

Article

Slope Stability Evaluation of Earthen Hydraulic Structures at the Dychów Pumped-Storage Power Plant by Electrical Resistivity Tomography and Finite-Element Modelling

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Abstract

Pumped-storage hydropower (PSH) remains the main grid-scale energy storage technology in Europe, yet much of the fleet is ageing and requires periodic verification against current geotechnical standards. The Dychów plant (88 MW, western Poland), in service since the 1930s and classified as national critical energy infrastructure, has a documented history of surface mass movements, including a 1997 landslide on the frontal dam. To reassess its condition, two earthen sections were analysed: lateral section of the frontal dam of the upper reservoir and the embankment of the derivation channel. Electrical resistivity tomography (ERT) profiles, measured using a gradient array in 2023 and further detailed in 2024 along the same GNSS-fixed lines, imaged the internal structure of both sections. The resistivity cross-sections, verified against shallow control boreholes and archival geological data, supplied the geometry of the finite-element (FEM) models in ZSoil: the confirmed layer boundaries became the material zones, and piezometric observations set the groundwater boundary conditions. The safety factor SF was then computed with the shear-strength reduction technique for four calculation variants and two groundwater scenarios per section. The resulting SF equals 1.75 for the side section of the frontal dam area and ranges from 1.80 to 2.10 for the channel embankment. A parametric reduction in the friction angle of saturated medium sand gives limit values of $\varphi = 12.3^\circ$ (dam) and $\varphi = 20.3^\circ$ (embankment), which are clearly below realistic in situ values. Overall, both structures meet the $SF \geq 1.50$ requirement for Class I hydraulic structures. The ERT-to-FEM workflow offers a non-invasive and repeatable tool for the periodic reassessment of ageing PSH infrastructure, which continues to balance variable renewable generation.

Keywords: geophysics; finite element method; earth dam; embankment; glaciotectionics

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1. Introduction

The decarbonisation of power systems across the European Union has strongly increased the demand for fast-reacting, large-scale energy storage that can complement the variable generation from wind and photovoltaic sources. Pumped-storage hydropower (PSH) is still the most mature and most widely deployed grid-scale energy storage tech-

nology [1,2], and its role in supporting frequency control, providing reserves and enabling black-start capability is now widely recognised in the ongoing energy transition [3]. Several recent studies have addressed the flexibility of hydropower in the Polish and wider European contexts—including specific Polish case studies such as the Jeziorsko reservoir on the Warta river [3,4], the evolution of Polish hydropower support schemes [5], and more broadly the development and current status of the Polish hydropower sector [6]—as well as the hydroclimatic constraints that will increasingly affect the operation of hydro schemes. What is perhaps less frequently discussed, but equally important, is that a large share of the European hydro fleet—including pumped-storage plants—is ageing and is already treated, in part, as industrial heritage [7]: many units were commissioned between the 1930s and the 1970s and were designed on the basis of codes that predate modern geotechnical standards such as Eurocode 7 [8,9]. The safe and economically attractive continuation of their operation depends, among other things, on the reliable periodic assessment of the earthen retaining structures that form their civil-engineering backbone.

The Dychów hydro complex (Lubusz Voivodeship, western Poland; Figure 1) is a representative example of such an asset. It consists of seventeen hydroelectric plants located on the Bóbr, Kwisa and Nysa Łużycka rivers, with a total installed capacity of 103 MW, of which 88 MW belong to the Dychów pumped-storage power plant (hereafter ESP Dychów). The plant is the sixth-largest facility of its kind in Poland by installed capacity and is part of the national critical energy infrastructure, supporting the synchronous operation of the Polish transmission system and providing peak-shaving and ancillary services to the interconnected European grid.

The Dychów plant was built between 1932 and 1936 and, after numerous modernisations, is still in operation. Its ninety-year history has brought several challenges, both in terms of technical maintenance and the safety of the earthen structures, the latter resulting from difficult subsoil and groundwater conditions [10]. In 1997, a dangerous surface mass movement took place on the frontal dam of the upper reservoir. The landslide was probably a consequence of long-lasting rearrangement of the soil skeleton within the dam body, caused by intensive and prolonged filtration. It destroyed about 60 m of the crest road and caused significant damage to the powerhouse building [11], with a direct impact on plant availability. Excessive seepage from the upper reservoir resulted from a marked deterioration of the bottom and upstream-face sealing in the dam area, combined with complex ground conditions in the foundation layer—in particular the naturally unfavourable, inclined contacts of soil layers disturbed by glaciotectionics. The top of the cohesive clays interbedded with till and non-cohesive sands and gravels, lying at the base of the slope, formed a preferential pathway both for groundwater flow and for moving landslide masses. Events of this kind underline that, from the point of view of the electric-power system, the reliability of a PSH unit depends not only on the electromechanical equipment but, to a large extent, also on the condition of the civil structures that impound the upper reservoir and convey the water.

The study addresses the stability of selected earthen slopes of the hydraulic structures of the Dychów plant, identified as particularly prone to damage or failure. The areas of interest and the calculation cross-sections were chosen on the basis of field reconnaissance and consultations with the technical staff responsible for the operation and maintenance of the plant. A cross-section of the lateral section of the earthen frontal dam was prepared and analysed with regard to the previously documented seepage and suspected loosening processes of the body soils. A second cross-section was developed for the stability analysis of the downstream slope of the embankment of the derivation channel that feeds the upper reservoir, within a reach where intensive, above-average growth of hydrophilic vegetation is systematically observed. The channel embankment crosses a natural depression whose morphology suggests a risk of surface runoff.

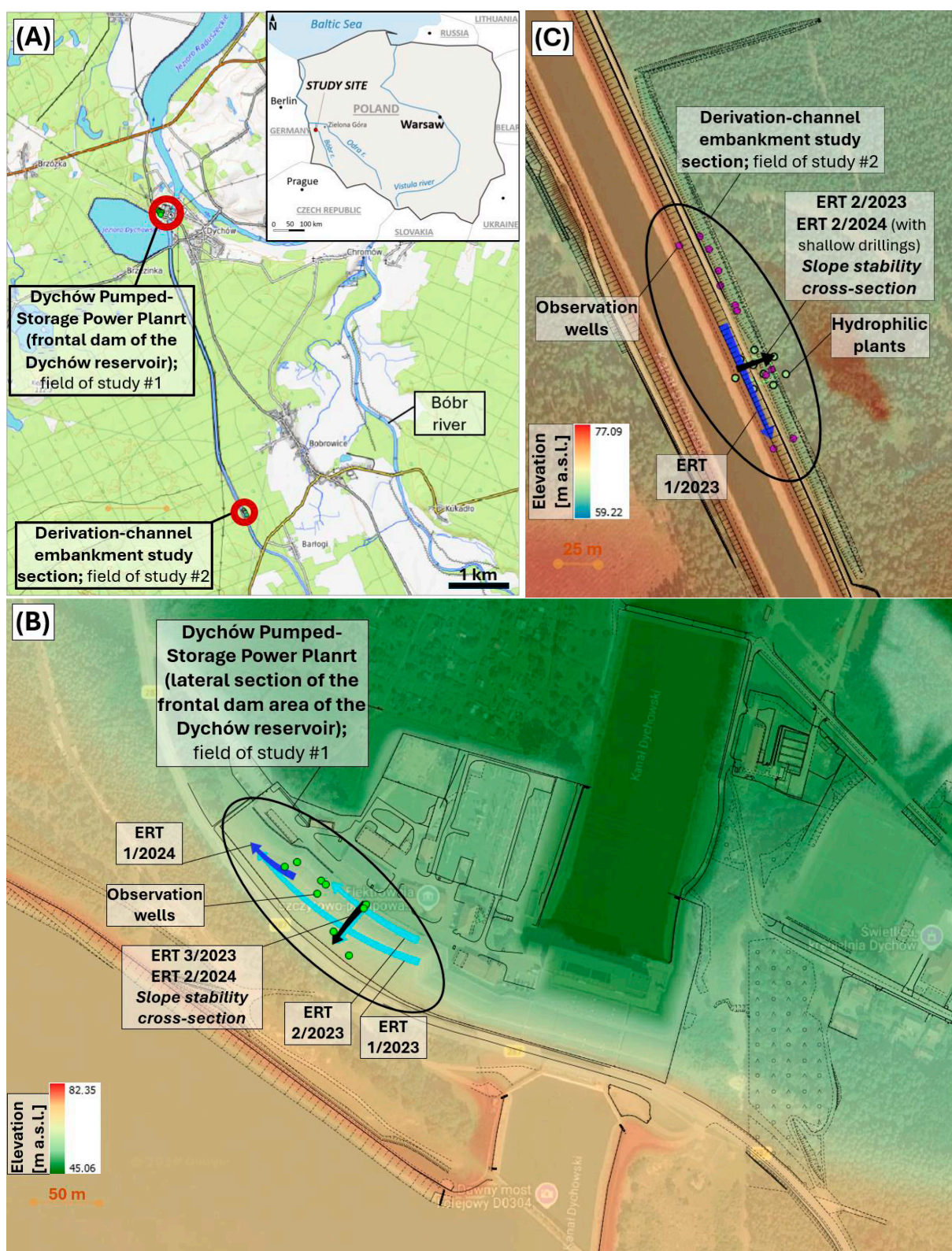


Figure 1. Fields of study: (A) Local map (based on OSM topography) with schematic country map; (B) Dychów Pumped-Storage Power Plant (frontal dam of the Dychów reservoir); (C) Derivation-channel embankment study section.

Slope stability was analysed for a set of variants covering different boundary conditions, both with regard to hydrogeological settings and the strength characteristics of the soils that form the structures and their subsoil. Numerical models were defined on the

basis of the relevant geological-engineering literature, the ERT results and supplementary hydrogeological observations.

The stability of the selected earthen retaining structures was analysed with regard to the recommendations of the Regulation of the Minister of Environment of 20 April 2007 on the technical conditions to be met by hydraulic structures and their location [12]. The analysis also took into account the most frequent causes of slope instability in Poland reported in the literature [13]: loading by buildings or embankments (30.8% of landslides), erosional undercutting (23.1%), infiltration of rainwater (20.0%), design errors (10.8%), undercutting by excavation (10.8%) and abrasion (4.5%).

The scope of the work covered the analysis of available archival materials and of the domestic and international subject literature. On this basis, the site conditions and the structures of interest were described. Next, the geophysical data (geoelectric cross-sections) were analysed together with a reinterpretation of the archival documentation. Hydrogeological measurements were then used to produce maps of the groundwater-table elevation (hydroisohypses). Both the hydrogeological maps and the spatial integration of data were prepared within a dedicated geographic information system (Quantum GIS). In this way, conceptual numerical models of the ground were established and used in the stability analyses aimed at determining the “global” safety factor SF . The calculations were performed with ZSoil, a specialised finite-element package implementing the shear-strength reduction method. The numerical approach itself has a long track record in geotechnics. Griffiths and Lane [14] demonstrated that finite-element strength reduction is a reliable alternative to the classical limit-equilibrium methods, and Cheng et al. [15] later compared the two techniques over a wide range of slope configurations, pointing out the numerical conditions under which the reduction procedure should be used with care. Stability analysis has also been extended to media whose behaviour is governed by geological interfaces, for example in the discrete-fracture model applied by Wang et al. [16] to jointed rock slopes; for the earthen structures considered here, a continuum description remains appropriate. The results, expressed both quantitatively and qualitatively, are summarised in the final part of the paper.

In parallel, research on the evaluation of slope stability has developed in several directions that go beyond the classical limit-equilibrium and strength-reduction schemes. Probabilistic formulations, in particular the random finite-element method, treat the spatial variability of soil strength explicitly and express the outcome as a probability of failure rather than a single safety factor [17]. Where the post-failure stage is of interest, large-deformation techniques such as the material point method make it possible to follow the runout of the sliding mass [18]. Machine-learning models, trained on large sets of computed or documented cases, are increasingly used as fast surrogates in such stochastic analyses [19]. Another active branch is formed by non-local methods, above all peridynamics, in which the spatial derivatives of continuum mechanics are replaced by integral operators, so that discontinuities can initiate and propagate in the model without a predefined path. Peridynamic formulations have already been applied to coupled water flow in unsaturated porous media [20], to mixed-mode fracture processes in rocks [21] and, recently, to ground deformation and fissuring driven by groundwater withdrawal [22], which makes them a promising research direction for hydro-geotechnical problems of the type considered here. At the same time, these approaches require input data (e.g., statistics of the spatial variability or non-local material parameters) and a computational effort that are rarely available in the routine verification of existing structures. For the periodic, code-based assessment of an ageing dam, the finite-element strength-reduction technique therefore remains the reference engineering tool and was adopted in this study.

The main contribution of this paper is not a new geophysical or numerical method, but a repeatable assessment procedure assembled from existing tools and adapted to age-

ing pumped-storage structures of critical energy-infrastructure rank. The procedure couples a fixed-profile ERT survey, repeated in a defined period (i.e., every one or two years) and co-registered with GNSS, with a hydrogeological reinterpretation and a finite-element shear-strength-reduction model. In the present study, the scheme was applied in two stages: a full reconnaissance survey in 2023, followed in 2024 by higher-resolution detailed profiles measured along the same lines on the lateral section of the frontal dam. Because the profiles keep the same position and are located with GNSS, the cross-sections obtained in different years can be compared, so the same workflow gives at once the current factor of safety and a first indication of slow changes in the soil structure, without any permanent instrumentation of the dam or the embankment. For this reason, the procedure can be applied, at a comparable cost, to a larger group of Class I hydraulic structures of an ageing hydropower fleet, and not only to a single object. The Dychów plant is used here as a worked example of such a reassessment.

In short, the paper offers three outcomes: (i) a repeatable, non-invasive assessment procedure that couples fixed-profile ERT, GNSS positioning and finite-element strength reduction; (ii) its complete demonstration on two earthen structures of an operating Class I pumped-storage plant, with the obtained safety factors checked against the current requirements; and (iii) an open statement of the infrastructure-ageing dimensions that the procedure captures and which require permanent monitoring or more advanced models.

The rest of the paper is organised as follows. Section 2 characterises the geological and engineering conditions of the study area. Section 3 describes the methods, from the field reconnaissance and the ERT surveys to the construction of the finite-element models. Section 4 presents the geophysical results, Section 5 defines the calculation variants and the soil parameters, and Section 6 reports the stability results for the lateral part of the frontal dam area and for the derivation-channel embankment. Section 7 discusses the results, their limitations and their wider implications, and Section 8 lists the conclusions.

2. Geological and Engineering Conditions of the Study Area

Two objects of the Dychów pumped-storage plant, both located in the Bobrowice region, are the subject of this study: the lateral part of the earthen frontal dam of the upper reservoir, and the embankment of the derivation channel connecting the Bóbr river to the Dychów reservoir.

The main parameters of the frontal dam are as follows: height 27 m, crest width 11.25 m, crest length 573 m. The upstream-slope inclination is 1:2.5 to 1:3, and the downstream slope is 1:2.

The Dychów channel is an artificial derivation channel running between the villages of Krzywaniec and Dychów. Its length is 20 km, width is 20 m and its depth is about 8 m. The cross-section has a trapezoidal shape; the banks and bottom are protected with reinforced-concrete slabs, forming a barrier that separates the channel water from the groundwater. The channel supplies water to Lake Dychów and is also a local tourist attraction.

Based on the analysis of archival materials [23] and the detailed geological maps of the surface [24–27], complex ground conditions can be identified for the area of interest with regard to glaciotelectonics. Partially complex conditions also occur in those reaches of the embankment where the top of the glaciotectionally disturbed deposits lies below the embankment base.

In the area of the frontal dam (Figure 2), made-up sand fills occur (with an average thickness of about 10 m), resting on sandy tills of considerable thickness (up to several tens of metres) with an irregular top surface. These conditions are responsible for ground-water damming in the upper part of the dam, which translates into an increased hydraulic gradient and a risk of developing internal erosion, as well as an increase in the plasticity index of the cohesive soils.

In the analysed reach of the channel embankment (Figure 3), the ground is more homogeneous. From the ground surface downwards, the profile consists of made-up medium sands, followed by clayey sands, saturated sands, and sandy tills. The embankment bodies of both objects are made up mainly of non-cohesive sands, for which the parameters describing the quality of compaction are of primary importance when their condition is assessed [28].



Figure 2. The location of the investigations on the lateral part of the frontal dam area of the Dychów reservoir with detailed geological map of Poland and Google satellite map in the background [24].

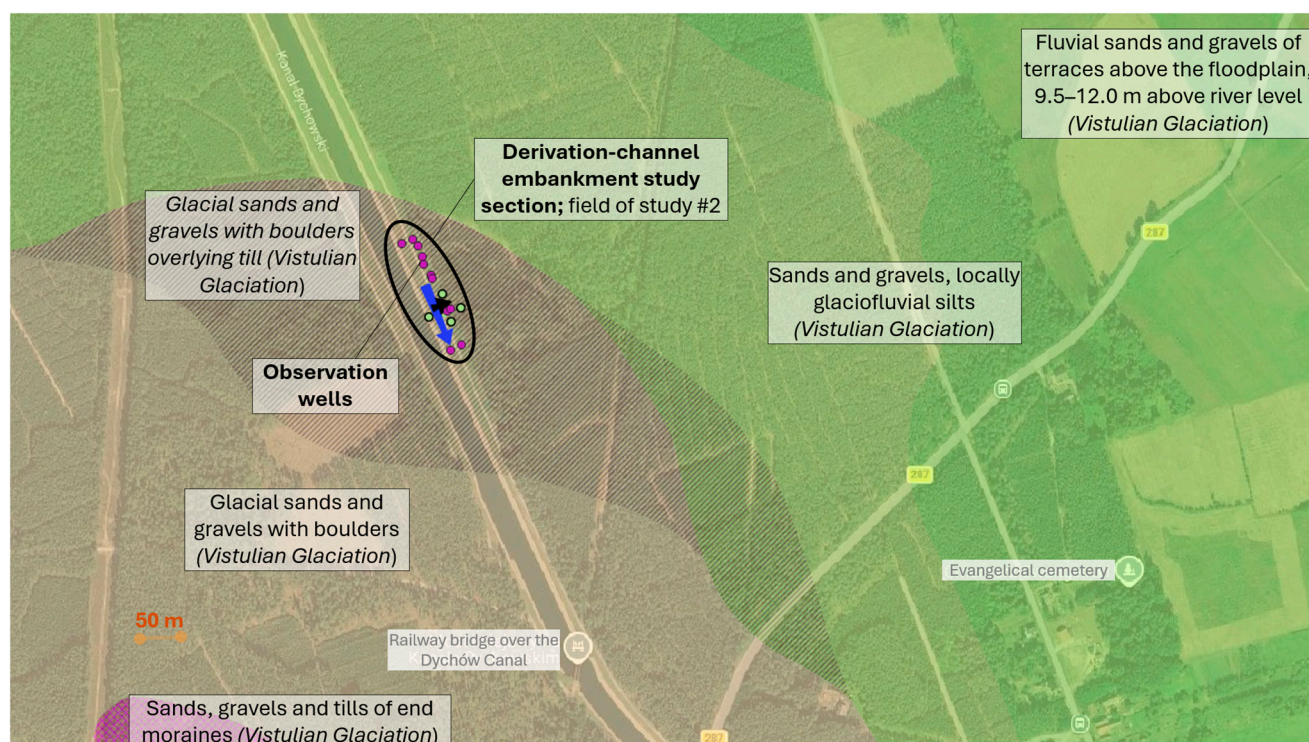


Figure 3. The location of the investigations on the derivation-channel embankment (arrows indicate ERT profiles; the black arrow also indicates the cross-section used for FEM calculations; background: Google satellite map) with detailed geological map of Poland in the background [24].

3. Methods

Following the provisions of Eurocode 7 (PN-EN 1997) [8,9], concerning geotechnics, and the calculation procedures used for the assessment of earthen and foundation structures, the analyses were based on advanced computational techniques employing the finite element method (FEM). This method allows a precise reproduction of complex ground conditions and the geometry of the analysed objects (the dam and the embankment). In the context of hydraulic structures, the shear-strength reduction (SRM) technique implemented within FEM codes [14,29] has been successfully applied for the slope stability evaluation of dam foundations under seepage and hydraulic–mechanical coupling. The reliability of such analyses depends largely on how well the models are constrained by field data. Geophysical surveys have proved effective in locating water escape zones within retention reservoirs and their surroundings [30]. In addition, advanced monitoring approaches, such as thermal and distributed fibre-optic monitoring, can be used to verify both the computed response of soil–structure systems and the adopted soil parameters [31]. In the present study, this constraining role is played by the ERT profiles and the piezometric observations described in the following sections.

The following procedure was applied in order to determine the safety factor:

1. Field reconnaissance (two campaigns) with hydrogeological observations (ground-water table, ER, TDS, temp.; September 2023 and September 2024; air temperature above 20 °C), GNSS measurements, shallow boreholes.
2. The digital terrain model was used to define the calculation profiles of the ground surface. The source data were obtained from airborne laser scanning, distributed by the Head Office of Geodesy and Cartography in Warsaw.
3. Based on the results of the geophysical surveys (electrical resistivity tomography, ERT, performed using an ABEM Terrameter LS 2 system manufactured by Guideline Geo AB, Stockholm, Sweden) [32], the division of the model into material zones (soil layers) was defined. The ERT profiles were measured in a gradient array; the elec-

trode spacing and the resolution were adapted to each profile, and their lengths, maximum prospecting depths and inversion errors are collected in Table 1. During post-processing, a topographic correction was applied. The ERT investigations were carried out in two stages: the 2023 survey provided full reconnaissance of both structures, and in 2024, selected sections of the lateral part of the frontal dam were measured again along the same lines (owing to the GNSS measurements) with shorter detailed profiles and the electrode spacing reduced to 0.5 m. The overlap of the two series also allows the repeatability of the resistivity image to be checked; the outcome of this comparison is discussed in Section 4. Details of the ERT prospecting are given in Table 1.

4. The model was supplemented with boundary conditions describing the position of the groundwater table. Piezometric observations, the hydrogeological map of Poland and a dedicated custom hydroisohypse map were used for this purpose (generated in QGIS, using the multilevel B-spline interpolation from the SAGA toolset [33]).
5. Finite-element discretisation schemes were prepared, and material properties were assigned to the corresponding zones. For the models, the factor of safety was calculated with the shear-strength reduction method (c – φ reduction) implemented in ZSoil software (GeoDev Sàrl, Lausanne, Switzerland).

Table 1. ERT prospecting methodological information.

Location	Profile	Profile Length (m)	Max. Prospecting Depth (m)	Electrode Spacing (m)	Approximation (Inversion) Error (RMS)
Dam (field study #1)	ERT 1/2023	160	~35	2	9.8% (iter. 7)
	ERT 2/2023	80	~17	1	3.1% (iter. 5)
	ERT 3/2023	40	~10	1	1.5% (iter. 7)
	ERT 1/2024	40	~9	0.5	3.4% (iter. 6)
	ERT 2/2024	40	~9	0.5	2.5% (iter. 7)
Embankment (field study #2)	ERT 1/2023	80	~16.5	1	2.2% (iter. 7)
	ERT 2/2023	28	~5	0.5	0.91% (iter. 7)

The successive stages of the calculations included: (i) the determination of the safety factor for the model reproducing the actual ground and water conditions; (ii) the determination of the safety factor for other possible variants of soil state and strength parameters; and (iii) the identification of the limit value of the internal friction angle of the sands at which the safety factor reaches 1.00 (e.g., as a consequence of a strength loss associated with internal erosion [34]).

As regards the numerical models themselves, both problems were treated in ZSoil (version 18) as two-dimensional plane-strain cases. The cross-sections were discretised with four-node quadrilateral continuum elements. The meshes were structured and locally refined along the slope surface and at the contacts between the soil layers, where strain localisation was expected. In this way, the dam model consisted of 2229 elements (the smallest with an area of about 0.2 m²) and the embankment model of 1740 elements (the smallest of about 0.1 m²).

The mechanical boundary conditions were applied in a manner typical of slope problems: horizontal displacements were restrained on both vertical edges of the models, the bottom edge was fixed in the horizontal and vertical directions, and the ground surface was left free. The initial stress state was generated by the self-weight of the soil, using the earth-pressure coefficients at rest, together with the pore-pressure field defined by the groundwater tables described in point (4). The lateral edges of the models also reproduced the groundwater inflow, while the reservoir and the concrete-lined channel were treated as sealed, without a direct hydraulic connection with the groundwater. The factor of safety

was then obtained by the proportional reduction in c and $\tan \varphi$ until the elastic–plastic solution lost convergence, which corresponds to the activation of the failure mechanism; no slip surface needs to be assumed in advance in this approach.

4. Geophysical Survey Results

The results of the geophysical surveys (ERT) are presented below as resistivity cross-sections, obtained on lateral part of the frontal dam area of the Dychów reservoir (Figure 4) and on the derivation-channel embankment (Figure 5).

The colour scale represents resistivity values, which mainly reflect lithology, followed by the saturation of the pore space and the soil porosity. In both cross-sections, zones of low resistivity (below about 80 Ωm) and of high resistivity (above about 500 Ωm) can be identified. The former correspond to cohesive soils (blue and green), while the latter correspond to soils dominated by the sand fraction (dark red and brown). The intermediate zones (80–500 Ωm) are associated with a larger content of silt or clay fractions, or with the presence of groundwater (yellow and orange).

In the area of the frontal dam, the top of the cohesive soils is strongly disturbed, which is related to the glaciotectionic history of the area. A similar observation holds for the derivation channel, where the top of the cohesive soils is not flat-parallel either; in the longitudinal geophysical profile taken along the embankment crest, this zone drops below the base of the embankment body.

The two measurement series had different purposes. The 2023 survey provided a full reconnaissance of both structures, with profiles up to 160 m long and prospecting depths down to about 35 m, while the 2024 series was a detailing survey: selected sections of the lateral part of the frontal dam area were measured again along the same GNSS-fixed lines, with shorter profiles and the electrode spacing reduced to 0.5 m, in order to increase the resolution of the near-surface part of the dam body (Table 1). At the same time, the overlapping sections made it possible to verify how repeatable the resistivity image is. Within the resolution of the method, the 2024 sections reproduced the geometry of the main resistivity units obtained a year earlier, i.e., the position of the top of the cohesive soils and the extent of the saturated sands, and no new low- or high-resistivity anomalies appeared. The differences between the series concerned mainly the absolute resistivity values in the uppermost, unsaturated part of the sections, up to about 1–2 m below the surface, which is the zone directly controlled by the seasonal soil moisture. Over the one-year interval, no changes that could be attributed to the evolution of the soil structure within the dam body were observed.

The influence of seasonal precipitation on this comparison was limited by the design of the surveys rather than removed numerically. Both series were carried out in the same part of the hydrological year (September 2023 and September 2024, at air temperatures above 20 °C), that is at the end of the summer low-flow period, and each of them was accompanied by direct observations of the groundwater table and of the basic physico-chemical parameters of the water (Section 3). The saturation state of the subsoil was therefore known from measurements and was comparable in both years. Below the groundwater table, where the soils stay fully saturated, the resistivity image does not respond to short-term rainfall. For this reason, the features that persist in both series are interpreted as lithological and structural. It should be kept in mind that the 2024 series was designed as a detailing survey, not as a time-lapse repetition; since the profiles differ in length and electrode spacing, the comparison presented here remains qualitative. A strict time-lapse evaluation would require the full 2023 profiles to be re-measured with identical acquisition geometry and with reference to precipitation records.

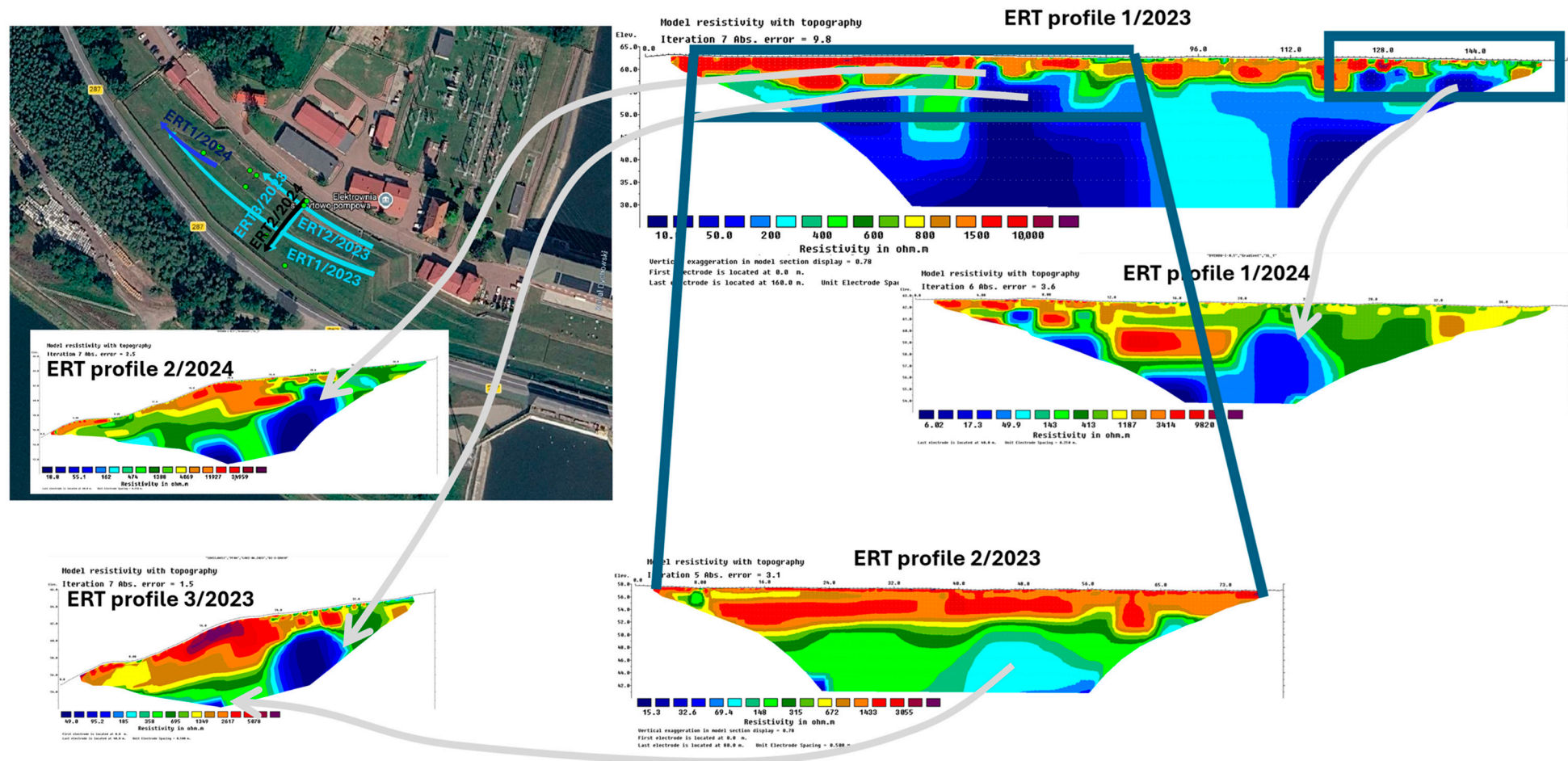


Figure 4. Resistivity cross-section of the profile measured on the lateral part of the frontal dam area of the Dychów reservoir.

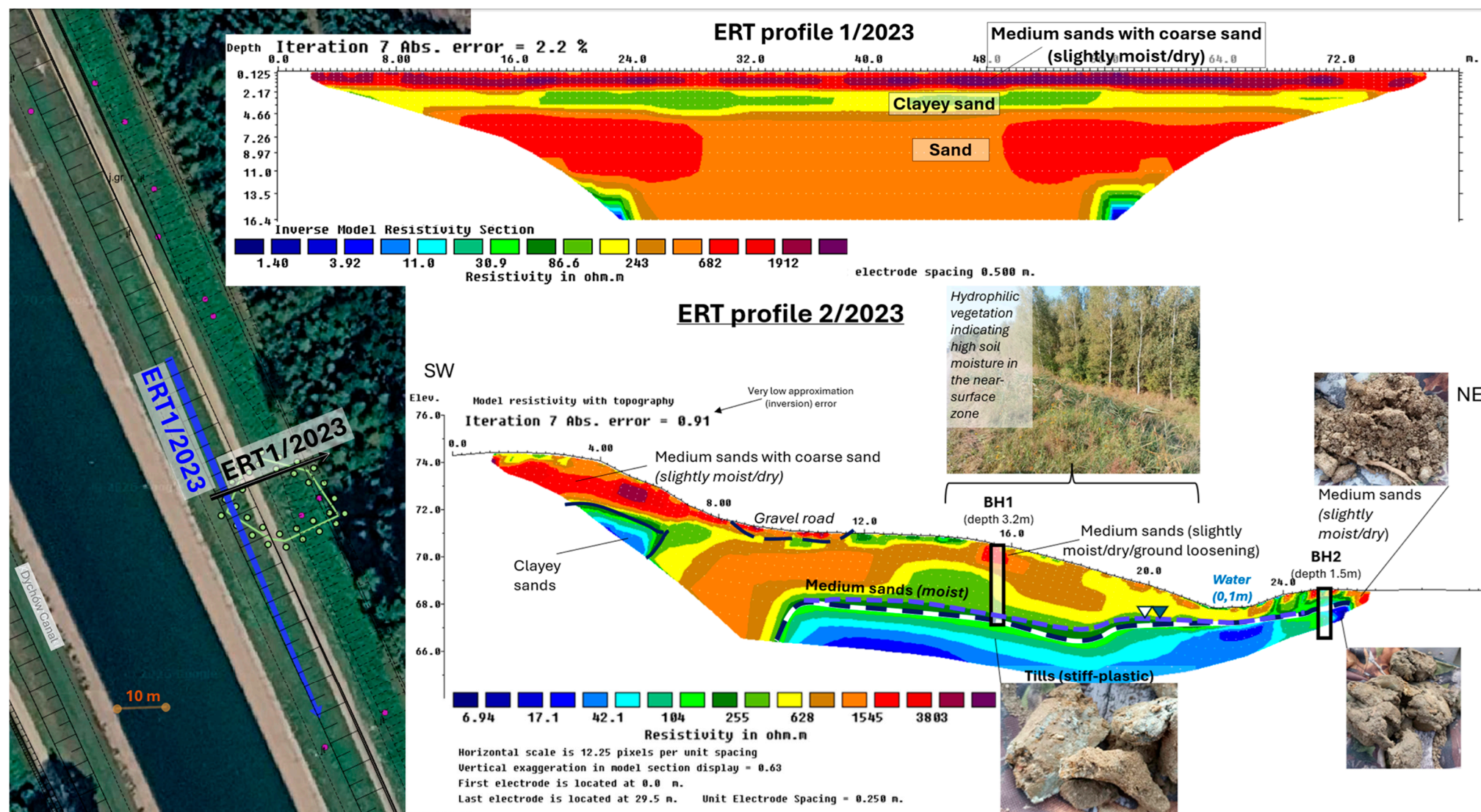


Figure 5. Resistivity cross-section of the profile measured on the derivation-channel embankment.

On the basis of the geological background, an interpretation was performed, the results of which were reproduced within the models implemented in ZSoil (Figures 6 and 7).

The calculation cross-section of the dam extends over a length of 120 m, encompassing the zone identified as a potential initiation site for mass movements that could lead to loss of stability (a steep slope combined with a local rise in the cohesive soils, sandy tills, whose top surface forms a potential preferential slip zone). In the second case, the stability analysis was restricted to the zone where the glaciotectionic disturbances appear at a shallow depth.

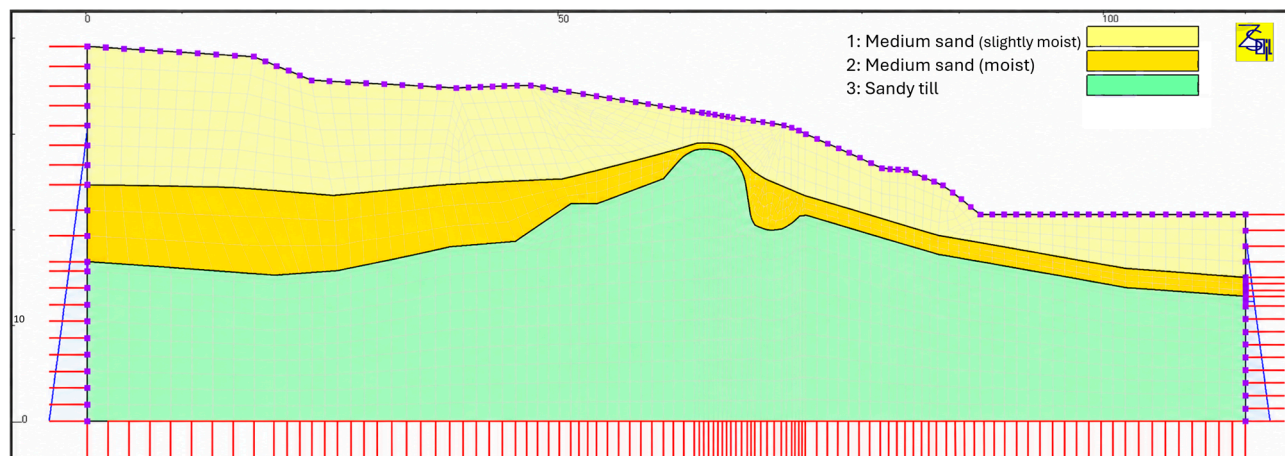


Figure 6. Cross-section of the lateral part of the frontal dam are with soil layers and ground-surface elevations.

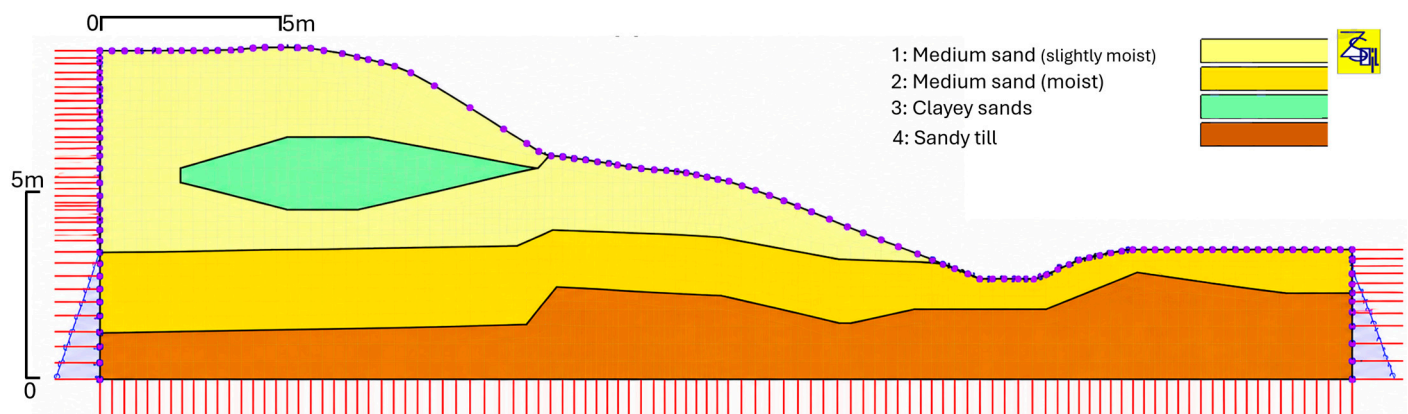


Figure 7. Cross-section of the derivation-channel embankment with soil layers and ground-surface elevations.

5. Calculation Variants and Soil Parameters

The variants and calculation assumptions adopted in the ZSoil models of the dam and the embankment are listed below:

Variant 1—soil layers from the ground surface downwards: moist medium sand; saturated medium sand; hard-plastic sandy till.

Variant 2—soil layers from the ground surface downwards: moist medium sand; saturated medium sand, with a reduced internal friction angle corresponding to $SF = 1.00$; hard-plastic sandy till.

Variant 3—soil layers from the ground surface downwards: moist medium sand; saturated medium sand; plastic sandy till (a 1 m thick top layer); then hard-plastic sandy till.

Variant 4—soil layers from the ground surface downwards: moist medium sand; saturated medium sand, with a reduced internal friction angle corresponding to $SF = 1.00$; plastic sandy till (about 1 m thick top layer); then hard-plastic sandy till.

The position of the groundwater table for variants 1–4 (Table 2) was defined on the basis of field piezometric measurements (scenario A) and the hydroisohypse pattern presented in the hydrogeological map of Poland [25,26] (scenario B).

Table 2. Groundwater-table elevations adopted as hydrostatic boundary conditions in the calculation variants of the numerical models (m a.s.l.).

Variant	Lateral Part of the Frontal Dam		Channel Embankment	
	Left Edge	Right Edge	Left Edge	Right Edge
Field measurements (A)	61.60	50.26	68.91	68.20
Hydrogeological map (B)	60.00	50.00	69.50	68.00

With reference to these elevations, the risk of static liquefaction of the sands present within the cross-sections was checked. The calculations showed a wide safety margin with respect to static liquefaction. However, this does not eliminate the risk associated with sudden and unfavourable weather conditions, or with accidental situations related to leakage from the reservoir or the derivation channel.

In variants 2 and 4, automated parametric calculations were performed, consisting of a stepwise reduction in the internal friction angle φ of the saturated medium sand until the limit value was reached. The reduction step was 1° , and the factor of safety was determined with an accuracy of 0.05.

The soil parameters (Table 3) adopted for the calculations (moist medium sand, saturated medium sand, clayey sand, hard-plastic sandy till, and plastic sandy till) were estimated based on the regional literature [35,36], consistent with the archival materials and the geological cross-sections. An ideally elastic, perfectly plastic model with the Mohr–Coulomb failure criterion was assumed for all soil layers.

Table 3. Summary of soil parameters adopted in the numerical stability analyses.

Soil Parameter	Medium Sand (Moist)	Medium Sand (Saturated)	Clayey Sand	Sandy Till (Hard-Plastic)	Sandy Till (Plastic)
Deformation modulus, E (MPa)	87.0	87.0	20.6	38.6	24.6
Poisson's ratio, ν (–)	0.25	0.25	0.30	0.25	0.25
Specific gravity, γ_s (kN/m ³)	26.5	26.5	26.5	26.7	26.7
Bulk unit weight, γ (kN/m ³)	18.5	20.0	21.5	21.5	20.5
Natural water content, w_n (%)	14.0	22.0	13.0	16.0	21.0
Hydraulic conductivity, k (m/d)	1.2×10^{-1}	1.2×10^{-1}	1.2×10^{-4}	1.2×10^{-4}	1.2×10^{-4}
Cohesion, c (kPa)	0.0	0.0	17.0	39.3	31.3
Internal friction angle, φ (°)	33.3	33.3	14.8	21.5	18.1
Dilatancy angle, ψ (°)	3.30	3.30	1.85	2.70	2.30
Earth-pressure coefficient at rest, K_0 (–)	0.45	0.45	0.75	0.63	0.69
Relative density, I_D (–)	0.55	0.55	–	–	–
Liquidity index, I_L (–)	–	–	0.20	0.20	0.40

6. Stability Analysis Results

The results of the stability calculations were examined both quantitatively (by listing the obtained factors of safety) and qualitatively (by considering the location, extent and shape of the potential slip surfaces bounding the possible surface mass movements). Within the calculation variants, the possible states of the top layer of sandy till and the influence of different values of the internal friction angle of the saturated medium sand

on the factor of safety were examined. Both the quantitative results and their qualitative analysis made it possible to assess the safety margin with respect to the activation of potential landslides and to identify the most vulnerable areas. For clarity, the results are presented in the same order for both structures: Section 6.1 concerns the lateral part of the frontal dam area and Section 6.2 concerns the derivation-channel embankment, and within each of them the variants 1–4 are discussed one by one.

6.1. Lateral Part of the Frontal Dam of the Dychów Pumped-Storage Power Plant

The position of the groundwater table for variants 1–4 in the analysed cross-section of the lateral part of the frontal dam area is shown in Figure 8. The differences are very small between scenarios A and B (Table 2).

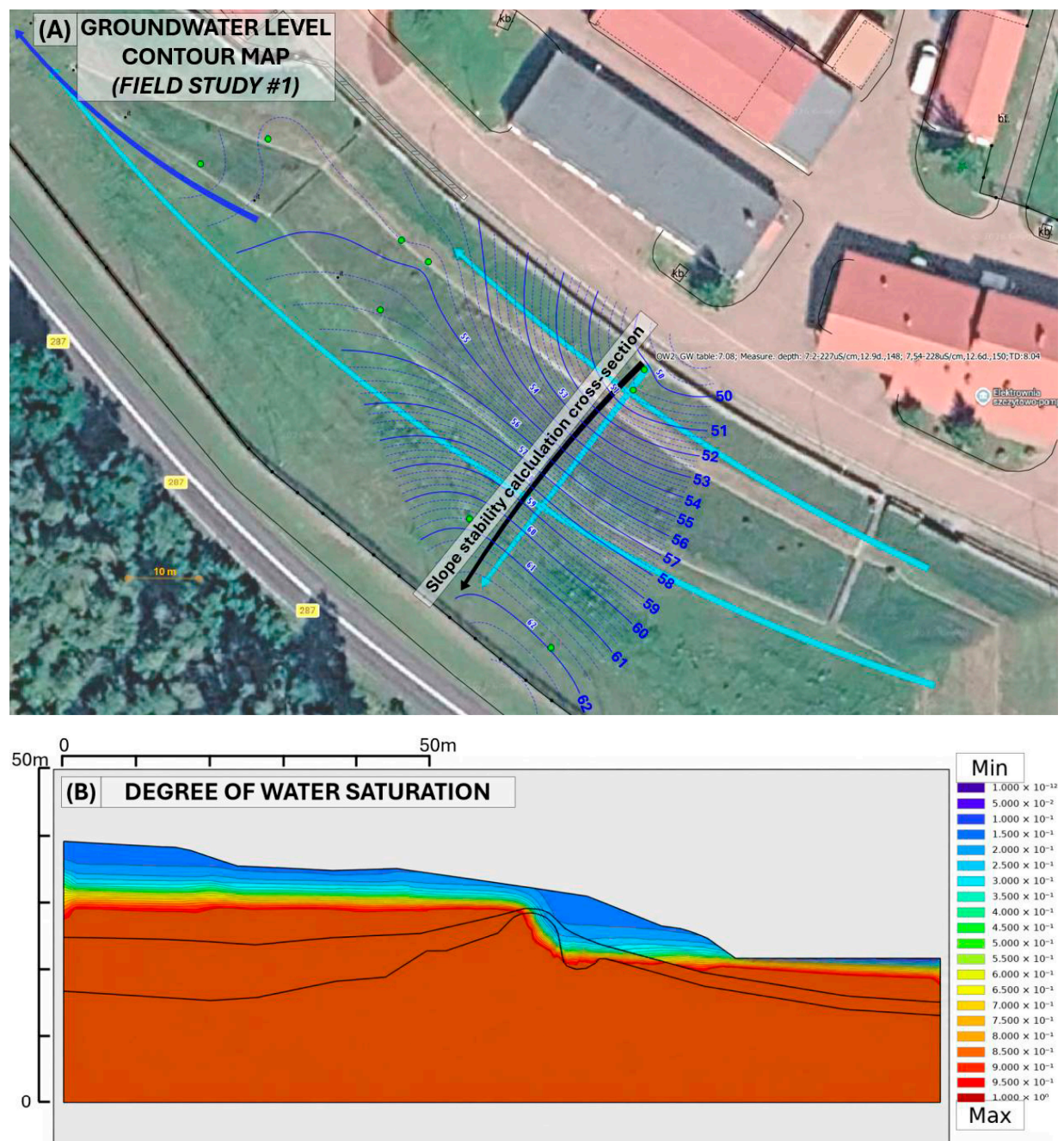


Figure 8. Groundwater conditions in the vicinity of the front dam: (A) groundwater-table contour map (arrows indicate ERT profiles); (B) degree of saturation of the subsoil for Scenario A, charac-

terised by a slightly higher hydraulic gradient than Scenario B (brown indicates fully saturated soil below the groundwater table).

In the first case (scenario A, Figure 8), the groundwater table adopted in the model was documented by observations in open piezometers. At the location of the largest glaciotectionic disturbance, a significant difference in groundwater levels is visible. Such a situation impedes groundwater flow and, during intense rainfall, may lead to a sudden increase in filtration under a high hydraulic gradient. As a consequence, temporary internal erosion could be initiated, which might in turn loosen the non-cohesive soil skeleton. Confirming such a scenario would require longer and more detailed observations of the groundwater table as well as granulometric analyses of the soils.

The second case (scenario B) differs from the first (scenario A), in that the groundwater table is lower by 1.6 m at the left edge of the model, with a proportional reduction in the hydraulic gradient. This scenario corresponds to the documented hydroisohypse pattern presented in the hydrogeological map of Poland. The resulting lowering of the groundwater level is not expected to have a negative impact on the safety of the dam with respect to landslide hazard. The stability calculations showed that the 1.6 m lowering of the groundwater table at the left edge of the model has practically no influence on the SF or on the shape of the slip surface; the only effect was a reduction in the limit internal friction angle of the sands by about 1° .

The potential slip surfaces on the lateral part of the frontal dam area, for the different variants of soil parameters (1–4) and groundwater positions (A, B), are shown in Figure 9.

In variant 1 (Figure 9A) the factor of safety equals 1.75. No significant influence of groundwater is observed, which is related to the location of the potential landslide. The calculations indicate a near-surface zone (with a shape close to a slide, Figure 9A, (Var. 1)) as the hypothetical zone of deformation development, lying entirely above the groundwater table (Figure 8A). The main destabilising factor is the inclination of the lower part of the downstream slope of the lateral part of the frontal dam are. The slip surface is about 10 m long, covering only the steepest portion of the lower slope and a layer of moist medium sand no more than a few metres deep.

In variant 2 scenario A (Figure 9A, (Var. 2)), the safety factor $SF \approx 1$ for the downstream slope of the lateral part of the frontal dam was obtained through stepwise reduction in the internal friction angle of the saturated medium sand down to $\varphi = 12.3^\circ$. With the groundwater table included in the model, the limit value still corresponds to $SF \approx 1$ —no “visible” (calculation step $\Delta SF = 0.05$) changes here, because the glaciotectionic impermeable tills uphill have an impact. The computed slip surfaces, regardless of the case, are very similar and take a circular-cylindrical shape. The extent of the theoretical landslide in this variant is about 32 m between the extreme points of the slip surface. This is larger than in variant 1 scenario A, and covers a wider part of the slope. In terms of depth, the slip surface reaches into the moist and saturated medium sand layers (the depth of the hypothetical landslide ranges from several to ten metres). In this case, the slip surface rests on the top of the cohesive soils (hard-plastic sandy till) that underlie the sands, which shows a significant influence of the state of this till, especially in the top zone, on the extent of the potential landslide.

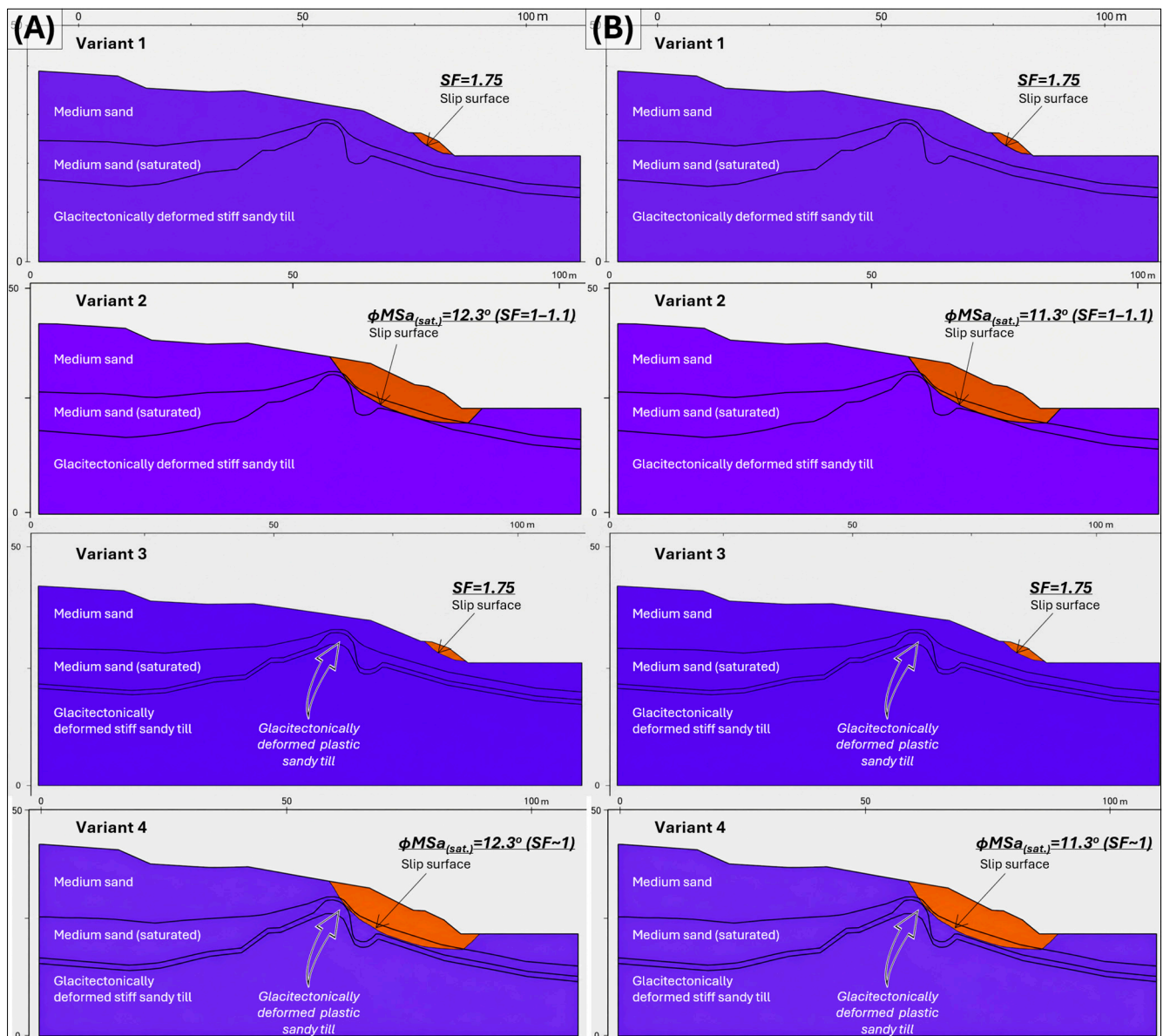


Figure 9. Potential slip surfaces of the hypothetical landslide on the lateral part of the frontal dam are (4 soil parameters variants): (A) scenario A—groundwater position based on field measurements; (B) scenario B—groundwater position based on the hydroisohypse pattern from the hydrogeological map of Poland.

In the scenario A; variants 3 and 4 differ from variants 1 and 2 by the presence of a layer of plastic sandy till in the top zone of the underlying hard-plastic moraine till. For variant 3, because the computed slip surface lies close to the ground surface, changes in the soil state at greater depth did not cause any noticeable change either in the factor of safety or in the geometry of the slip curve (Figure 9A, (Var. 3)). For variant 4 (Figure 9A, (Var. 4)), the limit value $\varphi = 12.3^\circ$ was again obtained for $SF \approx 1$; the mobilised soil masses again consisted mainly of sands resting on the till. It can therefore be stated that a reduction in cohesion and the internal friction angle by approximately 20% (Table 3), corresponding to a simulated change in the till state from hard-plastic to plastic, is not significant and does not produce noticeable qualitative differences with respect to variants 1 and 2.

The results for variant 1 scenario B are identical to those of variant 1 scenario A ($SF = 1.75$, and the slip surface is in the same position). In this case, as well as in variant 3B ($SF = 1.75$), no influence of the changes in the calculation scheme on the stability was observed.

This is due to the clear dominance of a potential slip surface located in the near-surface zone and to the lack of direct interaction between groundwater and this part of the structure.

In the remaining variants of the scenario B (2 and 4) the slip surfaces lie below the groundwater table. Consequently, the lowering of the groundwater level led to an even larger safety margin, expressed by a limit value $\varphi = 11.3^\circ$ (for $SF \approx 1$).

6.2. Derivation-Channel Embankment

The position of the groundwater table for variants 1–4 in the analysed cross-section of the derivation-channel embankment is shown in Figure 10A (scenario A) and Figure 10B (scenario B).

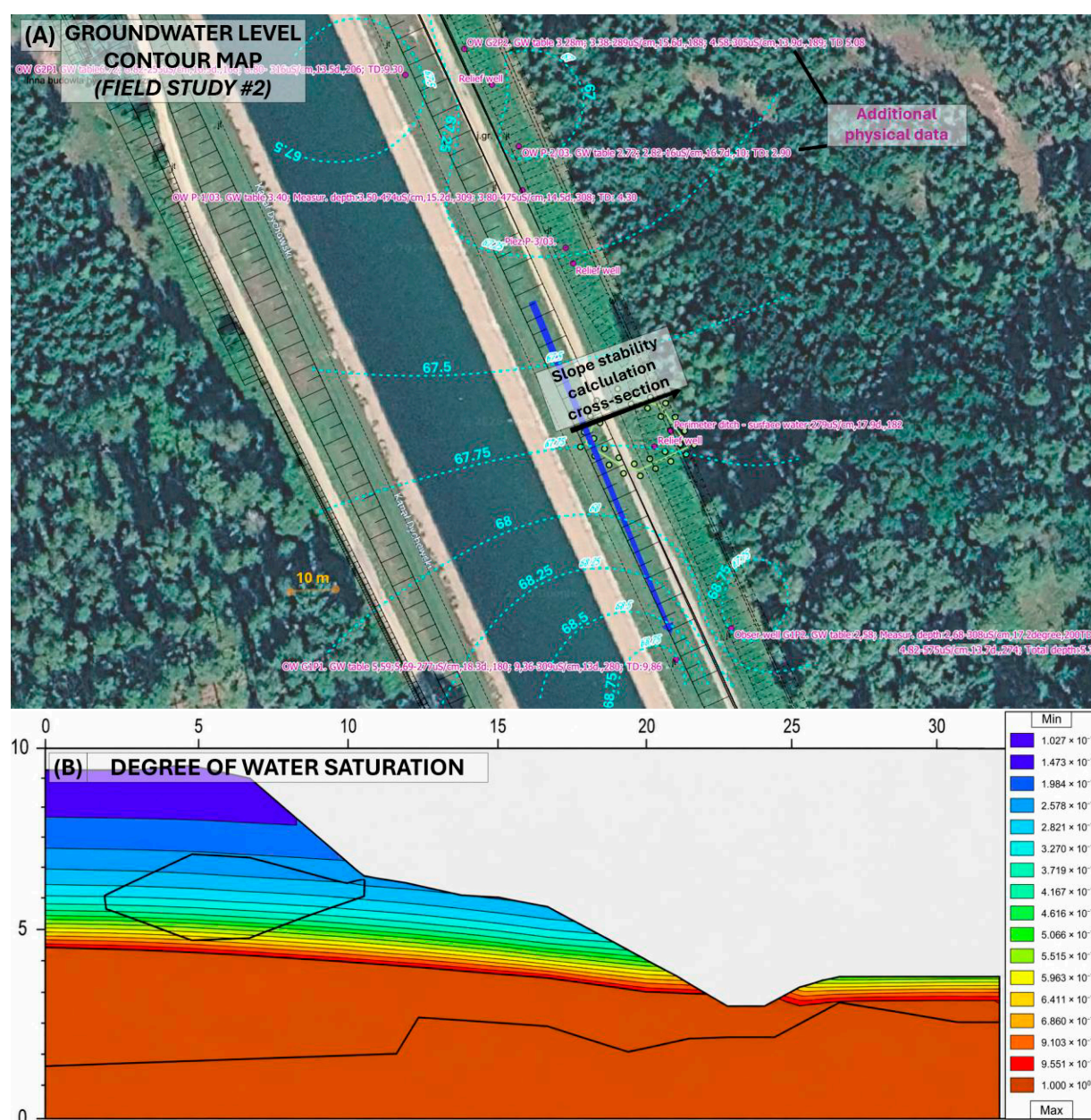


Figure 10. Groundwater conditions in the vicinity of the embankment slope: (A) groundwater-table contour map (arrows indicate ERT profiles); (B) degree of saturation of the subsoil for Scenario B, characterised by a slightly higher hydraulic gradient than Scenario A (brown indicates fully saturated soil below the groundwater table).

As in the case of the lateral part of the frontal dam are, the documented groundwater table in the analysed cross-section of the channel embankment lies below the base of the embankment.

The saturation boundary in scenario A (Figure 10A) has a small gradient (the difference in pressure head between the model edges equals 0.7 m), and a well-functioning drain is located at the foot of the embankment slope. Figure 10A shows the depression curve for scenario A (variants 1–4).

In the second calculation scenario (B) the hydraulic gradient was increased (the difference in pressure head between the side edges of the model was raised from 0.7 m to 1.5 m). Figure 10B shows the groundwater-table position for scenario B (variants 1–4).

The potential slip surfaces in the channel embankment, for the different variants of material parameters (1–4) and groundwater positions (A, B), are presented in Figure 11.

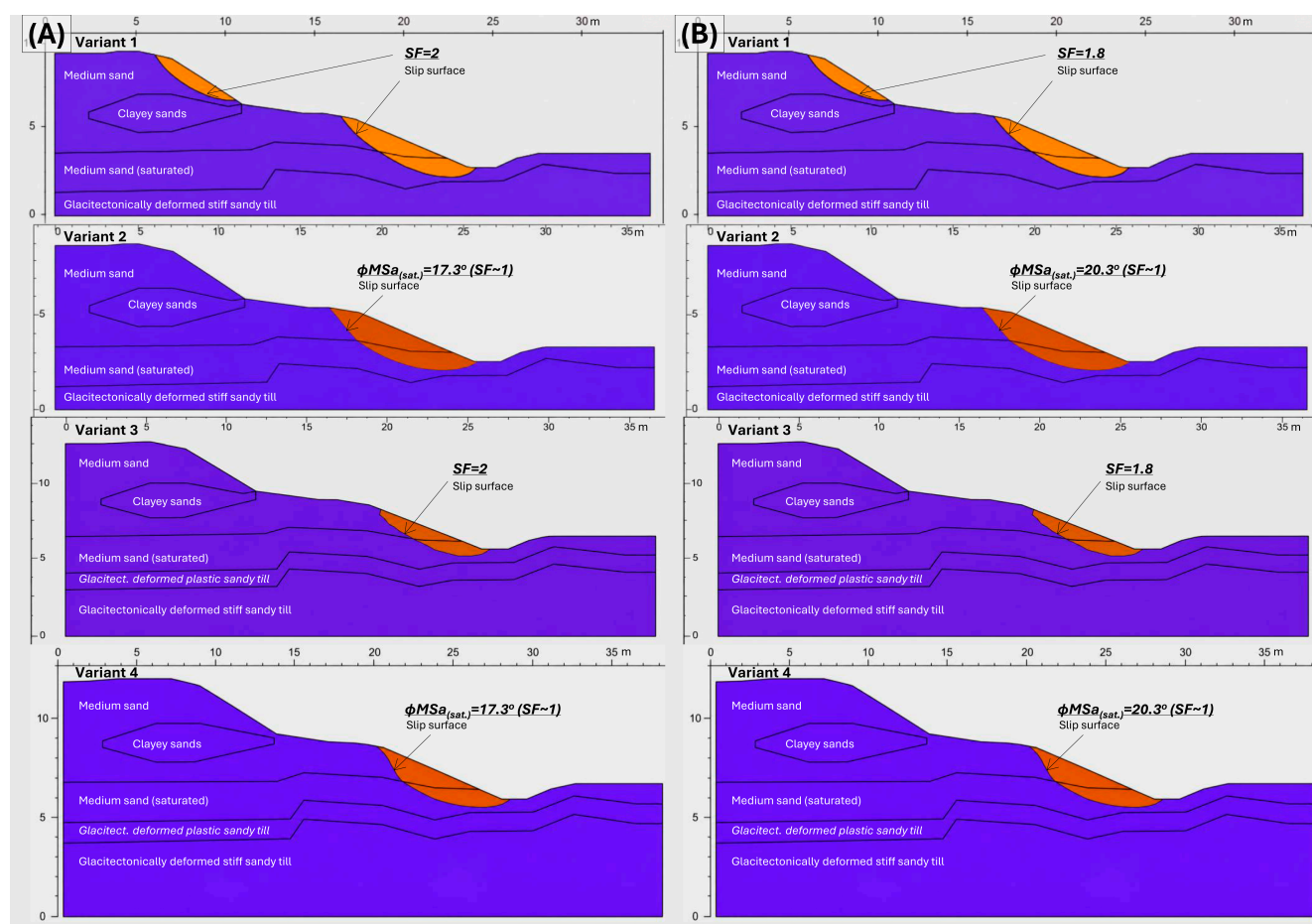


Figure 11. Potential slip surfaces of the hypothetical landslide on the embankment slope: (A) scenario A—groundwater position based on field measurements; (B) scenario B—groundwater position based on the hydroisohypse pattern from the hydrogeological map of Poland.

In the case without groundwater, with a shallow slip surface, the factor of safety does not change (Figure 11A, (Var. 1)). The landslide strip is about 6 m wide and would be located in the upper part of the slope, covering only the layer of moist medium sand.

Under realistic conditions, i.e., with groundwater present—even below the base of the embankment (Figure 11A, (Var. 1))—the safety level of the analysed slope varied depending on the assumptions of each variant.

In variant 1 scenario A the factor of safety decreased slightly, to a value of 2.0. The extent of the landsliding processes also changed: two potential landslides may develop (Figure 11A, (Var. 1)). The first one, as in the case without groundwater, is located in the

upper part of the body. The second one, in the lower part, is about 8 m wide and involves the layer of saturated medium sand. Both slide bodies are of the surface-mass-movement type, with a depth not exceeding a few metres. Although the “global” safety factors are similar, the potential landslide in the lower part of the slope should be considered more prone to activation. This is mainly due to the presence of the nearby service road designed for heavy vehicles, which may act as an additional destabilising factor of dynamic character directly affecting this lower part of the body (Figure 11A, (Var. 1)). It should be noted that for such shallow and relatively short slip surfaces, dynamic loading from heavy traffic may change the ratio between driving and resisting forces considerably more than for long and deep slip surfaces; therefore, in a more detailed design-level analysis, an additional road load should be applied to the slope in accordance with the road-engineering standard. Nevertheless, the obtained safety factor (2.0) still indicates a considerable reserve of capacity.

In the parametric analysis (variant 2 scenario A), in which the internal friction angle of the saturated medium sand was gradually reduced to reach the limit value $SF = 1.00$, a limit angle $\varphi = 17.3^\circ$ was obtained. This limit value corresponds to the case with groundwater included in the model.

When the groundwater is removed from the model while keeping $\varphi = 17.3^\circ$, the safety factor rises to 1.55, consistent with the fact that, in non-cohesive soils, the presence of water reduces the overall shear capacity of the slope. The slip surfaces in variant 2 scenario A are located similarly to those in the previously described variants. However, both with and without groundwater, a single slip surface is obtained in the lower part of the slope (Figure 11A, (Var. 2)). This surface cuts off a wedge about 8 m wide and no more than a few metres deep. This results from the degradation of the strength parameters of the saturated medium sand in the lower part of the slope.

The following variants in the scenario A (3 and 4) may illustrate the consequences of long-term groundwater action on the top of the moraine hard-plastic sandy till (Figure 11A, (Var. 3 and 4)). The top zone was modelled here as a layer of plastic sandy till. The calculations yielded factors of safety consistent with those of variants 1 and 2 (scenario A), which can be explained by the location of the potential slip surfaces above the top of the hard-plastic and plastic till layers. Despite the weakening introduced in the top of the till, no lowering of the slip surfaces—and therefore no deepening of the landslides—is observed. Although the factors of safety are similar, the qualitative assessment of variants 3 and 1 (scenario A) reveals meaningful differences. Without groundwater, the upper part of the body is more endangered, whereas with groundwater the lower part is (Figure 11A, (Var. 3 and 4)). In variant 4 (scenario A) the qualitative pattern remains the same as in variant 2 scenario A (only one slip surface in the lower part of the embankment).

Changes in the saturated zone in the model of the channel embankment for variants 1 and 3 (scenario B) led to quantitative changes in the results with respect to, respectively, variants 1 and 3 scenario A. In variants 1 and 3 of the scenario B the factor of safety with groundwater present decreased from 2.00 to 1.80, while without groundwater it remained the same (2.10).

Similar relationships were found for the remaining B variants compared with the first group (A variants). Despite the quantitative differences, no significant qualitative changes were observed. In variants 2 and 4 (scenario B, the step-wise reduction in the internal friction angle of the saturated medium sand led to a limit value $\varphi = 20.3^\circ$ (at $SF = 1.00$). Without groundwater, the factor of safety with $\varphi = 20.3^\circ$ equals 1.75.

7. Discussion

7.1. Interpretation of the Stability Results

The results show that, despite the complex glaciotectionic history of the cohesive soils in the area, both analysed structures preserve a safety factor well above the required value $SF = 1.50$ [12]. For the lateral part of the frontal dam area, $SF = 1.75$ in all considered variants, and for the channel embankment the values range from 1.80 to 2.10. This is in line with the general observation that ageing earthen hydraulic structures built on glacial and glaciotectionically disturbed subsoil in western Poland may still offer a sufficient safety reserve, provided that seepage control and drainage remain functional [10,13].

A parametric reduction in the internal friction angle of the saturated medium sand provided useful information on the robustness of the structures with respect to progressive weakening of the non-cohesive body soils. The limit values $\varphi = 12.3^\circ$ (dam) and $\varphi = 20.3^\circ$ (embankment) are clearly below the initial value $\varphi = 33.3^\circ$ adopted for medium sands (Table 2). Reaching such low values would require a drastic structural degradation of the sand skeleton, most probably related to sustained internal erosion. This observation is consistent with the 1997 event on the dam, which was attributed to long-term filtration processes [10,11], and underlines the need for continuous piezometric monitoring.

The logical consistency of the parametric results should also be emphasised. In variant 2 scenario A of the dam, the limit angle $\varphi = 12.3^\circ$ corresponds to $SF \approx 1$ with groundwater included in the model; in the same configuration without groundwater SF is higher (10%; a low increase related to the low position of the groundwater table). Similarly, in variant 2 scenario A of the embankment, the limit angle $\varphi = 17.3^\circ$ corresponds to $SF \approx 1$ with groundwater, whereas without groundwater $SF = 1.55$. The difference between the slope with and without groundwater is related to the level of the water table and to the localisation of the slip surface. Both outcomes agree with the general behaviour of non-cohesive soils, where the presence of water reduces the overall slope capacity.

With regard to the qualitative analysis of the channel-embankment results, attention should be drawn to the potential landslide in the lower part of the slope (variant 1 scenario A). Although its “global” safety factor ($SF = 2.0$) is high, the vicinity of the service road for heavy vehicles introduces additional dynamic loading, which—for shallow and short slip surfaces—may have a non-negligible impact on the ratio between driving and resisting forces. A more detailed, design-level analysis including a heavy road load, in accordance with the Polish road-engineering standard, is therefore recommended as a complementary step.

7.2. Dimensions of Ageing Captured by the Study

Three ageing dimensions are covered. The first is the operational age itself: the structures have been in service since the 1930s and have a documented failure history, which is the practical reason for their reassessment against current requirements. The second is the present state of the structures: the resistivity cross-sections image the cumulative effect of about ninety years of operation, including the loosened, water-saturated sandy zones and the disturbed top of the cohesive soils, so the ERT results describe the state reached by ageing rather than a design condition. The third is the remaining margin: the parametric reduction in the friction angle shows how much further strength degradation, for example, due to internal erosion, the structures could still accept before the safety factor drops to 1.00. What the study does not capture is the kinetics of ageing. No time-dependent increase in permeability and no long-term creep of the soils are simulated; the reduction in the friction angle is a surrogate for the final effect of such processes, not their physical description. In the presented workflow, the progression of ageing is followed by observation instead: the fixed, GNSS-located ERT profiles can be re-measured and compared

over the years, together with the piezometric record. A physical model of the ageing processes themselves would require the coupled transient formulations and the longer observation series discussed later in this section.

7.3. Input Data and Model Limitations

The combination of ERT prospecting and FEM-based stability analysis confirms its usefulness for the periodic assessment of hydraulic earthen structures. ERT allowed the top of the cohesive layers to be traced in a non-invasive way and, in consequence, the preferential zones of a potential slip surface to be identified. It should, however, be remembered that resistivity values strongly depend on water saturation and on the silt and clay content, so the interpretation of ERT cross-sections requires supporting information from boreholes or archival documentation. In the present study, the geological cross-sections from the literature [23–27] provided such a reference.

The boundaries between the material zones used in the ZSoil models were not taken from the resistivity images alone. The top of the cohesive layer and the position of the saturated sands read from the ERT cross-sections were checked against the shallow reference boreholes made during the field reconnaissance and against the archival borehole logs and geological cross-sections [23–27]. Only the boundaries that were confirmed by at least one of these sources were transferred to the numerical model; the resistivity ranges given in Section 4 were used mainly to follow the same layers between the control points. This limits, but does not fully remove, the well-known ambiguity of the resistivity–lithology relation. A second limitation concerns the soil parameters. The mechanical parameters listed in Table 3 were not obtained from laboratory tests on undisturbed, site-specific samples; they were estimated from the national standard tables and the regional literature [35,36], in agreement with the archival materials and the geological cross-sections. This choice follows from the non-invasive character of the procedure: taking undisturbed samples from the body of an operating Class I dam would require deep drilling through the structure, which the presented workflow deliberately avoids. The adopted values should therefore be read as characteristic estimates rather than as measured properties of the structure, and this uncertainty is one of the reasons why the analysis was organised as a set of variants: the two groundwater scenarios and the parametric reduction in the friction angle show directly how sensitive the safety factor is to the assumed strength values. The limit friction angles obtained in this way lie clearly below realistic values for medium sands, so the overall conclusion is not sensitive to the exact parameter choice. For a design-level verification, however, the parameters should be confirmed by laboratory mechanical testing of undisturbed samples taken from the dam body and its subsoil. A third limitation concerns the constitutive model. An elastic, perfectly plastic Mohr–Coulomb model with a steady-state groundwater table was used for all soil layers, without strain softening and without transient seepage. Such a model is sufficient for a screening of the safety reserve and for the comparison of the variants, which is the aim of this study, but it does not reproduce the progressive, time-dependent loss of strength that is expected during long-lasting internal erosion. A prediction of the actual development of a failure in the dam body would require a more advanced model, for example with strain softening, with soil creep or with coupled transient flow, supported by a longer series of piezometric and geodetic observations.

7.4. Implications for the Operation of the Plant and the Power System

From the perspective of electric-power engineering, the results presented here go beyond a purely geotechnical case. ESP Dychów, with 88 MW of pumping–generating capacity, belongs to a limited set of fast-responding assets on the western flank of the Polish transmission system. A forced outage caused by damage to the lateral part of the frontal

dam area or to the derivation-channel embankment would remove this capacity from the pool of grid-balancing reserves and from the black-start portfolio of the national TSO (Transmission System Operator). The historical landslide on the frontal dam destroyed communication infrastructure and affected the powerhouse [11], which shows that a single event localised in the earthen structures may translate directly into a measurable loss of energy and capacity availability. The factor of safety well above 1.50 obtained in the analyses is therefore not a purely geotechnical statement. It is also a quantitative argument for keeping Dychów in the set of dispatchable resources that stabilise frequency and voltage in the synchronous area, in particular under the growing share of variable wind and photovoltaic generation in the Polish mix [3,4,6]. The economic dimension of this conclusion is directly relevant in the context of the Polish hydropower support schemes discussed in [5]. In this sense, the presented workflow links the periodic safety reassessment of earthen hydraulic structures directly to the operational availability of pumped-storage capacity.

7.5. Transferability of the Workflow

Analogous ageing conditions characterise a large share of the European pumped-storage fleet, including plants in Germany, Austria and the Czech Republic that were commissioned in the same decades as Dychów and operate on glaciotectionally disturbed or moraine subsoils [1,2]. Many of these twentieth-century hydroelectric schemes are today discussed also as industrial-heritage assets whose operating life is approaching its original design horizon [7], which gives an additional argument for non-invasive reassessment methods. Recent Polish case studies such as the Jeziorsko reservoir on the Warta river [3,4] and, more broadly those from the Polish hydropower sector [5,6] point to a similar conclusion: the resilience of hydro assets to changing operating conditions depends not only on hydrology and on turbine characteristics, but equally on the condition of their civil structures. The ERT (with GNSS localization) plus FEM approach presented here is by construction transferable—it does not rely on dedicated instrumentation of the structure and can be repeated at regular intervals with the same set of profiles, which makes it particularly suitable for operators who manage several ageing hydro structures of Class I hydraulic-engineering rank. Repeated in a fixed two-year cycle, the same set of profiles also gives a monitoring-like signal of slow changes in the structure, which is normally available only from permanent instrumentation; this is the practical advantage that makes the procedure attractive at the scale of a whole ageing fleet rather than for a single dam.

8. Conclusions

On the basis of the study, the following conclusions can be drawn:

1. Despite the complex glaciotectionic history of the cohesive soil layers, both analysed structures are stable and exhibit a considerable safety reserve. The “global” safety factor equals 1.75 for the dam and 1.80–2.10 for the channel embankment, i.e., markedly higher than the required value of 1.50.
2. The potential presence of zones of reduced strength within the range of fluctuations of the groundwater table (e.g., loosened zones or locally higher plasticity of the cohesive soils) should not endanger the equilibrium of the analysed objects. The hypothetical slip surfaces were local and involved only a minor portion of the calculation cross-sections. The depth extent of the eventual landslides ranged from a few to ten metres, and their shape did not differ significantly from circular-cylindrical, although slide-type movements along shallower surfaces cannot be ruled out.
3. In the case of the lateral part of the frontal dam area, a sudden and prolonged rise in the groundwater table (e.g., as a consequence of a failure of the anti-filtration protection or of the drainage system) could adversely affect the stability of the structure.

For both analysed objects, continuous piezometric monitoring, combined with control geodetic measurements and inclinometer observations, is therefore recommended.

4. For the lower, potentially more active part of the channel-embankment slope (variant 1A), a complementary analysis including a dynamic road load in accordance with the road-engineering standard is recommended. For shallow and short slip surfaces, such loading may alter the ratio between driving and resisting forces more substantially than for deep and long surfaces.
5. Electrical resistivity tomography (ERT) is becoming one of the key and increasingly widely applied geophysical methods. It constitutes an effective tool supporting both the interpretation of traditional subsoil investigations and the identification of local anomalies that may affect design solutions, operational decisions or the interpretation of monitoring data—especially for hydraulic structures. Owing to its non-invasive character, efficiency and low cost, ERT still offers unused potential for improving the reliability of such structures, reducing maintenance costs and effectively counteracting the effects of ageing and wear.

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