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# Stability analysis of the Racibórz Dolny Dam during the water level changes in the reservoir

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**Abstract:** The design of embankment dams or levee should consider the analysis of slope stability under conditions of rapid water level changes in the reservoir. The pore water pressure and the saturation state of the soil with water changes within the earth hydrotechnical structures such as dry reservoir dams occur quite often and can result in instability of the dam slope, and the abutment.

Thus, in order to minimise the risk of such a situation in Racibórz Dolny Dam flood control reservoir, the stability calculations using various methods assuming different loading scenarios were carried out. The parameters for the calculations were determined on the basis of geotechnical investigations conducted during the design and construction phase of the dam, and by performing supplementary investigations for the sections selected for the calculations.

The results of the calculations indicated that stability will be maintained in each of the calculation cases, and the minimum value of the factor of safety obtained will be 1.63, while the minimum factor of safety to be achieved is $F > 1.5$. This means that the slope of the Racibórz Dolny Dam flood control reservoir is safe and there is no risk of instability when the analysed cases occur.

**Keywords:** critical slip surface, factor of safety, flood protection reservoir, geotechnical investigations, head dam, slope stability

## INTRODUCTION

Dry flood control reservoirs are structures characterised by the free flow of the river through the reservoir basin and discharge facilities. When a flow is greater than the capacity of the water-permeable devices, the excess is stored in the reservoir area. Once the surge has passed, the reservoir is emptied. An interesting fact about dry reservoirs is that they can be managed agriculturally between surges. This article focuses on the analysis of the slope stability of the Racibórz Dolny reservoir – the largest dry flood control reservoir in Poland, located on the Oder River, in the Silesian Province. The structure was built in 2013–2020 as a reaction to the catastrophic flood of 1997 and is now a key component of the regional flood protection system (Kwinta, 2020).

The reservoir has an area of 26.3 km$^2$ and a capacity of about 185 mln m$^3$ of water (Budimex S.A., no date). It is equipped with an advanced overflow-drainage structure with six main gates that allow controlled damming of water during the Oder River floods. Figure 1 shows the location of the reservoir with the area covered by the detailed analysis highlighted.

In Poland, the problems of flood protection are one of the key challenges of water engineering (Rozporządzenie, 2007). This has become particularly important, in the context of the two major floods that have occurred in the past 30 years – the most recent in 2024. For many years, periodic inspections of the technical condition of hydro-engineering structures have been required to identify and eliminate any deficiencies that could affect the safety of these structures. In 2020, the Polish Institute of Meteorology and Water Management (Pol.: Instytut Meteo-

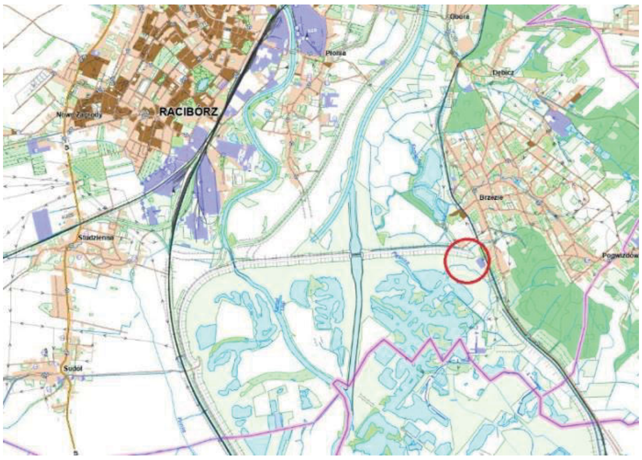


**Fig. 1.** Location of the analysed area on a section of the topographic map of Poland (GUGiK, no date)

rologii i Gospodarki Wodnej – PIB, IMGW-PIB) published guidelines for the technical assessment and safety evaluation of water dam structures. Their mission includes ensuring safe and efficient management of water resources, and this includes assessing the condition of dams, where one of the key points is stability analysis. Some factors affect the slope stability for the earth dam. It is well known that the values of factor of safety increase when the values of soil strength parameters (angle of internal friction and cohesion) increase and while the value of unit weight of the soil decreases. The values of factor of safety, decreasing fast in rapid draw down the water level (Utepov *et al.*, 2022). The present study uses the limit equilibrium methods to calculate the factor of safety of selected cross sections located on the right abutment of the dam. The main objective is to study the effect of water level changes in the reservoir on the factor of safety and how the applied ground buttress improves the stability conditions of the dam in the vicinity of the abutment. Reliable performance of such analyses requires prior, accurate investigation of the soil and water conditions of the subsoil. For this purpose, geotechnical borings, cone penetration test (CPT) soundings, laboratory tests and a number of other analyses are performed (DeGroot, 2014; Kulhawy and Mayne, 1990), based on which it is possible to derive the geotechnical parameters necessary for stability analysis under varying loading conditions (PN-EN 1997-1:2008; PN-EN 1997-2:2009).

## MATERIALS AND METHODS

### THE SCOPE AND METHODOLOGY OF SUBSOIL INVESTIGATION

For the analysed area of the natural slope of the Racibórz Dolny flood protection reservoir, 9 CPT soundings with a depth of 7–15 m and 6 boreholes with a depth in the range of 7–15 m were carried out. Below is a fragment of the map with marked test points and the designation of cross-sections through the slope (Fig. 2).

The boreholes were drilled with a mechanical system using a rotary drilling rig in accordance with the Polish standard (PN-B-04452:2002). This method involves the auger plunging into the ground while cutting or crushing the soil. At specified depth intervals, the auger and drill are withdrawn so that a qualified geologist can examine the soil to identify its type and characteristics such as colour, moisture content, and consistency in the case of cohesive soils, and collect samples for potential laboratory testing.

Soundings for the analysed site were performed using a Begemann-type mechanical cone in accordance with the European standard (ISO 22476-1:2013). Such a test is called


**Fig. 2.** Location of test points; source: Geoteko Ltd. (2019)

mechanical cone penetration test (CPTM), and the measurement is conducted at an interval equal to 20 cm of cone penetration. Subsequently, the test operators record the measured values as $q_c$ (the resistance at the tip of the cone) and $f_s$ (the friction along the sides of the cone). The results of these geotechnical tests are later presented in the article in the form of geotechnical cross sections.

## GEOTECHNICAL CONDITIONS

The investigations carried out enabled us to distinguish the following geotechnical layers within the analysed slope of the flood control reservoir: layer I – composed mainly of organic soils with local inserts of peats and silty clays. These soils are in a plastic state ($I_L$ = 0.4–0.5). Layer II – consisting of fine-grained cohesive soils, i.e. silty clays, clayey sands and dust. The layer is divided into two sub-layers according to the degree of plasticity. Layer IIb – soils in plastic state ($I_L$ = 0.25–0.35). Layer IIc – soils in the firm state ($I_L$ = 0.10–0.25). Layer III – represented by noncohesive soils, i.e. quaternary sands, silts, gravels. The layer was divided into two sub-layers due to the grain size and the degree of compaction $I_D$. Layer IIIa – predominantly fine and medium sands in a medium compacted state ($I_D$ = 0.33–0.4). Layer IIIc – predominantly sands with gravel, gravels and silt in compacted state ($I_D$ > 0.67). Layer IVc – composed of cohesive soils in a hard-plastic state ($I_L$ = 0.10–0.25) represented by silty clays, sandy clays and compacted clays. Layer V – forming the deeper subsoil, consisting of Neogene (Tertiary) soils developed as silty clay in a firm and semi-firm state ($I_L$ < 0.10). Layer nB – construction embankment consisting of silt and coarse sands in a compacted state ($I_D$ > 0.70).

## SUBSOIL MODEL AND GEOTECHNICAL PARAMETERS

Geotechnical cross sections II–II and III–III (Figs. 3, 4) were elaborated on the basis of archival studies as well as additional geotechnical investigations (Geoteko, 2005 and 2019). Stability calculations were carried out for these cross-sections assuming certain loading situations (changes of the water table in the reservoir). These cross sections were chosen due to their proximity to the reservoir's technical infrastructure facilities and the access road running along the embankment. In addition, local landslides were observed at the locations of these cross sections. Stability analysis was carried out in both cross sections II–II and III–III, for the following schemes and cases, which were considered critical: natural slope and slope with ground buttress in the conditions of the reservoir before filling and changes in the water level in the reservoir.

Geotechnical parameters were used for stability calculations, which were determined according to procedure given in PN-EN 1997-2:2009. Derived values of geotechnical parameters were obtained through the interpretation of CPT soundings and the analysis of archival laboratory tests (Abu-Farsakh *et al.*, 2008; Wierzbicki and Młynarek, 2015; Młynarek, Wierzbicki and

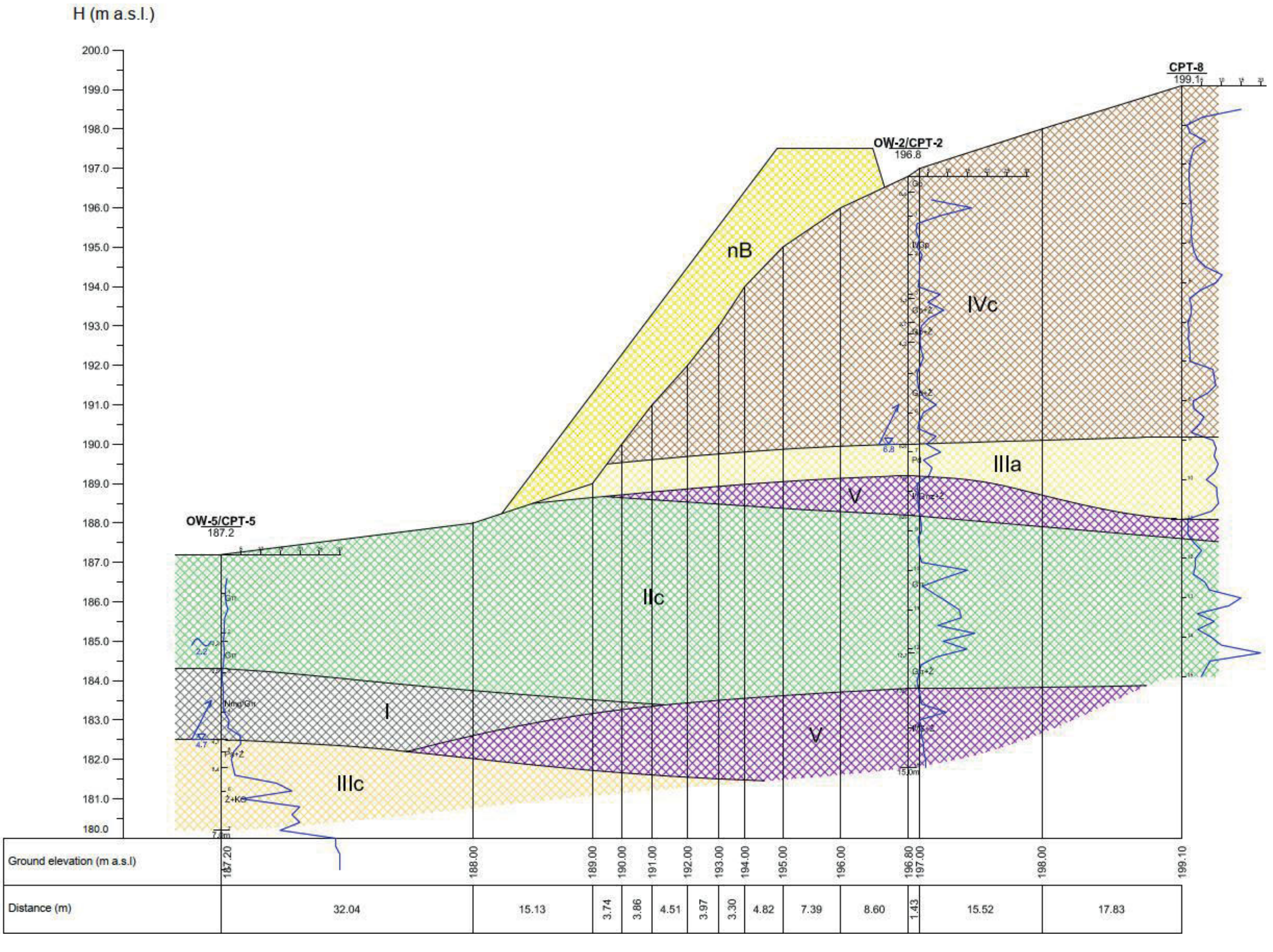


**Fig. 3.** Geotechnical cross-section II–II; layers as in Tab. 1; source: Geoteko Ltd. (2019)

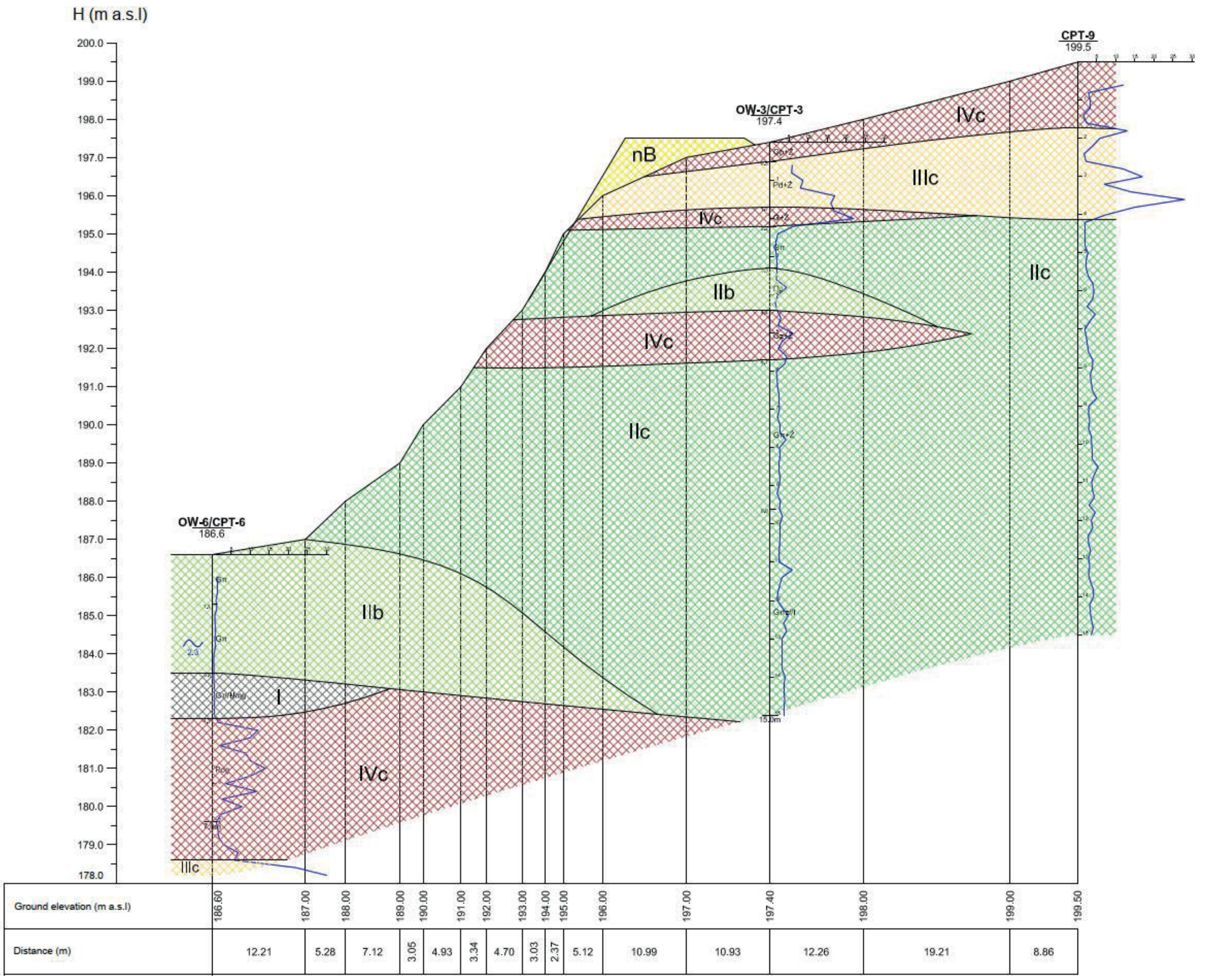


**Fig. 4.** Geotechnical cross-section III–III; layers as in Tab. 1; source: Geoteko Ltd. (2019)

Wołyński, 2018). To perform the stability analysis, it was necessary to identify parameters such as soil bulk density ($\rho$), effective internal friction angle ($\varphi$'), soil cohesion ($c$'), and the permeability coefficient ($k$) (Kosiński and Leśniewski, 2009). The parameters used in the stability calculations are summarised in Table 1.

**Table 1.** Geotechnical parameters used in stability calculations

| Layer | Soil type | Parameter value | | | | | |
|---|---|---|---|---|---|---|---|
| | | $I_L$ (–) | $I_D$ (–) | $\rho$ (g·cm$^{-3}$) | $\varphi$' (°) | $c$' (kPa) | $k$ (m·s$^{-1}$) |
| nB | noncohesive engineered fill | – | >0.70 | 2.00 | 40 | – | 0.001 |
| I | clayey silt//silty clay//peat | 0.40–0.50 | – | 1.85 | 10 | 5 | 1E–08 |
| IIb | silty clay, silt | 0.25–0.35 | – | 2.00 | 20 | 5 | 1E–08 |
| IIc | silty clay, silt | 0.10–0.25 | – | 2.00 | 20 | 5 | 5E–07 |
| IIIa | noncohesive soils (sands from fine to gravelly) | – | 0.33–0.40 | 1.80 | 32 | 0 | 0.005 |
| IIIc | noncohesive soils (sands from fine to gravelly) | – | >0.67 | 1.80 | 37.5 | 0 | 0.01 |
| IVc | sandy clay, stiff clay | 0.10–0.25 | – | 2.00 | 23 | 5 | 1E–07 |
| V | clay, silty clay | <0.10 | – | 2.10 | 20 | 10 | 1E–08 |

Source: own elaboration.
Explanations: $I_L$ = plasticity index, $I_D$ = density index, $\rho$ = bulk density of soil, $\varphi$' = effective internal friction angle, $c$' = soil cohesion, $k$ = coefficient of permeability.

## METHODOLOGY AND CALCULATION ASSUMPTIONS

Stability analyses were conducted using the GeoSlope software from the GEO-STUDIO package (Seequent, 2023). The calculations were carried out using the limit equilibrium methods (Bishop, Morgenstern–Price, and Fellenius), applying the computer program SLOPE/W to define the potential slip surface and calculate the factor of safety of selected cross-section under change water level condition in the reservoir. For the purposes of this article, only the Bishop method is described, which assumes the division of the potential landslide volume into vertical calculation blocks. The assumptions for the most popular used methods are as follows: the slip surface has a cylindrical shape (it is worth noting that the GeoSlope software gives us the possibility to carry out calculations also for an optimised slip surface – in our case we used a cylindrical curve), the calculations do not take into account friction between individual blocks, the forces acting between individual blocks are oriented horizontally and their projection on the vertical direction is zero, the stability factor is determined by the equilibrium equations of the moments of forces with respect to the centre of the slip surface.

The stability factor in the Bishop method is determined by the ratio of the retaining forces to the sliding forces. In addition, it is worth noting that the final result of the stability factor is obtained by consecutive iterations, as there is a factor on both sides of the Equation (1). The minimum value of the factor of safety that will provide a sufficient degree of safety is 1.5. A simplified formula for the Bishop's method taking into account the position of the ground water table is shown below.

$$F = \frac{1}{\sum W_i \cdot \sin\alpha_i} \sum \frac{[W_i - u_i \cdot L_i \cos(\alpha_i)] \cdot \tan(\varphi_i{}') + c_i{}' \cdot L_i \cos(\alpha_i)}{\cos(\alpha_i) \cdot \left[1 + \frac{\tan(\varphi_i')}{F} \cdot \tan(\alpha_i)\right]} \tag{1}$$

where: $F$ = slope stability coefficient (–), $W_i$ = weight of the block (kN·m$^{-1}$), $u_i$ = pore water pressure at the base of the block (kPa), $L_i$ = length of the base of the block (m), $\alpha_i$ = slope angle of the tangent to the base of the block (°), $\varphi_i'$ = effective internal friction angle (°), $c_i'$ = soil cohesion (kPa)].

Stability calculations were carried out in cross-sections II–II and III–III through the reservoir slope, in several calculation variants:

1a) natural slope – dry reservoir,
1b) natural slope – normal reservoir water impoundment level (*NPP*) water table elevation 191 m a.s.l.;
1c) natural slope – maximum reservoir water impoundment level (Max*PP*), water table elevation 195.2 m a.s.l.;
2a) natural slope with buttress embankment – dry reservoir;
2b) natural slope with buttress embankment – normal reservoir water impoundment level (*NPP*), water table elevation 191 m a.s.l.;
2c) natural slope with buttress embankment – maximum reservoir water impoundment level (Max*PP*), water table elevation 195.2 m a.s.l.

# RESULTS AND DISCUSSION

## RESULTS AND DISCUSSION OF STABILITY ASSESSMENTS

The results of stability calculations (FOS – factor of safety values) are shown in Table 2. Examples of (selected) stability calculation results showing the location of the critical slip surface in cross-section II–II for selected critical design variants are shown in Figures 5–7. The results of the calculations indicate that stability will be maintained in each calculation case, and the lowest value of the stability factor (1.63) was obtained for the scenario in which the water table position in the reservoir was at the elevation of 191 m a.s.l. (Tab. 2). The minimum factor of safety to be achieved by performing calculations on characteristic parameters is $F > 1.5$, under special operating conditions, for the flood regime, for fast and slow rainfall, the value can be in the range of 1.2–1.5.

The results of additional analysis of the impact of changing the internal friction angle of layer IIc or IVc shows that even when the value is reduced to 15 degrees, the stability factor does not fall below 1.5 (Fig. 6). From the stability calculations carried out, despite the assumed very safe values of geotechnical parameters, the most significant changes in the value of the stability factor result from changes in the water level in the reservoir. In order to show what is the real impact of an increase in the water level in the reservoir, followed by a sudden decrease for cross-section III–III an additional stability analysis carried out over time consisting of the following stages: stage 1 – filling the reservoir within 7 days to the ordinate of 195.2 m a.s.l., stage 2 – maintaining the filled reservoir at the ordinate of 195.2 m a.s.l. for the next 30 days, stage 3 – emptying the reservoir within 1 day to

**Table 2.** Stability calculation results

| Cross-section | Factor of safety (*F*) in different conditions, acc. to three calculation methods | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | dry reservoir | | | impoundment *NPP* = 191 m a.s.l. | | | max. impoundment Max*PP* = 195.2 m a.s.l. | | |
| | Bishop | MP | Fellenius | Bishop | MP | Fellenius | Bishop | MP | Fellenius |
| II–II (natural slope) | 2.42 | 2.43 | 2.24 | 1.99 | 1.99 | 1.89 | 2.62 | 2.72 | 2.48 |
| II–II (embedded counterfort) | 2.06 | 2.05 | 1.94 | 1.71 | 1.70 | 1.63 | 2.10 | 2.09 | 1.97 |
| III–III (natural slope) | 2.07 | 2.06 | 1.96 | 1.80 | 1.80 | 1.74 | 2.21 | 2.20 | 2.02 |
| III–III (embedded counterfort) | 1.99 | 1.98 | 1.92 | 1.73 | 1.72 | 1.64 | 1.96 | 1.95 | 1.84 |

Explanation: MP = Morgenstern–Price.

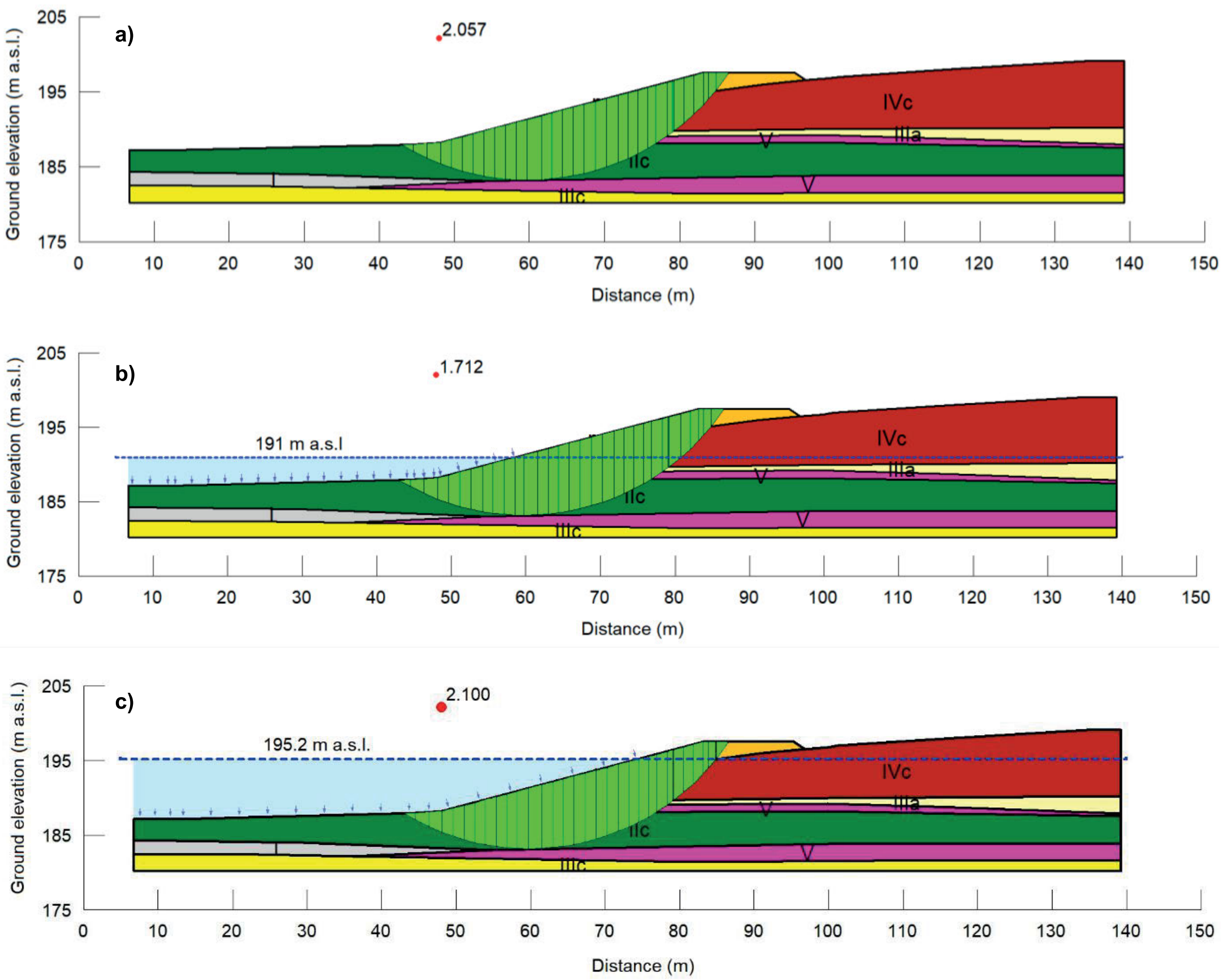


**Fig. 5.** Factor of safety in section II–II for the natural slope with an embedded counterfort in the conditions: a) variant 2a, b) variant 2b, c) variant 2c; source: own study

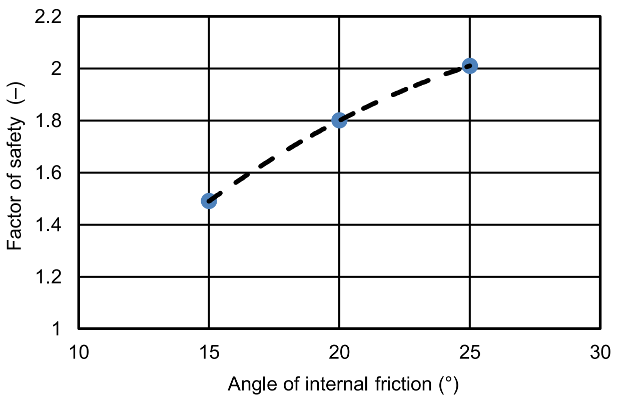


**Fig. 6.** Factor of safety vs. soil strength parameter (angle of internal friction); source: own study

the ordinate of 191 m a.s.l. (Fig. 7). This is surprising that coefficient for Max*PP* is higer than for *NPP* however, it is possible that the increase in water level in the reservoir in this case improves stability. Considering the shape and position of the critical slip curve, this results from the fact that the volume weight of the soil of the slope due to uplift ("loading part") caused by the increase in the water level, due to the filling of the reservoir. This is most likely due to the reduction in the weight of the lower part of the slope (below the water level) caused by the uplift of the water dammed in the reservoir. It should be noted that this analysis uses constant values of soil strength parameters, which, based on the results of the study, were assumed very safe (the lowest possible values). This also results from the assumptions of limit equilibrium methods. This means that the slope of the Racibórz Dolny Dam will be safe and there is no danger associated with the loss of stability when the analysed cases occur.

## RESULTS OF STABILITY CALCULATIONS VS. FLOOD 2024

In 2024, the Racibórz Dolny reservoir played a key role in protecting against flooding on the Oder River. During intensive rainfall and river flooding, the facility reached a filling level of around 80% of its capacity, which was approximately 148 mln $m^3$ of water (PGW Wody Polskie, 2025). The water level in the reservoir was then approximately 193.9 m a.s.l. Stability analysis was carried out in the range of water level changes in the reservoir from dry reservoir to filling at an elevation of 191 m a.s.l. to 195.2 m a.s.l.

There was no observed instability of the reservoir slopes during the crisis, and the reservoir fulfilled its function of flood

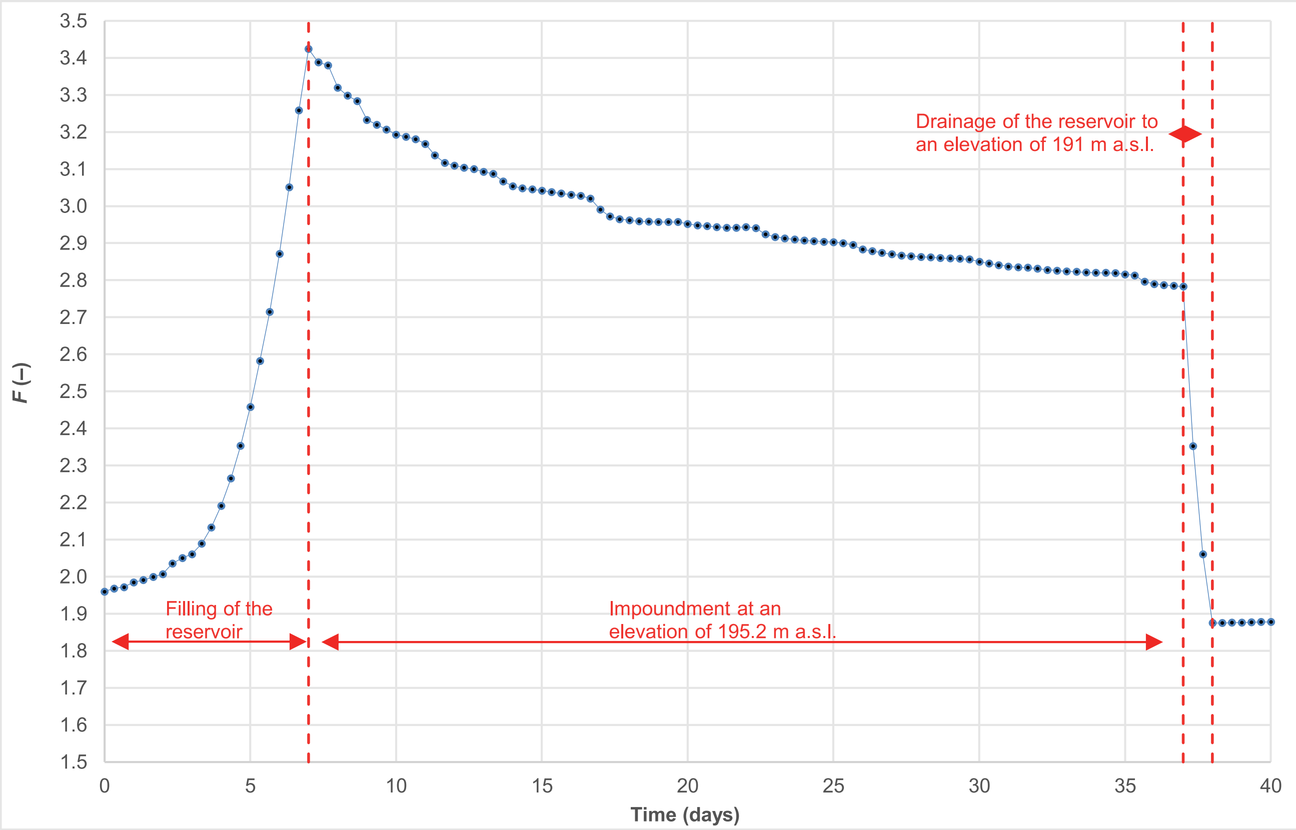


**Fig. 7.** Changes of factor of safety (*F*) with time in the conditions of water level changes for section III–III with an embedded counterfort; source: own study

protection, minimising potential losses and risks to residents in the surrounding area. For the critical slip surface, critical factors of safety were calculated and compared to the admissible factors of safety established in the legislation and the technical regulations in force in Poland (1.2–1.5 for the flood regime, for the rapid and the slow drawdown).

## CONCLUSIONS

The slope stability analysis of flood control reservoirs is an important part of the safety assessment of these structures and should be carried out on a regular basis, according to current standards. However, it should be emphasised that a reliable stability assessment requires consideration of many factors, adherence to well-established procedures and, most importantly, a thorough recognition of the geotechnical conditions of the subsoil and assuming possible scenarios of water level changes in the reservoir.

The results of the calculations indicate that stability will be saved in each calculated case. The lowest value of the stability factor (1.63) was obtained for the scenario in which the water table in the reservoir was at an elevation of 191 m a.s.l. (normal reservoir water impoundment level – *NPP*). This means that the slope stability of the reservoir is most affected by changes in the water level, especially for the *NPP* elevation. This is surprising that coefficient for maximum reservoir water impoundment level (Max*PP*) is higher than for *NPP* however, it is possible that the increase in water level in the reservoir in this case improves stability. This is most likely due to the reduction in the weight of the lower part of the slope (below the water level) caused by the uplift of the water dammed in the reservoir.

The events of the 2024 flood confirmed the effectiveness of control stability analyses, which provided reliable information on the safety level of flood reservoir slopes and their behaviour under extreme conditions that occurred that year.

## CONFLICT OF INTERESTS

All authors declare that they have no conflict of interests.

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