Resonant column and cyclic triaxial testing of tailing dam material
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ABSTRACT
A series of resonant column and cyclic triaxial tests has been conducted in the frame of the analysis of tailing dam stability during earthquakes. The investigation program for a silty sand from uranium tailings is presented. The paper describes the testing procedures and presents all significant results of these experiments. Single-stage and multi-stage resonant column tests were performed in order to determine the dependence of shear modulus and damping ratio on confining pressure and shear strain amplitude. Stress-controlled cyclic triaxial tests on the same material were conducted to determine the increase in pore pressure and the amount of accumulated strain in dependence of the cycles number and the load configuration. Approximate formulas are given for stiffness and damping to be used in seismic response analyses.

INTRODUCTION
When applying finite element methods for the seismic response analyses of dams and embankments, e.g. Seed\textsuperscript{1}, the nonlinear soil behaviour can be taken into account by use of equivalent elastic constants which vary as a function of stress and strain level. These parameters are obtained either from empirical relations, e.g. Hardin & Drnevich\textsuperscript{2}, or from laboratory tests on the particular soil material. Much of the published data deal with soils such as poorly graded sands, principal of alluvial origin. However, for soils that are dirty, gravelly, well graded or composed of relatively angular particles only a limited amount of test data is available in the
literature. In the frame of the stability analysis of tailing dams located in a moderate seismic risk region in the south-east part of Germany an extensive dynamic in-situ and laboratory investigation testing program was conducted. Since the tailings sand was placed hydraulically with little compaction the strength reduction due to pore pressure increase had also to be considered.

In the following, results of resonant column and cyclic triaxial tests are presented which were performed on samples of sand and silty sand from uranium tailings.

**MATERIAL TESTED**

The material tested was obtained from borings drilled in the crest or slopes of a major tailings dam. It consisted of sub-angular silty sand and had the following properties: i) grain size distribution: \( D_{50} = 0.2 \text{ mm}, D_{90} = 0.08-0.12 \text{ mm} \) and \( D_{95} = 0.5 \text{ mm} \), ii) particle density 2.77 Mg/m\(^3\) and iii) effective strength parameters \( \phi' = 31^\circ, c' = 5 \text{ kN/m}^2 \). The dry density \( \rho_d \) varied between 1.59 and 1.82 Mg/m\(^3\) corresponding to void ratios \( e \) from 0.74 to 0.52.

All specimens, 50 mm in diameter and 100 mm in height, were formed in an unsaturated state corresponding to the in situ water content and compacted to the required density.

**RESONANT COLUMN TESTS**

The tests were conducted on a Drnevich type apparatus, cf. Drnevich et al.\(^3\), according to the specification of the ASTM D4015 - 87 Standard. All specimens were isotropically consolidated using standard techniques. Multi-stage tests were conducted to determine the shear modulus and damping ratio, \( G_{max} \) and \( D_{min} \), respectively, at very low shear strain amplitudes \( \gamma \). The test was started at the lowest confining pressure and repeated for intermediate pressures up to a maximum confining pressure of 500 kPa. At the highest confining pressure under consideration the amplitude of excitation was gradually increased to study the shear modulus \( G \) as a function of shear strain amplitude. Damping was determined by the steady-state vibration of the specimen. Spot checks were made by the free vibration decay method. In addition, single-stage tests were performed at selected levels of confining pressure by stepwise increasing the excitation force up to a maximum shear strain amplitude of approximately \( 10^{-1} \% \). Densities and confining pressures for each test are summarized in Table 1, where the confining pressure \( \sigma_0' \) is normalized with respect to the atmospheric pressure \( p_a = 100 \text{ kPa} \). The measured shear moduli \( G_{max} \) at small strains of approximately \( 10^{-4} \% \) are depicted in Figure 1 as a function of confining pressure \( \sigma_0' \). Shear modulus reduction curves at two confining pressures are presented in Figure 2 versus shear strain.
Table 1: Summary of resonant column tests

All tests demonstrated the well-known dependence of \(G_{\text{max}}\) on effective confining pressure \(\sigma_0'\) and density expressed in terms of the void ratio \(e\). In order to obtain an analytical expression for the \(G_{\text{max}} = G_{\text{max}}(\sigma_0')\) relationship, the general equation suggested by Hardin* is adopted

\[
G_{\text{max}} = \frac{S}{0.3 + 0.7e^2P_a^{1-n}(\sigma_0')^n}
\]

where \(S\) is a stiffness coefficient. The experimental results presented in Figure 1 are closely approximated by setting

\[
S = 420 \quad \text{and} \quad n = 0.6
\]

It should be noticed that the value of the exponent \(n\) is higher than the widely used one for cohesive and cohesionless soils which ranges between 0.4 and 0.5.

Using regression analysis the following function is derived from the data of Figure 2 expressing the variation of the shear modulus with shear strain amplitude

\[
G = G_{\text{max}}/(1 + 200\gamma)
\]

The damping ratio \(D\) was found to be essentially independent on confining pressure and density. An average value \(D_{\text{min}} = 2\%\) was determined from all tests. The increase of damping with shear strain amplitude is depicted in Figure 3 and may be approximately expressed by

\[
\frac{D}{D_{\text{min}}} = 1 + \frac{A}{1 + \alpha/\gamma}
\]

with \(A = 6.2\) and \(\alpha = 6.5 \cdot 10^{-4}\).
Figure 1: Maximum shear modulus versus confining pressure

Figure 2: Shear modulus reduction versus shear strain
CYCLIC TRIAXIAL TESTS

These tests were conducted on anisotropically consolidated, saturated specimens under undrained conditions. All specimens were first consolidated under an effective confining pressure of 100 kPa. The axial load and the cell pressure were then progressively raised to the desired level under undrained conditions. The apparatus and testing procedure are described in detail by Savidis and Schuppe. A frequency \( f = 0.5 \) Hz was chosen for all tests. During each cyclic loading test deviator stress, pore pressure and axial strain were recorded by means of a computer.

For analysing the tests the anisotropic consolidation stress ratio \( K_c = \sigma_1/\sigma_3 \) and the cyclic stress ratio \( SR = \sigma_{dcy}/2\sigma_3 \) are introduced, whereby \( \sigma_3c \) and \( \sigma_{dcy} \) are the minor principal consolidation stress and the cyclic deviator stress, respectively. Following Seed et al. we distinguish between

\[
\frac{\sigma_{dcy}}{\sigma_3(K_c - 1)} \begin{cases} < 1 & \text{(i)} \\ > 1 & \text{(ii)} \end{cases}
\]

determining whether (i) or not (ii) a reversal in the direction of the applied shear stresses on potential failure planes is occurring during cyclic loading. For both cases, the anisotropically consolidated samples show
Table 2: Summary of cyclic triaxial tests

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>( \rho_d ) [Mg/m(^3)]</th>
<th>( \sigma_3 ) [kPa]</th>
<th>( K_c ) [-]</th>
<th>( SR ) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC1</td>
<td>1.70</td>
<td></td>
<td></td>
<td>0.550</td>
</tr>
<tr>
<td>TC2</td>
<td>1.71</td>
<td></td>
<td></td>
<td>0.700</td>
</tr>
<tr>
<td>TC3</td>
<td>1.58</td>
<td>100</td>
<td>2</td>
<td>0.550</td>
</tr>
<tr>
<td>TC4</td>
<td>1.56</td>
<td></td>
<td></td>
<td>0.750</td>
</tr>
<tr>
<td>TC5</td>
<td>1.59</td>
<td></td>
<td></td>
<td>0.750</td>
</tr>
<tr>
<td>TC6</td>
<td>1.59</td>
<td>150</td>
<td>2</td>
<td>0.750</td>
</tr>
<tr>
<td>TC7</td>
<td>1.71</td>
<td></td>
<td></td>
<td>0.667</td>
</tr>
<tr>
<td>TC8</td>
<td>1.64</td>
<td>200</td>
<td>2</td>
<td>0.625</td>
</tr>
</tbody>
</table>

a continuous increase in pore pressure and accumulated axial strain in contrast to the isotropic consolidated case where significant strains develop only when the pore pressure \( u_{cy} \) reaches about 60% of the confining pressure, Seed et al.\(^6\).

Static and cyclic loads for tests reported herein were chosen so as to produce a reversal of cyclic shear stresses during the test, as can be seen in Table 2 where a summary of the tests conducted is given. All specimens exhibited gradual progressive deformation. Additional tests, which are not presented here, were conducted under conditions of no shear stress reversal and showed the existence of limiting values for the pore-pressure. The values of the specimens density were chosen according to the respective in-situ values and were not low enough for liquefaction to occur.

The pore pressure ratio \( u_{cy}/\sigma_3 \) and the accumulated axial strain \( \varepsilon_1 \) are plotted in Figure 4 and 5, respectively, for distinct numbers of loading cycles. These plots do not presuppose any particular failure criterion. The results demonstrate the density dependency both of the pore pressure and the accumulated strain. However, when comparing tests with the same density and cyclic stress ratio \( SR \), susceptibility to pore pressure generation is essentially reduced at higher consolidation pressures. For the accumulated strain, on the other hand, the use of solely the cyclic stress ratio as an index of dynamic strength is more appropriate.
Figure 4: Pore pressure ratio versus number of loading cycles

Figure 5: Accumulated axial strain versus number of loading cycles
CONCLUSIONS

The results of the cyclic triaxial testing program presented herein indicate that the tailings material tested exhibit a relatively high resistance to cyclic loading. This is mainly due to the relatively high in-situ density resulting from the natural consolidation process. Scatter of the data is attributed to the difficulty in reconstituting the soil material in the laboratory. The resonant column tests, on the other hand, show a regular behaviour of the tailings sand leading to analytical expressions for stiffness and damping to be used in seismic response analyses.

REFERENCES


