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MEMO TO: G. F. Kemper  
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SUBJECT: Research Report No. 533, "Evaluation of Triaxial Testing Equipment and Methodologies of Three Agencies;" KYP-72-38; HPR-PL-1(15), Part III-B.

The analyses of the stabilities of slopes and of foundations under high embankments make use of shear strength data often obtained from triaxial tests. There are many potential sources of errors in the test methods, and questions can arise concerning the quality of triaxial test results obtained by geotechnical laboratories. The report attached describes a testing program in which triaxial tests were performed by the Division of Research and the Division of Materials of the Kentucky Department of Transportation and the Department of Civil Engineering at the University of Kentucky. The testing program can also provide a basis for evaluating the reliability of test procedures used by various consultants that may be doing triaxial testing for the Department. The report compares triaxial data from identical specimens tested by the three participating laboratories.

Respectfully submitted,

A handwritten signature in cursive script that reads "Jas. H. Havens".

Jas. H. Havens  
Director of Research

gd  
Attachment  
cc: Research Committee



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533

**EVALUATION OF TRIAXIAL TESTING EQUIPMENT  
AND METHODOLOGIES OF THREE AGENCIES**

**KYP-72-38; HPR-PL-1(15), Part III-B**

by

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December 1979



## INTRODUCTION

A reliable estimate of the shear strength of soil is essential in the design of earth structures such as retaining walls, building foundations, and such earthworks as highway and railroad embankments, dams, mining spoils, and cofferdams. The shear strength of soil is most often determined experimentally from laboratory tests. The most versatile and commonly used laboratory test for obtaining soil shear strength is the triaxial test. In this test, the soil specimen is confined laterally; the soil specimen can be sheared in various modes by varying the major and minor principal stresses.

Since 1960, the triaxial test has been used extensively for investigating the stability of earth structures; the test is performed "routinely" in many geotechnical laboratories. Although commonly used, it is not necessarily a 'routine' test. Careful attention and judgement must be given to testing procedures and equipment make-up. There are many sources of potential errors, especially when pore pressures are monitored; and a large degree of skill and training is required. Experience is required to insure that the results are reasonable. Different attitudes and equipment make-up can be found at different geotechnical laboratories. Consequently, a question inevitably arises concerning the quality of test results obtained by different laboratories.

The primary purpose of this report is to initiate and help establish a triaxial testing forum whereby any geotechnical laboratory engaged in triaxial testing can check the quality of their triaxial results, and therefore, evaluate their testing procedures and equipment make-up against the results obtained by other agencies. The establishment of such a forum will help in recognizing the causes of different results (if any). The forum will provide the Kentucky Department of Transportation (KYDOT) and, perhaps, other governmental agencies with the means to accredit any geotechnical laboratory which may perform triaxial tests for that agency.

To initiate the forum, the geotechnical laboratory of the Division of Research of KYDOT invited the geotechnical laboratories of the Department of Civil Engineering (CE) of the University of Kentucky

and the Division of Materials of KYDOT to participate in a triaxial testing study. Each of those agencies was asked to perform isotropically consolidated, undrained triaxial tests (with pore pressure measurements) on "standardized" soil specimens supplied by the Division of Research and to report their results (including all data) to the Division of Research. Specimens supplied to each laboratory consisted of remolded kaolinite. An effort was made to make the specimens as uniform as practical. The kaolinite was remolded in such a manner that the specimens had approximately the same water content, void ratio, and degree of saturation. Each of the agencies was selected because of its experience in triaxial testing. The three agencies have been involved over the past years in numerous research and public works projects requiring triaxial test results. A comparison of the results and analyses of the data by the Division of Research are reported herein.

## MATERIAL PROPERTIES

The kaolinite used in this study was purchased from the Edgar Plastic Kaolin Company, Edgar, Florida. Particle-size analysis was performed according to AASHTO T 88-51. All of the material was finer than 70  $\mu\text{m}$  and 62 percent was finer than 2  $\mu\text{m}$ . The particle-size distribution is shown in Figure 1. AASHTO standard tests T 89-60 and T 90-61 were used to determine the liquid limit and the plastic limit, respectively. The liquid limit was 55 percent and the plastic limit was 31 percent. The specific gravity was 2.68 (AASHTO T 100-60). The moisture-density relationship (from AASHTO T 99-61) is shown in Figure 2. The maximum dry density was 81.3 pounds per cubic foot (1,305  $\text{kg}/\text{m}^3$ ), and the optimum moisture content was 34.5 percent. According to the AASHTO Soil Classification System, the material was an A-7-5(36) soil. It was described as an MH material under the Unified Soil Classification System.

A sample of the material was submitted to the Agricultural Science Center of the University of Kentucky for x-ray diffraction analysis. The sample was 94 percent kaolinite, four percent quartz, and two percent talc.

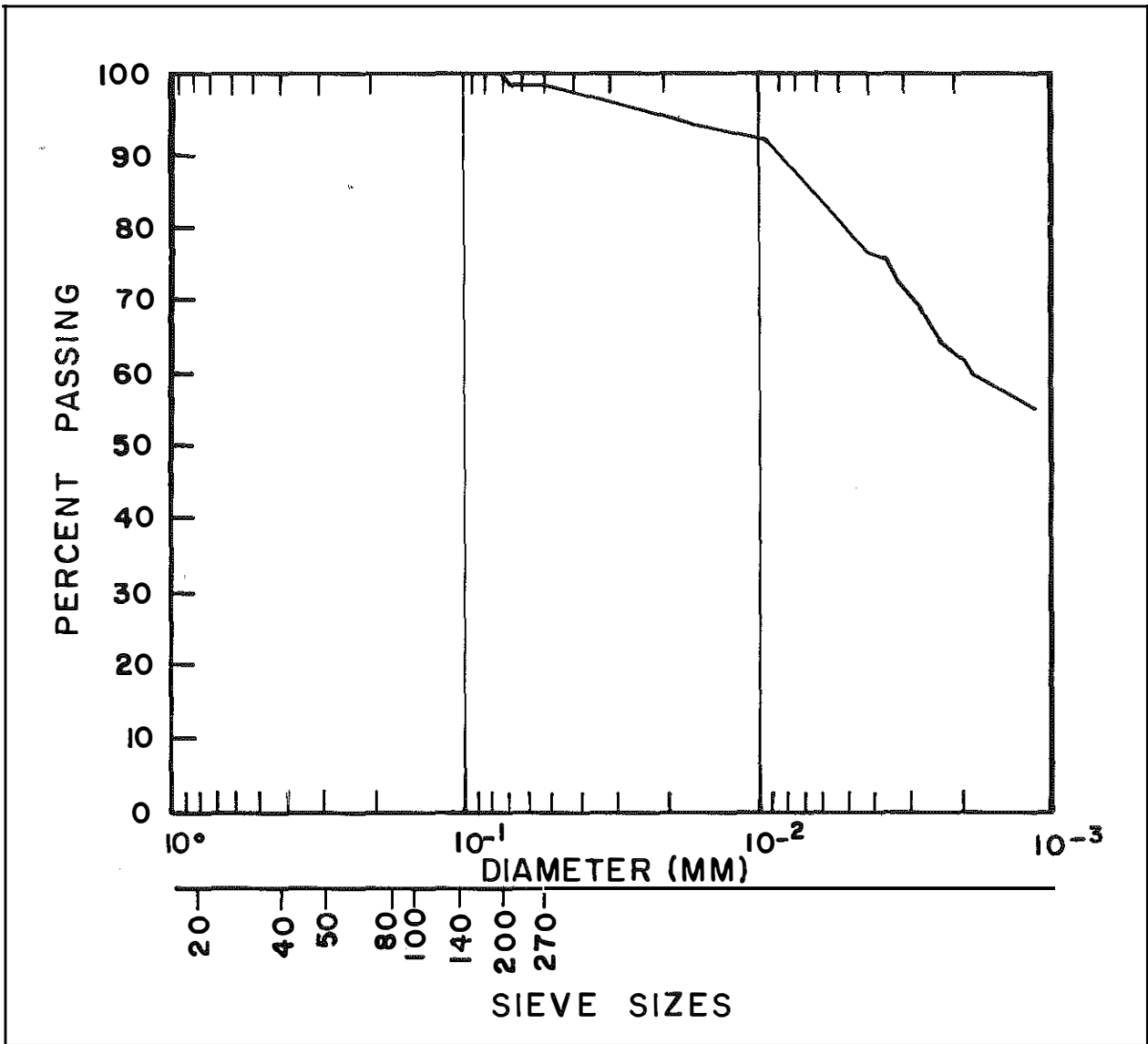


Figure 1. Particle-Size Distributions of the Kaolinite Used to Form the Triaxial Test Specimens.



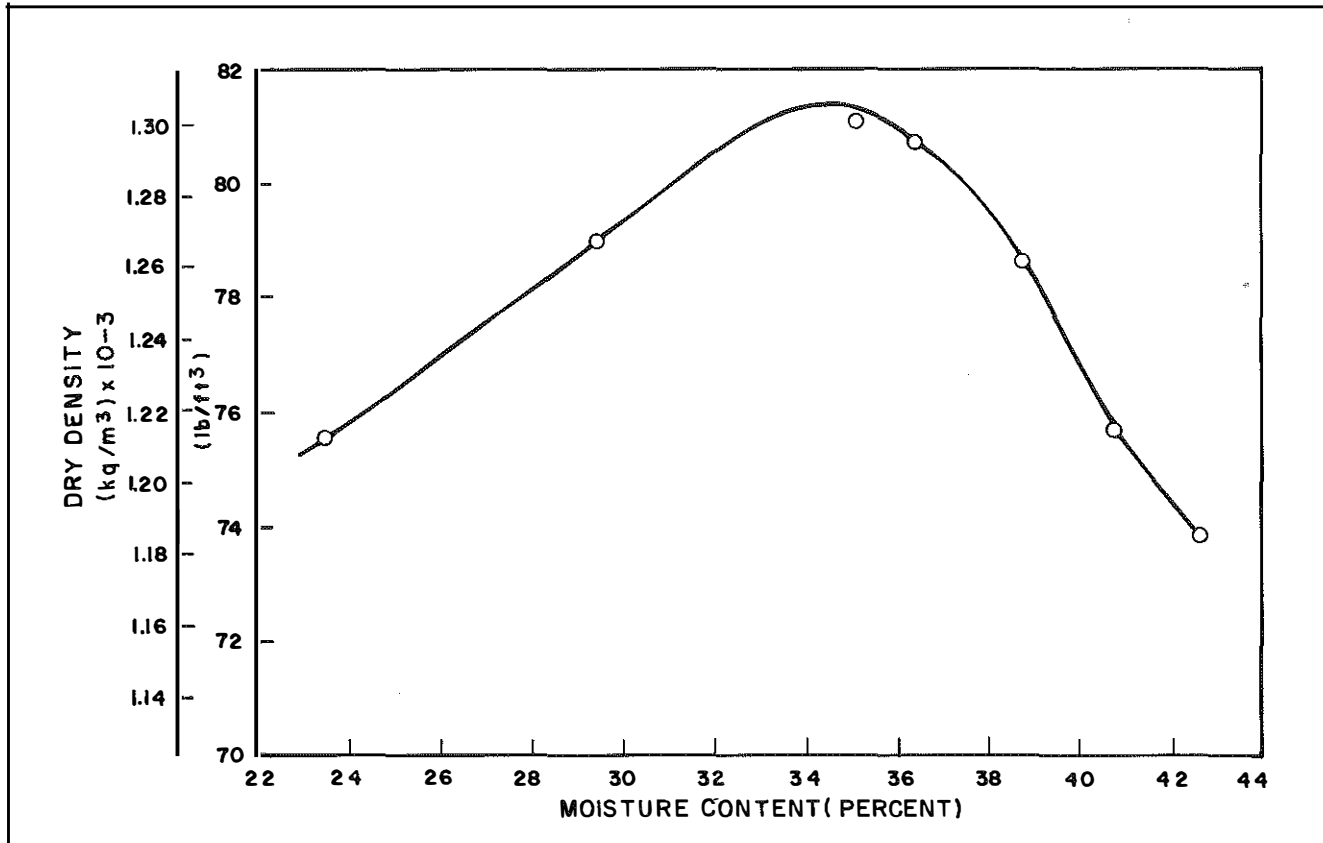


Figure 2. Moisture-Density Relationship of the Kaolinite Used to Form the Triaxial Test Specimens.

### SAMPLE PREPARATION

The kaolinite was received in dry powdered form. The desired amount of water (43 percent) was thoroughly mixed into the material and then covered with a plastic sheet and allowed to "cure" over night. This insured a uniform distribution of moisture. The percentage of moisture was selected that would, hopefully, provide 100-percent saturation. However, test specimens averaged only 96.4 percent of saturation.

To obtain triaxial results which could be compared, a large number of triaxial specimens having duplicate clay structures as practical were needed.

Additionally, void ratio, particle orientation, and pore water composition should be nearly identical. To insure uniformity, a "Vac-Aire" extrusion machine, Figure 3, capable of extruding clay up to 3 inches (76 mm) in diameter was used. The soil was mixed in a vacuum hopper on the machine and forced by augers through a die of desired size and shape. Mixing in a vacuum produced a high degree of uniformity and saturation. Cylindrical specimens approximately 2.4 inches (61 mm) in diameter and 5.0 inches (127 mm) long were extruded for this study. The specimens were immersed immediately in melted wax for protection; the waxed specimens were stored several weeks until testing.

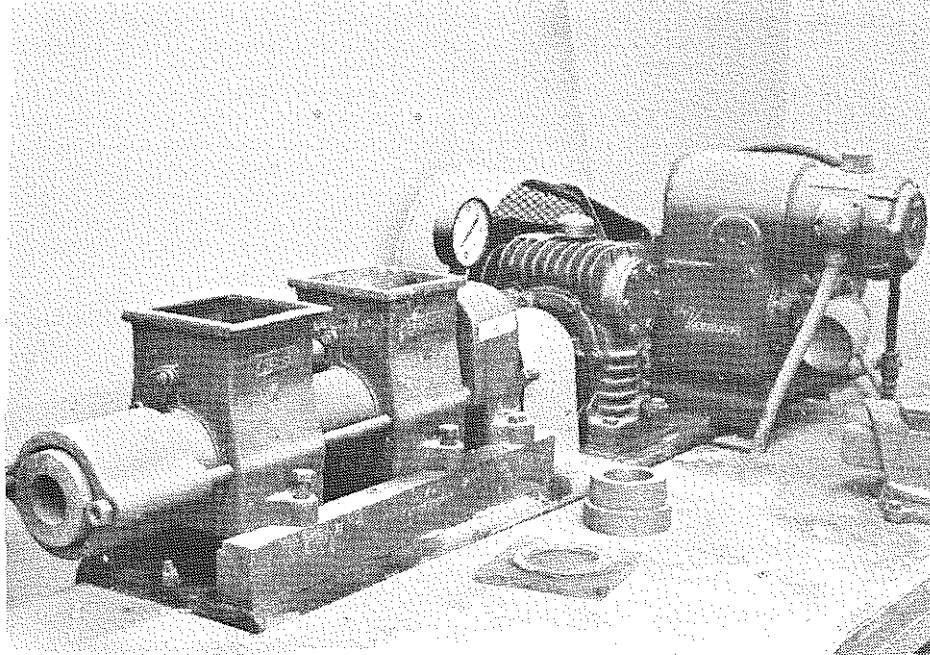


Figure 3. View of "Vac-Aire" Extrusion Machine Used to Mix and Extrude Uniform Kaolinite Specimens.

## EQUIPMENT

A comparison of some general features of the triaxial equipment used to perform triaxial tests on the "standardized" kaolinite specimens is shown in the left portion of Table 1 and described below in more detail.

### Division of Research

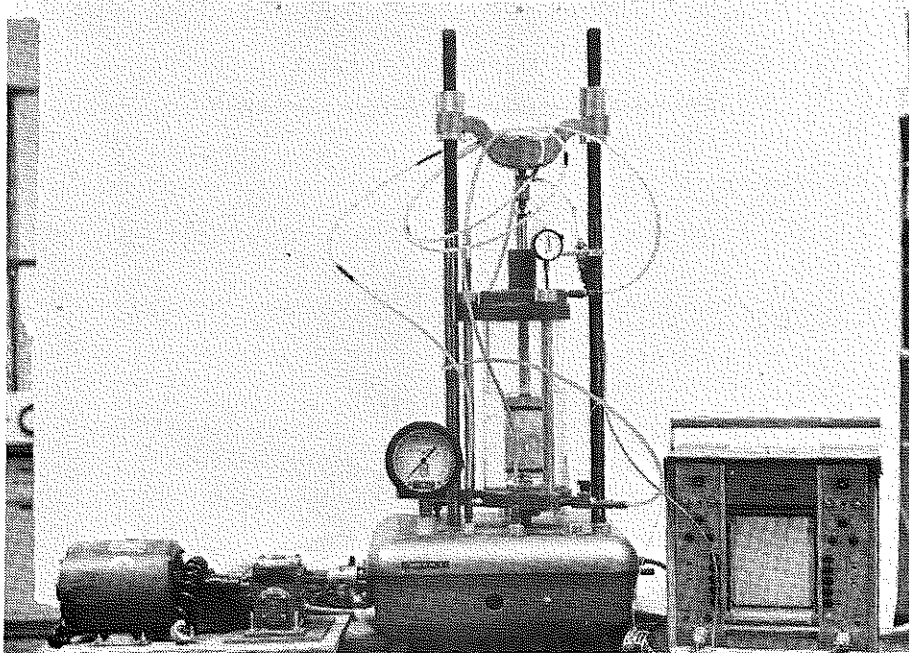
The Division of Research used a Model 530, Karol-Warner, loading frame (5,000-pound or 2,273 kg capacity) and chamber. The pressure chambers were equipped with 2.4-inch (61-mm) pedestals and headers. The bottom pedestal was provided with a relatively fine-grained porous stone 2.3 inches (58 mm) in diameter by 0.1 inch (2.5 mm) thick fitted snugly into an indentation. Two drainage lines -- 1/16-inch (1.6-mm) O.D. nylon tubing -- lead from outside the chamber to the bottom pedestal. Thus, water could be circulated through the lines and porous stone to purge the system of air. The drainage lines were continuous: that is, there were no fittings between the outside of the chamber and the porous stone as they might be difficult to de-air completely.

One of the drainage lines was provided with a no-volume-change valve outside the chamber which could be connected to a burette and back-pressure system. The other line could be connected to a pore-pressure transducer. Water was used as the confining fluid, and both the chamber pressure and back

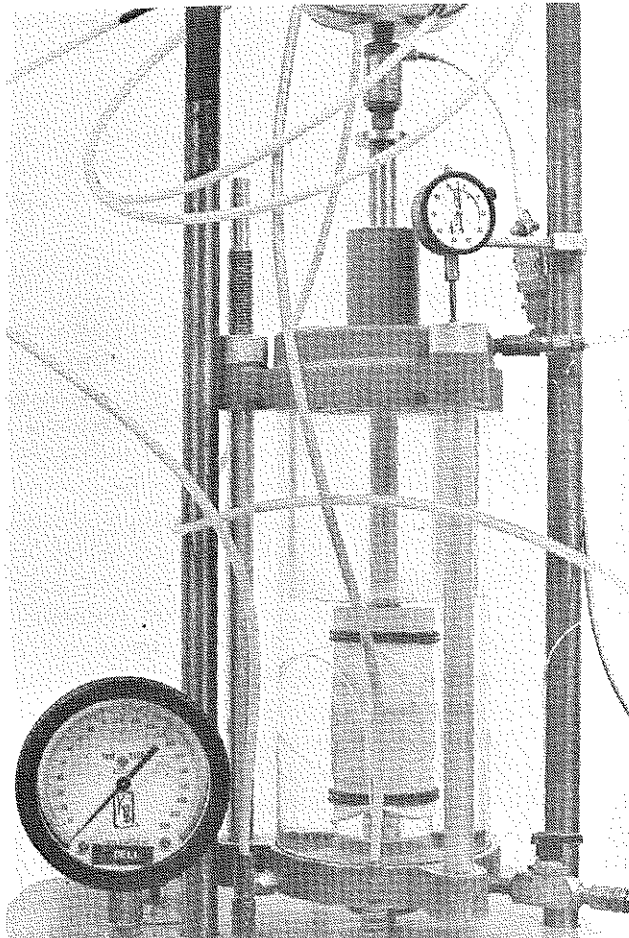
pressure were controlled by precision air-pressure regulators. The confining pressure was applied to the upper part of the chamber -- above the specimen -- which was not filled with water. A continuous supply of dry, filtered air was required. The supply pressure was not permitted to drop below 120 psi (827 kPa) nor to rise above 150 psi (1,034 kPa).

The load was applied to the top cap through a 3/4-inch (19-mm) steel piston guided through the top of the chamber by two Thompson ball bushings and a rubber "quad-ring" pressure seal. A variable-speed drive forced the chamber containing the specimen up against the piston, which was fixed to the top of the loading frame. The rate of deformation could be controlled to as low as 0.0002 inches per minute (5.1  $\mu\text{m}$  per minute) through the use of a 100:1 reduction gear box. The load was measured with strain-gage load cells mounted outside the chambers. Pore pressures were measured with strain-gage pressure transducers. Deformation was usually indicated on dial extensometers (having a resolution of 0.001 inch or 0.0254 mm); linear variable differential transformers could be used if it was desired to record data (1).

Test loads and pore pressures were monitored on either a Sanborn Model 321 dual channel oscillograph or a Brush Recorder, Mark II, with a dual strain-gage amplifier. This equipment is shown in Figures 4 and 5. Figure 6 is a schematic diagram of this equipment.



**Figure 4.** General View of Triaxial Equipment Used by the Division of Research.



**Figure 5.** Close-up View of Triaxial Chamber and Specimen, Division of Research.

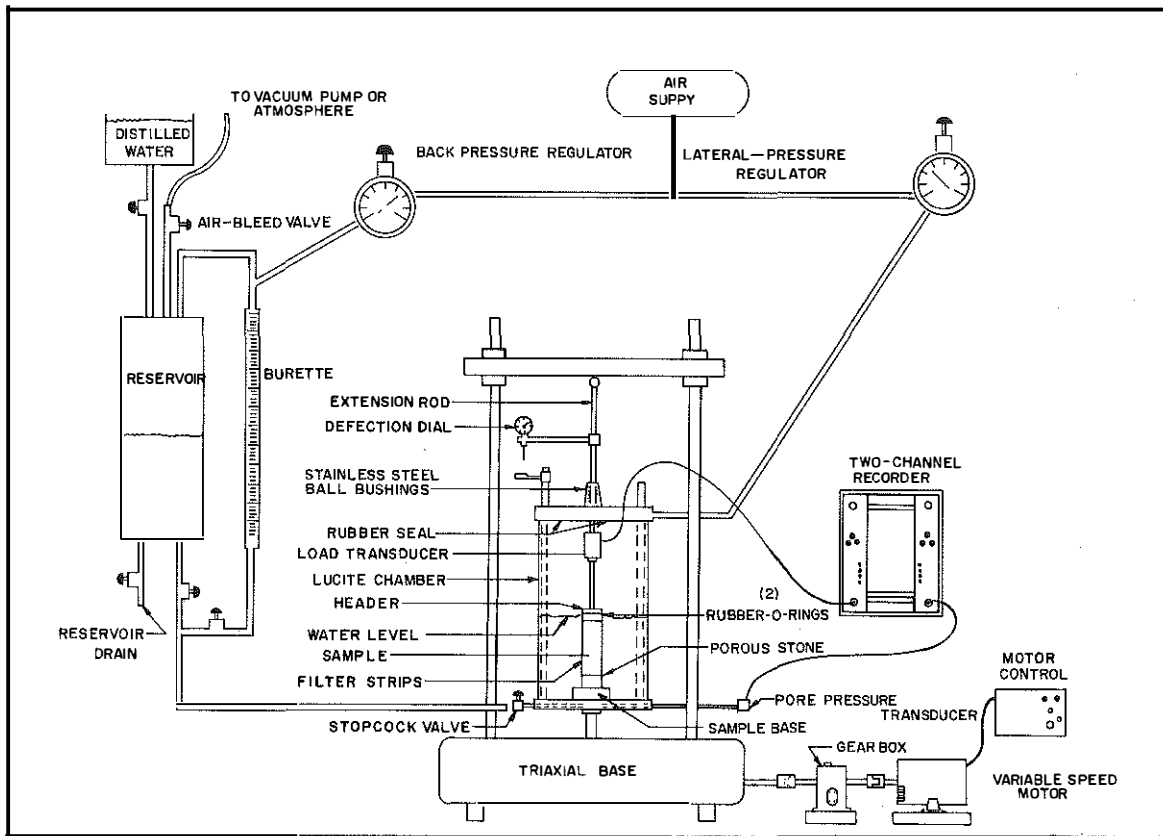


Figure 6. Schematic Diagram of Triaxial Equipment Used by the Division of Research.

#### Division of Materials

The Division of Materials uses a Model 500, Karol-Warner triaxial loading frame and chamber. This is similar to the Model 530 except the load capacity is 1,000 pounds (455 kg). The top cap, base plate, and pore pressure lines inside the chamber are similar to those used by the Division of Research. The burette system, Model KWPS-1, was manufactured by Karol-Warner. This device has three lucite pressure tubes. Two of these tubes are manifolded; the larger tube is intended for pressure saturation prior to testing; the smaller tube is used for sample volume-change measurements during the test. The third tube is for chamber volume-change measurements during the test. Water was used as the confining fluid in the chamber and precision air regulators were used to control back pressure and cell pressure.

The samples were strained using a variable-speed drive in tandem with a 100:1 ratio reduction gear box. The test load was monitored with either a 500-pound (227-kg) or 1,000-pound (455-kg) capacity, cast-aluminum load ring. Pore pressures were monitored with a Karol-Warner, Model KW53, pore-pressure device (null-indicator type). Strains were monitored by

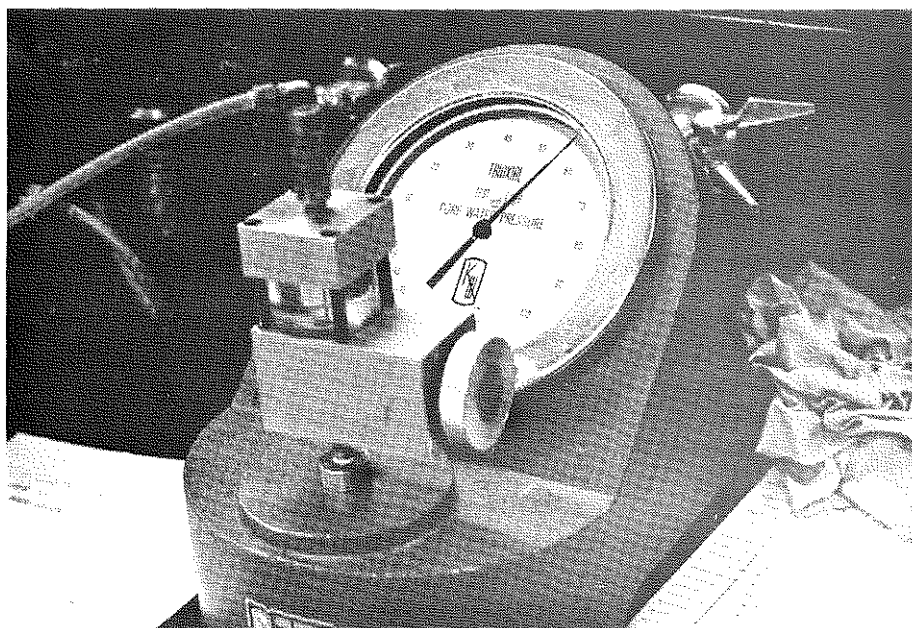
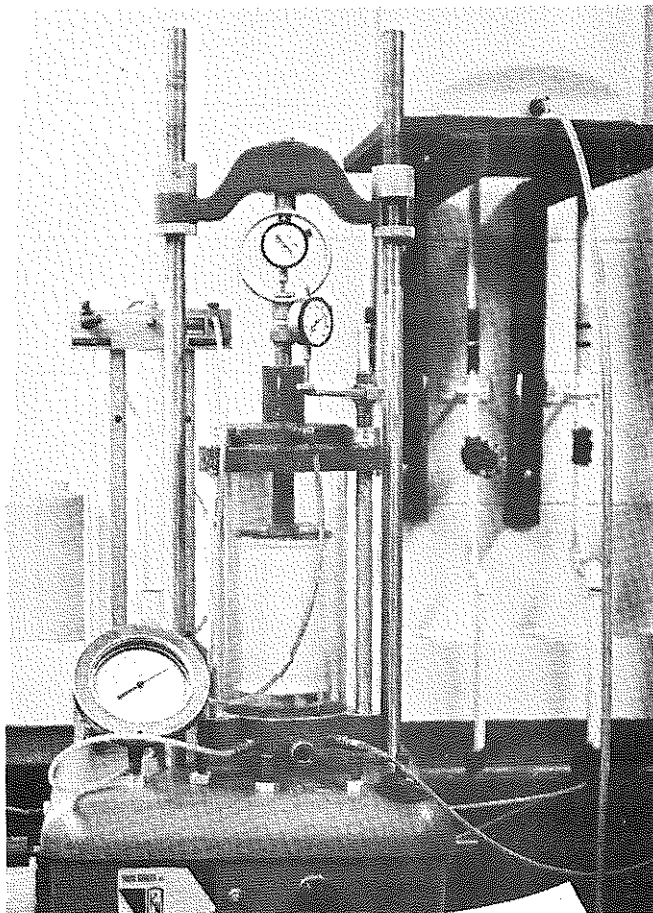
dial extensometers having a resolution of 0.001 inch (25.4  $\mu\text{m}$ ). The equipment and the pore pressure device are shown in Figures 7 and 8, respectively.

#### Civil Engineering Department (UK)

The basic equipment used by the CE Department was manufactured by Geonor of Oslo, Norway. The load was applied to the specimen by a variable-speed drive and a "transmission" box. The chamber was forced up against the head fixed to the top of the loading frame. Load and pore pressure were monitored by strain-gage transducers. Strains were monitored by linear variable differential transformers or dial extensometers. The confining fluid was water, and the back pressure and chamber pressure were controlled by precision air regulators. Changes in specimen volume were measured by a burette system mounted on the control board (see Figure 9).

Unlike the equipment used by Materials and Research, this equipment had only one drainage line at the base of the sample through which the system was de-aired. The pore pressure transducer was mounted at the base of the specimen, eliminating the need for a second drainage line. Figure 9 shows a complete triaxial set-up.

**Figure 7. General View of Triaxial Equipment Used by the Division of Materials.**



**Figure 8. View Showing Pore-Pressure Measuring Device, Null Indicator Type, Used by the Division of Materials.**

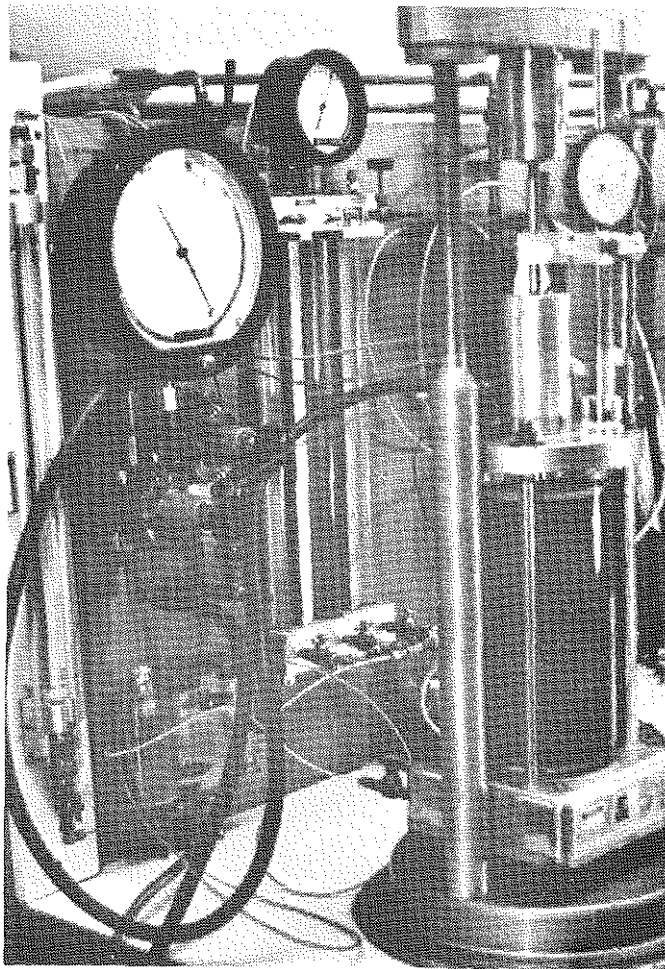


Figure 9. View of Triaxial Equipment Used by the CE Department.

### METHODOLOGY

Procedures used by the Division of Research are described below. Although the same basic testing method was used in the Division of Materials and the Civil Engineering Department, some significant differences exist among the three agencies because of differences in equipment and testing philosophy. However, only the procedures used by the Division of Research are reported in detail below.

The protective wax coating was removed, and the specimen was placed in an end-trimming mold and trimmed so that the ends were perpendicular to the longitudinal axis. Specimens were trimmed to be approximately 4.75 inches (121 mm) long.

Several measurements of the diameter and height of each specimen were made, and the weight of the test specimen was determined. The specimen was placed in the triaxial chamber on the pedestal which had been prepared as follows: (1) a saturated porous stone was placed in the indentation of the pedestal, (2) strips of filter paper were placed over the

porous stone; and (3) a polished plexiglass or teflon disk, slightly larger in diameter than the specimen, was placed over the porous stone on the pedestal. The disk was coated with a thin film of silicone grease to reduce end friction between the specimen and the end cap and thus to allow more uniform deformation. The specimen was placed on the coated disk, and the strips of filter paper folded up along the sides of the specimen to the top of the specimen to provide drainage paths around the polished disk. Hence, only radial drainage was used in the tests on the kaolinite specimens.

Using a vacuum membrane-expander, a leak-proof membrane was placed over the specimen. Two rubber O-rings were placed around the membrane at the pedestal to provide a seal. A coated, polished disk was placed on the specimen, and then a header or top cap was placed over this. Two O-rings were then placed around the membrane at the header to provide a seal at the top of the specimen.

The hollow cylinder was placed on the base, and the top cover was secured by three vertical rods and nuts. Large O-rings were used to form a pressure seal between the cylindrical chamber and the base and the top cover. The loading piston was then lowered until it entered the chamber, but it was not allowed to contact the indentation in the top end cap.

The chamber containing the specimen was placed in the loading machine and filled with water to an elevation approximately 1 inch (25 mm) above the O-rings around the header cap (see Figure 6). To de-air the drainage lines prior to starting the test, a vacuum was applied to one drainage line while the end of the other line was submerged in a beaker of water. This procedure removed air that may have been trapped between the sample and membrane and drew water from the beaker into the drainage lines. The vacuum line was then disconnected and a burette attached to the line. Water was allowed to flow back from the burette through the drainage lines until it was apparent that all air bubbles had been removed. A small pressure (approximately 2 psi (13.8 kPa)) was applied to the test chamber during this process to prevent water from entering the space between the sample and membrane. The porepressure line was then connected to the measuring device while water was running to prevent trapping air in the system.

Confining pressure was applied to the specimen by means of the regulator. Concurrently, a back pressure was applied to the top of the burette by means of another regulator. The consolidation pressure achieved was the difference between the two pressures. A back pressure was applied overnight to insure that the specimen was saturated.

After allowing the specimen to consolidate overnight, the valve between the specimen and back-pressure system was closed; and a check for leakage and completion of the consolidation process was made. If the consolidation was completed and there was no leakage through or around the membrane, the pore-pressure transducer would continue to read zero. If there were no leaks and the degree of consolidation were less than 100 percent, the pore pressure would rise slightly and approach a value considerably less than the consolidation pressure. If there were a leak, the pore pressure would rise slowly or rapidly, depending upon the leakage rate, and approach the consolidation pressure. In the case of incomplete consolidation, more time was allowed or the slightly lower consolidation pressure was accepted, and the test was completed. In this case, the consolidation pressure ( $\sigma_3$ ) was the chamber pressure minus the sum of the back pressure and the pore-pressure rise. In the case of a leak, the test was aborted; and the specimen was discarded.

After determining that consolidation was complete and there was no leakage, a final reading of the burette was made. The chamber pressure was then increased (usually 5 psi (34.5 kPa)), and the pore-pressure increase recorded for about ten minutes. The chamber pressure was reduced to the original value, and the valve was opened and left open while the machine was being prepared for the triaxial test. The balance and zero of the recorder were checked and the attenuators on both the load and pore-pressure channels set to the most sensitive scale. The crosshead of the testing machine was then lowered carefully by turning the nuts on the columns to bring the piston into contact with the top cap. A slight seating load -- 1/4 to 1/2 pound (0.11 to 0.23 kg) -- was applied to the specimen to insure firm contact of all load transfer parts. It was very important to obtain a firm contact without applying an appreciable load to the specimen. Before application of the load, the valve between the chamber and back-pressure system was closed so that the specimen would be sheared in an undrained condition.

The strain rate was two percent per hour. Additionally, this strain rate insured a test time of not less than four hours and in most cases approximately eight hours. The strain rate was sufficient to permit equalization of pore pressures within the specimen. Selection of a strain rate of two percent per hour was based partly on past experience and partly on computations made on numerous soil types using a method given by Bishop and Henkel (2). Readings of the deformation, applied load, and pore pressure were taken at approximately 5-minute intervals at the beginning of the test and at 30-minute intervals thereafter.

The test was continued until the stress decreased or remained essentially constant. Some specimens yielded under nearly constant stress while the load continued to increase slightly due to the increasing cross-sectional area. An examination of a few readings indicated whether or not the maximum stress had actually been reached. After failure, all pressures were released and the confining fluid drained from the test chamber. The testing apparatus was disassembled -- being careful not to disturb the specimen. The specimen was examined, and the mode of failure was sketched for future reference. The specimen was weighed and placed in an oven to dry to obtain data to calculate the moisture content and density (1).

## RESULTS

The Division of Research performed four tests at effective confining pressures of 60 psi (414 kPa), 40 psi (276 kPa), 30 psi (207 kPa), and 20 psi (138 kPa). The Division of Materials ran three tests using effective confining pressures of 50 psi (345 kPa), 30 psi (207 kPa), and 11 psi (76 kPa). Effective confining pressures of 40 psi (276 kPa) and 10 psi (69 kPa) were used by the Civil Engineering Department in two tests. All tests were run consolidated-undrained (CU) and with pore-pressure measurements.

Strain rates selected and used by the three participating agencies, the times to failure, and the failure strains of each test performed by the three agencies are shown in the right portion of Table 1. Failure times and strains are listed for both failure criteria -- the maximum principal stress difference and the maximum principal stress ratio. The strain rate used by the Division of Research was selected partly on the basis of past experience and partly on past calculations made on numerous soil types using the method

proposed by Bishop and Henkel (2). Strain rates used by the Civil Engineering Department were computed for each test specimen using the method proposed by Bishop and Henkel. The computations performed by the Civil Engineering Department have been reproduced in the APPENDIX; these calculations illustrate the use of Bishop and Henkel's approach to determine a suitable strain rate. The Division of Materials used a strain rate based on past experience. The strain rates used by the three agencies ranged from 0.00065 to 0.0016 inches per minute (16.5 to 40.6  $\mu\text{m}$  per minute). Based on the maximum principal stress ratio criterion, the failure times and strains ranged from 255 minutes to 1,062 minutes and 5.0 percent to 16.1 percent, respectively. Failure times and strains for the maximum principal stress difference criterion ranged from 385 minutes to 1,026 minutes and 10.7 percent to 18.0 percent. Generally, the maximum principal stress ratio yielded failure times and strains smaller than those obtained from the maximum principal stress difference criterion.

**TABLE 1. COMPARISON OF SOME GENERAL FEATURES OF TRIAXIAL TESTS AND EQUIPMENT OF THE THREE PARTICIPATING AGENCIES**

Agency	Equipment Manufacturer	Pore-Pressure Measuring Device	Axial-Load Measuring Device	Specimen Drainage Condition	Triaxial Test Identification (Kaolinitic Samples)	Strain Rates Used in Tests (Inches/Minute)	Time to Failure (Minutes)		Failure Strains (Percent)	
							$(\sigma_1/\sigma_3)_f$	$(\sigma_1-\sigma_3)_f$	$(\sigma_1/\sigma_3)_f$	$(\sigma_1-\sigma_3)_f$
Division of Research	Karol-Warner <sup>R</sup> Model 530	Pressure Transducer, Bell and Howell <sup>R</sup> , Type No. 4-312.001. Device Located About 1.5 ft (0.46m) From Specimen Base	Load Cell, Transducer Inc. <sup>R</sup> Model-BML2-25-3K-2221. Capacity = 3,000 lb (1,364 kg)	Radial	$\sigma_3^1 = 20$ psi (138 kPa)	0.001 <sup>1</sup>	431	890	8.8	18.9
					$\sigma_3^2 = 30$ psi (207 kPa)	0.001	370	640	7.6	13.3
					$\sigma_3^3 = 40$ psi (276 kPa)	0.001	414	825	8.0	15.9
					$\sigma_3^4 = 60$ psi (414 kPa)	0.001	670	670	16.1	16.1
Division of Materials	Karol-Warner <sup>R</sup> Model 530	Null-Indicator Type, Karol-Warner <sup>R</sup> Model KW-53	Load-Ring Karol-Warner <sup>R</sup> KW-500R and KW-1000 R, Capacity = 500 lb (227 kg) and 1,000 lb (455 kg) Respectively.	Radial and Ends (All Around)	$\sigma_3^1 = 11$ psi (76 kPa)	0.0016 <sup>2</sup>	255	385	5.0	12.9
				$\sigma_3^2 = 30$ psi (207 kPa)	0.0016	293	433	9.5	13.9	
				$\sigma_3^3 = 50$ psi (345 kPa)	0.0016	325	420	10.9	13.7	
U of K Department of Civil Engineering	Geonor <sup>R</sup> (Modified)	Pressure Transducer, Stathan <sup>R</sup> , Model 14032-100. Device Mounted Directly Under Specimen Base.	Load Cell, BLH <sup>R</sup> Model U2M. Sn 30946, Capacity = 500 lb (227 kg)	Radial and Ends (All Around)	$\sigma_3^1 = 10$ psi (69 kPa)	0.00065 <sup>3</sup>	396	564	7.5	10.7
				$\sigma_3^2 = 40$ psi (276 kPa)	0.00098 <sup>3</sup>	1062	1026	12.1	12.5	

1. Selected on the basis of past experience and calculations made on numerous soil types
2. Selected on the basis of past experience
3. Selected on the basis of calculations (APPENDIX A)



The initial moisture content of the nine test specimens are listed in Table 2. The values ranged from 42.8 to 45.3 percent. The average value was 43.7 percent, and the standard deviation was 0.89. The initial void ratios (Table 2) averaged 1.21 with a standard deviation of 0.04. The average dimensions of the four specimens tested by the Division of Research were 4.97 inches (126.2 mm) in height by 23.9 inches (60.7 mm) in diameter. The Division of Materials

tested three specimens having an average height of 4.65 inches (118.1 mm) and an average of diameter of 2.36 inches (59.9 mm). Average dimensions of the two specimens tested by the C. E. Department were 6.00 inches (152.4 mm) in height by 2.37 inches (60.2 mm) in diameter. The length-to-diameter ratios of the specimens tested by Research, Materials, and the CE Department were 2.08, 1.97, and 2.54, respectively.

TABLE 2. TRIAXIAL DATA SUMMARY

SAMPLE	MOISTURE CONTENT (PERCENT)		VOID RATIO		SHEAR STRENGTH PARAMETERS			
	INITIAL	FINAL	INITIAL	FINAL	$(\sigma_1 - \sigma_3)_f$		$(\sigma_1/\sigma_3)_f$	
					$\phi'$ - DEG	C' - psi (kPa)	$\phi'$ - DEG	C' - psi (kPa)
DIV Res $\sigma_3' = 20$ psi (138 kPa)	43.7	40.6	1.21	1.11				
DIV RES $\sigma_3' = 30$ psi (207 kPa)	44.7	39.6	1.28	1.20				
DIV RES $\sigma_3' = 40$ psi (276 kPa)	42.7	38.3	1.20	1.13	29.2	0.18 (1.2)	27.4	1.70 (11.7)
DIV RES $\sigma_3' = 60$ psi (414 kPa)	45.3	36.5	1.27	1.11				
DIV MAT $\sigma_3' = 11$ psi (76 kPa)	42.9	41.5	1.17	1.14				
DIV MAT $\sigma_3' = 30$ psi (207 kPa)	42.8	38.2	1.17	1.05	29.1	0.0 (0)	30.2	0.0 (0)
DIV MAT $\sigma_3' = 50$ psi (345 kPa)	43.1	36.8	1.18	0.99				
CE DEPT $\sigma_3' = 10$ psi (69 kPa)	44.5	42.6	1.22	1.17				
CE DEPT $\sigma_3' = 40$ psi (276 kPa)	43.3	37.6	1.17	1.02	28.7	1.04 (7.2)	28.6	1.21 (8.3)

To check the amount of consolidation of the nine tests relative to each other and to effective confining pressure, the ratio of initial moisture content to the moisture content after consolidation was plotted as a function of effective confining pressure. The ratio of the two moisture contents was used to remove the effects of variations in the initial moisture contents. The results are shown in Figure 10. The line drawn through the data is simply to indicate a trend and is not a regression line. All of the points are closely grouped around the trend line, indicating that each test consolidated as expected in relation to the other eight tests. This is also an indication that there were no leaks from the chamber pressure to the back pressure on any of the tests. Had one test developed a leak, the point representing that particular test would have fallen a considerable distance below the line. The amount of water forced into the burette system, measuring volume change of the sample, would have been unusually large, falsely indicating that a large amount of water had been forced from the specimen. This would indicate a lower final moisture content and, consequently, a lower moisture content ratio.

The results of the four tests performed by the Division of Research are shown in Figure 11. A computer program, presently under development in this Division, was used to calculate the shear strength parameters ( $c'$ ,  $\phi'$ ). Also, two failure criteria were used to determine those parameters -- the maximum principal stress difference  $(\sigma_1 - \sigma_3)_f$  and the maximum principal stress ratio  $(\sigma_1/\sigma_3)_f$ . The computer program fits regression lines ( $k_f$ -lines) through the p-q failure points defined by the two criteria and then calculates the shear strength parameters  $c'$  and  $\phi'$  for each criterion. The results, listed in Table 1 and plotted in Figure 11, yield a  $\phi'$  of 29.2 degrees and  $c'$  of 0.18 psi (1.2 kPa) for  $(\sigma_1 - \sigma_3)_f$  and a  $\phi'$  of 27.4 degrees and  $c'$  of 1.70 psi (11.8 kPa) for  $(\sigma_1/\sigma_3)_f$ .

The results of computer analyses on the three tests by the Division of Materials gave a  $\phi'$  of 31.1 degrees and  $c'$  of -0.97 psi (-6.7 kPa). This was for the failure criterion of  $(\sigma_1 - \sigma_3)_f$ . For the criterion of  $(\sigma_1/\sigma_3)_f$ ,  $\phi'$  was 31.1 degrees and  $c'$  equaled -0.47 psi (0.32 kPa). To avoid a negative cohesion value, the cohesion was constrained through zero; and the analysis was performed again. The resulting  $\phi'$ 's were 29.1 degrees and 30.2 degrees for  $(\sigma_1 - \sigma_3)_f$  and  $(\sigma_1/\sigma_3)_f$ , respectively; and, of course,  $c'$  was zero for both criteria. Test results for the Division of Materials are shown in Figure 12 and listed in Table 2.

Table 2 also lists results of the two tests performed by the CE Department (also see Figure 13). For  $(\sigma_1 - \sigma_3)_f$ , a  $\phi'$  of 28.7 degrees and  $c'$  of 1.04 psi (7.2 kPa) were calculated. For  $(\sigma_1/\sigma_3)_f$ ,  $\phi'$  and  $c'$  were 28.5 degrees and 1.21 psi (8.3 kPa), respectively.

In Figure 14, the p-q diagrams from the nine tests have been plotted and analyses made to determine the shear strength parameters. The collective  $\phi'$  equaled 28.8 degrees, and  $c'$  was 0.88 psi (6.1 kPa).

To effectively evaluate small anomalies in specimen composition and to evaluate different testing equipment and methods, it was necessary to compare each test with all others without the obvious effects produced by differences in effective confining pressures. This may be done by "normalizing" the p-q diagrams, stress-strain curves, and pore pressure-strain curves with respect to effective confining pressure,  $\sigma_3'$ . The variables p, q, axial stress, and pore pressure for each test are divided by  $\sigma_3'$  for that particular test. If the samples and test methods are reasonably alike, the curves should approach coincidence. Figures 15 through 17 are the normalized p-q diagrams, stress-strain curves, and pore pressure-strain curves, respectively.

Figure 15 indicates that two tests (Division of Materials at 11 psi (76 kPa) and CE Department at 10 psi (69 kPa)) did not match the remaining seven tests. The deviator stress was disproportionately large in both tests (Figure 16), and the Division of Materials test developed relatively small pore pressures (see Figure 17), causing the p-q diagram to be shifted to the right. This indicates the preconsolidation pressure on the two specimens was greater than 11 psi (76 kPa); therefore, the specimens were tested in a slightly overconsolidated condition.

An examination of Figure 15 will show that the test performed by the CE Department at 10 psi (69 kPa) is slightly above the  $k_f$ -line (well developed pore pressures with high deviator stresses); however, the test by the Division of Materials at 11 psi (76 kPa) is below the  $k_f$ -line (poorly developed pore pressures with high deviator stresses). This is the reason a small value of cohesion was calculated for the tests performed by the CE Department (Figure 13) and for the negative value of cohesion computed for the unconstrained data of the Division of Materials. A redefinition of the  $k_f$ -line, ignoring these two tests, gives a  $\phi'$  of 29.3 degrees and a  $c'$  of 0.018 psi (0.124 kPa) for  $(\sigma_1 - \sigma_3)_f$  and a  $\phi'$  of 28.8 degrees with a  $c'$  of 0.884 psi (6.095 kPa) for  $(\sigma_1/\sigma_3)_f$ .

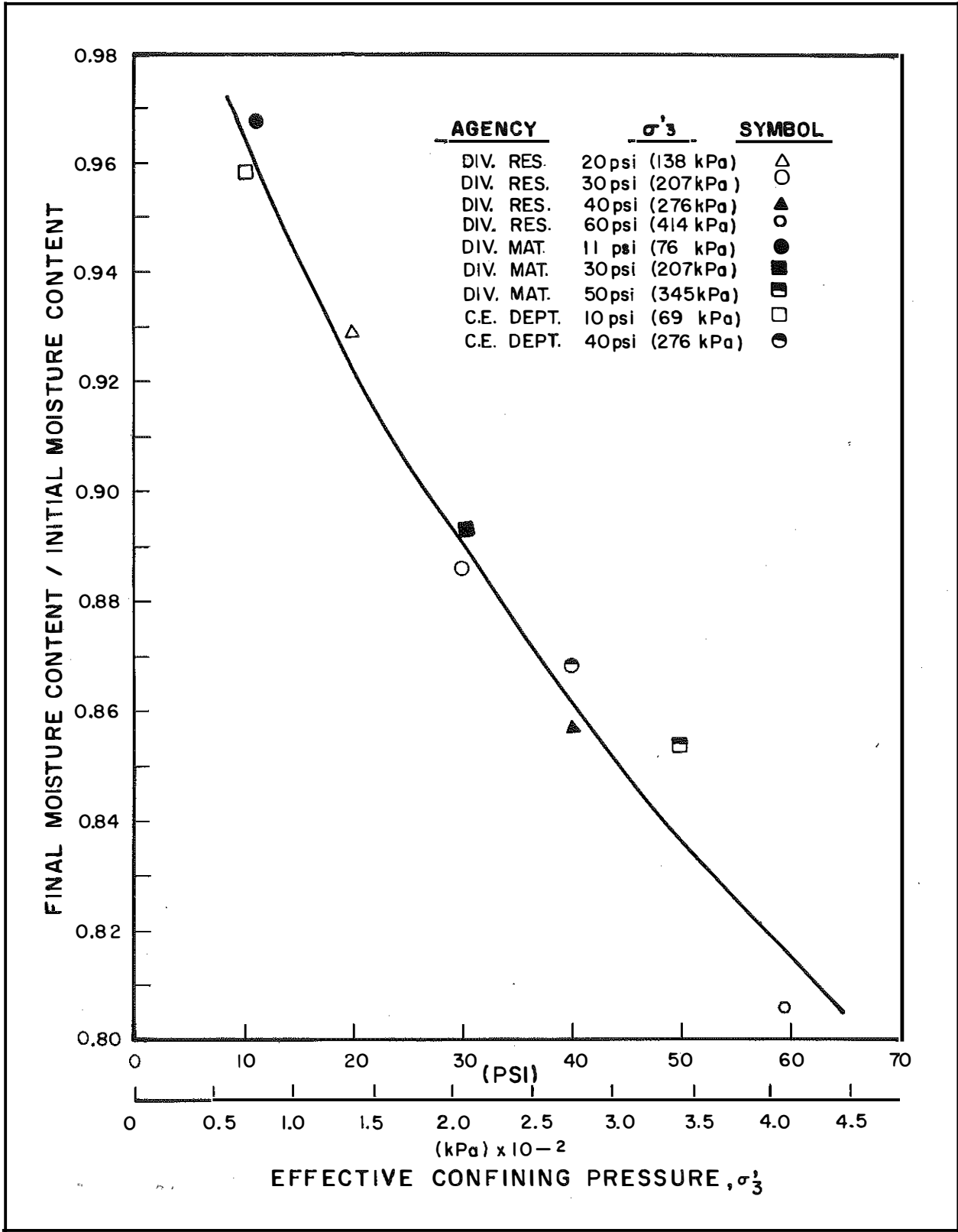


Figure 10. Ratio of Final Moisture Content and Initial Moisture Content as a Function of Effective Confining Pressure.

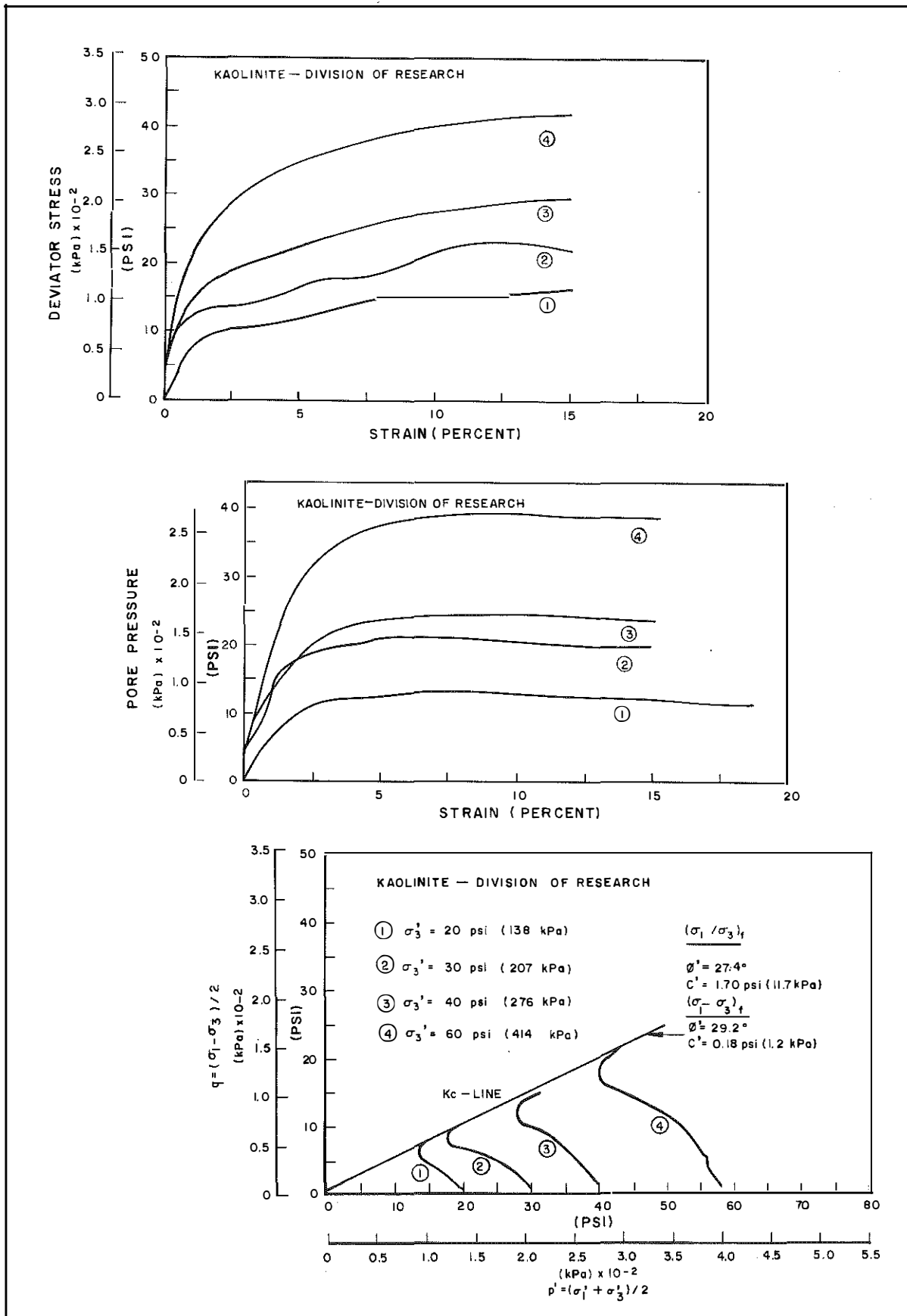


Figure 11. Triaxial Test Results Obtained by the Division of Research.

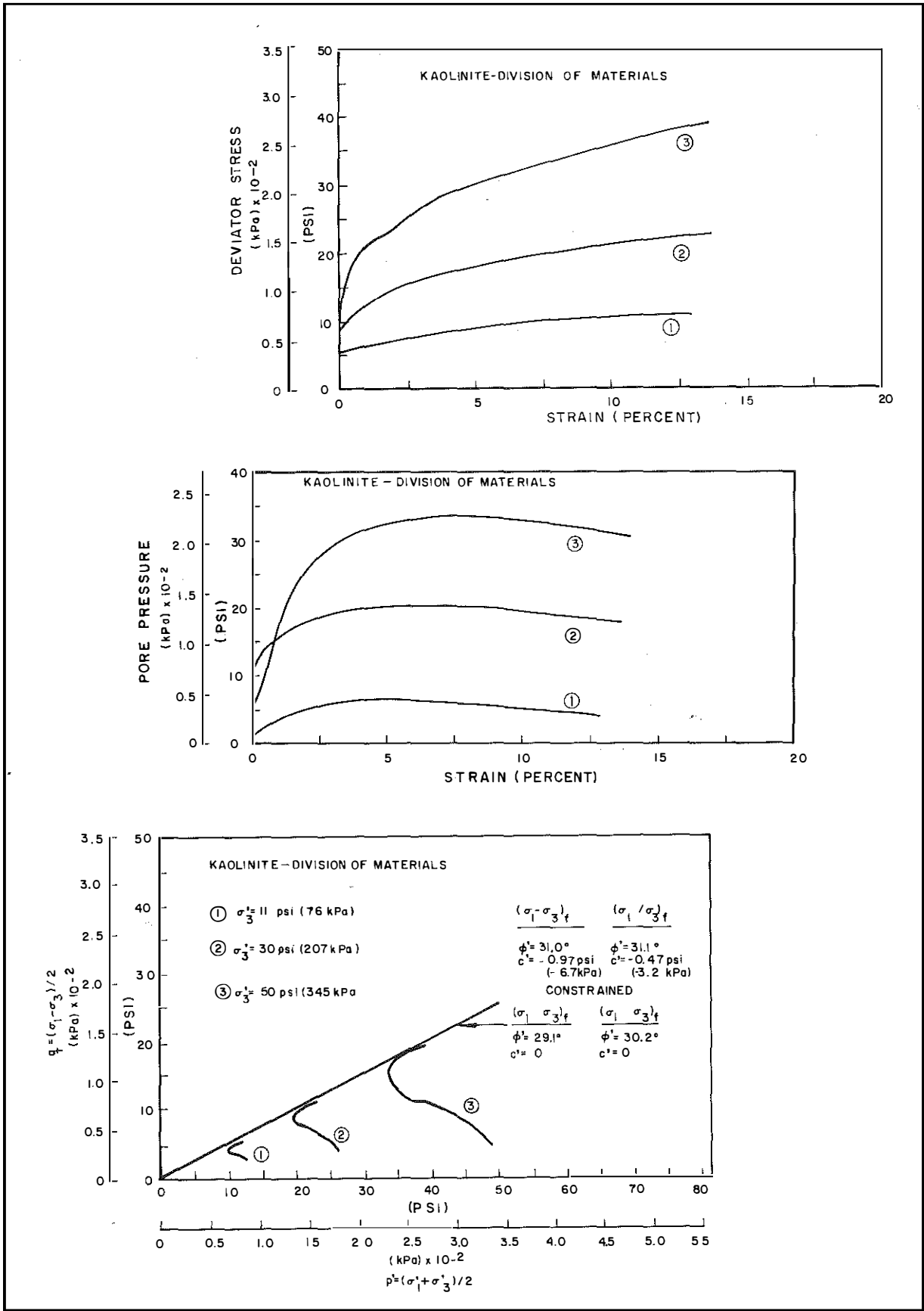


Figure 12. Triaxial Test Results Obtained by the Division of Materials.

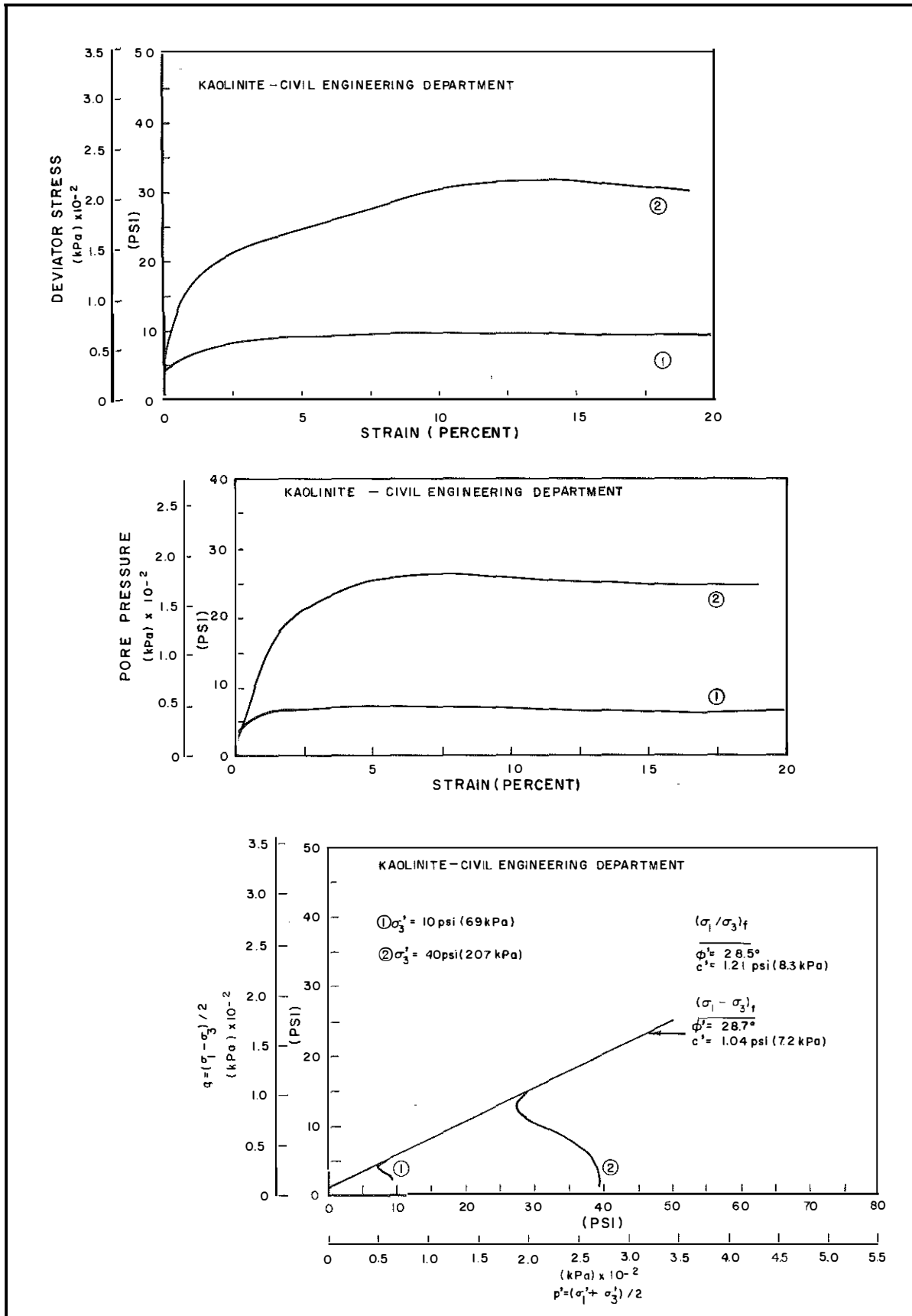


Figure 13. Triaxial Test Results Obtained by the CE Department.

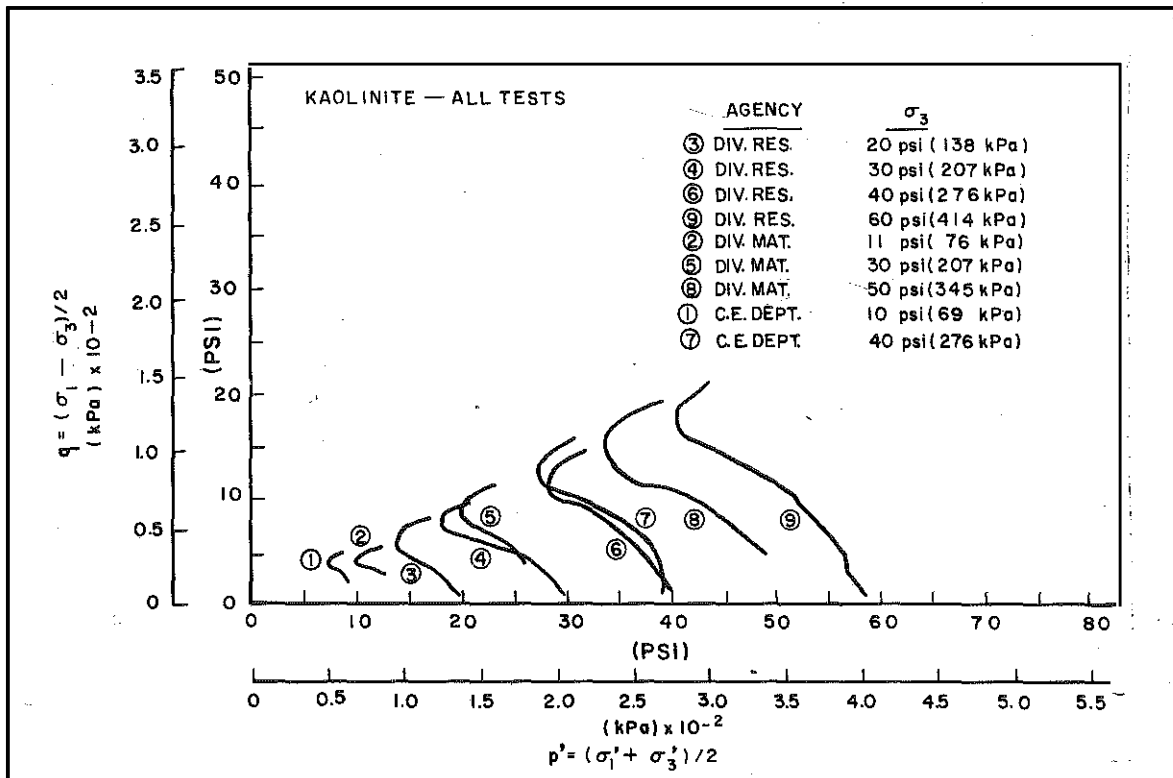


Figure 14. Composite Plot of Triaxial Results Obtained by the Three Participating Agencies.

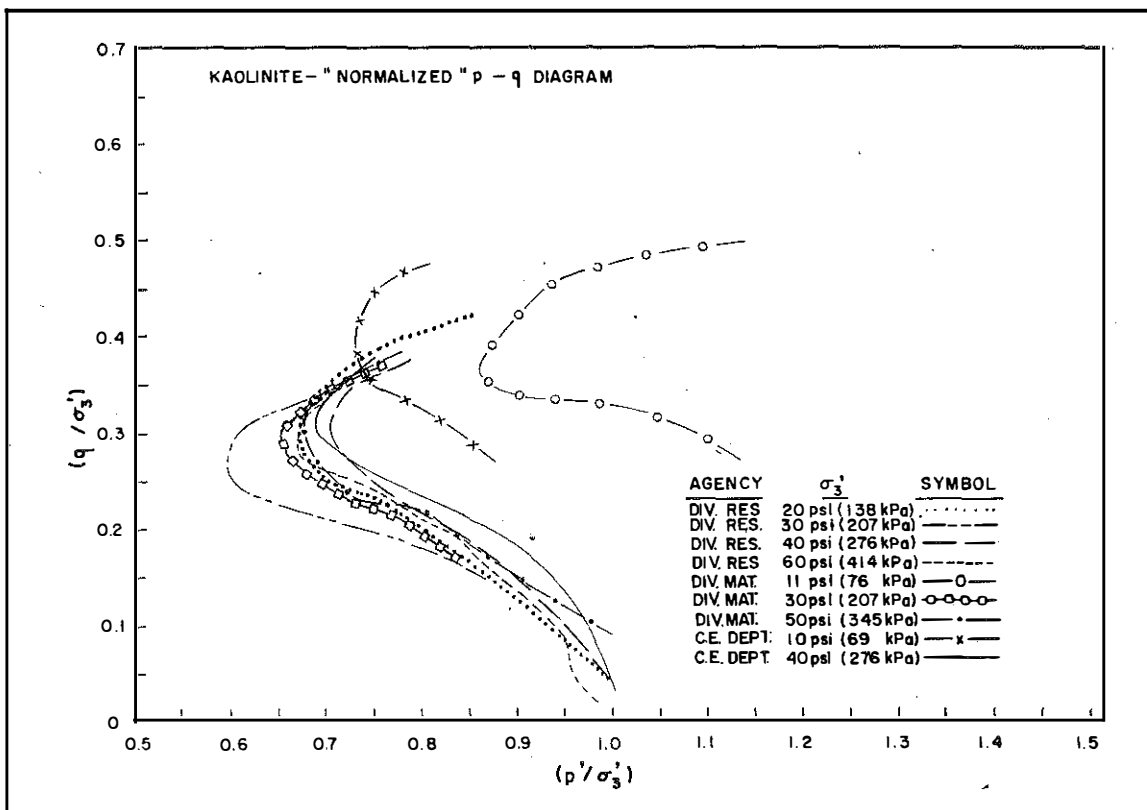


Figure 15. "Normalized" p-q Diagrams for the Nine Triaxial Tests Performed by the Three Participating Agencies.

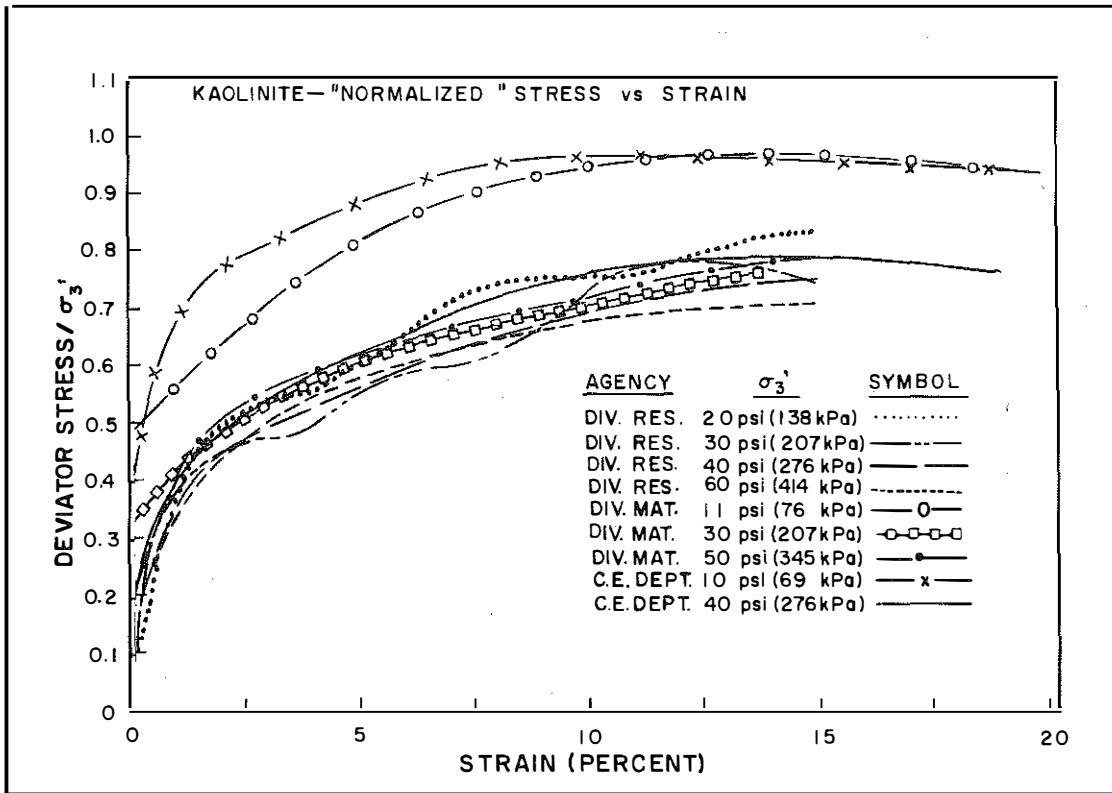


Figure 16. "Normalized" Deviator Stress-versus-Strain Curves for the Nine Triaxial Tests Performed by the Three Participating Agencies.

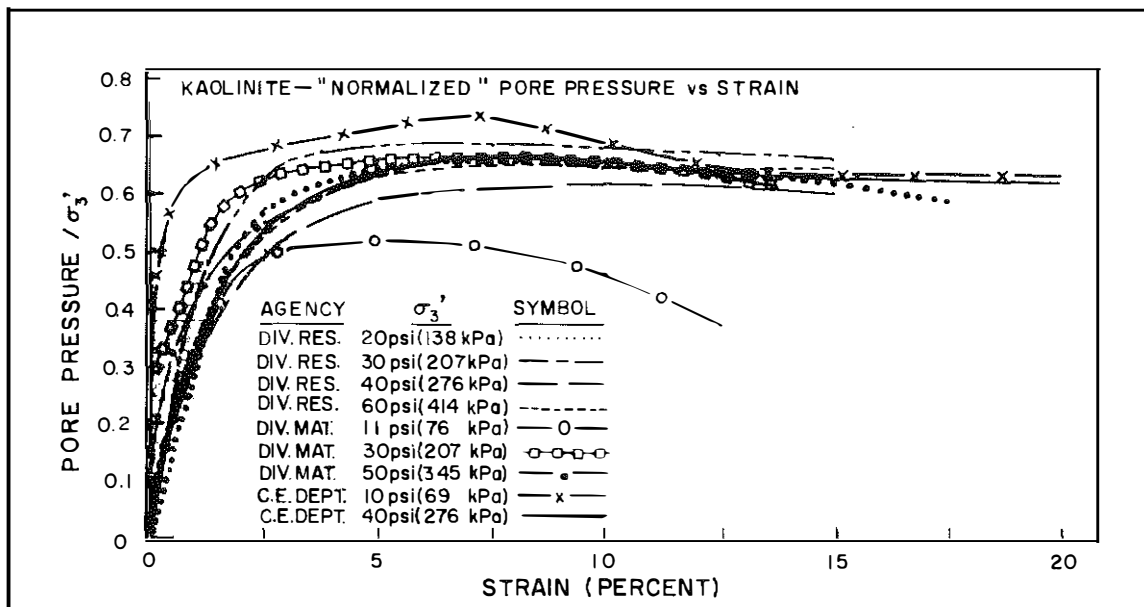


Figure 17. "Normalized" Pore Pressure-versus-Strain Curves for the Nine Triaxial Tests Performed by the Three Participating Agencies.



## IMPLEMENTATION

The test performed by the Division of Research at an effective confining pressure of 20 psi (138 kPa) corresponds well with the remaining tests. This would indicate that the specimens were normally consolidated. Therefore, it appears the preconsolidation pressure (produced by the vacuum extrusion process) must be between 12 psi (83 kPa) and 20 psi (138 kPa). However, unpublished data obtained by this Division, for consolidation tests on this material, indicate the preconsolidation pressure was about 22 psi (152 kPa). For purposes of further testing, a preconsolidation pressure of 20 psi (138 kPa) would be a reasonable assumption.

Figure 18 shows the average normalized p-q diagram, stress-strain and pore pressure-strain curves, for the seven normally consolidated tests. Any other testing agency interested in comparing their testing procedures and sample preparation procedures could do so by running triaxial tests on kaolinite, normalizing their data as described herein, and comparing the results with Figure 18. However, to make a valid comparison, the tests must be performed above the preconsolidation pressure of 20 psi (138 kPa).

## CONCLUSIONS

1. The two tests performed at effective confining pressures of 11 psi (76 kPa) were slightly overconsolidated.
2. The preconsolidation pressure of the remolded kaolinite specimens appeared to be approximately 20 to 22 psi (138 to 152 kPa).
3. The seven normally consolidated tests behaved in a similar fashion, with excellent agreement in test results. This indicated sample anomalies, testing methods, and test equipment of the three laboratories had little effect on results.
4. The shear strength parameters obtained from the maximum principal stress difference criterion and those obtained from the maximum principal stress ratio failure criterion were slightly different. The failure criterion of maximum principal stress difference  $(\sigma_1 - \sigma_3)_f$  gave a better correlation of shear strength parameters than did the maximum principal stress ratio  $(\sigma_1/\sigma_3)_f$ .
5. The shear strength parameters, using the seven normally consolidated tests and the maximum principal stress difference failure criterion, were a  $\phi'$  of 29.3 degrees and  $\phi'$  of 0.018 psi (0.124 kPa).
6. Normalizing the p-q diagrams, stress-strain curves, and pore pressure-strain curves is a good method of comparing results of tests performed at different effective confining pressures.

Any agency interested in participating in the triaxial testing forum proposed in this report should contact the Director, Division of Research, Kentucky Department of Transportation, 533 South Limestone Street, Lexington, Kentucky 40508 (Phone: (606) 254-4475). Each request will be considered on an individual basis, and "standardized" kaolinite specimens will be supplied to the interested agency, provided such request is considered to be of interest to the testing forum. Before specimens will be supplied to any agency, the interested agency must agree to submit data developed from testing the kaolinite specimens, including raw data, to the Division of Research. It should be understood by each interested agency that their data may be published at some future date.

## ACKNOWLEDGEMENTS

The Division of Research would like to thank Dr. Vincent Drnevich of the CE Department of the University of Kentucky and the Division of Materials of KYDOT for their participation in the triaxial testing study.

## REFERENCES

1. Scott, G. D.; *Rheological and Ultimate Strength Parameters of Cohesive Soils*, Division of Research, Kentucky Department of Highways, February 1970.
2. Bishop, A. W.; and Henkel, D. J.; *The Measurement of Soil Properties in the Triaxial Test*, Edward Arnold Ltd., London, second Edition, 1962.

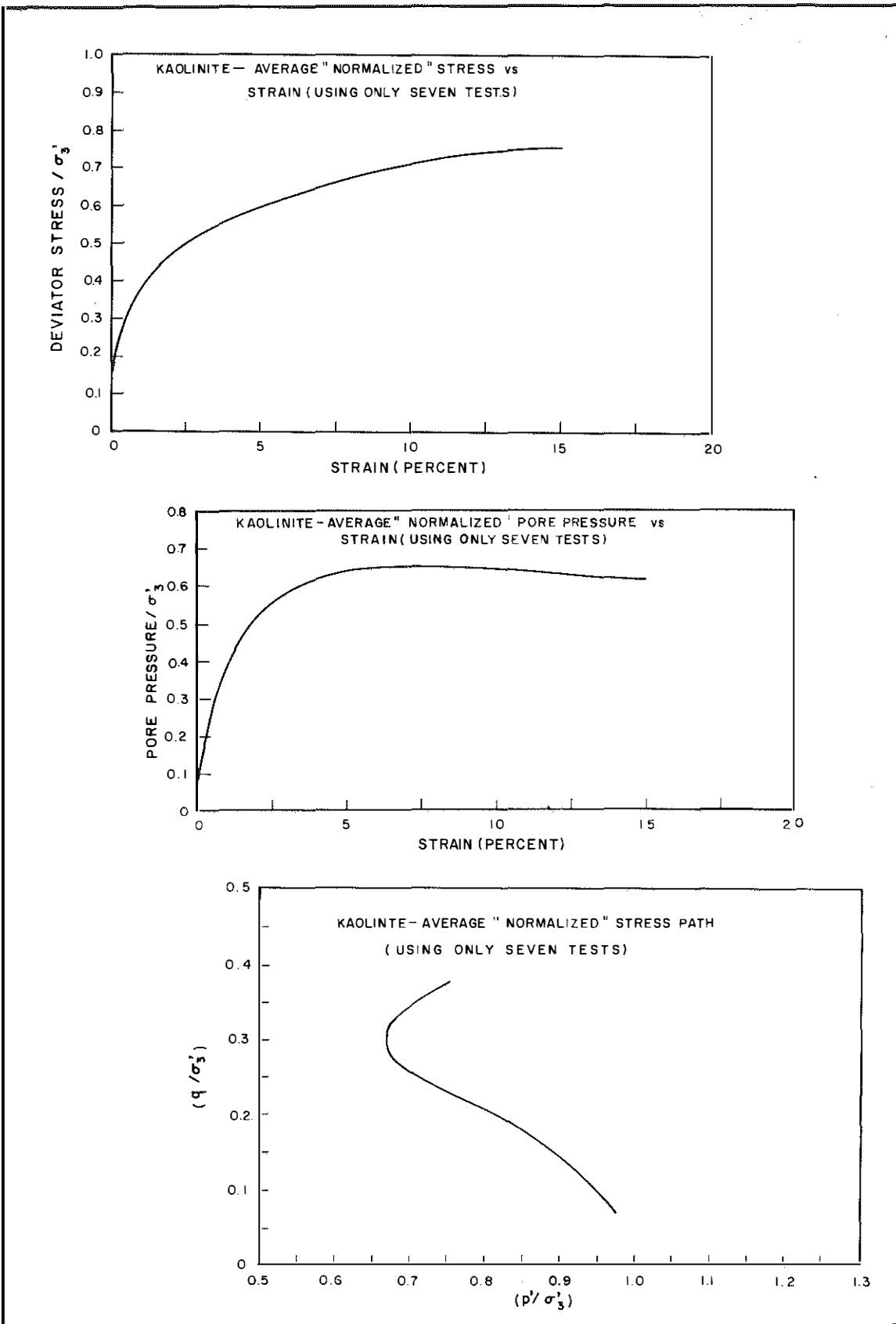


Figure 18. Average "Normalized" Curves for Seven Selected Triaxial Tests Performed by the Three Participating Agencies.

**APPENDIX**

**CALCULATIONS ILLUSTRATING  
A METHOD OF SELECTING STRAIN RATES  
FOR TRIAXIAL TESTING**

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**RATE OF TESTING FOR  
KYDOT TRIAXIAL TESTS ON  
KAOLINITE SOIL SPECIMENS**

by  
Dr. V. P. Drnevich  
Department of Civil Engineering  
University of Kentucky

Reference: Bishop, A. W.; and Henkel, D. J.; *The Triaxial Test*, Part III, Section 4, Item (2), pp 123-127 and Appendix 5, Section 4, pp 192-204.

The time required for 95-percent pore pressure equalization is given by (assuming both end and radial drainage)

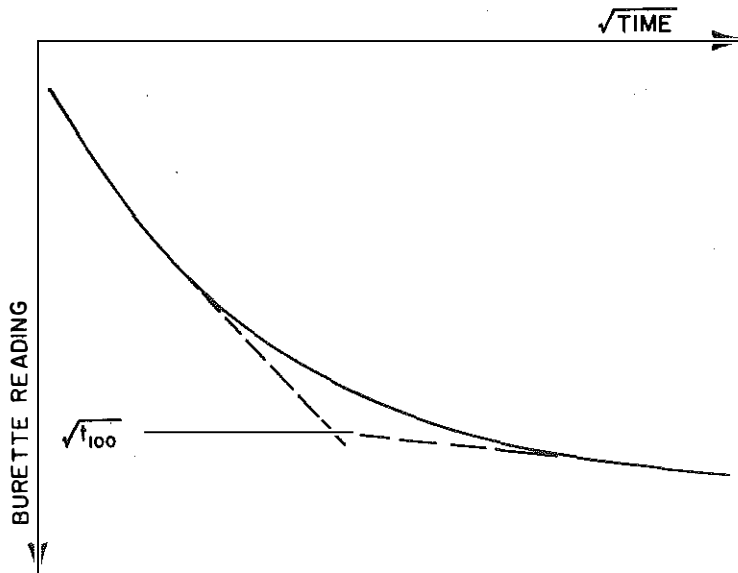
$$t_{95} = 0.071(h)^2/C_v \quad 1$$

where  $h$  = specimen half-height = diameter of specimen and  
 $C_v$  = coefficient of consolidation.

For the same boundary conditions,  $C_v$  may be estimated from consolidation-time relations from

$$C_v = \pi h^2/100 t_{100} \quad 2$$

where  $t_{100}$  can be obtained from the square-root-of-time plot, as shown below:



Substituting Equation 2 into Equation 1,

$$t_{95} = 0.71 h^2 / [(\pi/100)h^2/t_{100}] = 2.26 t_{100}$$

Let  $\epsilon\%$  be the strain at which equalization is desired and  $\dot{\epsilon}\%$  be the strain rate for testing,

$$\dot{\epsilon}\% = \epsilon\% / t_{95} = 0.442 \epsilon\% / t_{100}$$

Since soil is normally consolidated or lightly preconsolidated, let equalization occur at five-percent strain.

For KAOL1,

$$\begin{aligned} \dot{\epsilon}\% &= 0.442 \times 5\% \\ &= 0.017\%/\text{min or} \\ &1,176 \text{ min to 20-percent strain.} \end{aligned}$$

Considering a specimen length of 6.0 inches, then

$$\begin{aligned} \text{Rate of deformation} &= \dot{\epsilon}\% L/100\% \\ &= 0.0010 \text{ in./min (25 } \mu\text{m/min)} \end{aligned}$$

For KAOL2,

$$\begin{aligned} \dot{\epsilon}\% &= 0.442 \times 5\% / 368.6 \text{ min} \\ &= 0.006\%/\text{min or} \\ &3,336 \text{ min to 20-percent strain.} \end{aligned}$$

For a specimen length of 6.0 inches,

$$\begin{aligned} \text{Rate of deformation} &= \dot{\epsilon}\% L/100\% \\ &= 0.00036 \text{ in./min (9.1 } \mu\text{m/mm)} \end{aligned}$$

Use 0.00065 in./min (16.5  $\mu\text{m/min}$ ).