APPENDIX X:

TRIAXIAL COMPRESSION TESTS

1. PRINCIPLES OF THE TRIAXIAL COMPRESSION TEST. The triaxial compression test is used to measure the shear strength of a soil under controlled drainage conditions. In the basic triaxial test, a cylindrical specimen of soil encased in a rubber membrane is placed in a triaxial compression chamber, subjected to a confining fluid pressure, and then loaded axially to failure. Connections at the ends of the specimen permit controlled drainage of pore water from the specimen. The procedures presented herein apply only to the basic test conducted with limited drainage conditions, and do not include special types or variants of this test. In general, a minimum of three specimens, each under a different confining pressure, are tested to establish the relation between shear strength and normal stress. The test is called "triaxial" because the three principal stresses are known and controlled. Prior to shear, the three principal stresses are equal to the chamber fluid pressure. During shear, the major principal stress, σ_1, is equal to the applied axial stress (P/A) plus the chamber pressure, σ_3 (see Fig. 1). The applied axial stress, $\sigma_1 - \sigma_3$, is termed the "deviator stress." The

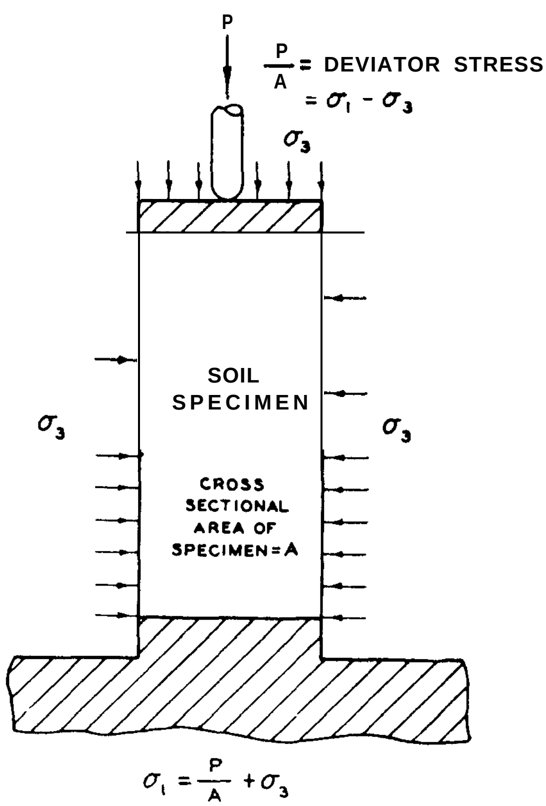


Figure 1. Diagram showing stresses during triaxial compression test

intermediate principal stress, σ_2, and the minor principal stress, σ_3, are identical in the test, and are equal to the confining or chamber pressure hereafter referred to as σ_3.

A soil mass may be considered as a compressible skeleton of solid particles. In saturated soils the void spaces are completely filled with water; in partially saturated soils the void spaces are filled with both water and air. Shear stresses are carried only by the skeleton of solid particles, whereas the normal stress on any plane is carried by both the solid particles and the pore water. In a triaxial test, the shear strength is determined in terms of the total stress (intergranular stress plus pore water pressure), unless (a) complete drainage is provided during the test so that the pore pressure is equal to zero at failure, or (b) measurements of pore pressure are made during the test. When the pore pressure at failure is known, the shear strength can be computed in terms of the stress carried by the soil particles (termed effective or intergranular stress). In recent years, significant advances have been made in the techniques of measuring pore pressures in the triaxial test and in the interpretation of the data obtained; however, difficulties still exist in this respect. Pore pressure measurements during shear are seldom required in routine investigations, as the basic triaxial tests are sufficient to furnish shear strengths for the limiting conditions of drainage. Procedures for measuring pore pressures in the triaxial test during shear are discussed elsewhere† and are beyond the scope of this manual.

2. TYPES OF TESTS. The three types of basic triaxial compression tests are unconsolidated-undrained, consolidated-undrained, and consolidated-drained, subsequently referred to as the Q, R, and S tests, respectively. As these names imply, they are derived from the drainage conditions allowed to prevail during the test. The type of test is selected

† A. W. Bishop and D. J. Henkel, The Measurement of Soil Properties in the Triaxial Test, 2nd ed. (London, Edward Arnold Ltd., 1962).

to closely simulate, or to bracket, the conditions anticipated in the field. In the basic tests, the initial principal stresses are equal; that is, no attempt is made to duplicate stress systems in the field in which the principal stresses are not equal.

a. Q Test. In the Q test the water content of the test specimen is not permitted to change during the application of the confining pressure or during the loading of the specimen to failure by increasing the deviator stress. The Q test is usually applicable only to soils which are not free-draining, that is, to soils having a permeability less than 10 $\times 10^{-4}$ cm per sec.

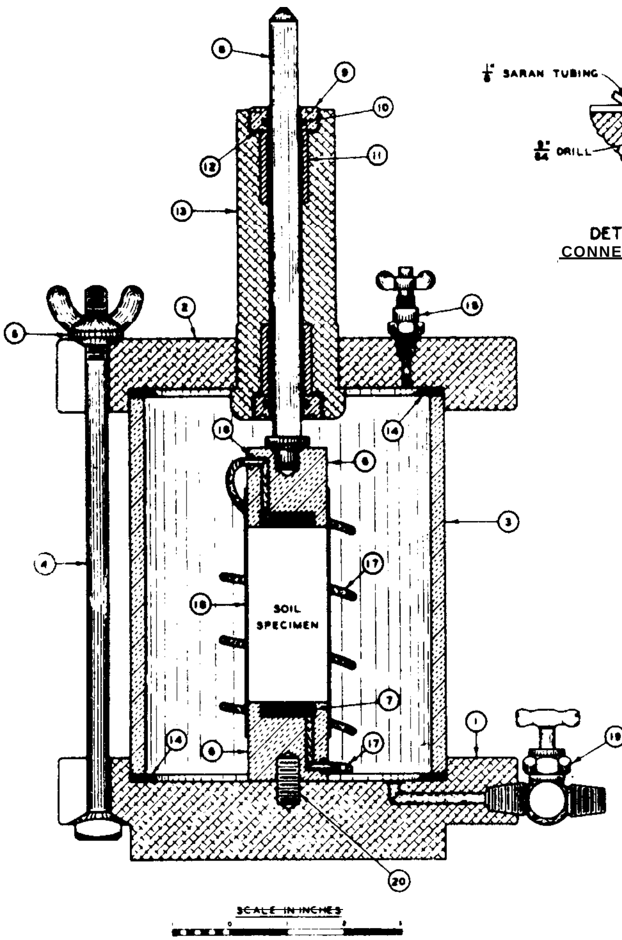
b. R Test. In the R test, complete consolidation of the test specimen is permitted under the confining pressure. Then, with the water content held constant, the specimen is loaded to failure by increasing the deviator stress. Specimens must as a general rule be completely saturated before application of the deviator stress.

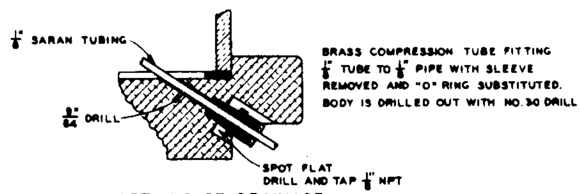
c. S Test. In the S test, complete consolidation of the test specimen is permitted under the confining pressure and during the loading of the specimen to failure by increasing the deviator stress. Consequently, no excess pore pressures exist at the time of failure.

3. APPARATUS. a. Loading Devices. Various devices may be used to apply axial load to the specimen. These devices can be classified as either apparatus in which axial loads are measured outside the triaxial chamber or apparatus in which axial loads are measured inside the triaxial chamber by using a proving ring or frame, an electrical transducer, or a pressure capsule. Any- equipment used should be calibrated to permit determination of loads actually applied to the soil specimen.

Loading devices can be further grouped under controlled-strain or controlled-stress types. In controlled-strain tests, the specimen is strained axially at a predetermined rate; in controlled-stress tests, predetermined increments of load are applied to the specimen at fixed intervals of time. Controlled-strain loading devices, such as commercial testing machines, are preferred for short-duration tests using piston-type test apparatus. If available, an automatic stress-strain recorder may be used to measure and record applied axial loads and strains.

b. Triaxial Compression Chamber. The triaxial compression chamber consists primarily of a headplate and a baseplate separated by a transparent plastic cylinder.* A drawing of a typical triaxial compression chamber for 1.4-in. -diameter specimens is shown in Figure 2. Chamber





DETAILS OF DRAINAGE CONNECTIONS THROUGH BASE

PART NO.	NAME	MATERIAL	QUANTITY
1	BASEPLATE	ALUMINUM	1
2	HEADPLATE	ALUMINUM	1
3	CYLINDER	TRANSPARENT PLASTIC	1
4	TIE ROD AND WING NUT	STAINLESS STEEL	3
5	TIE ROD WASHER	STAINLESS STEEL	3
6	SPECIMEN CAP AND BASE	PLASTIC	2
7	POROUS DISC	CERAMIC OR METAL	2
8	PISTON (GROUND AND POLISHED, CASE HARDENED) 1/2" DIAM	STEEL	1
9	"O" RING HOUSING	BRASS	2
10	11/16" O D "O" RING	RUBBER	2
11	LINEAR BALL BUSHING	STAINLESS STEEL	2
12	GASKET	FIBER	2
13	PISTON HOUSING	ALUMINUM	1
14	GASKET	RUBBER	2
15	1/8" N P T NEEDLE VALVE	BRASS	1
16	1/4" O D "O" RING	RUBBER	2
17	1/8" O D TUBING	PLASTIC	2
18	SPECIMEN MEMBRANE 1.4" I D , 5" LONG	RUBBER	
19	1/4" x 1/4" N P T NEEDLE VALVE OR A CIRCLE SEAL VALVE OF THE 9500 SERIES	BRASS	1
20	3/8" THREADED STUD	ALUMINUM	1

Figure 2. Details of typical triaxial compression chamber

* Adequate safety precautions should be taken, or the transparent plastic cylinder should be replaced by a metal cylinder, if chamber pressures in excess of 100 psi are used.

dimensions and type will vary depending on the size of specimen tested and on pressure and load requirements. The baseplate has one inlet through which the pressure liquid is supplied to the chamber and two inlets leading to the specimen base and cap to permit saturation and drainage of the specimen when required, The headplate has a vent valve so that air can be forced out of the chamber as it is filled with the pressure fluid. The cylinder is held tightly against rubber gaskets by bolts or tie rods connecting the headplate and baseplate.

In piston-type test apparatus in which the axial, loads are measured outside the triaxial compression chamber, piston friction can have a significant effect on the indicated applied load, and measures should be taken to reduce friction to tolerable limits. Pistons generally should consist of ground and polished case-hardened steel rods with diameters between 1/4 and 1/2 in. for testing 1.4-in.-diameter specimens; heavier pistons are required for larger specimens. The following measures have been found to reduce piston friction to tolerable amounts.

(1) The use of linear ball bushings as shown in Figure 2. The unique design of these bushings permits unlimited axial movement of the piston with a minimum of friction. Leakage around the piston is reduced by means of Q-rings, Quad-rings, flexible diaphragms, or other devices. A seal incorporating Q-rings is shown in Figure 2. The beneficial effects of using linear ball bushings in comparison with steel bushings are demonstrated by the data shown in Figure 3. The amount of lateral force

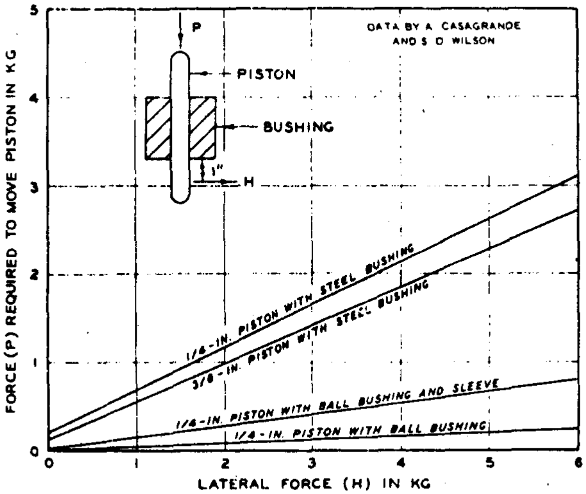


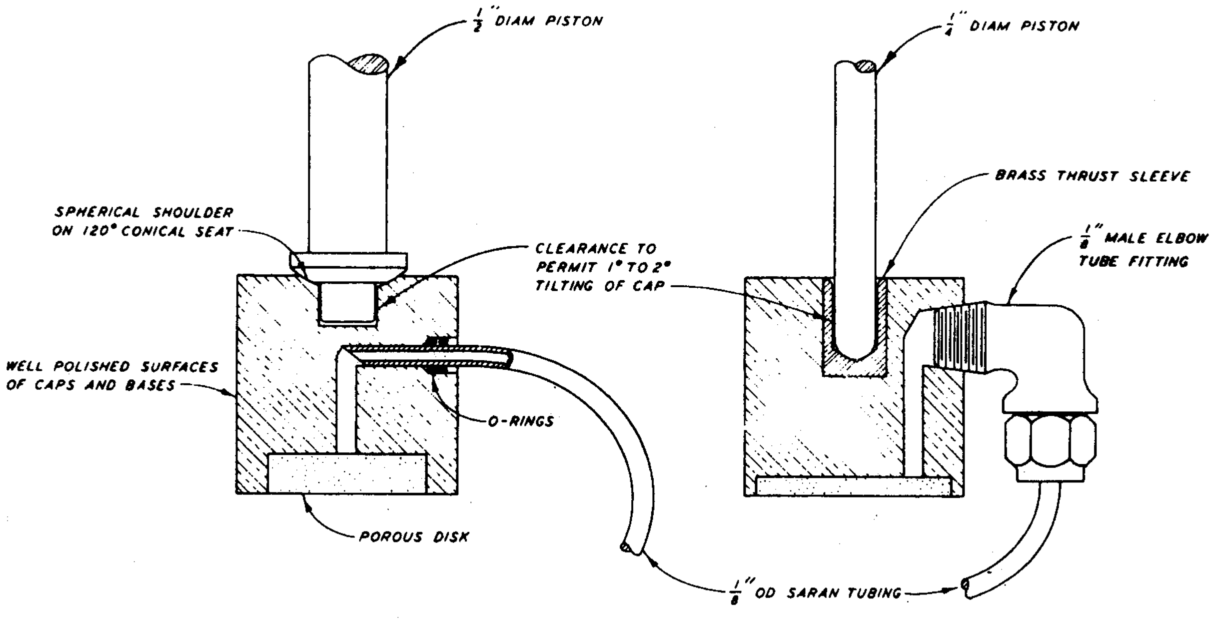
Figure 3. Effect of lateral force on piston friction in triaxial compression apparatus

transmitted to the piston, if the specimen cap tends to tilt during a test, cannot be determined; however, the data shown in Figure 3 indicate that the resulting piston friction would be negligible even for relatively large lateral forces.

(2) Rotation of the piston within the bushings during the application of the deviator stress. (Commercial devices are available to rotate the piston during the test.) This method is very effective in reducing friction; however, a more complex design of the specimen cap is necessary, and unless the piston is rotated continuously, appreciable friction would still develop during longtime tests. When linear ball bushings are used, the piston should never be rotated except under special conditions designated by the manufacturer.

Although these measures will reduce piston friction to negligible amounts during the course of the test, it is always preferable to measure the actual piston friction before the start of the test. This can readily be done by starting the axial load application with the bottom of the piston raised slightly above the top of the specimen cap. Thus any starting friction or residual friction, as indicated by the load necessary to move the piston down into contact with the cap, can be subtracted from the measured load.

c. Specimen Caps and Bases. Specimen caps and bases should be constructed of a lightweight noncorrosive material and should be of the same diameter as the test specimen in order to avoid entrapment of air at the contact faces. Solid caps and bases should be used for the Q test to prevent drainage of the specimens. Caps and bases with porous metal or porous stone inserts and drainage connections, as shown in Figure 4, should be used for the R and S tests. The porous inserts should be more pervious than the soil being tested to permit effective drainage. For routine testing, stones of medium porosity are satisfactory. The specimen cap should be designed to permit slight tilting with the piston in contact position, as shown in Figure 4.



a. WES TYPE

b. HARVARD TYPE

Figure 4. Details of typical 1.4-in.-diameter specimen caps showing drainage connections and piston seats

d. Rubber Membranes. Rubber membranes used to encase the specimen should provide reliable protection against leakage, yet offer minimum restraint to the specimen. Commercially available rubber membranes having thicknesses ranging from 0.0025 in. (for soft clays) to 0.010 in. (for sands or for clays containing sharp particles) are generally satisfactory for 1.4-in.-diameter specimens. Rubber membranes about 0.010 in. or greater in thickness are suitable for larger specimens. Membranes should be carefully inspected prior to use, and if any flaws or pinholes are evident, the membranes should be discarded. The use of two thin membranes separated by a thin film of silicone grease will afford protection against leakage through an undetected pinhole and will minimize the possibility of air leakage from the chamber fluid into the specimen during tests of relatively long duration. Since no rubber membrane is completely impervious, the use of special membranes or chamber fluids† may sometimes be necessary, such as during periods of undrained shear that exceed a few hours. The membrane is sealed against the cap and base by rubber O-rings or rubber bands. Leakage around the ends of the membrane, where it is sealed against the cap and the base, as well as through fittings, valves, etc., can develop unless close attention is given to details in the manufacture and use of the apparatus.†

e. Equipment for Preparing Specimens. (1) Cohesive soils. A specimen trimming frame is recommended for preparing specimens of most cohesive soils. The specimen is held in a vertical position between two circular plates containing pins which press into the ends of the specimen to prevent movement during trimming. The edges of the trimming frame act as vertical guides for the cutting equipment and control the final diameter of the specimen. Details of a typical trimming frame for 1.4-in.-diameter specimens are shown in Figure 5. Wire saws and knives of various sizes and types are used with the trimmer (see Fig. 7, p. 12). Split or solid cylinders with a beveled cutting edge can also be used to trim specimens. The use of a motorized soil lathe may be advantageous in reducing the time required for preparing specimens of certain types of soils. A miter box or cradle (see Fig. 8, p. 13) is required to trim the specimen to a fixed length and to insure that the ends of the specimen are parallel with each other and perpendicular to the axis of the specimen.

(2) Cohesionless soils. A forming jacket consisting of a split mold which incloses a rubber membrane is required for cohesionless soils. The inside diameter of the mold minus the double thickness of the membrane is equal to the diameter of the specimen required. A funnel or special spoon (see Fig. 5 of Appendix VII, PERMEABILITY TESTS) for placing the material inside the jacket and a tamping hammer or vibratory

† S. J. Poulos, Report on Control of Leakage in the Triaxial Test, Soil Mechanics Series No. 71, Harvard University (Cambridge, Mass., March 1964).

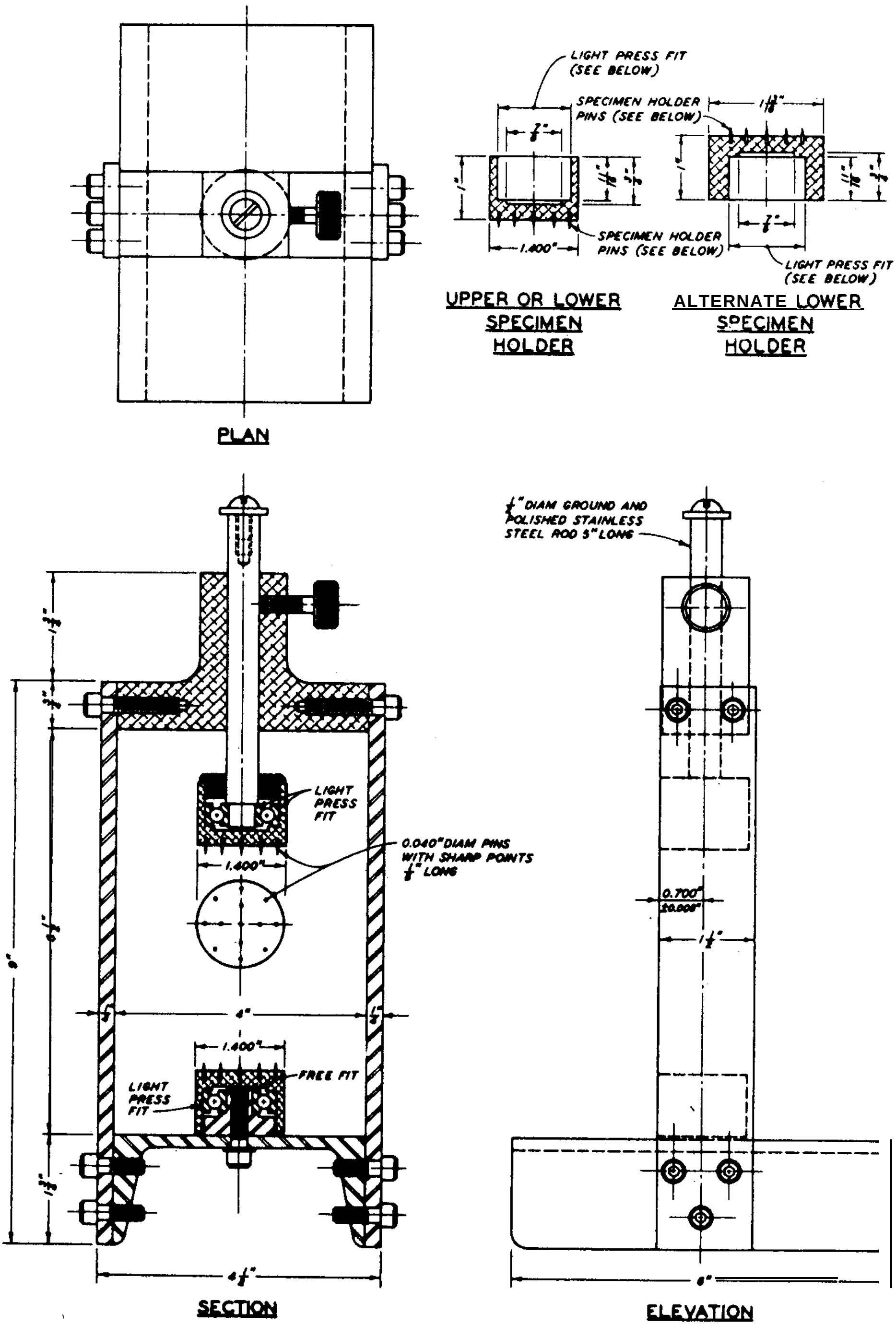


Figure 5. Details of a trimming frame for preparing 1.4-in.-diameter specimens

equipment for compacting the material are necessary.

(3) Soils containing gravel. Large-size forming jackets, the dimensions of which will depend on specimen size requirements subsequently described, are necessary for preparing specimens of material containing gravel. Special compacting equipment is also necessary for such soils, depending on the type of soil and the procedures used.

f. Equipment for Using Back Pressure to Saturate Specimens. Special equipment required for saturating specimens by using back pressures is described in paragraph 6a.

g. Miscellaneous Equipment. Other items of equipment needed for the triaxial compression tests are as follows:

(1) Membrane stretcher. A cylindrical tube, larger in diameter than the soil specimen, which has a tube connected to its side for application of a vacuum. Details of a membrane stretcher for 1.4-in.-diameter specimens are shown in Figure 6.

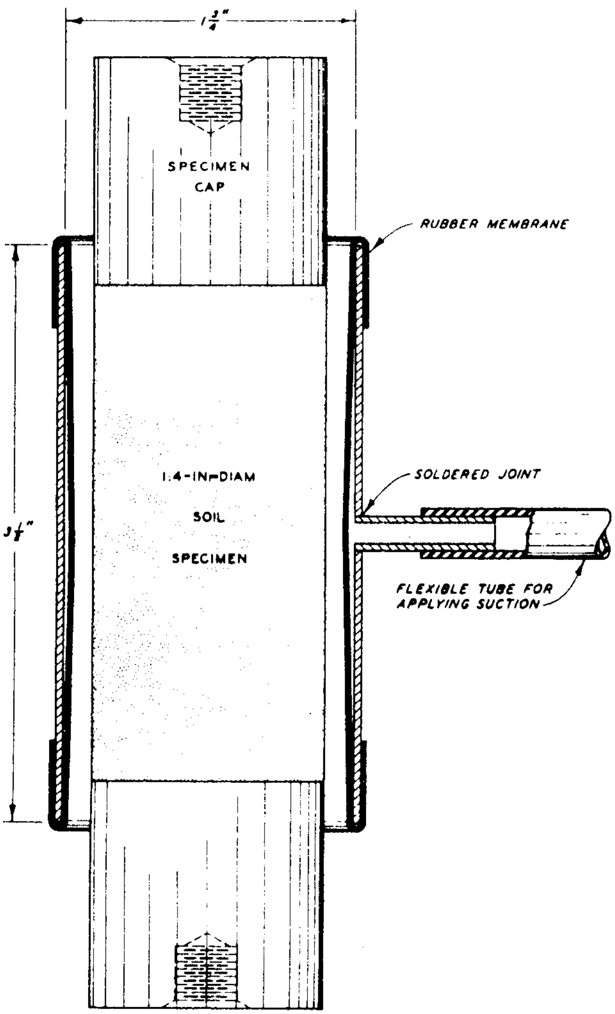


Figure 6. Details of a membrane stretcher for 1.4-in.-diameter specimens

(2) Pressure reservoir, generally a metal tank. The reservoir is filled with the fluid (usually deaired water) for applying the chamber pressure and is provided with a pressure regulator and a Bourdon gage. The

regulator should be capable of controlling pressures to within ±1/2 percent, though more precise methods of controlling and maintaining chamber pressures are required for tests of long duration.

(3) Measuring equipment, such as dial indicators and calipers. Precise instruments should be used for measuring the dimensions of a specimen with the desired accuracy.

(4) Deaired water, distilled or demineralized.

(5) Vacuum and air pressure supply.

(6) Bourdon gages of various sizes and capacities.

(7) A timing device, either a watch or clock with second hand.

(8) Balances, sensitive to 0.01 g and to 0.1 g.

(9) Apparatus, necessary to determine water content and specific gravity (see Appendices I, WATER CONTENT - GENERAL, and IV, SPECIFIC GRAVITY).

4. PREPARATION OF SPECIMENS. Specimens shall have an initial height of not less than 2.1 times the initial diameter, though the minimum initial height of a specimen must be 2.25 times the diameter if the soil contains particles retained on the No. 4 sieve. The maximum particle size permitted in any specimen shall be no greater than one-sixth of the specimen diameter. Triaxial specimens 1.4, 2.8, 4, 6, 12, and 15 in. in diameter are most commonly used.

a. Cohesive Soils Containing Negligible Amounts of Gravel. Specimens 1.4 in. in diameter are generally satisfactory for testing cohesive soils containing a negligible amount of gravel, while specimens of larger diameter may be advisable for undisturbed soils having marked stratification, fissures, or other discontinuities. Depending on the type of sample, specimens shall be prepared by either of the following procedures:

(1) Trimming specimens of cohesive soil. A sample that is uniform in character and sufficient in amount to provide a minimum of three specimens is required. For undisturbed soils, samples about 5 in. in diameter are preferred for triaxial tests using 1.4-in.-diameter specimens.

Specimens shall be prepared in a humid room and tested as soon as possible thereafter to prevent evaporation of moisture. Extreme care shall be taken in preparing the specimens to preclude the least possible disturbance to the structure of the soil. The specimens shall be prepared as follows:

(a) Cut a section of suitable length from the sample. As a rule, the specimens should be cut with the long axes parallel to the long axis of the sample; any influence of stratification is commonly disregarded. However, comparative tests can be made, if necessary, to determine the effects of stratification. When a 5-in.-diameter undisturbed sample is to be used for 1.4-in.-diameter specimens, cut the sample axially into quadrants using a wire saw or other convenient cutting tool. Use three of the quadrants for specimens; seal the fourth quadrant in wax and preserve it for a possible check test.

(b) Carefully trim each specimen to the required diameter, using a trimming frame or similar equipment (see Fig. 7). Use one side of the trimming frame for

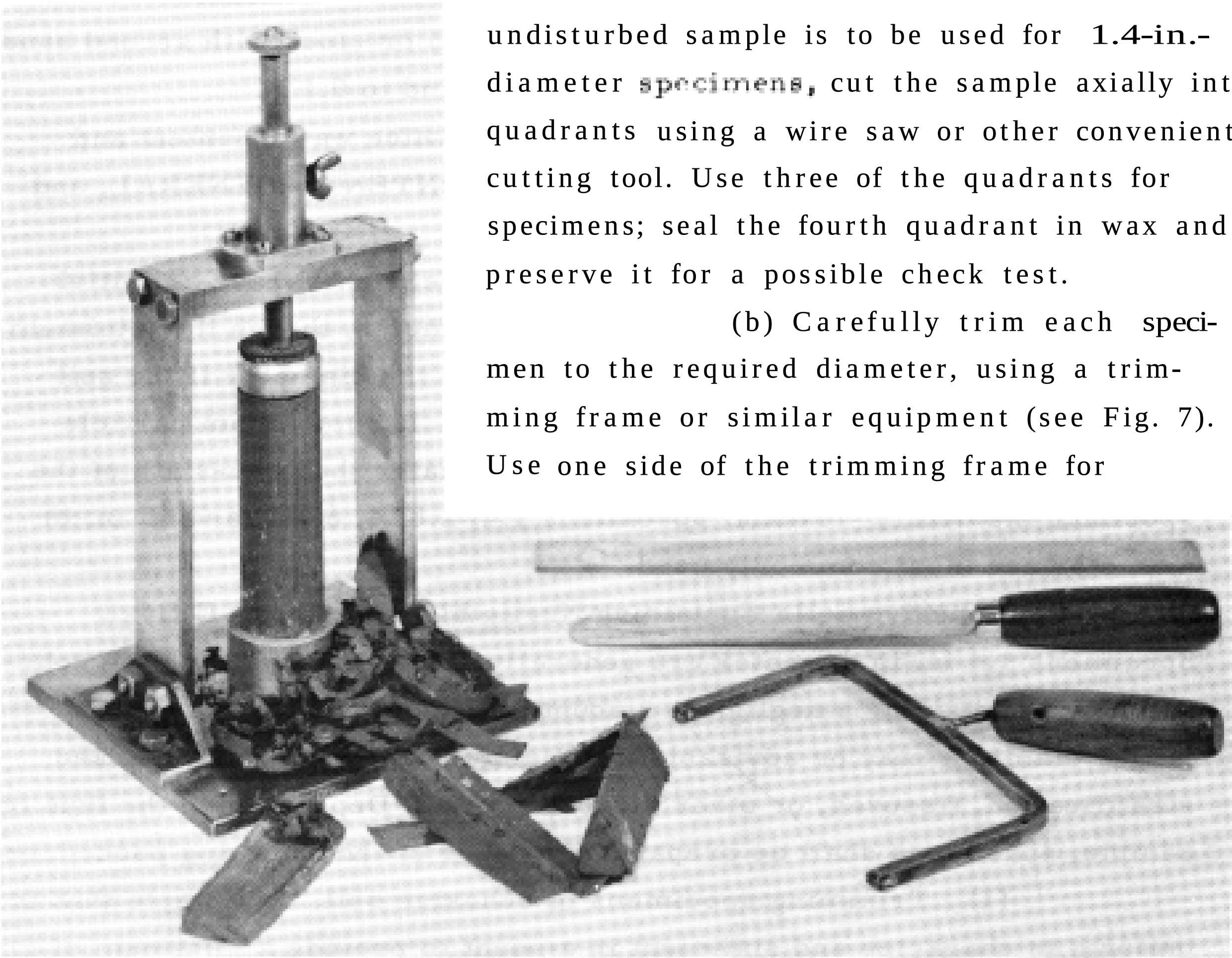


Figure 7. Prepared triaxial specimen, trimming frame, and cutting tools

preliminary cutting, and the other side for final trimming. A specimen after trimming is also shown in Figure 7. Ordinarily, the specimen is trimmed by pressing the wire saw or trimming knife against the edges of the frame and cutting from top to bottom. In trimming stiff or varved clays, move the wire saw from the top and bottom toward the middle of the specimen to prevent breaking off pieces at the ends. Remove any small shells or pebbles encountered during the trimming operations. Carefully fill voids on the surface of the specimen with remolded soil obtained from the trimmings. Cut specimen to the required length (usually 3 to 3-1/2 in. for 1.4-in. -diameter specimens and 6 to 7 in. for 2.8-in. -diameter specimens) using a miter box, as shown in Figure 8

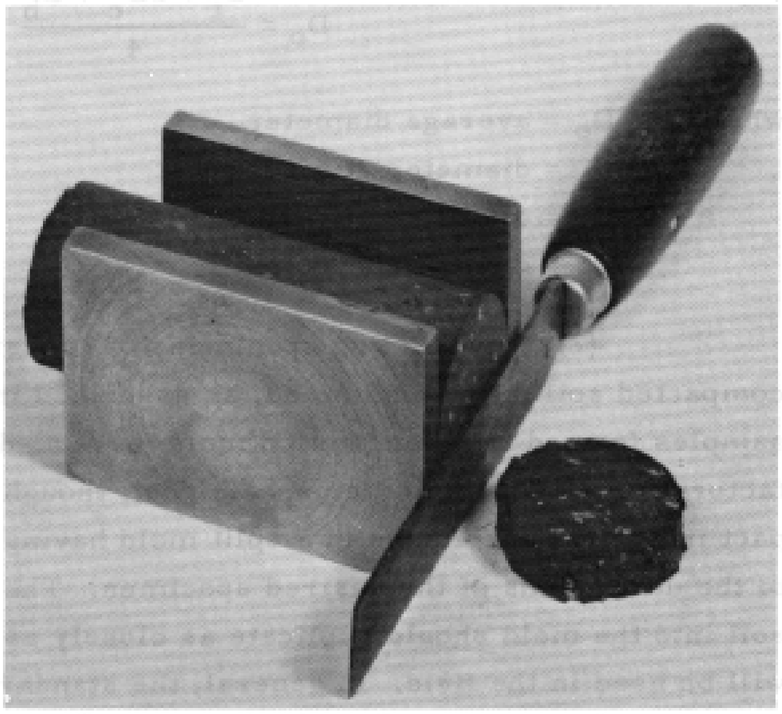


Figure 8. Squaring ends of specimen with a miter box

(c) From the soil trimmings, obtain 200 g of material for specific gravity and water content determinations (see Appendixes I, WATER CONTENT - GENERAL, and IV, SPECIFIC GRAVITY).

(d) Weigh the specimen to an accuracy of ±0.01 g for 1.4-in. -diameter specimens and ±0.1 g for 2.8-in.-diameter specimens.

(e) Measure the height and diameter of the specimen to an accuracy of ±0.01 in. Specimen dimensions based on measurements of the trimming frame guides and miter box length are not sufficiently accurate. The average height, H_o, of the specimen should be determined from at least four measurements, while the average diameter should be determined from measurements at the top, center, and bottom of the specimen, as follows:

$$D_o = \frac{D_t + 2D_c + D_b}{4}$$

where D_o = average diameter
D_t = diameter at top
D_c = diameter at center
D_b = diameter at bottom

(2) Compacting specimens of cohesive soil. Specimens of compacted soil may be trimmed, as described in paragraph 4a(1), from samples formed in a compaction mold (a 4-in.-diameter sample is satisfactory for 1.4-in.-diameter specimens), though it is preferable to compact individual specimens in a split mold having inside dimensions equal to the dimensions of the desired specimen. The method of compacting the soil into the mold should duplicate as closely as possible the method that will be used in the field. In general, the standard impact type of compaction (see Appendix VI, COMPACTION TESTS) will not produce the same soil structure and stress-deformation characteristics as the kneading action of the field compaction equipment. Therefore, the soil should preferably be compacted into the mold (whether a specimen-size or a standard compaction mold) in at least six layers, using a pressing or kneading action of a tamper having an area in contact with the soil of less than one-sixth the area of the mold, and thoroughly scarifying the surface of each layer before placing the next. The sample shall be prepared according to paragraph 2b of Appendix VI, COMPACTION TESTS, thoroughly mixed with sufficient water to produce the desired water content, and then

stored in an airtight container for at least 16 hr. The desired density may be produced by either (1) kneading or tamping each layer until the accumulative weight of soil placed in the mold is compacted to a known volume or (2) adjusting the number of layers, the number of tamps per layer, and the force per tamp. For the latter method of control, special constant-force tampers (such as the Harvard miniature compactor for 1.4-in.-diameter specimens† or similar compactors for 2.8-in.-diameter and larger speciments‡) are necessary. After each specimen compacted to finished dimensions has been removed from the mold, proceed in accordance with steps (c) through (e) of paragraph 4a(1).

b. Cohesionless Soils Containing Negligible Amounts of Gravel. Soils which possess little or no cohesion are difficult if not impossible to trim into a specimen. If undisturbed samples of such materials are available in sampling tubes, satisfactory specimens can usually be obtained by freezing the sample to permit cutting out suitable specimens. Samples should be drained before freezing. The frozen specimens are placed in the triaxial chamber, allowed to thaw after application of the chamber pressure, and then tested as desired. Some slight disturbance probably occurs as a result of the freezing, but the natural stratification and structure of the material are retained. In most cases, however, it is permissible to test cohesionless soils in the remolded state by forming the specimen at the desired density or at a series of densities which will permit interpolation to the desired density. Specimens prepared in this

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† A. Casagrande, J. M. Corso, and S. D. Wilson, Report to Waterways Experiment Station on the 1949-1950 Program of Investigation of Effect of Long-Time Loading on the Strength of Clays and Shales at Constant Water Content, Harvard University (Cambridge, Mass., July 1950).

‡ A. Casagrande and R. C. Hirschfeld, Second Progress Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays, Soil Mechanics Series No. 65, Harvard University (Cambridge, Mass., April 1962).

manner should generally be 2.8 in. in diameter or larger, depending on the maximum particle size. The procedure for forming the test specimen shall consist of the following steps:

(1) Oven-dry and weigh an amount of material sufficient to provide somewhat more than the desired volume of specimen.

(2) Place the forming jacket, with the membrane inside, over the specimen base of the triaxial compression device.

(3) Evacuate the air between the membrane and the inside face of the forming jacket.

(4) After mixing the dried material to avoid segregation, place the specimen, by means of a funnel or the special spoon, inside the forming jacket in equal layers. For 2.8-in.-diameter specimens, 10 layers of equal thickness are adequate. Starting with the bottom layer, compact each layer by blows with a tamping hammer, increasing the number of blows per layer linearly with the height of the layer above the bottom layer.† The total number of blows required for a specimen of a given material will depend on the density desired. Considerable experience is usually required to establish the proper procedure for compacting a material to a desired uniform density by this method. A specimen formed properly in the above-specified manner, when confined and axially loaded, will deform symmetrically with respect to its midheight, indicating that a uniform density has been obtained along the height of the specimen.

(5) As an alternate procedure, the entire specimen may be placed in a loose condition by means of a funnel or special spoon. The desired density may then be achieved by vibrating the specimen in the forming jacket to obtain a specimen of predetermined height and corresponding density. A specimen formed properly in this manner, when

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† Liang-Sheng Chen, "An investigation of stress strain and strength characteristics of cohesionless soils by triaxial compression tests," Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering, vol. V (Rotterdam, 1948), pp. 35-43.

confined and axially loaded, will deform symmetrically with respect to its midheight.

(6) Subtract weight of unused material from original weight of the sample to obtain weight of material in the specimen.

(7) After the forming jacket is filled to the desired height, place the specimen cap on the top of the specimen, roll the ends of the, membrane over the specimen cap and base, and fasten the ends with rubber bands or O-rings. Apply a low vacuum to the specimen through the base and remove the forming jacket.

(8) Measure height and diameter as specified in paragraph 4a(1)(e).

c. Soils Containing Gravel. The size of specimens containing appreciable amounts of gravel is governed by the requirements of paragraph4. If the material to be tested is in an undisturbed state, the specimens shall be prepared according to the applicable requirements of paragraphs 4a and 4b, with the size of specimen based on anestimate of the largest particle size. In testing compacted soils, the largest particle size is usuallyknown, and the entire sample should be tested, whenever possible, without removing any of the coarser particles. However, it may be necessaryto remove the particles larger than a certain size to comply with the requirements for specimen size, though such practice will result in lower measured values of the shear strength and should be avoided if possible. Oversize particles should be removed and, if comprising more than 10 percent by weight of the sample, be replaced by anequalpercentage by weight of material retained on the No. 4 sieve and passing the maximum allowable sieve size. The percentage of material finer than the No. 4 sieve thus remains constant (see paragraph 2b of Appendix VI, COMPACTION TESTS). It will generally be necessary to prepare compacted samples of material containing gravel inside a forming jacket placed on the specimen base. If the material is cohesionless, it should be oven-dried and compacted in layers inside the membrane and forming jacket using the procedure in paragraph 4b as a guide. When specimens of very high density are required, the samples should be compacted preferably by vibration to avoid rupturing the membrane. The use of two membranes will provide additional insurance against possible leakage during the test as a result of membrane rupture. If the sample contains a significant amount of fine-grained material, the soil usually must possess the proper water content before it can be compacted to the desired density. Then, a special split compaction mold is used for forming the specimen. The inside dimensions of the mold are equal to the dimensions of the triaxial specimen desired. No membrane is used inside the mold, as the membrane can be readily placed over the compacted specimen after it is removed from the split mold. The specimen should be compacted to the desired density in accordance with paragraph 4a(2).

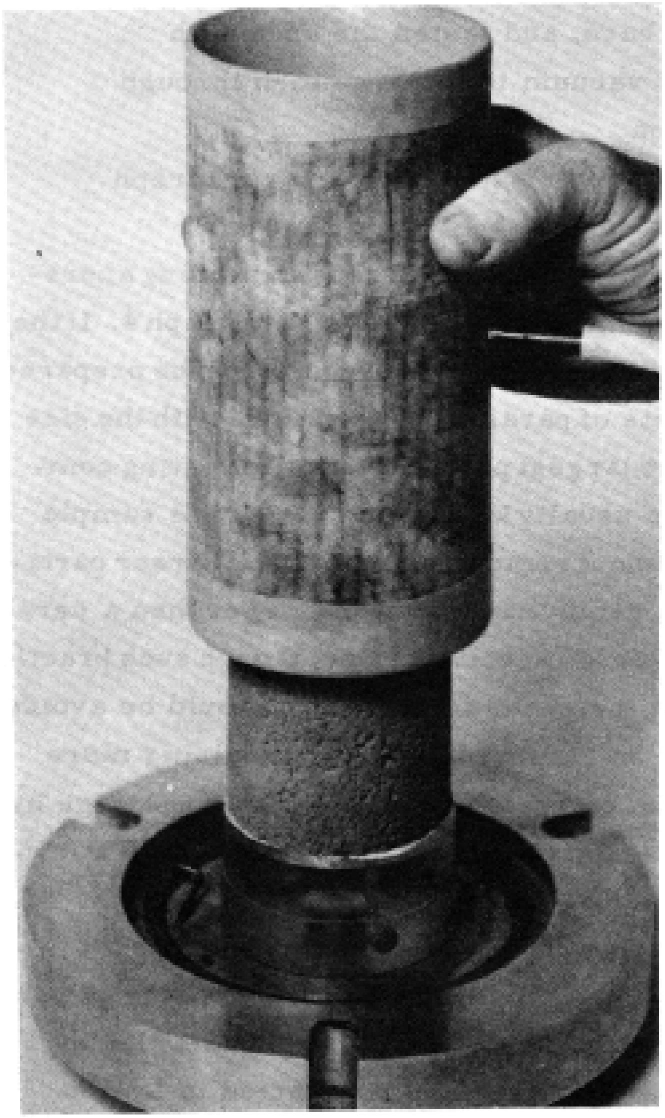

Figure 9. Placing rubber membrane over a 2.8-in.-diameter specimen using a membrane stretcher

5. Q TEST. a. Procedure. The procedure for the Q test shall consist of the following steps:

(1) Record all identifying information for the sample project number or name, boring number, and other pertinent data, on a data sheet (see Plate X-1).

(2) Place one of the prepared specimens on the base.

(3) Place a rubber membrane (see Fig. 9) in the

membrane stretcher, turn both ends of the membrane over the ends of the stretcher, and apply a vacuum to the stretcher. Carefully lower the stretcher and membrane over the specimen as shown in Figure 9. Place the specimen cap on the top of the specimen and release the vacuum on the membrane stretcher. Turn the ends of the membrane down around the base and up around the specimen cap and fasten the ends with O-rings or rubber bands. With 1.4-in. -diameter specimens of relatively insensitive soils, it is easier to roll the membrane over the specimen as shown in Figure 10.

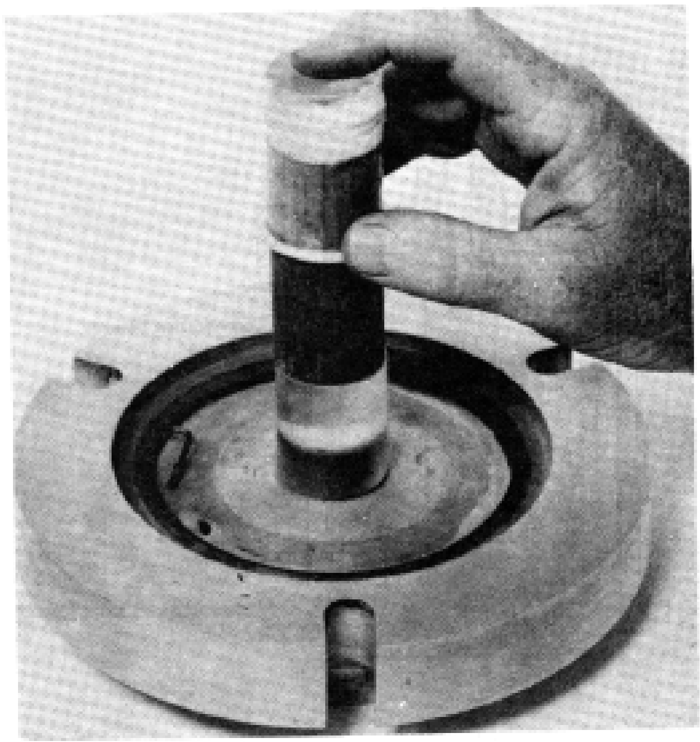

Figure 10. Rolling rubber membrane over a 1.4-in.-diameter specimen

(4) Assemble the triaxial chamber and place it in position in the loading device. Connect the tube from the pressure reservoir to the base of the triaxial chamber. With valve C (see Figure 11) on the pressure reservoir closed and valves A and B open, increase the pressure inside the reservoir and allow the pressure fluid to fill the triaxial chamber. Allow a few drops of the pressure fluid to escape through the vent valve (valve B) to insure complete filling of the chamber with fluid. Close valve A and the vent valve.

(5) With valves A and C closed, adjust the pressure regulator to preset the desired chamber pressure. The range of chamber pressures for the three specimens will depend on the loadings expected

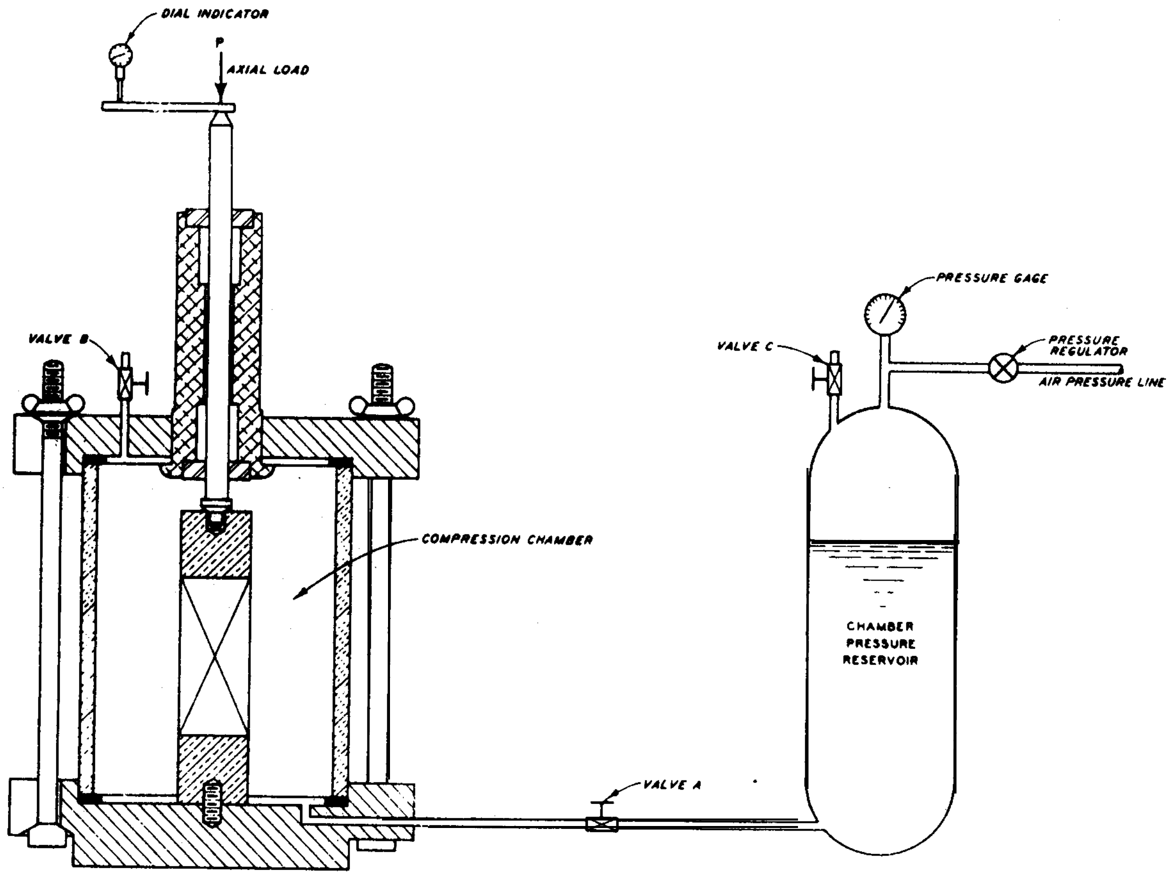

Figure 11. Schematic diagram of triaxial compression apparatus for Q test

in the field. The maximum confining pressure should be at least equal to the maximum normal load expected in the field in order that the shear strength data need not be extrapolated for use in design analysis. Record the chamber pressure on data sheets (Plates X-1 and X-2). Now open valve A and apply the preset pressure to the chamber. Application of the chamber pressure will force the piston upward into contact with the ram of the loading device. This upward force is equal to the chamber pressure acting on the cross-sectional area of the piston minus the weight of the piston minus piston friction.

(6) Start the test with the piston approximately 0.1 in. above the specimen cap. This allows compensation for the effects of piston friction, exclusive of that which may later develop as a result of lateral forces. Set the load indicator to zero when the piston comes into contact with the specimen cap. In this manner the upward thrust of the chamber pressure on the piston is also eliminated from further consideration. Contact of the piston with the specimen cap is indicated by a slight movement of the load indicator. Set the strain indicator and record on the data sheet (Plate X-2) the initial dial reading at contact. Axially strain the specimen at a rate of about 1 percent per minute (for plastic materials) and about 0.3 percent per minute (for brittle materials that achieve maximum deviator stress at about 3 to 6 percent strain); at these rates the elapsed time to reach maximum deviator stress would be about 15 to 20 min.

(7) Observe and record the resulting load at every 0.3 percent strain for about the first 3 percent and, thereafter, at every 1 percent, or for large strains, at every 2 percent strain; sufficient readings should be taken to completely define the shape of the stress-strain curve so frequent readings may be necessary as failure is approached. Continue the test until an axial strain of 15 percent has been reached, as shown in Figures 12a, 12b, and 12d; however, when the deviator stress decreases after attaining a maximum value and is continuing to decrease at 15 percent strain (Fig. 12c), the test shall be continued to 20 percent strain.

(8) For brittle soils (i.e., those in which maximum deviator stress is reached at 6 percent axial strain or less), tests should be performed at rates of strain sufficient to produce times to failure as set forth in paragraph 5a(6) above; however, when the maximum deviator stress has been clearly defined, the rate may be increased such that the remainder of the test is completed in the same length of time as that taken to reach maximum deviator stress. However, for each group of tests about 20 percent of the samples should be tested at the rates set forth

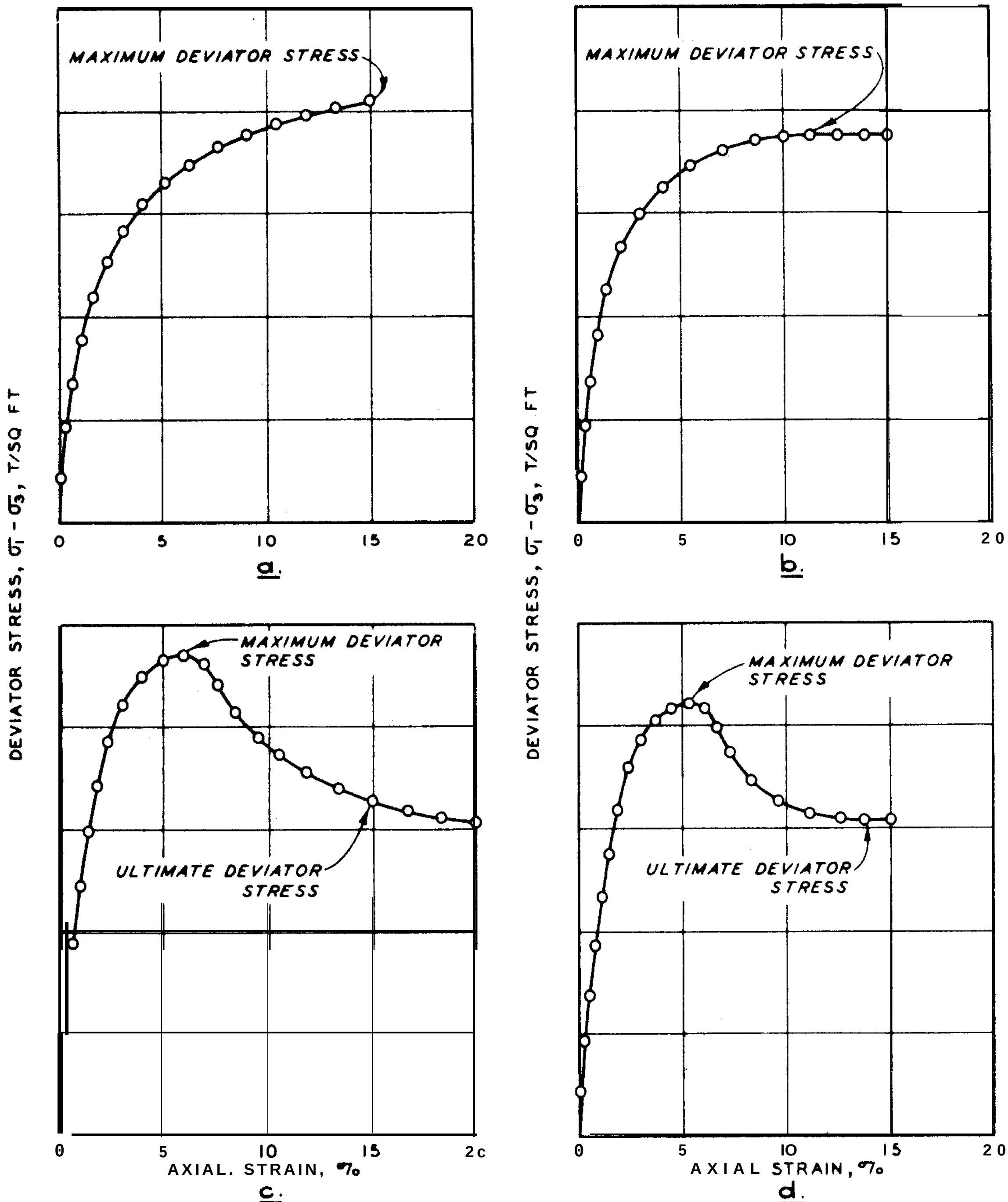

Figure 12. Examples of stress-strain curves

in paragraphs 5a(6) and 5a(7) above.

(9) Upon completion of axial loading, release the chamber pressure by shutting off the air supply with the regulator and opening valve C. Open valve B and draw the pressure fluid back into the pressure reservoir by applying a low vacuum at valve C. Dismantle the triaxial chamber. Make a sketch of the specimen, showing the mode of failure.

(10) Remove the membrane from the specimen For 1.4-in.-diameter specimens, carefully blot any excess moisture from the surface of the specimen and determine the water content of the whole

specimen (see Appendix I, WATER CONTENT - GENERAL). For 2.8-in.-diameter or larger specimens, it is permissible to use a representative portion of the specimen for the water content determination. It is essential that the final water content be determined accurately, and weighings should be verified, preferably by a different technician.

(11) Repeat the test on the two remaining specimens at different chamber pressures, though using the same rate of strain.

b. Computations. The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Plate X-1) the initial water content (see Appendix I, WATER CONTENT - GENERAL), volume of solids, initial void ratio, initial degree of saturation, and initial dry density, using the formulas given in Appendix II, UNIT WEIGHTS, VOID RATIO, POROSITY, AND DEGREE OF SATURATION.

(2) Compute and record on the data sheet (Plate X-2) the axial strain, the corrected area, and the deviator stress at each increment of strain, using the following formulas:

$$\text{Axial strain, } \varepsilon = \frac{AH}{H_o}$$

$$\text{Corrected area of specimen, } A_{corr}\text{, sq cm} = \frac{A_o}{2 - \varepsilon}$$

$$\text{Deviator stress, tons per sq ft} = \frac{P}{A_{corr}} \times 0.465$$

where AH = change in height of specimen during test, cm

H_o = initial height of specimen, cm, (Where a significant decrease in specimen volume occurs upon application of the chamber pressure, as in partially saturated soils, the height of the specimen after application of the chamber pressure should be used rather than the initial height.)

A_o = initial area of specimen, sq cm

P = net applied axial load, lb (the actual load applied to specimen after correction for piston friction and for the upward thrust of the fluid pressure in the triaxial chamber)

(3) Record the time to failure on the data sheet (Plate X-2).

(4) The rubber membrane increases the apparent strength of the specimen. Investigations† with specimens 1.5 in. in diameter and membranes 0.008 in. thick, for instance, indicate the increase in deviator stress to be 0.6 psi at 15 percent axial strain. The correction, σ_r, to be made to the measured deviator stress for the effect of the rubber membrane is computed as follows:

$$\sigma_r = \frac{\pi D_o M \, \varepsilon(1 - \varepsilon)}{A_o}$$

where D_o = initial diameter of specimen

M = compression modulus of the rubber membrane

ε = axial strain

A_o = initial cross-sectional area of the specimen

The compression modulus may, without great error, be assumed to be equal to that measured in extension. An apparatus for determining the extension modulus of rubber is described in another work.† In tests of very soft soils the membrane effect may be significant, and in these tests it is advisable to compute or estimate the correction and deduct it from the maximum deviator stress. For most soils tested using membranes of standard thickness, the correction is insignificant and can be ignored.

c. Presentation of Results. The results of the Q test shall be recorded on the report form shown as Plate X-3. Enter pertinent information regarding the condition of the specimen or method of preparing the

† Bishop and Henkel, op. cit., pp. 167-171.

specimen under "Remarks." Plot the deviator stress versus the axial strain for each of the specimens as shown in Figure 12. The peak or maximum deviator stress represents "failure" of the specimen; when the deviator stress increases continuously during the test, the deviator stress at 15 percent axial strain shall be considered the maximum deviator stress. When the deviator stress decreases after reaching a maximum, the minimum deviator stress attained before 15 percent axial strain shall be considered the ultimate deviator stress, as shown in Figures 12c and 12d. Construct Mohr stress circles on an arithmetic plot with shear stresses as ordinates and normal stresses as abscissas. As shown in Figure 13, the applied principal stresses, σ_1 and σ_3, are plotted on the abscissa, and the Mohr circles are drawn with radii of one-half the maximum deviator stresses $\left(\frac{\sigma_1 - \sigma_3}{2}\right)$ and with their centers at values equal to one-half the sums of the major and minor principal stresses $\left(\frac{\sigma_1 + \sigma_3}{2}\right)$. Plot a Mohr circle, or a sufficient segment thereof, for each specimen in

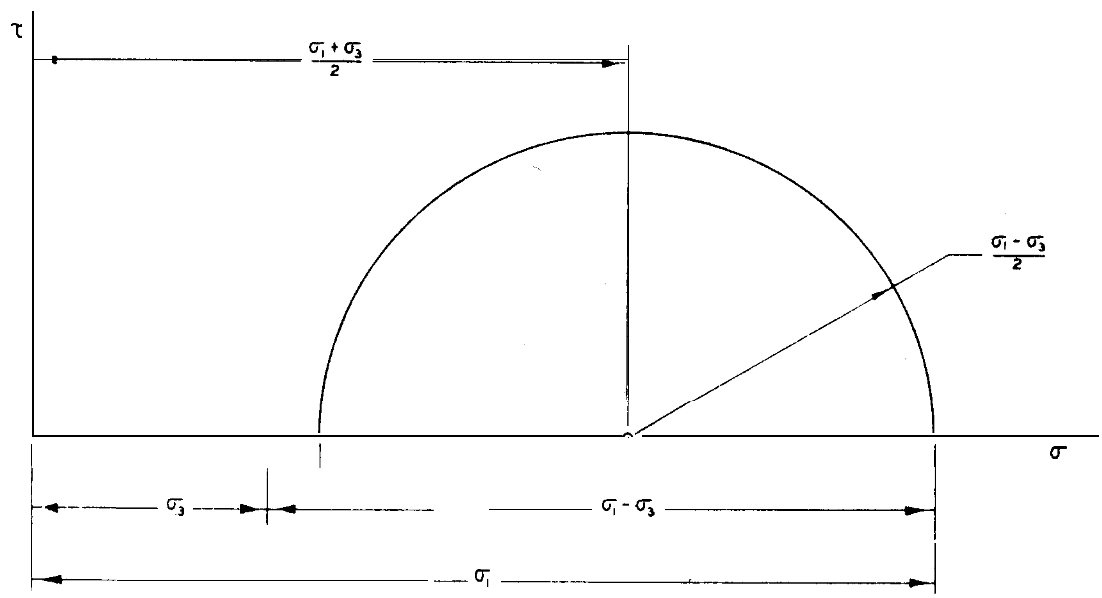

Figure 13. Construction of Mohr's circle of stress

the graph in the upper right corner of report form. A sketch of each specimen after failure should be shown above the Mohr circles (Plate X-3). The following procedures should be followed in drawing strength envelopes:

(1) Undisturbed specimens. For undisturbed specimens, the strength envelope should be drawn tangent to the Mohr circles. Q tests of saturated soils usually indicate a strength envelope that is parallel to the abscissa as shown in Figure 14a, so the angle of internal friction is usually equal to zero. Strength envelopes indicated by Q tests on partially saturated soils are usually curved as shown in Figure 14b, particularly for the lower normal stresses. When the curvature is pronounced, the shear strength parameters ϕ and c are not constants.

(2) Compacted specimens. For compacted specimens, the strength envelope should be drawn through points on the Mohr circles representing stresses on the failure plane as shown in Figure 14c.

6. Q TEST WITH BACK-PRESSURE SATURATION. In cases where a foundation soil exists that is partially saturated during exploration but which will become completely saturated without significant volume change either before or during construction, it is necessary to saturate Q test specimens, using back pressure, before they are sheared. Such field conditions may occur when, due to heavy rains or other reasons, the water table is raised above the level that existed during initial sampling: Construction of cofferdams, river diversions, and closure sections, or percolation of rainwaters can also create conditions that increase foundation water contents after exploration but before embankment construction and subsequent consolidation of the foundation.

For the Q test with back-pressure saturation, the apparatus should be set up similar to that shown in Figure 16 (page X-30). Filter strips should not be used and as little volume change as possible should be permitted during the test. After completing the steps outlined in paragraphs 4 and 5a(1) through 5a(4) (note that the procedures for attaching the membrane to the cap and base and for deairing the drainage lines are similar

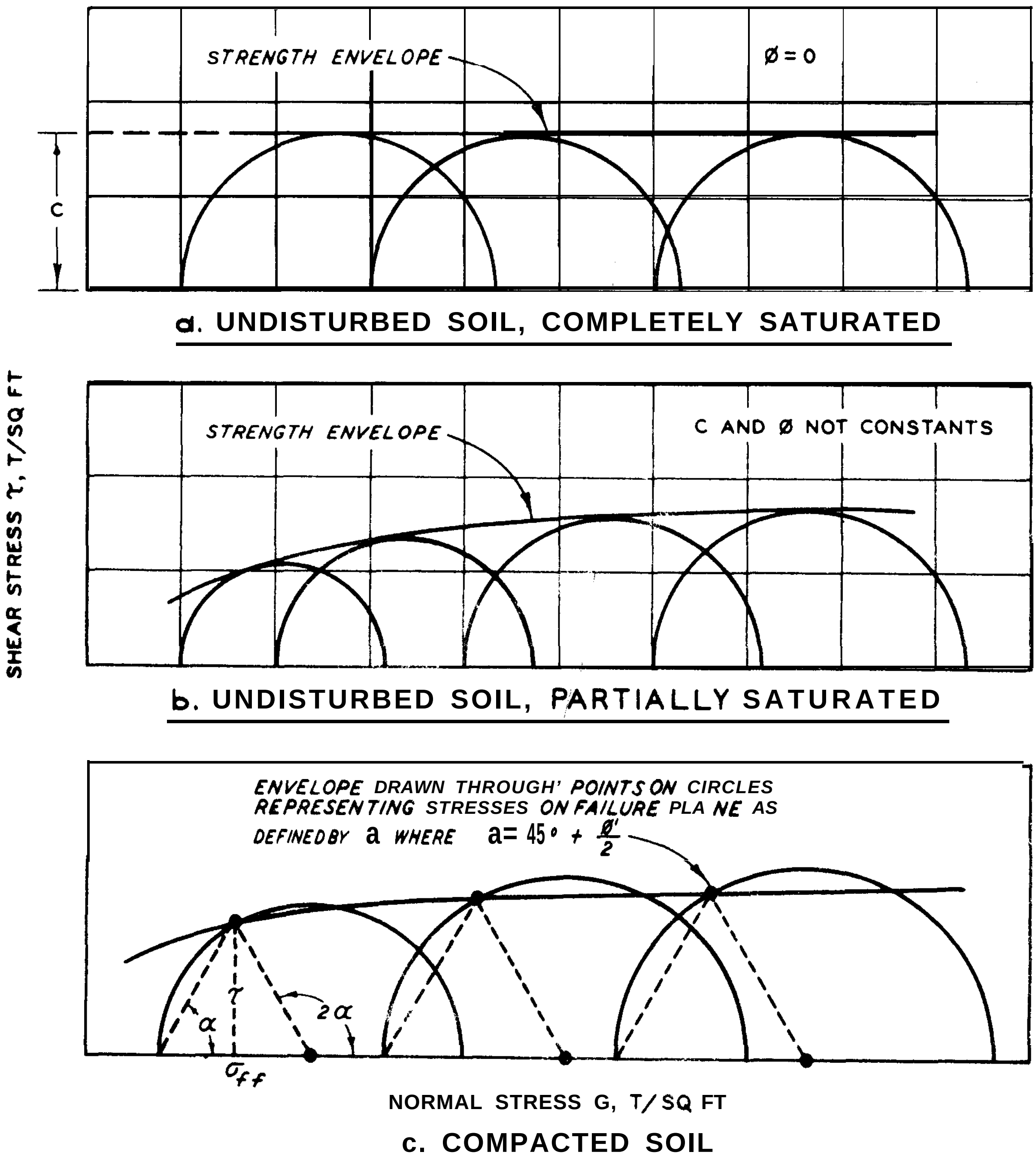

Figure 14. Examples of strength envelopes for Q tests

to those used in the R test), apply 3-psi chamber pressure to the specimen with all drainage valves closed. Allow a minimum of 30 min for stabilization of the specimen pore water pressure, measure AH, and begin back-pressure procedures as given in paragraphs 7b(2) through 7b(5). After verification of saturation, and remeasurement of AH, close all drainage lines leading to the back pressure and pore water measurement apparatus. Holding the maximum applied back pressure constant, increase the chamber pressure until the difference ‘between the chamber pressure and the back pressure equals the desired effective confining pressure (see paragraph 5a(5)). Then proceed as outlined in paragraphs 5a(6) through 5a(11).

7. R TEST. All specimens must be completely saturated before application of the deviator stress in the R test. A degree of saturation over 98 percent can be considered to represent a condition of essentially complete saturation; if pore water pressures are to be measured during shear, however, the specimens must be 100 percent saturated. Computations of

the degree of saturation based on changes of volume and water content are often imprecise, so complete saturation of a specimen should be assumed only when an increase of the chamber fluid pressure will cause an immediate and equal increase of pressure in the pore water of the specimen. In general, it is preferable to saturate the soil after the specimens have been prepared, encased in membranes, and placed within the compression chamber, using back pressure. A back pressure is an artificial increase of the pore water pressure which will increase the degree of saturation of a specimen by forcing pockets of air into solution in the pore water. The back pressure is applied to the pore water simultaneously with an equal increase of the chamber pressure so that the effective stress acting on the soil skeleton is not changed. In other words, the pressure differential across the membrane remains constant during the back pressure saturation phase. Thus, when the back pressure is increased sufficiently slowly to avoid an excessive pressure differential within the specimen itself, the degree of saturation will be increased while the volume of the specimen is maintained essentially constant. Figure 15 gives the back pressure theoretically required to produce a desired increase in saturation if there is no change in specimen volume. It is important to note that the relation shown in Figure 15 is based on an assumption that the water entering the specimen contains no dissolved air.

a. Apparatus. In addition to the apparatus described in paragraphs 3a through 3g, the following equipment are necessary for R tests utilizing back pressure for saturation:

(1) Air reservoir and regulator for controlling the back pressure, similar to those used to control the chamber pressure.

(2) Bourdon gage attached to the back pressure reservoir to measure the applied back pressure. As relatively large back pressures and chamber pressures are sometimes required, it is essential that these two pressures be measured accurately to insure that the precise

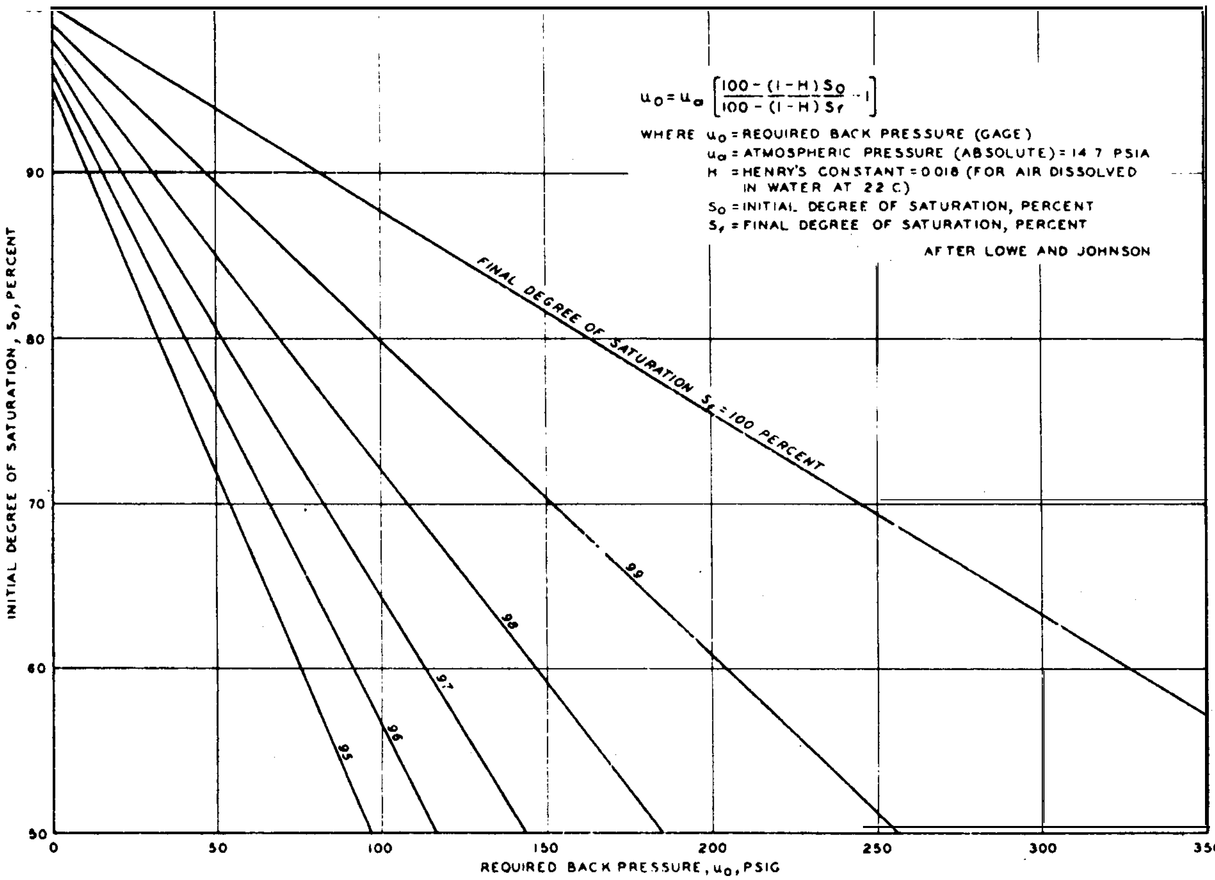

Figure 15. Back pressure required to attain various degrees of saturation

difference between them is known. A differential pressure gage† will permit this difference to be measured directly.

(3) Calibrated burette or standpipe capable of measuring volume changes to within 0.1 cc for 1.4-in.-diameter specimens, 0.5 cc for 2.8-in.-diameter specimens, and 1 cc for 6-in.-diameter specimens. This burette is connected in the back pressure line leading to the top of

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† John Lowe, III, and Thaddeus C. Johnson, "Use of back pressure to increase degree of saturation of triaxial test specimens," ASCE Research Conference on Shear Strength of Cohesive Soils, University of Colorado (Boulder, Colo., June 1960).

the specimen to measure the volume of water added to the specimen during saturation and volume changes of the specimen during consolidation. If the water added to the specimen becomes saturated with air, a higher back pressure will be required than that given in Figure 15. Therefore, precautions should be taken to minimize aeration of the saturation water by reducing the area of the air-water interface or by separating the air and water with a rolling rubber diaphragm.* A relatively long (over 6-foot) length of thick-walled, small-bore tubing between the burette and the specimen will also reduce the amount of air entering the specimen. Adequate safety precuations should be taken against breakage of the burette under high pressures.

(4) Electrical pressure transducer or no-flow indicator with which the pressure of the pore water at the bottom of the specimen can be measured without allowing a significant flow of water from the specimen. This is an extremely difficult measurement to make since even a minute flow of water will reduce the pressure in the pore water; yet the measuring device must be sensitive enough to detect small changes in pressure. Electrical pressure transducers, while relatively expensive, offer almost complete protection against flow, are simple to operate, and lend themselves to the automatic recording of test data. Several types of manually balanced pressure-measuring systems employing a no-flow indicator are being used successfully,‡ though a full discussion of their relative merits

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* H. B. Seed, J. K. Mitchell, and C. K. Chan, "The strength of compacted cohesive soils," ASCE Research Conference on Shear Strength of Cohesive Soils, University of Colorado (Boulder, Colo., June 1960).

‡ Bishop and Henkel, op. cit., pp. 52-63, 206-207.

A. Andersen, L. Bjerrum, E. DiBiagio, and B. Kjaernsli, Triaxial Equipment Developed at the Norwegian Geotechnical Institute, Publication No. 21, Norwegian Geotechnical Institute (Oslo, 1957).

A. Casagrande and R.C. Hirschfeld, First Progress Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays, Soil Mechanics Series No. 61, Harvard University (Cambridge, Mass., May 1960).

and shortcomings is not possible here.

b. Procedure. The procedure for the R test utilizing back pressure for saturation shall consist of the following steps:

(1) Proceed as outlined in paragraphs 5a(1) through 5a(4), with the exception that specimen bases and caps with porous inserts and drainage connections should be used and back pressure equipment should be included as shown in Figure 16. Saturated strips of filter paper (such as Whatman's No. 54) placed beneath the membrane and extending from the base along three-fourths of the specimen length will reduce the time required for saturation and consolidation. These strips must neither overlap and form a continuous circumferential coverage of the specimen

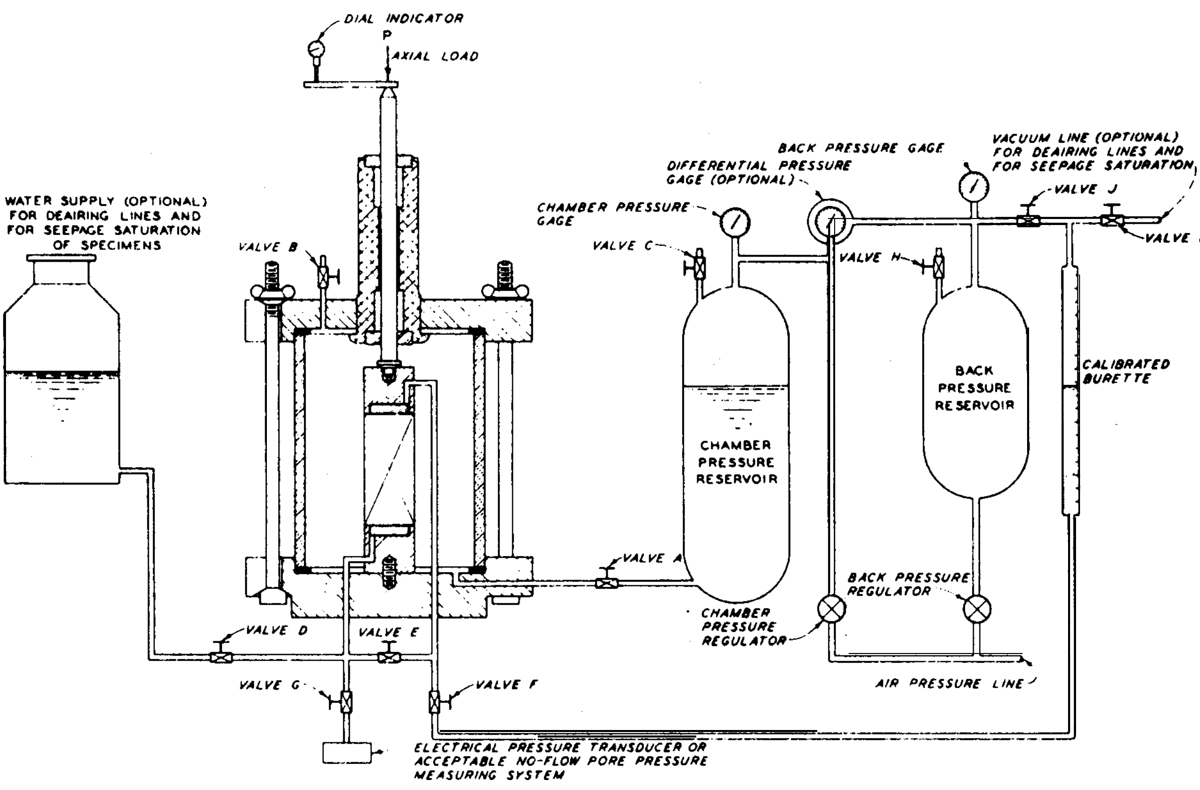

Figure 16. Schematic diagram of typical triaxial compression apparatus for R and S tests

nor form a continuous path between the base and the cap. Place saturated filter paper disks having the same diameter as that of the specimen between the specimen and the base and cap; these disks will also facilitate removal of the specimen after the test. The drainage lines and the porous inserts should be completely saturated with deaired water. The drainage lines should be as short as possible and made of thick-walled, small--bore tubing to insure minimum elastic changes in volume due to changes in pressure. Valves in the drainage lines (valves E, F, and G in Figure 16) should preferably be of a type which will cause no discernible change of internal volume when operated (such as the Teflon-packed ball valve made by the Whitey Research Tool Co.). While mounting the specimen in the compression chamber, care should be exercised to avoid entrapping any air beneath the membrane or between the specimen and the base and cap.

(2) Estimate the magnitude of the required back pressure by reference to Figure 15 or other theoretical relations. Specimens should be completely saturated before any appreciable consolidation is permitted, for ease and uniformity of saturation as well as to allow volume changes during consolidation to be measured with the burette; therefore, the difference between the chamber pressure and the back pressure should not exceed 5 psi during the saturation phase. To insure that a specimen is not prestressed during the saturation phase, the back pressure must be applied in small increments, with adequate time between increments to permit equalization of pore water pressure throughout the specimen.

(3) With all valves closed, adjust the pressure regulators to a chamber pressure of about 7 psi and a back pressure of about 2 psi. Record these pressures on the data sheet (Plate X-4). Now open valve A to apply the preset pressure to the chamber fluid and simultaneously open valve F to apply the back pressure through the specimen cap. Immediately open valve G and read and record the pore pressure at the specimen base. When the measured pore pressure becomes essentially

constant, close valves F and G† and record the burette reading.

(4) Using the technique described in step (3), increase the chamber pressure and the back pressure in increments, maintaining the back pressure at about 5 psi less than the chamber pressure. The size of each increment might be 5, 10, or even 20 psi, depending on the compressibility of the soil specimen and the magnitude of the desired consolidation pressure. Open valve G and measure the pore pressure at the base immediately upon application of each increment of back pressure and observe the pore pressure until it becomes essentially constant. The time required for stabilization of the pore pressure may range from a few minutes to several hours depending on the permeability of the soil. Continue adding increments of chamber pressure and back pressure until, under any increment, the pore pressure reading equals the applied back pressure immediately upon opening valve G.

(5) Verify the completeness of saturation by closing valve F and increasing the chamber pressure by about 5 psi. The specimen shall not be considered completely saturated unless the increase in pore pressure immediately equals the increase in chamber pressure.

(6) When the specimen is completely saturated, hold the maximum applied back pressure constant and increase the chamber pressure until the difference between the chamber pressure and the back pressure equals the desired consolidation pressure. Open valve F and permit the specimen to consolidate (or swell.) under the consolidation pressure. Valve E may be opened to allow drainage from both ends of the specimen. At increasing intervals of elapsed time (0.1, 0.2, 0.5, 1, 2, 4, 8, 15, and 30 min, 1, 2, 4, and 8 hr, etc.), observe and record (Plate X-5) the burette readings and, if practicable, the dial indicator readings (it may be necessary to force the piston down into contact with the specimen cap for each

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† If an electrical pressure transducer is used to measure the pore pressure, valve G may be safely left open during the entire saturation procedure.

reading). Plot the burette readings (and dial indicator readings, if taken) versus the logarithm of elapsed time, as shown in Figure 5 of Appendix VIII, CONSOLIDATION TEST. Allow consolidation to continue until a marked reduction in slope of the curve shows that 100 percent primary consolidation has been achieved.

(7) Close valve G, unless pore pressure measurements are to be made during shear, and valves E and F, and proceed according to paragraphs 5a(6) through 5a(10), except use a rate of strain for the R test of about 0.5 percent per minute (for plastic materials) and about 0.3 percent per minute or less for brittle materials that achieve a maximum deviator stress at about 3 to 6 percent strain; the strain rate used should result in a time to maximum deviator stress of approximately 30 min. Relatively pervious soils may be sheared in 15 min. These rates of strain do not permit equalization of induced pore pressure throughout the specimen and are too high to allow satisfactory pore pressure measurements to be made at the specimen ends during shear.† Therefore, these rates of strain are applicable only to R tests in which no pore pressure measurements are made during shear. Where pore pressure measurements are made at the ends of the specimens as in $\bar{R}$ tests, the time to reach maximum deviator stress should generally be at least 120 min; considerably longer time may be required for materials of low permeability. For brittle soils (i.e., those in which the maximum deviator stress is reached at 6 percent axial strain or less), after the maximum deviator stress has been clearly defined, the rate of strain may be increased so that the remainder of the test is completed in the same length of time as that taken to reach maximum deviator stress. However, for each group of tests in a given test program, at least 20 percent of the samples should be tested to final axial strain at rates of strain outlined in the first sentence of this paragraph.

c. Computations. The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Plate X-1) the initial water content, volume of solids, initial void

† Bishop and Henkel, op. cit., pp. 192-204.

ratio, initial degree of saturation, and initial dry density, using the formulas previously presented.

(2) Compute the cross-sectional area of the specimen after completion of consolidation according to the formula:†

$$\text{Area of specimen after consolidation, } A_c\text{, sq cm} = A_o \frac{H_o - 2\Delta H_o}{H_o}$$

or if the specimen is or has been completely saturated during the test, use the more accurate formula:

$$\text{Area of specimen after consolidation, } A_c\text{, sq cm} = \frac{V_o - V_a - \Delta V_w}{H_o - \Delta H_o}$$

where V_o = initial volume of specimen, cc

V_a = initial volume of air in specimen, cc = $V_o - V_s - V_w$

$V_o - V_s - V_w$ = initial volume of specimen minus volume of solids minus initial volume of water

ΔV_w = change in volume of water in the specimen during the saturation and consolidation phases of the test, cc. This value may be computed from the change in weight of the specimen before and after the test or from the burette readings from the start of saturation on to the end of consolidation

H_o = initial height of specimen, cm

ΔH_o = change in height of specimen during consolidation, cm

(3) Using the computed dimensions of the specimen after consolidation and assuming that the water content after consolidation is the same as the final water content, compute the void ratio and degree of saturation using formulas previously presented.

(4) Compute and record on the data sheet (Plate X-2) the axial strain, the corrected area, and the deviator stress at each increment of strain, using the following formulas:

† This formula is based on the assumption that axial and radial strains are equal during consolidation.

$$\text{Axial strain, } \varepsilon = \frac{\Delta H}{H_c}$$

$$\text{Corrected area of specimen, } A_{corr}\text{, sq cm} = \frac{A_c}{1 - \varepsilon}$$

$$\text{Deviator stress, tons per sq ft} = \frac{P}{A_{corr}} \times 0.465$$

where H_c = height of specimen after consolidation, cm = $H_o - \Delta H_o$
P = net applied axial load, lb (see paragraph 5b(2))

(5) Record the time to failure on the data sheet (Plate X-2).

(6) Correct the maximum deviator stress, if necessary, for the effect of membrane restraint (see paragraph 5b(4)).

d. Presentation of Results. The results of the R test shall be presented on the report form shown as Plate X-3, as described in paragraph 5c. A sketch of each specimen after failure should be shown above the Mohr circles. If pore pressure measurements were made during shear, plot the induced pore pressure versus axial strain for each specimen below the stress-strain curves. The procedures below should be followed in drawing strength envelopes:

(1) Undisturbed specimens. For undisturbed specimens, strength envelopes should be drawn tangent to the Mohr circles as shown in Figures 17a and 17b.

(2) Compacted specimens. For compacted specimens, strength envelopes should be drawn through points on the Mohr circles representing stresses on the failure plane as shown in Figure 17c.

8. S TEST. The S test using triaxial equipment, as a rule, shall be performed only with relatively pervious soils. The consolidation of triaxial specimens of relatively impervious soils proceeds so slowly that the time required to complete an S triaxial test inhibits its use in

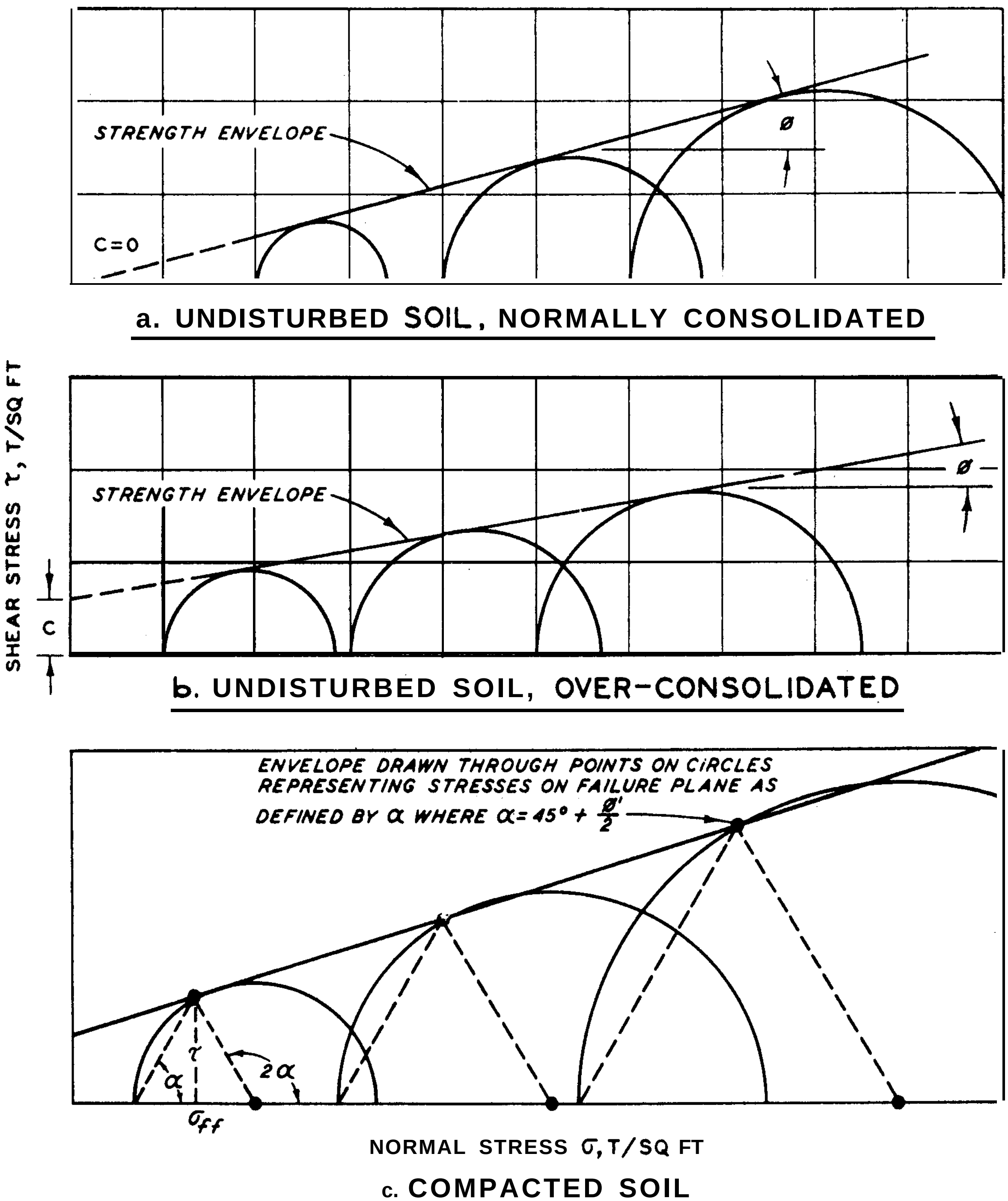

Figure 17. Examples of strength envelopes for R tests

routine laboratory work. Therefore, S tests of fine-grained impervious materials should normally be performed with direct shear equipment (see Appendix IX DRAINED (S) DIRECT SHEAR TEST). However, if scheduling permits, it may be desirable to perform companion S triaxial tests of impervious soils to compare the results with those from S direct shear tests. If the soil to be tested is relatively impervious and contains gravel which would preclude the use of direct shear equipment, consideration should. be given to using triaxial equipment with pore pressure measurements in order to obtain the drained shear strength parameters within a reasonable length of time. All specimens must be completely saturated before application of the deviator stress in the S test.

a. Apparatus. The apparatus used to perform the R test, as illustrated in Figure 16, will usually be satisfactory for the S test, though the equipment for saturation by back pressure will not be necessary for relatively pervious soils. In general, controlled-strain testing should be used for relatively pervious soils, and controlled-stress testing should be used for relatively impervious soils.

b. Procedure. The procedure for the S test shall consist of the following steps:

(1) For soils requiring saturation by back pressure, proceed as outlined in paragraphs 6b(1) through 6b(6). For pervious soils which can be effectively saturated by seepage, that is, by water percolating through the specimen under a small hydraulic head, omit the back pressure equipment described in paragraph 6a and proceed as follows:

(a) Proceed as outlined in paragraphs 5a(1) through 5a(4), with the exception that specimen bases and caps with porous inserts and drainage connections should be used and the apparatus should include a water supply container and a calibrated burette with a vacuum connection as shown in Figure 16. For specimens of cohesionless soil prepared as described in paragraph 4b, the porous inserts and drainage lines (including the burette) should be dry, and a low vacuum (less than 5 psi) should be maintained at both the specimen base and cap (with valves D and G closed) to support the specimen while assembling and filling the triaxial chamber.

(b) Keeping valves A and C closed, adjust the pressure regulator for a chamber pressure of about 5 psi and then open valve A to apply this pressure to the chamber.

(c) With valves E and G closed, maintain a low vacuum through the burette to the specimen cap. Then open valve D and elevate the water supply container so that a hydrostatic head of 1 to 2 ft is applied to the base of the specimen.

(d) When the saturation water rises into the burette, disconnect the vacuum from the burette. Permit seepage under the small head to continue until the rate of flow into the burette is constant, and then close valve D.

(2) With valves E and F open (see Fig. 16), lower the piston .nto contact with the specimen cap and increase the axial load at a relatively slow rate so that a fully drained condition exists at failure with controlled-strain loading or after each increment of load with controlled-stress application. As for the direct shear test, considerable experience and judgment are generally required in determining the proper rate of axial load application (see Appendix IX, DRAINED (S) DIRECT SHEAR TEST). Theoretical formulas are also available† for estimating the time required for failure in S tests. Special precautions may be necessary for tests requiring an axial loading duration in excess of a few hours to insure that the chamber pressure (as well as the back pressure, if used) is maintained constant, that temperature fluctuations are minimized, and that evaporation or aeration of the water in the burette is reduced as much as possible. Placing about 1 cc of oil or dyed kerosene over the water surface in the burette will minimize evaporation.

(3) Record the dial indicator and burette readings at

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† Bishop and Henkel, op. cit., pp. 124-127, 204-206.

increasing intervals of elapsed time under each increment of load. For relatively impervious soils, plot either or both of these readings versus the logarithm of elapsed time, as shown in Figure 5 of Appendix VIII, CONSOLIDATION TEST, to establish when primary consolidation has been essentially completed for each increment of load. Record the final dial indicator and burette readings for each axial load increment on a form similar to Plate X-6 prior to applying the next increment. With controlled-strain loading, periodically observe and record (Plate X-6) the resulting load and the dial indicator and burette readings; sufficient readings should be taken to completely define the shape of the stress-strain curve, Continue the test until an axial strain of 15 percent has been reached; however, when the deviator stress decreases after attaining a maximum value and is continuing to decrease at 15 percent strain, the test shall be continued to 20 percent strain (see Fig. 12).

(4) Upon completion of axial loading, close valves E and F and proceed as outlined in paragraphs 5a(8) through 5a(10), except measure the specimen diameter, as described in paragraph 4a(1)(e), after the compression chamber has been dismantled. While considerable difficulty may be encountered in measuring the diameter of the specimen after the test, such measurements will permit the most reliable computations of the specimen properties at failure.

c. Computations. The computations shall consist of the following steps:

(1) From the observed data, compute and record on the data sheet (Plate X-1) the initial water content, volume of soilds, initial void ratio, initial degree of saturation, and initial dry density using the formulas previously presented.

(2) Compute the cross-sectional area of the specimen, A_c, after completion of consolidation using the formulas presented in paragraph 6c(2).

(3) Using the dimensions of the specimen after consolidation

and the changes in volume as measured with the burette, compute the void ratio and degree of saturation after consolidation using the formulas previously presented.

(4) Compute and record on the data sheet (Plate X-6) the axial strain, the corrected area, and the deviator stress corresponding to the final readings under each increment of load for controlled-stress loading or for convenient intervals of strain for controlled-strain loading using the following formulas:

$$\text{Axial strain, } \varepsilon = \frac{\Delta H}{H_c}$$

$$\text{Area of specimen corrected for strain and volume change, } A'_{corr}, \text{ sq cm} = \frac{A_c}{1 - C\varepsilon}$$

$$\text{Deviator stress, tons per sq ft} = \frac{P}{A'_{corr}} \times 0.465$$

where C = correction for volume change during shear $= \frac{A_f}{A_e}$

A_f = area of specimen after test based on measurements

$$= \frac{\pi D_f^2}{4} = 0.7854\ D_f^2$$

A_e = area of specimen at end of test computed on basis of constant volume

$$= \frac{A_c}{1 - \varepsilon_e}$$

ε_e = axial strain at end of test $= \frac{H_c - H_f}{H_c}$.

P = net applied axial load, lb (see paragraph 5b(2))

(5) Record the time to failure on the data sheet (Plate X-6).

(6) Correct the maximum deviator stress, if necessary, for the effect of membrane restraint (see paragraph 5b(4)).

d. Presentation of Results. The results of the S test shall be presented on the report form shown as Plate X-3, as described in paragraph 5c. If volume changes of the specimens during shear were measured, plot the volumetric strain versus axial strain for each specimen below the stress-strain curves.

9. POSSIBLE ERRORS. Following are possible errors that would cause inaccurate determinations of strength and stress -deformation characteristics :

a. Apparatus. (1) Leakage of chamber fluid into specimen. Such leakage might occur through or around the ends of the membrane or through the drainage connections and it would decrease the effective stress in a specimen during undrained shear, Very little leakage is needed to cause a very large change in effective stress, and the longer the period of undrained shear, the greater the amount of leakage. (Leakage will not influence the effective stress during periods of specimen drainage, but it will introduce errors in volume change measurements,)

(2) Leakage of pore water out of specimen. This leakage might occur through fittings or valves and it would increase the effective stress in a specimen during undrained shear.

(3) Permeability of porous inserts too low.

(4) Restraint caused by membrane and filter paper strips.

(5) Piston friction.

b. Preparation of Specimens. (1) Specimen disturbed while trimming. Disturbance of the natural soil structure does not always result in strength measurements which are too low, that is, on the safe side; disturbed specimens will consolidate more under the effective consolidation pressure in R or S tests and the measured strengths will be too high.

(2) Specimen disturbed while enclosing with membrane. The techniques of placing the membrane around the specimen illustrated in Figures 9 and 10 may not be satisfactory for sensitive undisturbed soils since the specimen would tend to be flexed while binding the membrane to the unsupported cap. Alternatively, the specimen can be set upon an inverted cap clamped to a ringstand and the membrane placed over the specimen and bound to the cap; then the specimen and cap can be inverted onto the base and the lower end of the membrane secured.

(3) Specimen dimensions not measured precisely. Dial gages or micrometers are helpful in obtaining precise measurements. When the specimen diameter is measured after being enclosed by the membrane, twice the thickness of the membrane must be subtracted from the measurement. The cross-section area of large specimens may be determined most satisfactorily from circumference measurements.

c. Q Test. (1) Changes in specimen dimensions upon application of chamber pressure. Partially saturated specimens will compress under the chamber pressure so the change in height, ΔH_o, due to the application of chamber height should be recorded. When this change in height is significant, the area of specimen before shear, A_c, should be computed according to the formula:

$$A_c = A_o \frac{H_o - 2\Delta H_o}{H_0}$$

as given in paragraph 6c(2).

(2) Rate of strain too fast.

(3) Water content determination after test not representative. Friction between the soil and the cap and base restrains the radial deformation at the ends of the specimen and this end restraint induces a nonuniform pore pressure distribution which, in turn, causes pore water migration within the specimen. For relatively impervious soils, a

significant migration of pore water could occur only in a test of long duration (such as S and some R tests); however, for more pervious soils, an appreciable redistribution of water content can occur within the short duration of a Q test. Therefore, it may be desirable to determine the water content of the end sections (about 1/6 of the height at each end) separately from the middle portion. Correlations of strength with water content should be based on the water content of the middle portion, though the dry weight of the entire specimen is needed to compute the initial soil properties.1

d. R Test. (1) Back pressure increments too large in relation to effective consolidation pressure.

(2) Back pressure increments applied too rapidly.

(3) Chamber and back pressures not precisely maintained during consolidation phase. Variations in either or both of these pressures (often much larger than the difference between them) can result in overconsolidation of the specimen.

(4) Specimen not completely consolidated before shearing.

(5) Rate of strain too fast.

(6) Excessive variations in temperature during shear. An increase in temperature will decrease the effective stress in a specimen during undrained shear. This danger, obviously, increases with the duration of the test.

(7) Specimen absorbed water from porous inserts at end of test. As in a consolidation test or a direct shear test, the specimen will absorb water from the porous inserts and drainage lines at the end of the R or S test no matter how rapidly the apparatus is disassembled and the specimen removed. To obtain an accurate water content determination at

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† A. Casagrande and S. J. Poulos, Fourth Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays, Soil Mechanics Series No. 74, Harvard University (Cambridge, Mass., October 1964).

the end of the test, the specimen should be allowed to swell completely under a small (2 or 3 psi) chamber pressure and the increase in volume measured by means of the burette. This volume change can then be used to correct the water content measured after the test.

e. S Test. (1) Rate of strain or rate of loading too fast.

(2) Inaccurate volume change measurements during shear. Where volume changes are measured using a burette, inaccuracies may result from incomplete saturation of the specimen, leakage, evaporation, or temperature fluctuations.

TRIAXIAL COMPRESSION TEST
(SPECIMEN DATA)

Date ______

Project ______

Boring No. ______ Sample No. ______

Type of Test	Confining Pressure tons/sq ft
Test No.	Classification

		Before Test		After Test
		Specimen	Trimmings	Specimen
Tare No.				
Weight, g: Tare plus wet soil				
Weight, g: Tare plus dry soil				
Weight, g: Water	W_w		W_{wo}	W_{wf}
Weight, g: Tare				
Weight, g: Wet soil	W			
Weight, g: Dry soil	W_s			
Water content	w	%	w_o %	w_f %

Initial Condition of Specimen

Diameter, cm	D_o	Top	Center	Bottom	Avg

Height, cm	H_o		Volume of solids, cc	V_s	
Area, sq cm = $0.7854\ D_o^2$	A_o		Void ratio = $(V_o - V_s) \div V_s$	e_o	
Volume, cc = H_oA_o	V_o		Saturation, %	S_o	
Specific gravity of solids	G_s		Dry density, lb/cu ft	γ_d	

Condition of Specimen After Consolidation (R and S Tests)

Change in height during consolidation, in.	ΔH_o		Volume, cc = A_cH_c	V_c	
Height, cm = $H_o - 2.54\Delta H_o$	H_c		Void ratio = $(V_c - V_s) \div V_s$	e_c	
Area, sq cm	A_c		Saturation, %	S_c	

Condition of Specimen After Test (R and S Tests)

Diameter, cm	D_f	Top	Center	Bottom	Avg

Change in height during shear test, in.	ΔH		Volume, cc = A_fH_f	V_f	
Height, cm = $H_c - 2.54\Delta H$	H_f		Void ratio = $(V_f - V_s) \div V_s$	e_f	
Area, sq cm	A_f		Saturation, %	S_f	

$$W_s = \frac{W}{1 + \frac{w_o}{100}}, \quad V_s = \frac{W_s}{G_s}, \quad S_o = \frac{\frac{w_o}{100} \times \frac{W_s}{\gamma_w}}{V_o - V_s} \times 100, \quad S_c = \frac{\frac{w_c}{100} \times \frac{W_s}{\gamma_w}}{V_c - V_s} \times 100,$$

$$S_f = \frac{\frac{w_f}{100} \times \frac{W_s}{\gamma_w}}{V_f - V_s} \times 100, \quad \gamma_d = \frac{W_s}{V_o} \times 62.4, \quad A_c = A_o \frac{H_o - 2\Delta H_o}{H_o}$$

Remarks ______

______

Technician ______ Computed By ______ Checked by ______

PLATE X-1 ENG FORM 1 JUN 65 3852

TRIAXIAL COMPRESSION (Q AND R) TEST
AXIAL LOADING DATA

Date ____________

Project ____________

Boring No. ____________ Sample No. ____________ Test No. ____________

Type Test ____________ Confining Pressure ____________ tons/sq ft

Time	Elapsed Time min	Dial Reading 10^{-} in.	Cumulative Change (ΔH) 10^{-} in.	P Axial Load lb	Axial Strain* $\epsilon = \frac{\Delta H}{H}$	$1 - \epsilon$	$A_{corr} = \frac{A^{**}}{1 - \epsilon}$ sq cm	Deviator Stress $= \frac{P}{A_{corr}} \times 0.465$ tons/sq ft

* Use H_o for Q tests and H_c for R tests.
** Use A_o for Q tests and A_c for R tests.

H_o ______ cm = ______ in. A_o ______ sq cm

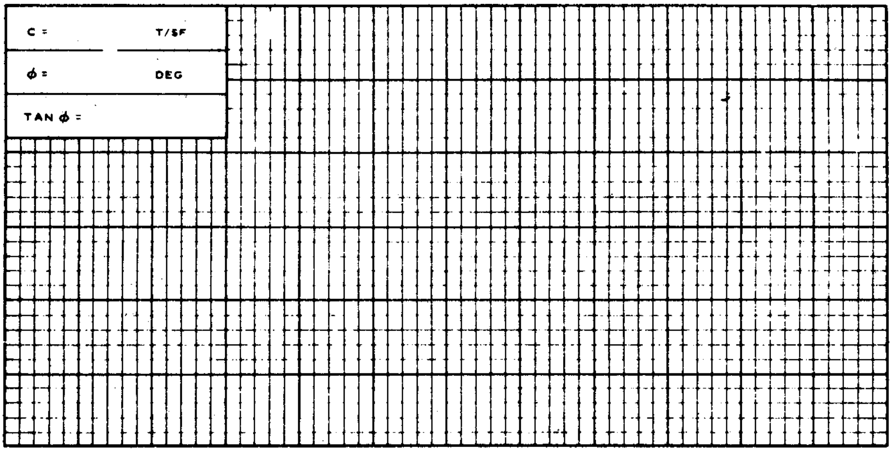
H_c ______ cm = ______ in. A_c ______ sq cm

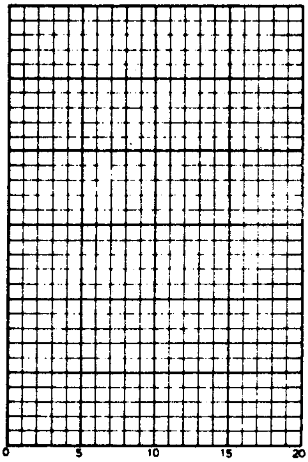
Test time to failure ____________ min Type failure ____________

Technician ____________

ENG FORM 1 JUN 65 3853

PLATE X-2





SPECIMEN NO.						
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/CU FT	γ_{d_o}				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3				
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$				
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST

REMARKS:

PROJECT

BORING NO. | SAMPLE NO.

DEPTH/ELEV

LABORATORY | DATE

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970

PLATE X-3

TRIAXIAL COMPRESSION TEST
(BACK PRESSURE AND PORE PRESSURE DATA)
(Saturation)

Date ____________

Project ____________ Boring No. ____________ Sample No. ____________

Test No. ____________ Consolidation Pressure ____________ tons/sq ft = ______ psi

Increment No.	Time	Elapsed Time min	Chamber Pressure psi	Applied Back Pressure psi	Measured Pore Pressure psi	Burette Reading cc	Volume Change (ΔV) cc

Technician ____________

ENG FORM 1 JUN 65 3854

PLATE X-4

TRIAXIAL COMPRESSION (R AND S) TEST
PRELIMINARY CONSOLIDATION

Date ____________

Project ____

Boring No. ____________ Sample No. ____________ Test No. ____________

Test Type ____________ Consolidation Pressure ____________ tons/sq ft

Date and Time	Elapsed Time min	Burette Reading cc	Volume Change (ΔV) cc	Dial Indicator Reading 10^{-} in.	Date and Time	Elapsed Time min	Burette Reading cc	Volume Change (ΔV) cc	Dial Indicator Reading 10^{-} in.

Dial indicator reading before consolidation ____________ in.
Dial indicator reading after consolidation ____________ in.
Change in height during consolidation (ΔH_o) ____________ in. = ______ cm

Remarks ____

____

Technician ____________

PLATE X-5 ENG FORM 1 JUN 65 3855

TRIAXIAL COMPRESSION (S) TEST
AXIAL LOADING DATA

Date ____________________

Project ____

Boring No. ______________ Sample No. ______________ Test No. ______________

Specimen No. ______________ Consolidation Pressure ______________ tons/sq ft

Date and Time	Dial Reading 10^{-} in.	Cumulative Change (ΔH) 10^{-} in.	Burette Reading cc	Volume Change (ΔV) cc	P Axial Load lb	Axial Strain $\epsilon = \frac{\Delta H}{H_c}$	$C\epsilon$	$1 - C\epsilon$	$A'_{corr} = \frac{A_c}{1 - C\epsilon}$ sq cm	Stress $\frac{P}{A'_{corr}} \times 0.465$ tons/sq ft

Area of specimen after test, A_f = __________ sq cm

Axial strain at end of test, ϵ_e = __________

Area of specimen at end of test, $A_e = \frac{A_c}{1 - \epsilon_e}$ = __________ sq cm

$C = \frac{A_f}{A_e}$ = __________

Test time to failure, t_f = __________

Remarks ____

____

____

____

Technician ____________________

ENG FORM 1 JUN 65 3856

PLATE X-6