Estimate of Settlement of a Tailings Dam Founded on Collapsible Soils: Case Study

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ABSTRACT

Collapsible soils are common in arid regions near the southern coast of Peru and often have a loose, metastable and brittle structure. They are usually of an eolian or alluvial origin with very low moisture content, negative pore pressures, and interparticle contacts that are partially cemented by salts. Thus, making them prone to sudden and substantial settlements and loss of shear strength by quenching of the negative pore pressures and dissolution of the salts if the moisture content is increased. Rapid rearrangement of the particles into a denser matrix can then occur causing the collapse phenomena. If kept dry and in their natural state they may remain metastable and can have moderate to high strength and stiffness. But in engineered applications, since these soils have physical and mechanical properties strongly influenced by moisture content, if wetting is a possibility, engineered mitigations must be adopted to either remove them or adequately accommodate their potential collapse.

The purpose of this paper is to present an estimate of the settlements that are expected to be experienced by a particular tailings dam founded on collapsible soils in southern Peru (case study). The estimates were made using a simplified approach based on the collapse index and collapse potential findings from laboratory testing (ASTM D5333) and were then corroborated by 2-D finite element numerical modeling using GeoStudio software. The results indicated that settlements in the order of 1.6 m could be expected at the time of start-up of the facility, which is the most critical period, when the supernatant water pond will be located against the dam and will lead to significant seepage and therefore wetting of the foundation materials.

Engineered solutions to mitigate the potential for settlement were adopted and included either excavating and replacing the soils with dense compacted fill or installing a geomembrane to substantially reduce potential wetting of the foundation.

INTRODUCTION

The case study is related to tailings storage facility (TSF) located on the coast of the region of Ica, 40 kilometers (km) southwest of Nazca city, Peru. The proposed area has a relatively flat topography, with elevations ranging from 750 to 800 meters (m) above sea level (masl), and consists of a broad valley with natural containment on West, North and East extents of the proposed impoundment basin. Construction of a relatively long earthen dam has been projected in the southern part of the TSF (as shown in Figure 1) providing containment for tailings and supernatant water during operations. The project is planning to produce thickened tailings over an estimated 18-year operational period. Tailings will be discharged in four stages: 1) first three years, (critical case due to potential first wetting of the foundation); 2) the following five years; 3) the next five years; and 4) the final five years.

The earthen dam will be built in four stages using the downstream construction method with a drainage system just upstream to intercept seepage. The material of construction will be of andesite waste rock from the proposed open pit as the main borrow source. The earthen dam has a total length of 2.6 km, with a maximum height of 35 m (southwest portion of the dam comprising the critical zone for settlement), and 25 m wide crest at each stage. Both upstream and downstream side slopes are designed to be 3:1 and 2.5:1, respectively. The extent of the dam foundation largely comprises potentially collapsible soils, which have been identified in alluvial deposits during the geotechnical site investigation conducted as a part of the project development. Upon identification it was important to understand the behavior and potential risks associated with these types of materials.

These soils are generally characterized by relatively low density, high porosity, low moisture content with negative pore pressure, and partial cementation of particle contacts due to salt deposits forming. Moreover, they can have moderate to high strength and stiffness in a dry state but are susceptible to significant strength loss and settlement as a result of wetting.

Significant settlement under the proposed dam due to collapse of these soils may trigger a potential loss of integrity and thus poses a risk of dam failure due to loss of freeboard, cracking, or development of piping channels in either the foundation or the dam. Therefore, this was the main motivation of the research which is aimed to estimate potential settlements under the dam foundation during the start-up of the facility, which is expected to be the most critical period due to the potential first wetting of the collapsible soil. This study includes a review of both existing literature and existing lab testing results to characterize potentially collapsible soils, such as index properties and degree of collapsibility.

To estimate the settlement a simplified method based on the results of laboratory testing (ASTM D-5333) was used, which involves 1-D consolidation testing of inundated and non-inundated samples of similar collapsible soils with measurement of the percent vertical strain experienced by each sample at different levels of vertical stress. The difference in percent strain between the two samples is the collapse potential, Ic. A collapse index, Ie, is the differential collapse strain measured under a vertical stress of 200 kPa, and this is used to make a distinction between severe, moderately severe, moderate, slight and no degree of collapsibility. In order to corroborate these results, a 2-D finite element numerical model was developed using the GeoStudio software package. Two scenarios were simulated to better understand the impact of these soils below the dam. One considered that during start-up the supernatant pond is always located at the dam which leads to significant potential seepage through the dam and into the foundation. Two considered that the supernatant water pond will be located at the dam only initially but then it will be displaced due to tailings deposition to approximately 670 m from the dam.

Additionally, engineering solutions were presented including excavation and replacement of the collapsible soil with compacted fill and installation of a geomembrane liner on the upstream face of the dam extending into the basin.

GEOTECHNICAL CHARACTERIZATION

A site specific geotechnical investigation was executed within the limits of the TSF footprint. Boreholes and test pits were executed as a part of the geotechnical investigation; including disturbed and undisturbed soil sampling, in-situ testing, such as permeability, standard penetration testing (SPT), and geophysical survey. The geophysical survey consisted of Multichannel Analysis of Surface Waves (MASW) testing. Field test results were analyzed together with laboratory test results on foundation materials, tailings samples, and potential borrow materials. Figure 1 shows the geotechnical site investigation plan, and Figure 2 shows the critical cross section of the dam which was later analyzed located in the valley bottom (Southwest portion of the dam), where thicker, potentially collapsible soils were identified in the form of alluvial deposits.

Foundation

The foundation of the tailings dam was characterized by the following geotechnical units:

(1) E**olic deposits (UG-1)**; superficial deposits mainly consisting of 1.5 to 4.0 m of silty sands (SM) and poorly graded sands (SP), non-plastic, loose to medium dense and dry;

(2) A**lluvial deposits (UG-2);** located under the eolic deposits mainly consisting of 5.0 m to 15.0 m of silty sands (SM) and poorly graded sands (SP), low plasticity, medium to very dense, and dry. Hydraulic conductivity varies from $6.4x10^{-5}$ to $1.6x10^{-2}$ centimeters per second (cm/s). This deposit includes inter-particle cementation by salt precipitation;

(3) A**lluvial gravels (UG-3);** below the alluvial deposits mainly consisting of up to 60 m of well-graded gravel (GW) and silty gravels with sand (GM), non-plastic, very dense, dry and brown. Hydraulic conductivity varies from $1.3x10^{-6}$ to $4.1x10^{-3}$ cm/s;

(4) C**onglomerate (UG-4);** below the alluvial gravels consisting of calcareous conglomerate rock, heavily to slightly weathered, with a low unconfined compressive strength. Hydraulic conductivity varies from $7.0x10^{-6}$ to $6.8x10^{-4}$ cm/s; and,

(5) B**edrock (UG-5);** below the conglomerate composed of andesite and arkoses formation, heavily to slightly weathered, heavily to lightly fractured, with medium to high unconfined compressive strength. Additionally, some fractured are found in this bedrock. Hydraulic conductivity varies from $1.7x10^{-7}$ to $1.6x10^{-3}$ cm/s.

Tailings

Tailings will be discharged with a total solid content of 58% into the impoundment. The tailings classify as a sandy silt (ML) with a fines content varying from 53 to 59%, a plasticity index (PI) of 7, and a hydraulic conductivity ranging from 3.0 x10^{-6} to 1.4x10^{-5} cm/s, based on the results of laboratory testing.

Dam fill

The TSF dam will be built using andesite waste rock for much of the fill which is classified as poorly graded gravel (GP) with a fines content ranging from 2% to 5%. Hydraulic conductivity is anticipated to be relatively high based on of the gradation testing, in an order of 1.0x10^{-1}.cm/s.

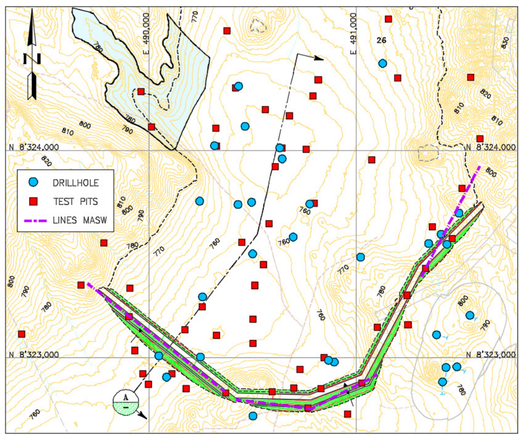

Figure 1 Geotechnical site investigation program executed within the TSF footprint

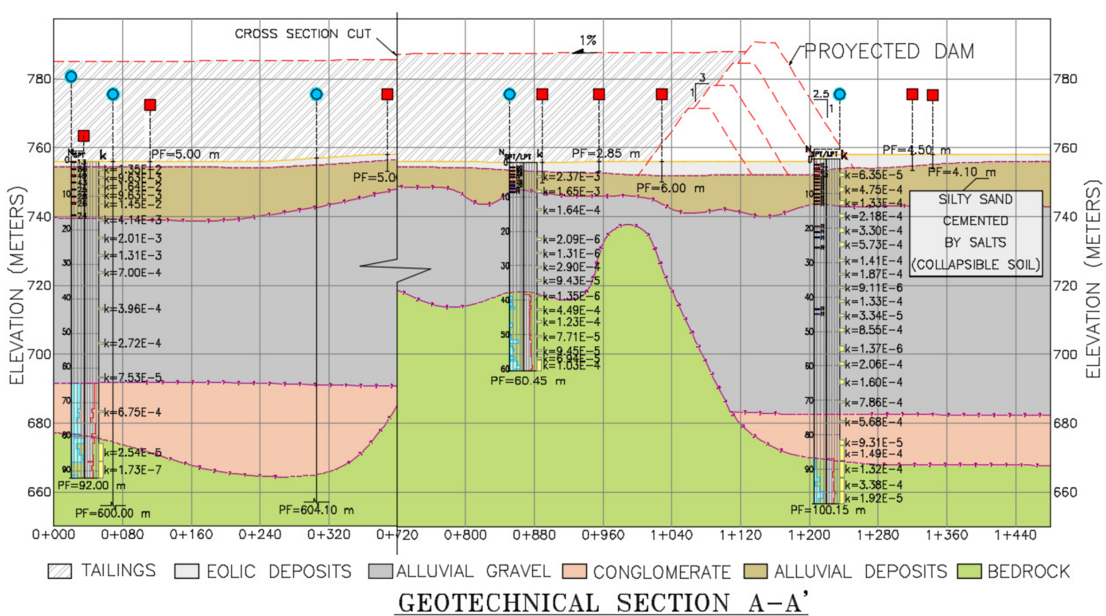

Figure 2 Stratigraphic profile (critical cross section)

COLLAPSIBLE SOIL CHARACTERIZATION

Undisturbed and disturbed sampling

Collapsible soils were identified in alluvial deposits (UG-2) during the geotechnical site investigation program. Undisturbed samples were obtained from test pits excavated at various locations within the valley bottom by means of block sampling (300 millimeter [mm] x 300 mm x 150 mm blocks), at depths ranging between 2.0 to 5.0 m. The samples were protected during transportation in a rigid box with foam insulation filled into the annual space. These samples were sent to a designated geotechnical laboratory to perform both collapse testing (ASTM D5333-03) [3] and direct shear testing (ASTM D3080). Additionally, disturbed samples were collected from boreholes and test pits for index testing (Atterberg limits (ASTM D4318), particle size distribution (ASTM D422), hydrometer tests (ASTM D422-63), specific gravity (ASTM D854)) and soluble salts content determination (NTP 339.152).

Collapsible soils classification

Index properties of the identified, potentially collapsible soils are summarized in Table 1. Figure 3 presents the particle size distribution range for these materials. They are classified as: poorly graded sand (SP), poorly graded sand with silt (SP-SM), and silty sand (SM), according to the Unified Soil Classification System (USCS- ASTM D2487). They are generally identified as cohesionless soils (i.e., show non-plastic behavior).

Standard Penetration Test (SPT) was developed during geotechnical investigation in the boreholes. They vary from 24 to more than 50 with rejection. Relative density was correlated with them; as a result, the material is very dense and varies from 80% to 100%.

Table 1 Summary of index test result of identified, potentially collapsible soils

Material	SUCS	Gravel %	Sand %	Silt/Clay %	LL	LP	IP	W %	Gs
Deposits alluvial	SM, SP, SP-SM	5 - 15	30 - 85	6 - 30	NP	NP	NP	2 - 5	2.6 – 2.9

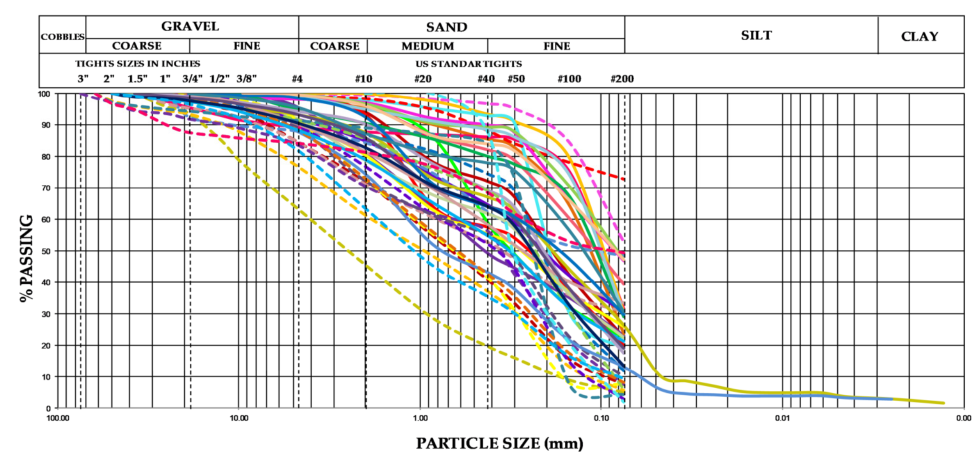

Figure 3 Particle size distributions of identified, potentially collapsible soils

Testing for collapse index and collapse potential of soils

The conventional oedometer test (ASTM D5333-03) [3] was conducted to assess the degree of potential soil collapse. The influence of the particle size distribution, void ratio, and density on the potentially collapsible soil was also, assessed using standard procedures (ASTM D5333-03) on undisturbed soil samples. One objective of the collapse testing is to determine the collapse index (Ie) of the sample, which is the percent stain experienced by an inundated sample minus the percent strain experience by a non-inundated sample (i.e. incremental percent strain) under a vertical applied stress of 200 kPa. The results are compared to the ranges below to classify the soil into one of the five categories describing the potential degree of collapse. Some pictures are shown in Figure 4 during of the test.

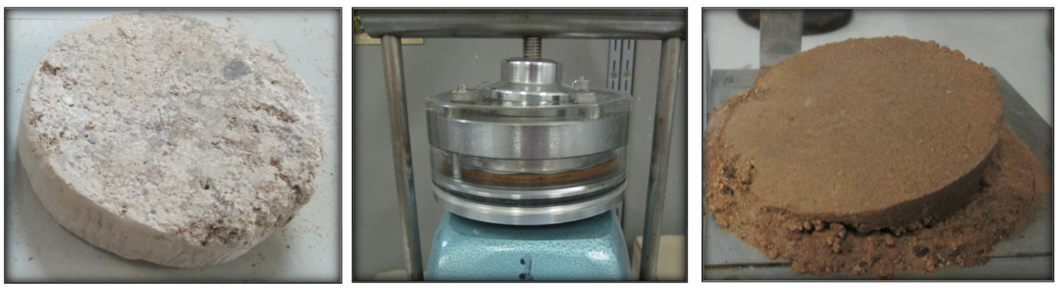

Figure 4 Pictures during of collapse test

Table 2 Collapse index classification categories by ASTM D-5333 [3]

Degree of collapse	None	Light	Moderate	Moderately severe	Severe
Collapse Index	0	0.0 – 2.0	2.1 – 6.0	6.1 – 10.0	>10.0

A total of eleven (11) collapse tests were performed on samples obtained from within the proposed dam foundation, with the results shown in Figure 5. Based on these results, the Ie was found to vary between 1.0% to 14%, these were obtained for a percentage of wetting that vary between 18.0% to 28.0%. According to Table 2, these results corresponds to a degree of collapse ranging from light to moderate, with a few tests ranging from moderately severe to severe. As such, the foundation soils at the site were deemed to have an elevated risk of collapse upon wetting [6, 7].

Additionally, the collapse potential (Ic) was also estimated according to ASTM D-5333, which defines the Ic as the relative magnitude of soil collapsibility determined at any stress level. Therefore, a relationship between stress and collapse strain could be developed to estimate the expected amount of potential collapse under different stresses representing the foundation loading progression. These stresses represent since the first to last construction raise of the dam (300, 400, 600 and 800 kPa). Based on these results, the collapse potential varies from 2.0% to 18% as shown in Figure 6.

Soluble salts content

Collapsible soils often contain cementing agents that help to bind the soil particles into a brittle structure. For the particular case of this project, the cementing agent was identified as salt precipitate. To quantify the salt content, soluble salts content testing was conducted according to standard procedures NTP 339.152-2002 on undisturbed and disturbed soils samples obtained from within the proposed dam foundation. Test results shown in Figure 7 present salt concentrations at depths ranging from 1.0 m to 9.0 m. In accordance with standard, a soluble salts content more than 15000 p.p.m is harmful (vertical line shown in Figure 7).

Direct shear test

Direct shear testing (ASTM D3080) was performed on specimens two different undisturbed soils samples at different normal stresses (300, 400, 600 and 800 kPa). Specimens were subject to wetting at each predetermined normal stress prior to the application of shear stresses. Wetting induced vertical displacement (collapse). When the vertical deformation ceased, a constant rate of horizontal displacement (shearing) was applied to the specimen [1, 9]. Mohr-Coulomb effective strength parameters (cohesion [C] and friction angle [∅]) were estimated to be C=0 kPa, ∅= 32°-35°.

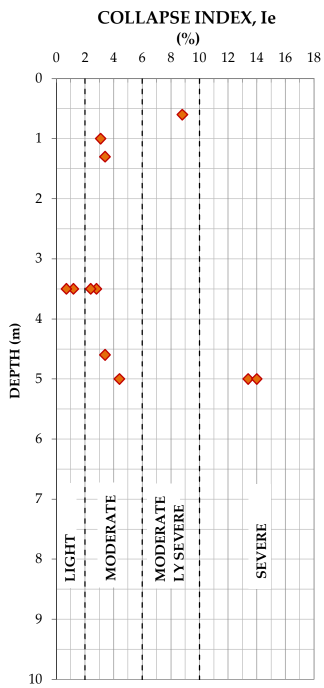

Figure 5 Collapse Index

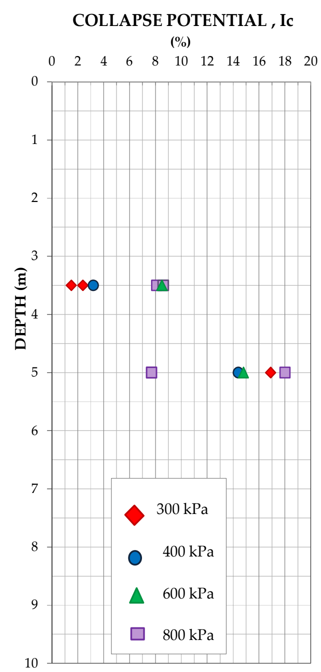

Figure 6 Collapse potential

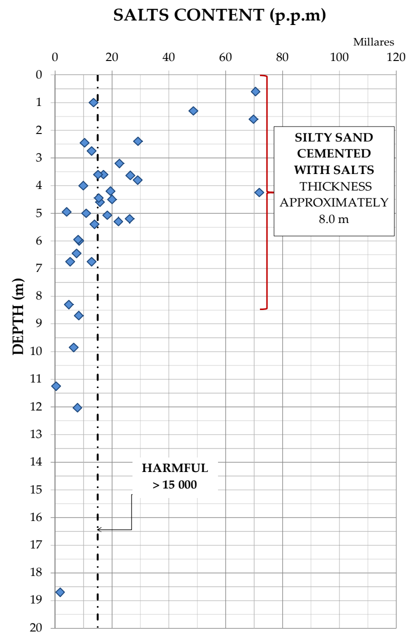

Figure 7 Content of soluble salts

SYMPLIFIED SETTLEMENT ESTIMATE BASED ON ASTM D-5333

Settlements under the TSF dam were estimated by utilizing the simplified equation proposed by ASTM D-5333 as follows:

$$S = \frac{H \text{ x } Ic}{100}$$

Where: S= settlement (m), H= thickness that represents the higher salt content material (m), as shown in Figure 7 and Ic= collapse potential (13.8 % in the first stage of loading and 18% in the fourth stage)

The settlements were estimated at each stage of construction; the settlements during the first stage (300 kPa) and fourth stage (800 kPa) are 1.3 m and 1.5 m, respectively.

To verify the results obtained by the simplified method, a 2-D finite-element numerical model was develop using GeoStudio software (SEEP/W and SIGMA/W).

SETTLEMENT ESTIMATE BY NUMERICAL MODELING

A numerical model was developed to estimate settlements in the dam foundation due to potentially collapsible soils using a finite-element method to compare with results obtained by the simplified method presented earlier. The model has the capability to combine the effect of transient water seepage into and through the dam due to TSF operation (water from the tailings void space and supernatant pond water), and use the resulting pore water distribution in the stress-strain model to estimate settlements. First, a transient seepage analysis utilizing SEEP/W was used to estimate the pore pressure distribution within the model in both saturated and unsaturated materials using Richards' equation [8]. Second, the resulting pore water (and pore pressure) distribution was coupled with SIGMA/W to perform a stress-strain response analysis [4]. In accordance with the tailings disposal plan, the numerical model was developed to represent each stage of dam construction: first stage (three years), second stage (five years), third stage (five years) and fourth stage (five years).

Model geometry and soil properties

Figure 8 shows a cross section of the TSF dam configuration as represented in the finite-element model. The depth of the potentially collapsible soil layer beneath the TSF dam range from 2.0 m to 15.0 m, approximately, and corresponds to UG-2. The cross section shows the geotechnical units identified during the geotechnical site investigation. Additionally, the TSF dam and tailings configuration at each stage of construction are also depicted.

Hydraulic properties

In order to model water flow through the porous media (e.g., soil), hydraulic properties such as the saturated hydraulic conductivity, unsaturated hydraulic conductivity function, and soil-water characteristic curve (SWCC) need to be estimated for each geotechnical unit.

The SWCC of collapsible soils was estimated using the method proposed by Aubertin et al (2003) [5], which predicts the volumetric water content function using basic material properties, and is appropriate particularly for preliminary analysis. These basic properties are the grain size distribution, liquid limit (LL) and saturated volumetric water content (Θ_s). The last is equivalent to the soil porosity (i.e. 0.48 for the collapsible soils based on the results of the collapse testing).

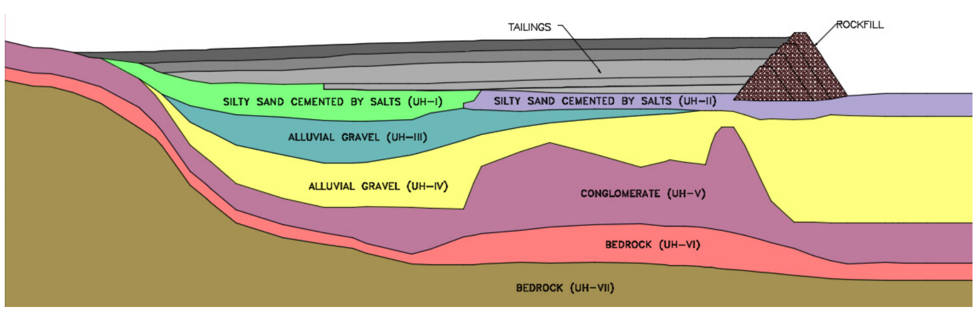

Figure 8 Geotechnical model for seepage and deformation analysis of tailings dam

Similarly, the unsaturated hydraulic conductivity function for each geotechnical unit was estimated using the methods proposed by Van Genuchten (1980) [10], providing an equation that relates both unsaturated hydraulic conductivity and matric suction. Table 3 summarizes the hydraulic properties for each geotechnical unit.

Table 3 Hydraulic properties used for seepage analysis

Material	Hydraulic Unit	θs (m3/m3)	D10 (mm)	D60 (mm)	LL (%)	k (m/s)
Tailings		0.53	0.0017	0.109	14	6.50E-08
Rockfill		-	-	-	-	1.00E-03
Silty sand cemented by salts	UH-I	0.48	(*)	(*)	(*)	1.00E-04
	UH-II					2.00E-05
Gravel alluvial	UH-III	0.41	-	-	-	2.00E-05
	UH-IV					2.00E-06
Conglomerate	UH-V	0.51	-	-	-	2.00E-06
Bedrock	UH-VI	0.51	-	-	-	2.00E-07
	UH-VII					1.70E-09

(*) Varies based on collapsible soils grain size distribution by Aubertin et al (2003) [5] method.

Geotechnical parameters

The material model representing the TSF dam fill (rockfill), conglomerate, alluvial gravel and bedrock were assumed to be linear elastic; however, the material model representing the potentially collapsible soils (silty sand cemented by salts) and tailings were assumed to behave as elastic-plastic, as they are expected to undergo significant deformation [11].

A summary of the geotechnical parameters used in the stress-strain modelling is shown in Table 4. The stress-strain parameters for most of the geotechnical units and TSF materials were estimated based on experience with laboratory testing on similar materials. In contrast, the potentially collapsible soil parameters, such as Young's modulus (E) and Poisson's ratio (ν) were estimated based on the results of site-specific geotechnical laboratory testing (collapse testing). By the way, soil collapsibility will be simulated using estimated Young's module in based to collapse test after sample saturation. The best way would be to represent to this material as a function between Young's modulus and pore pressure; as a result settlements would be estimated in progressive way with increase of pore pressure due to seepage of supernatant water pond. Neither constitutive models of SIGMA/W include this function in their equations for modeling the behavior of soil collapse influenced by wetting.

Table 4 Geotechnical parameters used for the stress-strain modeling

Material	Young's modulus (E) (kPa)	Poisson's ratio (ν)
Tailings	2 000	0.40
Rockfill	170 000	0.30
Silty sand cemented by salts	14 000-19 000 (*) 6 500-11 000 (**)	0.25
Gravel alluvial	70 000	0.20
Conglomerate	10 000 000	0.30
Bedrock	10 000 000	0.30

(*)Estimated from collapse test before sample saturation

(**)Estimated from collapse test after sample saturation

Simulated scenarios

Simulated scenarios were analyzed based on time of start-up of the facility of the TSF operation.

Scenario 1

During the initial period of the TSF operation, the supernatant water pond is assumed always to be located adjacent to the upstream face of the dam and lead to significant seepage through the dam and into the foundation (as shown in Figure 9). The main reason to consider this potential condition is the risk associated with the thickener plant which may be unlikely to operate at full efficiency at the time of mill start-up, and will not be able to generate tailings at the designed solids content. This will potentially lead to several months of operation below its full capacity discharging additional water into the TSF basin increasing the size and extent of the operational pond.

Scenario 2

This scenario assumes the supernatant pond will be located at the dam only initially, but then it will be displaced by tailings deposition to approximately 670 m from the dam (as shown in Figure 10), as designed. In the first stage neither saturation nor settlements were expected on the foundation due to supernatant water pond location. Thus, the four stages of the dam might be built. For the second, third and fourth stages, the supernatant pond will be located at 600, 600, 500 m, respectively. This scenario may reduce seepage through the TSF foundation by reducing the hydraulic gradient imposed by the supernatant pond; therefore, potentially reducing the amount of settlement relative to Scenario 1.

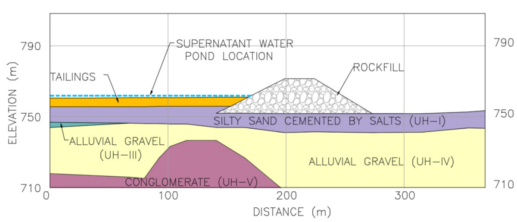

Figure 9 Configuration of Scenario 1

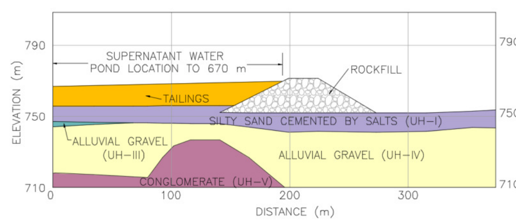

Figure 10 Configuration of Scenario 2

RESULTS AND DISCUSSION

This section summarizes the results after the analyses on each scenario to estimate settlements under the TSF dam in accordance with supernatant water pond location.

Scenario 1

Result from transient seepage analyses in this scenario confirm that the foundation is expected to saturate due to supernatant water pond location (i.e., against of the upstream face of the dam) at the end of the first stage tailings deposition, as shown in Figure 11. The wetting front reaches a depth of approximately 120 m, measured from the original ground surface during the last year of the first stage. Pore pressure distribution as estimated by the transient seepage analysis was used to estimate settlements using the stress-strain relationship of the materials. The results estimated a maximum settlement below the dam of 1.6 m, as shown in Figure 11.

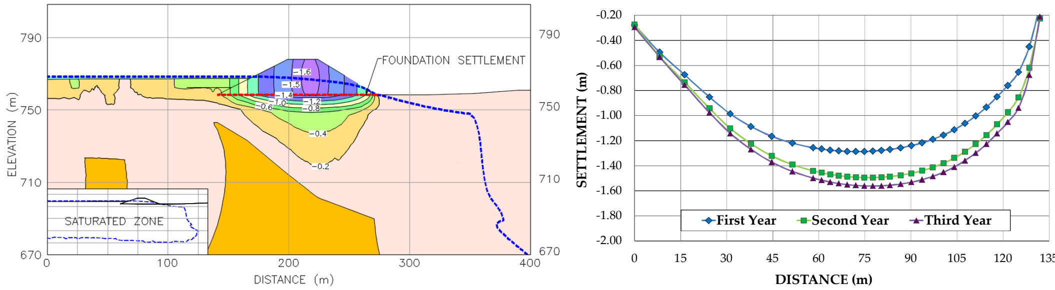

Figure 11 Results from seepage and stress-strain analysis for Scenario 1

Scenario 2

Results from seepage analysis considering a supernatant pond located at a significant distance from the dam, shown small zones of saturation upstream of the dam (as shown in Figure 12) and a maximum settlement below the dam of 0.2m, approximately. This settlement is smaller in comparison to scenario 1 as shown in Figure 12.

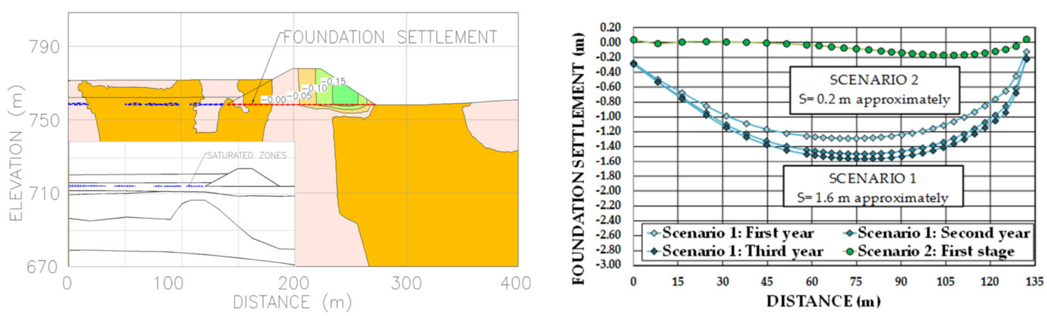

Figure 12 Results from seepage and stress-strain analysis for the first stage of Scenario 2

The saturation under the dam is expected at the last year of operation, as shown in Figure 13. The saturation front below the supernatant water pond reaches a depth approximately of 90 m, during the last year of the fourth stage. The settlements at the dam foundation vary from 0.2 m to 0.7 m for the first and fourth stage, respectively (as shown in Figure 13).

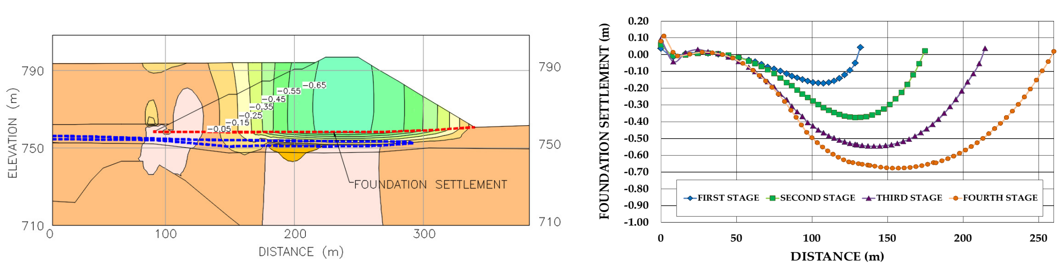

Figure 13 Results from seepage and stress-strain analysis for Scenario 2

Engineering solutions

In remedy to the results estimated for Scenario 1, which is assumed to be the most critical scenario based on the estimated settlements, foundation improvements were recommended which should be focused on mitigating the potential impacts of the collapsible soils. The following alternatives were proposed:

Alternative 1: Excavate and replace with compacted fill

Removal and replacement of the collapsible soil with compacted waste rock placed in layers of 1000 mm thickness was the first alternative recommended [2]. Two sets seepage and stress-strain analyses were performed for varying thickness of replaced material. The estimated settlements range from 1.0 m to 1.3 m and 0.8 m to 1.1 m for an improvement of 2.0 m and 4.0 m thickness, respectively, as shown in Figure 14.

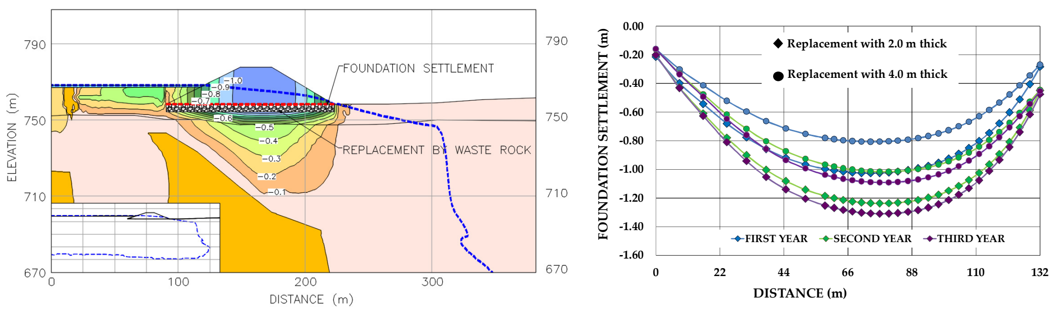

Figure 14 Results from Seepage and stress-strain analysis for Alternative 1

Alternative 2: Geomembrane liner installation

Incorporate a geomembrane liner on the upstream face of dam extending into the basin to reduce potential seepage through the dam and into the foundation; under it will be installed a drainage system. The analysis considered extending the geomembrane liner approximately 200 m to 400 m upstream into the TSF basin (with the intent to reduce significant seepage during start-up of the operations). Geomembrane condition neither estimated filtration into nor saturated zones developing in the dam foundation (as shown in Figure 15). The maximum estimated settlement was about 0.22 m due to dam self-weight.

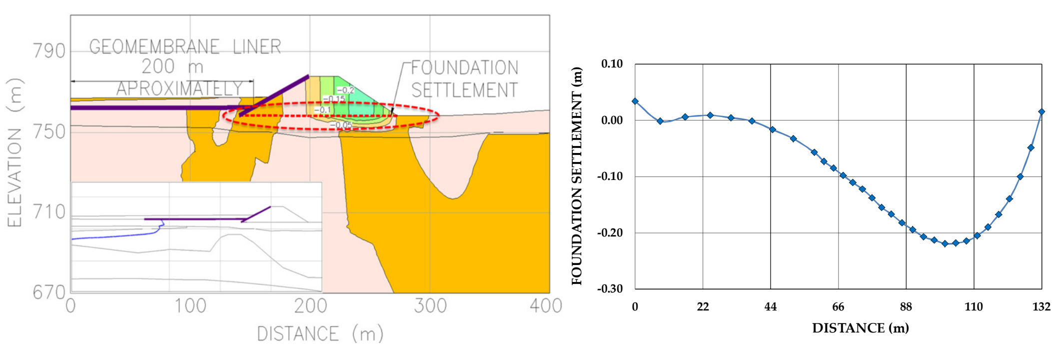

Figure 15 Results from Seepage and stress-strain analysis for alternative 2

CONCLUSIONS

The following conclusions can be drawn as a result of the analysis performed in this work:

1) As we stated before, the probability of finding collapsible soils in arid zones is extremely high. For this reason, professional expertise is recommended to identify and take counter-measures against collapsible soils.
2) According to the lab results, the "Ie" vary from light to moderate, with a few estimated as moderately severe to severe upon wetting. For this reason, the foundation soils are deemed to have an elevated risk of collapse upon wetting.
3) The "Ic" was assessed to estimate the expected settlements by collapse under different stresses. It was estimated that it will vary from 2.0% to 18%.
4) The most critical case is Scenario 1. Its importance lies in the need to define the geotechnical design criteria.
5) The settlements estimated by the simplified method and numerical modeling were 1.3 m and 1.6 m for the first stage, respectively. The difference of the results of settlements by both methods is because, with the simplified method, the lab made an attempt to find a tentative value of the foundation as a whole, rather than use a numerical model.
6) Developing the simplified method was important because it helped us understand the behavior of the material as a whole, and to propose alternative solutions.
7) Two alternatives were proposed in order to improve these conditions:
    a. Excavation and replacement, which may reduce expected settlements by about 20% and 30%, by 2 m and 4 m of improved thickness, respectively.
    b. It is expected that, with the installation of a geomembrane liner, the settlements will be by self-weight alone.
8) Since a 1.0m of dam freeboard is an acceptable value for the case study, the geomembrane liner alternative was recommended, it should be extended from 200 m to 400 m into the basin. Therefore, the other complementary alternative will be an additional fill placement on top of the crest.
9) The alternative to excavate and replace may further reduce settlements, but the costs associated with this alternative should be evaluated on a value-added basis due to the length of the dam.

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