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SYNOPSIS: The remediation of an inactive uranium mill tailings pile at the town of Andujar (Spain) has provided an opportunity to investigate the settlement characteristics of hydraulically-deposited uranium mill tailings. A test embankment was constructed on top of the existing tailings deposit and total stresses, settlements and pore pressures were measured. Settlements and pore pressure data were compared with the results obtained using an elastoplastic numerical model which allows the simulation of two dimensional consolidation processes. Backcalculated consolidation parameters were derived to provide agreement between the calculated and measured settlements and pore pressures. These parameters could then be used to predict the post-construction settlement of the remediated pile.

INTRODUCTION

The Empresa Nacional de Residuos Radiactivos, S.A. (ENRESA) is conducting the reclamation of an inactive uranium mill tailings disposal site at the town of Andujar in the South of Spain. The existing tailings deposit was placed hydraulically at the millsite which ceased active milling in 1981. The tailings pile was constructed by the upstream method and projects 20 meters above the surrounding ground surface in the central and eastern parts and 10 meters in the western part (Figure 1). For the lower area of the pile, the remedial action involves placement of compacted tailings and a protective cover on top of the existing hydraulically-deposited tailings. Differential settlements can affect the performance of the cover and therefore reliable post-construction settlement estimates are needed.

This paper describes the results and performance of a large-scale test embankment which was constructed on top of the existing tailings pile to determine the consolidation parameters of uranium mill tailings and evaluate the post-construction settlement of the remediated pile.

SITE CHARACTERIZATION

Site characterization activities were conducted to determine site stratigraphy and material properties. Approximately 20 boreholes, 5 cone penetration tests and 30 dynamic penetration tests were completed. Both disturbed and undisturbed samples were obtained along with the penetration test values. Site stratigraphy is generally described as tailings, overlying silty clay (1.8 to 4.0 m thick), sandy gravel (2.0 to 4.0 m thick) and Miocene marls, in descending order. The shallowest aquifer beneath the site lies in the gravel terrace formation.

Grain size and plasticity tests provided data for determining the distribution of the various material types within the tailings. From the data it was concluded that the tailings deposit consisting of sands, silty sands, clayey silts and silty clays is heterogeneous. Tailings sands are located around the perimeter of the deposit. However, the limited dimensions of the pile have permitted the deposit of fine beds of sand even at the central portion of the pile. This creates a banded texture of the tailings where the different types of materials appear finely interbedded.

The variation of interbedded materials is such that at the edges of the pile 60 % of the samples were sands and silty sands and 40 % were clayey silts/silty clays. At the center of the pile still 30 % of the samples were of sandy nature and the remaining 70% were clayey tailings.

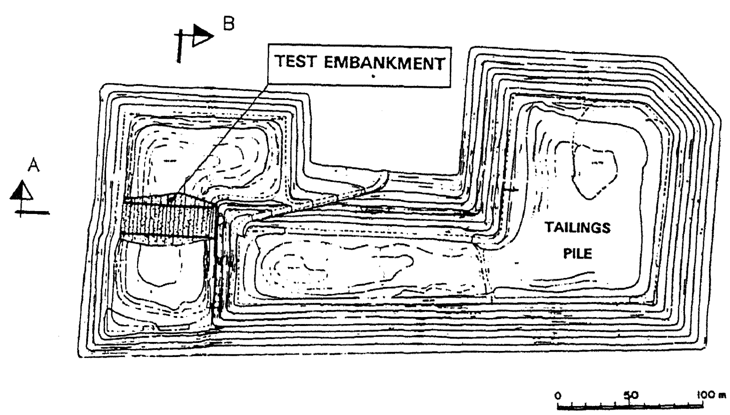

FIGURE 1

PLAN VIEW AND LOCATION OF TEST EMBANKMENT

The depth of tailings at the location of the test embankment varies between 4.0 and 5.5 m and the materials were partially saturated in the upper 2.0 m and nearly saturated at greater depths.

Tailings dry densities were fairly low, 1.36 t/m^3 for the sandy tailings and 0.94 t/m^3 for the clayey tailings, which indicate an open structure of the deposit.

A laboratory testing program was conducted to determine the engineering properties of the materials. The results of the tests and the material properties used in the settlement analyses are shown in Tables 1 and 2.

Total unit weight (γ_t), moisture content (w), degree of saturation (S_r), void ratio (e) and plasticity index (PI) were obtained from identification testing of the materials.

Estimates of the virgin compression index (C_c), recompression index (C_r), preconsolidation pressure (P_c) and coefficient of consolidation C_v) were based on laboratory consolidation testing of undisturbed samples. Estimates of permeabilities (K_v, K_h) were obtained from laboratory and in-situ permeability testing of the materials.

TABLE 1: Index properties

Material Type	γ_t (t/m^3)	w (%)	S_r (%)	e (-)	PI (-)
Silty/clayey tailings	1.65	30-70	60-100	1.0-2.1	10-30
Sandy tailings	1.85	30-40	25-65	1.1	0-10
Foundation soils (silty clays)	2.0	15-25	60	0.5-0.7	30

TABLE 2: Consolidation properties

Material Type	C_c	C_r	P_c (t/m^2)	C_v (cm^2/s)	K_v (cm/s)	K_h (cm/s)
Silty/ clayey tailings	0.45	0.035	12	2.45×10^{-3}	$10^{-6}-10^{-7}$	$10^{-4}-10^{-5}$
Sandy tailings	0.29	--	--	--	5×10^{-4}	5×10^{-4}

TEST EMBANKMENT CONSTRUCTION AND INSTRUMENTATION

In order to observe the consolidation behaviour of the tailings, a test embankment was constructed with a base measuring aproximately 46 meter by 35 meter, and 2(H):1(V) sideslopes. The maximum height of the embankment of 4 meters was reached after 12 days.

To effectively monitor the results of the test embankment an instrumentation network was installed consisting of:

- 24 vibrating wire piezometers
- 9 hydraulic settlement plates
- 5 total stress cells
- 5 inclinometers
- 21 topographic monuments

The basis configuration of the test embankment and the instrumentation is shown in Figure 2.

The piezometers were installed in eight borings, at three levels A (upper), B (middle) and C (bottom), in order to monitor the evolution of the pore pressures.

At the base of the test embankment, nine settlement plates were installed together with five stress cells for measurement of the settlement and applied load during construction and evolution in the post-construction period.

In addition, five inclinometers and 21 monuments were installed for controlling the horizontal and vertical movements of the embankment.

All instrumentation was installed at least one week before construction of the test embankment, in order to allow the instruments to equilibrate before initiating measurements.

Readings were taken on a daily basis for the first 60 days following installation. Once the measurements began to stabilize, reading were spaced further apart and were taken on a weekly basis for the following month and on a monthly basis for the subsequent three months.

The total monitoring period was five months and in the first two months, a strict sequence of readings was maintained.

TEST EMBANKMENT RESULTS

Key aspects of the geotechnical behaviour of the tailings were obtained from the instrumentation data. During the monitoring period, the observations were interpreted and evaluated qualitatively for comparison with prior analitical predictions, bases on best previous estimate parameters.

The scope of the test embankment monitoring directed towards obtaining consolidation parameters of the tailings to reliably predict post-construction settlement of the final disposal embankment. The result of the monitoring program is briefly described below.

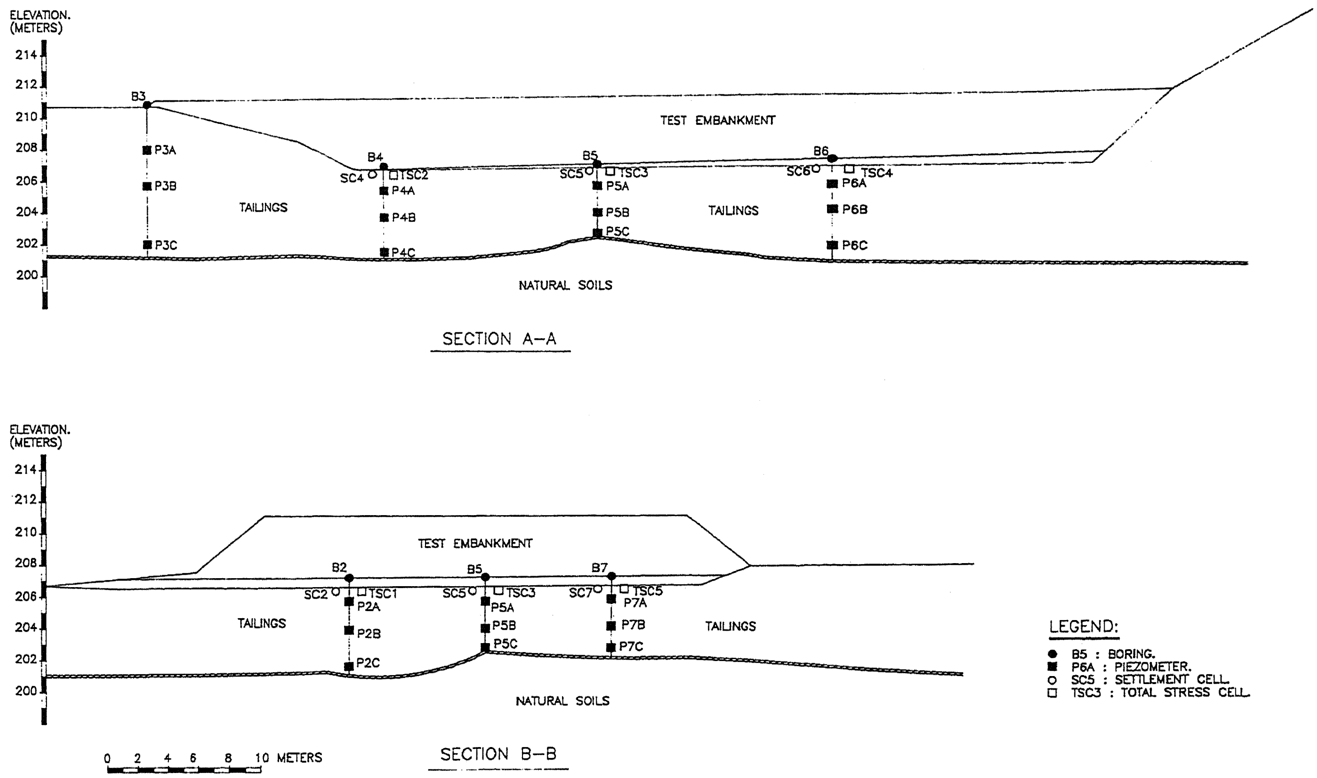

FIGURE 2

TEST EMBANKMENT CROSS-SECTIONS AND INSTRUMENTATION

During the construction of test embankment a record was kept on applied load for each compacted layer by means of in situ density tests and topographic monitoring. The results of the measurements in three total stress cells and the calculated loading rate curve are shown in Figure 3. The actual total stress measurements agreed adequately with the calculated loading rates based on the construction records.

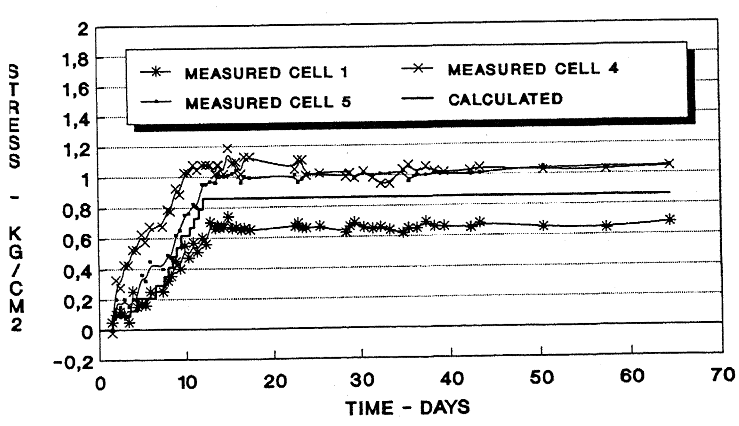

FIGURE 3
COMPARISON OF MEASURED AND CALCULATED TOTAL STRESSES

Figure 4 shows typical results of the evolution of the settlements in the tailings measured in three settlement plates located in a longitudinal cross-section of the test embankment. It should be noted that in general approximately eighty percent of the total settlement measured over the total monitoring period (6 months) was reached after sixty days. Most consolidation settlement in the tailings took place during the construction period of the test embankment.

The dissipation of the pore pressure has proved to occur faster than expected. In general, little variation in excess pore pressure was measured in the piezometers and this was independent of the depth of installation of the instrument. This could be explained due to the banded texture of the tailings, consisting of finely interbedded lenses of sands, silty sands, clayey silts and silty clays. An example of excess pore pressure measurements is presented in Figure 5.

CONSOLIDATION ANALYSIS

The evolution of settlement and pore pressure in uranium mill tailings is largely controlled by the deformation characteristics of the soil together with its permeability. It is important, therefore, to be able to reproduce both the nonlinear behaviour of the material and the coupling between the soil skeleton and the pore water pressure.

Here, a Cam-Clay model described in Roskoe et al. (1968) was used to reproduce soil behaviour. This model is one of the options included in the program CRISP, developed at Cambridge University (UK). The version used in the analysis corresponds to the two dimensional model described in Britto et al. (1987) and it allows the simulation of the embankment construction process. It was implemented in a personal computer, and it was supplemented by several preprocessing and postprocessing programs.

Constitutive model

Cam-Clay is an elastoplastic model with isotropic volumetric hardening. As every plasticity model, it relates the increments of effective stress and strain by :

$$d\underset{\sim}{\varepsilon} = \underset{\sim}{C}_{ep} \cdot d\underset{\sim}{\sigma} \tag{1}$$

where $\underset{\sim}{C}_{ep}$ is a fourth order constitutive tensor, given by:

$$\underset{\sim}{C}_{ep} = \begin{cases} \underset{\sim}{C}_e + \frac{1}{H} \cdot \left(\frac{\delta f}{\delta \sigma} \otimes \frac{\delta f}{\delta \sigma} \right) & if \ \frac{\delta f}{\delta \sigma} : d\sigma > 0 \\ \underset{\sim}{C}_e & if \ \frac{\delta f}{\delta \sigma} : d\sigma \le 0 \end{cases} \tag{2}$$

where $\underset{\sim}{C}_e$ is the elastic constitutive tensor and $f(\sigma, P_c) = 0$ is the yield surface. The plasticflow rule is, therefore of associate type. The scalar P_c gives the size of the surface and is referred to as the preconsolidation pressure.

The elastic behaviour is assumed to be isotropic, and it is characterized by the relations:

$$k = - \frac{\Delta e}{\Delta p / p} \tag{3}$$

$$\nu = Constant$$

where e represents the void ratio and p is the effective confining pressure, given by :

$$p = \frac{1}{3} tr (\sigma) \tag{4}$$

The yield function and plastic potential surface f is defined as:

$$f (\underset{\sim}{\sigma}, P_c) = (q^2 - M^2 p (P_c - p)) = 0 \tag{5}$$

where q is a measure of the deviatoric stress

$$q = \left(\frac{3}{2} s_{ij} s_{ij} \right)^{1/2} \tag{6}$$

$$\underset{\sim}{s} = dev (\underset{\sim}{\sigma}) \tag{7}$$

and M a material parameter which defines a straight line in the p-q plane referred to as the Critical State Line and which corresponds to the residual conditions for a given soil.

The plastic modulus H in equation (2) can be obtained by applying the consistency rule, taking into account that hardening of the soil is assumed to depend only on the volumetric plastic strain:

$$\frac{\delta P_c}{\delta \varepsilon_v^p} = P_c \cdot \frac{(1+e)}{(\lambda - k)} \tag{8}$$

Material behaviour is therefore characterized by five parameters which can be obtained from the laboratory tests.

Numerical analysis

Two representative sections, one longitudinal and one transversal, were analyzed in an attempt to approximate to the three-dimensional behaviour of the test embankment.

The geometry was discretized by quadrilateral elements, with eight displacement nodes and four pore pressure nodes, to avoid incompressibility problems. Four control points were selected, one for the displacement and one for the pore pressure in both longitudinal and transversal meshes. They were used as it will be shown later to compare the predicted displacement and pore water pressure against the measured results.

The construction process was assumed to take place in two weeks, during which dissipation of pore water pressure was allowed.

The soil parameters were selected as follows. The isotropic hardening parameter λ was obtained from oedometer tests, and a value of 0.45 was selected for the tailings. These tests included an unloading branch from which the elastic compressibility k was obtained, selecting a representative value of 0.035 for the tailings.
The initial slope observed in the deviatoric stress versus axial strain curves in undrained triaxial tests was used together with k to get the Poisson's ratio (υ), which was taken as constant during the whole loading process.

Failure or residual conditions are described by the parameter M, which can be obtained from triaxial tests as the ratio between the residual deviatoric stress q and the mean confining pressure p. The value selected was 1.84.

Finally, consolidation tests performed on undisturbed specimens provided the value of the preconsolidation pressure (P_c) to be used in the analysis. The preconsolidation pressure did not vary significantly in depth, and a value of 1.2 Kg/cm2 was adopted for the analysis.

The permeability was assumed to have different values along the horizontal and vertical directions, having chosen

$$K_h = 10^{-6} \text{ m/s}$$

$$K_v = 10^{-8} \text{ m/s}$$

A greater horizontal permeability was used for the analysis due to the presence of sandy horizons closely spaced within the pile.

Figure 4 shows the displacements predicted together with the measurements at the control points. It can be seen that the basic trend has been reasonably well obtained. It has to be noted that approximately eighty percent of the total displacement was produced by the end of the construction process.

Predicted and measured pore water pressures are given in Figure 5. One of the main limitations presented by the numerical model used is that the soil is assumed to be fully saturated, overpredicting the pore water pressure if a nonsaturated material is modelled.

It should be mentioned that all above referred parameters were obtained from laboratory and in situ tests in which an important scattering was observed.

It was considered important, therefore, to evaluate the influence of a possible deviation from the selected values, and a sensitivity analysis was performed.

Concerning the preconsolidation pressure, two simulations with P_c = 1.2 and 1.4 kg/cm2 were performed, and it was found that the predicted results showed a relatively small influence (Figure 6).

Two values of the volumetric hardening parameter λ were selected to evaluate its influence in the predicted response (Figure 7). Here, a larger value of λ caused higher deformation and pore pressures, as expected.

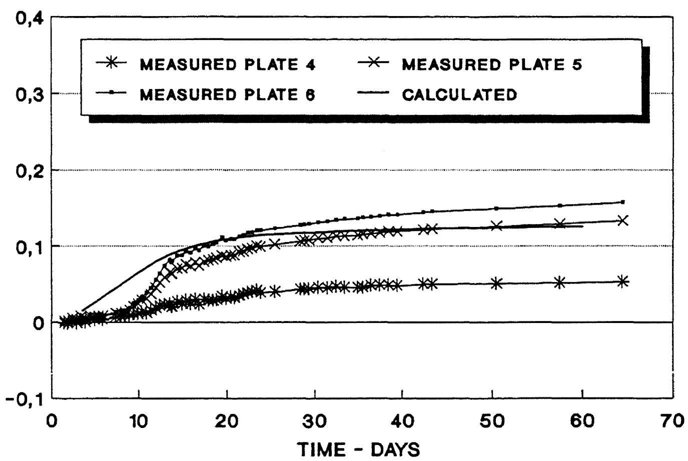

FIGURE 4
COMPARISON OF MEASURED AND CALCULATED SETTLEMENTS

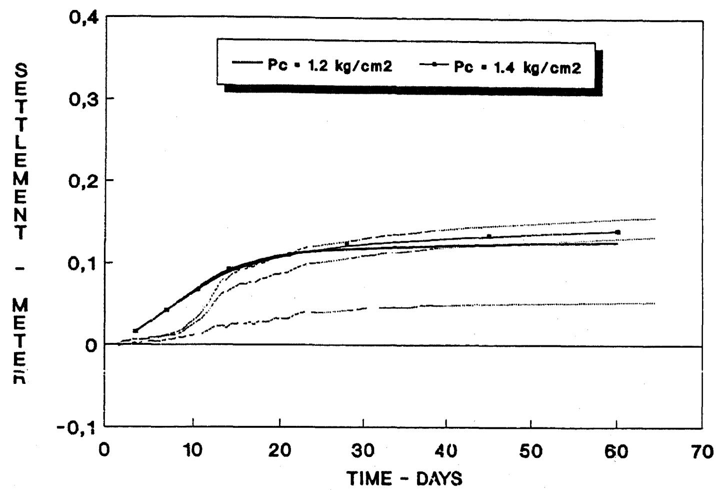

FIGURE 6
SENSITIVITY ANALYSIS - VARIATION OF Pc

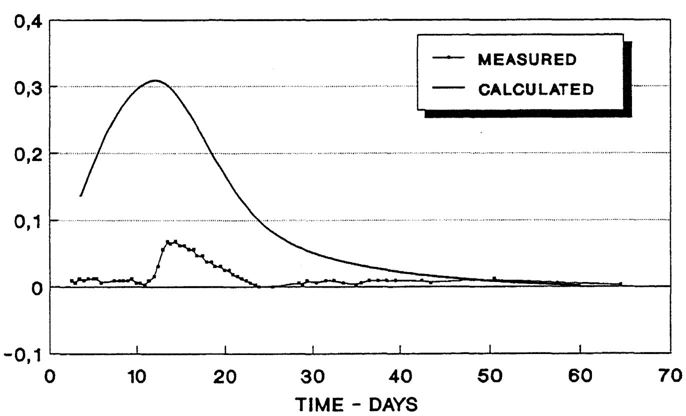

FIGURE 5
COMPARISON OF MEASURED AND CALCULATED EXCESS PORE PRESSURE

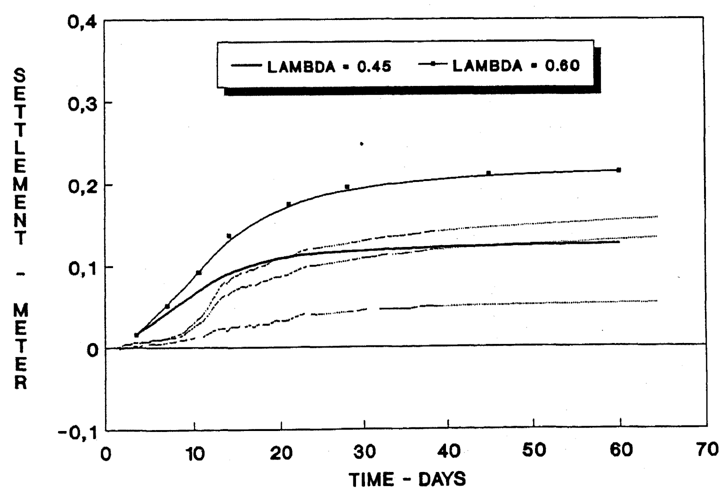

FIGURE 7
SENSITIVITY ANALYSIS - VARIATION OF λ

The key parameters controlling the consolidation were the horizontal and vertical permeabilities, which affected both the peak pore pressure and the shape of the consolidation curve. The simulations showed that the time required to reach a certain degree of consolidation was more affected by a change of permeability than the final settlement (Figure 8).

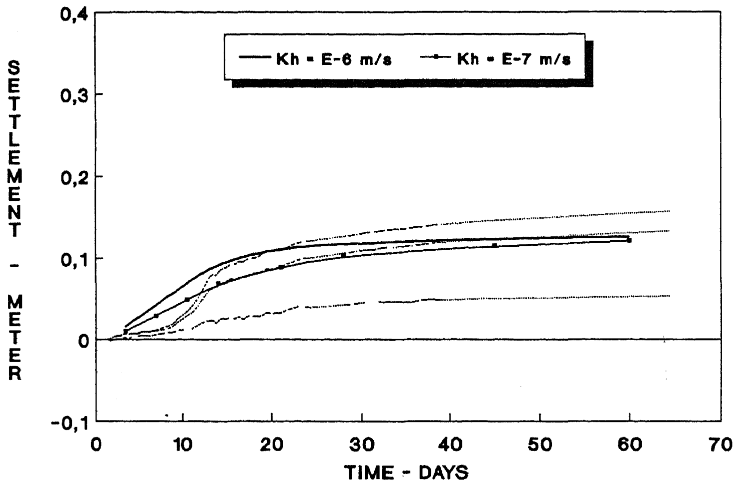

FIGURE 8
SENSITIVITY ANALYSIS - VARIATION OF Kh

CONCLUSIONS

The test embankment program was an effective tool for the investigation of the consolidation parameters of hydraulically-deposited uranium mill tailings. The main conclusions obtained from the interpretation and numerical modelling of the test data are the following:

- Total settlement of the tailings, as affected by C_c, C_r and P_c, can be accurately predicted based on the laboratory test results.

- The settlement rate of the tailings is significantly faster than the rate predicted using laboratory test data as a result of the heterogeneous placement of the material and the frequent presence of sand horizons within the tailings mass.

- Pore water pressures within the tailings are overestimated by the two-dimensional elastoplastic model due to the nonsaturated conditions of the material.

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