Seismic Stability and Liquefaction Analysis of Tailings Slope

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Abstract
The potential for liquefaction and slope stability of a thickened tailings slope, within a storage facility located on the shore of a natural glacial lake in Tehran, is evaluated based on field and laboratory testing. The tailings slope has the particularities of being located in a high seismicity area and founded on conventional slurry tailings from the previous operation of the facility. The in-situ state of the thickened tailings was investigated through field tests, including Standard Penetration Tests (SPT) and Cone Penetration Tests with pore pressure measurement (CPTu). Samples collected during the field investigation were used in an extensive geotechnical laboratory program to determine index properties and strength parameters for the thickened tailings. Testing included a series of isotropically consolidated undrained and drained (CIU and CID) triaxial tests, as well as a simulation of the consolidation process within the tailings through oedometer tests. The liquefaction potential was evaluated using the data from the CPTu testing, while slope stability was assessed using the limit equilibrium approach for an infinite slope, considering the variation of shear strength with the degree of consolidation (void ratio) of the tailings from a critical state approach. Based on the results of the liquefaction potential and the slope stability assessment, a design approach is suggested for facilities under similar conditions.

Keywords: Triaxial Tests, Liquefaction, Slope Stability, Tailings Slope, CPTu, SPT

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I. INTRODUCTION

Slope failures or landslides are common geotechnical hazards. These include failures of tailings slope of mine waste due to a lack of either adequate shear strength or excessive soil deformation. Liquefaction under either static or dynamic conditions has been considered one of the commonest failure mechanisms for tailings slopes and tailings dams ([1], [2], [3], [4], [5], [6]). Many procedures have been published for estimating the residual or liquefied shear strength of cohesionless soils. Some procedures require a laboratory testing of field samples obtained by ground freezing techniques e.g., [7] or samples obtained by high-quality tube samples coupled with procedures for “correcting” the shear strength for disturbance during sampling and testing [8]. Jefferies et al. [9] suggested a correlation to evaluate the liquefied shear strength using an equivalent clean sand normalized penetration resistance. To obtain equivalent clean sand penetration resistance values, [10] used the original standard penetration test (SPT)-based correction factors suggested by [11] and [12], although little justification was provided to support the proposed correction factors. Lade et al. [13] also included recommendations regarding potential but undefined void redistribution mechanisms.

Robertson [10] developed a chart to identify soil behavior type (SBT) based on normalized CPT parameters as shown in Fig. 1. The CPT parameters are normalized by the effective overburden stress to produce dimensionless parameters, $Q_t$ and $F_r$, where,

$$Q_t = (q_t - \sigma_{vo})/\sigma_{vo}$$  \hspace{1cm} (1)

$$F_r = \left[ f_s / (q_t - \sigma_{vo}) \right] \times 100\%$$  \hspace{1cm} (2)

where $q_t$=CPT corrected total cone resistance ([15]); $f_s$=CPT sleeve friction; $\sigma_{vo}$=preinsertion in situ total vertical stress; and $\sigma_{vo}$=preinsertion in situ effective vertical stress.

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Jeffries and Davies [9] identified that a SBT index, \( I_c \), could represent the SBT zones in the \( Q_t-F_r \) chart where \( I_c \) is essentially the radius of concentric circles that define the boundaries of soil type. Robertson and Wride [11] modified the definition of \( I_c \) to apply to the \( Q_t-F_r \) chart as defined by

\[
I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5}
\]  

II. SITE SPECIFIC STUDIES

The tailings storage facility is located on the western shore of a glacial lake in Tehran (Chitgar Lake) as shown in Fig. 2 at an average elevation of 600 meters above sea level. According to recent bathymetry surveys, the thickened tailings slope varies from 4% (near the discharge point) to 2% (near the pond).

Initially, the facility was operated following the conventional technology of slurry tailings disposal (31\% of solids content). Recently, a thickened tailings technology (69-71\% solids content) was implemented to improve tailings management during operation and closure. This modification required a redesign of the facility and therefore further field and laboratory tests. During the field testing program, the deposited thickened tailings constituted approximately the upper 15 m of the subsurface profile.

This paper presents a procedure to evaluation of liquefaction and stability analysis of a thickened tailings slope subjected to a static and seismic conditions using triaxial tests and field tests, including Standard Penetration Tests (SPT) and Cone Penetration Tests with pore pressure measurement (CPTu). The cyclic triaxial tests were conducted in accordance with ASTM D5311 [13] using the automatic triaxial test, as shown in Fig. 3.

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Fig. 1. SBT chart based on normalized CPT parameters (modified from [13]).

\[ I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5} \]  

Fig. 2. Location of study area in the Tehran province and Iran: (a) and (b) Chitgar Lake in Tehran; (c) General cross section of a slope.
III. FIELD EXPLORATION AND SAMPLING

Field investigations in the tailings facility consisted of two SPT and two CPTu soundings. The Field SPT blow counts “N” were corrected due to energy losses and overburden pressure to obtain (N1)60 values [16]. Cone penetration tip resistance “qc” was normalized to obtain the “qc1” parameter and incorporate the action of the shear modulus increase with depth [17]. Through the SPT testing, six thickened tailings samples were recovered in Shelby tubes (Shelby samples) at various depths ranging from 6 to 32 m. The in-situ state profile was defined by analyzing the results of void ratio, vertical effective stress, Atterberg limits from each SPT Shelby sample (as shown in Fig. 5), SPT (N1)60 blow counts, and CPTu (qc1) normalized tip resistance (presented in Fig. 5).

The profiles presented in Fig. 4 and 5 show no noticeable behavior difference between conventional slurry tailings and thickened tailings.

Fig. 3. Cyclic triaxial system and its associated units used in this study: (a) Triaxial cell, computer for controlling and data acquisition system and pressure control unit; (b) Triaxial cell saturation board

Fig. 4. Tailings in-situ state.

Fig. 5. SPT (N1)60 and CPTu - qc1
IV. LABORATORY TESTING AND TAILINGS PROPERTIES

Standard classification testing was carried out in addition to oedometer and triaxial compression testing (drained and undrained) on undisturbed Shelby samples of thickened tailings. The testing program was meant to determine index properties, evaluate the compressibility behavior and define the peak (yield) and post-liquefaction strength under undrained loading of the tailings samples. The properties of the tailings are presented in Fig. 4 and summarized in Table 1. The tailings compressibility behavior was modeled using a function that relates void ratio and mean effective stress (Equation (4), (6)). The calibration data was taken from the oedometer testing performed at an initial void ratio of 0.76. It should be noted that this void ratio value was the highest feasible to remold the sample into the oedometer ring. Fig. 6 shows the tailings compressibility function used.

\[ e = a - b_{10} \left( \frac{\sigma_m}{P_{ref}} \right)^{\alpha} \]

Where \( \sigma_m \) is the mean effective stress; \( P_{ref} \) is the atmospheric pressure (101.3 kPa); \( a \) is the tailings initial void ratio, and \( b_{10} \& \alpha \) are fit parameters.

Where \( e_c \) is the critical void ratio; \( \Gamma \) is the reference void ratio on CSL at 1 kPa; \( \lambda \) is the slope of the critical state line; and \( \sigma_m \) is the mean effective stress. Table 2 summarizes the results from the performed triaxial testing and Fig. 7 shows the CSL for the thickened tailings samples.

\[ e_c = \Gamma - \lambda \cdot \ln(\sigma_m) \]
TABLE I. AVERAGE PROPERTIES OF COLLECTED TAILINGS SAMPLES

<table>
<thead>
<tr>
<th>SUCS</th>
<th>Gravel (%)</th>
<th>Sand (%)</th>
<th>Fines (%)</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>o (%)</th>
<th>Gs</th>
<th>Void Ratio at the Tailings Surface (e_d)</th>
<th>Dry Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CL</td>
<td>0</td>
<td>15</td>
<td>85</td>
<td>31</td>
<td>19</td>
<td>12</td>
<td>34</td>
<td>2.99</td>
<td>0.9</td>
<td>1.50</td>
</tr>
</tbody>
</table>

TABLE II. RESULTS FROM CONSOLIDATED ISOTROPICALLY Drained AND UNDrained TRIAXIAL TESTING

<table>
<thead>
<tr>
<th>Triaxial Test</th>
<th>Condition</th>
<th>Before Consolidation</th>
<th>After Consolidation and Before Shearing</th>
<th>At Critical State</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Dry Density (g/cm³)</td>
<td>Void Ratio after saturation</td>
<td>Confining Pressure (kPa)</td>
</tr>
<tr>
<td>CIU</td>
<td>Loose</td>
<td>1.453</td>
<td>1.060</td>
<td>1.815</td>
</tr>
<tr>
<td>CID</td>
<td>Loose</td>
<td>1.450</td>
<td>1.064</td>
<td>1.851</td>
</tr>
</tbody>
</table>

V. LIQUEFACTION ASSESSMENT

The liquefaction potential was initially analyzed using the CPTu data obtained from the field investigation program. The analysis consisted of the estimation of the cyclic stress ratio (CSR) due to a magnitude (8) seismic event and the cyclic stress resistance (CRR), which is the soil resistance to liquefaction. The ratio between these two values provides an indicator of liquefaction potential or a factor of safety.

Given that no noticeable behavior difference was observed in the field test results (Fig. 4 and 5), penetration testing data from conventional slurry tailings and thickened tailings was considered indifferent. Considering the procedure proposed by [22], an equivalent CSR was calculated as 65% of the peak value and modified for a magnitude (8) seismic event using equation (6):

\[ CSR = 0.65 \left( \frac{a_{max}}{g} \right) \left( \frac{\sigma_v}{\sigma_v^e} \right) r_d \frac{1}{MSF} \]  

(6)

Where: \( a_{max} \) is the maximum horizontal acceleration; \( g \) is the acceleration of gravity (9.81 m/s²); \( \sigma_v \) is the total vertical stress, \( \sigma_v^e \) is the effective vertical stress; \( r_d \) is the reduction factor and MSF is the magnitude scaling factor.

The stress reduction factor (\( r_d \)) was estimated using equations (7) and (8), suggested by [23] and improved by [24]. The equations were initially developed by Golesorkhi based on seismic response analyses performed for soil deposits using equivalent-linear and nonlinear methods and (35) different earthquake records, and then statistically extending the \( r_d \) values to consider different earthquake magnitudes. Subsequently, [23] improved these formulations by adding different soil profiles to the database and presenting Golesorkhi’s findings in an \( r_d \) empirical correlation as shown below:

\[ \ln(r_d) = \alpha(z) + \beta(z)M \]  

(7)

Where,

\[ \alpha(z) = -1.012 - 1.126 \sin \left( \frac{z}{11.73} + 5.133 \right) \]

\[ \beta(z) = 0.106 + 0.118 \sin \left( \frac{z}{11.28} + 5.142 \right) \]

Also, for cases where locations deeper than 34 m:

\[ r_d = 0.12 e^{0.22M} \]  

(8)

Where \( z \) is the depth and \( M \) represents the earthquake magnitude.

The CRR of the tailings samples was determined using the correlation proposed by [24], who developed equation (9) to estimate the soil resistance from the adjusted cone tip resistance (q_C) and the soil type index (IC) based on previous publications ([25], [26], [27], [28], [29]).

\[ CRR = 0.05 + e^{A+B \left( q_C/100 \right)^C} \]  

(9)

Where,

\[ A = I_C (q_C/100) \]

\[ B = 0.909 I_C^2 + 0.901 I_C + 19.28 \]

\[ C = 0.059 + 0.015 I_C^2 \]

Fig. 8 presents the estimated values for CSR and CRR along with the associated factor of safety.
The estimated factors of safety suggest that the tailings would liquefy if a magnitude (8) event occurs. Consequently, to prevent the tailings from overtopping slope, a stable tailings slope under residual (post-seismic) conditions was evaluated. The slope stability under static conditions was also analyzed.

VI. SLOPE STABILITY ANALYSIS

A. Static Conditions

The static stability analysis considered the pre-failure peak (yield) condition for the tailings. In this condition, the undrained strength is known as yield shear strength [30]. The yield shear strength was estimated using the correlation proposed by [31] and the results from the SPT, CPTu, and CIU triaxial testing. Fig. 9 presents the estimated yield strength ratio profiles. The yield strength ratio values range from 0.15 to 0.35 and average at 0.24.

An infinite slope failure assessment was performed for three different slopes: 1.5% (representing a slope flatter than those observed on site), 2.0% (slope near the pond), and 4.0% (maximum observed near the discharge point). The factors of safety were calculated using equation (10), proposed by [32].

\[
FS = \frac{S_{yield}}{h \cdot \gamma \cdot \sin\beta \cdot \cos\beta}
\]  

(10)

Where: \( h \) is the failure surface depth; \( \gamma \) is the tailings unit weight; \( S_{yield} \) is the tailings yield strength, and \( \beta \) is the tailings beach slope. Fig. 10 shows factors of safety profiles under static conditions.
B. Seismic Conditions

Several researchers ([31] and [32], [33], [34]) support critical state soil mechanics to be an adequate theoretical framework to assess the post-liquefaction residual strength of soft soils, such as thickened tailings. In general, the residual strength of the liquefied material can be characterized as a function of the state parameter [40] which is the difference in void ratio between the in-situ state and the critical state.

Residual strength was calculated using the relationships proposed by [31] and by [32]. A procedure similar to that used to estimate the yield strength ratio was followed to estimate the residual (post-liquefaction) strength ratios using the SPT and CPTu data (Fig. 11). Estimated post-liquefaction strength ratios scatter in a wide range of values with a maximum of 0.20 and an average of 0.06, which lies within the range of historical data collected by [26]. Similar to the static case, factors of safety against sliding (see Fig. 12) were estimated using equation (10) considering the post-liquefaction resistance ($S_{\text{post-liquefaction}}$) instead of the yield strength ($S_{\text{yield}}$).

Post-liquefaction strength values and associated factors of safety were also calculated from the state parameter as proposed by [35] to a depth of 15 m. Post-liquefaction strength ratios and associated factors of safety are plotted in Fig. 13 and 14.

Fig. 11. Post Liquefaction Strength Ratio.

Fig. 12. Post Liquefaction Factors of Safety.
VII. CONCLUSION

This paper proposed the results of a comprehensive geotechnical study of a highly saturated and loose thickened tailings beach slope. The tailings state was determined from field and index laboratory classification testing. Penetration tests were extensively used to evaluate the potential for liquefaction as well as peak and post-liquefaction strength of the tailings.

Stability analyses were carried out to evaluate the performance of the tailings beach slope under static and seismic conditions. Results from these analyses included adequate static factors of safety for slopes between 1.5 and 4.0%. However, if liquefaction occurs, measured field slopes would be unstable and slope rearrangement would take place until slopes flatten to around 1.5%, when a stable condition would again be reached.

This paper has compared the liquefaction factor of safety based on SPT, CPT, site shear wave velocity and triaxial test. The selected stochastic parameters are corrected SPT blow count, saturated unit weight, shear wave velocity and corrected CPT tip resistance which is modeled using truncated normal probability distribution functions and earthquake magnitude which is considered to have exponential probability distribution function. The results show that the method based on the triaxial test is more reliable than other methods.

Finally, this document highlights the importance of performing the sequence of analyses presented herein for the design of stable thickened tailings slopes. The authors recommend and encourage design engineers to consider the design approach presented for similar facilities. Special attention should be given to liquefaction analysis as its occurrence can lead to flow failures of tailings beaches.

REFERENCES


