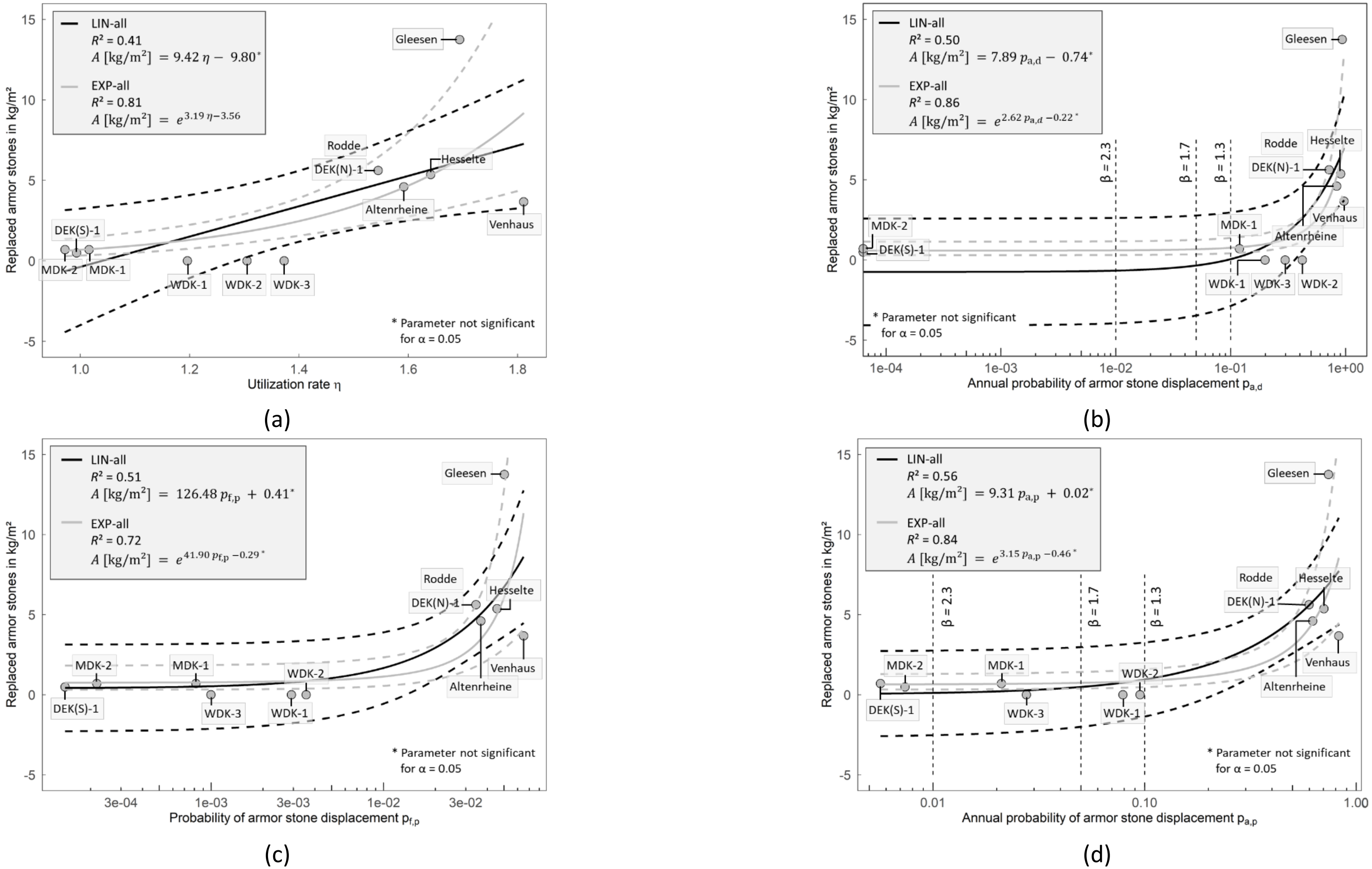

Figure 6: Correlation between calculated failure probabilities and elicited maintenance in terms of replaced armor stones in kg/m². Displayed are the regression equations (solid lines) and 95 % confidence intervals (dashed lines) of the two most suitable regression models (LIN-all, EXP-WDK) with (a) the utilization rate and (b) the annual probability of failure determined with the deterministic approach and the failure probability (c) and the annual failure probability (d) determined with Monte-Carlo simulations as regressors.

3.3 Comparison of deterministic and probabilistic analyses

To compare the regression equations determined with the “deterministic” and “probabilistic” failure probabilities in more detail, the linear and exponential equations that are based on the complete data are employed in a small parameter study. For this purpose, the required maintenance measures predicted by the different regression equations are compared for a total of 15 different traffic volume and p_v combinations.

The required maintenance that is predicted by the linear equation based on the deterministic analyses is smaller than the required maintenance predicted by the linear equation based on the probabilistic analyses (see Table 12). The differences between the modes LIN-DET and LIN-PROB range from 0.00 kg/m² for small p_v values to 1.40 kg/m² for large p_v values. As a result of the same intercept $b = 0$ in the LIN-DET and LIN-PROB model, the percentual difference between the two approaches stays constant with an increasing number of vessels or an increasing p_v.

The required maintenance that is predicted by the exponential equation based on the deterministic analyses is also smaller than the required maintenance predicted by the exponential equation based on the probabilistic analyses. The differences between the modes LIN-EXP and LIN-PROB range from 0.00 kg/m² for small p_v values to 9.60 kg/m² for large p_v values. In contrast to the linear models, the percentual difference between the two approaches increases with an increasing p_v and an increasing number of vessels. These results confirm a drawback of the deterministic calculation.

Table 12: Results of parameter study for a comparison of the linear and exponential equations found with the deterministically and the probabilistically determined failure probabilities. Regression equations that were determined with the “deterministic” failure probabilities are denoted by “-DET” and $p_{a,d}$, whereas “-PROB” and the $p_{a,p}$ in the equation indicates that the equation is derived with probabilities of failure obtained from Monte-Carlo simulations.

Case studies			Linear regression			Exponential regression		
Vessel passages per year	p_v	p_a	LIN-DET $A = 7.89\, p_{a,d}$	LIN-PROB $A = 9.31\, p_{a,p}$	Diff	EXP-DET $A = e^{2.62 p_{a,d}}$	EXP-PROB $A = e^{3.15 p_{a,p}}$	Diff
	--	--	kg/m²	kg/m²	%	kg/m²	kg/m²	%
1000	1.00E-06	1.00E-03	0.01	0.01	15.3	1.00	1.00	0.1
5000	1.00E-06	4.99E-03	0.04	0.05	15.3	1.01	1.02	0.3
10000	1.00E-06	9.95E-03	0.08	0.09	15.3	1.03	1.03	0.5
15000	1.00E-06	1.49E-02	0.12	0.14	15.3	1.04	1.05	0.8
20000	1.00E-06	1.98E-02	0.16	0.18	15.3	1.05	1.06	1.0
1000	1.00E-04	9.52E-02	0.75	0.89	15.3	1.28	1.35	4.9
5000	1.00E-04	3.93E-01	3.10	3.66	15.3	2.80	3.45	18.8
10000	1.00E-04	6.32E-01	4.99	5.89	15.3	5.24	7.32	28.5
15000	1.00E-04	7.77E-01	6.13	7.23	15.3	7.66	11.56	33.8
20000	1.00E-04	8.65E-01	6.82	8.05	15.3	9.64	15.24	36.8
1000	1.00E-02	1.00E+00	7.89	9.31	15.3	13.73	23.33	41.1
5000	1.00E-02	1.00E+00	7.89	9.31	15.3	13.74	23.34	41.1
10000	1.00E-02	1.00E+00	7.89	9.31	15.3	13.74	23.34	41.1
15000	1.00E-02	1.00E+00	7.89	9.31	15.3	13.74	23.34	41.1
20000	1.00E-02	1.00E+00	7.89	9.31	15.3	13.74	23.34	41.1

4 Discussion

Firstly, it is must be noted that maintenance itself may not be an objective measure. The conducted maintenance is always a response to predominately visual and, thus, subjective inspections of the structures by field engineers. In few cases, the replaced material may not meet the quality criteria or the installation of the material was unsatisfactory. Also, for MDK and WDK the elicited maintenance covers only six years. It is expected that maintenance measures can be low, but over a longer period of time it is assumed that they are non-zero. Particularly with the limited available data, these uncertainties may have an impact on the correlation between deterministically or probabilistically determined revetment stability and maintenance. In addition, the presented analyses are limited to a correlation between maintenance and stability assessment along German inland waterways. However, since long-term data on maintenance is difficult to collect and rarely published, the authors believe that, despite all limitations, the publication of the data is useful for the scientific community.

For the stability assessment, it is assumed that the stern waves characterize the load situation at the canal well. Yet, ship-induced currents, in particular the slope supply flow, can also cause armor stone displacements. Thus, the presented analyses may yield an optimistic estimate of the actual utilization degree or probabilities of failure. Due to the moderate blockage ratios, it is not expected that an assessment with ship-induced currents will change the stability assessment significantly. The available data is therefore suitable for first investigations of the correlation between stability assessment and maintenance.

Moreover, the presented analyses are limited to a correlation between maintenance and stability assessment assuming that a complete loss of slope stability does not occur. In view of the existing data, the existing design equations and from the perspective of the current approach to design and maintenance, this assumption seems reasonable.

When considering annual probabilities of failure, representative load assumptions are required for a period of one year. In the presented investigations, the annual probability of armor stone displacement is extrapolated spatially and temporarily. This implies that the site-specific probability density functions of the wave heights represent the annual traffic over an entire section. However, the distributions are statistical approximations of the wave events observed in real life. The Monte Carlo simulations yield an error of +/- 5 % in relation to the determined probabilities of failure. Since for revetments these uncertainties do not translate into failure, but into a change in maintenance, for now, these uncertainties are acceptable. The accuracy of the predictions can be increased by supplementing the database.

Furthermore, based on probabilistic calculations by Sorgatz (2021), at least 250 measured wave events are required for a reliability-based hydraulic revetment design. In the case of the deterministic analyses, the number of required measurements is higher (Sorgatz, 2021) which is supported by the fact that the deterministic analysis yields $p_{f,d} = 0$ for two case studies with less than 250 observations (DEK(S)-1 and MDK-2). For 9 out of 12 case studies the requirement of more than 250 measurements is fulfilled (see Table 2). For DEK(S)-1, MDK-1 and MDK-2 only 200 observations are available. Despite this fact, we decided to use all data for the analyses since the required number of measurements also depends on the variability of the sample. The results of Sorgatz (2021) are a robust estimate for cov = 3.0. For lower variabilities as considered in the present investigations (cov < 0.5), it seems acceptable to work with a smaller number of samples.

Revetments are simple structures. However, since they cover large areas at canals or rivers, the relative cost of safety measures is comparatively high. If larger armor stones have to be installed, the costs rise quickly as a result of the corresponding increase in required armor layer thickness. The risk associated with damage, on the other hand, is low under the constraint that inspections are conducted on a regular basis. If one applies the risk classification according to JCSS (2001), see Table 7, the structures should at least satisfy a reliability index $\beta > 1.3$ ($p_a < 0.1$).

The annual probabilities of armor stone displacements obtained with the deterministic approach (see Table 9) exceed this target reliability. On the one hand, this can be a result of generally low reliability standards at inland waterways which, in (conservative) deterministic analyses, are included by the B_B'* values and partial factors equal 1. On the other hand, the results emphasize that the deterministic approach may not cope well with the limited number of observations.

In contrast, the probabilistic approach seems to be characterized by a greater robustness regarding the underestimation of actions. The probability functions account for the possibility that values occur that exceed in nature observed loads. In the case that only maintenance efforts are of relevance for design considerations, an annual target reliability of $p_a = 0.1$ ($\beta = 1.3$) may be derived from the presented probabilistic analyses. Figure 6 shows low maintenance measures for $p_a <$ 0.1. Only for $p_a > 0.5$ maintenance measures are observed on a larger scale. It is emphasized that the proposed target reliability is a first estimate which is affected by the uncertainties discussed in the previous paragraphs. Certainly, further data and analyses are required to specify a generally valid target reliability. This is particularly important as the analyses of DEK (North) show that maintenance costs can be quite high in the case of heavy traffic and/or large hydraulic loads.

Finally, it must be noted that there is a slightly different notion of the probability of failure in JCSS (2001) and in this paper. JCSS (2001) specifies annual probabilities of failure. Yet, in the case of canals, it is essential to consider the probability of failure in the context of the traffic loads and the traffic volume.

5 Conclusions

In this paper, elicited maintenance measures are linked to a stability assessment of bank revetments at German inland waterways. In total, 12 case studies are investigated for which data on maintenance for at least six years and measurements of ship-induced waves for at least seven days are available. A deterministic and a probabilistic approach to determine the annual probabilities of failure are presented. The annual probabilities of failure are then correlated to maintenance measures by means of a regression analysis to investigate the relation between revetment stability and maintenance.

The analyses show that at least a positive (linear, exponential) relationship between revetment stability and required maintenance must be assumed. The results emphasize that for a detailed analysis of different canal sections, site-specific data should be available for each section. In addition to the armor stone characteristics, this includes information on the slope inclination, wave heights and the number of annual ship passages. To achieve comparability between different locations, it may be helpful to introduce one common design standard that does not rely on empirical stability factors.

Especially when the number of observations is small, the probabilistic approach should be favored over the deterministic approach. Yet, an improvement in the correlation between revetment stability and maintenance measures is also noticeable when using the frequency of limit state exceedances with moderate sample sizes instead of the utilization rate. If only maintenance efforts are of relevance for design considerations, the results indicate that $\beta = 1.3$ ($p_f \approx 10^{-1}$) may be a suitable first estimate for a target reliability resulting in moderate maintenance. However, once more it is emphasized that further data and analyses are required to specify a generally valid target reliability.

Further research is required to assess the transferability to different infrastructure than German inland waterways. In addition, it is strongly recommended to work towards a standardization of field observations and maintenance documentation. The application of methods such as Monte Carlo simulations or Bayesian networks may allow to establish a “standard” canal-specific vessel fleet based on previous traffic observations replacing costly traffic observations. For a risk-driven decision making which aims for the optimal lifetime costs as the sum of investment, maintenance, monitoring and failure costs, it is vital to establish a mathematical relationship between traffic and damage development. A joint consideration of different modes of failure as well of the construction, the monitoring and the maintenance costs may eventually result in a comprehensive risk- driven decision-making scheme.

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Author contributions (CRediT)

JS: Formal Analysis, Methodology, Visualization, Writing – original draft. JK.: Data acquisition, Writing – review & editing, Supervision.

Notation

Name	Symbol	Unit
Significance level	α	
Reliability index	β	
Angle of slope inclination	δ	$\circ$
Utilization degree	η	
Mean value	μ	
Density of armor stones	ρ_s	kg/m^3
Density of water	ρ_w	kg/m^3
Standard deviation	σ	
Cumulative standard Gaussian distribution	Φ	
Effective angle of repose of the armor stones	$\phi'_{\mathrm{D,hydr}}$	$\circ$
Amount of required armor stones	A	kg/m^2
Cross-sectional area of the waterway	A_w	m^2
Cross-sectional area of the submerged part of a vessel	A_s	m^2
Regression slope	a	
Empirical factor considering the revetment stability	$B'_{\mathrm{B}}, B^*_{\mathrm{B}}$	
Regression constant	b	
Gaussian copula	C	
Empirical factor considering the slope inclination	C_{slope}	
Probability density function	f	
Mean armor stone diameter	D_{50}	mm
Required armor stone diameter	$D_{50,req}$	mm
Armor stone diameter in-situ	$D_{50,site}$	mm
Limit state function	g	
Stern wave height	H_{stern}	m
Indicator function	I	
Slope inclination	m	
Blockage ratio	n	
Number of observed vessels	n_{m}	
Total number of observations / simulations	n_{total}	
Number of vessels per year	n_{y}	
p-value for hypothesis tests	p	
Annual probability of failure or armor stone displacement	p_a	
Probability of failure or armor stone displacement	p_{f}	
Probability of failure or armor stone displacement per vessel	p_{v}	
Coefficient of determination	R^2	
Spearman correlation matrix	$\boldsymbol{R}$	
Vector of (random) variables	$\boldsymbol{X}$	

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Appendix

Table A.1: Elicited maintenance costs for the period from 2001 to 2014.

Canal	Section	from km	to km	Costs 2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	Sum
				€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/m²	€/(m²a)
DEK (North)	Bever-gern	108.4	109.3	0.66	0.00	0.26	0.03	0.05	0.00	0.00	0.04	1.74	0.24	0.00	0.00	0.00	1.39	0.32
	Rodde	109.3	112.5	0.41	0.00	0.23	0.00	0.06	0.70	0.00	0.16	0.37	0.37	0.00	0.00	0.00	0.29	0.19
	Alten-rheine	112.5	117.9	0.32	0.00	0.02	0.03	0.06	0.44	0.00	0.23	0.43	0.23	0.00	0.00	0.00	0.04	0.13
	Venhaus	117.9	126.6	0.25	0.12	0.08	0.01	0.07	0.00	0.05	0.02	0.27	0.15	0.00	0.00	0.00	0.14	0.08
	Hesselte	126.6	134.5	0.27	0.33	0.16	0.02	0.02	0.79	0.01	0.18	0.28	0.17	0.00	0.00	0.00	0.17	0.17
	Gleesen	134.5	137.8	0.25	0.22	0.37	0.08	0.24	1.15	0.45	0.51	0.95	0.35	0.00	0.00	0.00	1.13	0.41
	unterh. Gleesen	137.8	138.3	0.00	2.19	0.00	0.00	0.00	0.00	0.00	0.00	1.02	0.55	0.00	0.00	0.00	0.40	0.30
DEK (South)	Los 14	79.4	84.0	0.00	0.08	0.00	0.07	0.00	0.00	0.51	0.06	0.00	0.02	0.00	0.00	0.00	0.02	0.05
	Los 15	84.0	89.0	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.09	0.16	0.09	0.00	0.00	0.00	0.03	0.03
MDK	Strullen-dorf	21.8	25.4				0.00	0.19	0.09	0.00	0.00	0.00						0.05
	Forch-heim	26.4	32.0				0.01	0.01	0.00	0.00	0.00	0.00						0.00
WDK	I	35.5	37.9					0.00	0.00	0.00	0.00	0.00	0.00					0.00
	II	39.7	42.8					0.00	0.00	0.00	0.00	0.00	0.00					0.00