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Variability of Soil Strength Parameters and its Effect on the Slope Stability of the Želazny Most Tailing Dam

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***Abstract*—**The Želazny Most tailing pond is one of the largest facilities worldwide for waste disposal from the copper mines located in South-West Poland. A potential failure of the dam would allow more than 10 million cubic meters of contaminated slurry to flow to the valley, causing immense environmental problems to the surrounding area. Thus, the determination of the strength properties of the dam's soils and their variability is of utmost importance.

An extensive site investigation consisting of more than 480 cone penetration tests (CPTs) with or without pore water pressure measurements were conducted within a period of 13 years to study the mechanical properties of the tailings body. The present work investigates the point variability of the soil strength parameters (effective friction angle φ') of the three soil layers consisting the dam's slope as well as the spatially-averaged variation of ϕ' over the slope's slip surface; this information is based on the well-established relationships between the CPT measurements and the strength parameters from literature. The study showed that the friction angle of the tailings exhibits smaller variation than the natural soils, which is consistent with the literature. As far as the spatial variability, the top and medium layers show a small fluctuation scale, while the bottom layer shows a ten-time greater correlation among its measurements.

The effect of the soil variability on the stability of the dam is determined by performing a probabilistic analysis on a dam's cross-section, while using the available knowledge about the soil properties of the dam.

***Keywords*—**Soil strength variability, friction angle spatial variability, Želazny Most tailing dam.

I. INTRODUCTION

THREE copper mines exist in Lower Silesian Voivodeship (province) in Poland – see Fig. 1 – to explore the great ore body, namely the Lubin, the Rudna and the Polkowice - Sierszowice mine.

The separation of the copper minerals from the rest of the ore is performed at the mill using the flotation method. According to KGHM's website [1], about 4 – 6 % of the weight of the extracted ore is copper. The great majority of the extracted material is waste, called tailings that contains heavy metals and other contaminants. Since no large-scale application for the tailings has been found so far, the waste has to be safely deposited on the ground surface; this purpose for the three KGHM mines is served by the Želazny Most tailing pond.

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The tailing dam is constructed according to the upstream method; a starter dike is constructed and then the tailings are discharged in the perimeter of the structure using spigots or cyclones. As the tailing slurry is released, the coarser materials settle quickly forming the new perimeter dike and the wide beach area, while the finer material moves along the beach to the pond. Each new perimeter dike is founded on the currently existing dam beach. Thus, the height of the dam is continuously increasing, while the dam's area remains the same throughout the structure's life. Part of the cleaned water of the tailing pond is send back into the mill.

The Želazny Most tailing dam started operating in 1977 [1]. Since then, it receives approximately 80,000 tonnes of tailings in liquid form per day, causing its height to increase by about 1.25-1.50m per year [2]. At its current state the dam's perimeter has a length of 14.3km and the area occupied by the structure is about 13.94 km2 [1]. Reference [2] states that the dam's crest is around 170m above sea level and that it is 22-60m poked out from the ground natural terrain, forming the largest tailing dam in Europe. The tailing dam is a key feature of the mining process because incapacity of the dam to receive the waste will result to an abrupt stoppage of the whole mining activity, with great economical consequences to the mine owner. At the same time, a failure of the tailing dam can cause huge environmental problems and jeopardise the health of the nearby town population.In the present study, the variability of the strength properties of the dam's body soil layers in time and space is investigated and its impact on a cross-section's slope stability is determined.

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Fig. 1 Location of the Želazny Most tailing pond (from Google maps); the Lubin town, the Polkowice town and the Rudna region are also shown

II. Input Data

A. Geological profile

The tailing dam is divided into 116 different cross-sections based on their geology. Fig. 2 shows the geological profile of one cross-section (cross-section no.19) whose location is highlighted by a thick box in Fig. 3. Similar profiles can be observed for the majority of the cross-sections.

As can be seen in Fig. 2, the tailing dam is founded on a complex sequence of geological layers. Based on [2], the first 100m consist of Pleistocene deposits (lake clays, sand interbedded with silt and clay and gravel), while Pliocene deposits are encountered in higher depths (thick plastic clays and thin brown coals). With regard to the dam's body, the starter dike that is shown in very dark grey in the bottom right part of the slope (Fig. 2) exhibits similar properties with the white top soil layer of the slope since both consist of the coarser materials that are released from the spigots (soil layer A). Moving towards the centre of the structure, the soil layers alter from sand to silty sand and clayey silt, forming the light grey (soil layer B) and the dark grey (soil layer C) soil layers of Fig. 2; this happens because the finer materials can travel longer distances before settlement.

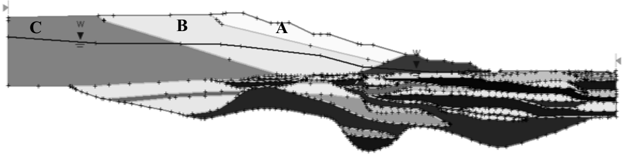

Fig. 2 The geological profile of cross-section no. 19

B. Cone penetration tests

Cone penetration tests with or without pore water pressure measurements (CPTU/CPT) were performed in more than 480 boreholes in the Želazny Most tailing pond within a period of 13 years (1995-2008). The majority of the test locations was at the slope of the dam, even though tests on the beach and over the perimeter of the dam were carried out. The locations of the CPT can be seen in Fig. 3 marked with black dots.

From the extensive dataset, only the CPTs located close to the centre of each cross-section and matched perfectly with the cross-sections' topography were selected for further analysis. Finally, 125 CPTs located close to 33 cross-sections were selected as the input dataset – noted in light grey dots in Fig. 3.

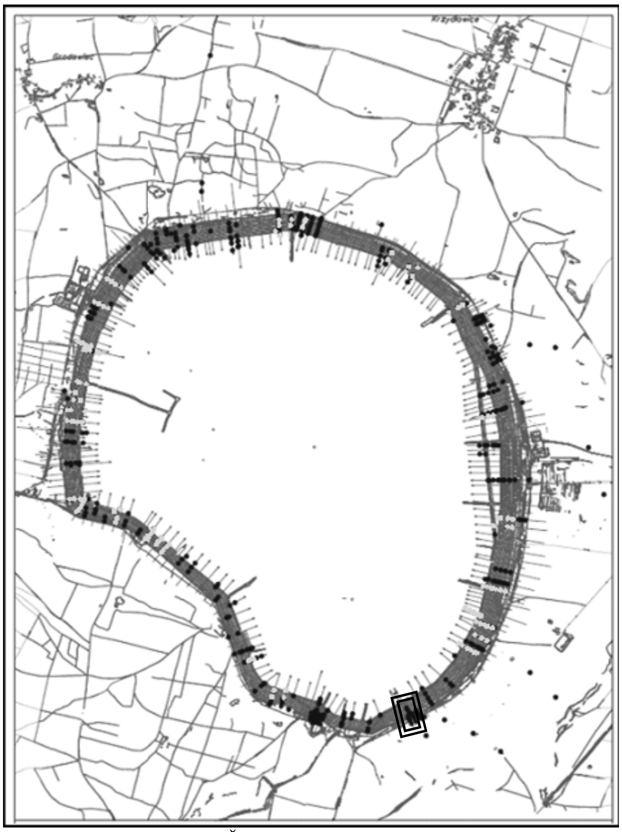

Fig. 3 Plan view of the Želazny Most tailing dam – the black dots show the locations of the CPTs performed and the light grey dots show the locations of the CPTs used in further analysis

III. Point-Variability of Strength Parameters

To describe the strength of the three granular soil layers under study, namely soil layers A, B and C, the effective friction angle φ' is required.

For each soil layer, all measurement points (CPTs are performed every 2cm in vertical direction) are firstly classified according to their soil type. Reference [3] provides a classification system of post-flotation sediments especially developed for the case of the Želazny Most tailing dam; this classification is based on the comparison of CPTs with laboratory results relevant to the dam's area. Indeed, they define a boundary line in terms of quantities measured at the CPTs; soils that lay above the boundary (fine sand, silty sand and sandy silt) are classified as granular, while the rest (silt and clayey silt) are classified as cohesive. The boundary line equation is given by (1).

$$f_s = 7.88q_c + 0.32q_c^2 \tag{1}$$

where f_s the sleeve friction in KPa and q_c the cone resistance in MPa.

For the measurement points that belong to the granular group, φ' can be determined through the next two steps. Firstly, the relative density D_r as a function of cone resistance q_c and total vertical stress σ_{vo} is given by (2) based on [4].

$$D_r = a_1 + b_1 \cdot \log(q_c) + c_1 \cdot \log(\sqrt{\sigma_{vo}}) \tag{2}$$

where a_1, b_1, c_1 are constants.

Secondly, the friction angle is determined according to (3) provided by [5].

$$\phi = a_2 + b_2 \cdot D_r \tag{3}$$

where a_2, b_2 are constants.

The values of the constants in (2) and (3) can be determined according to a diagram provided in [4] once the friction ratio R_f (=f_s/q_c) is known. The values of q_c and sleeve friction f_s are provided by the CPTs, whereas the total vertical stress is estimated by assuming a constant value of soil unit weight equal to γ=19kN/m^3 – the results are practically insensitive to the soil's unit weight.

For the clayey soils, there is extensive research that relates the CPT results with the undrained strength S_u of the clayey soils but nothing has been done in terms of the strength under drained conditions. Thus, the CPT tests were transformed to standard penetration test (SPT) results. The SPT energy ratio N_{60} has been estimated from the cone resistance q_c derived by the CPT test according to (4).

$$N_{60} = \left(\frac{q_c}{p_a} \right) / a \tag{4}$$

where α is a coefficient equal to 2.5 for sandy silt to clayey silt and p_a is a reference stress equal to 0.1MPa [6].

The SPT value is further corrected to take account of the overburden stress [7]:

$$N_{1,60} = N_{60} \cdot \sqrt{\frac{95.76}{\sigma'_{vo}}} \tag{5}$$

where σ'_{vo} the overburden pressure in kPa.

To finally determine the friction angle, the graph proposed by [8] that relates φ' with the $N_{1,60}$ has been utilized.

The aforementioned procedure has been applied to the 122,714 measurement points of the more than 2.4 km of CPTs performed. The results of the statistical analysis of the CPTs for the three different soil layers are shown in Table I. The mean friction angle determined ranges from 34^0 to 36^0. Moving towards the centre of the structure, the mean value of φ' is reduced from 36^0 to 34^0 as the soil layers transform from sand to clayey silt. The coefficient of variation is much less than the one proposed from literature – which is 10% for natural soils [9]. This is in accordance to the statement given in [10]: 'There is usually little variation between φ' for sands and slimes tailings. This is because the slimes are generally composed of ground-up hard minerals and so they are angular and no flat'.

Tabulated in Fig. 4 are the histograms of the granular and of the clayey effective friction angles for the three soil layers based on the CPT statistical analysis.

TABLE I
RESULTS OF THE STATISTICAL ANALYSIS OF CPTS

soil layer	mean φ' (°)	CV_{φ}	no. of points
A	36.0	8%	35965
B	34.4	5%	49009
C	34.0	6%	37740

Fig. 5 presents the histogram of the friction angle for the two soil types combined. It is obvious that the granular soils present mean values of φ' around 35^0 and very small variation (3-5%). This is not the case for the histogram of the clayey soils where high deviations from the mean value are observed (CV=8-11%). At the combined histogram of Fig. 5, the high proportion of granular soils over clayey (clayey soils are met at about the 17% of the cases) dominate the histogram, leading to smaller coefficients of variation than the ones observed at the clayey soils.

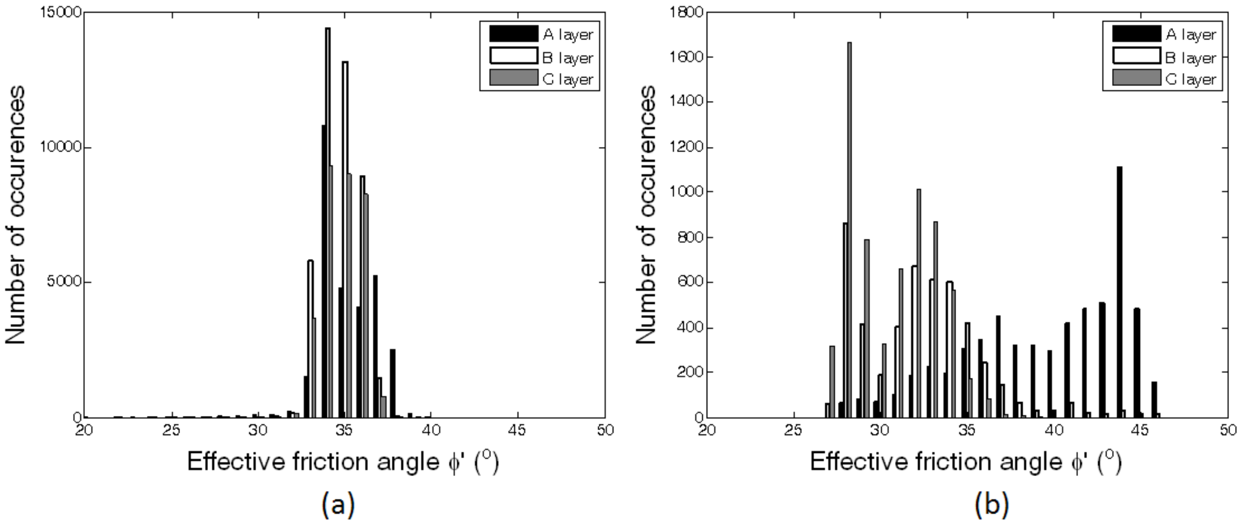

Fig. 4 Histogram of the effective friction angle for the cases of (a) the granular measurement points and of (b) the clayey measurement points for the three soil layers based on the CPTs

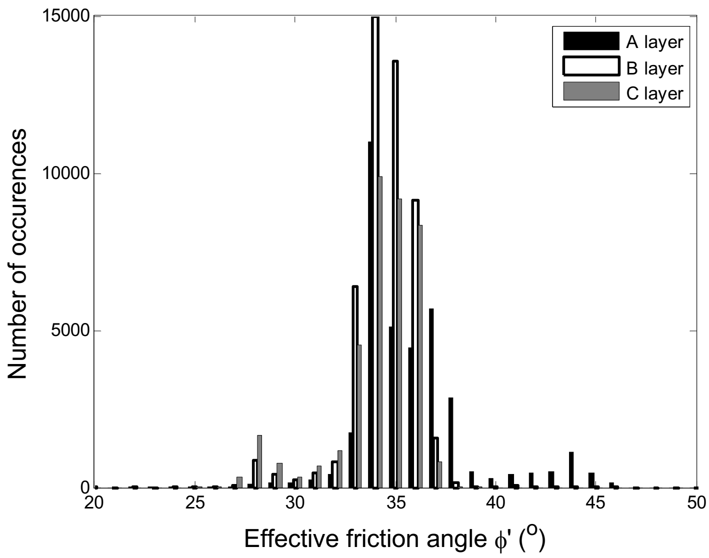

Fig. 5 Histogram of the effective friction angle for the three soil layers based on the CPTs

It should be noted that the results obtained are based on all the 125 CPTs together even though they have been conducted within a great time spam of 13 years. This is based on the fact that the static loading of the tailings with time by the slurry release cannot compact the granular soil layers by a significant amount and thus does not affect greatly the effective friction angle. Indeed, as is shown in Fig. 6, the effective friction angle is almost insensitive to the time the CPTs were conducted.

The coefficient of variation of each soil layer is increased by 2% to be on the safe side. This increase may accommodate the CPT measurement errors as well as the discrepancies from the equations used. Thus, the final probabilistic characteristics of friction angle for the three soil layers used in the subsequent analysis are given in Table II.

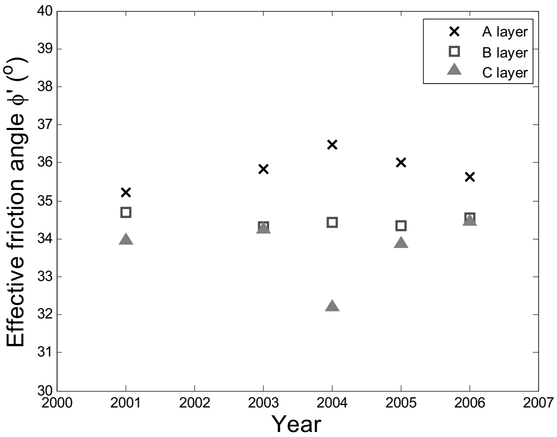

Fig. 6 Mean value of effective friction angle based on the CPTs results conducted within each year

IV. Spatially-Averaged Variation of Friction Angle

In the previous section, the variability of the effective friction angle at a specific point has been estimated. However, the variability of the strength parameters will be used for the determination of the slope stability and thus the variability of the whole cross-section is more appropriate than the variability at a specific point. Indeed, across the failure surface extremely high and extremely low values of φ' can be encountered; but what is actually needed for the analysis is the φ' of the whole failure surface, which intuitively is expected to have the same mean but less standard deviation compared to the point φ'. To take into account this effect, the methodology proposed in [11] is adopted in this section and the results are presented.

Based on [11], the ratio of standard deviation of a parameter X in a specific area A $\sigma(X_A)$ over the point standard deviation of this parameter σ(X) equals Γ(A), where Γ is named the root-mean-squared (rms) reduction factor. The Γ actually is a measure of the reduction of $\sigma(X_A)$ relative to the point σ(X). For the one-dimensional case, the rms reduction factor for a length segment Δz is given by (6):

TABLE II
Results of the Spatially Averaged Statistical Analysis of CPTs

soil layer	mean φ' (°)	CV_φ	δ (m)
A	36.0	10%	51
B	34.4	7%	27
C	34.0	8%	198

$$\Gamma(\Delta z) = \begin{cases} 1 & for\ \Delta z < \delta \\ \sqrt{\dfrac{\delta}{\Delta z}} & for\ \Delta z \geq \delta \end{cases} \tag{6}$$

where δ is the scale of fluctuation of parameter X.

The correlation function ρ(Δz) (essentially the autocorrelation function) of the parameter under study, the effective friction angle in the present analysis, is given by the exponential function of (7).

$$\rho(\Delta z) = \exp\left[-(\Delta z / b)^2\right] \tag{7}$$

where b equals $\delta / \sqrt{\pi}$.

For the selected formula of the correlation function (7), the rms reduction factor over the slip surface Γ(A) can be expressed as the product of two one-dimensional rms factors.

$$\Gamma(A) = \Gamma(\Delta x, \Delta y) = \Gamma(\Delta x) \cdot \Gamma(\Delta y) \tag{8}$$

If isotropy in the horizontal dimension is assumed, then (8) is further simplified to (9).

$$\Gamma(A) = \Gamma(\Delta x) \cdot \Gamma(\Delta y) = \Gamma(\Delta x) \cdot \Gamma(\Delta x) = \Gamma^2(\Delta x) \tag{9}$$

Once the correlation function is determined based on (7), the following formula is used to estimate Γ [11].

$$\Gamma^2(\Delta x) = \left(\frac{b}{\Delta x}\right)^2 \left[\frac{\Delta x}{b}\sqrt{\pi}\Phi\left(\frac{\Delta x}{b}\right) + e^{\left(-\frac{\Delta x^2}{b}\right)} - 1\right] \tag{10}$$

where Φ(·) is the error function.

In the analysis it was assumed that the soil is locally homogeneous in the vertical dimension. So, for each borehole for which the cone penetration test was conducted, the average value of the effective friction angle of the test results that passed through each layer was selected. This mean value for each borehole was then used to estimate the correlation function. It should be noted that the distance among test locations was estimated as the resultant distance of the radial and circumferential distance of test locations.

The least squares method was used to estimate the scale fluctuation δ of the correlation function for each soil layer that best suits the experimental data. The results are given in Table II and illustrated in Fig. 7.

From the results listed in Table II, it is apparent that the top and medium layers show a small scale δ, i.e. the strength parameters fluctuate every about 51 and 27m respectively, i.e. 'two points that lie within this distance are likely to be either both above or both below the average friction angle value' [11]. The bottom layer shows a greater correlation among measurements, since δ is significantly larger.

Fig. 7 presents the correlation function of the effective friction angle for the three soil layers.

It should be noted that the sizes of the markers are analogical to the number of couples used to determine each point. The crooked lines correspond to the experimental data and the dashed lines to the correlation function determined according to (7) that matches best the experimental data. Fig. 7 shows the very abrupt reduction of the correlation function for layers A and B that lead to small δ values and the smoother reduction of ρ(Δz) in the case of the C layer. Fig. 8 represents the variation of the effective friction angle in the horizontal direction for the three soil layers. The first two layers with the small δ fluctuate a lot, while the bottom layer presents more smooth fluctuations.

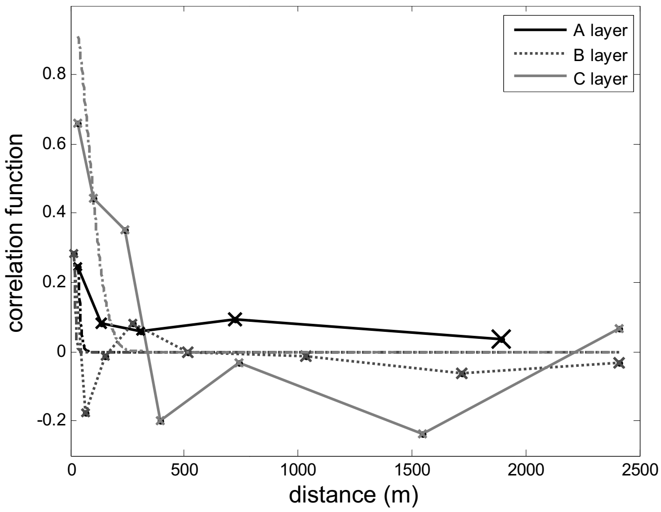

Fig 7 The correlation function of the friction angle for the three soil layers

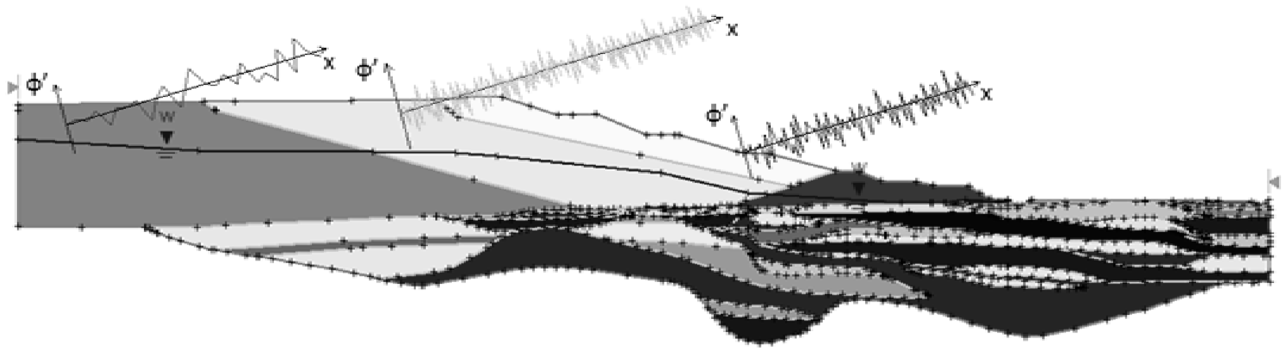

Fig. 8 Representation of the spatial variation of the friction angle for the three soil layers

V. Probabilistic Slope Stability Analysis

A probabilistic analysis was conducted to investigate the slope stability of cross-section no. 19, given the uncertainty of the effective friction angle of the three soil layers. 5000 Monte Carlo iterations were performed to determine the variability of the slope stability safety factor for three uncertainty cases. In the first case, it was assumed that the CPTs are not available and so the friction angle variability is exclusively based on the literature - a coefficient of variability equal to 10% is selected for each soil layer according to [9]. In the second case, the point variability of the effective friction angle shown in Table III is used. Last, the slope stability safety factor is estimated by taking into account the spatially-averaged variability of ϕ' according to [11] and [12]. The water table is assumed to be at its mean elevation as is illustrated in Fig. 2 and the safety factor is estimated according to the Bishop simplified method (Cornforth, 2005).

TABLE III
COEFFICIENT OF VARIATION FOR THE THREE CASES

soil layer	mean φ' (°)	CV_{φ}		
		case 1	case 2	case 3
A	36.0	10%	10%	3%
B	34.4	10%	7%	1.5%
C	34.0	10%	8%	5%

Table IV presents the results of the probabilistic slope stability analysis. The mean safety factor remains the same in the three cases, but the coefficient of variation CV_{SF} of the safety factor decreases from case 1 to case 3 following the reduction of the effective friction angle uncertainty. The probability of failure P_f is about 8% for the first case and is significantly reduced to 0.06% when the spatial-averaged effective friction angle variation is used.

TABLE IV
RESULTS OF THE PROBABILISTIC SLOPE STABILITY ANALYSIS

case	mean SF	min SF	max SF	CV_{SF} (%)	P_f (%)
1	1.055	0.927	1.199	3.7	8.04
2	1.055	0.955	1.164	2.8	3.28
3	1.055	0.980	1.113	1.6	0.06

VI. CONCLUSIONS

In the present study, the strength parameters of the tailings and their spatial variability have been investigated based on a great number of available CPTs. The tailings show variability smaller than the natural soils. The bottom layer fluctuates smoothly in the horizontal direction, while the upper soil layers present more abrupt fluctuations, thus forcing the spatially-averaged friction angle variation to significant reduction.

To illustrate the effect of the friction angle variability to the probability of failure, a probabilistic slope stability analysis was performed. If no CPT data were available, a coefficient of variation of 10% would have led to a non-acceptable value of the probability of failure (8%). The conventional use of the CPT results by estimating the point variation of φ' reduced the probability of failure to 3%. When the friction angle variation is spatially-averaged along the slip surface, the probability of failure is significantly reduced, highlighting the beneficial effect of the concept of statistical homogeneity.

In the present case, the thorough understanding of the effective friction angle's statistical properties through geotechnical investigation led to a computationally estimated substantially larger reliability of the structure, thus minimizing the necessity for stability measures.

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