REPORT NO. 1

LABORATORY SOIL TESTING

BIG RIVER DAM

WEST GREENWICH, RHODE ISLAND

Submitted to

U.S Army Corps of Engineers
New England Division

Presented by

Geotechnical Engineers Inc.
1017 Main Street
Winchester, Massachusetts 01890
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Project 78484

October 18, 1978

Karl Dalenberg
Assistant Project Manager

Gonzalo Castro
Principal

GEOTECHNICAL ENGINEERS INC.
1017 MAIN STREET · WINCHESTER · MASSACHUSETTS 01890 (617) 729-1625

October 20, 1978
Project 78484

Mr. Joe B. Fryer, Chief
Engineering Division
U.S. Army Corps of Engineers, NED
424 Trapelo Road
Waltham, MA 02154

Subject: Big River Dam Testing - Phase I

Dear Mr. Fryer:

Enclosed please find four copies of our report entitled "Report No. 1 Laboratory Soil Testing, Big River Dam, West Greenwich, Rhode Island."

We appreciate having had the opportunity to perform this testing.

Very truly yours,

GEOTECHNICAL ENGINEERS INC.

Gonzalo Castro
Principal

GC:ck

Enclosures

Copy to: Charles G. Tiersch
Chief, Foundations & Materials Branch
w/enc.

GEOTECHNICAL ENGINEERS INC.

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October 24, 1978
Project 78484

Mr. Joe B. Fryer, Chief
Engineering Division
U.S. Army Corps of Engineers, NED
424 Trapelo Road
Waltham, MA 02154

Subject: Big River Dam
West Greenwich, Rhode Island

Dear Mr. Fryer:

The purpose of this letter is to comment on the results of the laboratory testing performed on a sand sample for the subject project and presented in our Report No. 1 dated October 18, 1978.

The tests were performed on a uniform fine sand obtained by the USCE from a trench at the dam site. The tests were performed on specimens compacted to a unit weight equal to 100% of the maximum density obtained by the USCE in a standard compaction test. For the density at which it was tested, the test results indicate the following:

1) The sand is strongly dilative for confining pressures of up to at least 4 kg/cm^2 and, therefore, is not susceptible to liquefaction under these conditions.

2) The potential for deformations which can be induced by earthquake loading is a function of the design earthquake and can be analyzed using the test results summarized in Figs. 11, 12, and 13 in our Report No. 1.

Very truly yours,

GEOTECHNICAL ENGINEERS INC.

Gonzalo Castro, Principal

GC:ck

Copy to: Charles G. Tiersch
Chief, Foundations & Materials Branch
w/enc.

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1. INTRODUCTION

1.1 Purpose

The purpose of this report is to present the results of stress-controlled cyclic triaxial tests and stress-controlled consolidated-undrained triaxial compression tests performed on reconstituted specimens of sand taken from Foundation Trench FT-1 at the site of the Big River Dam in West Greenwich, Rhode Island.

1.2 Scope

Five bag samples, each containing about 50 pounds of moist sand, were delivered to Geotechnical Engineers Inc. by the U.S. Army Corps of Engineers for this testing program. The five bag samples, taken from the same stratum of material at a depth of 8 to 10 ft in Trench FT-1, were thoroughly mixed together and the following tests were performed:

- 7 Stress-controlled Cyclic Triaxial Tests
- 2 Stress-controlled Consolidated-Undrained Triaxial Compression Tests
- 1 Grain-Size Analysis

1.3 Authorization

This work was authorized by Contract No. DACW-33-78-M-3395 between the U.S. Army Corps of Engineers and Geotechnical Engineers Inc. (GEI).

2. OUTLINE OF TEST PROGRAM

2.1 Index Properties

Five bag samples of moist sand taken from a depth of 8 to 10 ft in FT-1 were thoroughly mixed together prior to performing any tests.

A grain size analysis was performed on the mixed sand and is shown on Fig. 1. The sample consists of a uniform fine sand (coefficient of uniformity = 3.0) with about 5% medium sand and 5% silt.

A Standard Proctor compaction test was performed by the U.S. Army Corps of Engineers. The maximum unit weight was 99.0 pcf at a water content of 17.2%.

2.2 Experimental Compaction of Triaxial Test Specimens

Prior to preparation of the triaxial test specimens, experimental compactions were performed to determine the distribution of density in the triaxial specimens.

Two specimens were compacted in a split mold lined with five individual rings. The rings were 1.4 in. high by 2.75 in. diameter and were fitted tightly in the mold. They effectively divided the compacted specimen into five sections of known volume so that the specimens could be easily segmented to determine the dry unit weight of each individual section.

Each specimen was compacted at the optimum water content of 17.2% in seven layers of equal weight and height. A layer was carefully placed in the mold, and its surface roughly leveled. A surcharge producing a vertical pressure of $\sim$1 psi was lowered to the soil. Each layer was compacted using an Ingersoll-Rand pneumatic hammer. A regulated air pressure of $\sim$15 psi was used to achieve low frequency, high amplitude vibrations. Vibrations were applied vertically through the surcharge until the surface of the layer settled to achieve the desired height corresponding to a unit weight equal to the Standard Proctor maximum unit weight. The next layer was then placed and the procedure repeated until all seven layers had been compacted to form the specimen of the desired unit weight, 99.0 pcf (100% of ASTM D-698).

The mold was subsequently removed, and the soil within each of the five rings was carefully removed and oven-dried to determine the dry density distribution within the total specimen.

The results of the two specimen compactions are presented in Fig. 2. The data indicate that some variation in density did occur over the length of the specimen, but that no trend in density variations vs height occurred. Had the lower layers shown a larger density than the average, it would have indicated that the compaction of each successive layer was increasing the density of the previous layers. Since this did not occur, which is typical in dense sands, the compaction procedure used for the trial compactions was also used for the test specimens.

The test procedures and results of the two consolidated-undrained triaxial compression tests and the seven cyclic triaxial tests are presented in Sections 3.0 and 4.0, respectively.

3. STRESS-CONTROLLED CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS

3.1 General

Two stress controlled consolidated-undrained triaxial compression tests ($\bar{R}$-tests) were performed on the combined sample of sand from FT-1. The consolidation stress conditions used for these tests were as follows:

$$\bar{\sigma}_{3c} = 1.0 \text{ kg/cm}^2, \; K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c} = 2.0$$

$$\bar{\sigma}_{3c} = 4.0 \text{ kg/cm}^2, \; K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c} = 1.0$$

3.2 Procedure

The required amount of air-dried soil was thoroughly mixed with water to achieve its optimum water content of 17.2%. This soil batch was subsequently divided into portions of equal weight for each layer and placed in closed containers. The first layer was then carefully placed into a split mold in which a rubber membrane had been inserted. The membrane was held tightly against the inside of the mold by means of a vacuum. The layer was leveled off and compacted, as described in Section 2.2. This process was repeated for each layer.

When the compaction was completed, the top drainage cap was mounted in place. The mold, specimen, and top cap were then placed in the triaxial cell. The rubber membrane was fitted around the top and bottom drainage caps and a vacuum of about 15 in. Hg was applied to the sample to provide some strength to the specimen. The mold was then removed and the dimensions of the specimen were measured. The cell was assembled and filled with water. After application of a small chamber pressure of 0.5 kg/cm^2, the vacuum was released by permitting water to enter the specimen. Height changes were observed and measured with a dial indicator. Water was percolated through the specimen to increase the degree of saturation. Back pressures ranging from 4.0 to 12.0 kg/cm^2 were applied to achieve a B-parameter $\geq$0.95. The test specimens were then consolidated to the desired effective stress conditions as indicated in Section 3.1. The anisotropic stress was applied in several small increments to minimize deformations due to undrained shear.

The drainage valves were then closed and an increment of deviator stress was applied. The increments ranged from 0.5 to 3.0 kg/cm^2, the smaller increments being applied at the beginning of the test. After each load increment was applied, readings of pore pressure were taken periodically until they stabilized, which occurred within 3 to 10 minutes. Subsequent load increments were applied until a strain of about 7 to 8 percent was reached.

3.3 Results

The results of the two $\bar{R}$ tests are presented in Fig. 3 by means of three plots.

Shear stress in the 45^o plane versus axial strain

Induced pore pressure versus axial strain

Stress path

3.4 Comments

The rate of loading during the test was slow enough so that the pore pressure could equalize throughout the specimen during each loading increment.

The test specimens were both strongly dilative at the effective stress levels used for the tests. The angles of internal friction were 37.0^o and 37.5^o for the two tests performed.

4. STRESS-CONTROLLED CONSOLIDATED-UNDRAINED CYCLIC TRIAXIAL TESTS

4.1 General

Seven stress-controlled consolidated-undrained cyclic triaxial tests ($C\bar{R}$ tests) were performed on specimens made from the combined samples from FT-1. The consolidation stress conditions imposed on the specimens were as follows:

Number of Tests	$\bar{\sigma}_{3c}$ kg/cm^2	$K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c}$
4	1.0	2.0
3	4.0	1.0

4.2 Procedure

The procedure for specimen preparation, saturation, and consolidation was identical to that described for the $\bar{R}$ tests in Section 3.2.

After consolidation was complete, the drainage valves were closed, a symmetrical cyclic deviator stress was applied to the specimen, and a continuous record was obtained of axial load, pore pressure, and deformation by means of a strip chart recorder.

4.3 Results

The results of the individual tests are presented in Figs. 4 to 10. Each figure is a plot of the following:

Peak cyclic deviator stress in compression and extension versus cycle number.

Peak axial strain in compression and extension and double amplitude versus cycle number.

Maximum induced pore pressure during each cycle versus cycle number.

Table 1 summarizes the $C\bar{R}$ test results. Three summary plots were prepared which show the relationship between the applied cyclic deviator stress and the number of cycles to reach a double amplitude strain of 2.5, 5 and 10%, Figs. 11 through 13, respectively.

4.4 Comments

The test specimens appear from visual observations to develop uniform cyclic straining at strains less than several percent. As cyclic strains increased, they concentrated in a rather wide zone generally within the top half of the specimen.

The pore pressure was measured at the ends of the specimen, thus it is not necessarily representative of the pore pressure in the most highly strained zone of the specimens because of the relatively rapid rate of loading normally used in these types of tests.

In general, the recorded pore pressure shows a substantial increase in the first cycle, particularly for the isotropically consolidated specimens and a gradual increase afterwards. In the first few cycles of the tests on isotropically consolidated specimens, the maximum pore pressure occurs under maximum compressive load. When the axial strains become about 1 to 2% during tests on isotropically consolidated specimens and for all cycles during tests on anisotropically consolidated specimens, the maximum pore pressure occurs whenever the deviator stress passes through zero with dilation occurring under compression and extension loads. The increase in the maximum pore pressure continues until it becomes almost equal to the confining pressure with the effective minor principal stress being reduced to 3% to 5% of the consolidation pressure. Thereafter it rises more slowly until it becomes equal to the confining pressure several cycles later. When the pore pressure becomes almost equal to the confining pressure, the stress strain behavior shows for the first time that some deformation occurs with little load increase followed by a stiffening both in compression and extension. Since this stress strain and pore pressure behavior is indicative of practically zero effective stress somewhere in the specimen, the cycle number in which this stress strain behavior was observed for the first time is reported in Table 1 as the number of cycles to reach $\bar{\sigma}_3 = 0$ rather than the number of cycles where the actual record shows $\bar{\sigma}_3 = 0$.

TABLE 1 - STRESS-CONTROLLED CYCLIC CONSOLIDATED-UNDRAINED TRIAXIAL TESTS BIG RIVER DAM, WEST GREENWICH, RHODE ISLAND

FOUNDATION TRENCH FT-1

DEPTH 8-10 ft

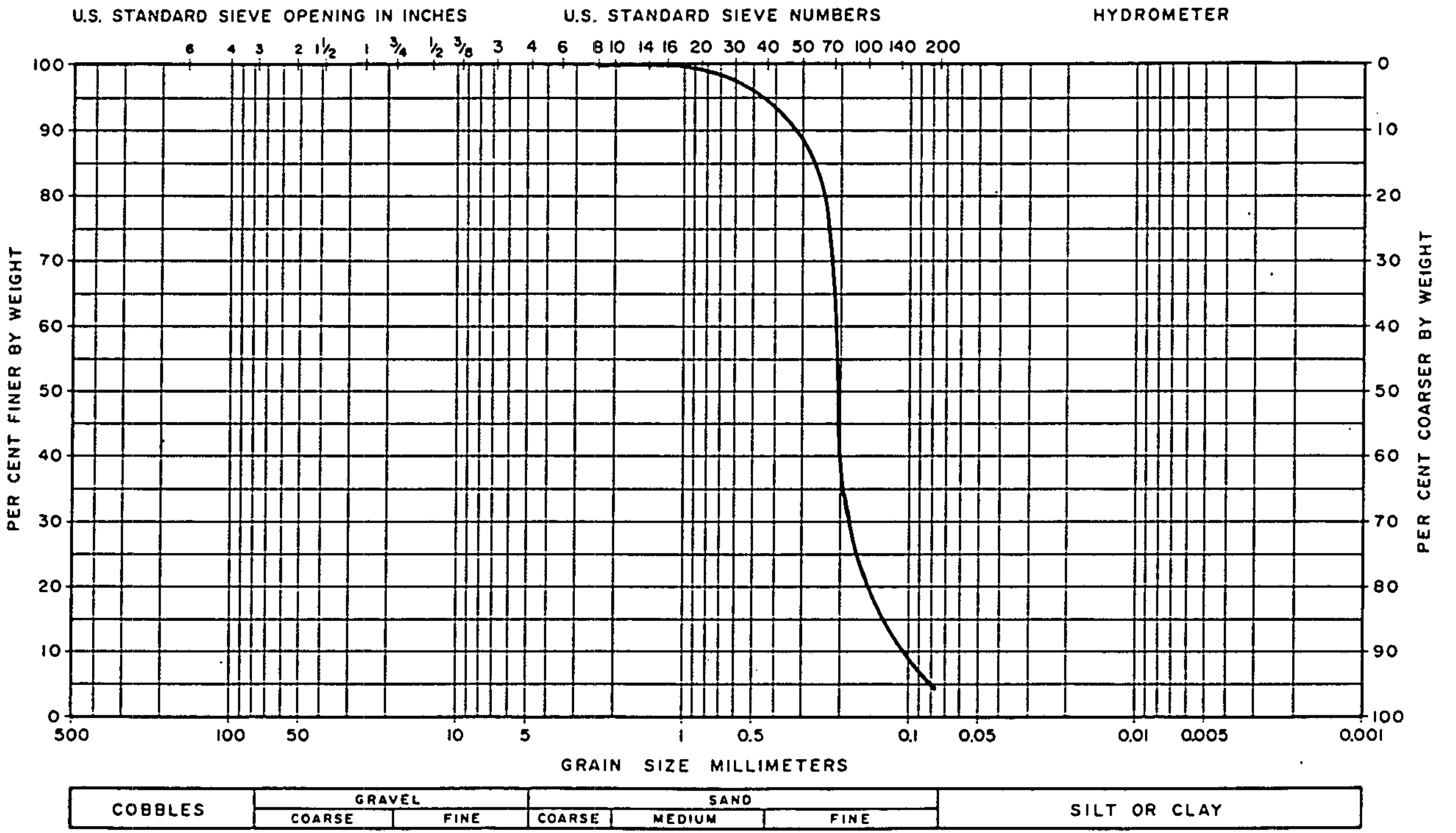
Test No.	Initial Water Content	Dry Unit Weights			Effective Consolidation Pressure	Consolidation Stress Ratio	Back Pressure	B Value	Cyclic Deviator Stress	Cyclic Stress Ratio	Number of Cycles to Reach			
		In Compaction Mold	In Triaxial Cell								$\bar{\sigma}_3 \doteq 0$	Double Amplitude Strain Equal to		
			Initial (1)	After Consolidation	$\bar{\sigma}_{3c}$	K_c $\bar{\sigma}_{1c}/\bar{\sigma}_{3c}$	u_o		$(\sigma_1 - \sigma_3)_{cy}$	$\frac{(\sigma_1 - \sigma_3)_{cy}}{2\bar{\sigma}_{3c}}$		2.5%	5%	10%
	%	pcf	pcf	pcf	kg/cm^2		kg/cm^2		kg/cm^2					
C$\bar{R}$-1	17.2	98.7	98.8	99.4	4.0	1.0	4.0	0.99	2.21	0.28	11	8	11	14
C$\bar{R}$-2	17.2	98.7	99.1	99.5	4.0	1.0	4.0	0.95	1.93	0.24	23	18	22	25
C$\bar{R}$-3	17.2	99.0	99.3	100.0	4.0	1.0	4.0	0.96	2.51	0.31	11	6	10	13
C$\bar{R}$-5	17.2	98.7	99.2	99.4	1.0	2.0	12.0	0.96	1.14	0.57	100	99 (2)	103 (2)	107 (2)
C$\bar{R}$-6	17.2	98.8	99.6	99.8	1.0	2.0	11.0	0.95	1.37	0.69	28	24	29	34
C$\bar{R}$-7	17.2	99.1	100.5	100.6	1.0	2.0	11.0	0.95	1.71	0.85	10	8	10	13
C$\bar{R}$-8	17.2	98.9	99.4	99.7	1.0	2.0	11.0	0.96	1.82	0.91	10	5	8	11

(1) Unit weight determined with the specimen under a vacuum of about 15 in. Hg.
(2) Number of cycles to reach a maximum compressive strain equal to 2.5, 5, and 10%, respectively.

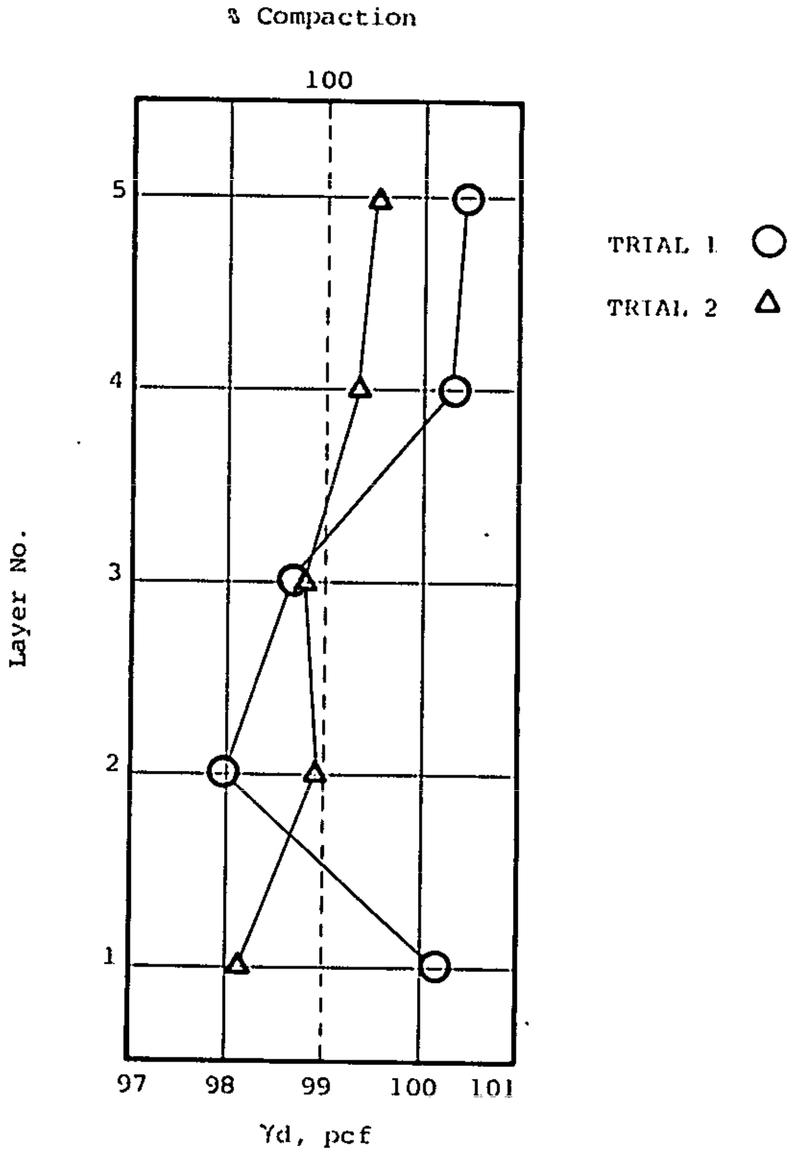
Geotechnical Engineers Inc.

Project 70484
October 13, 1978

Lab. 4-3 rev. 0 28 May 74

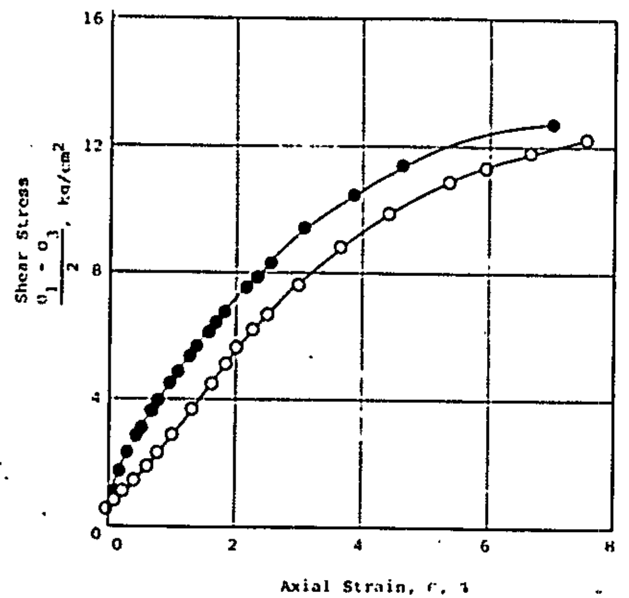


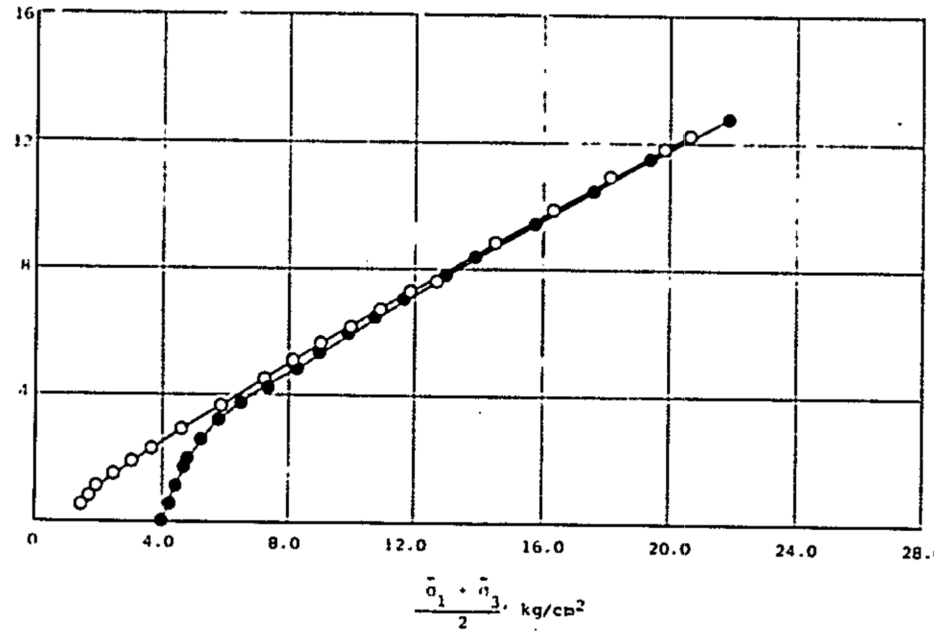
U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	GRAIN SIZE CURVE TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 10, 1978 Fig. 1

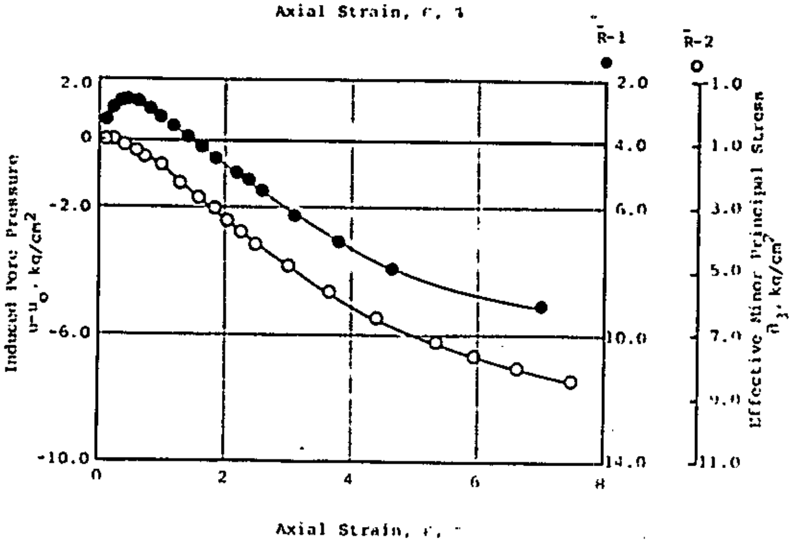


Trial No.	Target Dry Unit Weight pcf	Average Dry Unit Weight pcf	Maximum Variation pcf
1	99.0	99.5	2.4
2	99.0	98.9	1.4

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	UNIT WEIGHT DISTRIBUTION TRIAXIAL SPECIMENS TRENCH FT-1, DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 11, 1978 Fig. 2

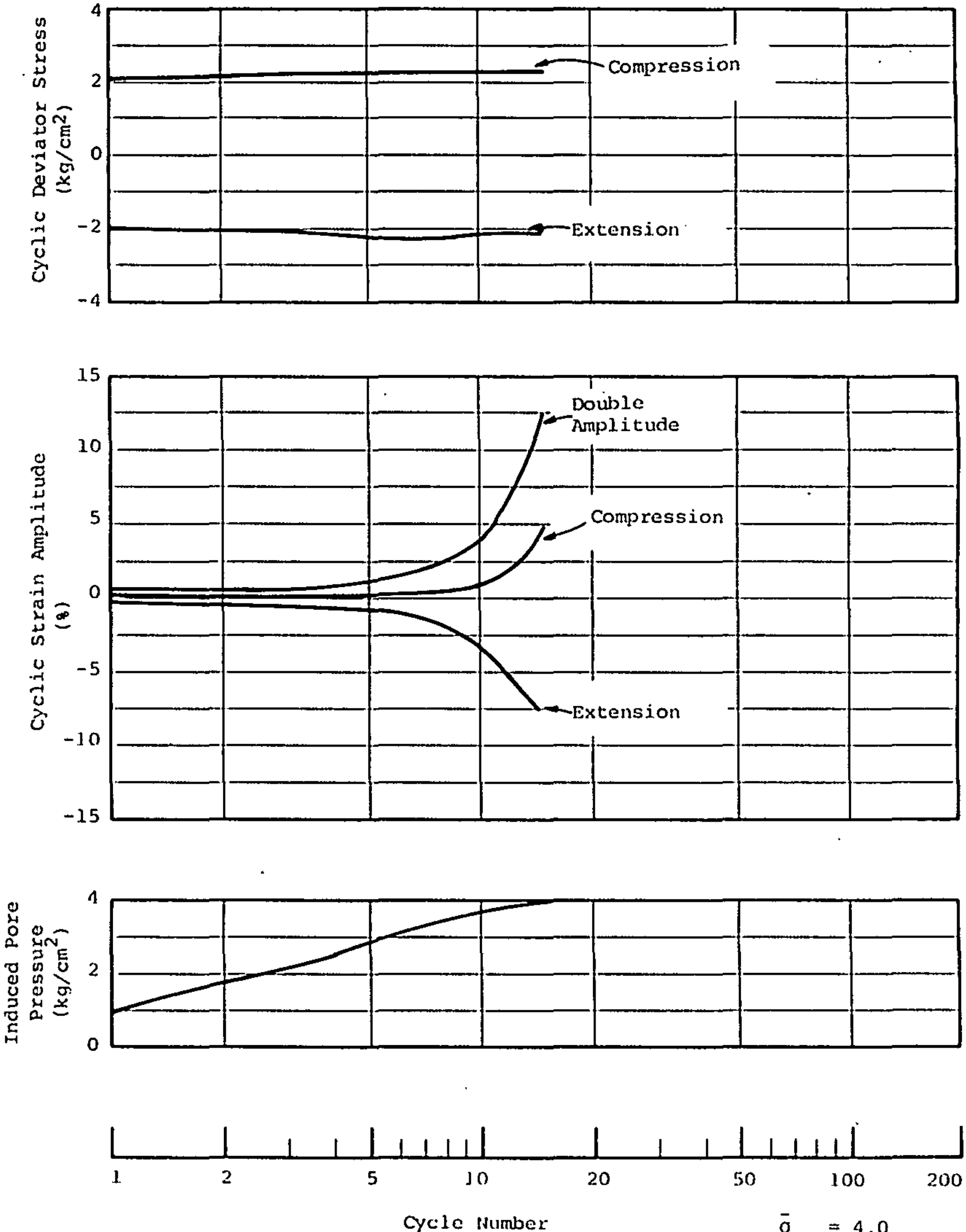






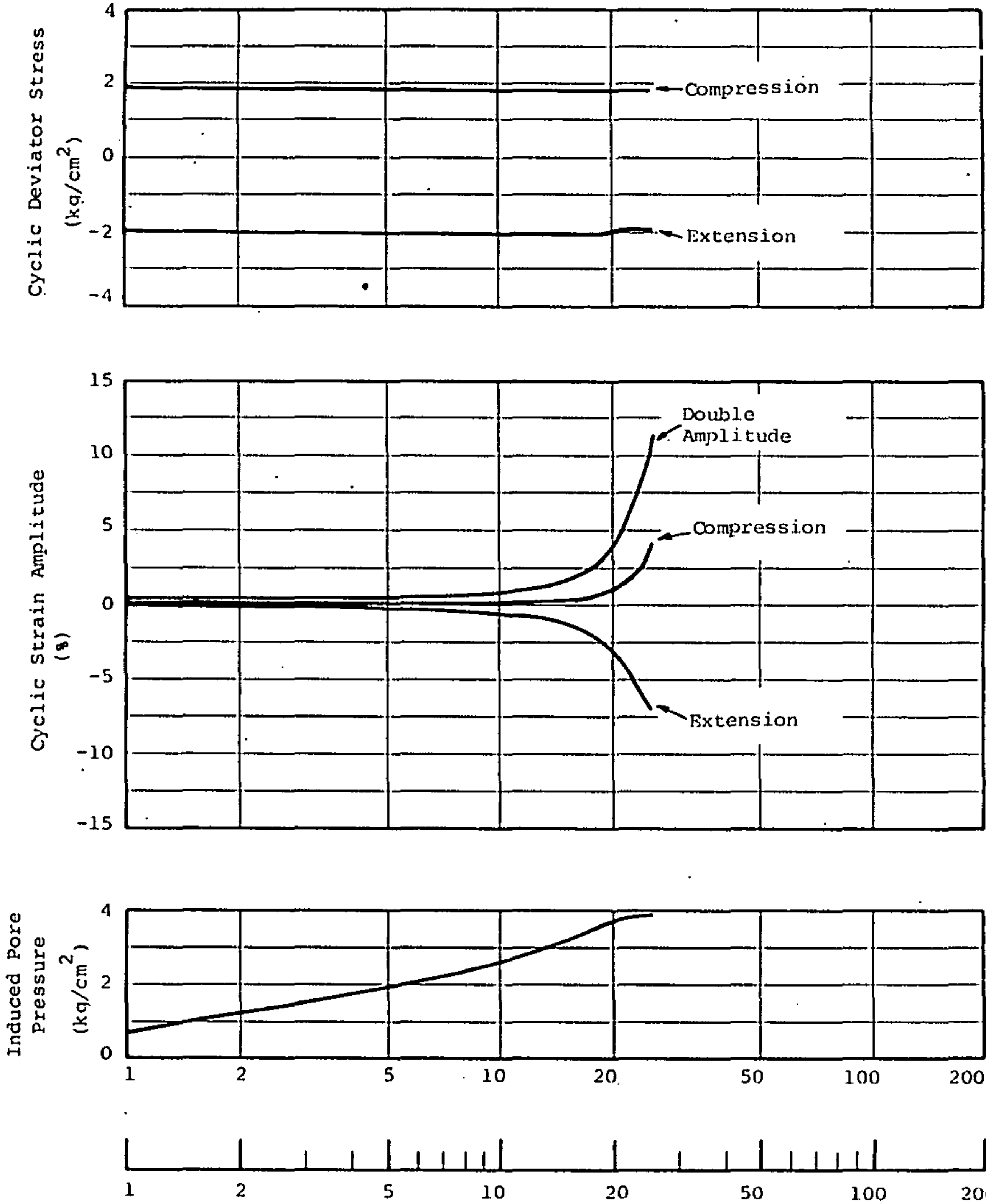
Symbol	Test No.	Initial Water Content	Dry Unit Weights			Effective Consolidation Pressure	Consolidation Stress Ratio	Peak Angle of Internal Friction
			In Mold	Triaxial Specimen				
				Initial	After Consolidation	$\bar{\sigma}_{3c}$	K_c	$\bar{\phi}$
		%	pcf	pcf	pcf	kg/cm²	$\bar{\sigma}_{1c}/\bar{\sigma}_{3c}$	degrees
●	R̄-1	17.2	98.9	99.4	100.0	4.0	1.0	37.0
○	R̄-2	17.2	99.2	99.9	100.1	1.0	2.0	37.5

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	R̄ TEST TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 17, 1978 Fig. 3



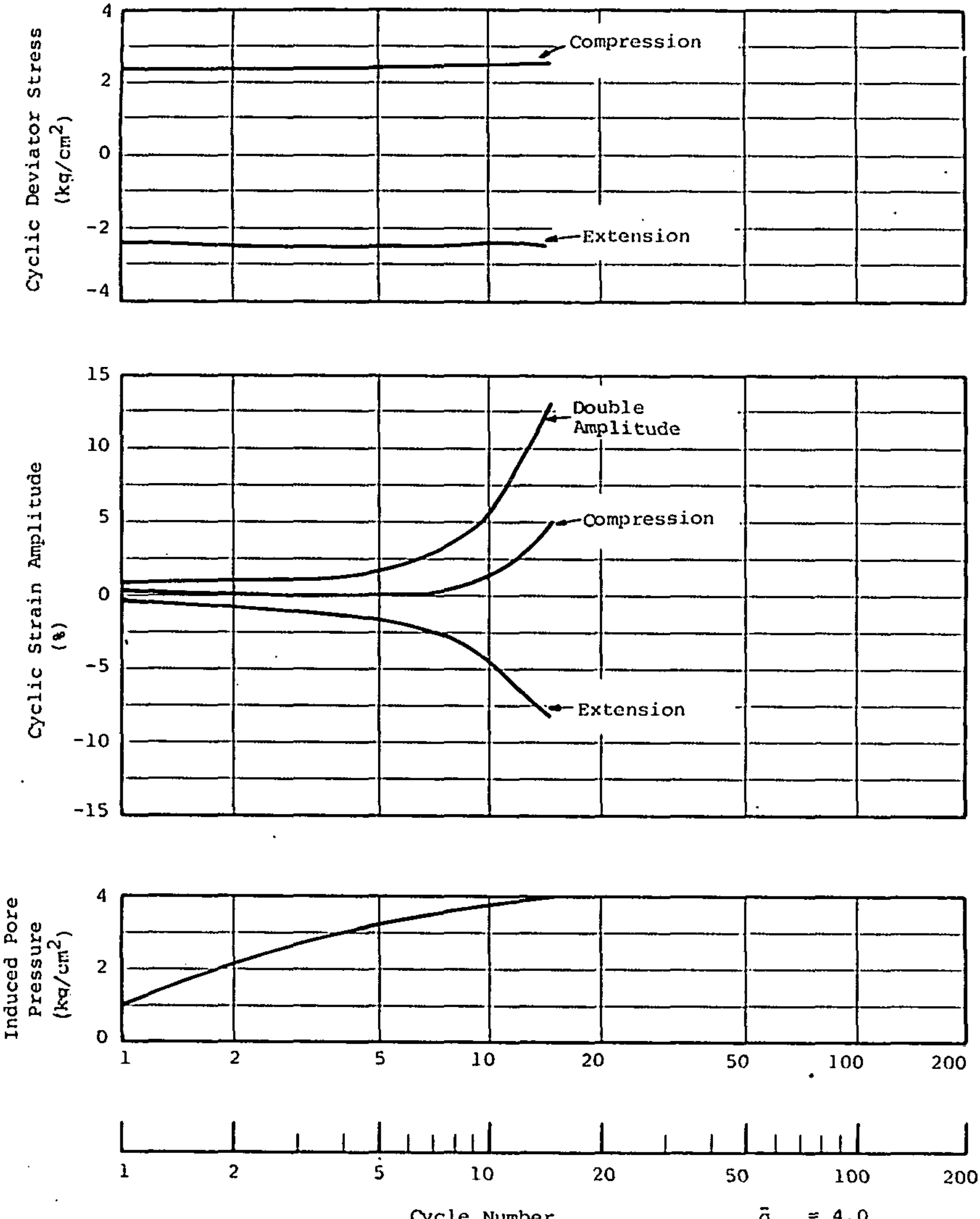
$\bar{\sigma}_{3c} = 4.0$
$K_c = 1.0$

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	CYCLIC LOAD TEST $\bar{\text{CR}}$- 1 TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 11, 1978 Fig. 4



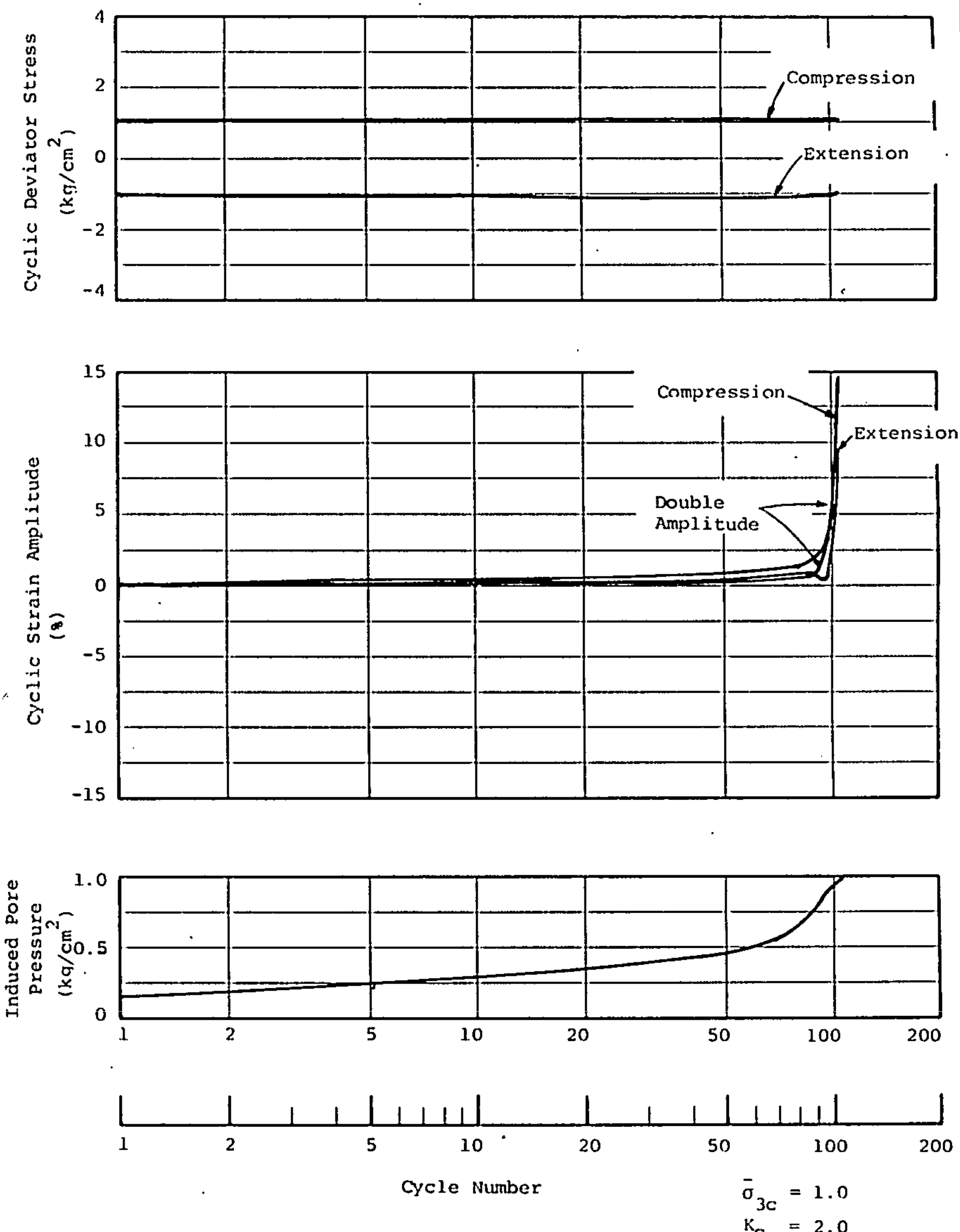
$\bar{\sigma}_{3c} = 4.0$
$K_c = 1.0$

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	CYCLIC LOAD TEST C$\bar{R}$-2 TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 11, 1978 Fig. 5

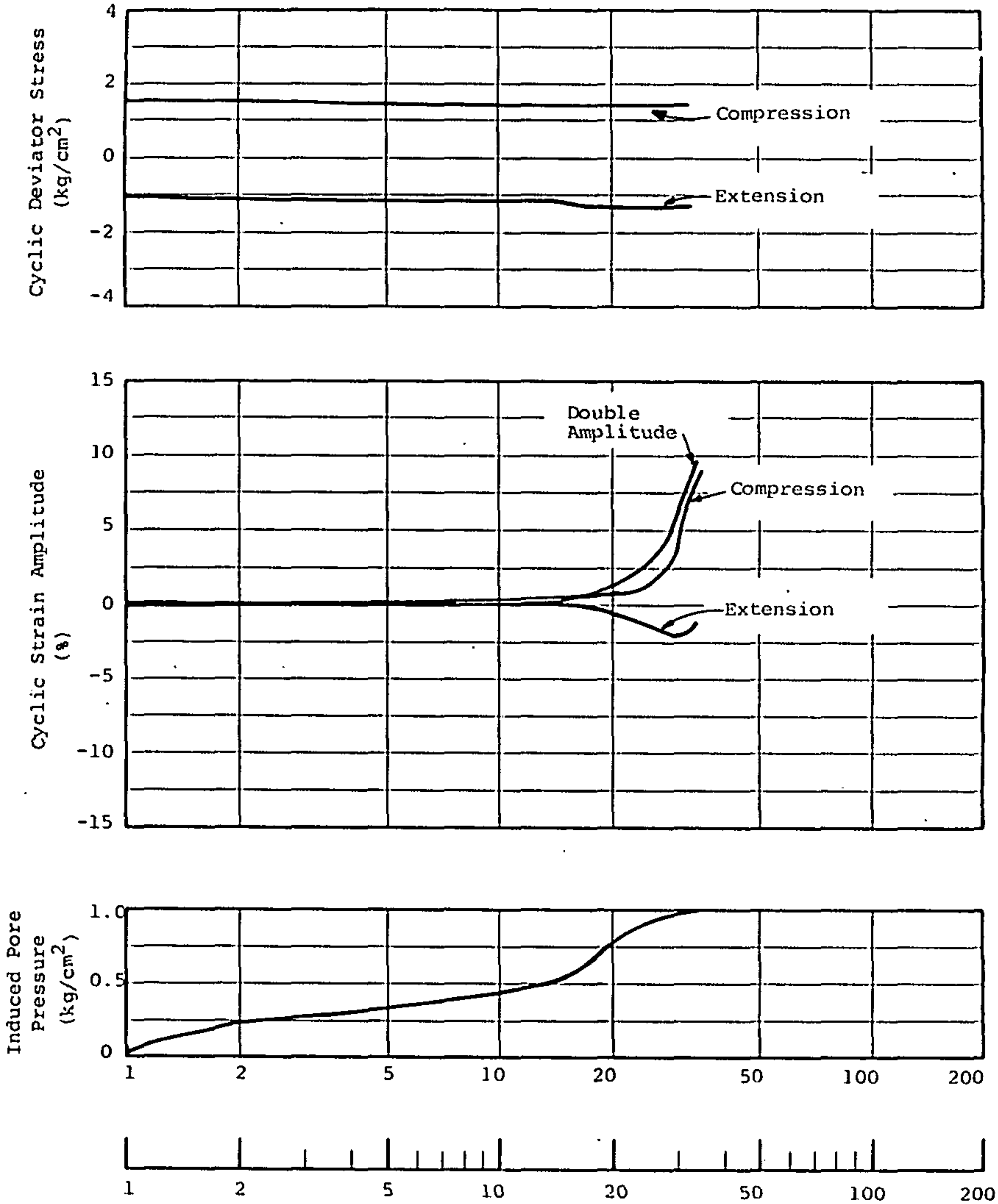


$\bar{\sigma}_{3c}$ = 4.0
K_c = 1.0

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	CYCLIC LOAD TEST CR-3 TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 11, 1978 Fig. 6



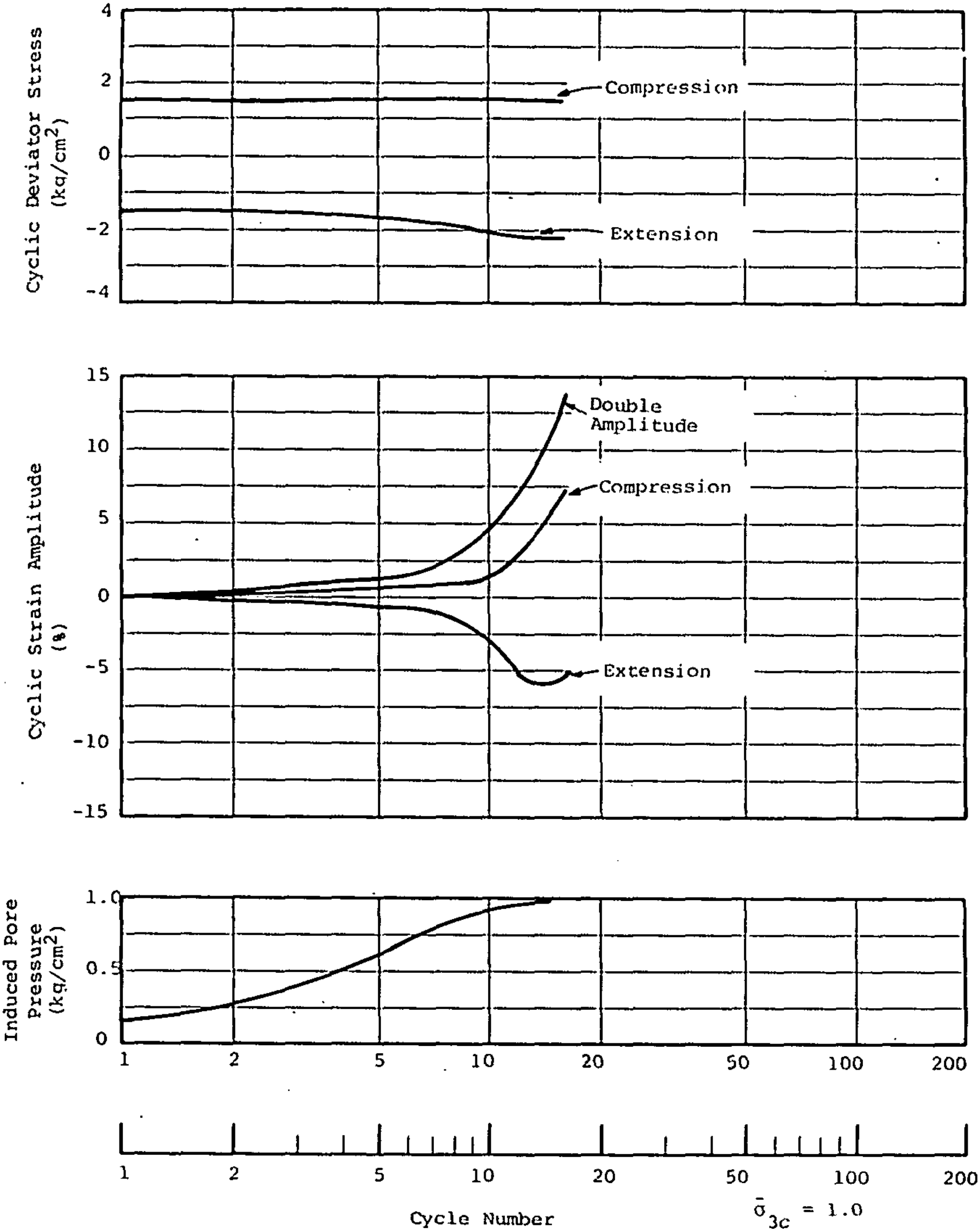
U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	CYCLIC LOAD TEST CR-5 TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 11, 1978 Fig. 7



$\bar{\sigma}_{3c} = 1.0$

$K_c = 2.0$

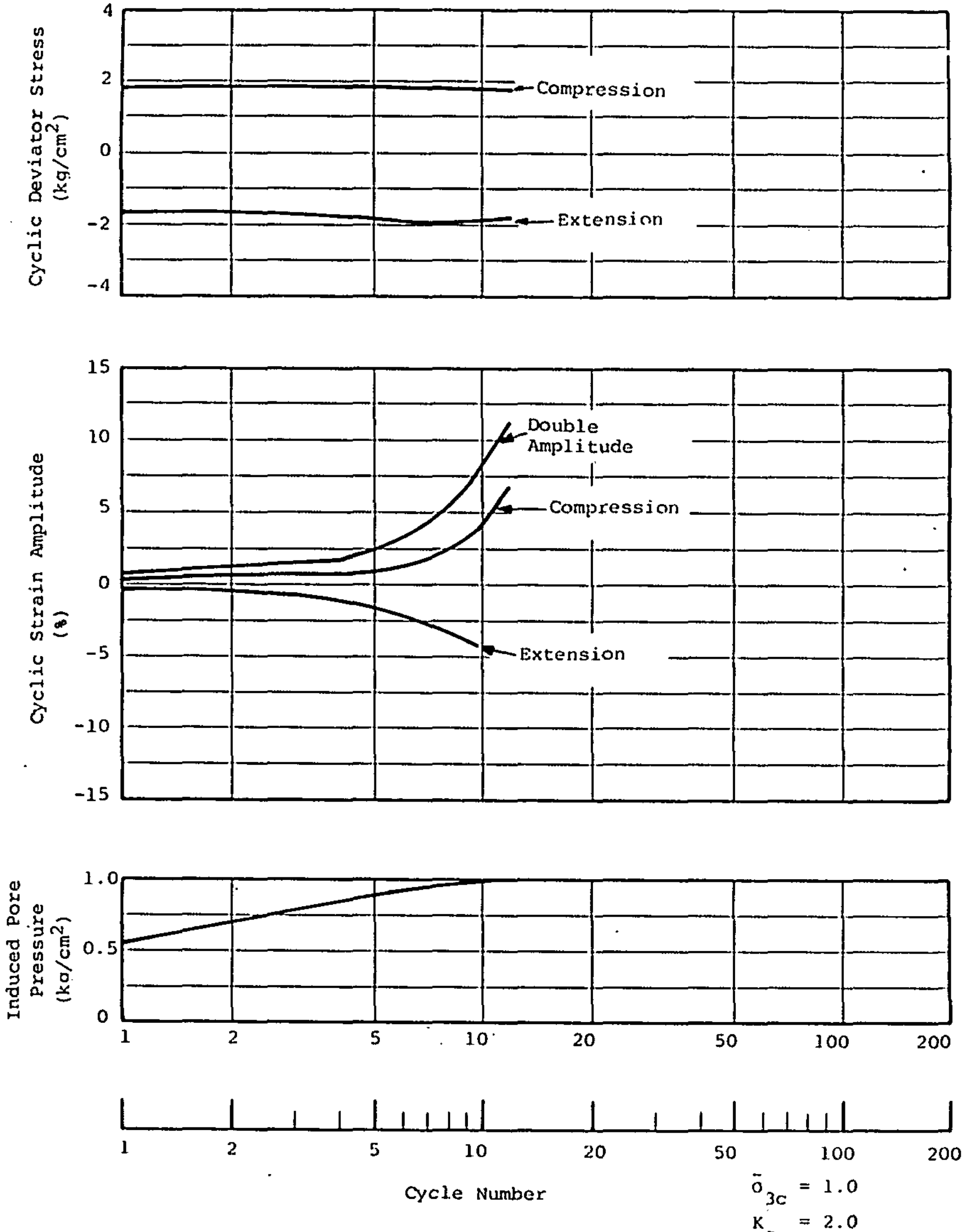
U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	CYCLIC LOAD TEST CR-6 TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 11, 1978 Fig. 8



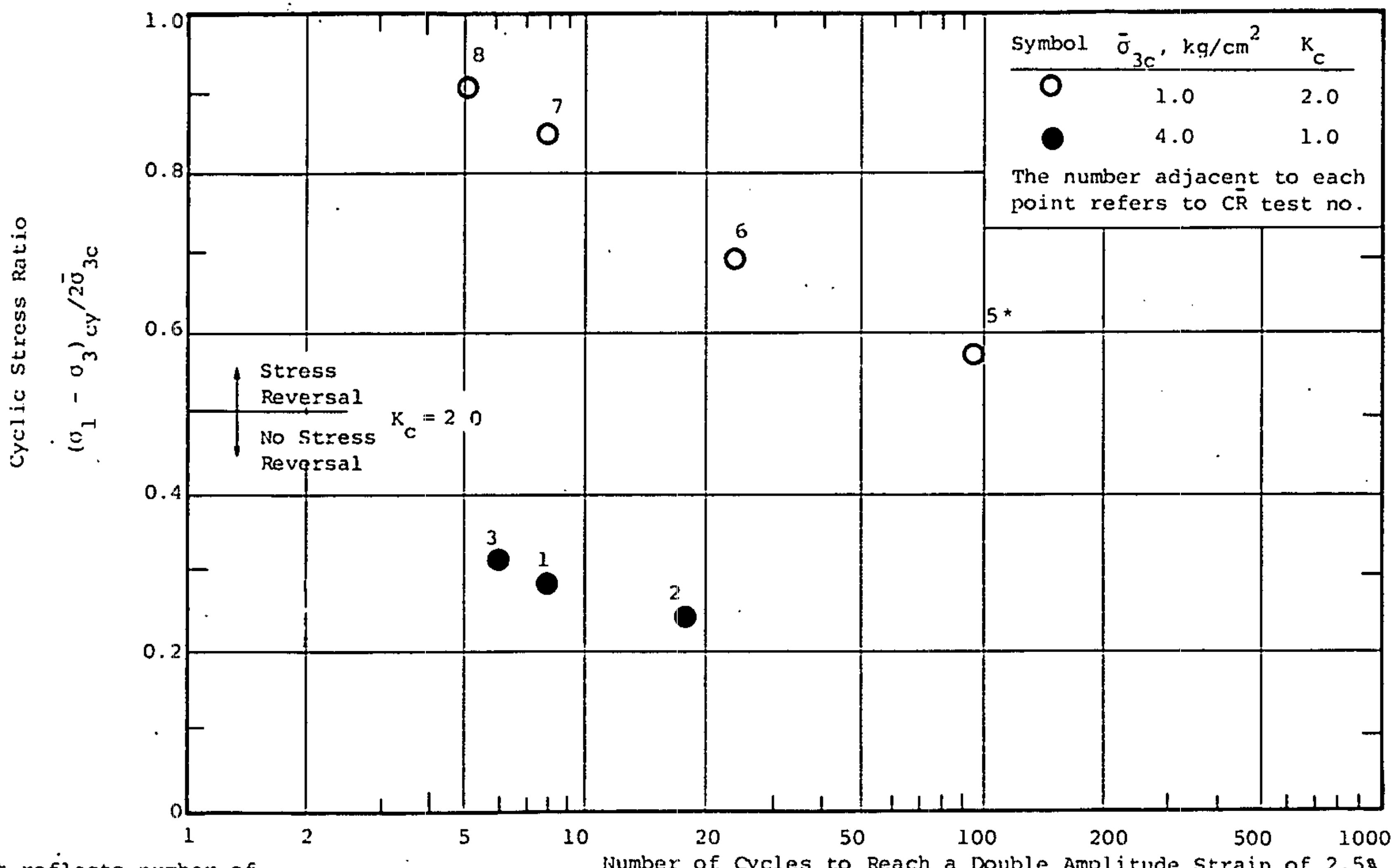
$\bar{\sigma}_{3c} = 1.0$

$K_c = 2.0$

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	CYCLIC LOAD TEST CR-7 TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 11, 1978 Fig. 9

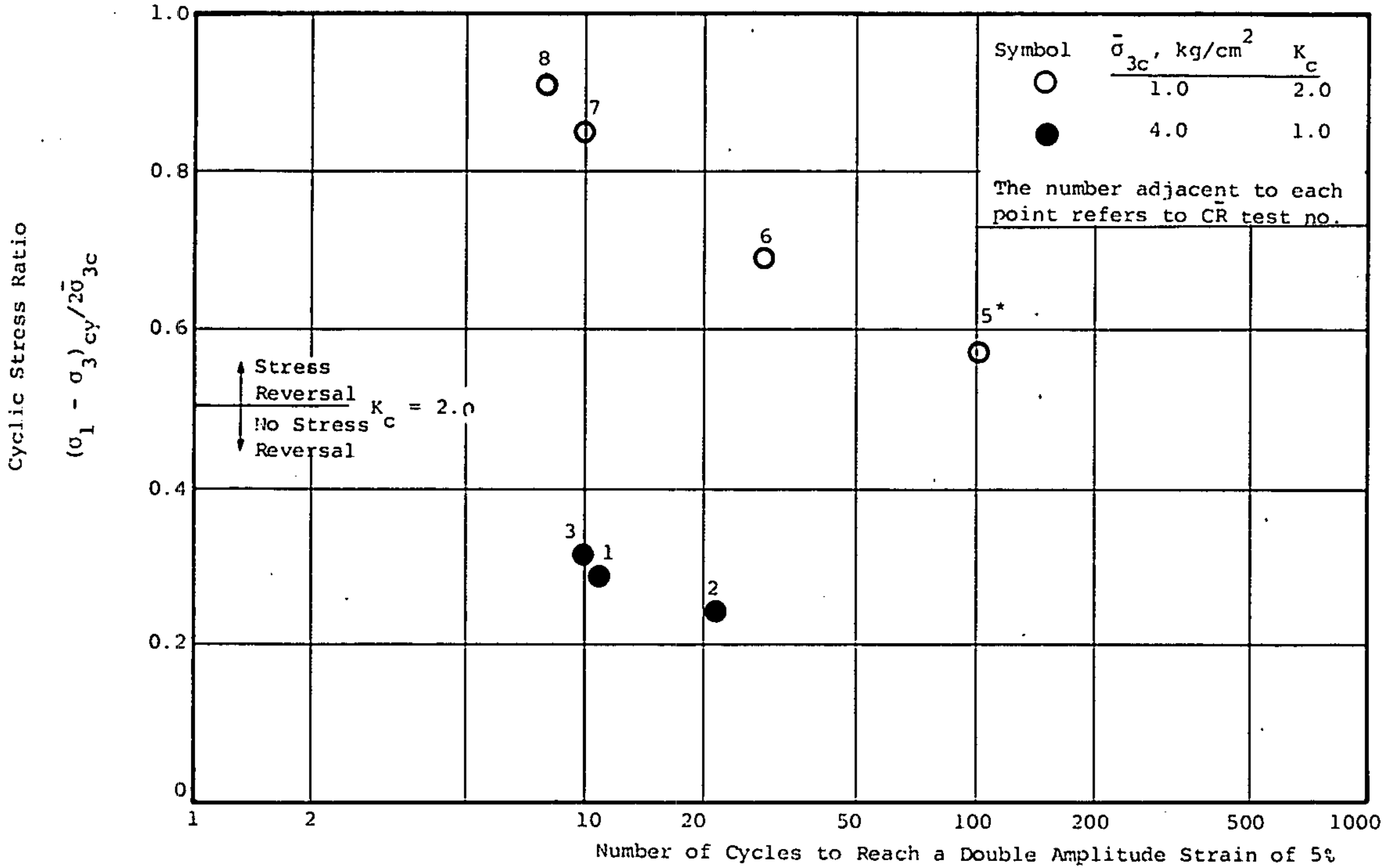


U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	CYCLIC LOAD TEST CR̄-8 TRENCH FT-1 DEPTH 8-10 ft
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 11, 1978 Fig. 10



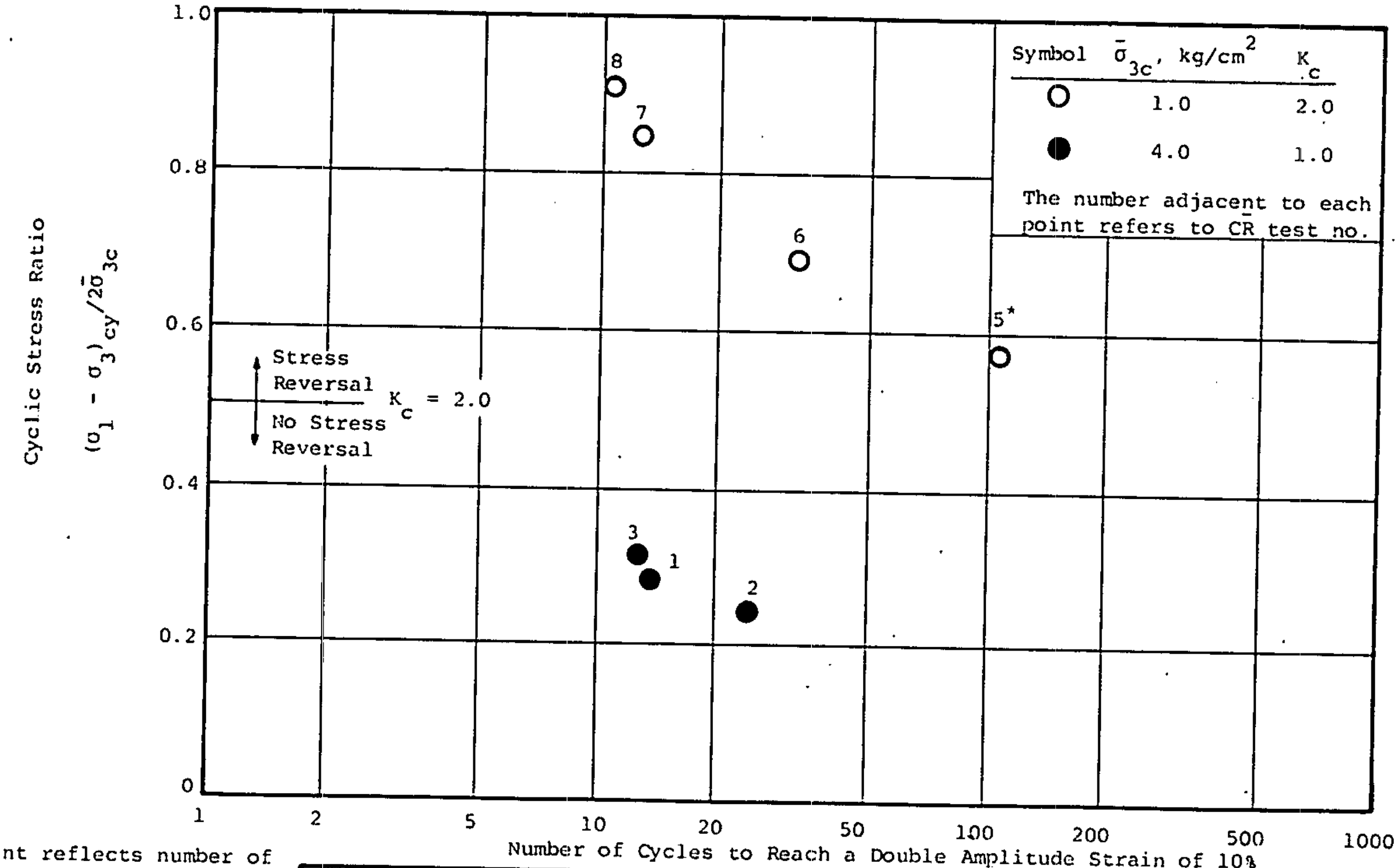
*Point reflects number of cycles to reach a maximum compressive strain of 2.5% which occurred before a double amplitude strain of 2.5%.

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	SUMMARY $\bar{CR}$ TESTS Double Amplitude Strain of 2.5%
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 17, 1978 Fig. 11



* Point reflects number of cycles to reach a maximum compressive strain of 5% which occurred before a double amplitude strain of 5%.

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	SUMMARY C$\bar{R}$ TESTS Double Amplitude Strain of 5%
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 17, 1978 Fig. 12



*Point reflects number of cycles to reach a maximum compressive strain of 10% which occurred before a double amplitude strain of 10%.

U.S. Corps of Engineers, NED Waltham, Massachusetts	Big River Dam	SUMMARY $\bar{CR}$ TESTS Double Amplitude Strain of 10%
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 78484	October 17, 1978 Fig. 13