Intensification of the Ash Impoundment and its Stability Analysis

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Abstract. The paper presents specific example of method of intensification of ash impoundment. The impoundment for permanent bedding of ashes is situated on the left side of the Danube river. Part of the dam system of impoundment is based upon the original flood bank of Danube river. At the selection of materials for elevation of the impoundment, the decision was made primarily according to two indicators with an economic aspect – the transportation distance and the increase of the accumulation volume of the impoundment. Finally, the decisive criterion was the occurrence of the failure rate of the ash dam systems in comparison with the dams built from loess soils. As the impoundment is situated in the immediate vicinity of the Danube river, safe and failure-free operation has the priority. There is compared the geotechnical characteristics of the loess soils in the original deposition and the loess soils compacted at optimum water content by Proctor Standard compaction test. The proposed method of intensification of ash impoundment is analyzed. The stability analysis provides an example of the utilization of the results of the monitoring of the geotechnical properties of the materials forming the impoundment’s body and the surrounding rock mass.

1. Introduction

The impoundment for permanent bedding of ashes from the power-producing operations is situated on the left side of the Danube river in the municipality of Štúrovo. The impoundment is partially flown through by surface waters, it is planar, with a circulation system of transport water, with dams on its southern, western and eastern sides. From the north, the impoundment is bounded by a natural slope. The relative position of the impoundment and Danube river is presented in Figure 1 (Google Earth).

The original surface of the terrain was at the ground elevation of 104.8 m above the sea level. The dam system of the impoundment consists of the basic dam with its crest at the elevation of 109.2 m altitude and of elevation dams with crests at elevations of 113.5 m and 116.2 m above the sea level, elevated by a method against water and built from local soils, with inclination of downstream slopes of 1:2.5. The lateral section across the dam system of the impoundment is presented in Figure 2. At the southern side, the dam system of the elevation dams is based upon the original flood bank of Danube. In the direction north – south it is divided by a dam into two cassettes. Deposition into the cassette No.1 was finished in the year 1983 at elevation of 120.0 m above the sea level and at present, only the cassette No. 2 is operated occasionally.
Figure 1. The relative position of the impoundment and Danube river

Figure 2. The lateral section across the dam system of the impoundment

2. Elevation of the ash impoundment via earth embankment

The ash impoundment was intensified in the cassette No.2 by dam elevation from the level of 116.2 m above the sea level to the level of 120.0 m above the sea level, i.e. approximately by 4 m. The slope of the elevation dam was realized with inclination of downstream slopes of 1:2, at the upstream side 1:3 with the crest width of 4 m. At the selection of materials for elevation of the impoundment, the decision was made primarily according to two indicators with an economic aspect – the transportation distance and the increase of the accumulation volume of the impoundment. It was possible to fulfill those requirements in two ways:

- to elevate the dam system by the ash sediment from the impoundment;
- to accomplish the elevation from the soils found in the slope of the northern boundary of the impoundment.

Finally, the decisive criterion was the occurrence of the failure rate of the ash dam systems in comparison with the dams built from soils. At the elevation of the impoundment, as fill materials there were used the soils excavated from the borrow-pit forming the northern boundary of the cassette No.2. As the impoundment is situated in the immediate vicinity of the Danube river, safe and failure-free operation has the priority.

3. Properties of the soil being built in

For mining of soils to the elevation dam, in the northern part of the cassette No.2 a borrow-pit was opened, with a total area of 4300 m² with the mining depth of 7 to 9 m. The mining was preceded by exploration works which verified the geotechnical properties of soils and considered the suitability of
their usage for the future ground construction [1], [2], [3]. In the site of the prospective borrow-pit, three exploration wells were dug (to the depth of ~ 3 m) and, in its immediate vicinity, one drilled boring (to the depth of ~ 4.5 m), with a withdrawal of 6 disturbed and 4 non-disturbed samples of soils.

3.1 Grain size distribution and index characteristics of soil
The prospective borrow-pit consisted of loess soils – silts and sandy silts classified as silts with low plasticity (8 samples), respectively, of sandy silts (2 samples) of rigid to hard consistency. From the grain size distribution (Figure 3) it follows that the soils contain 8-18% of clayey grains, 52-64% of silt particles and 18-40% of sands. The determined geotechnical characteristics of the borrow-pit soils are given in Table 1.

![Figure 3. Grain size distribution of soils in the borrow-pit](image)

The contents of carbonates specified in the laboratory ranged between $O_{an} = 28-31\%$. The contents of organic substances is small, within the interval of $O_{om} = 0.25-0.39\%$, t. e. it does not reach the limit norm value of 5% (soils with organic substances contents > 5% are not appropriate for the fillings). Another positive fact was shown by the tests of shrinkability. The soils are practically non-shrinkable, they are volumetrically stable, shrinkage ratio $s = 0.231-0.297\%$.

3.2. Mechanical properties of the soils
In the course of building of the ground construction it is very important to determine by experimental measurements the parameters of compactibility ($\rho_{d,max}$; $w_{omc}$) of the soils found in the borrow-pit. The tests of compatibility were performed by the Proctor-Standard method [4] at six soil samples (two samples each from every borrow-pit well). The value of the optimal moisture content ranged between $w_{omc}=13.2-15.0\%$ and the achieved maximum dry density $\rho_{d,max}=1725-1800\text{kg.m}^{-3}$. From all six measurements, the average parameters of soils compactibility $w_{omc} = 13.5\%$; $\rho_{d,max} = 1750\text{kg.m}^{-3}$ (Figure 4) were determined.

The parameters of shear strength of soils that determine the feasibility of the stability prognoses were determined by the direct shear tests [5] (each test on 4 testing samples, dimensions 60/60/25 mm), at actuation of a normal stress 20-250 kPa and speed of direct shear testing 0.00975mm.min$^{-1}$. The shear strength of soils was determined on four samples, prepared by compacting by Proctor-Standard energy at approximately optimal moisture content, and on two non-disturbed soil samples in the original bedding. The average parameters of the shear strength of the compacted samples of soils were $\varphi_{ef} = 33.1^\circ$; $c_{ef} = 16.3$ kPa; and of the soils in original bedding $\varphi_{ef} = 28.0^\circ$; $c_{ef} = 13.3$ kPa (Figure 5). The influence of compacting is evident, the measured average angle of internal friction has increased by $5.1^\circ$ and the average cohesion by 3.0 kPa.
Table 1. Geotechnical characteristics of soils in the borrow-pit

| Variable / dimension | Soils in the original bedding (low plasticity silts - 4 samples) | Compacted soils (low plasticity silts - 4 samples sandy silts - 2 samples) |
|----------------------|---------------------------------------------------------------|---------------------------------------------------------------------------|
|                      | Average | Value span | qty          | Average | Value span | qty          |
| $\gamma_s$ [kN.m$^{-3}$] | 17,4    | $\langle$16,6;18,2$\rangle$ | 3            |          |            |              |
| $\gamma_d$ [kN.m$^{-3}$] | 15,7    | $\langle$15,0;16,3$\rangle$ | 3            |          |            |              |
| $\rho_s$ [g.cm$^{-3}$] |          | 2,67       | 1            |          |            |              |
| $w$ [%]                | 12,3    | $\langle$10,9;14,7$\rangle$ | 4            | 10,2    | $\langle$7,6;12,9$\rangle$ | 6            |
| $n$ [%]                |          | 43,8       | 1            |          |            |              |
| $w_p$ [%]              | 22,6    | $\langle$22,1;23,1$\rangle$ | 4            | 22,7    | $\langle$21,7;23,5$\rangle$ | 6            |
| $w_L$ [%]              | 26,3    | $\langle$25,0;28,6$\rangle$ | 4            | 25,6    | $\langle$24,3;27,2$\rangle$ | 6            |
| $I_p$ [%]              | 3,7     | $\langle$2,6;5,8$\rangle$ | 4            | 2,9     | $\langle$1,9;4,3$\rangle$ | 6            |
| $I_c$ [-]              | 4,2     | $\langle$2,4;5,5$\rangle$ | 4            | 5,6     | $\langle$4,0;7,6$\rangle$ | 6            |
| $O_m$ [%]              |          | 0,32       | 3            |          | $\langle$0,25;0,39$\rangle$ | 3            |
| $O_o$ [%]              |          | 29,4       | 3            |          | $\langle$28,4;30,9$\rangle$ | 3            |
| $s$ [%]                |          | 0,268     | 3            |          | $\langle$0,231;0,297$\rangle$ | 3            |
| $k_{o-10^C}$ [m.s$^{-1}$] | 2,2.10$^{-9}$ | $\langle$0,89;3,37.10$^{-9}$ | 3            |          |            |              |
| $w_{opt}$ [%]          |          | 13,5       | 6            |          | $\langle$13,2;15,0$\rangle$ | 6            |
| $\rho_{d_{max}}$ [kg.m$^{-3}$] | 1750 | $\langle$1725;1800$\rangle$ | 6            |          |            |              |
| $\varphi_{ef}$ [%]     | 28,0    | $\langle$27,1;28,9$\rangle$ | 2 tests | 33,1    | $\langle$31,6;35,4$\rangle$ | 4 tests |
| $c_{ef}$ [kPa]         | 13,3    | $\langle$6,7;19,9$\rangle$ | n=8          | 16,3    | $\langle$8,9;20,2$\rangle$ | n=16       |
| $E_{mod}$ [MPa]        | 7,8-13,6 | $\langle$7,2;13,6$\rangle$ | 2            | 10,7-20,2 | $\langle$9,0;22,5$\rangle$ | 6            |

Figure 4. Determination of average parameters of soils compactibility
Figure 5. Determination of average parameters of shear strength

The deformation properties of soils were determined in oedometers [6] with a diameter of $\phi = 100.0\text{mm}$ and sample height of $30.0\text{mm}$ at three samples prepared by compacting by Proctor-Standard energy at approximately optimal moisture content, and on one non-disturbed soil sample in the original bedding. The stepwise additional load of the compacted samples was done in steps of $0 \rightarrow 50 \rightarrow 150 \rightarrow 350\text{kPa}$, and at the non-disturbed sample in the original bedding in steps of $0 \rightarrow 25 \rightarrow 75 \rightarrow 175 \rightarrow 375\text{kPa}$. The average values of the oedometric modules of the compacted samples and of the non-disturbed sample of soil are evaluated in Figure 6. The deformation characteristics will be influenced by compacting positively; the oedometric modulus of the compacted samples will increase by 1.3-1.5-multiple of the oedometric modulus of the non-disturbed soil in original bedding.

Figure 6. Determination of average oedometric modules
4. Technologic procedure of building a soil construction
At incorporation of the soils into the elevation dam of the impoundment, at the beginning a smooth-surfaced vibratory roller VV 1100D was used (weight 11495kg; vibration amplitudes 1.85/1.15kN; vibration frequencies 32/35Hz). Later, compacting was accomplished by a taper foot (trader) put-on of the smooth roller. The soil has been fed into the elevation body at natural moisture, since the difference between the average value of moisture of the samples taken of the borrow-pit (w = 10.2-12.3%) and the average value of optimal moisture content (w_omc = 13.5%) was only 1-3%. The thickness of the layer was 50cm at the beginning, and later on the basis of the results of the compacting process in situ it has been modified to 40cm. The number of roller travels through one layer was 8-10 at the maximum value of vibration. In the initial phase of dam elevation, a compacting experiment was accomplished in situ, to verify effectiveness and the necessary number of travels of the used compacting instrument to achieve the required degree of compacting (the project required a degree of compacting of D ≥ 92%).

5. Defining the procedure and input parameters of the stability analysis of the impoundment
For a stability analysis of an impoundment, the profile at the southern side was chosen. In this place, the dam system of the elevation dams is based upon the original flood bank of Danube. In the calculations by the GEO 2017 software [7], the Petterson [8], Bishop [9] and Spencer [10] slice methods for cylindrical slip surfaces and the Sarma [11] and Spencer [10] method for polygonal slip surfaces were applied. The stability analysis of the entire impoundment is performed per partes – the foot of the dam system, the centre of the dam system with a new elevation, and the entire dam system. For the selected profile of the impoundment and its individual parts defined, a set of cylindrical and polygonal slip surfaces was optimized, the most unfavorable of which (critical slip surfaces) are depicted in the profile of the impoundment in Figures 7a, b.

![Figure 7a. Selected profile of the impoundment – the critical cylindrical slip surface](image)

![Figure 7b. Selected profile of the impoundment – the critical polygonal slip surfaces](image)
The subsoil of the impoundment consists of fine-grained sediments of quaternary age. The soils, which by their occurrence in the subsoil significantly affect the stability relations of the impoundment, represent the contact of the impoundment body with the subsoil. These are fine-grained loess soils – silts and sandy silts classified as silts with low plasticity of ML, MS, SM types. The elevation stages of the dam system were built by upstream method – „against the water” (Figure 2). The basic dam and elevation dams of impoundment are built from local surface fine-grained quaternary loess soils of the same types such as the built subsoil. This fact is confirmed by all survey works carried out on impoundment. The decisive characteristics for the stability of the impoundment being considered are the parameters of the shear strength of the deposited ashes, the soils in the dam system, and the soils at the contact of the impoundment body with the subsoil. The average values of the parameters of the shear strength of the sedimented ashes and soils of the dam system and subsoil are specified from the results of the direct shear tests evaluated from the regular monitoring stages performed during 2003 – 2004 [1], [2]. The average peak shear strength parameters are expressed from the average line of strength by statistical processing of all the measured data on Figure 8 for soils of the dam system and subsoil and on the Figure 9 for sedimented ashes.

Figure 8. Expression of the average peak shear strength parameters of dam system soils and subsoil

Figure 9. Expression of the average peak shear strength parameters of sedimented ash
To summarize, all the parameters of the geomaterials considered in the stability analysis are expressed in Table 2.

| LAYER                     | AVERAGE PARAMETERS |
|---------------------------|--------------------|
|                           | $\gamma_n$ [kN.m$^{-3}$] | $\gamma_{sat}$ [kN.m$^{-3}$] | $\phi_{ef}$ [$^\circ$] | $c_{ef}$ [kPa] |
| SOIL OF ELEVATION DAMS    | 20.0               | 21.5              | 24.0             | 13.0         |
| SEDIMENTED ASH            | 14.0               | 15.5              | 34.0             | 0.0          |
| SUBSOIL                   | 20.0               | 21.5              | 27.0             | 15.0         |

The limit water level is considered according to the valid Handling Regulation and the Measurements Project. In the stability analysis of the impoundment, the influence of the seismic load corresponding to the normative seismic load of the given locality is also considered. According to the map of the areas of seismic load on the territory of Slovakia [12], the region of the impoundment belongs to an area of seismic acceleration with a value of $a_{R} = 1.1$ m.s$^{-2}$. Upon consideration of the stability of the ground slopes by means of so-called simplified pseudo-static methods, the seismic effects are introduced into the calculations by using a so-called factor of horizontal acceleration $K_h = a/g$, where $a$ is seismic acceleration, and $g$ is the acceleration of gravity 9.807 m.s$^{-2}$. The seismic effects are accepted in the calculations by means of the factor of horizontal acceleration $K_h = 0.11$.

6. Results and discussions of the stability analysis of the impoundment

According to Eurocode 7 [13], it should be verified that limit states are not exceeded. In a stability analysis of an impoundment, the limit state is the failure of the dam slope’s system. When considering the limit state of the rupture of a structural element or section of soil, it should be verified that the design value of the corresponding resistance is more than the design value of the effects of all the actions. The current Slovak standard for impoundment [14] requires the expression of the stability of an impoundment through a safety factor. The resulting critical slip surfaces, selected from the overall computing set of the surfaces (after optimization of the calculations) for the selected profile of the impoundment dam system examined are presented in Figure 9 and Figure 10. The respective safety factors for the critical cylindrical slip surfaces in Figure 3 and for the critical polygonal slip surfaces in Figure 4 are presented in Table 3.

| SLIP SURFACE No. | SAFETY FACTOR - $F_s$ WITH LIMITING WATER LEVEL |
|------------------|-----------------------------------------------|
| CYLINDRICAL SLIP SURFACES | POLYGONAL SLIP SURFACES |
| 1                | 2.47 / 2.74 / 2.73 / 2.67 / 2.65 |
| 2                | 1.64 / 1.90 / 1.88 / 1.79 / 1.81 |
| 3                | 1.94 / 2.13 / 2.12 / 1.96 / 2.01 |
| 1*               | 1.54 / 1.72 / 1.73 / 1.72 / 1.70 |
| 2*               | 1.12 / 1.32 / 1.31 / 1.27 / 1.27 |
| 3*               | 1.23 / 1.37 / 1.38 / 1.31 / 1.32 |

LEGEND : Petterson / Bishop / Spencer method for cylindrical slip surfaces
Sarma / Spencer method for polygonal slip surfaces
* - seismic load - factor of horizontal acceleration $K_h = 0.11$

Required safety : $F_{s,min} > 1.5$ $F_{s,min} > 1.1$
The safety factors for the limit water level are for the alternative with the average parameters of the shear strength of the geomaterials $F_S = (1.64 ; 2.74)$ (required safety $F_{S,\text{min}} > 1.50$). At the same time, the safety requirements are fulfilled at the seismic load of $F_{S,\text{min}} > 1.1$ for the average parameters of the shear strength of the geomaterials and for the limit water level in the impoundment body $F_S = (1.12 ; 1.73)$.

7. Conclusions

Impoundments are technical works influencing the quality of environment, therefore they demand increased attention from the viewpoint of the whole society. The construction of elevation of dams of cassette No.2 of the ash impoundment is an example of professional approach and cooperation of the investor, project engineer and, most of all, the contractor. The quality of the extensive ground works is documented not only by the final state, but also by the results of the proven and control checks of compacting of soils built into the elevation of the dam of the ash impoundment, as well as its further smooth and trouble-free operation. The safety of an impoundment is expressed on the basis of its stability analysis. Its feasibility is conditioned by the aptness of the computing model of the impoundment, which must realistically describe not only the constructional arrangement but also the properties of the geomaterials of the construction of the impoundment and the surrounding rock mass. The information database of the properties of the impoundment, expressed on the basis of the results of regular monitoring stages, is the basic input for the creation of a computing model, taking into account the construction and material composition of the impoundment.

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References

[1] M. Masarovičová, I. Slávik, “Geotechnical survey of the KAPPA a.s. Štúrovo impoundment“, STU Bratislava, Slovakia, p. 54, 2003, (in Slovak).
[2] M. Masarovičová, I. Slávik, “Assessment of selected geotechnical properties of the soil for intensification of the KAPPA a.s. Štúrovo impoundment“, STU Bratislava, Slovakia, p. 42, 2004, (in Slovak).
[3] I. Slavík, “Monitoring of the geotechnical properties of geomaterials of the impoundments“, Habilitation thesis, STU Bratislava, Slovakia, p. 173, 2012, (in Slovak).
[4] Slovak standard – STN 72 1015 – Laboratory determination of compactibility of soils, p. 16, 1988, (in Czech).
[5] Slovak standard – STN 72 1030 – Laboratory direct shear box drained test of soils, p. 20, 1987 (in Czech).
[6] Slovak standard – STN 72 1027 – Laboratory determination of soil compressibility in the oedometer apparatus, p. 24, 1983, (in Czech).
[7] GEO 2017 – user guide manual – FINE – Civil Engineering Software.
[8] K.E. Petterson, “The early history of circular sliding surfaces“, Geotechnique 5, p.275–296, 1955.
[9] A.W. Bishop, “The use of the slip circle in the stability analysis of slopes“, Geotechnique 5, p. 7-17, 1955.
[10] E. Spencer, "A Method of analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces", Geotechnique 17, p. 11-26, 1967.
[11] S.K. Sarma, “Stability analysis of embankments and slopes“, Geotechnique 23, p. 423-433, 1973.
[12] STN EN 1998-1 Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings.
[13] Eurocode 7 : Geotechnical design, Part 1 : General rules.
[14] Slovak standard – STN 75 3310 – Impoundment (1991) (in Slovak).