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CYCLIC TRIAXIAL COMPRESSION TESTS
NEWBURGH LOCK AND DAM, OHIO RIVER
INDIANA AND KENTUCKY

by

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The fixed-weir portion of the Newburgh Lock and Dam project is to be constructed of concrete-capped cofferdam cells. These cells are to be filled with a cohesionless foundation material which is a poorly graded, gravelly sand (SP-SM) with about 5 percent nonplastic fines. Because the site is in a seismic zone 3 (Algermissen), a laboratory study was conducted to evaluate the earthquake response of the gravelly sand at various densities. About 60 lb of the material was obtained from the Louisville District, and (Continued)
20. ABSTRACT (Continued).

cyclic triaxial compression tests were conducted on the sand compacted to nominal 40, 60, and 80 percent relative densities. Based on the results of these tests, it appears that an in-place relative density of at least 75 percent is desirable. Therefore, it is expected that some means of underwater compaction will be required.
Preface

The study reported herein was performed by the U. S. Army Engineer Waterways Experiment Station (WES) at the request of the U. S. Army Engineer District, Louisville, and was authorized in Intra-Army Order No. DC-B-73-110 dated 20 Dec 1972.

The engineers of WES who were actively engaged in this study were Drs. F. C. Townsend and W. F. Marcuson III and Messrs. P. A. Gilbert and S. A. Collins. The work was conducted under the general supervision of Messrs. R. W. Cunny, Chief, Earthquake Engineering and Vibrations Division, and J. P. Sale and R. G. Ahlvin, Chief and Assistant Chief, respectively, Soils and Pavements Laboratory. This report was prepared by Dr. Marcuson and Mr. Gilbert and was reviewed by Mr. S. J. Johnson, Special Assistant, Soils and Pavements Laboratory.

Directors of WES during the conduct of this study and the preparation and publication of this report were BG E. D. Peixotto, CE, and COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.
Contents

Preface ................................................. 1
Conversion Factors, U. S. Customary to Metric (SI) Units of Measurement ............... 3
Objective and Scope ........................................... 4
Material ..................................................... 4
Testing Equipment ........................................... 5
Testing Procedures ........................................... 5
  Specimen preparation ........................................ 5
  Saturation .................................................. 6
  Consolidation .............................................. 7
  Cyclic loading ............................................. 8
Specimen Behavior ........................................... 8
Test Results .................................................. 8
Discussion of Results ........................................... 11
Table 1
Figures 1-12
Appendix A: Notation
Conversion Factors, U. S. Customary to Metric (SI)
Units of Measurement

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

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<th>Multiply</th>
<th>By</th>
<th>To Obtain</th>
</tr>
</thead>
<tbody>
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</table>
CYCLIC TRIAXIAL COMPRESSION TESTS
NEWBURGH LOCK AND DAM
OHIO RIVER, INDIANA AND KENTUCKY

Objective and Scope

1. The objective of this study was to evaluate the earthquake response of a foundation sand proposed for use in the fixed-weir portion of Newburgh Lock and Dam, which is to be located in a seismic zone 3 (Algermissen). Cyclic load triaxial tests were conducted on the sand, which had been compacted to nominal 40, 60, and 80 percent relative densities. These test results will be used in a subsequent analysis to determine the minimum density at which the sand can be placed to avoid the danger of a liquefaction failure created by earthquake loadings.

Material

2. Two canvas bags containing about 60 lb* of material were received from the U. S. Army Engineer District, Louisville. The two bags of material were combined, and a gradation test was performed on the resulting material. A grain size distribution of the composite material is shown in Figure 1. The material was a poorly graded, gravelly sand (SP-SM) with about 5 percent nonplastic fines. The sand consisted predominantly of light-colored, gray and tan quartz and granite particles with a smaller amount of light and dark igneous rock fragments and a few cinders. The large particle sizes were subrounded to subangular (predominantly subrounded); as the particle size decreased, the angularity increased such that the material retained on the No. 200 sieve consisted of about 90 percent angular to subangular particles.

3. Since the grain size analysis indicated that only about 3 percent of the material was retained on the 1/2-in. sieve, this material was removed and discarded so that 2.8-in.-diam test specimens could be

*A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.
used. The maximum and minimum densities for the -1/2-in. material, as determined by procedures outlined in EM 1110-2-1906,* were 121.4 and 109.3 pcf, respectively.

**Testing Equipment**

4. In cyclic triaxial tests, the deviator stress is uniformly increased and decreased while a constant chamber pressure is maintained. The cyclic axial loads were applied by the U. S. Army Engineer Waterways Experiment Station (WES) pneumatic loading unit, which consists of regulators and solenoid valves actuated by a cam-operated microswitch. The solenoid valves provide alternating air pulses to a double-acting load cylinder so that a load is cyclically applied to the specimen through a connecting piston from the load cylinder. For these tests, the frequency of loading was 2 Hz and the repeated loads were consolidated-undrained. Four variables were monitored continuously during the test: axial load, axial deformation, pore pressure, and chamber pressure. The variables were monitored with electronic sensors and were recorded using a high-speed continuous-line recorder. Care was taken to calibrate the equipment in order to obtain accurate measurements over the range of data generated by the tests.

**Testing Procedures**

5. Each of the sixteen tests consisted of four stages: specimen preparation, saturation, consolidation, and cyclic loading. Each stage is described in detail in the following paragraphs.

**Specimen preparation**

6. The test specimens were compacted inside a split forming jacket mounted on a triaxial chamber base. To compact specimens to a relative density of less than 50 percent, the forming jacket was tapped

---

lightly with a spoon. A hand-held pneumatic vibrator was used to compact specimens to a relative density of greater than 50 percent. The amount of oven-dry sand utilized in each layer was predetermined based upon the volume of the split mold (with appropriate corrections for a 0.012-in.-thick membrane) and the desired density. A vacuum was used to secure the membrane flush against the forming jacket during compaction. After the specimen had been molded and sealed by placing the top cap and securing the membrane with an O-ring, 10 in. of vacuum were applied to the specimen through the top cap. Subsequently, the forming jacket was removed, and the specimen was supported by the vacuum while the specimen was measured at the top, midheight, and bottom to determine its area and volume. The diameter and height of the specimens tested in this investigation were approximately 2.8 and 7 in., respectively.

**Saturation**

7. The process of saturation consisted of two phases: seepage saturation and back-pressure saturation. After measurement of the specimen volume, the triaxial chamber was assembled and a chamber pressure of 2 psi was applied to the specimen. The vacuum on the specimen was then increased to 20 in. A line containing de-aired distilled water was opened to the bottom of the specimen, and water was allowed to seep into the soil very slowly, displacing the air in the specimen. When water had completely filled the specimen voids and about 100 cc had been allowed to seep through the soil, the valve in the water line was closed and the vacuum on the specimen was increased to about 27 in. while the chamber pressure was reduced to zero. Air bubbles could then be observed coming through the line from the top of the specimen; the vacuum was maintained until air was no longer observed coming from the specimen. At this time, the vacuum line was closed and the vacuum was slowly reduced by allowing water to reenter the specimen while the chamber pressure was increased to 5 psi to support the specimen. When sufficient water had entered the soil to dissipate the vacuum completely, back-pressure saturation, consisting of simultaneously increasing the chamber pressure and the back pressure, was used to saturate the
specimen further. Saturation of the soil was obtained with a back pressure of about 65 psi.

8. The degree of saturation for this testing is expressed in terms of Skempton's B-parameter. The B-parameter is the ratio of the change in pore pressure $\Delta u^*$ to an induced change in chamber pressure $\Delta \sigma_3$ in an undrained state, i.e.,

$$B = \frac{\Delta u^*}{\Delta \sigma_3}$$

The value was determined by closing the drainage line, increasing the chamber pressure by 10 psi, and observing the increase in pore water pressure. The minimum acceptable B-value for this testing was set at 0.96; however, generally the B-value was 0.98 or higher. During saturation, any change in height of the specimen was measured with a dial indicator read to the nearest 0.001 in.

Consolidation

9. After saturation had been achieved, the specimen was consolidated to the effective confining pressure under which it would be tested. Consolidation was accomplished by increasing the chamber pressure while allowing drainage such that the final difference between the chamber pressure and the back pressure was the desired effective confining pressure. For the specimens consolidated isotropically, the effective confining pressure (40 psi) was reached by applying successive increments of 5, 10, and 20 psi (a differential of 5 psi was maintained during saturation); for the specimens consolidated to 10 psi, only one increment of 5 psi was used. Consolidation for the one anisotropically consolidated specimen consisted of increasing the axial load and chamber pressure while allowing drainage such that the final consolidation ratio $K_c$ of $\sigma_1$, major principal stress, to $\sigma_3$, confining pressure, was the desired condition of anisotropy; that is,

$$K_c = \frac{\sigma_1}{\sigma_3}$$

* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).
where
\[
\sigma_{1c} = \text{effective axial stress at the end of consolidation}
\]
\[
\sigma_{3c} = \text{effective confining stress at the end of consolidation}
\]

**Cyclic loading**

10. Cyclic loading consisted of the cyclic application and reduction of axial stress on the specimen in an undrained condition. The specimen was loaded in this manner through a double-acting air cylinder driven by the loading unit described previously.

**Specimen Behavior**

11. Initial liquefaction is defined as the stage at which the pore pressure first becomes equal to the chamber pressure. For the sand specimens tested in this program, the axial deformation and pore pressure increased with the number of cycles of loading. Tests on this sand showed that large deformations occurred within just a few cycles after initial liquefaction for specimens with low densities; but as the density increased, the number of cycles required to develop large deformation after initial liquefaction increased dramatically. This behavior is shown in Figure 2. For this testing, maximum deformation was limited to about 20 percent strain* due to constraints in the testing equipment.

**Test Results**

12. The results of all tests are summarized in Table 1. Figures 3, 4, and 5 are plots of the triaxial data expressed as stress ratio versus the number of cycles of loading for average relative densities \(D_d\) of 43.3, 58.8, and 78.2 percent, respectively. The stress ratio is expressed as

* Strain indicated is the sum of both extension and compression strain.
where
\[
\frac{\sigma_{dc}}{2\sigma_a}
\]

\(\sigma_{dc}\) = cyclic deviator stress, psi
\(\sigma_a\) = ambient confining pressure, psi, computed on the basis of in situ overburden weights

The value of \(\sigma_a\) for the tests presented in these figures was 40 psi. The curves in Figures 3 and 4 are for tests 1-6 for initial liquefaction and 10 and 20 percent strain. The curves in Figure 5, tests 7, 8, and 9a, are for initial liquefaction and nominal 10 percent strain. The right ordinate of these figures is peak ground acceleration in g's which was converted from stress ratio in accordance with Seed's simplified procedure\(^*\) for a level ground surface. These data indicate, for example, that at 58 percent relative density, initial liquefaction would occur in the foundation with 5 cycles of 0.2-g ground acceleration; and when the sand was at 78 percent relative density, 8 percent strain would occur with 25 cycles of 0.35-g ground acceleration.

13. Tests 11 and 12 were stress-controlled (monotonically loaded), undrained shear tests.\(^**\) These tests were performed to determine if the undrained shear behavior of the material at a relative density of 75 percent was that of a dilatant material. This type of test relates to behavior of a material after the period of ground shaking has stopped. During test 11, the membrane ruptured; hence no useful information was obtained except that at a relative density of 75 percent, the material dilates under a monotonic load. The results of test 12 are shown in Figure 6. The test specimen was molded at about 75 percent relative density, and it is seen from the figure that the material dilated under a monotonic load and developed negative pore water pressure. It should be noted that this test was conducted at a back pressure of 100 psi.

and that the undrained strength of a dilating sand that does not fail by crushing the grains is a function of the back pressure under which the material is tested. This indicates that for this material, a lower back pressure (such as the in situ hydrostatic pressure) would have yielded an apparently lower strength than that indicated by this test.

14. Tests 10 and 14 were performed to investigate the behavior of the material at a very high relative density. These two specimens were molded at an average relative density of 83 percent (the highest relative density conveniently obtained in the laboratory) and were tested at stress ratios near the maximum ratio (0.5) at which cyclic tests could be performed without applying a tensile stress to the soil. Initial liquefaction never occurred in these dense specimens; both specimens failed by necking. A few cycles before necking occurred, the specimens had undergone less than 5 percent strain. These tests are shown in the summary plot in Figure 7, which also includes plots of the initial liquefaction and strain behavior at lower relative densities.

15. Figure 8 is a plot of relative density versus cyclic deviator stress for initial liquefaction and 10 and 20 percent strain achieved in 20 cycles. The point at a relative density of 83 percent was estimated. Since this sample did not experience initial liquefaction in 20 cycles, the curve had to pass to the left of the plotted point. The point for 10 percent strain at a relative density of 78 percent was extrapolated from Figure 5. Figures 2 and 8 show how the stability of material increases substantially as the relative density increases.

16. Samples 13 and 16 were consolidated to an effective confining pressure of 10 psi to investigate the behavior of this material at a lower effective confining pressure. Figure 9 shows these tests expressed as a stress ratio and plotted with results of tests consolidated to an effective confining pressure of 40 psi. The effect on the stress ratio of the effective confining pressure is not believed to be significant, although these tests indicate the material to be slightly more stable at the lower confining pressure. Additional tests would be required for a more definitive evaluation.

17. Specimen 15 was consolidated anisotropically to a consolidation
ratio $K_c$ of 2.0 ($\bar{c}_{1c} = 10$ psi, $\bar{c}_{3c} = 5$ psi) and then given the most intense cyclic loading possible without producing a tensile stress in the specimen; i.e., $\sigma_{dc} = 9.91$ psi. The specimen reached initial liquefaction in 2 cycles at 1.4 percent strain,* but 60 cycles were required for 20 percent strain.

18. Figures 10, 11, and 12 show significant portions of the actual test records of tests 2, 4, and 9a, respectively. These are typical tests at each of the three relative densities investigated. The records show how the tendency to deform under cyclic loading is much greater in test 2 at 40.5 percent relative density than in test 9a at 77.1 percent relative density.

**Discussion of Results**

19. Specific conclusions and recommendations can be made when the seismological and related geological investigations have been conducted. These will form the basis for selecting seismic ground motions for design purposes. However, based on the data obtained, the advantage of obtaining an in-place relative density of at least 75 percent is obvious (see Figure 8). Since an average density of only 50 to 60 percent** is obtained by placing sand underwater without compaction, it can be expected that sand fills placed underwater will require compaction. Means for underwater compaction are discussed by Johnson, Compton, and Ling** and are being further reviewed at WES.

20. After the structure has been analyzed for static conditions and a preliminary seismic analysis has been made for the ground motions selected for seismic design, more anisotropic cyclic triaxial tests may be required to determine the response of material in the embankment and inside the cofferdam cells. At that time, the need for a dynamic

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* For anisotropic consolidated tests, strain is measured zero to peak and is always compressive.

analysis can be determined together with compaction criteria required for fill placed underwater, including that placed in the cells.
### Table 1
Summary of Test Results, Newburgh Lock and Dam Liquefaction Study

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<th>Test No.</th>
<th>( \gamma )pcf</th>
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<th>Back Pressure</th>
<th>( \sigma_a ) or ( \sigma_{3r_c} ) psi</th>
<th>( \sigma_{3c} ) psi</th>
<th>( \sigma_{dc} ) psi</th>
<th>Acceleration ( g )'s</th>
<th>Cycles to Initial Liquefaction</th>
<th>Cycles to 10 Percent Strain*</th>
<th>Cycles to 20 Percent Strain*</th>
<th>Remarks</th>
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<td>40</td>
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<td>6.0</td>
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<td>65</td>
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<td>5</td>
<td>9.91</td>
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<td>--</td>
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<td>1.4</td>
<td>1.4</td>
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<tr>
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<td>0.451</td>
<td>0.340</td>
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<td>2.7</td>
<td>1.8</td>
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</table>

Note: Symbols defined in Appendix A, Notation.

* The number shown in parenthesis is the actual strain.
Figure 1. Grain size curve for material used for cyclic triaxial testing
Figure 2. Axial strain versus cycles of loading for tests 2, 4, and 9a
Figure 3. Cyclic triaxial tests of material at a relative density of 43.3% (average of tests 1, 2, and 3)
Figure 4. Cyclic triaxial tests of material at a relative density of 58.8% (average of tests 4, 5, and 6)
Figure 5. Cyclic triaxial tests of material at a relative density of 78.2% (average of tests 7, 8, and 9a)
The back pressure was 100 psi; therefore when pore pressure measured -100 psi, the pore water pressure was at atmospheric pressure.

Figure 6. Monotonic load test of material at a relative density of 74.8% where \( \sigma_{3c} = 10 \) psi (test 12)
Figure 7. Summary of all tests at $\sigma_{3c} = 40$ psi
Figure 8. Cyclic deviator stress versus relative density

- $n = 20$ cycles
- $\sigma_{3c} = 40$ psi

Cyclic Deviator Stress $\sigma_{dc}$, psi

Relative Density ($D_d$)

10% Strain
20% Strain
Initial Liquefaction

Stress Ratio $\sigma_{dc}/\sigma_{ca}$

0.500
0.375
0.250
0.125
0.000
Figure 9. Comparison of test results at $\sigma_{3c} = 40$ psi and $\sigma_{3c} = 10$ psi, relative density = 76.5%
Figure 10. Record for test 2, relative density = 40.5%
Figure 11. Record for test 4, relative density = 58.8%
Figure 12. Record for test 9a, relative density = 77.1%
Appendix A: Notation

\[ B \]  
Skempton's B-parameter, the ratio of the change in pore pressure \( \Delta u \) to an induced change in chamber pressure \( \Delta \sigma_3 \) in an undrained state

\[ D_d \]  
Relative density

\[ K_c \]  
Consolidation ratio

\[ \gamma_d \]  
Dry density

\[ \Delta u \]  
Change in pore pressure

\[ \Delta \sigma_3 \]  
Change in chamber pressure

\[ \sigma_a \]  
Ambient confining pressure, psi, computed on the basis of in situ overburden weights

\[ \sigma_{dc} \]  
Cyclic deviator stress, psi

\[ \sigma_1 \]  
Major principal stress

\[ \sigma_3 \]  
Confining stress

\[ \sigma_{1c} \]  
Effective axial stress at the end of consolidation

\[ \sigma_{3c} \]  
Effective confining stress at the end of consolidation