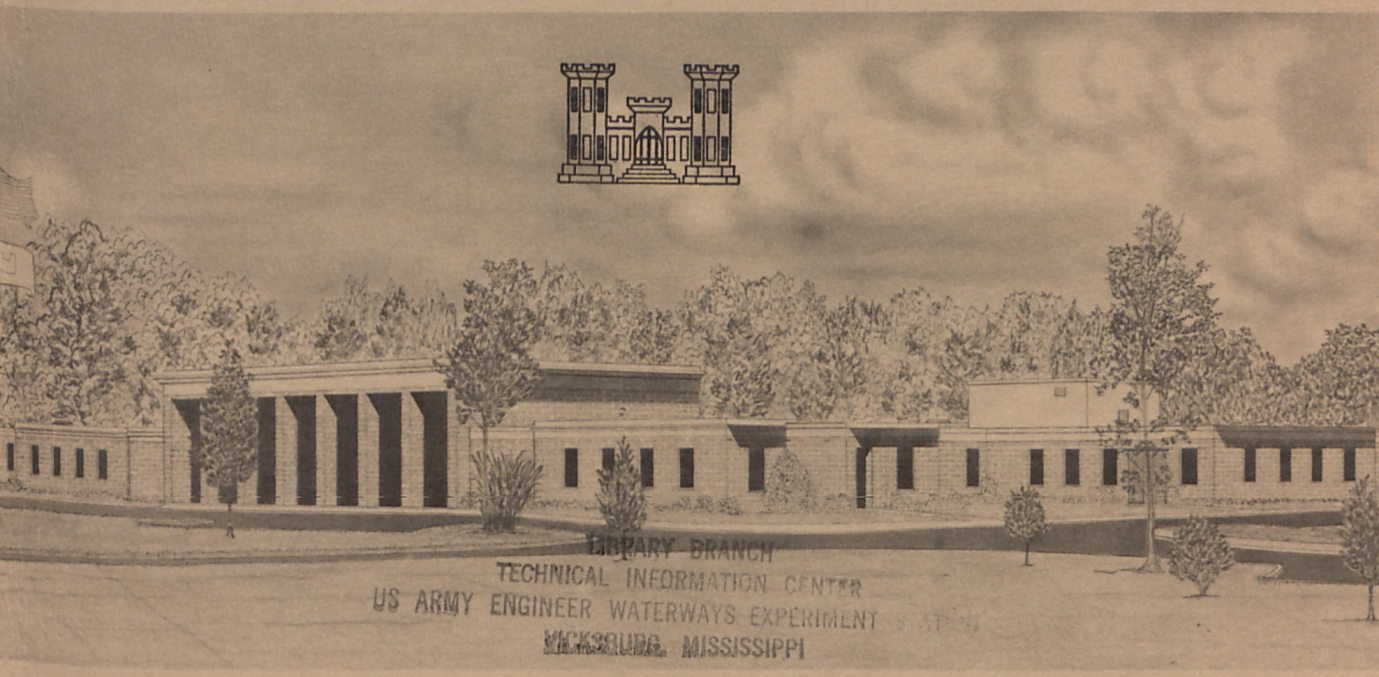
MISCELLANEOUS PAPER S-74-5

CYCLIC TRIAXIAL COMPRESSION TESTS CENTER HILL DAM, DEKALB COUNTY TENNESSEE

by

W. F. Marcuson III, S. A. Collins



April 1974

Sponsored by U. S. Army Engineer District, Nashville

Conducted by U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
Vicksburg, Mississippi

MISCELLANEOUS PAPER S-74-5

CYCLIC TRIAXIAL COMPRESSION TESTS CENTER HILL DAM, DEKALB COUNTY TENNESSEE

by

W. F. Marcuson III, S. A. Collins

April 1974

Sponsored by U. S. Army Engineer District, Nashville

Conducted by U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
Vicksburg, Mississippi

ARMY-MRC VICKSBURG, MISS.

Foreword

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, Nashville, and was authorized in Intra-Army Order No. 73-C-4, dated 21 Aug 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 R. G. Ahlvin and J. P. Sale, Assistant Chief and Chief, respectively, Soils and Pavements Laboratory. This report was prepared by Dr. Marcuson and Mr. Collins and was reviewed by Mr. S. J. Johnson, Special Assistant, Soils and Pavements Laboratory.

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

Contents

Conversion Factors, British to Metric Units of Measurement

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
pounds (force) per square inch	0.689476	newtons per square centimeter
pounds per cubic foot	16.01846	kilograms per cubic meter

Summary

The Center Hill Dam on the Caney Fork River, DeKalb County, Tennessee, is an earth-fill structure with a concrete overflow portion. A laboratory study was conducted to evaluate the earthquake response of the embankment material.

To evaluate the earthquake response, the U. S. Army Engineer District, Nashville, supplied the U. S. Army Engineer Waterways Experiment Station (WES) with undisturbed samples obtained from beneath the center line of the embankment. The samples were classified as either CH or CL soils. Natural dry densities ranged from 95 to 125 pcf.

Cyclic triaxial compression tests were conducted on the undisturbed samples. Based on the results of these tests, it appears that the material is relatively stable with respect to liquefaction. Due to the variation in physical properties and dry density, however, it is felt that more tests should be performed if a dynamic finite element analysis of the dam is to be performed.

CYCLIC TRIAXIAL COMPRESSION TESTS, CENTER HILL DAM, DEKALB COUNTY, TENNESSEE

Purpose and Scope

1. The purpose of this study was to obtain laboratory test data for a preliminary evaluation of the stability of Center Hill Dam during earthquake loading. The data were generated by performing cyclic load triaxial compression tests on undisturbed samples obtained by the U. S. Army Engineer District, Nashville, from beneath the center line of the embankment. The testing program consisted of the cyclic loading of test specimens consolidated under isotropic and two anisotropic stress conditions.

Material

2. Four 6-in.*-diam undisturbed samples were received from the Nashville District. These samples, numbered 18, 19, 21, and 23, were taken in boring UD-1 and corresponded to depths below the crest of 85.0-86.4, 90.0-92.0, 97.8-99.8, and 110.0-110.8 ft, respectively. Each sample consisted of yellowish-tan clay with occasional 2-in.-diam and smaller chert chips. The tests were performed on undisturbed specimens trimmed from these samples. Tests 1 and 2 used specimens from sample 21; test 3 was a specimen from sample 19; test 4 was a specimen from sample 23; and tests 5 and 6 were performed on specimens from sample 18. Grain-size distribution curves for specimens used for tests 1 and 3 are shown in figs. 1 and 2. Two chert chips were found in the specimen after test 1 and were removed before the grain-size analysis was made. Specimens tested were classified as either CH or CL and their densities ranged from 95.3 to 124.4 pcf.

---

* A table of factors for converting British units of measurement to metric units is presented on page vii.

Classification, water contents, Atterberg limits, and other soil-property data are summarized in table 1.

Testing Equipment

3. In cyclic triaxial compression tests performed at WES, the deviator stress is uniformly increased and decreased, while a constant chamber pressure is maintained. The cyclic axial loads are applied by a pneumatic loading unit which consists of regulators and solenoid valves actuated by a cam-operated microswitch. These solenoid valves provide alternating air pulses to the double-acting load cylinder so that a load is cyclically applied from the load cylinder through the connecting piston to the specimen. In testing the Center Hill material, the frequency of loading was 2 Hz for isotropic conditions and 1 Hz for anisotropic conditions;* the repeated loads were applied in a rectangular stress-time wave form. The tests performed were of the consolidated-undrained type. 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 recorded using a high-speed continuous line recorder. The equipment was carefully calibrated prior to testing.

Testing Procedures

Specimen preparation

4. Six specimens were trimmed from the four available samples. Each specimen was initially cut from the sample to a length of 6.5 in. This resulted in a cylinder 6.5 in. high with a 6-in.-diam base. The base was placed upon a pedestal that had vertical guides for trimming. Excess material was trimmed away to obtain a specimen with a base

---

* WES data indicate that a variation in frequency of 1 Hz has little effect on specimen response.

diameter of approximately 2.7 in. and a height of 6.5 in. Because of sample disturbance, only six specimens could be trimmed from the samples furnished. After the diameter of each specimen was measured at top, midheight, and bottom, it was placed in a triaxial testing chamber. Vertical filter strips were placed around the specimen to aid in saturation and consolidation. A membrane stretcher was used to place a 0.012-in.-thick rubber membrane over the specimen and filter strips; the membrane was sealed to the top and base caps by an O-ring.

Saturation

5. The process of saturation consisted of two phases: air evacuation and application of back pressure. The triaxial chamber was assembled and a vacuum applied to the top and base caps. This removed air from the top and base caps, filter stones, and filter strips. The vacuum was relaxed and distilled water was allowed to replace the air that had been evacuated. When this phase had been completed, back-pressure saturation, consisting of simultaneously increasing the chamber pressure and back pressure, was used to saturate the specimen. Back pressures ranged from 76 to 143 psi as seen in table 2. The degree of saturation under applied back pressure is expressed in terms of Skempton's B-parameter. The B-parameter is the ratio of the change in pore water pressure Δu to an induced change in hydrostatic chamber pressure, $\Delta\sigma_3$, in an undrained state, i.e.,

$$B = \frac{\Delta u}{\Delta\sigma_3} \tag{1}$$

The B-value was determined by closing the drainage line, increasing the chamber pressure 10 psi, and observing the increase in pore water pressure. The B-value for these tests ranged from 0.40 to 1.00, as shown in table 2. The back pressure remained on the specimen for a maximum period of three days. Three B-values of less than the desired minimum of 0.95 were reluctantly accepted due to time

constraints. Maximum back pressures applied were governed by the apparatus chamber material and sensitivity of the pore pressure transducer. During saturation, the change in height of the specimen was measured with a dial indicator read to the nearest 0.001 in.

Consolidation

6. After completion of the saturation phase, each specimen was consolidated to the effective confining pressure under which it would be tested. Consolidation was accomplished by increasing the chamber pressure and axial load while allowing drainage such that the final difference between the chamber pressure and the back pressure was the desired effective confining pressure, $\bar{\sigma}_3$. For anisotropically consolidated specimens, after the desired chamber pressure was achieved, the axial load was further increased while allowing drainage until the desired anisotropic consolidation stress ratio K_c was achieved, i.e.,

$$K_c = \frac{\bar{\sigma}_{1c}}{\bar{\sigma}_{3c}} \tag{2}$$

where

$\bar{\sigma}_{1c}$ = effective axial stress at end of consolidation

$\bar{\sigma}_{3c}$ = effective confining stress at end of consolidation

Cyclic loading

7. Cyclic loading consisted of the cyclic application and reduction of an increment of axial stress σ_{dc} 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.

Test Results

8. The results of all tests are summarized in table 2. A description of special aspects of selected tests follows.

Isotropic consolidation

9. Test 1. This test was performed on an isotropically consolidated specimen with a B-value of 0.40 and a dry density γ_d of 95.3 pcf. Pore pressure increased 45 psi during the test, but the pore pressure was always at least 20 psi less than the chamber pressure. After the test, the specimen was examined. It had sheared along a 45-deg plane and two oblong chert chips about 1 in. in diameter were found in the shear zone.

10. Test 2. Test 2 was performed on an isotropically consolidated specimen with a B-value of 0.51, a dry density of 104.3 pcf, and a cyclic stress of 35.3 psi.

11. Test 6. Test 6 was performed on an isotropically consolidated specimen with a B-value of 0.98, a dry density of 124.4 pcf, and a cyclic stress of 52.4.

Anisotropic consolidation

12. Test 3. Test 3 was performed on an anisotropically consolidated specimen with a K_c of 1.5, a B-value of 0.97, and an initial dry density of 108.2 pcf. The first cyclic stress applied (test 3a) was 15.2 psi, and the specimen underwent 0.3 percent strain ε (zero-to-peak compression) in 500 cycles. The pore pressure increased 38.5 psi during the test, at which time it was 5 psi below the chamber pressure. The loading was stopped, the excess pore water pressure was relieved, and the specimen retested (test 3b) at a cyclic stress of 22.2 psi. After 500 cycles, the zero-to-peak compressive strain was 0.2 percent and the pore pressure had increased 28.5 psi. The loading was stopped, excess pore water pressure was relieved, and the specimen retested (test 3c) at a cyclic stress of 30.2 psi. A zero-to-peak compressive strain of 5 percent occurred in 660 cycles. After 1030 cycles, the strain was 8.5 percent (zero-to-peak compression) and cyclic testing was stopped. The specimen was taken out of the triaxial apparatus and tested in unconfined compression; it failed under a load of 700 lb or 108 psi.

13. Test 4. Test 4 was conducted on a specimen consolidated anisotropically to a K_c of 2.0 with a B-value of 0.43, a dry density

of 115 pcf, and a cyclic stress of 48.0 psi.

14. Test 5. This test was conducted on a specimen anisotropically consolidated to a K_c of 1.5 with a B-value of 1.0, a dry density of 119.1 pcf, and a cyclic stress of 32.7 psi. This test is similar to test 3.

Specimen Behavior

15. At the beginning of a cyclic triaxial compression test, the effective $\bar{\sigma}_{3c}$ stress is the difference between the pore pressure (equal to the back pressure) and the chamber pressure. As the specimen is cyclically loaded, the pore pressure increases, consequently reducing the effective stress. For loose specimens of sand and silt, large deformations, accompanied by a severe loss of strength, occur soon after the pore pressure first becomes equal to the chamber pressure. Probably because of the cohesive nature of the material tested, there was no sudden loss of strength when the pore pressure first equaled the chamber pressure. The strain increased slowly with each cyclic application of load and continued to do so even after the pore pressure equaled the chamber pressure. This behavior under isotropic consolidation is shown in the record for test 2 (fig. 3), which reached 5 percent strain (peak-to-peak) in 11 cycles, 10 percent strain in 23 cycles, and 20 percent strain in 98 cycles.

16. Fig. 4 shows the record for test 4 (anisotropic consolidation), which reached 5 percent compressive strain (zero-to-peak) in 1 cycle, 10 percent strain in 2 cycles, and 20 percent strain in 20 cycles. In test 5 (anisotropic consolidation), the specimen underwent 66 cycles before reaching 5 percent compressive strain (zero-to-peak) and 113 and 254 cycles before reaching 10 and 20 percent strains, respectively. This strain behavior is also true for a specimen whose pore pressure never equaled the chamber pressure as in test 1, illustrated in fig. 5. Test 1 is valid only to 81 cycles because the compressive strain equaled the limit of travel permitted for this test as can be seen in fig. 5.

17. Fig. 6 is a plot of the stress ratio versus number of cycles of loading for specimens consolidated isotropically. Because of the variations in density from 95.3 to 124.4 pcf and in the types of material (see table 1) of the specimens tested, no definitive curves could be drawn through the data points for each strain level; instead dashed curves were drawn that estimate the probable behavior at 5 percent strain for the various materials tested.

18. Fig. 7 is a summary plot of stress ratio versus number of cycles of loading for all the tests conducted for this study. Again, the variations of the densities and the types of material of the specimens tested prevented drawing quantitative conclusions. Data from tests 3 and 5, shown in table 2, indicate that the stress history during testing may be significant; the cyclic stress ratios of these two tests were quite similar, but the responses were significantly different. Test 5 experienced 5 percent strain in 66 cycles, while test 3 took 10 times that many cycles (660) to strain 5 percent. Test 3 (see paragraph 12) experienced an atypical stress history and this may be responsible for the increase in the number of load cycles; however, the density of the specimen in test 3 was about 10 pcf less than the specimen used in test 5. Further, the data from tests 3 and 5 shown in fig. 7 indicate that material similar to that tested but located away from the center line of the embankment, with a K_c of 1.5, would be somewhat more resistant to cyclic deformation than material at the center of the dam, with a $K_c = 1.0$. On the other hand, test 4 at a K_c of 20 indicates that this particular material would experience large deformations with a small number of applications of a relatively large deviator stress.

Recommendations

19. The tests conducted have provided insight into the dynamic behavior of the Center Hill material tested, but the data are insufficient to support quantitative conclusions concerning the dynamic behavior of the earth dam during an earthquake. If a dynamic finite

element analysis of the embankment is planned, additional laboratory tests should be conducted.

20. Undisturbed samples should be obtained from two additional depths at the center line of the embankment and at two depths under the upstream slope if possible. Cyclic triaxial tests should be conducted at consolidation stress ratios K_c of 1.0 and 1.5 for the center-line material and K_c of 1.0, 1.5, and 2.0 for the slope material. Because of the variations in the type and density of materials in specimens representative of relatively small changes in depth in a borehole, sufficient quantities of material should be obtained to run duplicate tests in order to obtain reliable data.

Table 1

Soil Classification Data

Test No.	Sample No.	Depth ft	Classification	Specific Gravity	Natural Water Content W, %	Natural Dry Density γ_d, pcf	Atterberg Limits LL	PL	PI	% <0.002 mm
				Isotropic Consolidation						
1	21	97.8- 99.8	CH	2.75	30.0	95.3	58	23	35	50
2	21	97.8- 99.8	CH	--	23.0	104.3	--	--	--	--
6	18	85.0- 86.4	CL	--	16.1	124.4	--	--	--	--
				Anisotropic Consolidation						
3	19	90.0- 92.0	CL	2.69	17.5	109.0	29	16	13	20
4	23	110.0-110.8	CL	--	14.4	115.0	--	--	--	--
5	18	85.0- 86.4	CL	--	13.8	119.1	--	--	--	--

Table 2

Summary of Cyclic Triaxial Test Results

Test No.	Density γ_d pcf	Back Pressure psi	B	$\bar{\sigma}_{3c}$ psi	σ_{dc} psi	$\frac{\sigma_{dc}}{2\sigma_3}$	Cycles to 5% Strain*	Cycles to 10% Strain*	Cycles to 20% Strain*	K_c
					Isotropic Consolidation					
1	95.3	95.0	0.40	65.0	25.9	0.199	64(5.0)	81(9.4)	--	1.0
2	104.3	86.3	0.51	64.6	35.3	0.273	11(5.2)	23(10.1)	98(18.7)	1.0
6	124.4	75.5	0.98	65.1	52.4	0.403	6(5.1)	14(20.0)	90(20.0)	1.0
					Anisotropic Consolidation**					
3(a)	108.2	101.3	0.97	43.5	15.2	0.175	500(0.3)	500(0.3)	--	1.5
3(b)	108.8	101.1	0.97	43.4	22.2	0.252	500(0.2)	500(0.2)	--	1.5
3(c)	109.0	101.1	0.97	44.1	30.2	0.342	660(5.0)	1030(8.5)	--	1.5
4	115.0	142.9	0.43	32.3	48.0	0.743	1(7.1)	2(10.4)	20(20.0)	2.0
5	119.1	101.8	1.00	43.6	32.7	0.374	66(5.0)	113(10.0)	254(20.0)	1.5

Note: Terms defined in text.

* Number of cycles shown is the cycle closest to 5%, 10%, and 20% strain for the two columns. The number in parentheses is the actual peak-to-peak or zero-to-peak strain, whichever was greater for that cycle. Peak-to-peak strain is the sum of compression and extension strains.

** $\bar{\sigma}_{1c}$ is approximately the same as for isotropic consolidation.

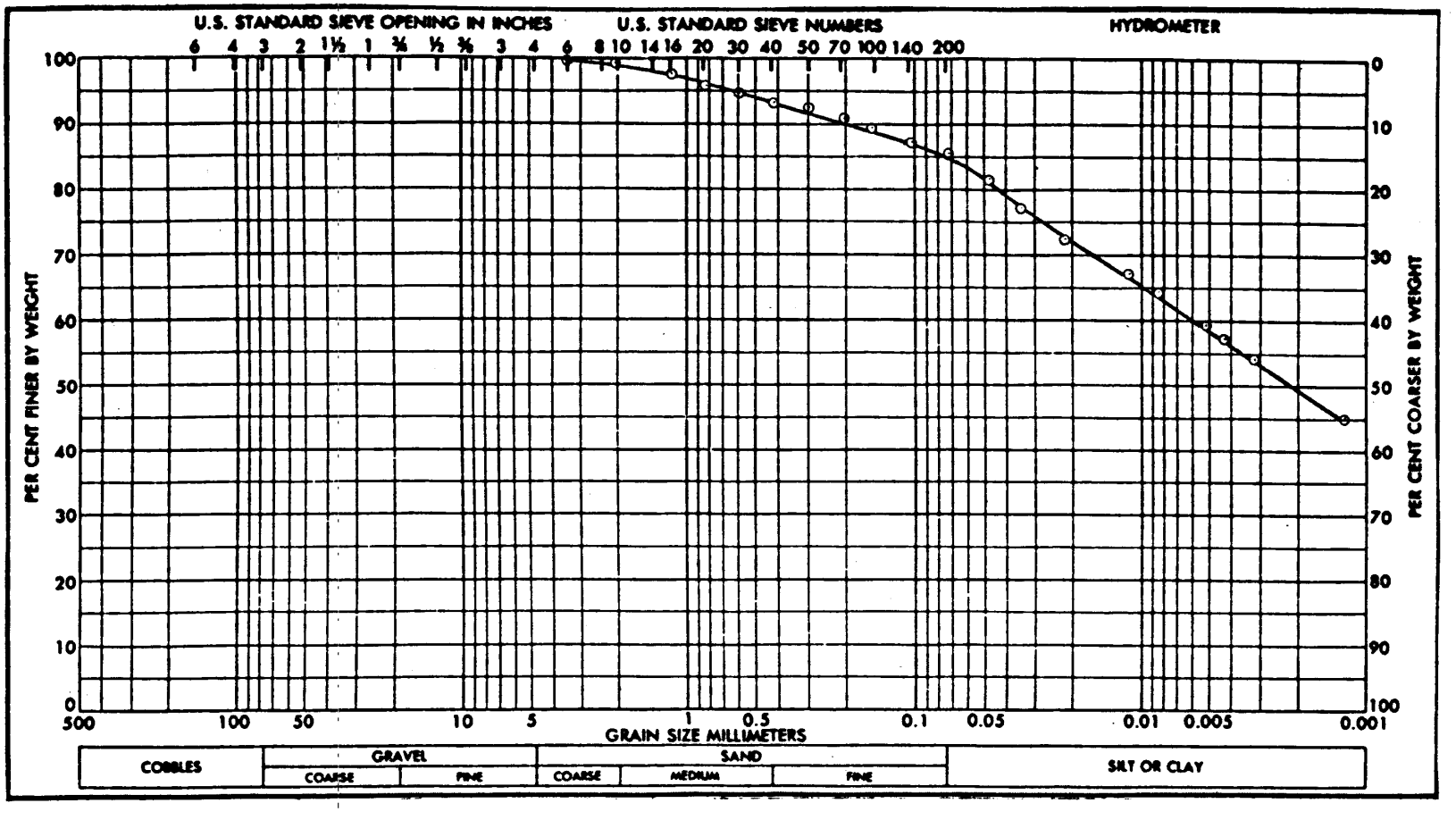


Fig. 1. Grain-size curve of material used for test 1

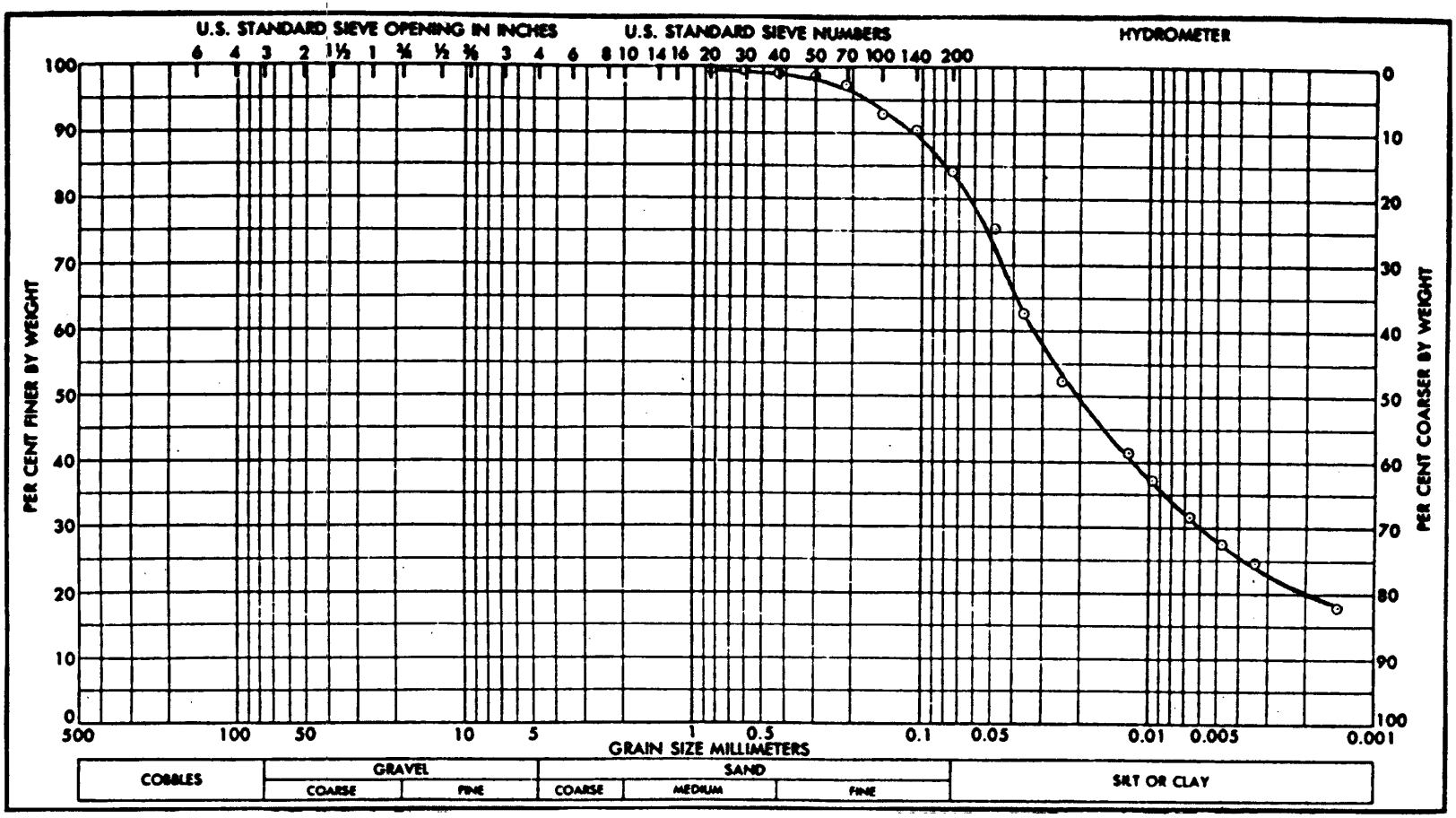


Fig. 2. Grain-size curve of material used for test 3

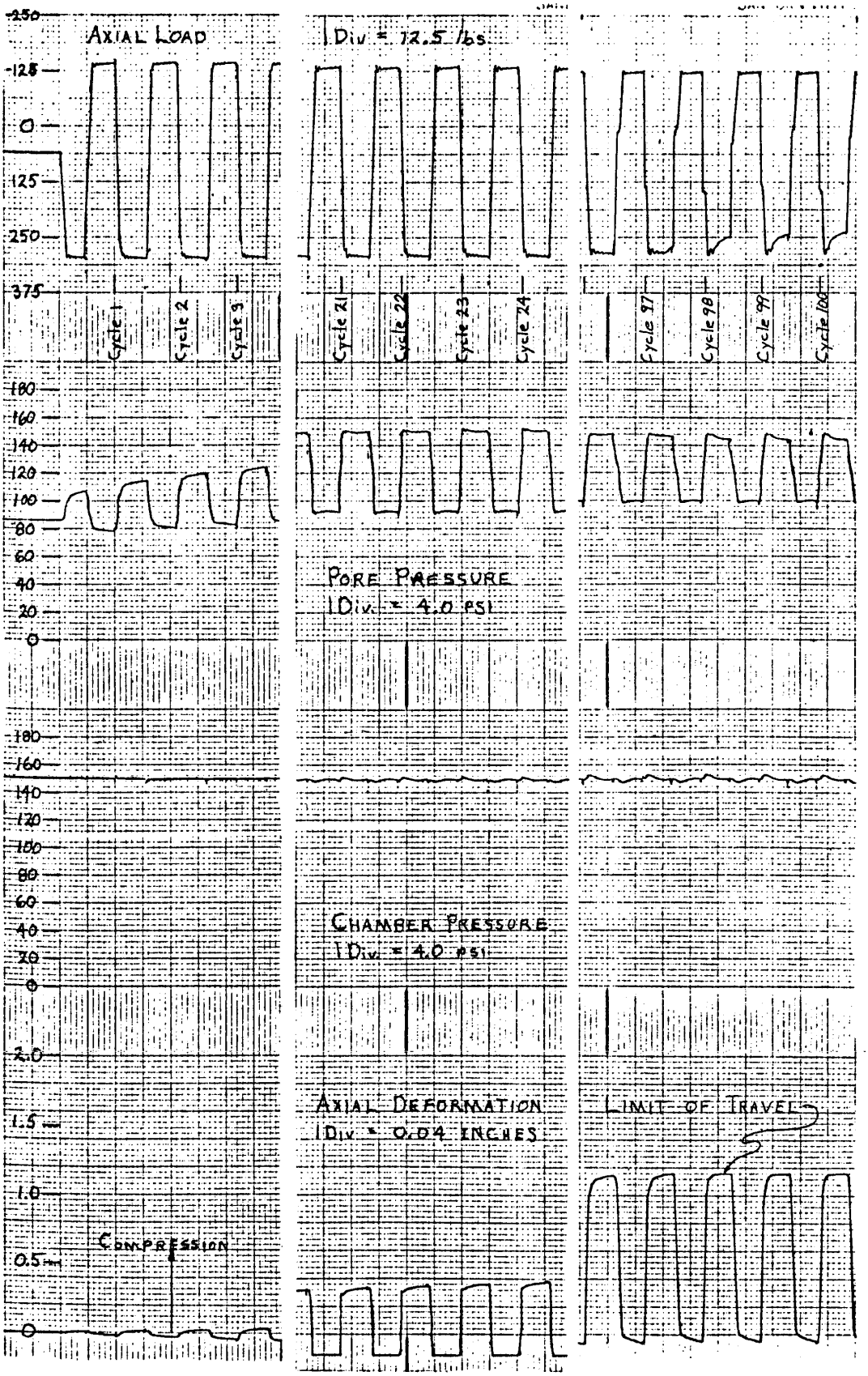


Fig. 3. Record for test 2, isotropic consolidation

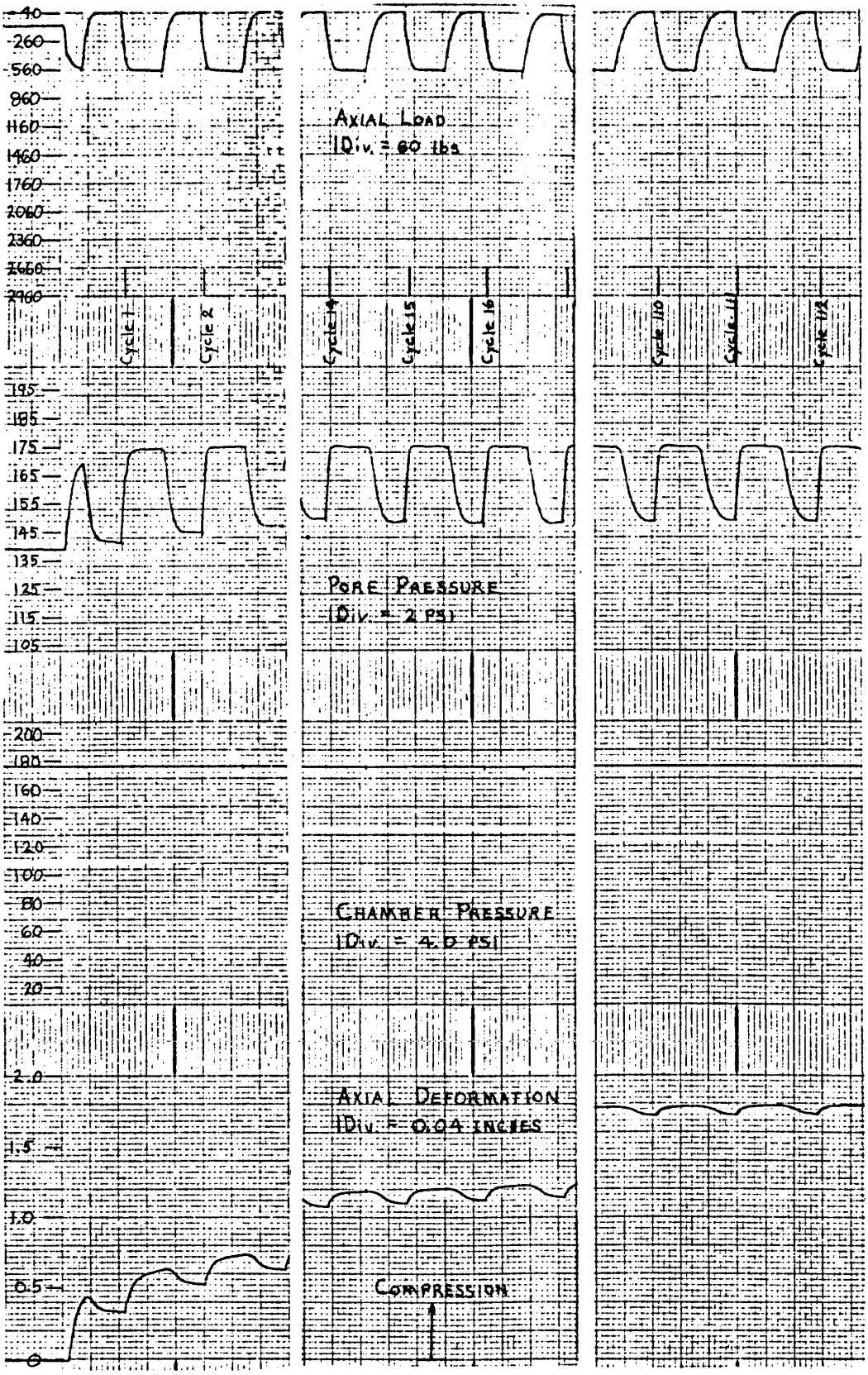


Fig. 4. Record for test 4, anisotropic consolidation

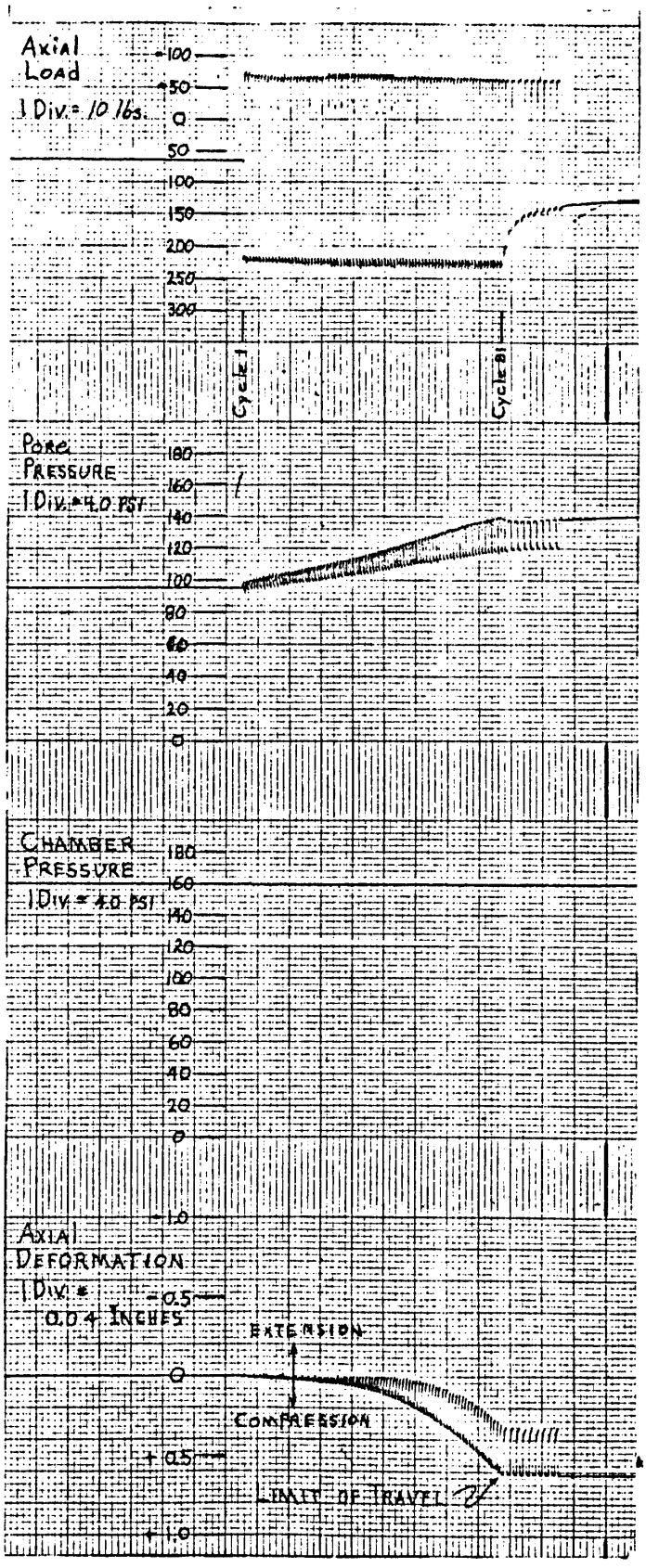


Fig. 5. Record for test 1, isotropic consolidation

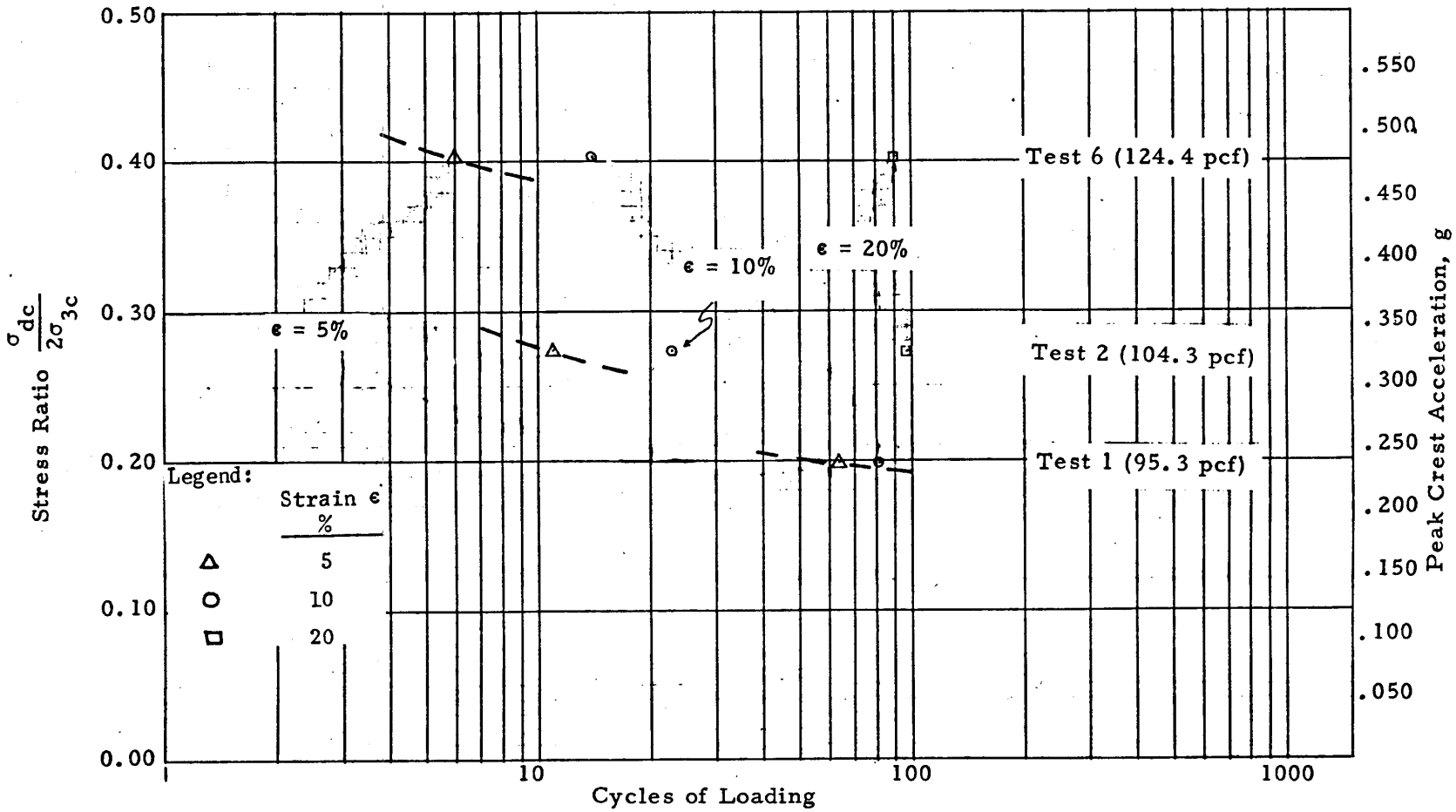


Fig. 6. Stress ratio versus cycles of loading for isotropic consolidation tests 1, 2, and 6 (K_c = 1.0)

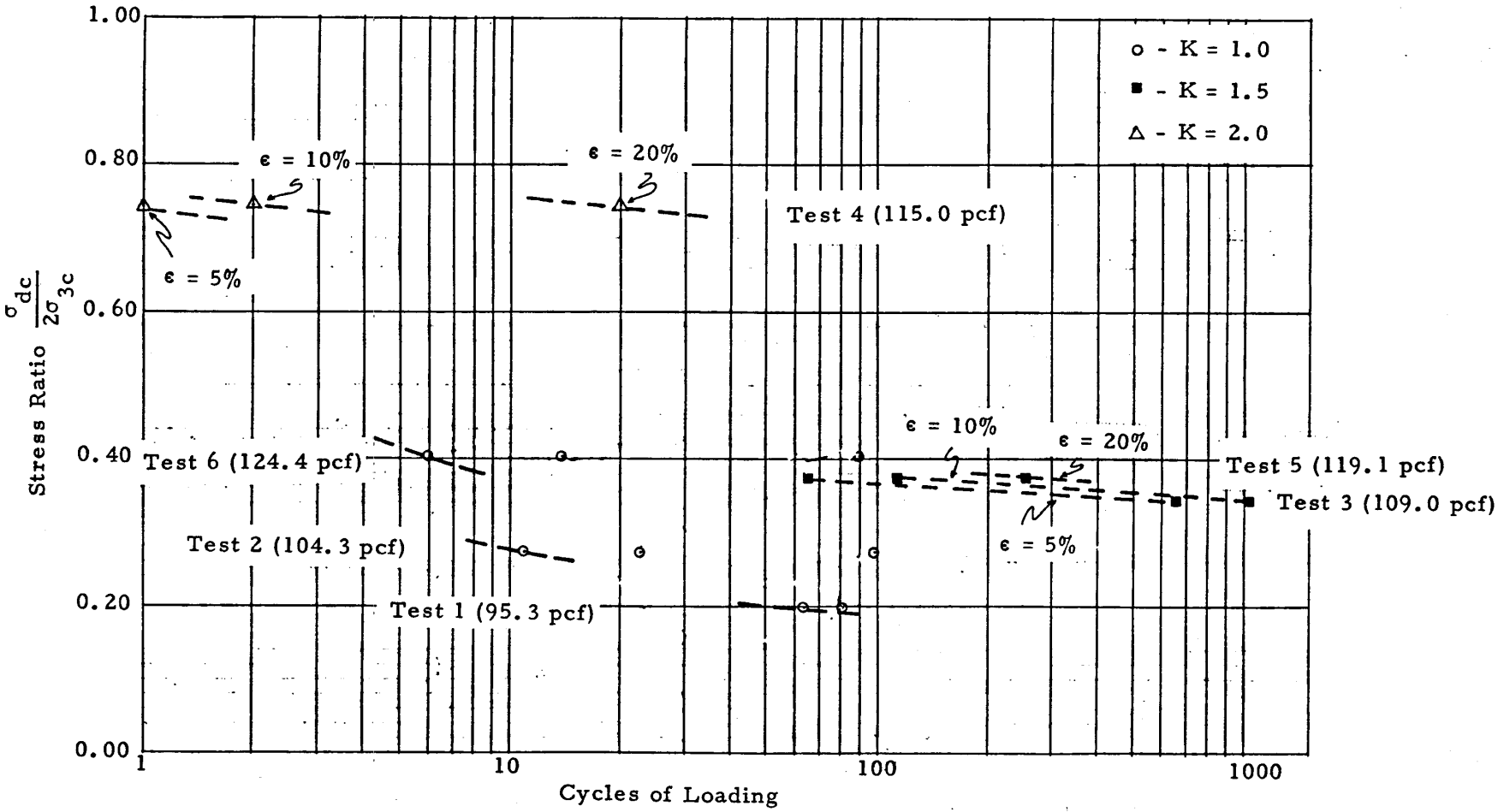


Fig. 7. Stress ratio versus cycles of loading

Unclassified

Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY *(Corporate author)*

U. S. Army Engineer Waterways Experiment Station
Vicksburg, Mississippi

2a. REPORT SECURITY CLASSIFICATION

Unclassified

2b. GROUP

3. REPORT TITLE

CYCLIC TRIAXIAL COMPRESSION TESTS, CENTER HILL DAM, DEKALB COUNTY, TENNESSEE

4. DESCRIPTIVE NOTES *(Type of report and inclusive dates)*

Final report

5. AUTHOR(S) *(First name, middle initial, last name)*

William F. Marcuson III
Stephen A. Collins

6. REPORT DATE

April 1974

7a. TOTAL NO. OF PAGES

22

7b. NO. OF REFS

8a. CONTRACT OR GRANT NO.

b. PROJECT NO.

c. Intra-Army Order No. 73-C-4

d.

9a. ORIGINATOR'S REPORT NUMBER(S)

Miscellaneous Paper S-74-5

9b. OTHER REPORT NO(S) *(Any other numbers that may be assigned this report)*

10. DISTRIBUTION STATEMENT

Approved for public release; distribution unlimited.

11. SUPPLEMENTARY NOTES

12. SPONSORING MILITARY ACTIVITY

U. S. Army Engineer District, Nashville
Nashville, Tennessee

13. ABSTRACT

The Center Hill Dam on the Caney Fork River, DeKalb County, Tennessee, is an earthfill structure with a concrete overflow portion. A laboratory study was conducted to evaluate the earthquake response of the embankment material. To evaluate the earthquake response, the U. S. Army Engineer District, Nashville, supplied the U. S. Army Engineer Waterways Experiment Station (WES) with undisturbed samples obtained from beneath the center line of the embankment. The samples were classified as either CH or CL soils. Natural dry densities ranged from 95 to 125 pcf. Cyclic triaxial compression tests were conducted on the undisturbed samples. Based on the results of these tests, it appears that the material is relatively stable with respect to liquefaction. Due to the variation in physical properties and dry density, however, it is felt that more tests should be performed if a dynamic finite element analysis of the dam is to be performed.

DD FORM 1 NOV 65 1473 REPLACES DD FORM 1473, 1 JAN 64, WHICH IS OBSOLETE FOR ARMY USE.

Unclassified

Security Classification

Unclassified

Security Classification

14. KEY WORDS	LINK A ROLE	LINK A WT	LINK B ROLE	LINK B WT	LINK C ROLE	LINK C WT
Center Hill Dam						
Earth dams						
Earthquake resistant structures						
Dynamic tests						
Soil tests						
Triaxial shear tests						
Vibration						

Unclassified

Security Classification