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ASSESSMENT OF SAFE EXPLOITATION OF POST-FLOATATION STORAGE FACILITIES

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Abstract: The *Żelazny Most* tailings pond, as the largest post-floatation waste storage facility of that type in Europe, requires a particularly versatile supervision of exploitation safety. This supervision should include the assessment of the pond's present condition as well as the prospective construction of its dams' superstructure. This work presents the assessment of the dam stability and the estimation of its further deformation process, on the basis of the authors' own interpretation of the displacement and deformation of the tailings pond eastern dam and its protection area, taking into account systematic observations of benchmarks displacement.

Streszczenie: Składowisko *Żelazny Most*, jako największy w Europie tego typu zbiornik odpadów poflotacyjnych, wymaga prowadzenia szczególnie wszechstronnego nadzoru bezpieczeństwa eksploatacji, uwzględniającego stan obecny jak i wynikający z dalszych, planowanych etapów nadbudowy jego zapór. W pracy, na podstawie własnej interpretacji przemieszczeń i deformacji zapory wschodniej zbiornika i jej przedpola, uwzględniającej systematycznie prowadzone obserwacje przemieszczeń reperów, dokonano oceny stateczności zapory oraz oszacowania przebiegu dalszych deformacji.

Резюме: Складской двор *Железный Мост*, как самый большой такого типа бассейн послефлотационных отбросов в Европе, требует особенно всестороннего надзора за безопасностью экс-плуатации, учитывающего как актуальное состояние, так и состояние, вытекающее из дальнейших, планированных этапов надстройки его заграждений. В настоящей работе на основе личной интерпретации сдвигов и деформаций восточной части заграждения бассейна и его предгорья, учитывающей систематически веденные наблюдения сдвигов реперов, сделана оценка устойчивости заграждения, а также течения дальнейших деформаций.

1. INTRODUCTION

Polish copper industry, which holds an important position nationwide and worldwide, developed in the 1950s in the area of the North-Sudeten Trough (i.e., Grodzieck and Złotoryja synclines). The mines of the so-called *Old Copper Basin* were gradually closed in consequence of copper ore exploitation. The Lubin–Głogów deposit of copper ore (of sedimentary type), located on the Sudeten foreland monocline in the south-eastern part of Poland (the so called *New Copper Basin*), was discovered in 1957 in Permian limestone formations. It belongs to the largest copper ore deposits in Europe and is rated among the largest in the world. Copper ore, excavated from the deposit in the shape of a mining output, is not fit for being used in metallurgical processes. It has to undergo the process of concentration beforehand. The remaining post-floatation waste, so far, has not found any large-scale application that would eliminate the necessity to store it on the surface. Therefore, attention should be mainly paid to the problem of the safe and economical construction and exploitation of post-floatation waste storages (tailings ponds). At present, there are five exhausted waste storages. The only functioning post-floatation storage of copper ore in the area of LGOM (Lubin–Głogów Copper Basin) is *Żelazny Most* tailings pond with the target capacity of 350 million m^3 and planned embankment height of 80 m (figure 1).

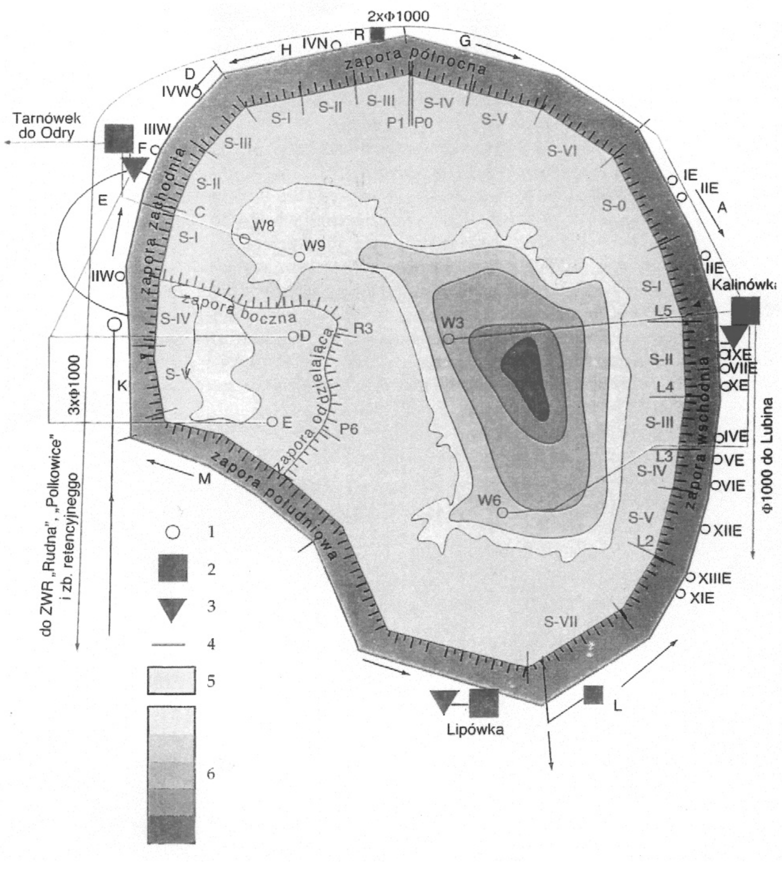

Fig. 1. *"Żelazny Most"* tailings pond. Description: 1 – wells, 2 – pump house, 3 – infiltration water tank, 4 – ditch running along the dam, 5 – beach, 6 – water area

The questions of the stability of post-floatation waste storage dams are extremely important and complex problems. The storages, similarly to hydrotechnical reservoirs, constitute a potential threat to the surrounding environment to a greater extent than the other engineering objects. The experience gained in the course of storage embankment construction and exploitation, as well as through disasters occurring with dramatic consequences, confirms that those endeavours typically require the solving of many geotechnical, geological-engineering and hydrogeological problems. The issue that should be taken into account in the assessment of the stability of waste storage dams is the excessive deformation of embankment and base frames, which, in turn, results from such factors as: the diversity of particle size distribution, consolidation degree and strength parameters of the waste (figure 2), excessive filtration, the influence of water-pumping in mines, static waste fluidization, the occurrence of weak geological formations at considerable depths (tectonic faults, slits), glacio-tectonically displaced soils, mining damage and mining exploitation. The survey and observations [2], [3] confirm that the largest storage stability is ensured by those geodesic methods that are most trustworthy as far as the determination of displacement and deformation of the embankment and base frames is concerned.

Fig. 2. The water area of *"Żelazny Most"* tailings pond

The *Żelazny Most* tailings pond, as the largest post-floatation waste storage facility of that type in Europe, requires a particularly versatile supervision of exploitation safety. This supervision should include the assessment of the pond's present condition as well as the prospective construction of its dams' superstructure.

This work presents the assessment of the dam stability and the estimation of its further deformation process, on the basis of the authors' own interpretation of the displacement and deformation of the tailings pond eastern dam and its protection area, taking into account systematic observations of benchmarks displacement (figure 3).

Fig. 3. Benchmarks on the eastern dam

2. DETERMINING DISPLACEMENTS OF POST-FLOATATION STORAGE FACILITY DAMS

The displacements of dams of copper ore tailings ponds are determined by means of geodesic methods. The method of precise geometric leveling is a tool for determining vertical displacements. Horizontal displacements are determined separately in trigonometric nets, in precise polygonal traverses or in control lines. For the purpose of marking the components of horizontal displacement values (the displacement that is transverse to the dam axis), the alignment observation method is applied (constant straight-line or datum line method). The forecasts of deformations that may occur in the course of the construction of dams accompanied by the simultaneous exploitation of tailings ponds are prepared with the use of previously determined displacement and deformation values by means of geodetic methods. The reliability of those forecasts is influenced by: reference system stability, the arrangement of reference marks and their stability, the precision of geodetic observations in an initial survey and in periodical surveys, as well as the time and frequency of periodical surveys [3].

This analysis refers to the oldest dam of *Żelazny Most* tailings pond – the eastern dam (figure 4). The existing documentation [4] constituted the basis for the analysis of two sections in which the local dam (and its base) displacements and deformations occurred to increased measures:

Fig. 4. The eastern dam – the area of section XVI

• section XVI E (km 11+109), in which geodetic survey entails the reference marks on the dam's top and protection area, a survey line that is perpendicular to the dam axis, and a group of deepwater benchmarks (figure 5).

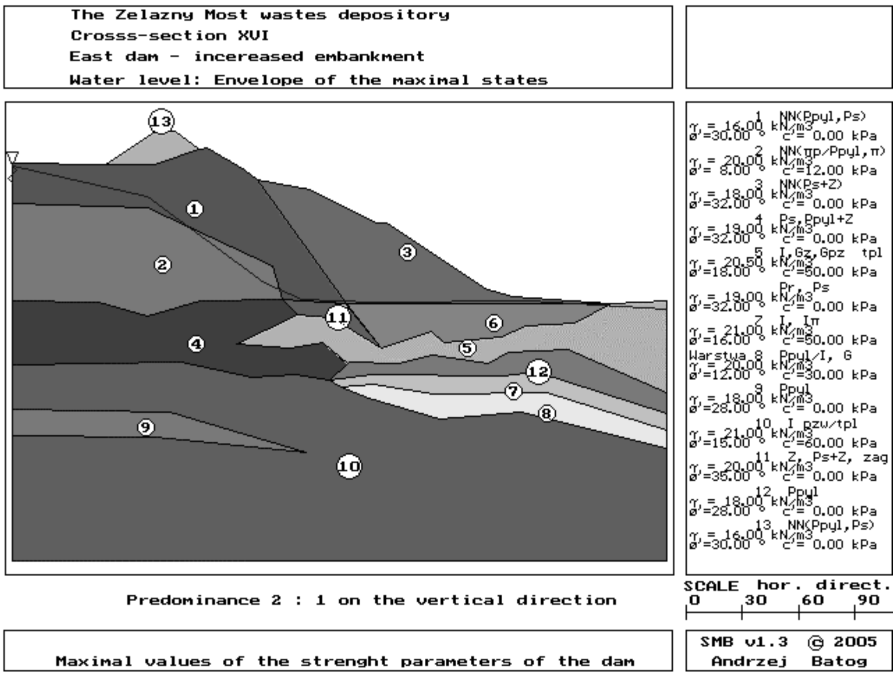

Fig. 5. A computational scheme of section XVI

• section XVIII E (km 10+807), in which the geodetic survey comprises the main reference marks on the dam's top and protection area, a survey line (perpendicular to the dam axis) on the load increase, and a group of deepwater benchmarks (figure 6).

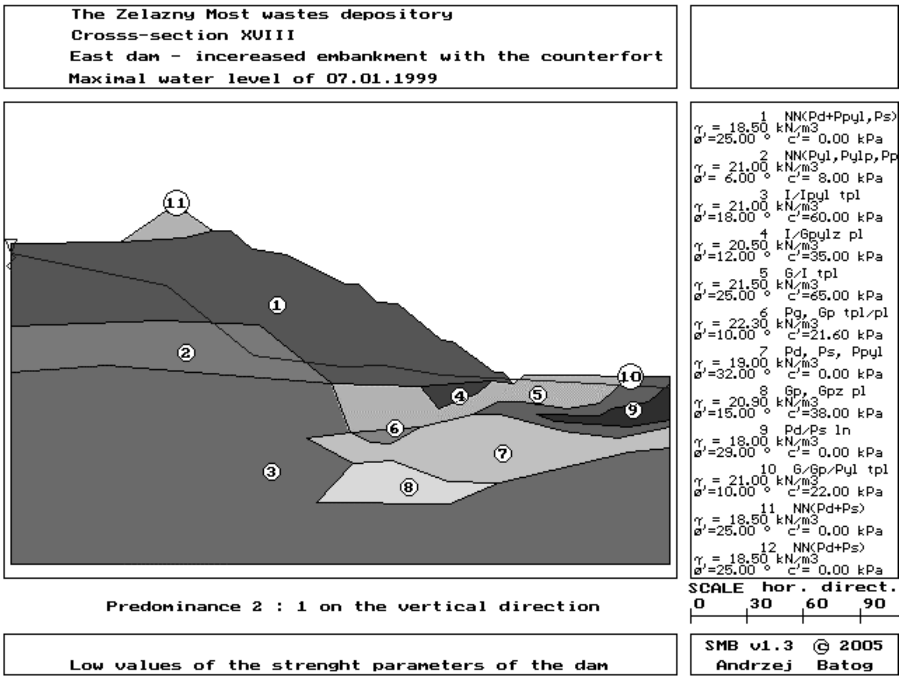

Fig. 6. A computational scheme of section XVIII

3. STABILITY ANALYSIS

3.1. SELECTING COMPUTATIONAL METHOD

The computations of stability of hydrotechnical constructions such as the dams of *Żelazny Most* tailings pond are one of the main premises for further assessment of their safe exploitation. There are numerous methods of computing slope and soil massif stability, such as block methods (Fellenius, Bishop and Janbu methods), to begin with, and ending with numerical methods used with growing frequency in computer programs, employing mainly the finite element method.

The value of the basic stability parameter, that is, stability factor F, may vary within the same problem, depending on the computational method applied. The results obtained by means of the finite element method have a considerably larger scope than those resulting from block methods, since, apart from the safety stability values, they also entail data regarding stress pattern, deformation, pore pressures, subsoil effort, the extent of softening zones, etc.

However, only in the case of block methods is it possible to apply the theorems about the bearing capacity [1], in accordance with which the safest assessment may be accomplished by means of Fellenius method. Regarding the stability factor computed with the use of the finite element method, it is difficult to produce such an assessment. Owing to that major limitation, in calculations the classical Fellenius method has been applied.

3.2. STABILITY ASSESSMENT

In order to assess the stability of the eastern dam in the selected sections XVI and XVIII, the computations have been carried out; their results determine the dependence of the minimal stability factor value $F_{\min}$ on:

– geometric diagrams (three diagrams were considered: *A* – taking into account the height of the dam from 1999, *B* – with the superstructure, and *C* – with the artificially embanked support);

– the level of a filtration curve (three situations were considered: 1 – taking into account the water levels in computational diagram *A*, minimal and maximal of the ones observed in the course of the pond superstructure construction);

– the values of geotechnical parameters of the embankment; the values of subsoil parameters, since these were recognized properly, were assumed in keeping with the existing documentation [4].

The results of these computations make it possible to determine the envelope of the results of the dam stability assessment. The calculations were performed with the use of the authors' original computer program SMB v. 1.3.

The stability factor regarding the equilibrium condition in the selected Fellenius method is described by the formula:

$$F = \frac{\Sigma[W_i(\gamma) + W_i(\gamma')]\cos\alpha_i \tan\varphi_i' + \Sigma c_i' l_i}{\Sigma[W_i(\gamma) + W_i(\gamma_{sr})]\sin\alpha_i} \tag{1}$$

where:

$W_i(\gamma)$ – weight of the *i*-th part of the soil block located above the water level (with bulk density of γ) and below the water level (with bulk density of γ', γ_{sr}, respectively, taking into account the hydrostatic lift and when the pores were saturated with water),

ϕ', c' – efficient parameters of soil strength, decreased by the value of pore pressure,

l_i, α_i – length and inclination of the slip surface in the *i*-th block.

The results of calculations may be shown by means of an isohypse diagram of the isolines of the stability factor constant values, determined inside a given area of the location of the centres of circular-cylindrical slip surface, together with the location of the slip surface, for the particular values of the stability factor. Some of the results of the minimal stability factor in accordance with formula (1) are presented in tables 1–3.

Table 1

The dependence of the stability on the water level (the highest soil strength parameters; the geometry of the dam – diagram *A*)

Variant	Minimal safety factor F_{min}	
	section XVI	section XVIII
The lowest water level	1.322	1.293
The highest water level	1.303	1.280
Water level (*A*)	1.323	1.280

Table 2

The influence of the alteration of embankment soil parameters on the stability of section XVI (the highest water level)

Variant	Minimal safety factor F_{min}	
	geometry (*A*)	hightened embankment
The lowest parameter values	1.269	1.220
Mean parameter values	1.289	1.241
The highest parameter values	1.322	1.250

Table 3

The influence of the alteration of embankment soil parameters on the safety/stability of section XVIII (the highest water level)

Variant	Minimal safety factor F_{min}		
	geometry (*A*)	hightened embankment	hightened embankment and counterfort
Low parameter values	1.269	1.164	1.430
Mean parameter values	1.289	1.188	1.454
High parameter values	1.322	1.216	1.466

The results suggest that the changes in the water table level affect the stability only to a minor degree (table 1). When the tailings pond is treated as hydrotechnical object of the 1rst class, in keeping with [5], one should note that the dam in the sections analysed does not have the required stability margin. The most disadvantageous slip surfaces, as far as sections XVI and XVIII are concerned, are illustrated by figures 7 and 8.

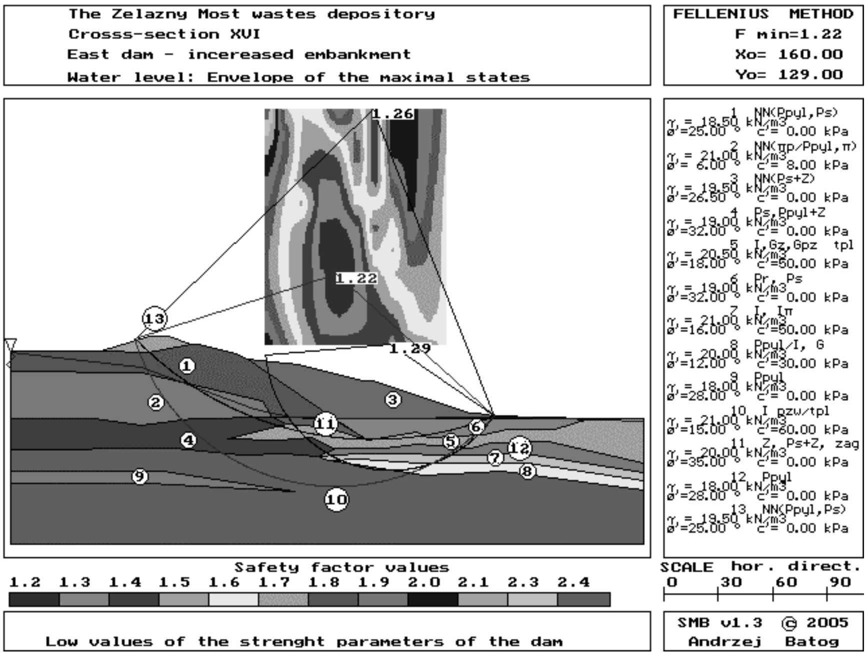

Fig. 7. The most hazardous slip surfaces of section XVI

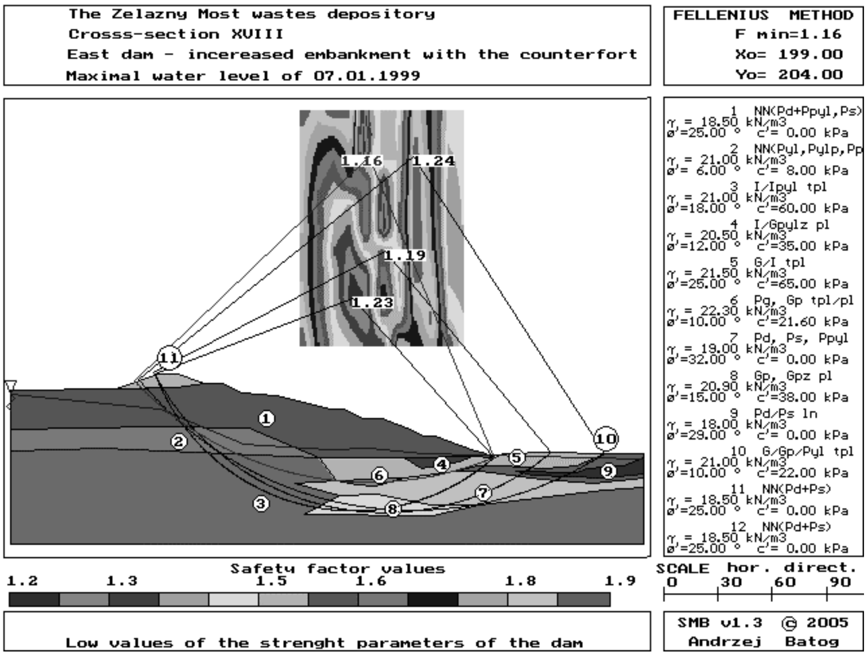

Fig. 8. The most hazardous slip surfaces of section XVIII

4. DEFORMATION FORECAST

4.1. ASSUMPTIONS

In order to analyse deformation of the eastern dam, a Swiss programme ZSOIL [6] was employed. The purpose of the computations was to obtain the estimation of the deformation process line on the basis of the systematic observations of benchmark displacement. It was assumed that the deformations are connected mainly with the ongoing rheological processes and occur chiefly in the embankment. The most difficult part is the assumption of creep parameters in computational sections. They were determined on the basis of a "backward" analysis, anchored in the systematic dam deformation surveys. The computations were thus carried out in two stages. The first stage consisted in determining soil deformations that occurred when the embankment was being formed, the second stage aimed at determining those deformations that resulted from the creeping process.

For the needs of numerical computations the soil was described by means of an elastic-plastic body model, and the limit state of stress – using the Drucker–Prager criterion. In the calculations of deformation values the Kelvin rheological model was employed. The creep was divided into a volumetric component and a deviatoric component (figural creep). Volumetric creep was described by means of Bjerrum's law, whereas deviatoric creep – using Singh–Mitchell's law

$$\varepsilon_v^{cr} = \sigma_m \frac{E}{K} A_v (t - t_o^v)^{m_v} , \tag{2}$$

$$e^{cr} = S\left(\frac{E}{2\mu}\right) A_d (t - t_o^d)^{m_d} , \tag{3}$$

where:

E, K, μ – elasticity parameters,
σ – volumetric component of stress (isotropic component of a tensor),
S – deviatoric component of stress (the stress deviator tensor),
t – time,
t_o^v – the time of volumetric creep initiation, volume creep,
t_o^d – the time of deviatoric creep initiation,
A_v – volumetric creep coefficient,
m_v – the exponent of volumetric flow rule,
A_d – deviatoric creep coefficient,
m_d – the exponent of deviatoric flow rule.

4.2. ASSESSMENT OF CREEP PARAMETERS

The manner in which the creep parameters were determined has been shown by the example of section XVI of the eastern dam. The analysis of deformation was carried out with the use of the read-outs at the base points (working benchmarks): 208, 208.4 and 208.5. At those points vertical displacement (settlement) values as well as the length of survey lines were determined, using precise geodetic methods. The values selected for the computations are juxtaposed below:

- total settlement values:
  – point 208 $s = 0.367$ m during 17 years,
  – point 208.4 $s = 0.192$ m during 8 years,
  – point 208.5 $s = 0.180$ m during 4 years;
- side length alteration:
  – side 208–208.4 $\Delta L = 0.185$ m during 15 years,
  – side 208.4–208.5 $\Delta L = 0.243$ m during 15 years.

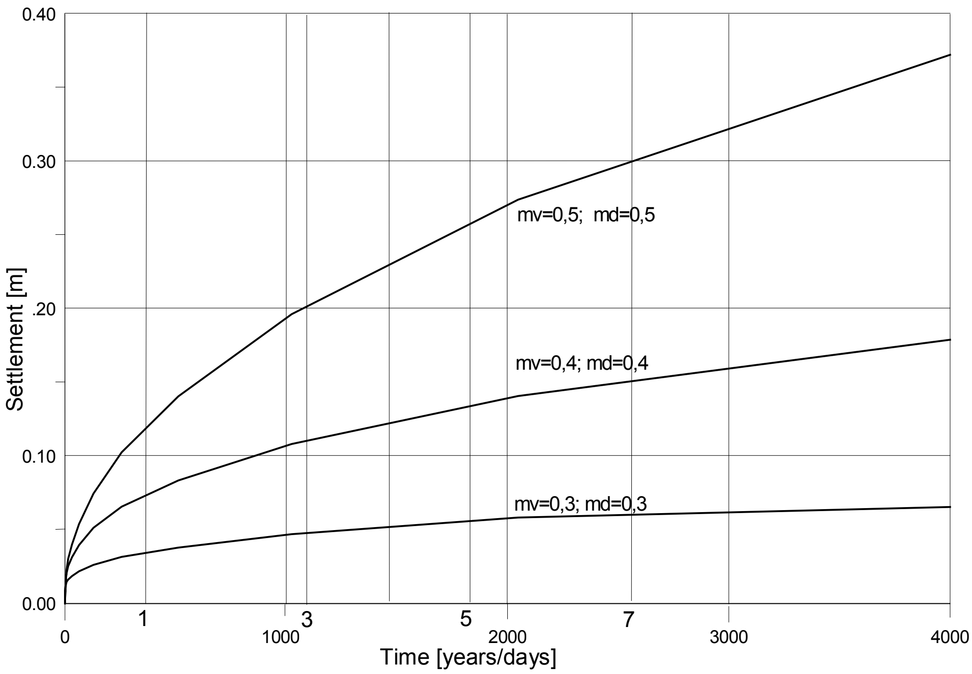

Fig. 9. Determination of creep parameters (m_v and m_d) for soil no. 3 (when $A_v = A_d = 1.2\cdot10^{-6}$)

The parameters describing the embankment soil creep were initially estimated on the basis of base point settlements. These parameters may be subject to correction on the basis of the surveyed alterations in the length of base sections ΔL. Preliminary calculations were carried out taking into account different values of the embankment soil creep (for both creep rules): A_v, m_v and A_d, m_d. The results of those calculations are illustrated by creep curves, for example, the ones that were determined at point 208.04 for soil no. 3 (figure 9).

The settlements caused by the creep of the selected base points contribute to the solution surface. Figure 10 shows the dependence of point 208.4 settlement on the soil creep parameters for embankment 3, which occurred within an 8 years' period. Inspecting this diagram, it is possible to read the preliminary soil creep parameter values for embankment no. 3 on the basis of the already known actual settlement value, measured in that time span. Thus, the parameters obtained must be corrected in order to minimize the differences between the calculated and measured embankment deformations in the whole section.

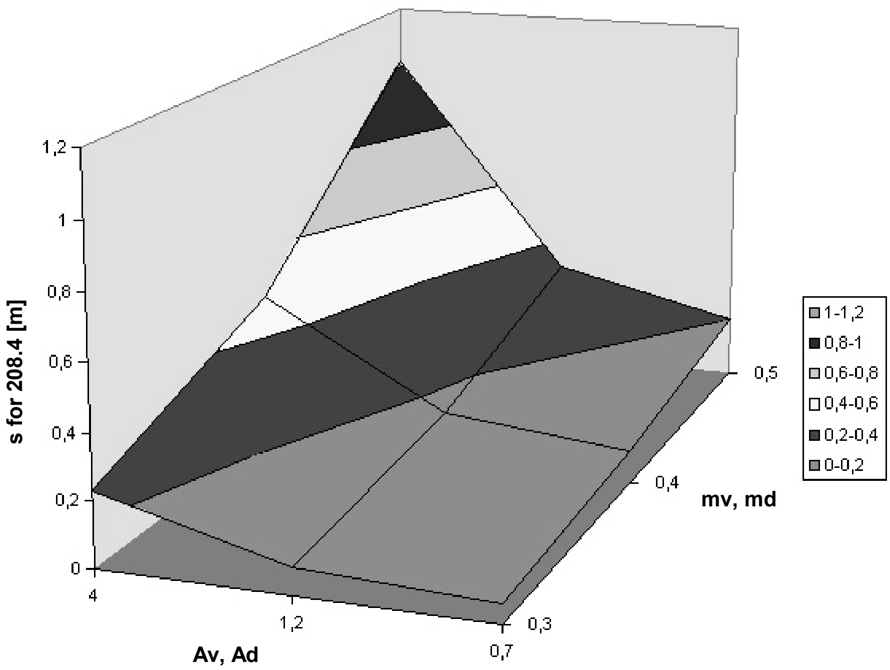

Fig. 10. The settlements of point 208.4 for varying values of creep parameters after the period of 8 years

4.3. COMPUTATION RESULTS

Once the soil creep parameters had been determined, it was possible to calculate changes in the embankment deformation over time $T = 0\div30\ 000$ days (82 years). As an example, figure 11 demonstrates the deformation distribution in section XVI caused by the creep over the period of 4 years. Table 4 provides the settlement values in section XVI of the eastern dam.

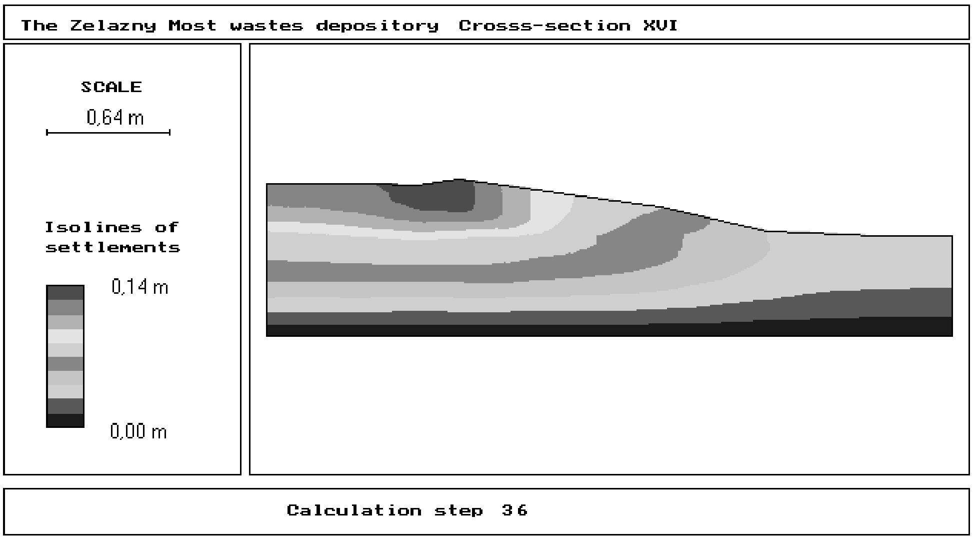

Fig. 11. The distribution of settlements in section XVI after a 4 years' time period

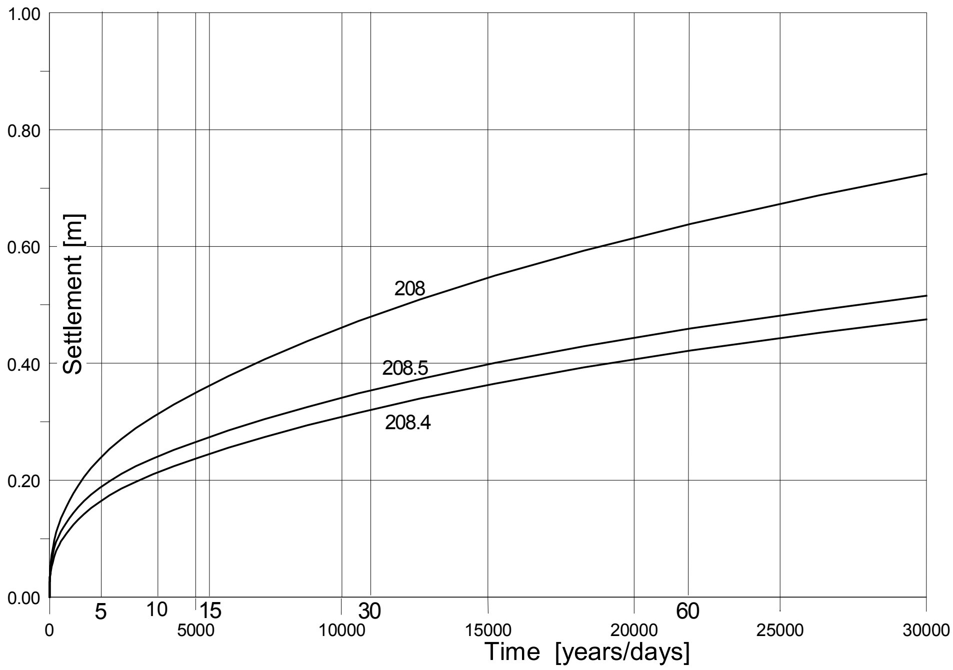

Fig. 12. Creep (settlement) curves in section XVI

Table 4

Computational results of the settlement of section XVI in the eastern dam

Point (benchmark)	Time [years]	Settlement [m]	
		computed	measured
208.5	4	0.175	0.180
208.4	8	0.198	0.192
208	17	0.378	0.367

The results are illustrated by the creep curves that show the settlement and the horizontal displacement alterations calculated within the period of 30 000 days. Figure 12 presents the curves of base points (benchmarks) settlement. Relying on the comparison of measured and calculated values, it may be concluded that it was possible to accomplish such computational model for section XVI that is characterized by large conformity to the actual state.

5. SUMMARY

In related literature, the authors have not come across the assessment of the stability of waste storage superstructures that would take advantage of a geodetic survey of the base and embankment line displacement. Due to the lack of geodetic survey and measurement results it was impossible to explain in an explicit way the reasons of disasters of many tailing pond dams (e.g., in 1967, in Iwiny waste storage facility, near Bolesławiec). The knowledge of such survey results would contribute greatly to the issue of safe superstructure construction and exploitation of such engineering objects.

From the numerical computations of stability and the analysis of deformation of the tailings pond dams, we can draw the following conclusions:

- the results of geodetic surveys may be a good basis for a “backward” analysis determination method – presented in this work – of the parameters (describing rheological processes in soil) that are essentially difficult for laboratory identification;
- the results of numerical computations show great conformity with those of the geodetic survey;
- the eastern dam of *“Żelazny Most”* tailings pond does not have the required stability margin (see [5]);
- if the geometry of dam section from the year 1 998 is preserved (the condition considered in these computations), the settlement will increase twofold within the period of the following 35–60 years.

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