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Reliability-based sliding failure analysis of a concrete dam using level II methods: The Zatonie Dam

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Abstract: Hydraulic structures, with their ability to dam up and store water, perform crucial social, economic, and environmental functions. Their construction cost is usually significant, and the consequences of failure are catastrophic, which incorporates an inherent risk analysis component into their management. However, Poland's current regulatory design approach is essentially deterministic – it does not consider the variability of important design parameters, instead assuming arbitrary safety margins. In comparison, reliability-based methods incorporate random variables based on available statistical data, leading to an estimate of the probability of failure and a relatively straightforward transition to risk analysis. This paper exemplifies the application of the so-called reliability index (β) according to level II methods. The presented analysis concerns potential sliding failure based on the example of the Zatonie concrete dam in Poland, as assessed by both deterministic and probabilistic methods. The calculated safety factor n and the β index are 1.28 and 7.21, respectively, and the probability of failure is of the order of 10^{-13} per year. The results were discussed in light of various standard requirements and good practices, e.g., Dutch flood protection guidelines.

Keywords: concrete dam, dam safety, failure probability, reliability index, risk analysis

INTRODUCTION

According to the Regulation (Rozporządzenie, 2007), the method for verifying dams' stability in Poland is presently based on equilibrium equations and *de facto* deterministic quantities. As such, it is close to the traditional approach, e.g., according to Fanti *et al.* (1972). From this perspective, the measure of safety, considering the assumed failure mechanism, is the ratio of stabilising actions to destabilising ones (safety factor n). The Regulation implemented, at least in part, the limit state method with partial safety coefficients for loads and materials. The state of standardisation and regulation in Polish hydraulic engineering is described in more detail in Kledyński and Krysiak (2017).

The Regulation (Rozporządzenie, 2007) does not address using probabilistic methods for stability analysis, even if only as a parallel approach. In the Eurocode system, these reliability-based methods enable the calibration of partial safety factors, as described in the standard PN-EN 1990:2004 in Annexes B and C (Polski Komitet Normalizacyjny, 2004). According to ICOLD Bulletin (ICOLD, 2007), many countries had, at the time, regulations or guidelines for risk assessment in dam safety management. However, not Poland – and this is still the case today. Thus, standardisation and regulation in hydraulic engineering design require critical analysis and evaluation of potential updates, considering the current state of knowledge and the extensive needs for structural safety assessment.

The essential challenges facing the industry lie not only in the design and construction of dams but also (and perhaps predominantly) in the assessment and maintenance of existing structures. According to the Centre for Technical Inspection of Dams, in 2023, 21% of Class I and II hydraulic structure complexes in Poland were in an unsatisfactory technical condition, and 17% of them were in a safety-threatening condition (Radzicki *et al.*, 2024). Suppose one compares these data with earlier reports, e.g. Dmitruk *et al.* (2022). In that case, the picture is even more alarming, as it indicates a negative trend in the safety condition of the mentioned group of facilities.

The uniqueness of hydraulic structures lies in their individual character due to unique hydrological and geological conditions, different purposes, and structural arrangements. The long expected service life of dams, which are often critical infrastructure, as well as the significant damage in the event of a major failure or disaster, mean that these structures should be carefully considered in terms of safety and durability. To address these challenges, one can use probabilistic reliability assessment methods, including level II methods (Kledyński, 2024). These are based on the mean value and variance of random variables and employ the so-called reliability index β as a measure of reliability. In the cited work, this approach was used to analyse the stability of concrete dams as a function of their geometry. The authors of this article are not aware of other Polish works dealing with reliability methods in structural calculations in hydraulic engineering. However, several foreign studies are available, including a doctoral dissertation on the reliability analysis of the stability of concrete dams (Westberg, 2010).

This paper supplements the analyses described in the paper Kledyński (2024), based on the so-called Cornell β-index, with the determination of the Hasofer-Lind β-index and the generalised β-index (Madsen *et al.*, 2006). The presented calculations and a brief description of the methods provide a helpful illustration of the provisions of Annex C of Eurocode 0 (Polski Komitet Normalizacyjny, 2004). Real data from the Zatonie Dam in Poland was used (Hrabowski, 2012). The obtained β indices (different variants) are compared with the standard requirements. An example of the probabilistic approach according to the Dutch flood protection guidelines is mentioned, as well as a proposal for implementing probabilistic methods in Polish hydro-engineering.

STUDY MATERIALS AND METHODS

ZATONIE DAM CHARACTERISTICS

The Zatonie Dam, completed in 1966, is a concrete buttress dam, continuously storing water. The maximum damming height is 34.5 m, and the length is 306 m. More information on the structure, including the cross-sectional geometry of the dam's section no. 7 (the subject of the presented analysis – Figure 1) can be found in the monograph Hrabowski (2012).

The analysed failure mechanism is the sliding of the dam section along the foundation-subsoil interface, taking into account the inclination of the foundation base (with interlocking) at an angle θ from the horizontal (here: 9°). The load values used in the calculations are based on the source study (Hrabowski, 2012) from an 'as-built' (*a posteriori*) analysis (Tab. 1).

DETERMINISTIC STABILITY ANALYSIS

Stability verification according to § 34. 1. of the Regulation (Rozporządzenie, 2007) is, in essence, based on a traditional deterministic approach (described, for example, by the Equation

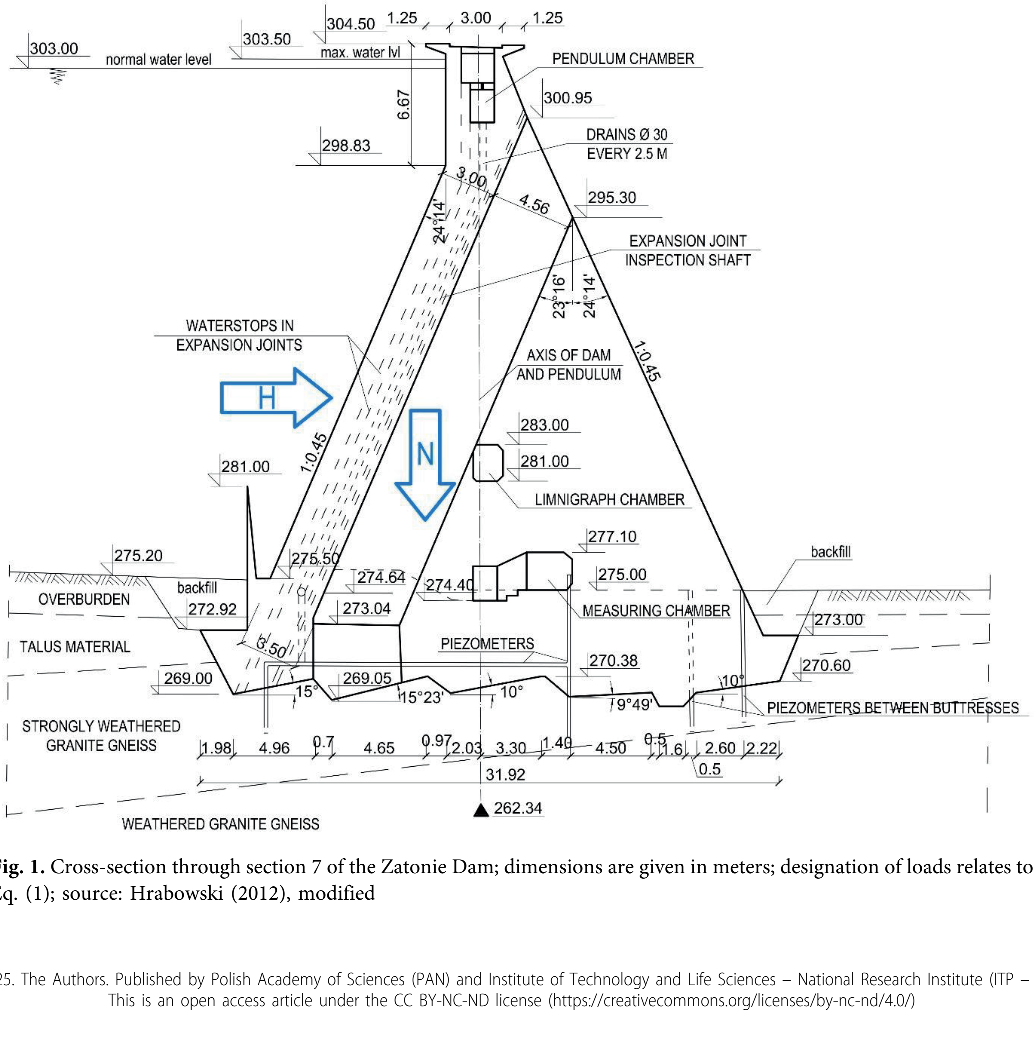

Fig. 1. Cross-section through section 7 of the Zatonie Dam; dimensions are given in meters; designation of loads relates to Eq. (1); source: Hrabowski (2012), modified

Table 1. Summary of variables in the Zatonie Dam section 7 stability analysis

Variable i in the stability analysis	Mean value μ_i	Standard deviation σ_i
Section self-weight G (kN)	101,830	0 (deterministic)
Uplift W (kN)	22,100	0 (deterministic)
Water overburden (vertical) G_w (kN)	32,700	0 (deterministic)
Headwater pressure (horizontal) P (kN)	69,400	0 (deterministic)
Ice pressure P_l (kN)	1,200	0 (deterministic)
Friction coefficient, foundation-soil interface f (–)	0.587	0.0208
Concrete unit weight γ_c (kN·m^{-3})	24.09	0.371[a)]

[a)] Example data – industrial concrete mixing plant, 1993 (Kledyński, 2024).
Note: The water overburden is due to the sloping of the upstream face.
Source: own study based on Hrabowski (2012).

(2-63) in Fanti *et al.* (1972, p. 124). The difference, however, is that it is now required to separate stabilising and destabilising actions (assumed in their design values) in the calculations. The required safety factor consists of two coefficients – γ_n and m, which can be combined into one n. The stability check requires the value of the deterministic n to be greater than one, with an appropriate margin (Eq. (1)):

$$n = \frac{E_{\text{stab}}}{E_{\text{dest}}} = \frac{(\sum N\cos\theta + \sum H\sin\theta)f + \sum N\sin\theta}{\sum H\cos\theta} \geq \frac{\gamma_n}{m} \tag{1}$$

where: E_{stab} and E_{destab} = total stabilising and destabilising actions (kN), ΣN = total vertical load, $\Sigma N = G + G_w - W$ (kN) (see Tab. 1), ΣH = total horizontal load, $\Sigma H = P + P_l$ (kN) (see Tab. 1), f = friction coefficient (foundation-soil interface) (–), θ = foundation base inclination (°), γ_n = coefficient of consequences (–), and m = corrective coefficient.

The issues of partial safety coefficients (needed to obtain design values) and the adhesion of concrete to rock were ignored in the analysis. The monograph Hrabowski (2012) describes more formulas used at the time to verify stability, but it is beyond the scope of this article.

PROBABILISTIC STABILITY ANALYSIS

In practice, virtually all parameters included in a structure failure model are random variables, as it is impossible to establish their values precisely (deterministically). Thus, in addition to deterministic methods, reliability theory includes so-called level II (probabilistic with limited information) and level III (fully probabilistic) methods (Madsen *et al.*, 2006). The primary roots of randomness in dam stability analysis stem from uncertainties in determining, *inter alia*, design flows (and therefore water levels), ice phenomena, soil characteristics, uplift, section geometry, concrete self-weight, and the concrete-soil friction coefficient. In this study, only two random variables are included (Tab. 1), which is conducive to the clarity of the analysis (especially in graphical form) but also results from the limited availability of data. The random variables here are assumed to be independent. Level II reliability methods, the focus of this paper, assume a lack of complete information about the distributions of the variables. Instead, they use only the random variables' mean values and variances (if applicable, covariances).

For reliability analysis, the selected failure mechanism should be described in terms of the so-called limit state function g, taking positive values for safe states and negative values for failure states. In this case, the obvious choice of such a function is described by Equation (2). To increase the clarity of calculations, the function was simplified by introducing constants A to E (Eq. (3)). The analysed function g of two random variables – γ_c and f (in bold) – is a second-degree polynomial in this case.

$$g = E_{stab} - E_{dest} \tag{2}$$

$$g(\gamma_c, f) = A\boldsymbol{f}\boldsymbol{\gamma_c} + \mathrm{B}\boldsymbol{f} + \mathrm{C}\boldsymbol{\gamma_c} + \mathrm{D} - \mathrm{E} \tag{3}$$

where: f = friction coefficient (–), γ_c = unit weight of concrete (kN·m^{-3}), A–E = deterministic values (constants): A = $V\cos\theta$ (m^3), B = $(G_w - W)\cos\theta + (P + P_l)\sin\theta$ (kN), C = $V\sin\theta$ (m^3), $D = (G_w - W)\sin\theta$ (kN), $E = (P + P_l)\cos\theta$ (kN), and V = volume of the section, $V = G/\mu_{\gamma c}$ (m^3).

The primary objective of reliability analyses is to estimate the probability of failure of the system P_f, i.e. the probability a given form of failure will occur in the relevant time interval (or the probability of survival, also called reliability: $P_s = 1 - P_f$). A frequently used alternative measure of reliability in level II methods is the so-called reliability index β, related to the probability of failure according to Equation (4) (Polski Komitet Normalizacyjny, 2004; Madsen *et al.*, 2006):

$$P_f = P(g < 0) = \Phi(-\beta) \tag{4}$$

where: $P(E)$ = probability of the event E, Φ = cumulative distribution function of the standard normal distribution, β = reliability index.

The exact calculation of P_f poses difficulties, even with complete knowledge of the distributions of random variables. Instead, in the 1960s and 1970s, a number of methods for approximating the β-index were developed. Several of these, in order of increasing generality, are recalled below, and applied to the case study of the Zatonie Dam.

The β-index according to Cornell (1969), referenced, among others, in PN-EN 1990:2004 (Polski Komitet Normalizacyjny, 2004) and Kledyński (2024), is defined as the ratio of mean to standard deviation, i.e., the inverse of the coefficient of variation of the margin of safety (here, for simplicity, equated to g) – Equation (5).

$$\beta_c = \frac{\mu_g}{\sigma_g} \tag{5}$$

where: μ_g = mean value of g, σ_g = standard deviation of g.

The approach described by Equation (5) is, in principle, applicable only when function g is linear (is a hyperplane) (Madsen *et al.*, 2006). The simplest way to estimate the reliability index for a non-linear function is to linearise it using a first-degree Taylor polynomial, usually around the mean values (around the mean point). Then, an approach similar to that described by Eq. (5) can be used. The β-index determined this way is called the mean-value first-order second-moment reliability index (Madsen *et al.*, 2006); its determination is not shown here.

A more generalised approach to the nonlinear failure surface problem is offered by the β-index as defined by Hasofer and Lind (1974). It is based on the transformation of the set of base variables to uncorrelated normalised variables. The failure surface is also transformed, and the β-index is equivalent to the distance from the mean value point to the nearest point on the failure surface $g = 0$ (the so-called design point). To apply transformation in the analysed case, it is sufficient to divide the values of the variables by the corresponding standard deviation, as represented, e.g., in Fig. C2 (Polski Komitet Normalizacyjny, 2004, p. 52). Next, the task reduces to finding the minimum distance between the mean value point and the failure surface, expressed as a multiple of the standard deviation.

The β_{HL} index, like the previously mentioned method, relies on the linearisation of the failure surface around a particular point, in this case, the design point (see Fig. 2). Thus, it provides a measure of the distance to the actual failure surface, which is however approximated by the tangent plane (line) shown in Figure 2. The algorithm for finding the minimum distance and the design point is described in Madsen *et al.* (2006), among others. It is an elementary task in the studied case with two random variables; the distance can also be determined graphically.

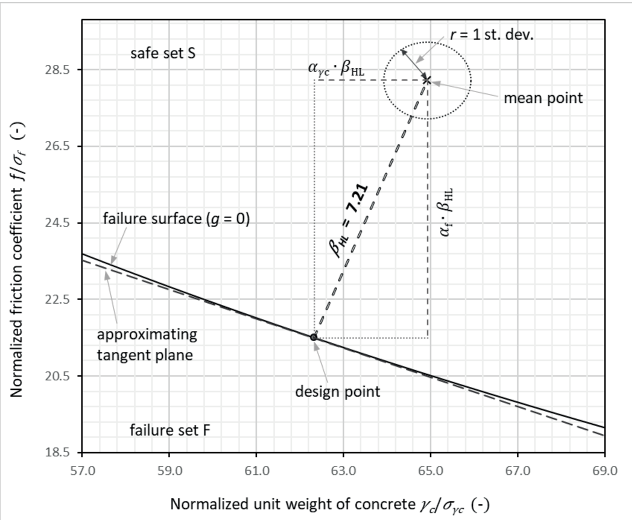

Fig. 2. Graphical determination of the reliability index β_{HL} as the distance from the mean point to the failure surface (in the system of normalised variables γ_c and f); the values of the sensitivity coefficients are: $\alpha_{\gamma c} = 0.36$, $\alpha_f = 0.93$; source: own study

Where the failure surface deviates significantly from a hyperplane, the β_{HL} index may provide a poor measure of reliability. In that case, the generalised reliability index β_G can be used, which preserves the actual shape of the failure surface (Ditlevsen, 1979). The method is based on transforming the variables into independent variables with zero mean and unit standard deviation and then assuming the joint probability density function of all variables ψ_n as the product of their probability densities. The ψ_n is then the n-dimensional standardised normal probability density function. This is purely a pragmatic, arbitrary assumption for computational purposes since the distributions of the variables are still unknown (Madsen *et al.*, 2006). The probability of failure can be estimated by integrating this joint function over the area F: $g < 0$ (the so-called failure set) – Eq. (6). The equivalent value of β_G is then determined from Eq. (4) (see also Eq. (9)).

$$P_f \approx \int_F \psi_n\,(x_1, x_2 \ldots x_n) dF = \int_F f_1(x_1)\, f_2(x_2) \ldots f_n(x_n) dF \tag{6}$$

where: $\psi_n(x_1, x_2, \ldots x_n)$ = joint probability density function of n variables, f_i = probability density function of the variable x_i (here – of normal distribution), F = failure set – the area covering all variable combinations for which $g < 0$.

Further improvement of estimation would require the specification of the distributions of random variables. The popular FORM (First Order Reliability Method) is based on transformation to the space of standardised normally distributed and uncorrelated variables, and then finding the design point and β in a manner analogous to that shown in Figure 2 (Madsen and Egeland, 1989; Rackwitz, 2001).

RESULTS AND DISCUSSION

RESULTS

To find the β_C-index, the numerator in Equation (5) is obtained by substituting mean values and not, for example, design values, as indicated in the Regulation (Rozporządzenie, 2007) and the Limit State Method more broadly. The denominator (standard deviation) in the studied case can be determined based on the mathematical properties of the variance (σ_g^2) of the sum and product of two random variables with known means and variances (e.g., Ang and Tang, 2006, pp. 180–182). The symbolic expression and the numerical result are shown by Equation (7):

$$\beta_c = \frac{g(\mu_{\gamma c}, \mu_f)}{\sqrt{\sigma_g^2}} = \frac{A\mu_f\mu_{\gamma c} + B\mu_f + C\mu_{\gamma c} + D - E}{\sqrt{A^2\sigma_f^2\sigma_{\gamma c}^2 + \left(A\mu_{\gamma c} + B\right)^2\sigma_f^2 + \left(A\mu_f + C\right)^2\sigma_{\gamma c}^2}} = 7.00 \tag{7}$$

where: $\mu_{\gamma c}$ = mean unit weight of concrete, $\sigma_{\gamma c}$ = standard deviation of the unit weight of concrete, μ_f = mean friction coefficient, σ_f = standard deviation of friction coefficient, other symbols as described in Equations (3) and (5).

The graphical representation of the β_{HL}-index is shown in Figure 2.

Calculations of the β_G-index for the Zatonie Dam are shown by Equations (8) and (9); no transformation was needed here for numerical integration. The limits of integration follow from the delimitation of the failure set, bounded by the curve $g = 0$ (see Fig. 2). In the case study, the indices are in the relation $\beta_G < \beta_{HL}$ (the difference being negligibly small) since the actual failure set is larger than that resulting from the linear approximation of the failure surface.

$$P_f \approx \int_{-\infty}^{\infty} \left(\int_{\frac{E-Bf-D}{Af+C}}^{\infty} f_{\mu_{\gamma c}, \sigma_{\gamma c}}(\gamma_c)\, f_{\mu_f,\ \sigma_f}(f)\, d\gamma_c \right) df = 2.8 \cdot 10^{-13} \quad (8)$$

where: $f_{\mu,\sigma}(x_i)$ = probability density function of normal distribution of the variable x_i, with mean value μ and standard deviation σ.

$$\beta_G = -\Phi^{-1}(P_f) = -\Phi^{-1}(2.82 \cdot 10^{-13}) = 7.21 \quad (9)$$

where: Φ^{-1} = the inverse cumulative distribution function of the standard normal distribution.

The results of all analyses are summarised in Table 2.

DISCUSSION

According to the deterministic approach, structural reliability is deemed satisfactory if the resistance exceeds the effect by a sufficiently high safety margin. If the values of model variables are based on selected quantiles and probabilistically calibrated partial safety coefficients, then such an approach is sometimes called semi-probabilistic (Jongejan and Calle, 2013). However, this occurs on a general basis, and the safety factor n does not permit quantification of reliability for a given structure.

The β indices in the presented analysis provide a practical, albeit arbitrary, measure useful for ranking different structures' reliability. As this approach relies on mean values and variances, it is crucial to estimate them with a reasonable level of confidence; this problem, however, is not touched upon in this paper. A more complete description of level II methods can be found in (Madsen *et al.*, 2006). To estimate the actual probability of failure in a given model, it is necessary to provide information on the probability distributions of all relevant random variables and apply the level III reliability methods. Acquiring adequate data is a major obstacle to the broader implementation of such an approach. The challenge of the statistical description of so-called rare events (i.e., the tails of distributions) is well exemplified by the issue of estimating maximum flows known from hydrology (Korbutiak *et al.*, 2023).

It follows from the definition that the higher the β-index, the lower the failure probability and, therefore, greater structural safety regarding the specific failure mode. Satisfying the requirements in the deterministic method (e.g., during design) means ensuring an appropriate value for the safety factor n ($n > n_{\min} \geq 1$). Similarly, the requirement of $\beta > \beta_{\min}$ is imposed in the probabilistic approach. The latter does not necessitate using any other values of model parameters besides the mean (such as characteristic or design values). This prevents the ambiguities present in the limit state method when certain variables influence both the resistance and the effect, which, for example, happens in Eq. (1) – the component $H\sin\theta$ is stabilising, while $H\cos\theta$ is destabilising. The β-index allows for estimating the probability of failure, adapting the structure to individual reliability-related requirements set at the design stage, as well as assessing the safety of existing structures. Thus, it provides a qualitative advantage over the deterministic safety factor.

Various requirements for $\beta_{\min}$ are provided depending on the referenced document. Eurocode 0 (Polski Komitet Normalizacyjny, 2004) links β to the reliability class RC (and therefore the consequences of failure), as well as the type of limit state (Tabs. B2 and C2 in Polski Komitet Normalizacyjny, 2004, p. 45 and p. 51); the most strict requirement being $\beta_{\min} = 5.2$. In addition to the consequences of failure, the Probabilistic Model Code (JCSS, 2001) includes a dependency on the relative cost of safety measures, essentially introducing an element of risk management (greatest $\beta_{\min} = 4.7$). Schneider (2006), on the other hand, recommends β values depending on the type of failure, stating the highest $\beta_{\min} = 6.0$ for extreme consequences and brittle, non-redundant failure mode. Given the ramifications of their destruction, the β-index required for large dams should obviously be among the highest.

Some standards specify reliability index values for assessing the condition of existing buildings, different than those for the newly designed ones. For example, the Dutch standard NEN8700 allows $\beta \geq 3.3$ for reconstruction of existing buildings (Scholten and Vrouwenvelder, 2013), lower than that applicable to newly designed buildings in the same RC2 class: $\beta \geq 3.8$ according to PN-EN 1990:2004 (Polski Komitet Normalizacyjny, 2004). The same Dutch standard also states a value of $\beta < 2.5$ as resulting in the rejection of a building (Scholten and Vrouwenvelder, 2013).

Probabilistic methods enable calibration of safety coefficients in the limit state method. The so-called sensitivity factors assigned to each of the variables in the model can be used for this. Based on the results $\alpha_{\gamma c} = 0.36$ and $\alpha_f = 0.93$ obtained in the analysed case (see Fig. 2), it is possible to determine the design values of each variable that would ensure reliability at the desired level (here $\beta \approx 7.2$) – Eqs. (10) and (11) (the formulas work for

Table 2. The safety factor and reliability indices regarding the sliding failure of section 7 of the Zatonie Dam

Method	Index value (–)	Equivalent probability of failure P_f (–)	Comments on the method
Deterministic n	1.28	–	unknown probability of failure P_f
Cornell β_C	7.00	$1.3 \cdot 10^{-12}$	only for linear limit state functions g
Hasofer-Lind β_{HL}	7.21	$2.7 \cdot 10^{-13}$	decently approximates g functions not deviating significantly from linearity
Generalised β_G	7.21	$2.8 \cdot 10^{-13}$	most adequate in the analysed case; fit for any kind of function g

Source: own study.

normal distribution). The design values are also the coordinates of the design point. The closer the value of $|\alpha|$ to unity, the larger the contribution of the uncertainty of a random variable to the overall probability of failure. In the analysed case, the key parameter is the coefficient of friction f, with an influence on P_f several times higher than the unit weight of concrete γ_c.

$$f_d = \mu_f - \alpha_f \beta_G \sigma_f = 0.447 \tag{10}$$

$$\gamma_{c,d} = \mu_{\gamma c} - \alpha_{\gamma c} \beta_G \sigma_{\gamma c} = 23.14 \tag{11}$$

where: f_d = design value of friction coefficient, $\gamma_{c,d}$ = design value of unit weight of concrete, α_f = sensitivity factor for f, $\alpha_{\gamma c}$ = sensitivity factor for γ_c, other symbols described in Eq. (7) and (9).

An exemplary application of reliability methods in hydrology can be found in the Dutch flood protection guidelines, which have been in effect since 2017 (Kok *et al*., 2016). The earlier paradigm limited the probabilistic analysis to hydraulic loading (overflow). The new, expanded approach focuses on the maximum permissible probability of flooding, including structural safety and hydraulics (piping, slope stability, erosion, and mechanism failure, among others). This probability, along with a more detailed assessment of the potential consequences of embankment failure, allows for stricter risk management. A directive goal of the Dutch government was to ensure that all areas at risk have a probability of loss of life of no more than 1/100,000 per year (Slomp, 2016). The total probability of flooding is divided according to general recommendations (Kok *et al*., 2016) into individual failure modes (so-called probability budget), which makes it possible to specify the required reliability index $\beta_{\min}$ for each mode separately.

The guidelines (Kok *et al*., 2016) allow the use of the limit state method (referred to as semi-probabilistic) based on the proper calibration of partial coefficients. For its purposes, a large-scale VNK-2 database was necessary (Jongejan *et al*., 2013), containing the results of reliability analyses (including α coefficients) of a significant number of test embankment cross-sections per various failure modes. The calibration process is described in (Jongejan and Calle, 2013), among others.

In comparison, the aforementioned Polish Regulation (Rozporządzenie, 2007) does not take probabilistic reliability methods into account at any point; the situation will not change with its pending amendment (Projekt Rozporządzenia, 2025). In addition, there are other unresolved ambiguities, such as regarding the safety coefficients and combinations to be used in structural calculations (Kledynski and Krysiak, 2017).

CONCLUSIONS

The article presents level II reliability calculations on the example of the sliding stability of the Zatonie concrete dam. The purposes that the reliability-based methods can serve in the broader context of ensuring the safety of structures and managing flood risk are briefly discussed.

Based on the available (and limited) data characterising section 7 of the Zatonie Dam, the reliability index regarding failure by sliding was estimated at β = 7.21, corresponding to a probability of failure of about $3{\cdot}10^{-13}$. Such an order of magnitude roughly corresponds to the requirements set in developed countries for critical structures such as large dams. The simplified analysis presented here cannot be regarded as an actual safety assessment of the Zatonie Dam. It should be treated as a practical example illustrating the principles of the discussed methods.

Probabilistic reliability methods offer significant advantages over deterministic methods, including:

- individual analysis of the reliability of a structure (or even of each failure mode separately), which is particularly important for hydraulic structures;
- avoiding the ambiguity of setting representative values, design values, etc.;
- calibration of coefficients (e.g., γ and ψ in Eurocodes);
- easy application of reliability measures in further risk analyses.

Based on available good practices (the example of the Dutch guidelines), one can propose that long-term planning for developing modern design methods for hydraulic structures should be considered in Poland, taking into account the theory of reliability. A useful starting point would be collecting data and experience as part of the periodic evaluation of the condition of existing structures and the design of new ones, including parallel calculations according to probabilistic methods. In the short term, such a practice would allow a tighter, quantitative assessment of the safety (and risk) status of existing hydraulic structures. In the long term, it would enable the calibration of safety coefficients, modernisation of design methods and raising the level of reliability of structures, in line with the country's development. At the same time, it should be emphasised that large sets of measured data are needed to characterise probability distributions.

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CONFLICT OF INTERESTS

All authors declare that they have no conflict of interests.

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