



Research paper

Optimization of technological measures increasing the safety of the Żelazny Most tailings pond dams with the combined use of monitoring results and advanced computational models

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Abstract: The paper presents the approach for optimization of preventive/technological measures increasing the safety of tailings pond dams. It is based on the combined use of monitoring results as well as advanced 3D finite element (FE) modeling. Under consideration was the eastern dam of Żelazny Most Tailings Storage Facility (TSF). As part of the work, four numerical models of the dam and the subsoil, differing in the spatial arrangement of the soil layers, were created. For this purpose, the kriging technique was used. The numerical models were calibrated against the measurements from the monitoring system. In particular the readings acquired from benchmarks, piezometers and inclinometers were used. The optimization of preventive measures was performed for the model that showed the best general fit to the monitoring data. The spatial distribution and installation time of relief wells were both optimized. It was shown that the optimized system of relief wells provides the required safety margin.

Keywords: dam, FE modeling, hydrotechnics, kriging, relief wells

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1. Introduction

The main purpose of constructing tailings pond facilities is to enable the storage of post-mining waste. Ensuring the safe use of such facilities is important [5, 17, 24] due to the continuity of the production process, but most of all the lives of people, both miners and local residents, as well as from the point of view of natural environment protection [26]. Numerous failures of such facilities have taken place all over the world in recent decades causing large scale damages [1, 14]. As stated by Rico et al. [18, 19], failures on facilities in operation occur much more frequently than on “inactive” ones. All this implies the necessity of large investments in ensuring the safety of these facilities. This can be carried out through appropriate designs, geotechnical investigation, numerical analyzes and finally continuous monitoring of various parameters, e.g. benchmarks displacements, in-depth displacements in inclinometers or water head levels in piezometers [25].

A certain difficulty in ensuring the safety of these structures is that, unlike typical reservoirs, these structures are constantly being expanded by raising their dams. It triggers that their safety factor gradually decreases as the elevation of the dam crest rises [5, 11, 16]. Moreover, it is necessary to take into account the non-stationary state of filtration, in particular pore pressure distribution within the sediments, dam body and the subsoil [8, 10, 20]. The stability is strongly affected by the pore pressure distribution [12], in particular, the water table localization in the ground [4, 21, 28].

The paper considers the possibility of the safe construction of dams of the Żelazny Most TSF up to an elevation of 205 m a.s.l. This object is one of the largest facilities of this type in the world and, at the same time, it is a crucial element in the technological line of copper production in KGHM Polska Miedź S.A. Due to the necessity of ensuring the required safety margin of the dam, preventive/technological measures have been applied on the individual areas/sections of facility, including: construction of loading berms, installation of relief wells system or movement of the dam crest towards the center of the pond.

In this work, the possibility of further safe expansion of the dam is verified. In particular, a comprehensive approach for the optimization of preventive measures, in the form of relief wells, is provided. The aim of the optimization is to find the solution for the system of relief wells enabling the safe construction of the dam up to the elevation of 205 m a.s.l. As a result, the following characteristics of the system are to be determined: the number of relief well, their spatial arrangement, installation time of relief wells as well as the depth of the filter bottom. Analyses are performed using hybrid approach, namely, the one which combines monitoring results and advanced numerical computations. The former are used to calibrate 3D FE computational model. For that purpose the measurements from 11 benchmarks, 24 piezometers and 11 inclinometers are utilized. Moreover the calibration process involves also differentiation in the spatial arrangement of soil layers in the dam subsoil. Reconstructions of subsoils are created with the use of kriging technique [9, 15, 27]. The condition for the safe raise of the dam is adopted according to the national regulations, i.e., the factor of safety (FOS) should be greater than or equal to the minimum one (FOS_{min}) equated to 1.5. The conducted numerous simulations confirmed the possibility of expanding the reservoir to the required elevation with the use of the relief wells only.

The paper is organized as follows. In the next section a brief characteristic of the Żelazny Most TSF is provided. This includes: technical parameters of the reservoir, description of preventive measures applied in the facility and geological conditions of the subsoil. Calibration of the numerical model against monitoring results is presented in Section 3. The methodology and results of the optimization of preventive measures, i.e. the relief wells system, is discussed in Section 4. Final conclusions end the paper.

2. Brief characteristic of the facility and the subsoil

2.1. Technical conditions of the Żelazny Most TSF

The Żelazny Most TSF is a key element in the technological line of copper production in KGHM Polska Miedź S.A. (Fig. 1). The copper flotation waste from the Ore Concentration Plant are stored in the facility. Its construction began in 1974. The exploitation resulting in continuous increase of the height of the dams has been ongoing since 1977 until now. The average annual height increment ranges from 1.2 to 1.5 m. Currently (September 2020), the facility is characterized by technical parameters listed in Table 1.

Table 1. Technical parameters of TSF Żelazny Most (last update in September 2020)

| Parameter | Value | Unit |
|--|-----------|--------------------|
| The volume according to the damming curve | 660.0 | mln m ³ |
| Water volume | 8 | mln m ³ |
| Tailings volume | 652 | mln m ³ |
| Pond water elevation | 184.50 | m a.s.l. |
| The crest elevation | 187.50 | m a.s.l. |
| The area of the TSF Żelazny Most (Main Object) with infrastructure | ~1580 | ha |
| The area on the inner side of the crest | 1180 | ha |
| The reservoir area | 560 | ha |
| Beach area | 620 | ha |
| The dam length in axis | 14330 | m |
| The height of the dams | 40.0–73.0 | m |

Analyses conducted in the paper concern eastern dam, in particular sections E1 and E2. The partition of the dam perimeter into technical sections is presented in Fig. 2. Area under study is marked with green rectangle.



Fig. 1. The Żelazny Most Tailings Storage Facility

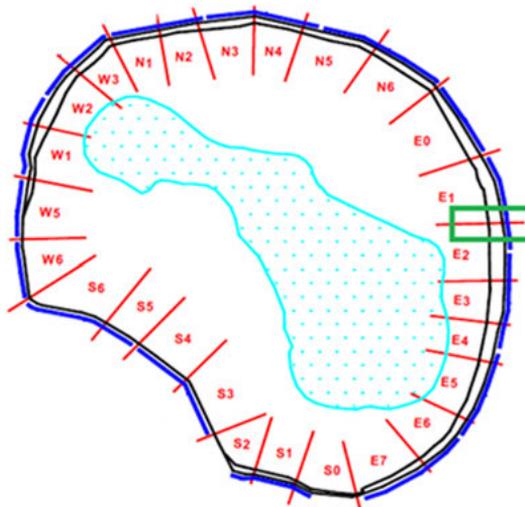


Fig. 2. Technological section of the Żelazny Most dam

2.2. Preventive measures applied in the facility

In recent years, the occurrence of increased horizontal and vertical deformations has been observed in the facility. In order to restrain them and improve the dam stability, three types of technical/preventive measures have been implemented, i.e.:

- construction of loading berms,
- installation of relief wells,
- moving the dam crest towards the center of the pond.

In the area of sections E1 and E2, three stages of loading berm have been planned. So far, stages I (2009) and II (2017) have been completed. Stage III is additionally divided into

three phases. First of them is still in progress. Results of the monitoring, carried out after the execution of the berm of stage I and II, indicated the reduction of the rate of displacement. This confirms the effectiveness of such solution in selected areas of the facility. Figs. 3 and 4 present the stages of loading berm construction in the considered area of E1 and E2 sections.

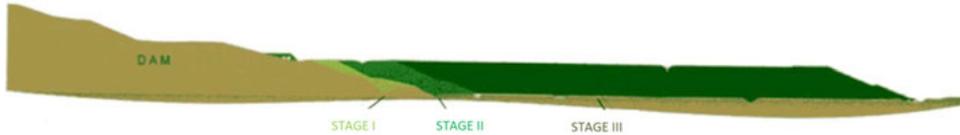


Fig. 3. Construction stages of loading berm on the eastern dam – cross sectional view

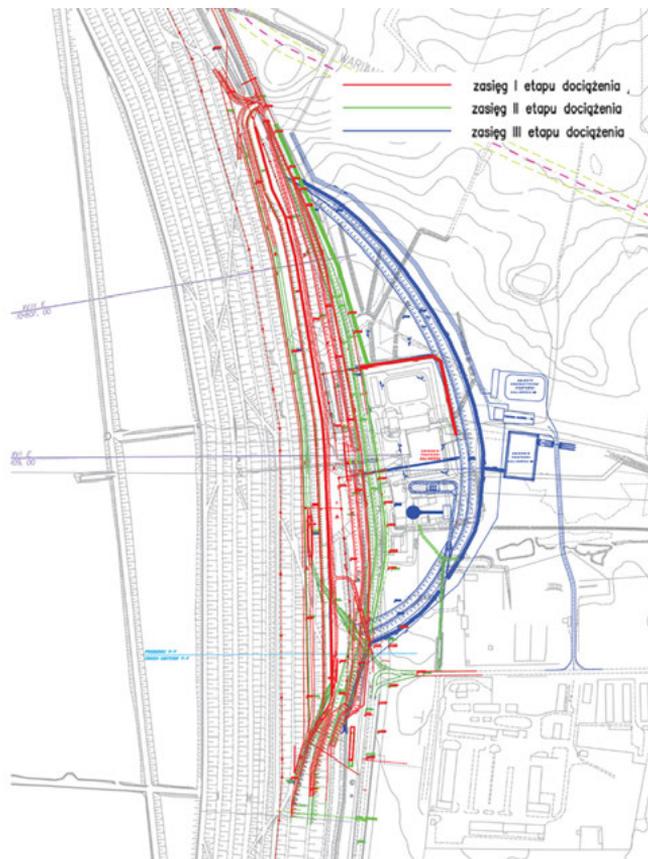


Fig. 4. Construction stages of loading berm on the eastern dam – top view

In order to mitigate the phenomenon of increasing pore pressure, a system of drainage wells (relief wells) was installed at sections E1 and E2 of the facility as the dam super-

structure progressed. There are 24 relief wells within the area under consideration. Four of them are permanently disabled. The wells are presented in Fig. 5 where they have been divided into two groups: group I (marked in blue) includes wells installed before 2011, while group II (marked in violet) includes wells installed in 2016. As a consequence of relief wells installation a dissipation of pore pressure was observed as the lowering of the water level in piezometers.

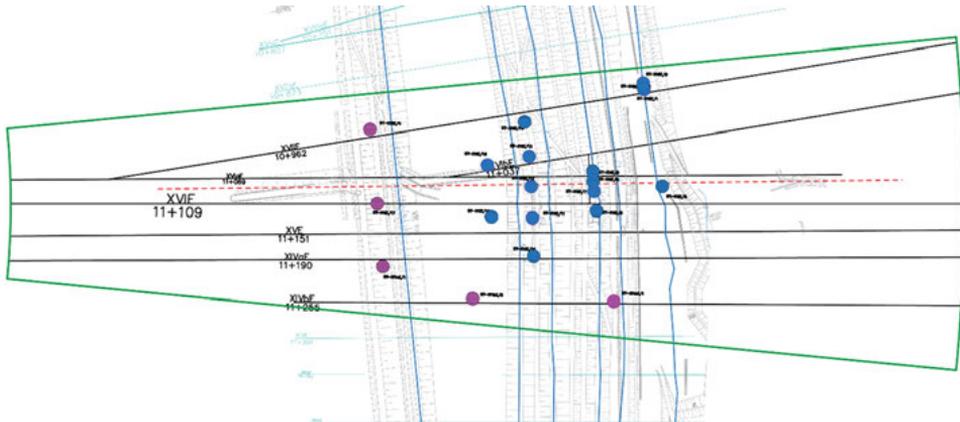


Fig. 5. Location of relief wells (in operation) in eastern dam

2.3. Geological conditions of the subsoil

The Źelazny Most TSF is located in south-western Poland, near Polkowice, Lubin and Głogów towns, approximately 80 kilometers to the west from Wrocław. An important role in the selection of the location was the possibility of using the natural depression of the terrain (the valley of the Kalinówka River) in the south-eastern region of the Dalkowskie hills being a Permian-Mesozoic structural unit as a part of the Fore Sudetic Monocline. The subsoil of the facility was formed as a result of the South Polish and Middle Polish glaciations. The geological structure should be considered as complicated due to the glaciotectionic and periglacial processes that occurred in this area. This, in particular, led to a significant disturbance of the arrangement of geological layers and their deformation. The detailed description of geological conditions is presented in the paper of Jamiolkowski [7] and in the previous work of Łydźba et al. [11].

Based on the extensive geological documentation (developed over many years), a total of 12 geotechnical layers of the subsoil were distinguished in the area under consideration. i.e.: 7n, 8n, 9n, 10n, 12n, 13n, 14n, 15n, 16n, 17n, 18n and 19n, as well as anthropogenic layers: 1, 2, 3, 4, and 5.

Layer 7n includes Quaternary clays and solifluction clays. Layer 8n is formed by cohesive soils of fluvial or fluvio-glacial origin. The 9n and 10n layers were defined as Neogene clays, where 10n corresponds to the soils with weakened strength parameters in the

areas of the concentrated shear zones. Quaternary layers 12n and 13n are glaciolacustrine deposits and tills. The Neogene silty soils are described as a layer of 15n. Layers 16n and 17n are non-cohesive Quaternary fluviglacial sediments of various grain sizes. Neogene sands from fine and silty to coarse form the layers of 18n and 19n. Anthropogenic layers 1 and 2 denote dam superstructure. Layer 3 corresponds to a beach and layer 4 the tailings. Layer 5 is the loading berm. The geotechnical parameters (dry unit weight γ_d , unit weight at saturation γ_{sat} , angle of internal friction ϕ , cohesion c , dilation angle ψ , Young modulus E , Poisson ratio ν , horizontal and vertical hydraulic conductivities k_h and k_v , parameter α of van Genuchten model [23]) of the individual subsoil layers adopted for the numerical calculations are summarized in Table 2.

Table 2. Geotechnical parameters of the subsoil and anthropogenic layers

| | Parameter | | | | | | | | | |
|--------------|------------------------------------|--|---------------|--------------|---------------|--------------|--------------|---------------------|---------------------|-------------------|
| | γ_d [kN/m ³] | γ_{sat} [kN/m ³] | ϕ [°] | c [kPa] | ψ [°] | E [MPa] | ν [-] | k_h [m/s] | k_v [m/s] | α [1/m] |
| Dam (1) | 19.00 | 20.50 | 34 | 1 | 9 | 80 | 0.20 | $5 \cdot 10^{-4}$ | $5 \cdot 10^{-4}$ | 5 |
| Dam (2) | 19.00 | 20.50 | 34 | 1 | 9 | 80 | 0.20 | $1 \cdot 10^{-4}$ | $1 \cdot 10^{-4}$ | 5 |
| Beach (3) | 18.00 | 19.50 | 34 | 1 | 9 | 80 | 0.20 | $3.4 \cdot 10^{-8}$ | $1.1 \cdot 10^{-8}$ | 0.75 |
| Tailings (4) | 17.00 | 18.50 | 34 | 1 | 9 | 80 | 0.20 | $1 \cdot 10^{-11}$ | $1 \cdot 10^{-11}$ | 0.5 |
| Berm (5) | 21.00 | 22.00 | 35 | 1 | 10 | 80 | 0.20 | $5 \cdot 10^{-4}$ | $5 \cdot 10^{-4}$ | 5 |
| Layer 7n | 16.06 | 20.00 | 18 | 1 | 1.8 | 96 | 0.25 | $1 \cdot 10^{-11}$ | $1 \cdot 10^{-11}$ | 2 |
| Layer 8n | 16.06 | 20.00 | 12 | 1 | 1.2 | 100 | 0.30 | $5 \cdot 10^{-11}$ | $5 \cdot 10^{-12}$ | 0.1 |
| Layer 9n | 17.30 | 20.50 | 14.5 | 5 | 1.5 | 36 | 0.25 | $5 \cdot 10^{-11}$ | $5 \cdot 10^{-12}$ | 0.1 |
| Layer 10n | 16.80 | 20.00 | 10 | 1 | 1.0 | 36 | 0.25 | $1 \cdot 10^{-8}$ | $1 \cdot 10^{-8}$ | 2 |
| Layer 12n | 16.80 | 20.00 | 27 | 1 | 2.7 | 96 | 0.25 | $1 \cdot 10^{-8}$ | $1 \cdot 10^{-8}$ | 0.5 |
| Layer 13n | 17.67 | 21.00 | 28 | 2 | 2.8 | 96 | 0.25 | $1 \cdot 10^{-8}$ | $1 \cdot 10^{-8}$ | 0.5 |
| Layer 15n | 17.97 | 21.30 | 24 | 3 | 2.4 | 96 | 0.25 | $1 \cdot 10^{-4}$ | $1 \cdot 10^{-4}$ | 5 |
| Layer 16n | 16.87 | 20.50 | 36 | 1 | 11.0 | 240 | 0.20 | $1 \cdot 10^{-4}$ | $1 \cdot 10^{-4}$ | 5 |
| Layer 17n | 16.06 | 20.00 | 31 | 1 | 6.0 | 240 | 0.20 | $1 \cdot 10^{-4}$ | $1 \cdot 10^{-4}$ | 5 |
| Layer 18n | 15.86 | 19.80 | 30 | 1 | 5.0 | 240 | 0.20 | $1 \cdot 10^{-8}$ | $1 \cdot 10^{-8}$ | 2 |
| Layer 19n | 16.80 | 20.00 | 24 | 1 | 2.4 | 96 | 0.25 | $1 \cdot 10^{-11}$ | $1 \cdot 10^{-11}$ | 2 |

The data on the “dominant” layer 9n (fine-grained soils) in the subsoil come from in situ investigations reaching a depth of approx. 20 m below the ground surface. As an extrapolation for greater depth, it was assumed in the analyses that the deformation modulus of this layer is linearly increasing proportionally to the depth as shown in Fig. 6.

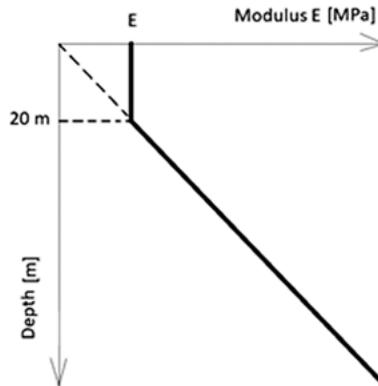


Fig. 6. Scheme of the increase of modulus E with depth for layer 9n

3. Calibration of numerical model

As mentioned before numerical calculations were carried out for the eastern dam in the area of the sections E1 and E2 (see Fig. 2). Furthermore, in Fig. 5 the boundary of the computational model is marked green. The computations were performed in the framework of FE analyses using ZSoil software [22]. The poro-mechanical coupling was adopted in the model. Constitutive relations in the unsaturated zone correspond to van Genuchten model [23]. Mohr-Coulomb failure criterion was assumed for the subsoil and sediments. The dam erection was simulated in the model as a staged, layer-by-layer construction. Due to the limited volume of the paper, the detailed formulation of the initial boundary value problem was omitted.

The aim of the calibration was to obtain the best possible compliance (fit) of the numerical simulation results with the monitoring data. For this purpose, the results obtained from the model were compared to measurements from benchmarks, inclinometers and piezometers. Their location is presented in Fig. 7.

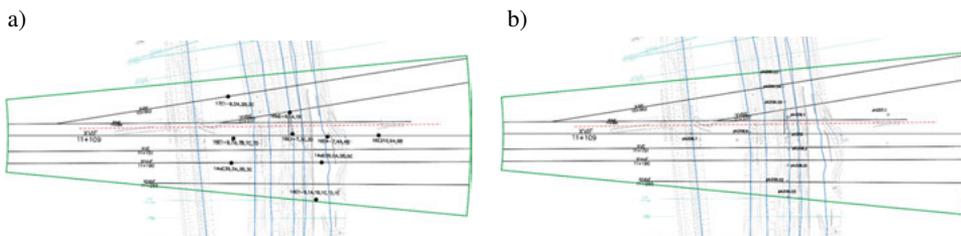


Fig. 7. Location of: a) piezometers and inclinometers; b) benchmarks

The calibration of the model was performed by parametric analyzes due to the strength, deformation and filtration properties as well as the spatial arrangement of layers in the subsoil. Data from geological investigation usually come from point boreholes or soundings.

Even if a geological interpretation is available, it is most often in the form of plain (2D) subsoil cross-sections. As a consequence, when performing 3D calculations, such information is not sufficient to unequivocally reproduce the spatial arrangement of layers in the substrate – there exists an area of uncertainty. In order to overcome this issue, it has been proposed to use the geostatic interpolation technique, namely kriging. Within the framework of the parametric analyzes, four different subsoil layer arrangements were created, denoted as: BH1, BH2, BH3 and BH4. For that purpose, geotechnical cross-sections (both longitudinal and transverse) were utilized as a given (fixed) data. Furthermore, spherical semivariogram function (being a measure of the correlation between the boreholes in considered space – cf. [13]) was adopted for interpolation. Moreover, the interpolation by kriging technique was carried out with the use of built-in “boreholes” module of ZSoil.

The BH1 model reproduces, as good as possible, the profiles (transverse and longitudinal cross-sections) of the subsoil under the eastern dam. To accomplish that, in addition to the “real” boreholes (Fig. 8a) from drilling, a set of “virtual” boreholes, spaced approximately every 20 meters, was created (Fig. 8b). These “virtual” boreholes were extracted from 2D geological profiles. In the contrast to above stated, the BH2, BH3 and BH4 models used only “real” boreholes. These models were differentiated by the parameter R of the semivariogram function, standing for the correlation length. The following values were adopted: $R = 500$ m, $R = 100$ m and $R = 50$ m for BH2, BH3 and BH4 models, respectively. The reconstructed layer systems in the BH1–BH4 models are shown in Fig. 9.

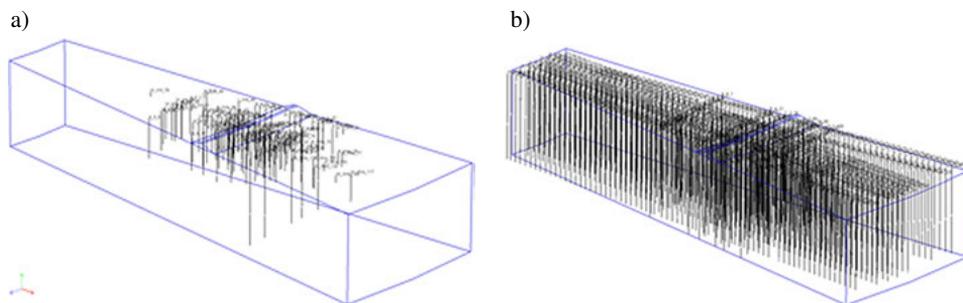


Fig. 8. Location of boreholes in the model: a) “real”; b) “virtual”

Exemplary results of numerical model calibration are presented in Figs. 10–12. In particular, Fig. 10 presents horizontal and vertical displacements against dam elevation for selected benchmarks. In Fig. 11 water head levels versus dam elevation for selected piezometers are shown whereas Fig. 12 presents in-depth ground displacements from inclinometers. In all figures, monitoring results are displayed with black markers and the simulation results with solid or dashed color lines: red – BH1, blue – BH2, green – BH3, yellow – BH4.

As mentioned in Figs. 10–12 only selected results concerning calibration are shown. In the entire procedure of the calibration much more results were taken into account, namely the data over time acquired from: 11 benchmarks, 24 piezometers and 11 inclinometers. As a result of the calibration it was stated that the best general fit to the monitoring data was

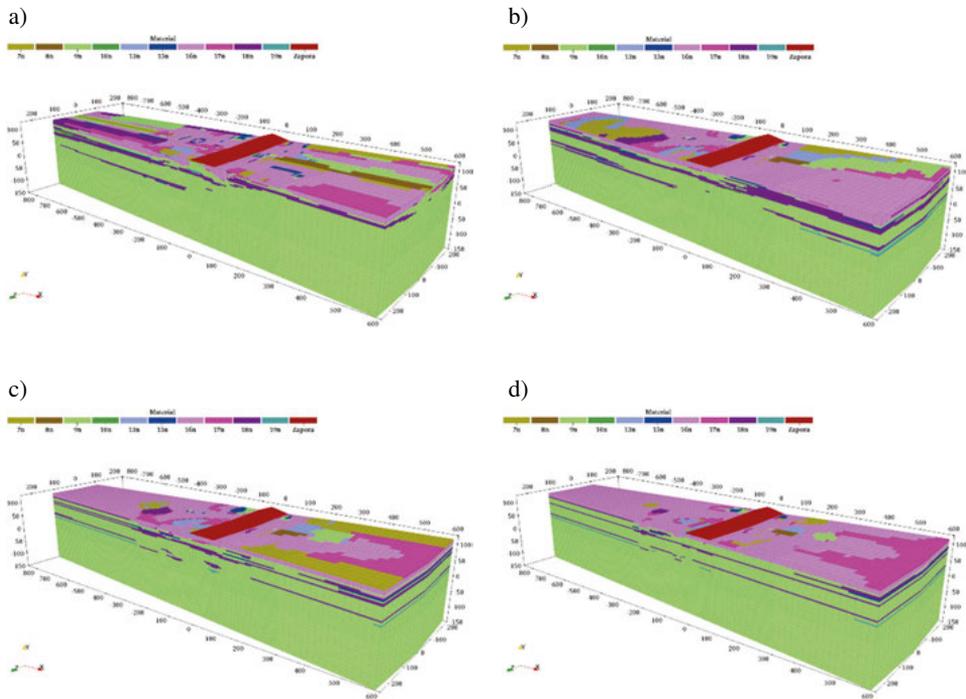


Fig. 9. Soil layer arrangements in the model: a) BH1; b) BH2; c) BH1; d) BH2

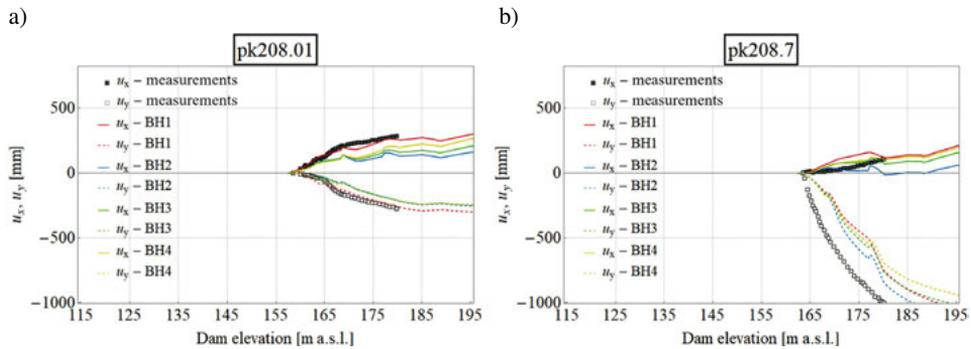


Fig. 10. Horizontal and vertical displacements for two selected benchmarks: a) pk208.01; b) pk208.7

obtained in the BH1 model. It can be seen in particular in the selected results presented before, e.g. Fig. 11 where BH1 model results almost perfectly fit the monitoring data. Therefore, this model is found to be representative for the considered section of the dam. Consequently, further analyses concerning optimization of relief wells were carried out for BH1 model.

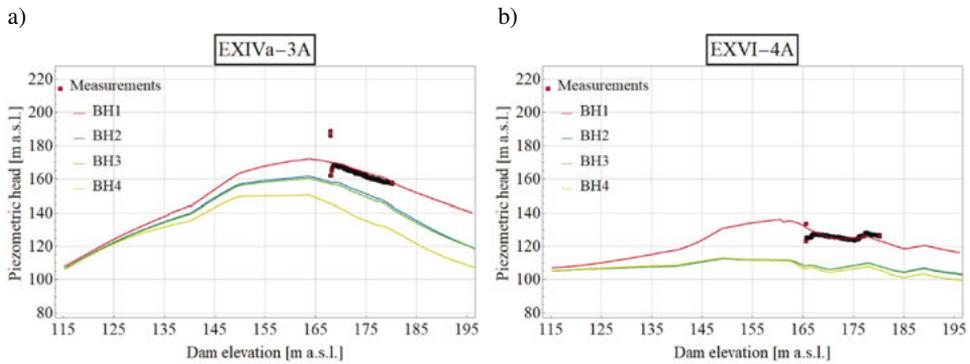


Fig. 11. Water head levels for two selected piezometers: a) EXIVa-3A; b) EXVI-4A

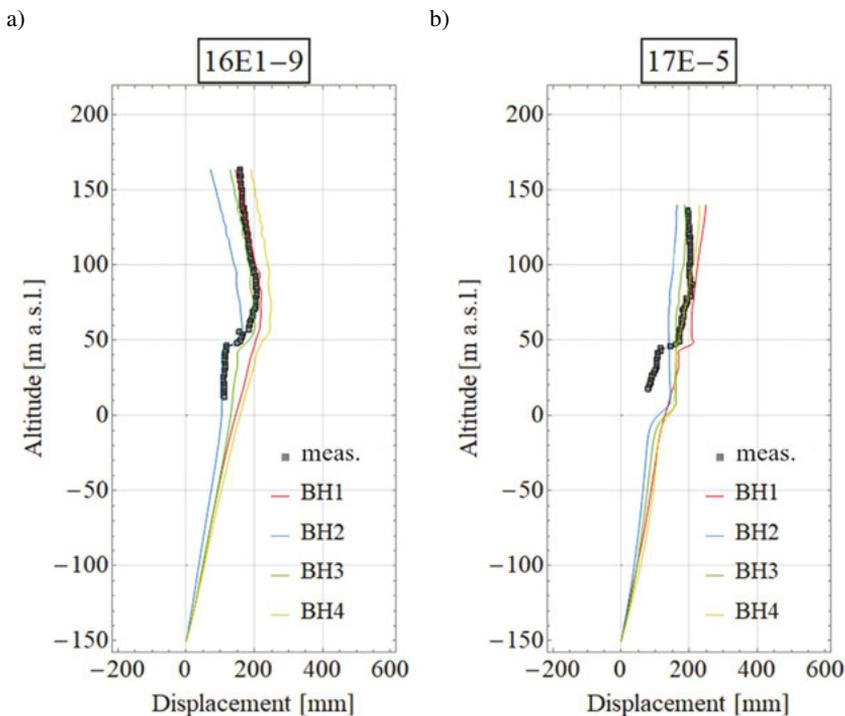


Fig. 12. In-depth displacements for two selected inclinometers: a) 16E1-9; b) 17E-5

4. Optimization of the relief wells

The main assumption of the analyses was to find the optimal solution concerning the application of relief wells, enabling the safe construction of the dam up to the elevation of 205 m a.s.l. The optimization consisted in determination of a system of new relief wells in

terms of their number, spatial arrangement, depth of filters as well as installation time. The analyzes also took into account the operation of the existing wells in the area of sections E1 and E2 (see Fig. 5). The factor of safety $FOS \geq FOS_{min}$ was adopted as a condition for the safe raise of the dam. This factor was determined using the shear strength reduction (SSR) method [2, 3, 6] in the framework of FE computations (performed in ZSoil). According to the appropriate national regulations $FOS_{min} = 1.5$ for the considered engineering structure. The maximum possible number of additional relief wells to be applied in considered area is 66. This number results from the technological conditions adopted for the facility. All relief wells (both, existing and additional) are presented in Fig. 13. This figure presents a maximum set of relief wells to be optimized. Thus, it constitutes the initial configuration for the optimization procedure.

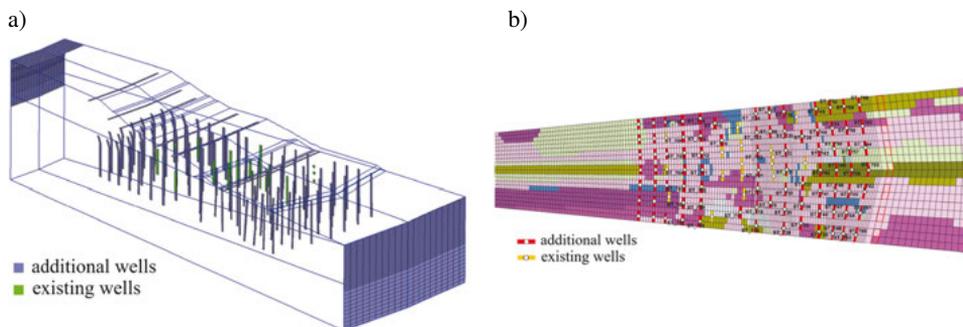


Fig. 13. Initial configuration of system of relief wells to be optimized: a) perspective view; b) top view (with FE mesh)

First, calculations were performed for the initial configuration in which all wells were activated. The purpose of these computations was to verify the maximum possible effect in terms of FOS increase. The value of $FOS = 1.64$ was obtained for dam elevation of 195 m a.s.l. On the other hand, the reference case, with the existing wells only (i.e. with no additional wells), provides $FOS = 1.46$. Thus, a significant improvement was indicated by the model. The next stage of the optimization was the verification of the effectiveness of groups of wells depending on their distance from the basin. For that purpose the groups of wells in predefined zones (Fig. 14) were activated/deactivated and the FOS were evaluated. This allowed to identify the zone where the installation of the wells has the largest contribution to the FOS increase. Consequently, it is the area closest to the basin (zone 3 in Fig. 14). At the same time, the left boundary of zone 3 is determined by the minimum distance of the wells to the reservoir due to technological limitations.

The considerations presented above indicated the area of detailed analyzes with respect to the number, distribution and depth of the wells to be carried out subsequently. Then, the wells within the zone 3 were divided into subzones presented in Fig. 15. Consequently the rows of the wells (in the individual subzones) were activated/deactivated and the FOS was assessed for different configurations. This in some sense follows the previously conducted procedure.

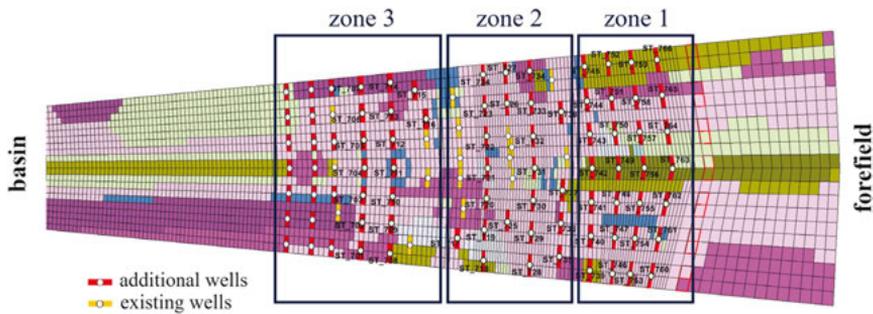


Fig. 14. Zones/groups of wells activated/deactivated during optimization

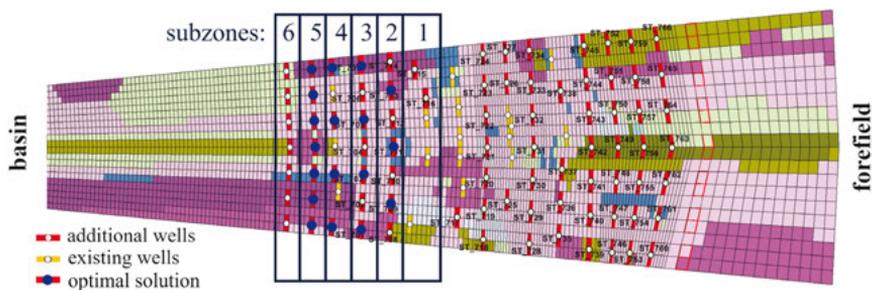


Fig. 15. Subzones of wells/groups activated/deactivated during primary optimization

Moreover, additional modifications were implemented, including: activation/deactivation of every second well in a row, differentiation of filter depth and installation time of the wells in the individual subzones. As a result of the conducted analyzes, the following optimal solution regarding relief well system was proposed:

- installation of 7 relief wells in subzones 2 and 3 (see Fig. 15) at the time moment corresponding to crest elevation 187.00 m a.s.l. and further construction of the dam to the elevation of 195.00 m a.s.l.,
- installation of 4 relief wells in subzone 4 (see Fig. 15) at the time moment corresponding to crest elevation 195.00 m a.s.l. and further construction of the dam to the elevation of 200.00 m a.s.l. ,
- installation of 7 relief wells in subzone 5 (see Fig. 15) at the time moment corresponding to crest elevation 200.00 m a.s.l. and further construction of the dam to the elevation of 205.00 m a.s.l.

According to the above, it means that it is proposed to further raise up the reservoir to 205 m a.s.l. with the use of 18 new relief wells, while the installation of individual groups of wells is spread over time. Taking into account the fact that the rate of dam raise is about 1.5 m per year, the installation of the wells in particular stages will take place every few (3.5–5) years. Another important information is also the fact that the depth of filter bottom was determined as 0 m a.s.l.

For such a program of installation of system of relief wells, the following values of factors of safety were obtained: FOS = 1.56, FOS = 1.54, FOS = 1.52, for the dam elevation 195.00 m a.s.l., 200.00 m a.s.l. and 205.00 m a.s.l., respectively. Finally, it should be stated that the application of 18 relief wells, according to the abovementioned scheme, ensures the safety margin required by the relevant national regulations (FOS \geq 1.5).

5. Conclusions

The paper proposed a comprehensive approach for optimization of preventive measures increasing the safety of tailings pond dams. Under consideration was the eastern dam of Żelazny Most TSF. In particular, sections E1 and E2 were investigated in details. Before starting the primary optimization it was necessary to create a 3D numerical FE model for which the full poromechanical coupling was adopted. This model was at first step calibrated against monitoring results, i.e., 11 benchmarks, 24 piezometers and 11 inclinometers. It is worth noting that in the parametric analyses, within the calibration process, 4 different reconstructions of the soil layer arrangements were taken into account. They were created with the use of geostatistical interpolation technique, namely kriging. The model denoted as BH1 was found to be representative for the considered section of the dam and therefore used for the purpose of optimization of preventive measures.

The main goal of optimization was to find the solution with respect to the system of relief wells enabling the safe construction of the dam up to the elevation of 205 m a.s.l. In particular, the following characteristics of the system had to be determined: the number, spatial arrangement and installation time of relief wells as well as the depth of the filter bottom. The condition for the safe raise of the dam was adopted according to the national regulations, namely: factor of safety FOS \geq FOS_{min} = 1.5. It was determined that the system of relief wells for the considered section of the dam should consist of 18 wells (with a filter reaching a depth of 0 m a.s.l.), while the installation of individual groups of wells can be spread over time. In particular, it was proposed to install the wells in three stages, i.e. at the elevation of the dam: 187.00 m a.s.l. (7 wells), 195.00 m a.s.l. (4 wells) and 200 m a.s.l. (7 wells). In accordance with the rate of the dam raise, it means that subsequent stages of wells installation are time-shifted by about 5 years and next 3.5 years.

The above stated results are based on the predictions of the FOS calculated using 3D numerical model. With regard to the idea of the simultaneous use of monitoring results with advanced 3D modeling it is strongly advised to verify the model against actual in-situ measurements before the subsequent stages of the wells installation. Then, the model should be recalibrated based on updated measurements of displacements and pore pressures in accordance with the procedure presented earlier in the paper. In addition, due to the fact that potential failure mechanism is getting larger (reaches deeper subsoil layers) as the elevation of dam crest rises, it is furthermore recommended to successively perform geotechnical investigations according to the appropriate program. In particular the geotechnical conditions at great depths (at which the failure mechanisms are located) should be identified from in situ tests. In such a case, the model should also be supplemented

with these new results concerning the geotechnical layers arrangement and their mechanical parameters. Before proceeding with the further expansion of the facility (above 190 m a.s.l.) and the installation of the last stage of relief wells, the possibility of raising up the dam (up to 205 m a.s.l.) should be verified again based on the BH1 model updated with new geotechnical data.

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Optymalizacja działań technologicznych zwiększających bezpieczeństwo zapór OUOW Żelazny Most przy jednoczesnym wykorzystaniu wyników monitoringu i zaawansowanych modeli obliczeniowych

Słowa kluczowe: hydrotechnika, kriging, metoda elementów skończonych, studnie odciążające, zapora

Streszczenie:

W artykule przedstawiono podejście do optymalizacji zabiegów technologicznych zwiększających bezpieczeństwo zapór zbiorników odpadów wydobywczych. Opiera się ono na jednoczesnym wykorzystaniu wyników monitorowania oraz modelowania numerycznego 3D w ujęciu metody elementów skończonych. Rozpatrywana jest wschodnia zapora OUOW Żelazny Most. W ramach pracy stworzono cztery modele numeryczne zapory i podłoża, różniące się przestrzennym układem warstw gruntowych. W tym celu wykorzystano technikę krigingu. Na podstawie wyników monitoringu skalibrowano modele numeryczne i wybrano ten, dla którego przeprowadzono procedurę optymalizacji. Zoptymalizowano zarówno rozkład przestrzenny, jak i czas instalacji studni odciążających. Wykazano, że takie rozwiązanie zapewnia, wymagany stosownymi rozporządzeniami, zapas bezpieczeństwa.

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