



Mechanical properties of soft tailings from different depths of a Swedish tailings dam: Results from triaxial tests

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KEYWORDS

Soft tailings;
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Abstract. In an upstream tailings dam, loose layers might occur at different depths due to the melting of frozen layers deposited during freezing temperatures in Sweden. The reduced shear strength of such layers in a tailings dam might cause stability problems. Due to the slow consolidation process, it remains to be seen whether self-weight of a high tailings dam could affect the strength and stiffness of soft tailings located at different depths or not. For numerical modeling, appropriate strength and stiffness properties of soft tailings are required. To this end, loose layers in an upstream tailings dam were identified based on the results of cone penetration tests. Consolidated Drained (CD) triaxial tests were conducted on undisturbed soft tailings collected from different depths of the dam. The results indicated that depth did not have considerable effect on the strength and stiffness of tailings. Hardening Soil Model (HSM) at high confining pressures and axial strains underestimated the stiffness of soft tailings under CD triaxial state. This study shows that (i) proper care is required for evaluating strength and stiffness parameters for soft tailings and (ii) the application of the HSM is likely to predict more deformations, which could give an early warning before an actual failure of a tailings dam.

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1. Introduction

Tailings dams may be constructed using three methods such as upstream, downstream, and centerline [1]. Tailings dams raised using the upstream construction method are relatively economical as compared to those constructed with other two methods [1]. It is generally

understood that soil becomes stronger in deep layers as compared to the surface. This implies that lower layers in an embankment dam might be stronger than those at the top. This is because the strength is dependent on the confining pressure coming from the layers above. On the contrary, in an upstream tailings dam raised in freezing weather such as that in Sweden, the deposition process of tailings is carried out throughout a year. New layers of tailings are placed on old frozen layers. Due to the low permeability of tailings, the consolidation process is slow in a tailings dam. In the summer time, the melting of frozen layers may occur. Because of this, it is possible that there might be loose layers with more permeability and less shear strength. Such loose layers in a tailing dam might be sensitive

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to internal erosion, which could lead to failure. Today, it still remains to be seen what the effect of confining pressure on the strength and stiffness properties of the loose layers of tailings present at various depths of a tailings dam is.

Several studies on geotechnical properties of tailings material are reported in the literature (e.g., [2–15]). None of these studies has considered the effect of confining pressure on strength and stiffness properties of tailings.

To evaluate the stability of an upstream tailings dam, advanced numerical tools based on finite element methods are generally preferable to the traditional methods based on the limit equilibrium approach (see, e.g., [16–27]). At present, commercial finite element programs provide several constitutive models that cover various properties of soils in a wide range of applications in geotechnical structures. At present, due to the availability of commercial finite element software, it is possible to perform numerical modeling of tailings dams to compute realistic stability and deformations [28,29]. To predict reliable results of stability and deformations of a tailings dam, the selection of a proper constitutive model plays a key role. This is because tailings may show a different stress strain behavior as compared to natural materials of equivalent grains size [1]. To evaluate the suitability of a constitutive model for tailings material, reliable laboratory tests should be carried out on undisturbed samples.

The focus of the present study is to investigate mechanical behavior and the effect of confining pressure on strength and stiffness parameters of undisturbed loose tailings obtained at different depths of Aitik tailings dam located in northern Sweden. The capability of Hardening Soil Model (HSM) [30] to simulate the response of loose tailings in drained triaxial compression tests is discussed.

2. Materials studied

The undisturbed samples of tailings were collected from various locations of Aitik tailings dam. Void ratio and bulk density of tailings obtained from various depths are shown in Table 1. Samples were collected from those layers of the dam, which were described as loose

according to CPT results. Particle size distribution curves conducted by Bhanbhro et al. [6] showed that the tailings particles at shallow depths were more angular than those obtained from deeper depths. All samples were fully saturated with water content from 15 to 44%. The average specific gravity and bulk density of the studied tailings were 2.83 and 1.73–1.98 t/m³, respectively.

All tests were subject to an axial strain rate of 5×10^{-3} mm/min. The axial strains were measured indirectly by volume changes in lower chamber, and radial strains were calculated from changes in specimen's height and volume. Similarly, volumetric strains were calculated from changes in back volume of specimen by measuring the amount of water coming into or going out of specimen and initial sample dimensions. The axial deformations in the test specimen were also calibrated by external linear variable differential transducer and were assumed to be equal.

The membranes used to surround the specimen were made up of natural latex rubber. The membrane used is of 0.35 mm in thickness and 50 mm in diameter. The stiffness of the rubber membrane is about 0.38 N/mm [31], which may affect the sample stiffness. Hence, the membrane correction was applied [31]. The corrections of –1 to –5 kPa were applied at axial strain rates of 4 and 20%, respectively.

3. Test procedure

All tests in this study were conducted as Consolidated Drained (CD) triaxial tests on undisturbed samples. Before mounting the sample, all controllers and pipes attached to the apparatus were properly de-aired. The samples were put into a stretched membrane mold. The porous stone filters were provided to allow drainage from both ends. After providing porous stone filters, the bottom end of the sample was fixed with a triaxial cell by unfolding membrane on the bottom part of the triaxial cell. O-rings were applied and mold was then taken out. The sample was then surrounded by a split mold to install top cap. This was done because of the sensitivity of material and to avoid any disturbance during the installation of the sample. The process of mounting is shown in Figure 1. The triaxial cell was filled with de-aired water by opening the top cap and allowing water to enter and de-air the cell. Once the cell was filled in with water, the top outlet was closed, assuring that there was no air entrapped in the cell and pipes. The bottom drainage outlet was connected to the back-pressure controller to saturate the material. The saturation process was done at a pressure difference of 5–10 kPa for coarser specimens only.

This was the only process where drainage line at the top end was kept open. In the rest of the

Table 1. Void ratio and bulk density of soft tailings sampled at various depths of Aitik tailings dam [6].

Dam section	Depth (m)	Void ratio	Bulk density (t/m ³)
GH section	7–10	0.94–0.96	1.73
EF section	18–20	1.02–1.09	1.92
EF section	21–22	1.22–1.61	1.85
GH section	38–47	0.72–1.06	1.98

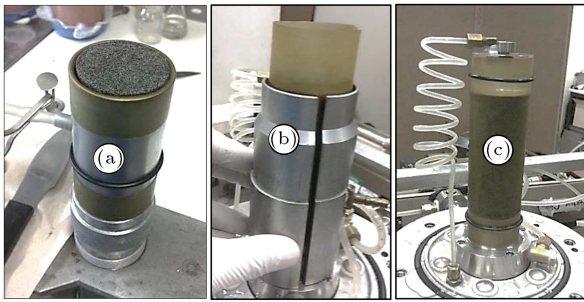


Figure 1. Sample taken out from sampler and mounted on triaxial cell: (a) Membrane stretcher, (b) split mold, and (c) sample after mounting.

process, the back-pressure controller was connected to the top outlet of the specimen for applying back pressure, and the bottom outlet was connected to the pore-pressure recorder. Pre-consolidation was carried out by increasing the radial stresses up to 20 kPa and back-pressure to 10 kPa; then, the effective stresses were kept in the range of 10 kPa.

3.1. Saturation

Saturation of the specimen was conducted by increasing the backpressure and cell pressure simultaneously with a linear rate by keeping the effective stresses at 10–15 kPa. The backpressure of 210–220 kPa was used in all of the tests. The rate of the application of backpressure and cell pressure was kept at 1.2–1.8 kPa/min for all the tests. To ensure the required saturation of the specimen, a B-check was performed. This was done by increasing the cell pressure by 50 kPa and measuring the change in pore pressure. The *B*-value [31] can be calculated as follows:

$$B = \frac{\Delta u}{\Delta \sigma_3}, \quad (1)$$

where Δu is the change in pore pressure, and $\Delta \sigma_3$ is the change in cell pressure. Typical *B*-values calculated in this study were in the range of 0.95–0.98; only two tests showed *B*-value as 0.89. The specimen with *B* values of 0.9 to 1 can be considered as fully saturated [31].

3.2. Consolidation

After the *B*-check, if the sample was considered saturated, the backpressure for all the tests would be kept at 250–300 kPa. The radial stresses were then increased to achieve the required effective stresses for the test being carried out. If the required effective stresses were more than 100 kPa, the radial stresses would be applied in steps with each step of 100 kPa. Back volume changes were monitored in each step with a condition that no change in volume should occur for five minutes to proceed to the next step.

3.3. Axial strain

Once the effective stresses were set to desired values and no volume changes were observed for five minutes,

axial strains were applied with a constant rate of 5×10^{-3} mm/min. The tests, which were performed at a confining stress of 100 kPa, were subjected to loading, unloading, and reloading. The initial loading was applied based on the assumption (based on assumed strength parameters) of 75% of maximum deviator stress followed by unloading. Reloading was then applied once the specimen was unloaded back to deviator stress equivalent to zero. Axial strains in unloading and reloading were kept at the same rate as those in the primary loading. The loading, unloading, and reloading were conducted to determine the parameters of HSM. All of the tests performed were subjected to axial strains of 20% except that which was subjected to 10%, since it was terminated due to the limit exceeded in the pressure-volume controller.

4. Results and discussions

4.1. Stress-strain behavior of soft tailings

CD triaxial compression tests were conducted for all of the samples taken from various depths (cf. Table 1). The deviatoric stress versus axial strain and volumetric strain for the tests conducted at confining pressures of 50, 100, 250 and 400 kPa are presented in Figure 2. Of note, for all of the samples taken from different depths,

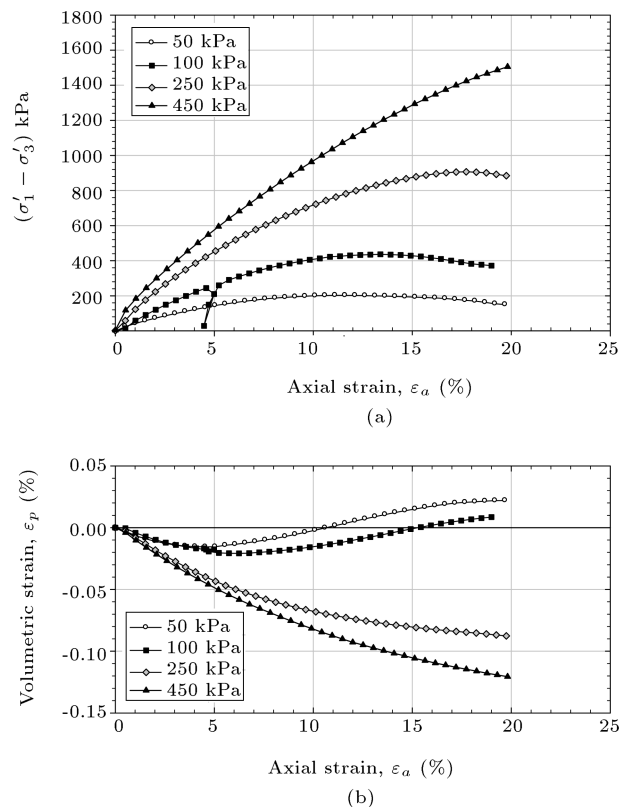


Figure 2. Stress strain response of soft tailings in consolidated drained triaxial test. Deviator stress (a) and volumetric strains (b) versus axial strains.

the stress strain behavior was almost the same. With an increase in confining pressures, the axial strains at which failure occurred increased; this was observed in all of the tests. This is consistent with common observations such that the tests performed at higher confining stresses show failure at larger strains. The figure also shows volumetric strains plotted versus axial strains. The dilatant and contractant behaviors were observed for the tests that were performed at low confining and high confining pressures, respectively.

4.2. Effective principal stress ratios

The effective principal stress ratio versus axial strain curves for CD triaxial compression tests conducted on tailings sampled at different depths are plotted in Figure 3. It can be observed that almost the same effective principal stress ratios at failure were obtained for soft tailings irrespective of the depth where samples were collected. This implies that the stress strain behavior of these soft tailings is independent of the depth.

Higher effective principal stress ratios were observed for all of the tests conducted at low confining stresses as compared to high confining pressure. High

principal stress ratios were observed for the specimen with a high value of the void ratio (i.e., 1.6) at low confining stresses (Figure 3(c)). There was no visible peak identified until 10% of axial strains in most of the tests conducted except those which were conducted at lower confining pressures and with void ratios of 0.9. The specimens collected from the shallow depth (i.e., 7–10 m) of the dam with a void ratio of 0.9 showed a peak stress ratio near 5% of axial strains (Figure 3(a)). In general, the lower effective principal stress ratios were observed for specimens that were tested at high confining pressures; this trend is in agreement with the studies reported in the literature (e.g., [4]).

4.3. Strength parameters

The values of the effective friction angle of tailings collected at different depths in the dam are presented in Table 2. The values were found to be in the range of 39.1 to 41.1 degrees. These values are in agreement with a previous study reported Jantzer [32]. No considerable influence of the depth was observed on values of effective friction angle of the tailings despite the fact that the specimens at shallow depth were more angular than those located deeper.

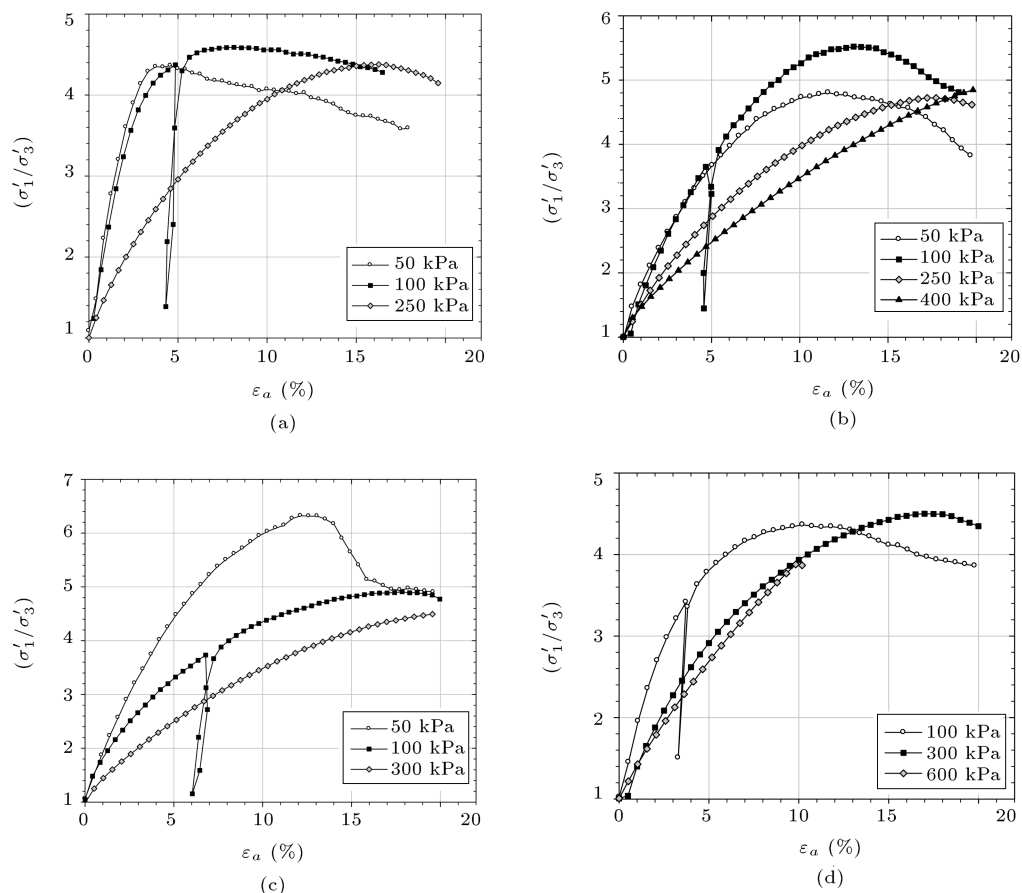


Figure 3. Principal effective stress ratios versus axial strains: (a) GH section at 7–10 m depth, (b) EF section at 18–20 m depth, (c) EF section at 21–22 m depth, and (d) GH section at 38–47 m depth.

Table 2. Variation of effective friction angle of tailings with respect to depth.

Sample location	Depth from surface of dam (m)	Friction angle (deg)
GH section	7–10	39.1
EF section	18–20	41.1
EF section	21–22	40.2
GH section	38–47	39.7

4.4. Effective stress paths

Figure 4(a) presents the effective stress paths for CD triaxial tests on tailings material subjected to different confining pressures (EF section, 18–20 m depth). The figure also shows the best curve fit for the critical state line passing through the origin. The slope of the critical state line (here written as M) was found to be in the range of 1.61 to 1.73. One set of results from GH section at 7–10 m depth showed a certain cohesion intercept (here written as k) on the q -plane when p' was zero. When $p' = 0$, the deviator stress observed was $q = k$; this may not hold in the case of granular soils. The critical state line slope in this case was observed as $M_k = 1.55$, which was then plotted as the best fit curve that passed through the origin and showed the value as $M = 1.63$, as presented in Figure 4(b).

4.5. HSM parameters

The HSM is capable of simulating the stress strain response of soft and hard soils subjected to the static loading condition in various geotechnical applications [30]. The HSM captures the associated decrease and increase of the mean effective stress exhibited by soft and hard soils, respectively. Deformations and failure of soils could be predicted reliably using the HSM [30].

The HSM simulates hyperbolic stress-strain relationship in a more realistic way than Mohr-Coulomb model does [30]. For drained triaxial loading, the relation between axial strain and deviator stress can

be defined as a hyperbolic function and is described as follows [29]:

$$\varepsilon_1 = \frac{q_a}{2E_{50}} - \frac{q}{q_a - q} \quad \text{for} \quad q < q_f,$$

where ε_1 is the strain, q is the deviatoric stress, and q_f is the ultimate deviatoric stress described with Mohr-Coulomb failure criterion, $q_f = \frac{6 \sin \phi'}{3 - \sin \phi'} (\sigma'_3 + c' \cot \phi')$, and q_a is the asymptote of hyperbola.

$$q_a = \frac{q_f}{R_f}, \quad (2)$$

where R_f is the failure rate relationship between failure stresses q_f and q_a , and $q_f = q_a$ when $R_f = 1$.

The evaluation procedure of parameters of the HSM [33] is described in Table 3. The parameters of HSM were evaluated based on the drained triaxial tests, and standard oedometer tests were conducted on tailings material. The unloading and reloading responses of a CD triaxial test at a confining stress of 100 kPa are shown in Figure 5.

The HSM can capture hyperbolic stress strain behavior exhibited by drained triaxial compression tests on soil (Figure 6). The stress-strain relationship in the unloading-reloading state is controlled by E_{ur}

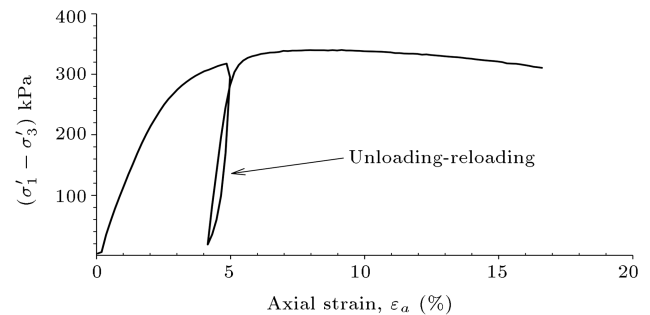
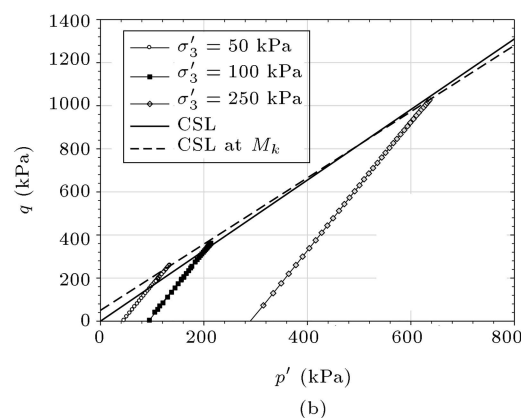
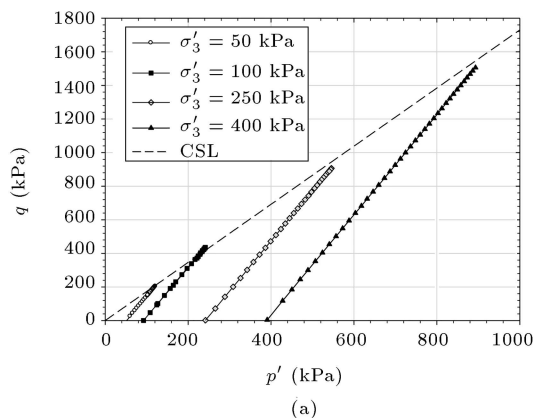
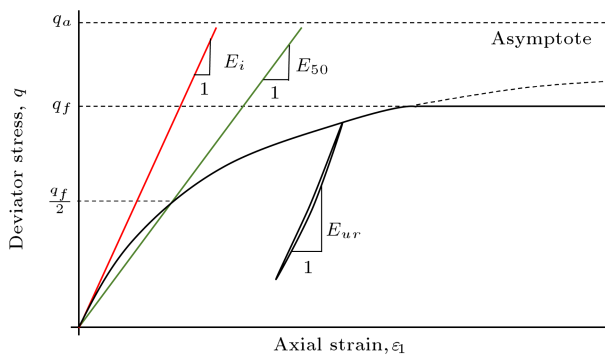
**Figure 5.** Loading, unloading, and reloading behaviors of a consolidated drained triaxial compression test performed on soft tailings at a confining pressure of 100 kPa.**Figure 4.** The effective stress paths for CD triaxial tests on tailings samples taken from (a) EF section at 18–20 m depth and (b) GH section at 7–10 m depth.

Table 3. Evaluation procedure of parameters of hardening soil model [33].

Parameter symbol	Description	Parameter evaluation
ϕ'	Internal friction angle	Slope of failure line from MC failure criterion
c'	Cohesion	y -intercept of failure line from MC failure criterion
Rf	Failure ratio	$(\sigma_1 - \sigma_3)(\sigma_1 - \sigma_3)_{ult}$
ψ	Dilatancy angle	Function of ε_a and ε_v
E_{50}^{ref}	Reference secant stiffness from drained triaxial test	y -intercept in $\log(\sigma_3/p^{ref}) - \log(E_{50})$ space
E_{oed}^{ref}	Reference tangent stiffness for oedometer primary loading	y -intercept in $\log(\sigma_1/p^{ref}) - \log(E_{oed})$ space
E_{ur}^{ref}	Reference unloading/reloading stiffness	y -intercept in $\log(\sigma_3/p^{ref}) - \log(E_{ur})$ space
m	Exponential power	Slope of trend-line in $\log(\sigma_3/p^{ref}) - \log(E_{50})$ space
ν_{ur}	Unloading/reloading Poisson's ratio	0.15 to 0.3
K_0^{nc}	Coefficient of earth pressure at rest (NC state)	$1 - \sin \phi'$ (default setting)

**Figure 6.** Hyperbolic stress strain behavior in drained triaxial test.

(Young's modulus for unloading and reloading) and ν_{ur} (Poisson's ratio for unloading and reloading). Another feature of the HSM is the stress-dependent modulus corresponding to reference confining stress (p^{ref}), and the amount of stress dependency is given by power ' m '. The values of m values of silts/sands and soft clays may be taken as 0.5 and 1, respectively [34]. The stiffness modulus is stress dependent by power m [30] and can be written in the following generalized form [29]:

$$E_{50} = E_{50}^{ref} \left(\frac{c' \cos \phi' - A \sin \phi'}{c' \cos \phi' + p^{ref} \sin \phi'} \right)^m. \quad (3)$$

For E_{50} and E_{ur} : $A = \sigma'_3$ for E_{oed} : $A = \sigma'_1$.

4.6. HSM calibration

SoilTest tool in PLAXIS 2D [34] was used to calibrate the parameters for the HSM from the conducted laboratory tests. The simulations were performed on CD triaxial and oedometer tests.

Initially, the HSM parameters were obtained from the results of the oedometer test and CD triaxial test at a confining pressure of 100 kPa. Slight

modifications were then made to the model parameters to find the best match between simulated behavior and laboratory tests. The evaluated HSM parameters are presented in Table 4.

Figure 7 shows the behavior of the oedometer test and simulated behavior using the HSM. Initially, it was observed that the simulated vertical strains in the oedometer test were low as compared to laboratory observations. When the oedometer modulus decreased, the best curve fit was achieved (see Figure 7). However, the application of the decreased oedometer modulus caused a slight increase in volumetric strains in simulated triaxial tests.

The deviatoric stress versus the axial strain response simulated with the HSM is compared with CD triaxial tests performed on soft tailings (Figure 8). It can be observed that the deviatoric stresses simulated with the HSM were slightly higher at low axial strains, whereas, at higher axial strains, the simulated deviatoric

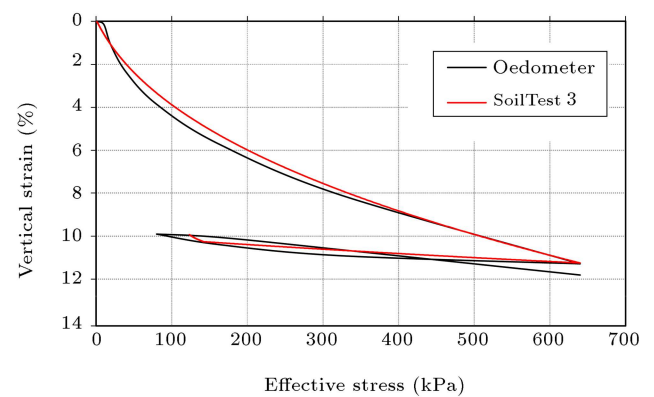
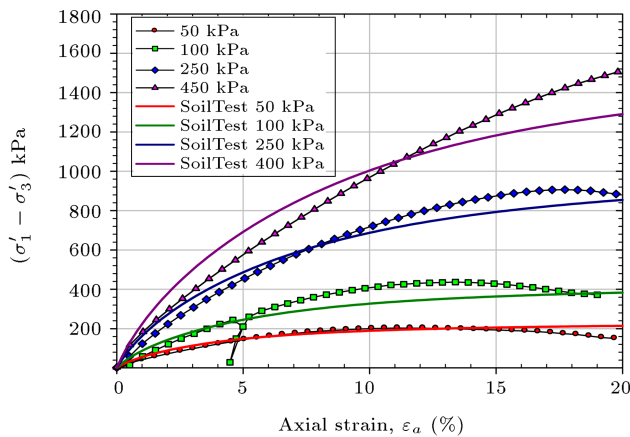
**Figure 7.** Comparison of loading and unloading responses of the oedometer test with Hardening Soil Model (HSM) simulations.

Table 4. Hardening soil model parameters for soft tailings located at various depths of Aitik tailings dam [29].

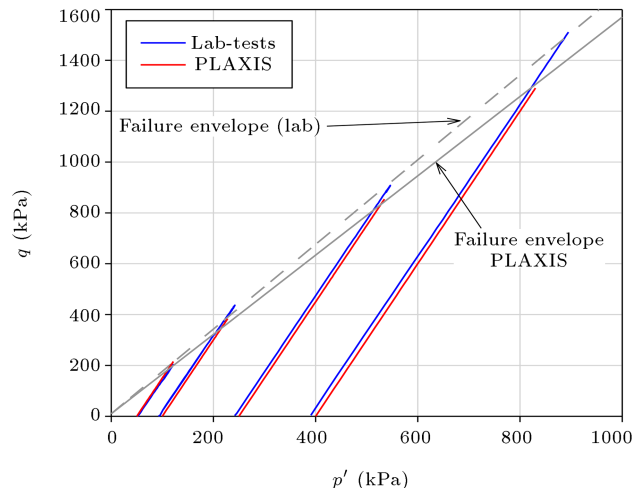
Parameter	Material location			
	GH (7–10 m)	EF (18–20 m)	EF (12–22 m)	GH (38–47 m)
E_{50}^{ref} (kPa)	6900	6032	5500	8254
E_{oed}^{ref} (kPa)	4000	3500	3950	6000
E_{ur}^{ref} (kPa)	20000	32600	22000	40000
m	0.7	0.6	0.57	0.5
P^{ref} (kPa)	100	100	100	100
ν_{ur}	0.3	0.15	0.15	0.15
c' (kPa)	0	7.5	0	0
ϕ' (deg)	39.5	40.2	40	38.66
ψ (deg)	2	2.5	2.5	1
R_f (kN/m ³)	0.8	0.9	0.9	0.9
γ_{unsat} (kN/m ³)	14.9	16.2	15.7	14.3
γ_{sat} (kN/m ³)	19.5	20.5	20.1	19.3

**Figure 8.** Stress strain behavior of consolidated drained triaxial test on soft tailings material compared with hardening soil model simulation.

toric stresses were lower than that of laboratory tests. Figure 9 shows the comparison of effective stress paths simulated with the HSM and obtained from CD triaxial tests. It can be observed that both types of effective stress paths are similar. On the other hand, the HSM slightly overestimated volumetric behavior of the soft tailings, compared to that of CD triaxial tests.

5. Discussion

The peak deviatoric stress was observed at high strains for the tests performed at high confining pressures of 250 and 400 kPa. However, the tests performed at lower confining pressures showed peak stress followed by softening behavior. When particles have higher strength than the confining pressure, the tailings may have greater stiffness due to the interlocking of particles [35]. During the application of axial loads, the stiffness of particles increased to the peak value. Post-

**Figure 9.** Comparison of effective stress path simulated using hardening soil model with that of consolidated drained triaxial test results.

peak behavior of tailings shows crushing or sliding of particles. Consequently, the stiffness of tailings was reduced [33].

The HSM slightly overestimated stiffness at small strains and underestimated stiffness at larger strains (Figure 9). However, the underestimated values of stiffness could predict exaggerated deformations, which might give an early warning before the occurrence of true failure of a tailing dam.

6. Concluding remarks

In this study, the effect of confining pressure of increased height of an upstream tailings dam was investigated on soft tailings located in deeper layers. In addition, the capability of Hardening Soil Model (HSM) was evaluated to simulate stress strain and

volumetric responses of soft tailings. The following conclusions can be made:

- No considerable influence of self-weight of the dam was observed on strength and stiffness properties of soft tailings located at different depths;
- At high confining pressure and axial strains, as would be expected during future raisings of the dam, the HSM overestimated the stiffness of undisturbed soft tailings, obtained at different depths, as compared to the results of CD triaxial tests;
- Numerical analysis of a tailings dam using the HSM may predict slightly more deformations in those zones, where soft tailings are located;
- A future study is suggested to evaluate the suitability of the HSM to simulate the consolidated undrained response of soft tailings.

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