minerals

Article

Tailings Behavior Assessment Using Piezocone Penetration Test

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Abstract: Intensive economic development is associated with an increasing demand for raw materials, including minerals. An illustrative example of this issue is the development of the copper industry. A significant problem arising from the scale of copper production is the management of an ever-growing amount of post-flotation tailings. This necessitates the need to ensure the continuity of safe storage. This study presents the results of studies on the behavior of deposits in the Żelazny Most Tailings Storage Facility (Poland). The primary objective of this study was to estimate the settlements of tailings under variable deposition conditions. The results were assessed using two methods: indirect and direct; this was based on cone penetration test (CPTU) results. The results were verified using Modified Cam Clay (MCC) modeling. Depending on the type of test, settlements ranged from several dozen centimeters to over three meters. Despite the observed differences, the results of both CPTU methods indicate a convergent trend in tailings behavior. Conversely, the results estimated using the direct method and numerical modeling demonstrate a high level of agreement.

Keywords: post-flotation tailings; tailings storage facility; settlements; TSF; CPTU; MCC

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1. Introduction

The intense economic development of the country is associated with a growing demand for resources, including minerals. An illustrative example of this issue is the development of the copper industry in Poland. The main national producer of copper is KGHM Polska Miedź S.A. Over a period of more than 50 years of operation, KGHM Polska Miedź S.A. has produced approximately 20 million tons of copper for the domestic and export markets. The mass of extracted rock is estimated to be around 1 billion tons. Based on the assessment of deposit resources, the exploitation of copper ore can continue for another 50 years. In light of such extensive exploitation, a significant problem arises in the management of a large amount of post-flotation tailings generated in the copper production process. In a historical context, this issue was addressed through the construction of specialized facilities called wet storage facilities. The utilized storage facilities underwent reclamation. Currently, the only active repository for all post-flotation tailings from the copper mines is the Żelazny Most Tailings Storage Facility (TSF). The construction of the TSF took place between 1974 and 1977. This type of hydrotechnical structure serves as both an integral part of continuous exploitation and is susceptible to damage and even catastrophic failures [1,2]. Therefore, to ensure safe operation, the facility undergoes comprehensive monitoring, both technical and environmental. As part of technical monitoring, geotechnical monitoring is conducted, including non-invasive in situ testing. This allows for the assessment of the behavior of tailings, both deposited in the facility and those used in the upstream method of its dam construction. Geotechnical research includes, among others, the Cone Penetration Test—CPT [3,4]. The knowledge compendium on this test is provided by the "Guide to Cone Penetration Testing" [5]. The test standard is not complicated, but the interpretation of test results can be problematic. The issue arises because the mentioned standard pertains to natural soils, not anthropogenic soils, which include tailings. Nevertheless, CPT has many advantages: testing is conducted in the current stress state, various soil characteristics are preserved (including structure, cementation, pre-consolidation), the test is repeatable, and it is comprehensive (various independent parameters are obtained). Most interpretative procedures and classification systems are based on a four-step scheme: registration of parameters, correction of registered parameters, normalization of corrected parameters, and the final determination of geotechnical soil parameters.

The first penetration tests, conducted using a mechanical cone, were carried out in the 1930s [5,6]. Over the subsequent decades, cones underwent improvements and numerous modifications [7,8]. The modern cone is dated to 1963, and the current nomenclature, CPT (Cone Penetration Test), has been applied since 1965. The geometry and dimensions of the electric cone (1965) formed the basis for the construction of modern cones, which were standardized. In 1974, the piezocone (piezoelectric cone) was introduced [5]. Cones vary in dimensions, from mini cones with a cross-sectional area of 2 cm^2, used for shallow investigations, through standard and commonly used cones with cross-sectional areas of 10 cm^2 or 15 cm^2, to special cones with a cross-sectional area of 40 cm^2, used for investigations of coarse-grained and rocky soils. The primary objectives of the Cone Penetration Test include geological, hydrogeological, and geo-environmental characteristics. Additionally, the results of piezocone penetration tests are used to determine a wide range of geotechnical parameters utilized in conducting various analyses, including liquefaction analysis [9,10].

In the standard Cone Penetration Test, the cone is pushed into the ground at a constant rate of 2 cm/s (allowing for the investigation of approximately 20 m of the profile in half an hour). CPT can be conducted both from the ground surface and within ponds.

The main goal of this study is to elucidate the interaction occurring between the subsoil, composed of post-flotation tailings, and the additional load structure also constructed from tailings. To achieve this objective, Cone Penetration Tests were conducted. The interpretation of CPTU results was carried out using three methods: (1) a direct method and (2) an unconventional indirect method. The unconventional interpretation involved the development of new empirical formulas that can be used to estimate the settlements of tailings. The results of the conducted research were further verified (3) using MCC modeling.

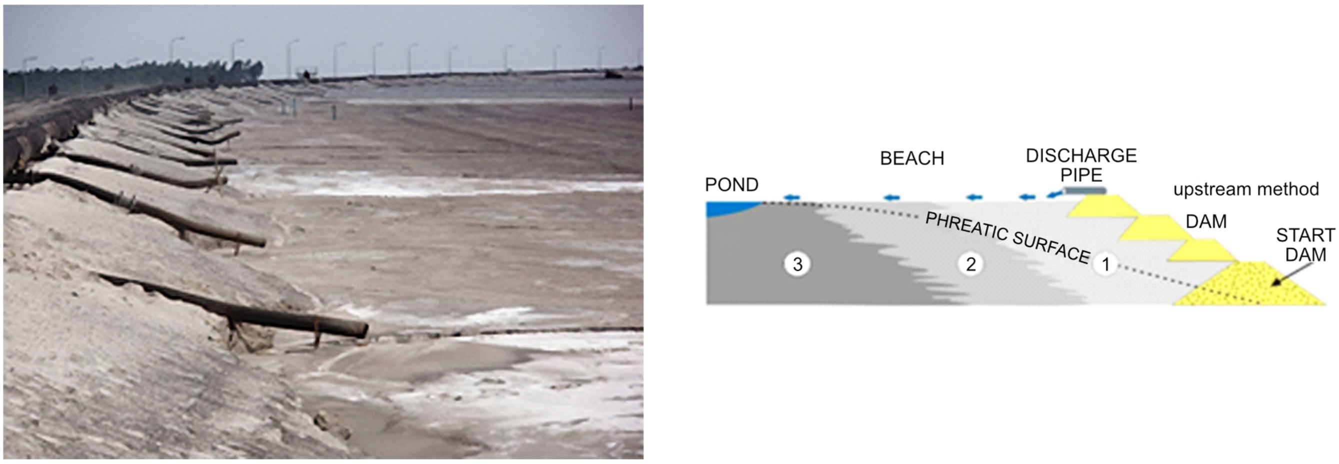
2. Materials and Methods

2.1. Study Site

The Żelazny Most Tailings Storage Facility (Figure 1) was put into operation on 12 February 1977, during the final phase of the Gilów storage operation. The storage facility is located within the territories of three municipalities belonging to two counties in the Lower Silesian Voivodeship (51.515634 N, 16.207030 E).

Figure 1. Three-dimensional graphic presentation of Żelazny Most TSF (excessive vertical scaling).

The Żelazny Most TSF is currently the largest hydrotechnical structure of its kind in Europe and the second-largest globally [11]. It covers an area of over 2000 hectares. The storage facility is surrounded on all sides by earthen embankments with a total length exceeding 20 km and heights ranging from 54 m to 76 m. Approximately 30 million tons of post-flotation tailings, equivalent to about 94% of the extracted rock, is deposited into the facility each year [12]. The tailings are transported hydraulically through pipelines (Figure 2). By the year 2038, approximately 950 million cubic meters of tailings can be deposited in the storage facility. Due to complex hydrogeological, geological, and geotechnical conditions prevailing in the facility subsoil, the facility is characterized as exceptionally intricate. The TSF is classified as an aboveground earth structure, constructed in stages with selectively fractionated tailings, primarily composed of sandy tailings fractions. The continuous expansion of the facility is carried out using the upstream method, while the recently constructed part of the facility is built using the downstream method.



(**a**) (**b**)

Figure 2. Żelazny Most TSF: (**a**) wet deposition of tailings, photo: KGHM Polska Miedź; (**b**) diagrammatic section: 1—coarse tailings (fine sands), 2—transitional zone (silt and sand), 3—fine tailings and slimes (clay and silt) [1].

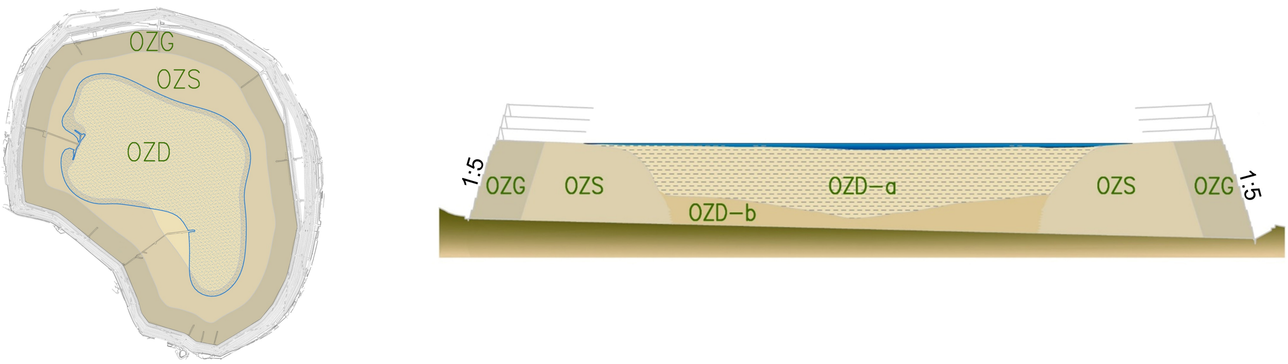
The simultaneous expansion and operation of the storage require continuous monitoring, including the prevention of failures. The Żelazny Most TSF is currently one of the best-monitored structures of its kind globally. Multi-directional monitoring enables the acquisition of a very large amount of data and observations.

The elements of geotechnical monitoring mainly consist of specialized in situ tests, deep drilling, a geodetic network, GPS stations, an inclinometer network, a seismometric network, a piezometer network, and others [13–17]. An extensive monitoring database allows for the assessment of the actual response of the storage facility to the substrate. Any change in the operation of the storage facility, such as further expansion, may initiate negative processes. To simulate such a change in the storage facility, an embankment was constructed. It serves the function of additional loading, utilizing selected sandy tailings previously deposited on the storage facility's beach. The length of the embankment is approximately 1 km, and its width is 15 m. Due to these parameters, the embankment loads all three zones of the storage facility. These zones were delineated, among other factors, based on grain size distribution: the beach, the transitional zone between the beach and the tailings pond (transitional zone), and the tailings pond (Figure 2). A detailed characterization of the embankment was presented in [18].

2.2. Tailings Characteristics

The flotation process relies on recovering mainly copper and other minerals from the ore, which are characterized by high floatability. Since the content of these minerals in the ore is low, their petrographic, mineral, and chemical composition resembles that of tailings material, but they still contain trace amounts of chemicals used during the flotation process [19]. Research on the chemical composition shows that the tailings exhibit high variability in chemical composition. The chemical composition of the tailings is predominantly composed of SiO_2 and CaO. The tailings exhibited an alkaline pH, with values ranging from 8.07 to 8.20, and revealed a high content of calcium carbonate, ranging from 7.56 to 16.23%. A high calcium carbonate content may affect the structure of the tailings by forming aggregates. As a result, the basic physical characteristic of the tailings, grain size distribution, may change. Additionally, this may impact the permeability of the tailings.

On the storage facility beach, the coarsest fractions settle, exhibiting sand-like properties. Sandy tailings are used in the construction of TSF dams and can also be successfully applied in earthworks and the production of geopolymers [20–22]. As one moves away from the beach, the volume of fine fractions increases, while the silty fraction migrates to the third zone. In the tailings pond, the finest tailings particles settle freely (Figure 3). This implies that the properties change with distance from the dam. These properties depend on factors such as the type of rock and the processes the rock has undergone: fragmentation, grinding, flotation. The TSF receives tailings from three different mining facilities: Lubin, Polkowice–Sieroszowice, and Rudna (Table 1). Understanding the properties of tailings plays a crucial role in assessing their behavior, response to loads, and paraseismic shock reactions. More information about the tailings and their detailed characteristics are provided in the bibliography concerning Żelazny Most TSF [1,18].



(**a**) (**b**)

Figure 3. (**a**) Generalized grain size distribution of tailings deposited in the TSF; (**b**) an example cross-section through the storage facility, where OZG—coarse tailings used for dam construction, OZS—coarse tailings with some amount of fines, OZD—fine tailings deposited in the pond: OZD–a—fine tailings in liquid state, and OZD–b—consolidated fine tailings.

Table 1. Approximate composition of post-flotation tailings from various mining plants.

Mining Plant	Lubin	Polkowice–Sieroszowice	Rudna
Type of rock and its content [%]		sandstones	
	65–70	4–14	79
		dolomites	
	20–23	75–86	17
		shales	
	10–12	10–11	4
Average content of fraction [%]		sandy	
	59	16	42
		silty	
	29	79	56
		clay	
	12	5	2

2.3. Research Method

2.3.1. Cone Penetration Test

According to the standard for the Cone Penetration Test, two main types of test are distinguished: CPT—without measuring pore water pressure in soil, and CPTU—with the measurement of this pressure. Below are three steps leading to the determination of geotechnical soil parameters.

1. Parameter recording During the test, fundamental penetration characteristics are continuously recorded: measured cone resistance—q_c and unit sleeve friction resistance—f_s (Equations (1) and (2), Figure 4).

- measured cone resistance $$q_c = Q_c/A_c \quad (1)$$

- unit sleeve friction resistance $$f_s = Q_s/A_s \quad (2)$$

where: Q_c—total force acting on the cone; A_c—cross-sectional area of the cone; Q_s—total force acting on friction sleeve; A_s—friction sleeve surface area.

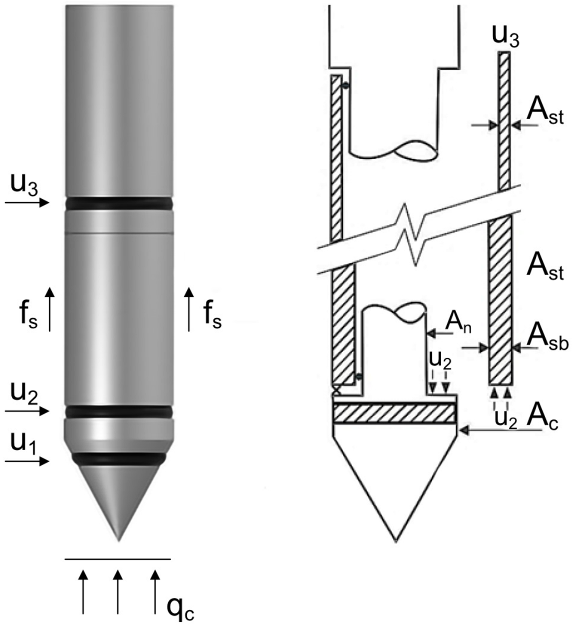


Figure 4. (**a**) Cone used for penetration test, author's own drawing; (**b**) the influence of pore water pressure on measured penetration parameters [8], where A_n, A_{st}, A_{sb}—cross-sectional areas.

The third characteristic that distinguishes CPTU from CPT is the pore water pressure, u_c (measured). It is measured using a filter embedded in the cone at one of three standard locations: u_1—pore pressure measured on the cone; u_2—pore pressure measured behind the cone; and u_3—pore pressure measured behind the friction sleeve (Figure 4a).

2. Correction of recorded parameters The recorded parameters of the q_c and f_s tests are subject to correction, among other things, due to the cone design (Figure 4b) and its susceptibility to the influence of excess pore water pressure induced during penetration (Equations (3)–(5), Figure 4b) [5,8,23]. Another type of correction results from the stress state (Equation (7)) or a reduction in the parameters to the form of effective parameters (Equations (6) and (8)). The group of corrected parameters also includes the friction ratio—R_f (Equation (9)), which is used in the SBCS (Soil Behavior Classification System).

- corrected cone resistance $$q_t = q_c + (1 - a_n)\, u_2 \quad (3)$$

where: $a_n = A_n/A_c$, (4)

- corrected sleeve friction $$f_t = f_s - (u_2 \cdot A_{sb} - u_3 \cdot A_{st})/A_s \quad (5)$$

- excess of pore water pressure $$\Delta u = u_c - u_0 \quad (6)$$

- net cone resistance $$q_n = q_t - \sigma_{v0} \quad (7)$$
- effective cone resistance $$q_e = q_t - u_2 \quad (8)$$
- friction ratio $$R_f = (f_t/q_t) \times 100\% \quad (9)$$

where: a_n—net area ratio of the cone, u_0—in situ pore pressure, σ_{v0}—total overburden stress.

3. Normalization of corrected parameters In the next step of the interpretative procedure, the corrected parameters undergo normalization to obtain dimensionless coefficients. The normalizing parameter is usually the vertical component of effective or total geostatic stress (Equations (10)–(14)) or the hydrostatic pressure of groundwater (Equations (15) and (16)). Formally, this parameter can also be the reference pressure–atmospheric pressure (Equation (11)).

- normalized cone resistance $$Q_t = (q_t - \sigma_{v0})\ \sigma'_{v0} \quad (10)$$
- normalized cone resistance, dimensionless $$q_{c1} = (q_c/p_a)(p_a/\sigma'_{v0})^{0.5} \quad (11)$$
- normalized friction ratio $$Fr = [f_s/(q_t - \sigma_{v0})] \times 100\% \quad (12)$$
- normalized pore water pressure $$u_t = (u_c - u_0)/\sigma' v_0 \quad (13)$$
- normalized, effective cone resistance $$Q_e = (q_t - u_c)/\sigma'_{v0} \quad (14)$$
- pore pressure parameter $$B_q = (u_2 - u_0)/q_n \quad (15)$$
- normalized pore water pressure difference $$NPPD = (u_1 - u_2)/u_0 \quad (16)$$

where: σ'_{v0}—effective overburden stress, p_a—reference stress (≈100 kPa).

Predicting settlements of natural soils based on CPTU is often recommended [4,24–34]. The recommendation typically pertains to estimating settlements according to Equation (17). The constrained modulus M (M_{CPTU}) is determined using the method proposed by Mayne [26] according to Equation (18).

$$s_{CPTU} = \Sigma(\Delta\sigma_v/M_{CPTU})\ \Delta z \quad (17)$$

$$M = \alpha\ (q_t - \sigma_{v0}) \quad (18)$$

where: s_{CPTU}—settlements (from CPTU), σ_v—total vertical stress, z—depth.

CPTU tests conducted at Żelazny Most TSF (Figure 5) were carried out in two stages: (I) reconnaissance of the geological (engineering) conditions of the embankment subsoil during the initial loading period (control tests), and (II) continuation of the tests: refining the scope and purpose of the research based on the results obtained in stage I.

2.3.2. Modified Cam Clay

A solution to engineering problems is the application of numerical methods using FEM (Finite Element Method). Currently, there are many more or less advanced computational methods in use. Among them, some allow for the analysis of the evolution of deformation in the subsoil. Obtaining all the required input data for the model is often a recurring difficulty. To conduct an analysis of embankment–subsoil interaction, the constitutive model for continuum MCC was employed. Most of the tailings' geotechnical parameters were obtained through CPTU testing. Additional parameters were determined in laboratory conditions using the TXT (Triaxial Test) [35]. The less advanced CM (Coulomb–Mohr) model does not require additional parameters. The choice of the MCC model is justified by the fact that, unlike the CM model, this model is usually applied to analyze weak cohesive soils. Although post-flotation tailings are not natural soils, they are most similar to this type of soil. Numerical calculations were performed using the ZSoil 2023 v23.54 multi–purpose finite element software for geotechnical professionals (see https://zsoil.com (accessed on

24 September 2023)). ZSoil is Windows-based finite element software, offering a unified approach to the numerical simulation of soil and rock mechanics.

(**a**) (**b**) (**c**)

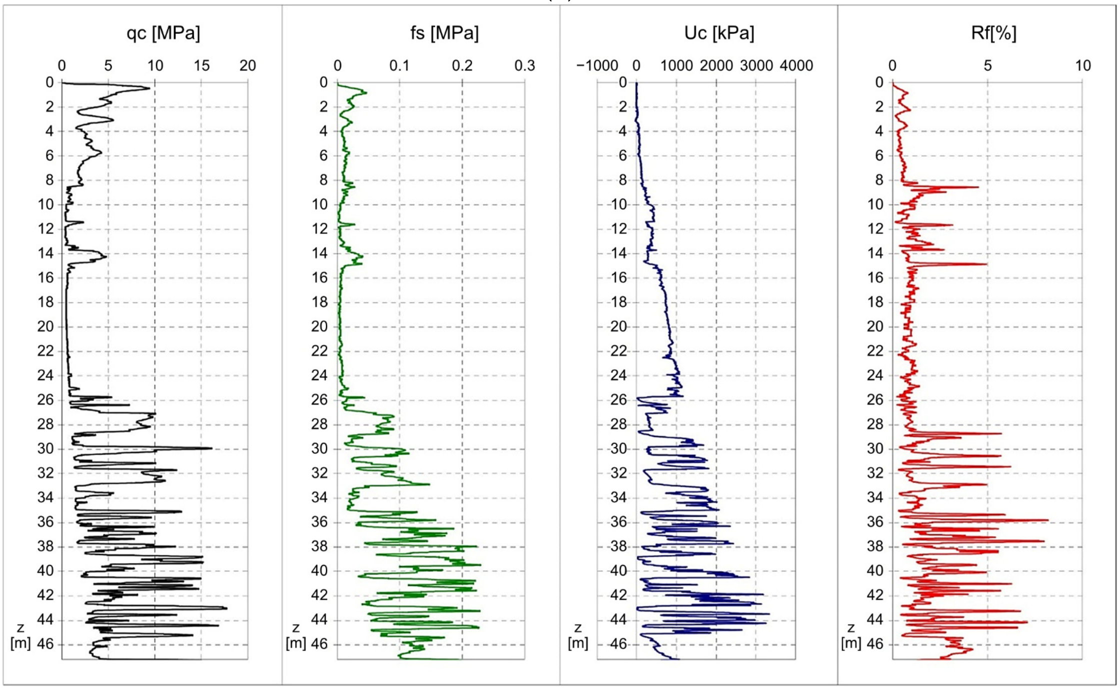
Figure 5. (**a**) Hyson 200 kN track-truck type by a.p. van den Berg company (Heerenveen, Netherlands); (**b**) touch screen by a.p. van den Berg company (Heerenveen, Netherlands); (**c**) interior of the Hyson, photo: KGHM Polska Miedź (Lubin, Poland).

3. Results

3.1. Preliminary Assumptions

An important factor influencing the analysis of research results is the adopted embankment construction technology and the methodology of control tests. According to the adopted concept, the research was conducted on a formed and stabilized embankment. Based on the analysis of tailings reactions to additional loading (stage I of the research), it was observed that the consolidation process varied in different zones of the storage facility. Minor settlements were observed in the beach area, resulting from volumetric deformations of the subsoil. In contrast, the largest deformations were found in the tailings pond zone, indicating a high excess of pore water pressure in the tailings. This suggests an unfinished consolidation process. These observations influenced the research program for stage II. Therefore, the research was limited to the transitional zone and the tailings pond, from chainage 0+600 to 1+100. CPTU was complemented by the collection of three hundred tailings cores using the MOSTAP sampler. The collection of cores was conducted using the Hyson 200 kN produced by a.p. van den Berg. The penetration rate was standardly set at 2 cm/s (Figure 6a). CPTU was performed using electric cones, enabling continuous recording of characteristics: q_c, f_s, and u_2 as a function of depth (Figure 6b). To understand the subsoil reaction to additional loading, research continuity was maintained. CPTU was conducted a minimum of two and a maximum of seven times (test series: a–f) at the same research points. All test series were conducted at one point—1+000.

(**a**)



(**b**)

Figure 6. (**a**) Hyson 200 kN during test conducted from a barge within the Żelazny Most TSF pond; (**b**) CPTU characteristics at the point 1+000.

3.2. Assessing the Settlements of Tailings Based on the Interpretation of CPTU Results.

The recommendation for conducting CPTU testing raises no objections; however, an open issue pertains to the selection of reliable interpretative procedures. The chosen procedures should enable obtaining soil characteristics that correspond to real conditions. In the case of a specific material, such as post-flotation tailings, commonly used interpretative procedures were subjected to verification. This is due to the fact, as mentioned earlier, that standard procedures were developed for natural soils. In the context of using CPTU for identifying the behavior of loaded tailings, two methods can be applied:

- Direct method: this involves assessing qualitative changes in penetration curves—graphical method,
- Indirect method: this relies on the quantitative assessment of geotechnical parameters—analytical method.

3.2.1. Direct Method

The direct method compares the penetration characteristics obtained from at least two independent CPTU tests conducted at the same location. The method is based on the analysis of the cone resistance–depth curve, identifying locations referred to as "indicators". Indicators are points of abrupt changes in cone resistance. Additional dissipation tests enable the determination of pore water pressure excess in tailings and the assessment of the dispersion of this excess over time.

The settlement analysis of tailings by the direct method was carried out at five research points. The results of the analysis are presented in Figures 7 and 8, as well as in Table 2. The results indicate variations in the values of tailings settlement at different research points (Figure 7): from 0.5 m at point 0+600 (nearest to the beach) to 3.0 m at point 1+000. Theoretically, the largest settlements should occur at point 1+100 (the point closest to the center of the tailings pond). As mentioned earlier, all test series were conducted only at point 1+000 (Table 2). This fact justifies the highest settlements at this point (Figure 8). In addition to fewer test series conducted at point 1+100, their duration was approximately half of that in point 1+000.

Table 2. Summary of tailings settlements determined by the direct method [m].

Point	0+600	0+800	0+900	1+000	1+100
Series			***s* [m]**		
start	0.0	0.0	0.0	0.0	-
a	-	-	0.4	1.3	0.0
b	-	0.5	0.6	1.8	0.2
c	-	0.6	0.7	2.1	2.1
d	0.5	0.7	0.9	2.7	2.2
e	-	-	-	2.9	-
f	-	-	-	3.0	-

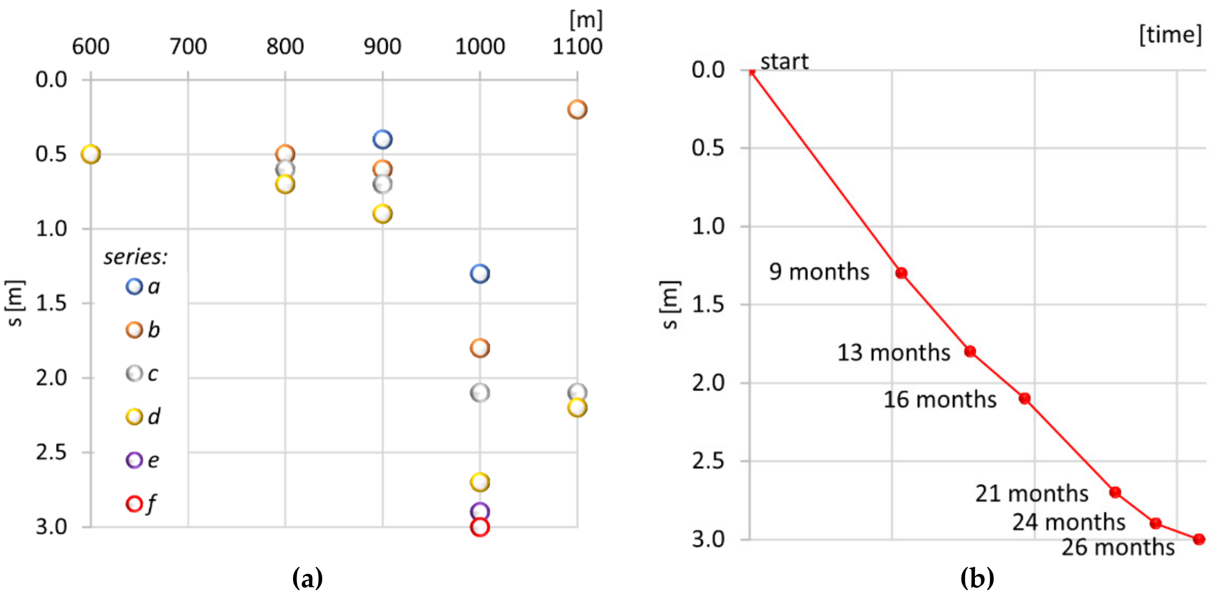
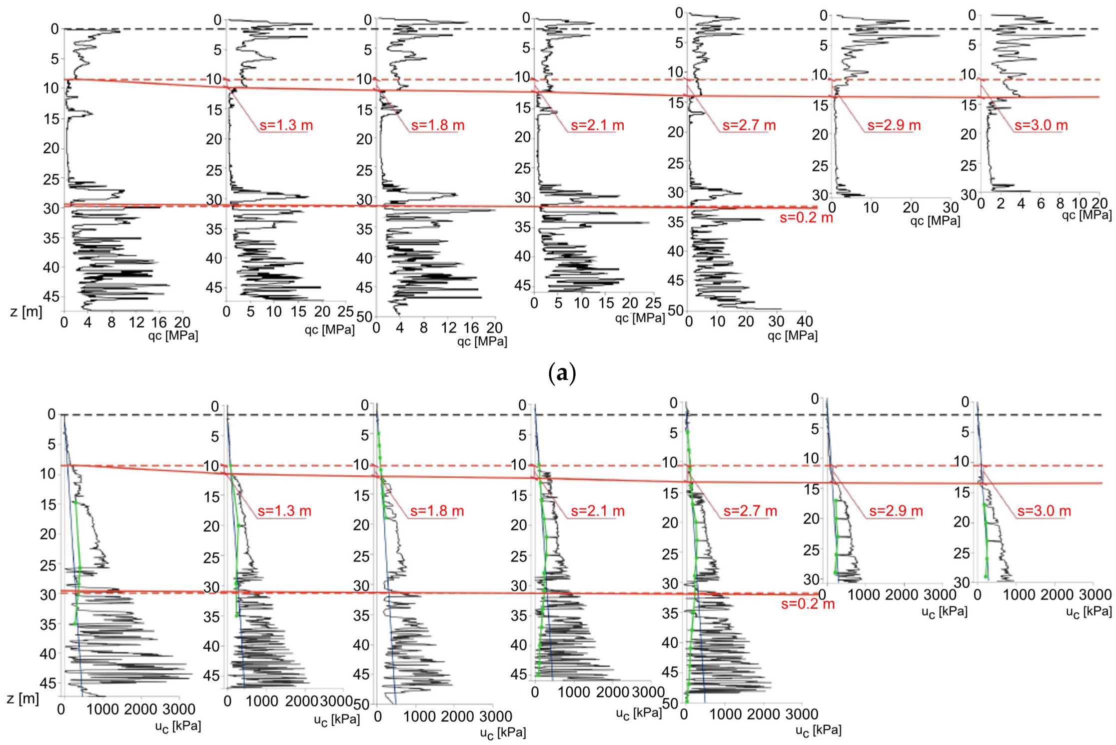


Figure 7. Graphical interpretation of settlements determined by the direct method: (**a**) points 0+600–1+100; (**b**) point 1+000.



(**a**)

(**b**)

Figure 8. (**a**) The settlements assessed by the graphical direct method at point 1+000 over a period of more than 2 years: q_c curves; (**b**) dissipation curves u_c. The dashed line corresponds to a value of s = 0.0 m, and the distance between the dashed line and the solid line represents the value of s [m].

3.2.2. Indirect Method

In the indirect method, values of geotechnical parameters are identified. Equation (19) was used to estimate total settlements. Calculations were conducted at all points where CPTU tests were performed.

$$s = \Sigma[(\sigma_{vi} \cdot h_i)/M_i] \quad (19)$$

where: h—layer thickness.

The results of the CPTU tests were used, among others, to determine M. Due to the specificity of the tailings, this required conducting calibration tests. In this case, oedometer tests were performed [36]. It should be noted that it is crucial to accurately determine the parameters from penetration curves at the same depth from which tailings samples were taken for oedometer tests.

The relationship between the dependent variable M and the independent variable (from the CPTU), q_c, q_t, and q_n, demonstrates linear dependency. For each cone resistance, a relationship was formulated, assuming that M is proportional to the cone resistance. The relationship is expressed as follows: $M = f(q_c)$, $M = f(q_t)$, $M = f(q_n)$. The results of the analysis and additional statistics, obtained using tools available in the Statistica by StatSoft (https://www.statsoft.pl (accessed on 16 August 2023)), are presented graphically and tabulated.

- q_c

  The value of M determined for the tested series of tailings is presented using Equation (20) (Figure 9a, Table 3).

  $$M = 4.97\ q_c \quad (20)$$

- q_t

  Taking into account the factor determining the cone characteristics (the influence of the recorded excess of pore water pressure of the tailings on the measured value of cone resistance) leads to the determination of another equation (Equation (21), Figure 9b, Table 3).

$$M = 4.24\ q_t \tag{21}$$

- q_n

  The next step involves considering two factors determining the cone characteristics: both the influence of the recorded excess of pore water pressure of the tailing on the measured value of cone resistance and the influence of the stress state in the subsoil on the cone resistance. This approach leads to the determination of a more complex but, at the same time, better-fitted relationship (Equation (22), Figure 9c, Table 3).

  - $q_n > 200$ kPa

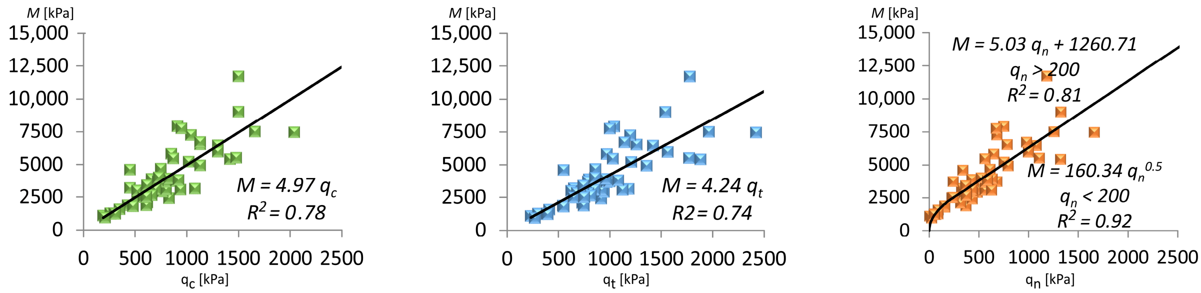
    $$M = 5.03\ q_n + 1260.71 \tag{22}$$

  - $q_n < 200$ kPa

    The above results of linear regression analysis pertain to the part of the q_n variable population, where $q_n > 200$ kPa. However, in the range where $0 < q_n < 200$ kPa, the function takes the form presented by Equation (23) (Figure 9c).

$$M = 160.34\ {q_n}^{0.5} \tag{23}$$

Based on the results of the conducted analysis, it can be observed that in the case of the weakest tailings, where the consolidation process has not yet been completed (under-consolidated tailings), the relationship $M = f(q_n)$ for $q_n < 200$ kPa is nonlinear. This is not typical for natural soils. This may indicate that if $q_n < 200$ kPa, the excess of pore water pressure of the tailings is proportional to the net cone resistance value.



(**a**) (**b**) (**c**)

Figure 9. Relationship between M and (**a**) q_c; (**b**) q_t; (**c**) q_n.

Table 3. Results of the statistical analysis of cone resistance from CPTU.

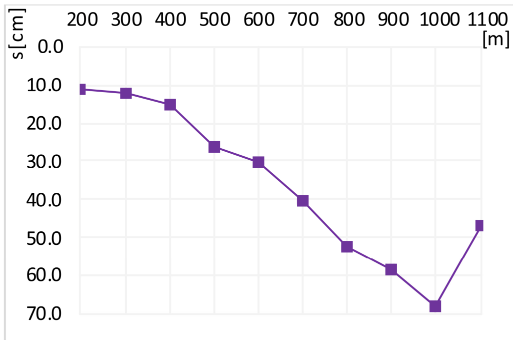
Intercept	b^*	Standard Error b^*	b	Standard Error b	t^1	p
q_c	-	-	0.00	340.25	0.00	1.00
	0.88	0.06	4.97	0.35	13.95	0.00
q_t	-	-	0.00	380.07	0.00	1.00
	0.86	0.07	4.24	0.33	12.54	0.00
q_n	-	-	1260.71	252.71	4.99	0.00
	0.90	0.06	5.03	0.33	15.13	0.00
Regression model is statistically significant at the p-level of $\alpha = 0.05$						
				q_c	q_t	q_n
				Correlation coefficient R: 0.88	0.86	0.90
				Coefficient of determination R^2: 0.78	0.74	0.81
				Coefficient of determination (adjusted) R^2: 0.77	0.73	0.80
				Standard error of estimation S_e: ±1292	±1402	±1212

where: b^*—standardized regression coefficient, b—unstandardized regression coefficient, p—probability value (p-value), α—level of significance. [1] 55 for q_c, 55 for q_t, and 47 for q_n.

For the purposes of settlement calculations using Equation (19), the value of M for each layer of the profile was determined based on cone resistances. The discretization of the subsoil into layers with a thickness of 0.5 m was performed using cone penetration test curves. Equations (22) and (23) were used to estimate the values of M. The results are presented in Table 4. Settlements estimated by this method are significantly smaller than those determined by other methods and range from 11 cm to 68 cm. Such small settlement results can be justified, on the one hand, by neglecting the effect of the long-term consolidation process and, on the other hand, by not accounting for significant deformations.

Table 4. Settlements comparison at points 0+200 to 1+100.

Point	Number of layers	σ_v [kPa]	s [cm]
0+200	119	50	11.10
0+300	152	50	12.05
0+400	153	70	15.19
0+500	148	75	26.18
0+600	160	80	30.15
0+700	187	75	40.19
0+800	186	80	52.59
0+900	181	85	58.57
1+000	159	93	68.15
1+100	85	115	46.87



where: σ_v—overburden stress, s—settlements.

3.3. Modified Cam Clay Verification Model

The input parameters for the MCC model consist of physical, mechanical, and filtration parameters of tailings, determined based on the results of CPTU (Table 5, columns 5–17). As mentioned earlier, the application of the MCC model requires additional parameters, which were compiled based on the TXT tests (Table 5, columns 18–20). The analysis was conducted based on the results of the tests carried out at point 1+000. The layer arrangement is presented by the cone resistance curve (Figure 10a). The profile was divided into seven layers (Figure 10b), and the geotechnical parameters of the tailings are compiled in Table 5. The computational grid was built with 4268 elements (Figure 10c). Due to the symmetry of the model, only one half was analyzed.

A three-stage loading scheme was adopted:

Stage I—loading progressively increased over 3 weeks to 50 kPa after the completion of Stage I settlement shapes at a level of 165 cm (Figure 11a). Additionally, the largest horizontal displacements, reaching up to 60 cm, are generated in the less-deformable zone of the tailings subsoil (Figure 11b). The greatest total deformations occur in the axis of the embankment and in the slope (Figure 11c).

Table 5. Comparison of input parameter values for the MCC model.

Layer No.	Layer: from-to [m]	Layer Thickness [m]	Soil Type	φ [°]	ψ [°]	c [kPa]	ν [-]	E [MPa]	β_f [kPa]
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
I	0.0–1.3	1.3	Tailings: embankment	41	18	-	0.22	65	-
II	1.3–3.5	2.2		35	9	-	0.24	32	2.2×10^7
III	3.5–10.0	6.5		29	0	-	0.30	12	
IV	10.0–26.5	16.5	Tailings: subsoil	10	0	5	0.25	1.9	
V	26.5–31.3	4.8		30	5	-	0.25	35	
VI	31.3–35.0	3.7		19	0	7	0.22	14	
VII	35.0–42.2	7.2		34	8	-	0.24	50	

Table 5. *Cont.*

Layer No.	k_x [m/s]	k_y [m/s]	Sr [-]	α [m^{-1}]	ρ [g/cm^3]	ρ_d [g/cm^3]	e [-]	λ [-]	κ [-]	M_c [-]
1	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
I	0.9×10^{-5}	0.9×10^{-5}	0.14	12.5	1.86	1.68	0.59	0.021	0.001	1.68
II	1.2×10^{-5}	1.1×10^{-5}	0.09	10.2	1.90	1.53	0.75	0.027	0.002	1.42
III	2.1×10^{-6}	2.5×10^{-6}	0.07	8.5	1.86	1.48	0.81	0.033	0.003	1.16
IV	3.1×10^{-9}	4.7×10^{-9}	0.19	0.5	1.95	1.44	0.90	0.200	0.015	0.37
V	2.6×10^{-6}	7.5×10^{-7}	0.09	6.8	1.89	1.53	0.74	0.056	0.004	1.20
VI	1.1×10^{-8}	9.1×10^{-8}	0.15	1.0	2.08	1.63	0.66	0.100	0.008	0.49
VII	2.2×10^{-6}	8.8×10^{-7}	0.10	7.0	1.91	1.57	0.69	0.046	0.003	1.37

where: φ—friction angle, ψ—state parameter, c—cohesion, υ—Poisson's ratio, E—modulus of elasticity, β_f—model parameter, k_x and k_y—permeability coefficient in the x and y directions, Sr—degree of saturation, α—model parameter, ρ—density of soil, ρ_d—density of dry soil, e—void ratio, λ—slope of the normal consolidation line, κ—slope of the swelling line, M_c—slope of the critical state line.

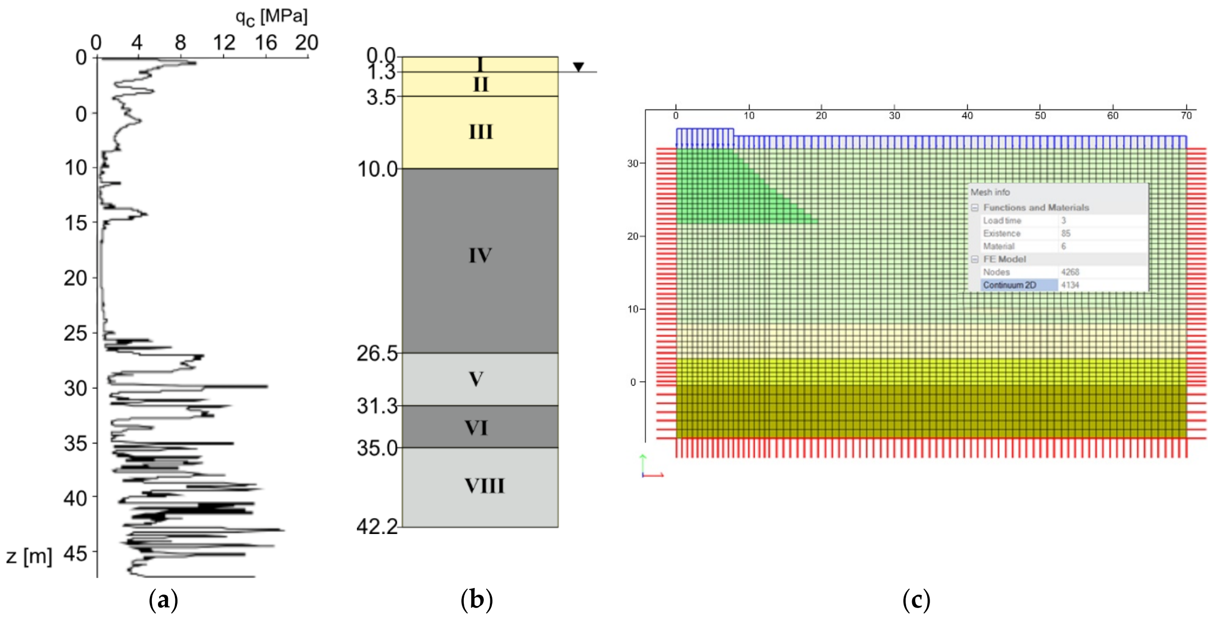
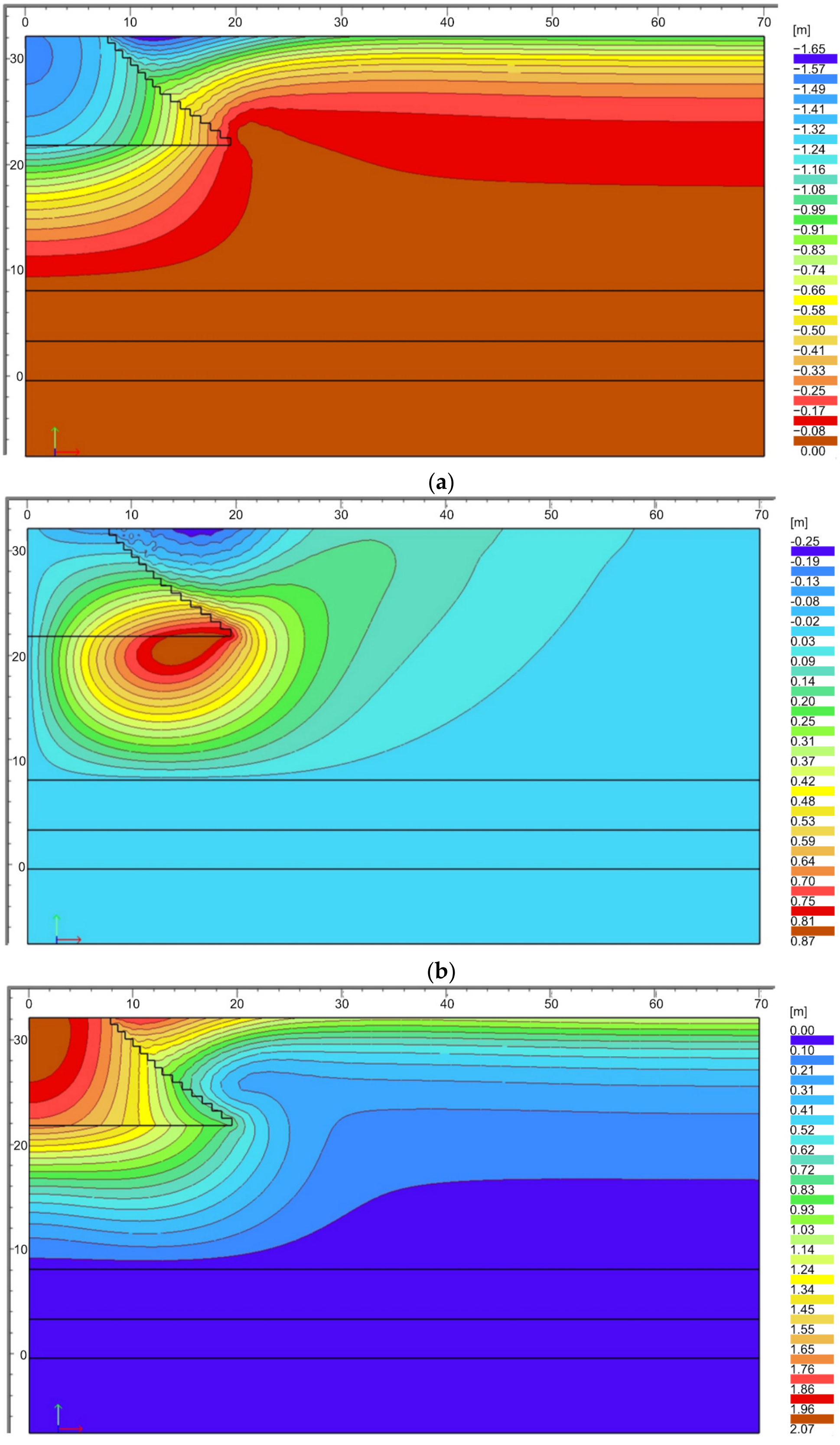


Figure 10. Point 1+000: (**a**) cone resistance distribution q_c; (**b**) calculation scheme based on q_c; (**c**) generated mesh of elements.

Stage II—after 16 months, the load was increased by 20 kPa, generating an increase in load over 2 weeks to a value of 70 kPa. Settlements in the embankment axis reached 207 cm (Figure 12a). Horizontal displacements increased to a value of 81 cm (Figure 12b). The distribution of total deformations is shown in Figure 12c.

Stage III—after an additional 10 months, the loading from Stage II was repeated, reaching a total load of 90 kPa. As a result, settlements in the embankment axis increased to 326 cm (Figure 13a). The same trend applies to the distribution of horizontal (Figure 13b) and total deformations (Figure 13c).

To validate the results, the distribution of pore water pressure excess in tailings during the initial phase of Stage I (Figure 14a,b) was compared with the actual distribution, determined based on CPTU with pore pressure measurements (Figure 14c). An analysis of Figure 14a indicates that the highest excess of pore water pressure occurs in the highly compressible silty layer, initially reaching 35 kPa (Figure 14a,b). The distribution of pore water pressure excess, as demonstrated by the results of dissipation tests, exhibits a similar shape with proportionally higher values (Figure 14c).



(**a**)

(**b**)

(**c**)

Figure 11. Displacements in Stage I: (**a**) vertical; (**b**) horizontal; (**c**) total.

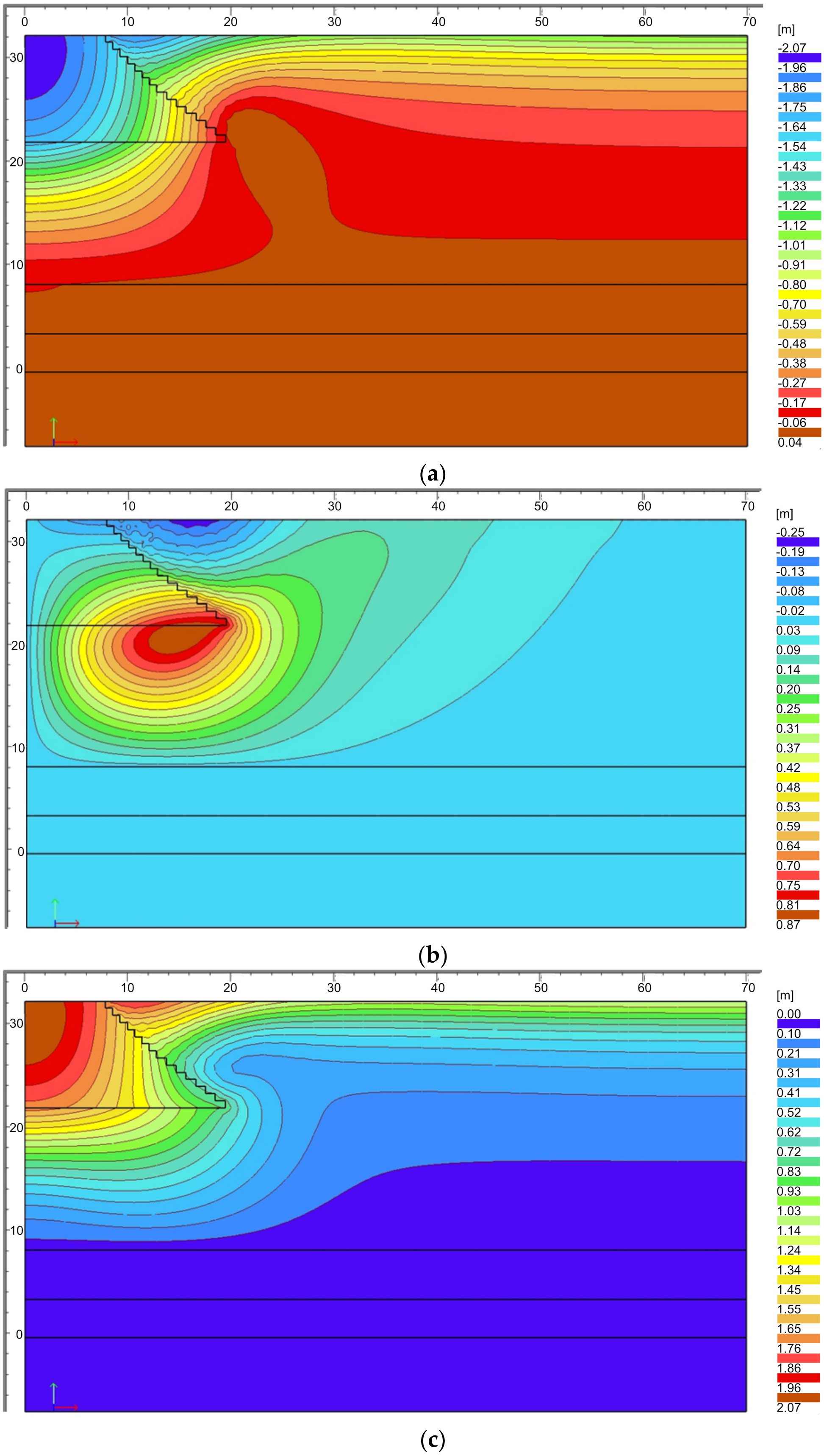
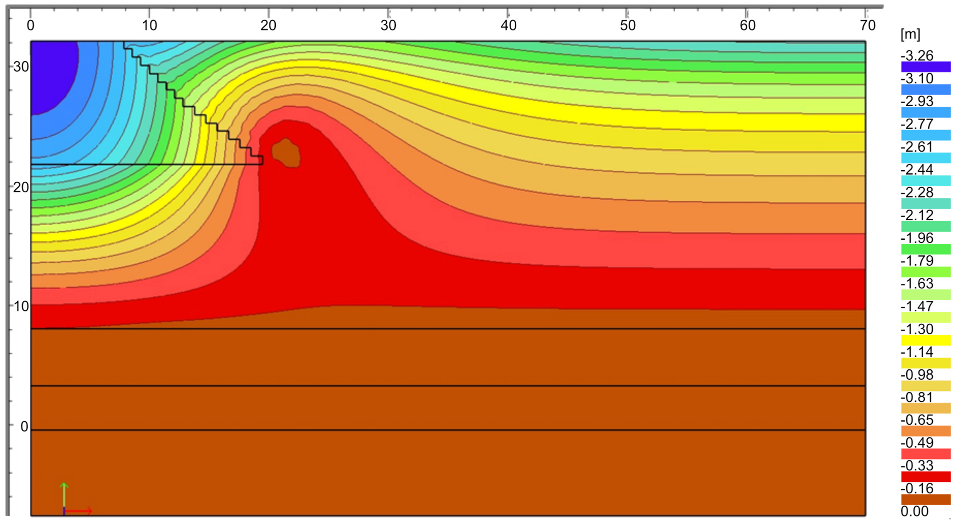
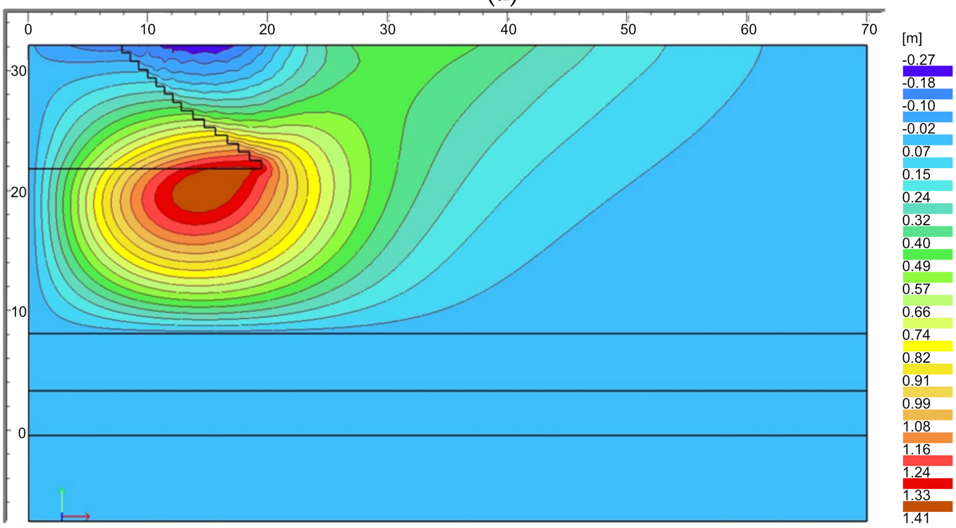


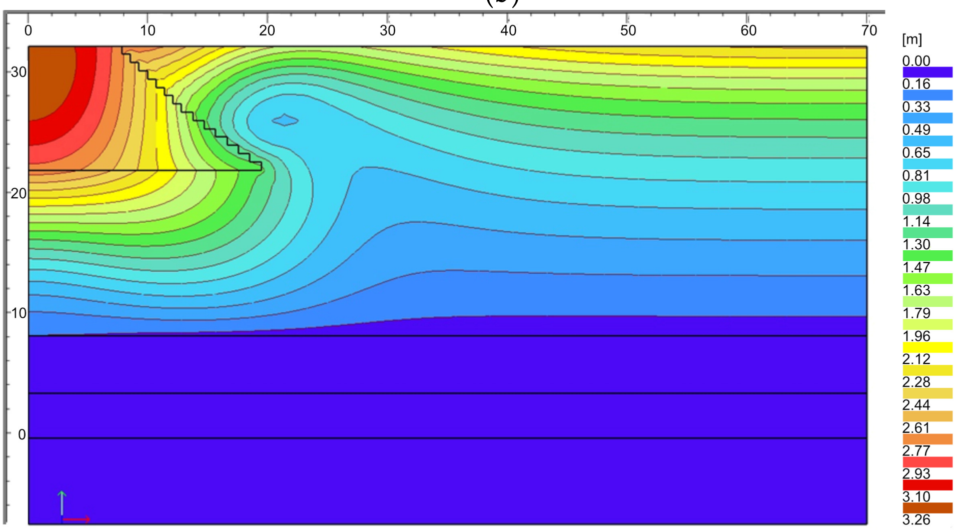
Figure 12. Displacements in Stage II: (**a**) vertical; (**b**) horizontal; (**c**) total.



(**a**)

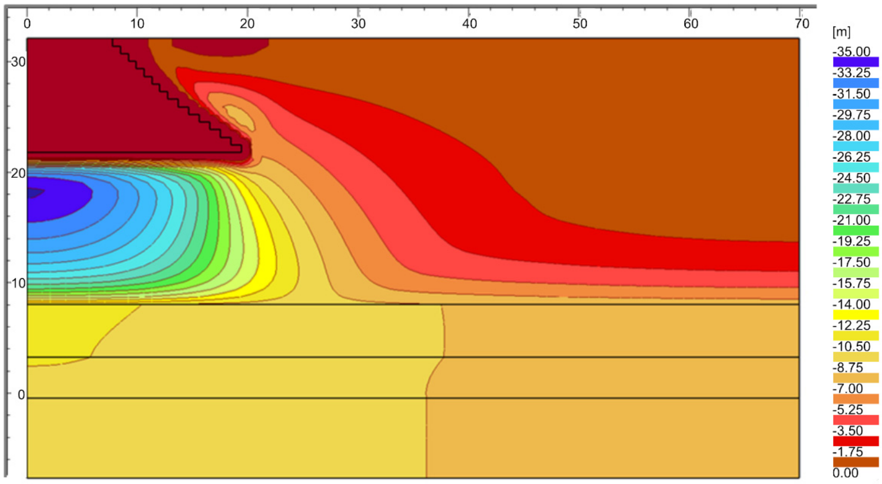


(**b**)

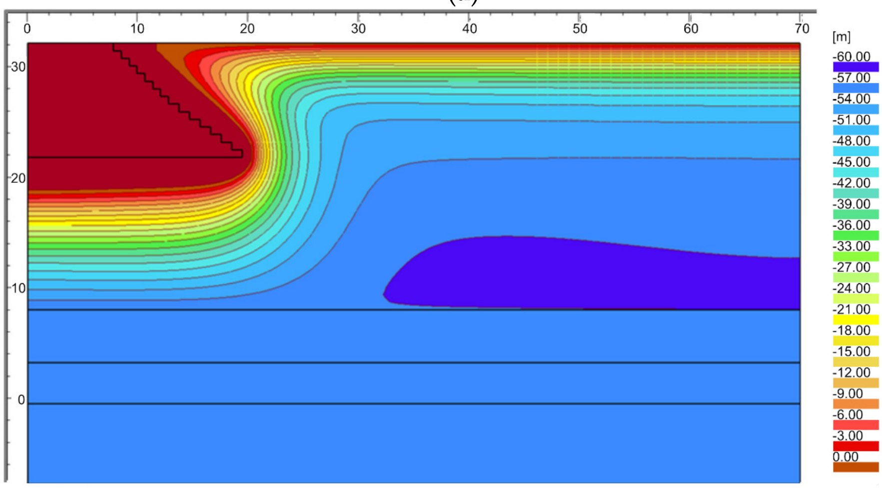


(**c**)

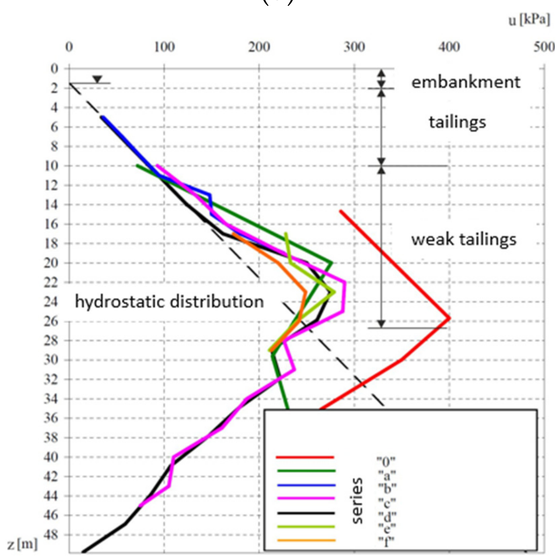
Figure 13. Displacements in Stage III: (**a**) vertical; (**b**) horizontal; (**c**) total.



(**a**)



(**b**)



(**c**)

Figure 14. Pore water pressure excess: (**a**) at the beginning of Stage I; (**b**) at the end of Stage I; (**c**) actual distributions of pore water pressure (dissipation tests). The dashed line illustrates the hydrostatic pressure distribution.

Analysis of the figure indicates that at the end of Stage I, the degree of consolidation will be 55%, and by the end of Stage II, it will increase to 65%. It is estimated that achieving a consolidation degree of 90% in Stage III would require a longer time (approximately 10 years, Figure 15). The figure also illustrates that the maximum settlements, already after the consolidation process, will be approximately 204 cm for 50 kN/m^2, 260 cm for 70 kN/m^2, and 326 cm for 90 kN/m^2 (Figure 15).

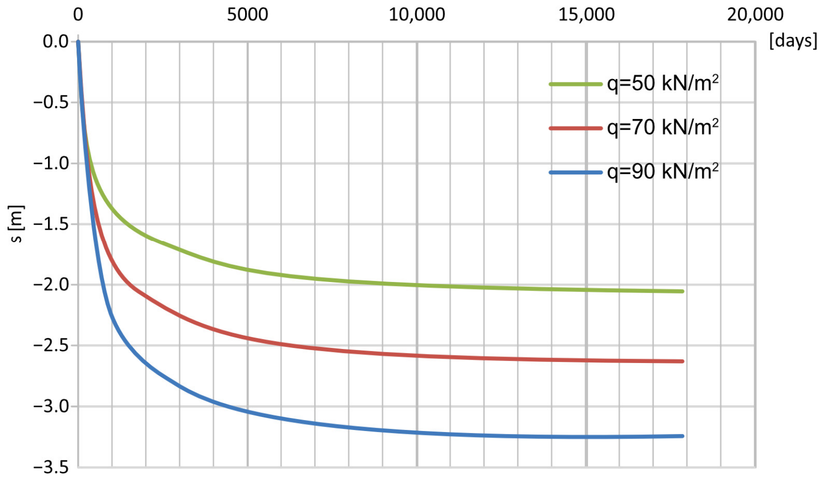


Figure 15. Settlements after the consolidation.

4. Discussion and Conclusions

The ultimate goal of the interpretative procedures for the CPTU investigation is to obtain reliable geotechnical information regarding the analyzed subsoil. Bibliographic analysis indicates that, in the case of CPTU, there are broad possibilities for using the results [4,8,37–42]. Due to the complexity of the methods for interpreting CPTU results, various concepts of identifying geotechnical parameters can be presented, both for natural soils and flotation tailings.

Estimates of soil settlements have been undertaken by many researchers who have pointed out both the advantages and disadvantages of CPTU tests. Lambrechts and Leonards [43] demonstrated that additional loading on the soil can increase the value of *M* by even an order of magnitude, with a slight increase in cone resistance values. Robertson's studies also confirm the above statement, indicating that estimating *M* solely based on cone resistance values can be inaccurate and even subject to significant errors. This is most commonly related to pre-consolidated soils [37]. According to Jamiolkowski, without knowledge of the stress history, it is not possible to accurately estimate the *M* value based solely on cone resistance [44]. Leonards and Frost [45] draw attention to secondary stresses, which, in turn, can contribute to overestimating settlements based on the correlation between cone resistance and *M*. An example of such overestimation is presented by Arroyo et al. [46]. According to the authors, a solution to this problem could involve the application of the local correlation method of CPTU and another test, providing a more accurate estimation of settlements.

Been et al. [3] states that to estimate the parameters necessary for determining settlement values, CPTU tests must be complemented with laboratory tests or other research methods. In contrast to previous approaches, Bastani et al. [38] recommends estimating settlements using the *M* from CPTU tests. The effectiveness of this method is demonstrated by the results presented by the authors—the actual (measured) settlement value was 20.1 mm, while the predicted value based on CPTU was 19.3 mm.

The essence of the issue related to the appropriate selection of the type of investigation for subsoil composed of tailings and loaded with embankment is emphasized by the fact that the reliability of assessing the results of these studies directly influences the determination of the proper parameters of the tailings. A suitable method for a reliable assessment of the geotechnical parameters of post-flotation tailings may be in situ testing using CPTU penetration tests. The usefulness of this test is particularly evident when conducting the following:

– Qualitative assessment, based on the analysis of penetration curve changes;
– Quantitative assessment, parametric assessment.

Both direct and indirect methods show a trend that settlements are greater the closer they are to the center of the pond (Table 6). Additionally, just like in the direct method, the highest settlements of tailings were estimated at the same point in the indirect method. Disproportionately smaller settlements, estimated by the indirect method, may result in immediate deformations, excluding the effect of long-term consolidation. The settlement ratio ($s_{indirect}/s_{direct}$) ranges from 0.60 to 0.76 (points 0+600–0+900), sharply decreasing (from 0.21 to 0.23) as it approaches the center of the pond—points 1+000–1+100 (Table 6). This indicates that estimating settlements for the weakest tailings is the most challenging and requires further research and analysis. It can be assumed that higher settlement values were estimated as a result of reducing M values of tailings under the embankment.

Table 6. A comparison of the settlements of tailings.

Point	CPTU Methods		MCC	Settlement Ratio		
	Indirect	Direct				
		s [cm]		$s_{indirect}/s_{direct}$	$s_{indirect}/s_{MCC}$	s_{direct}/s_{MCC}
0+600	30	50	-	0.60	-	-
0+800	53	70	-	0.76	-	-
0+900	59	90	-	0.66	-	-
1+000	68	300	326	0.23	0.21	0.92
1+100	47	220	-	0.21	-	-

The results obtained by the indirect and direct methods were verified using the MCC model. The model is recommended for analyzing the behavior of even weak soils, including tailings in this case. For the analysis of sandy tailings, for example, deposited on the beach of the facility, the MC model appears to be sufficient. The results obtained using the MCC model confirm the validity of estimating tailings settlements using the independent direct method. In this case, the direct method (s = 300 cm) and the MCC method (s = 326 cm) clearly indicate high agreement (settlement ratio = 0.92, Table 6). Nevertheless, using the indirect method allows for determining the approximate and overall trend of tailings settlement values, in line with the trend determined based on the direct method, especially in the preliminary stage of the analysis. Based on the results of the conducted research, it can be concluded that the CPTU test, recommended for determining the settlement of natural soils, can also be used to estimate the settlement trend of post-flotation tailings.

The conducted analyses led to the following general conclusions being drawn:

– The lack of a single, universal model for the tailings storage facility arises from (1) the zonal structure of the facility on a global scale and (2) the profile heterogeneity of post-flotation tailings on a local scale. It should be noted that there is no universal method for determining the behavior of tailings under variable conditions.
– Interpretation procedures for the CPTU test are intended for analyzing the results of natural soil investigations. To enhance the validity of interpretative procedures, the distinct characteristics of post-flotation tailings, such as alternative correlational dependencies, should be considered.
– The recommended method for identifying tailings in their disposal site may be the CPTU test; however, it is important to emphasize that this is not a universally applicable method.
– Recommended methods for estimating tailings settlements include the following:

   - Based on penetration characteristics from CPTU testing for rigid tailings;
   - Using the direct method based on CPTU and numerical modeling for weak tailings. However, it should be noted that the direct method requires experience.
- It is crucial to consider the timeframe in which the tests were conducted. Firstly, the analysis requires specifying a specific period for conducting the research. Secondly, and more importantly, when conducting a comparative analysis, it is essential to standardize the lengths of the periods during which the tests and observations were carried out.
- Future research directions may involve attempting to integrate the results of different, independent research methods, such as the outcomes of the CPTU test and the Marchetti Dilatometer test.

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