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Shear strength behavior in silts and clay mixtures

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SHEAR STRENGTH BEHAVIOR IN SILT AND CLAY MIXTURES

by

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A

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SYNOPSIS

A series of consolidated undrained triaxial tests with pore pressure measurements were performed on laboratory sedimented samples having various known silt to clay content ratios. The clay used was commercial grundite clay from Goose Lake, Illinois, and the silt used was extracted from Lebanon Silt Loam from Rolla, Missouri. Silt was mixed with clay in the following percentages by weight: 0%, 20%, 40%, 60%, and 80%.

Results indicate an increasing angle of internal friction with an increase in silt content. The unconfined compressive strength of the samples remained constant, as would be expected in a clay, with increasing silt content up to the 80% silt mixture. At the 80% silt content, the soil behaved as a granular material.

INTRODUCTION

The foundation engineer frequently must deal with problems associated with the shear strength of silt[1] and clay[2] mixtures because there is an abundance of these mixtures in natural soil deposits. Adequate information is available on the separate treatment of shear strength in silt and clay; however, there is a lack of information available on mixtures. The purpose of this paper is to evaluate the change in shear strength behavior when increased amounts of a silt are combined with a clay.

Samples were mixed artificially and sedimented because of the obvious difficulties involved in finding a homogeneous soil of predetermined grain size. A sedimentation unit was used similar to the one described by Olson[2]. A diagram of the unit is shown in Fig. 3. The sedimentation unit made it possible to obtain 100% saturated one-dimensionally consolidated samples of any desired mix. After samples were consolidated one dimensionally, they were placed in a triaxial cell for three-dimensional consolidation and shearing.

SOIL

Commercial grundite clay was procured from the Illinois

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[1]Penman, A. D. M., "Shear Characteristics of a Saturated Silt Measured in Triaxial Compression," Geotechnique, Volume 3, 1953, p. 312.

[2]Olson, R. E., "The Shear Strength Properties of Calcium Illite," Geotechnique, Volume 12, No. 1, 1962, p. 23.

Clay Products Company. X-ray diffraction tests show it to be an illite. The liquid limit was found to be about 56%; the plastic limit, 27%; the shrinkage limit, 15%; and it has a specific gravity of 2.73. The grain size distribution curve as determined by hydrometer analysis is shown in Fig. 1.

Silt was obtained from the "A" horizon of Lebanon Silt Loam. Silt size particles were extracted from the silt loam by means of a sedimentation tank. A schematic diagram of the 95 gallon stock watering tank used is shown in Fig. 2. The soil was passed through a #10 U. S. Standard Sieve to remove pebbles and coarse sand before being placed in the tank. The tank was filled 10 inches above the top of the vertical portion of the drain pipe. The soil and water were mixed thoroughly during filling by water jets, and by means of a paddle after filling. Approximately 220 grams of calgon were added to the water, making a 4% calgon solution. Calgon was used to avoid flocculation of the clay particles. Soil particles settled to the bottom of the tank at velocities related to particle size in accordance with Stokes' law. Enough time was computed for the silt size particles to fall below the top of the vertical portion of the drain pipe. Then the water containing particles finer than silt were drawn off through the drain and discarded. Hydrometer readings were taken while the particles finer than silt were being drained. The tank was then refilled with the 4%

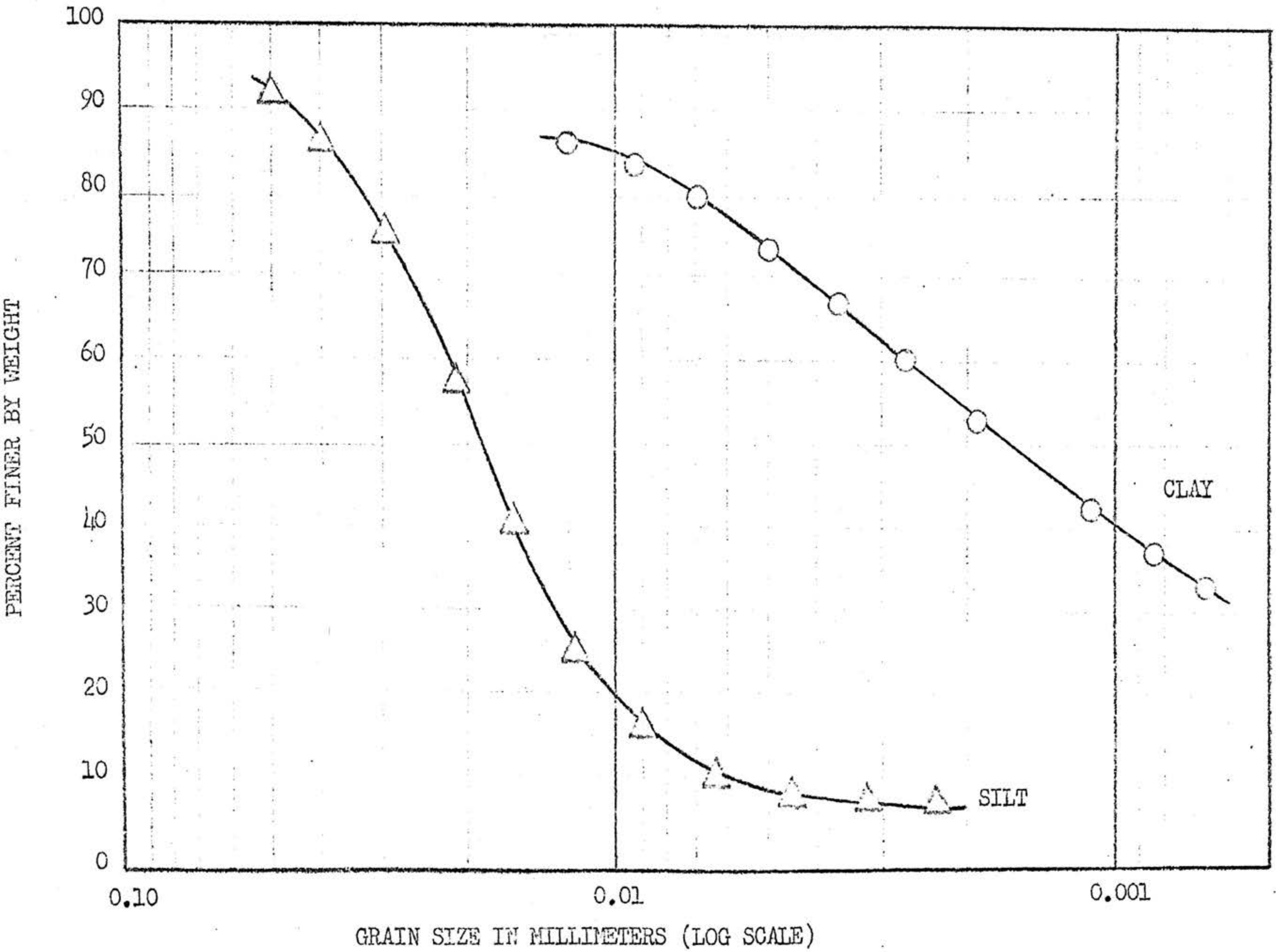
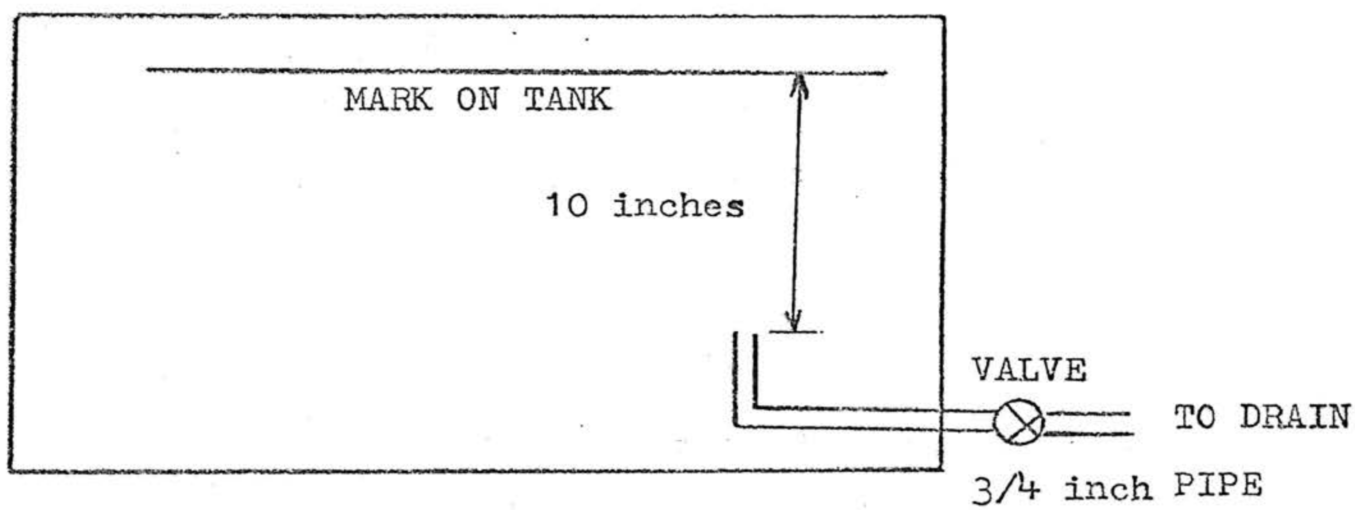


FIG. 1.-GRAIN SIZE DISTRIBUTION CURVES OF SILT AND CLAY.



STOCK WATERING TANK

FIG. 2.-SEDIMENTATION TANK FOR REMOVING CLAY FROM SILT.

calgon solution and the washing process repeated. By the time the washing process was repeated 20 times for each batch of soil, there were no changes in hydrometer readings and the wash water appeared clear. The washed product was dried and passed through a #200 U. S. Standard Sieve. The grain size distribution curve determined by hydrometer analysis for the silt is compared with that of clay in Fig. 1. The specific gravity of the silt was found to be 2.65. X-ray diffraction tests revealed that the silt is primarily quartz with less than 10% feldspar.

The physical properties of the silt and clay mixtures are shown in Table 1.

TABLE 1

PHYSICAL PROPERTIES OF SILT AND CLAY MIXTURES

% Silt	% Clay	Specific Gravity	Liquid Limit	Plastic Limit	Shrinkage Limit
	100	2.73	56	27.0	15.0
20	80	2.71	45	23.0	15.4
40	60	2.70	35	19.8	16.0
60	40	2.68	29	19.0	23.0
80	20	2.67	--	22.0	25.0
100					

SAMPLE PREPARATION

Samples were mixed in percentages by weight using the formula: $\frac{\text{Weight Silt}}{\text{Weight Silt} + \text{Weight Clay}} \times 100 = \%$ Silt, as shown in Table 1. The soil was mixed in a dry condition in a sealed quart glass jar. The jar was rotated until the silt and clay appeared to be a homogeneous mixture. The dry mixture was then poured into sufficient deaired, distilled water to make a solution having a viscosity about that of thin gear oil. When the soil was 100% saturated with distilled water, the solution was mixed thoroughly with a spatula. Then it was poured through a rubber tube one inch in diameter into the sedimentation unit. The rubber tube was used as a tremie to help prevent segregation. Each percentage mix contained varying amounts of soil and water. Table 2 shows the silt, clay and water content of the mixture, sample heights, and axial stresses used for one-dimensional consolidation in the sedimentation unit. The 80% silt samples were not trimmed because the disturbance created by trimming caused the samples to lose their shape which caused a considerable amount of remolding.

TABLE 2

PROPORTIONS BY WEIGHT, SAMPLE HEIGHT, AND AXIAL STRESS OF SAMPLES PREPARED FOR ONE-DIMENSIONAL CONSOLIDATION

% Silt	Weight Clay gr	Weight Silt gr	Weight Water gr	Height Sample in	Axial Stress psi
0	140	0	140	3.8	7.1
20	116	29	130	3.7	7.1
40	90	60	120	3.7	7.1

TABLE 2 - (Cont'd.)

% Silt	Weight Clay gr	Weight Silt gr	Weight Water gr	Height Sample in	Axial Stress psi
60	60	90	100	3.6	14.2
80	27	108	70	3.2	21.3

The sedimentation unit shown in Fig. 3 is designed to perform one-dimensional consolidation. Drainage is through porous stones at the top and bottom of the sample. A light coating of chemically inert silicone oil was applied to the inside of the plastic cylinder before the soil water slurry was introduced to help reduce ring friction between the piston and the plastic cylinder. The 20 inch long plastic cylinder forms a 1.4 inch sample diameter that requires trimming on the ends only for use in the triaxial cell. After the sample was poured into the sedimentation unit, a vacuum was created in the unit for about one minute to de-air the sample. In the 60% silt and 80% silt samples, particles lodged between the piston and cylinder wall caused the piston to stick. This (side) friction was overcome by increasing the axial stress as shown in Table 2.

After the sample was fully consolidated, it was extruded from the sedimentation unit. The sample was then trimmed at both ends leaving a sample height of about 3 inches. The triaxial samples were consolidated to a water content near the liquid limit. Care had to be taken not to over-disturb

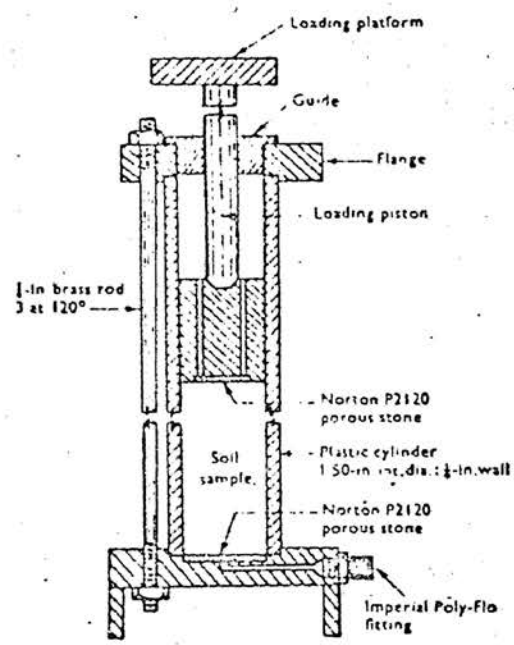


FIG. 3.-SEDIMENTATION UNIT FOR PREPARING TRIAXIAL SAMPLES.

them when they were being handled during trimming and placing in the triaxial machine.

Samples were checked for variations in water content from top to bottom after they were extruded from the sedimentation unit. The water contents checked as shown in Table 3 indicate less variation than reported by Olson[2].

TABLE 3

DISTRIBUTION OF WATER CONTENT IN ONE-DIMENSIONALLY CONSOLIDATED SPECIMENS

Position of sample	100% Clay Water Content %	40% Silt Water Content %	60% Silt Water Content %
Top	38.5	34.4	31.7
	41.1	34.7	33.3
	42.8	35.9	34.0
	46.8	36.7	34.4
	45.0	38.0	34.4
	45.4	38.5	33.8
Bottom	43.7	36.8	33.2
Average	43.2	36.4	33.5

Samples were also checked for segregation. After extrusion from the sedimentation unit hydrometer analyses were performed on the top and bottom halves of samples of each percentage and the amount of segregation was found to be negligible.

SHEAR TESTS

Shear tests were performed using a Geonor triaxial machine developed by the Norwegian Geotechnical Institute.[3] A special rotating bushing in each triaxial cell was used to reduce friction between the piston and the bushing as the sample was being sheared. The initial sample diameter was about 1.4 inches, and the length about 3.1 inches. During consolidation in the triaxial cell, the sample was drained by means of a slotted filter paper on the side and a porous filter stone on the bottom. A solid cap with no drainage connections or filter stone was placed on the top of this sample. A single Trojan brand rubber membrane of .002 inch thickness encased each sample. Results were not corrected for the filter paper, piston friction, or the membrane, because the corrections were believed to be negligible.

The triaxial machine was equipped with a pore pressure device for measurement of pore pressures in the sample without allowing volume changes. The equipment consisted of: a capillary "U" tube filled with mercury, a Bourdon gauge for measuring pressures, and a screw control.[4] The entire

[3]Andresen, A. and Simons, N. E., "Norwegian Triaxial Equipment and Technique," Research Conference on the Shear Strength of Cohesive Soil, ASCE, Jun, 1960, p.p. 695-708.

[4]"Instructions for the Assembly, Maintenance and Use of the Triaxial Equipment Developed at the Norwegian Geotechnical Institute," Geonor, A. S., Blindern, Norway, Sept, 1963.

system is filled with distilled water which is considered to be incompressible; as a result, any changes in pore water pressure are immediately reflected on a Bourdon gauge. The screw control consists of a metal cylinder with an internal piston fitted with two cup packings so that it is leak-tight when moving in or out. The piston is moved by turning the screw control that is attached to a threaded rod. The piston movement develops either a pressure or vacuum in the system. One branch of the "U" is connected directly to the sample. The other end of the "U" is connected to the screw control and Bourdon gauge. By observing the mercury level in the branches of the "U" tube, it is possible to detect incipient egress or ingress of pore water from or to the sample. By using the screw control, the mercury level can be maintained at a constant level, thereby preventing any pore water movement. The pressure required to hold the mercury column in position is, therefore, the pore pressure within the sample.

The samples for each different mix were further consolidated in the triaxial cell to pressures of 14.22 psi, 28.44 psi, and 42.66 psi. Consolidation, as indicated by the water draining from the sample into a calibrated burette, took place in each case even though the 60% silt and 80% silt samples were subjected to an axial stress of 21 psi in the sedimentation unit.

All samples were sheared at the constant strain rate of

.172 in/hour. Deviator stress, pore water pressure, percent strain and final void ratio were calculated and summarized by computer. Sample programs including plotter programs for stress-strain, pore pressure-strain, and Mohr failure circles are shown in the appendix.

Each sample was subjected to a back pressure of 28.4 psi to ensure 100% saturation as recommended by Bishop and Henkel.[5] The advantage of using a back pressure is that it will dissolve entrapped air in the sample or the space between the sample and the rubber membrane. When consolidation has been completed, all drainage connections to the sample are closed and the sample is connected to the pore pressure device. The reading on the Bourdon gauge of the pore pressure device will be zero, indicating the sample has fully consolidated. The confining pressure in the triaxial cell is then raised in increments of 5 psi to 28.4 psi. Only when all entrapped air has dissolved will the cell pressure increase be reflected in a pore pressure increase. After the Bourdon gauge of the pore pressure device reads 28.4 psi, back pressure is completed, and the shearing test is ready to proceed.

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[5]Bishop, A. W. and Henkel, D. J., "The Measurement of Soil Properties in the Triaxial Test," Edward Arnold (Publishers) LTD, London, 2nd Edition, p. 209.

DISCUSSION OF RESULTS

Mohr-Coulomb failure envelopes denoting increasing proportions of silt are shown in Fig. 4. There is a slight cohesion intercept that should not have occurred in all but the 80% mix. This intercept is possibly caused by experimental or equipment error, or changing temperatures that took place during testing.

The envelopes indicate an increase in the angle of internal friction as the percentage of silt increases. This bears out the soil structure hypothesis reported by Trollope and Chan,[6] who suggest that: "shear strength = colloidal friction + intergranular friction." As suggested by this hypothesis, the addition of silt to a clay matrix increases the shear strength by the addition of intergranular friction to colloidal friction. A diagrammatic representation of this concept is shown in Fig. 5. When a sufficient quantity of silt is added, it can be seen that colloidal friction has little influence on shear strength. In these experiments, the percentage mix in which the shear strength due to intergranular friction becomes dominant is the 80% silt mix. It is interesting to note from Table 2 that a liquid limit test could not be performed on the 80% silt mix.

Samples from 0% to 60% silt failed by bulging in a manner similar to the samples of sedimented calcium illite

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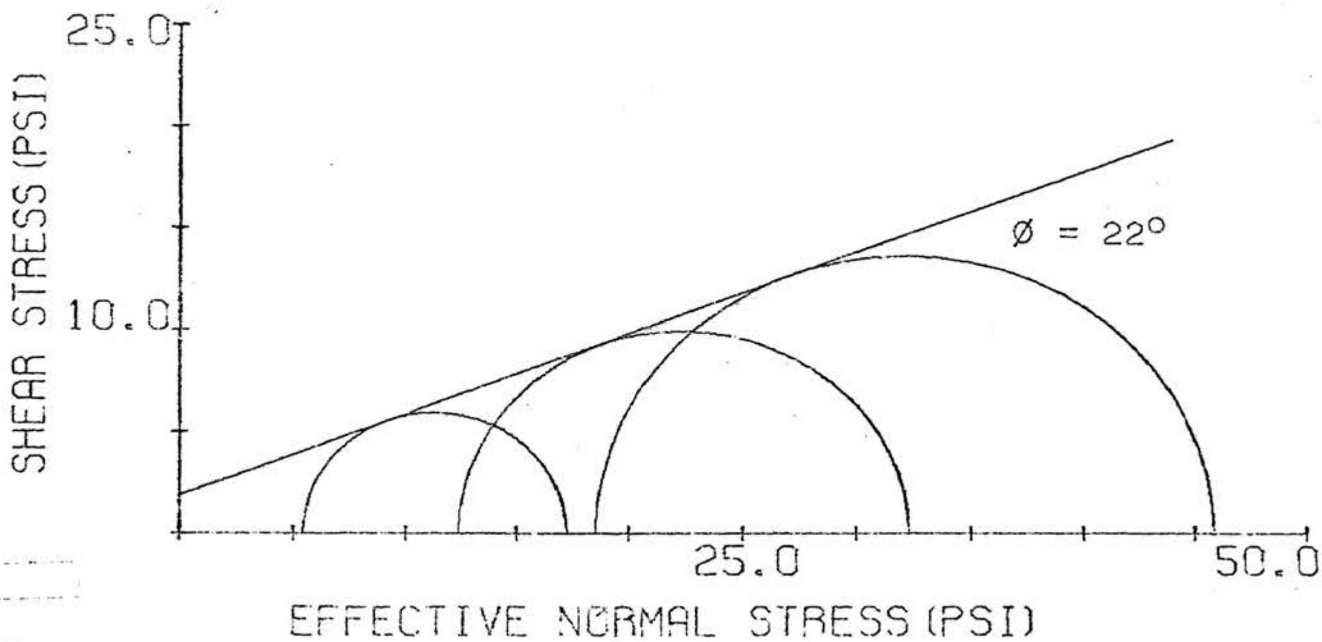
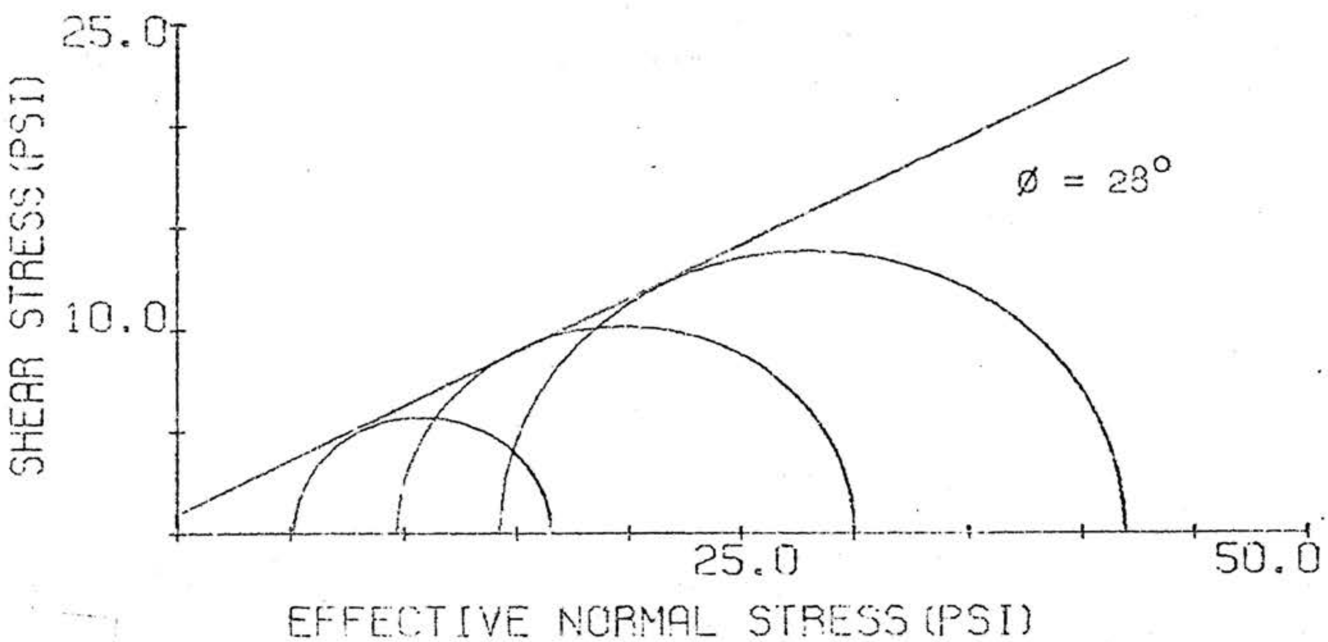
[6] Trollope, D. H., and Chan, C. K., "Soil Structure and the Step Strain Phenomenon," Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 86, No. SM 2, Proc. Paper 2431, April, 1960, p. 15.

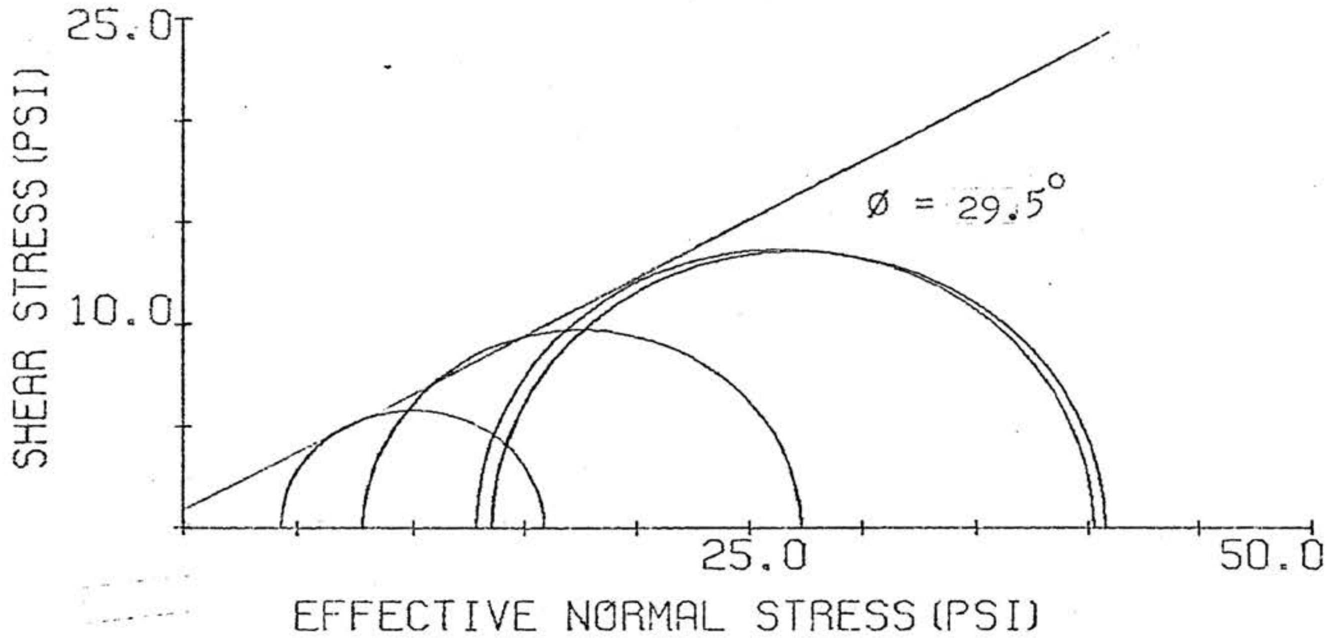


(a) 100% Clay.

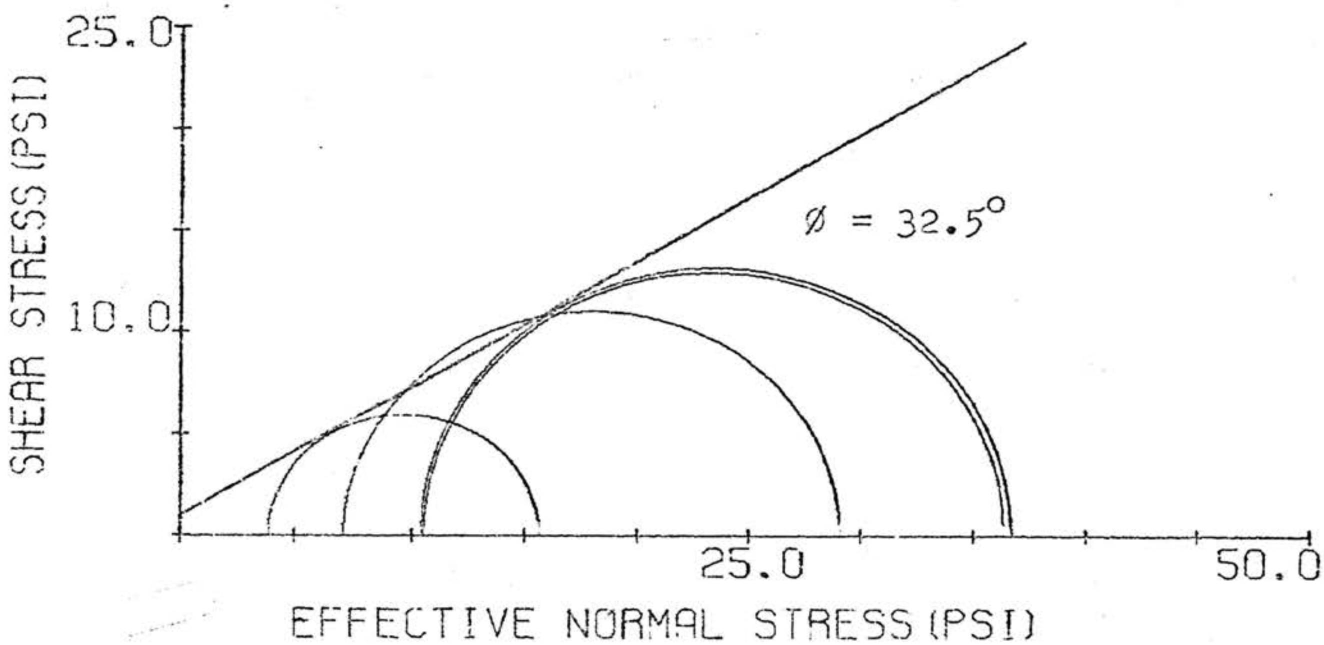


(b) 80% Clay, 20% Silt.

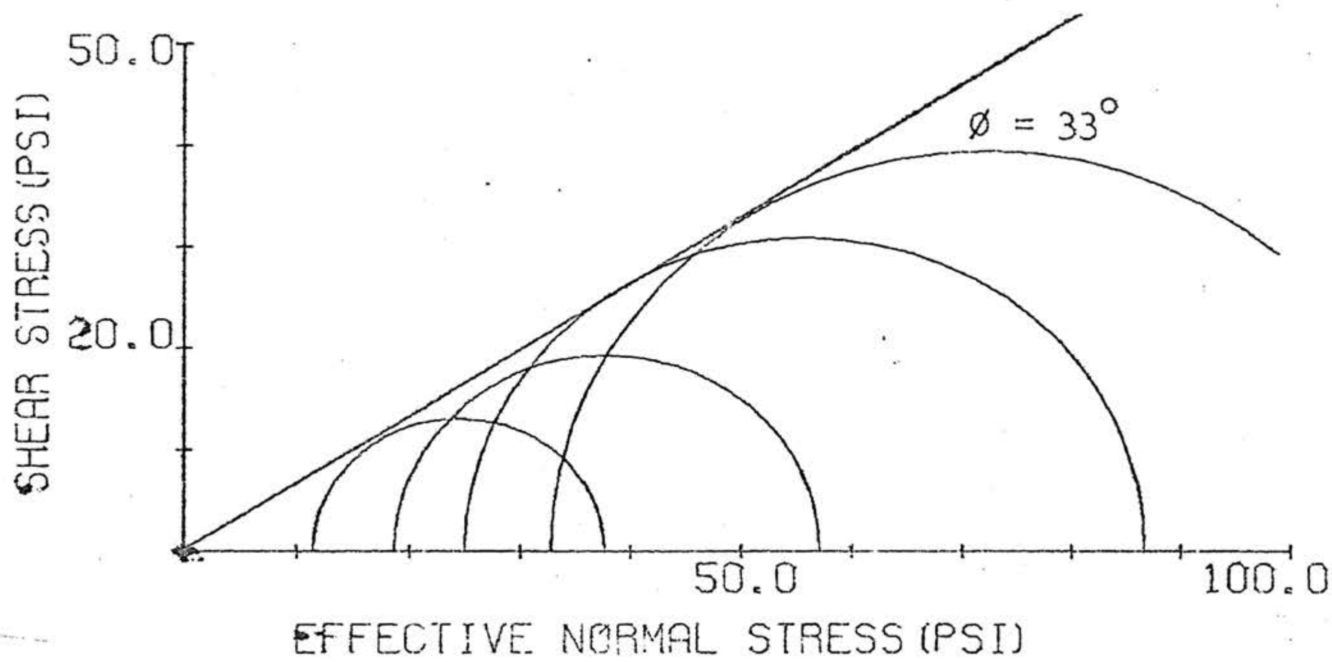
FIG. 4.-MOHR-COULOMB $\overline{R}$ TYPE FAILURE DIAGRAMS.



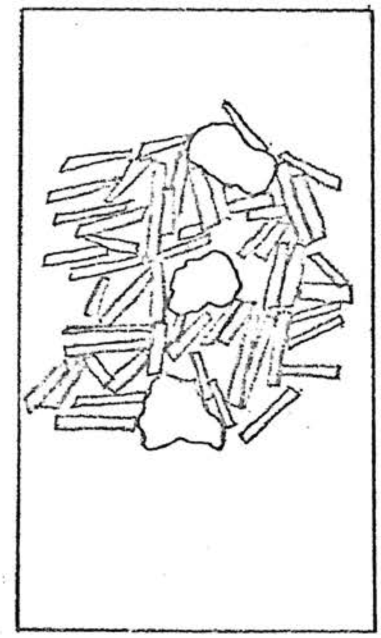
(c) 60% Clay, 40% Silt.



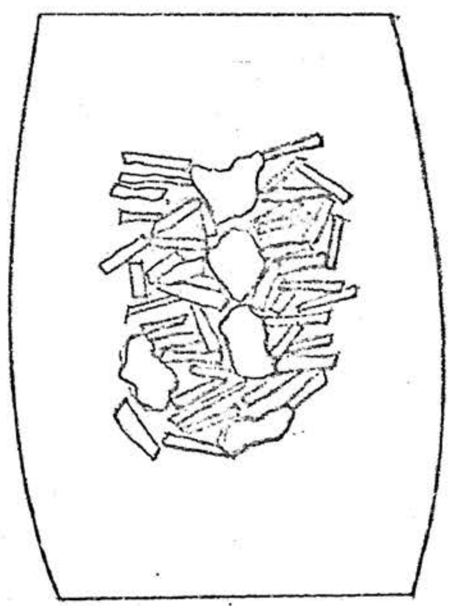
(d) 40% Clay, 60% Silt.



(c) 20% Clay, 80% Silt.



(a) Before matrix yield strength.



(b) After matrix yield, some intergranular friction.

FIG. 5.-DIAGRAMMATIC REPRESENTATION OF BUILD UP OF GRANULAR FRICTION AT FAILURE.

reported by Olson.[2] The value of 22 degrees for the angle of internal friction of the 100% grundite clay falls within the range of those cited in soils texts.[7]

The 80% silt samples behaved as a granular soil and would not stand unaided without collapsing unless an internal vacuum was created giving the rubber membrane rigidity by suction. To prevent over-disturbance from handling, it was necessary to extrude the sedimented sample into a sample former that was placed over the pedestal of the triaxial cell. A vacuum source of 4 psi was connected to the base of the sample through the pedestal and porous stone. The vacuum source was connected for about 15 minutes before the sample former could be removed. The vacuum was not released until the confining pressure in the triaxial cell was applied to the sample. Filter paper side drainage was not used for the 80% silt sample. Four tests were performed on the 80% silt samples at the same consolidation pressure of 42.66 psi. Two of the samples failed by bulging and two had visible failure planes. The application of the vacuum in all but one test caused less than 100% saturation as observed when a back pressure was applied. The angle of internal friction indicated by the four tests was about 33 degrees, which falls within the range of those reported for silt.[5]

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[7]Terzaghi, K., and Peck, R. B., "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., N. Y., 2nd Edition, p.p. 107 & 112.

Deviator stress-strain and pore pressure strain curves are shown in Fig. 6. The stress-strain curves for all samples appear typical of an initially flocculated structure. There is an apparent steeper slope in the stress and pore water pressure curves with an increase in silt content which indicates a more flocculent structure with the addition of silt. A study of the diagrammatic representation of a silt and clay mixture in Fig. 5 shows that the irregular shaped granular particles of silt disrupt, or allow flocculation, of the flat plate-like clay particles that surround the silt. The flocculent structure agrees with the hypothesis suggested by Olson[2] for similarly sedimented samples. Olson used X-ray diffraction studies, measurement of shrinkage limits, and observations of drying cracks to substantiate his findings.

The flocculated structure hypothesis can also be substantiated by comparing the stress-strain and pore water strain curves in Fig. 6, with results on compacted silty clay samples obtained by Seed, Mitchell, and Chan.[8] Seed, Mitchell, and Chan state that dispersed structures of clay particles in relatively parallel array are generally associated with high molding water contents and compaction proceedures inducing large shear strains; whereas,

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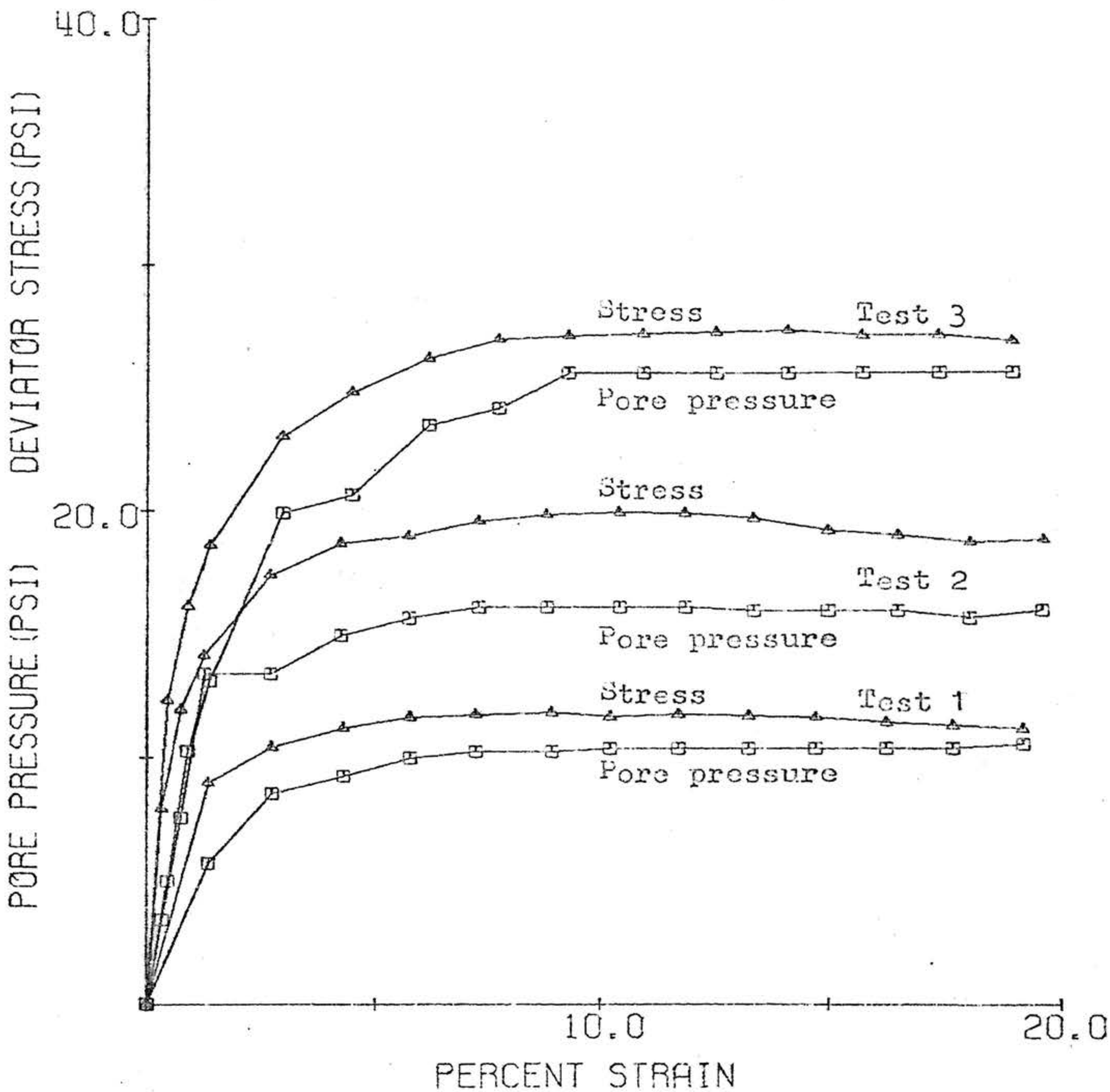
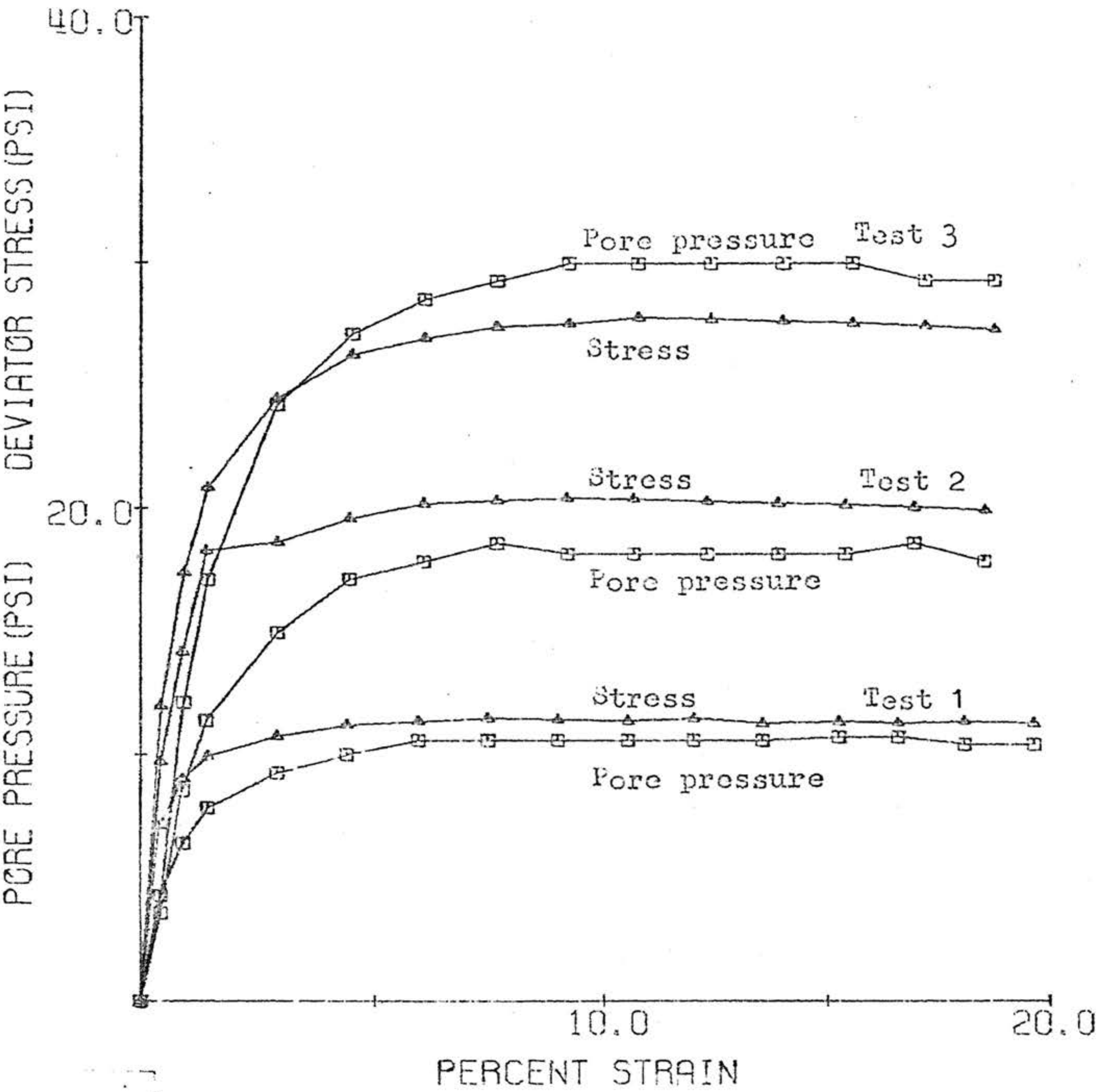
[8] Seed, H. B., Mitchell, J. K., and Chan, C. K., "The Strength of Compacted Cohesive Soils," Research Conference on The Shear Strength of Cohesive Soil, ASCE, Jun, 1960, p. 880.

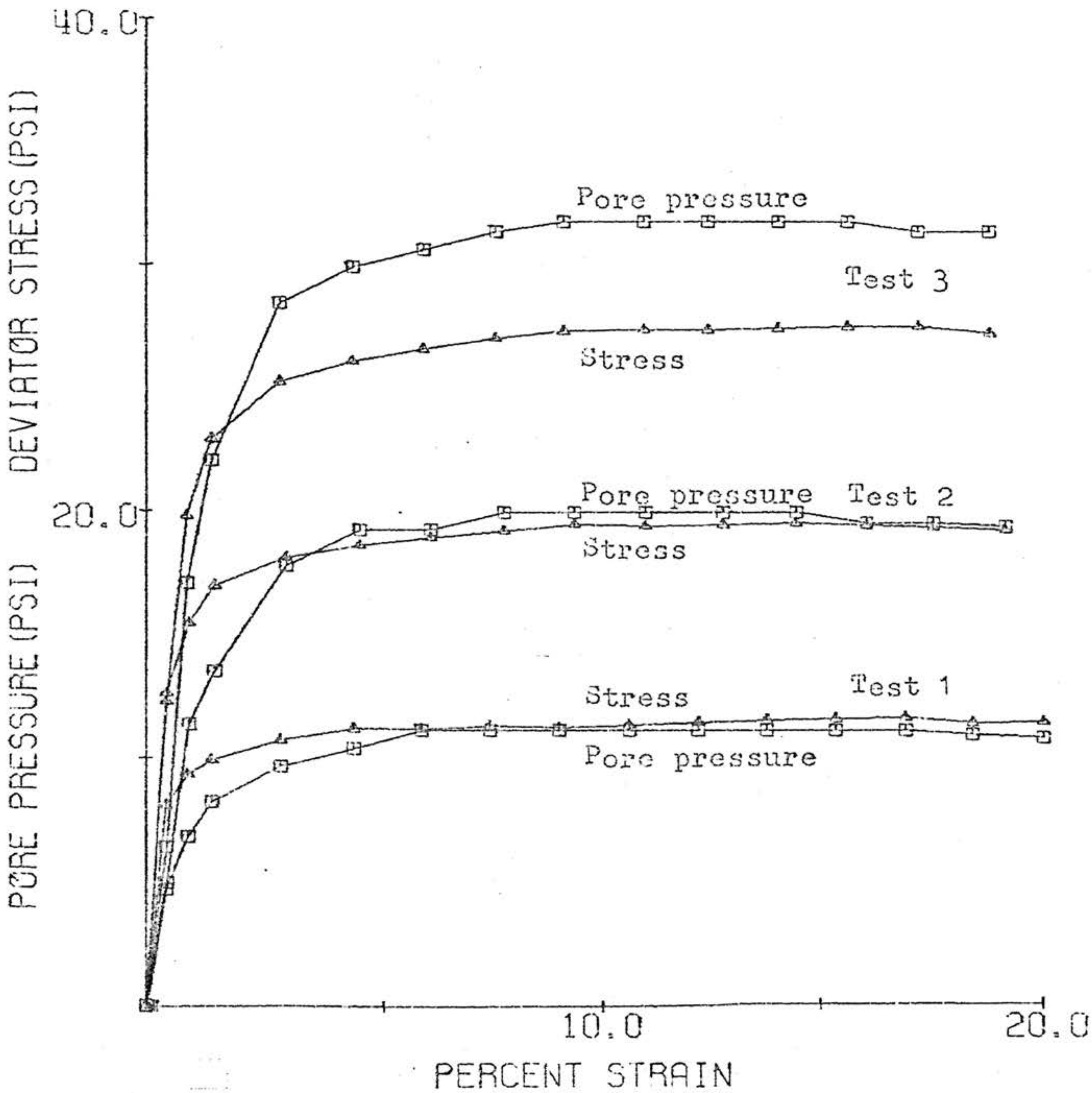


(a) 100% Clay. Confining pressures 1, 2, and 3 KG/SQCM

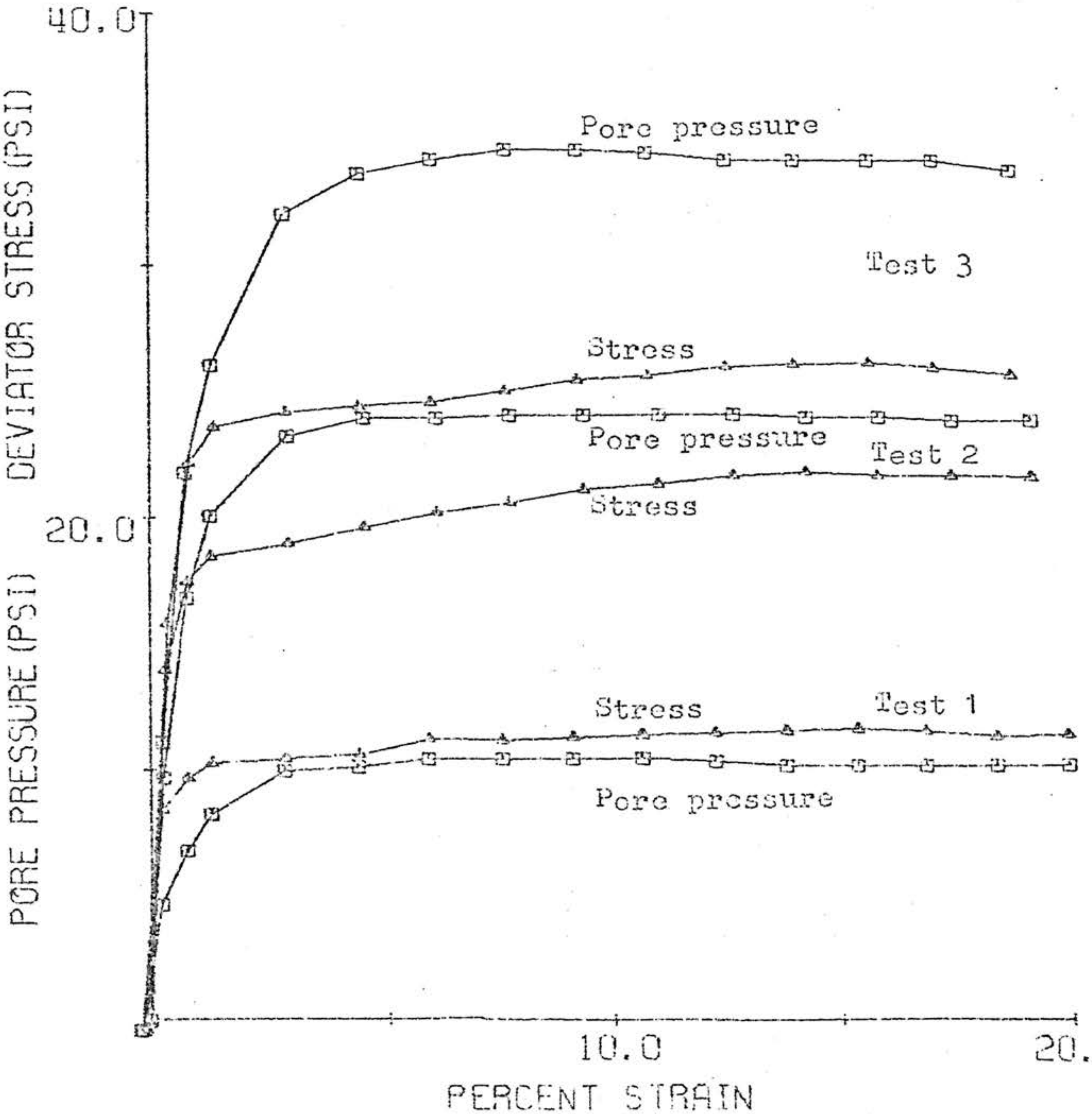
FIG. 6.-STRESS STRAIN AND POREWATER STRAIN CURVES.



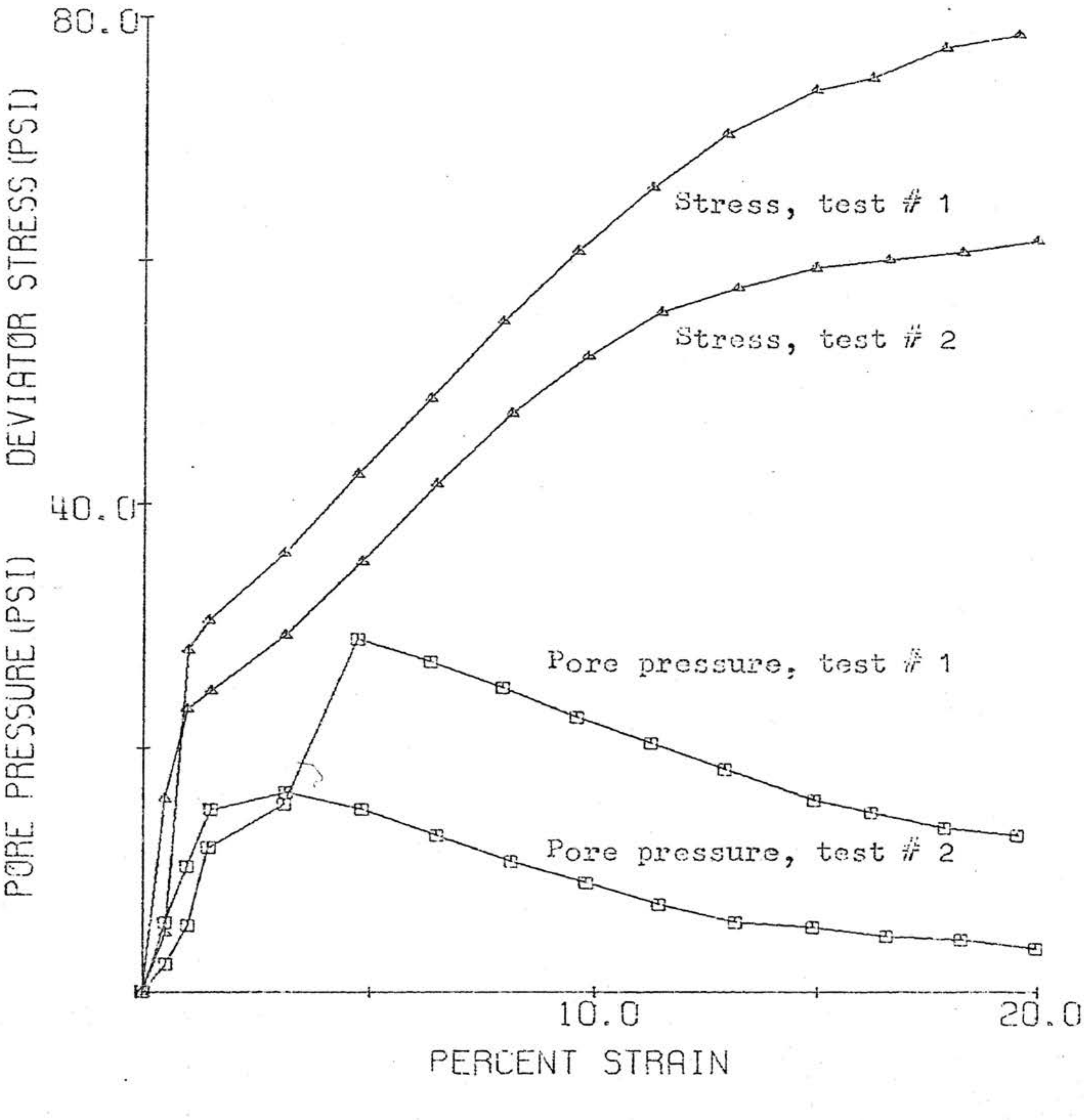
(b) 80% Clay, 20% Silt.
Confining pressures 1, 2, and 3 KG/SQCM.



(c) 60% Clay, 40% Silt.
Confining pressures 1, 2, and 3 KG/SQCM.



(a) 40% Clay, 60% Silt.
Confining pressures 1, 2, and 3 KG/SQCM.



(e) 20% Clay, 80% Silt.
Confining pressure for both tests 3 KG/SQCM.

flocculated structures, with clay particles in random array, result from low water contents and compaction procedures inducing little shear strain. In the early stages of the consolidated undrained triaxial shear tests on silty clay samples, the pore water pressures within the samples compacted wet of optimum (dispersed structure) are considerably greater than those in the samples compacted dry of optimum (flocculated structure), even though both samples had the same water content during the test. The curves shown in Fig. 6 of this paper compare favorably with the curves of an initially flocculated structure reported by Seed, Mitchell, and Chan.

The A-coefficient is the ratio of the pore water pressure to the stress difference. The A-coefficient at the point of failure or maximum shearing stress is defined as A_f. A_f coefficients are shown in Table 4. The A_f coefficients at failure are less than one for 100% clay and go above one as the percentage of silt increases. The increase in the A_f coefficient with the addition of silt to a clay matrix can be seen by looking at a theoretical mixture as shown in Fig. 5, and noting that the structure becomes more flocculent with the increase of silt. When the clay structure around the silt collapses, there is an increase of pore pressure caused by a tendency of the sample to decrease in volume. When the soil becomes more granular in nature as in the case of the 80% mix, the A_f coefficient is reduced because the intergranular contact allows less structure collapse.

The A_f coefficient in the fine grained samples, 0% to 60% silt, increased as the consolidation pressure increased. When the samples were consolidated one dimensionally and then subjected to further consolidation in the triaxial cell, there was a tendency for the clay particles to become more parallel (dispersed) with the increase in consolidation pressure, apparently causing the rise in pore water pressure and the A_f coefficient. Seed, Mitchell, and Chan[8] substantiate this by observing that dispersed structures develop higher pore pressures during shear than flocculated structures.

TABLE 4

RELATIONSHIP BETWEEN A_f COEFFICIENTS AND CONSOLIDATION PRESSURES

Consolidation pressures	A_f 0%	A_f 20%	A_f 40%	A_f 60%	A_f 80% Silt
14.22 psi	0.87	0.92	0.96	0.88	--
28.44 psi	0.81	0.89	1.02	1.10	--
42.66 psi	0.94	1.08	1.14	1.31	0.15

By plotting the maximum deviator stress versus the percent mix, it is interesting to note that the stress at failure is independent of the gradation. This $\phi = 0$ condition is valid only for mixes containing less than 80% silt. Thus the shearing strength under the $\phi = 0$ condition may be evaluated on the basis of unconfined compression tests as: $s = c = 1/2q_u$. This relationship is shown in Fig. 7.

Final void ratio changes at failure versus percent mix are shown in Fig. 8. The transition from a clay or fine grained material to a granular material can be seen by comparing the variation in void ratios due to the consolidation pressure for each percent mix. Wide variation in final void ratios for the 100% clay decrease to a relatively small variation for the 60% silt mix just before granular behavior is observed in the 80% silt mix. The void ratio in the clay mixtures is affected by the confining pressures. The silt in the mixes before the 80% silt mix is apparently floating in the clay matrix. In the 80% mix, the confining pressure no longer affects the void ratio, which is typical of intergranular contact in a granular soil.

CONCLUSIONS

The results of this study indicate several conclusions which apply to the material used.

It is apparent that the angle of internal friction increases as the silt content is increased. The addition of silt apparently allowed a more flocculent clay structure with an increase in pore pressures. Clay-like behavior dominated until a high percentage of silt was introduced. In this study it was not until 80% silt was added that the soil behaved as a granular material. The unconfined compressive

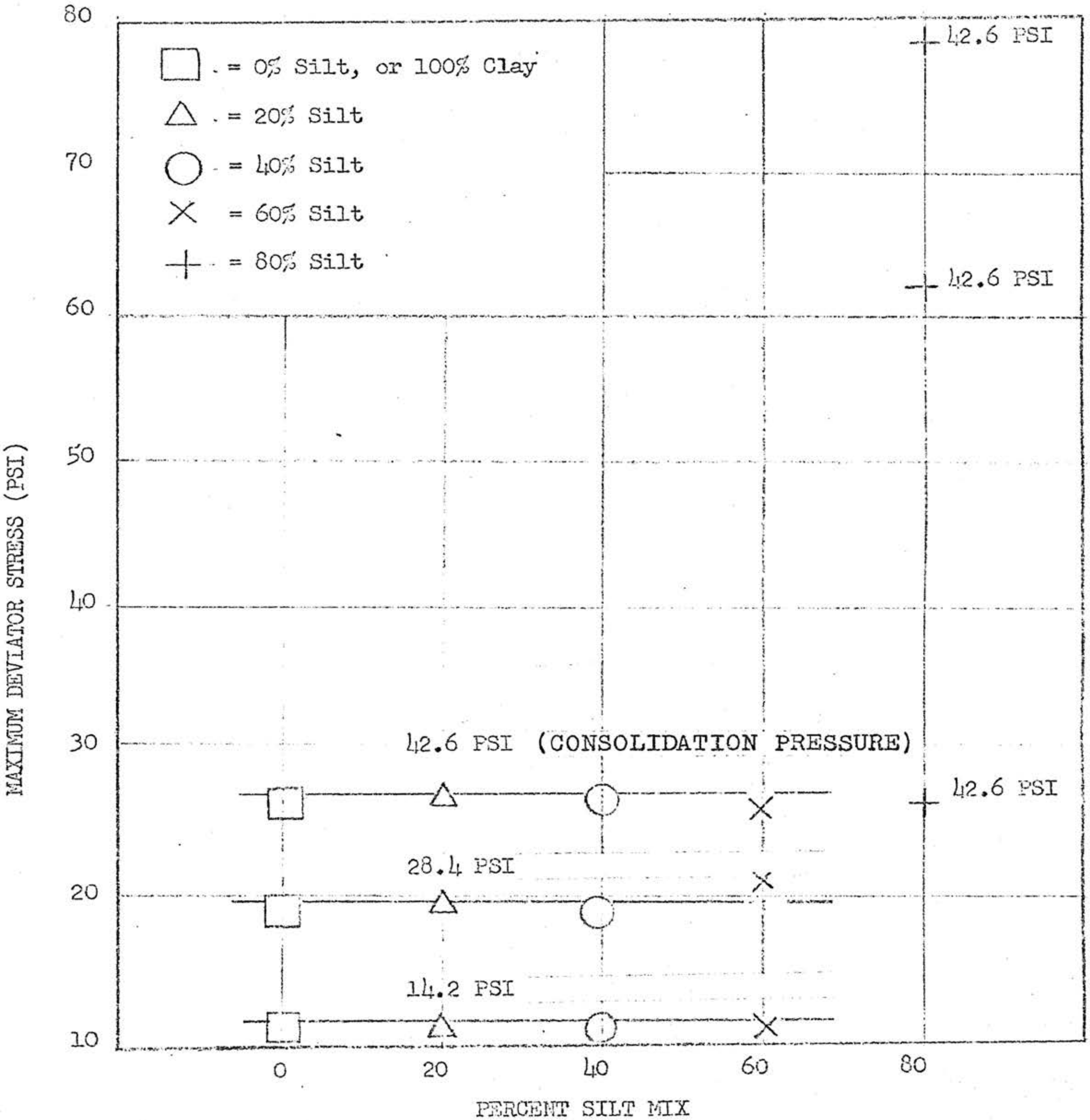


FIG. 7.-MAXIMUM DEVIATOR STRESS vs SILT CLAY COMPOSITION.

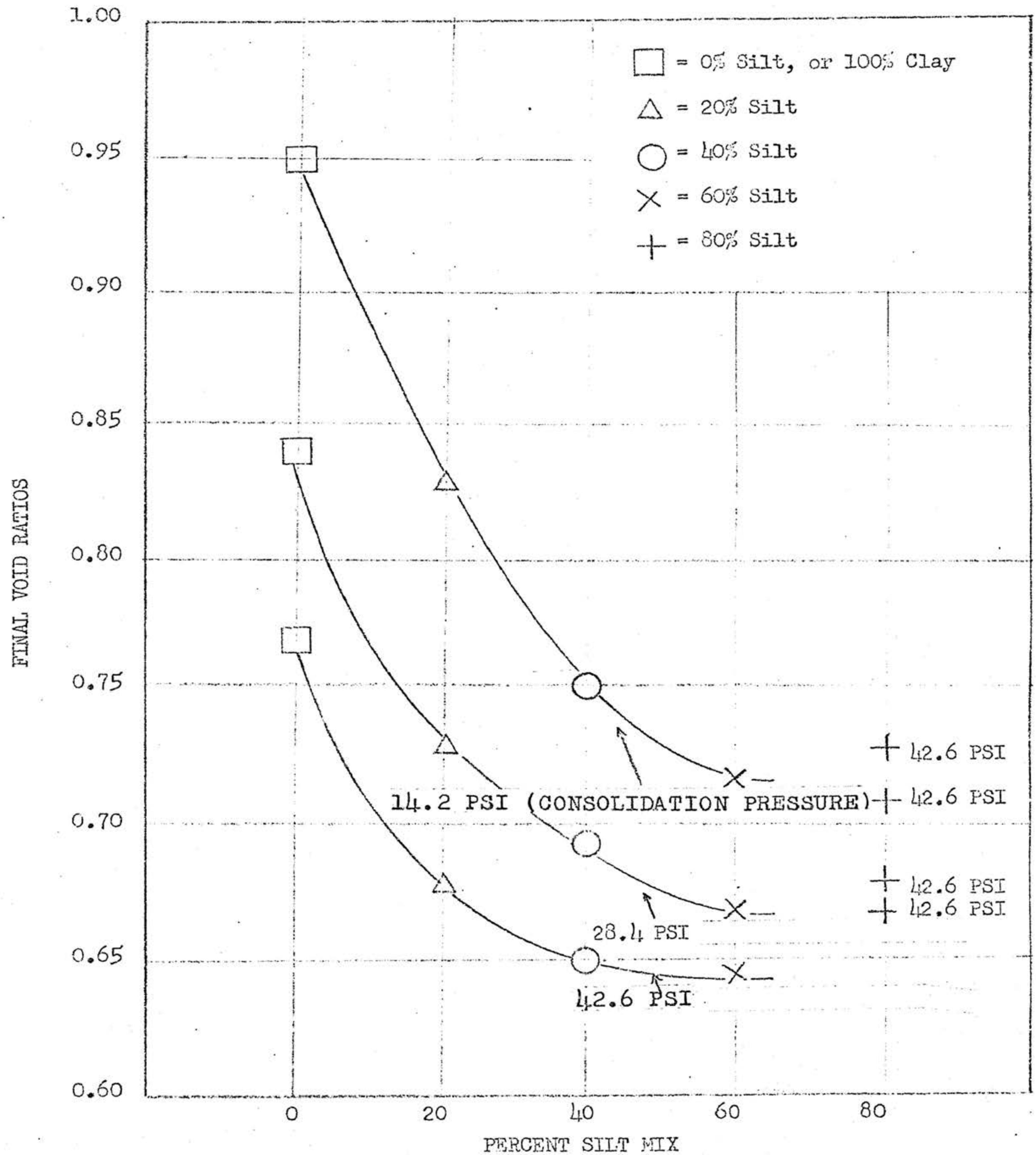


FIG. 8.-VOID RATIO vs SILT CLAY COMPOSITION.

strength of clay silt mixtures is independent of gradation until the mixture behaves like a granular material.

It must be noted that these tests were performed on artificial soil containing particles passing through the #200 U. S. Standard Sieve in which the width of the opening is 0.074 mm. It should also be noted that remarks regarding micro-structure of the mixes are hypothesized and were not subject to direct observation.

APPENDIX.--COMPUTER PROGRAMS

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 TRIAXIAL SHEAR TEST, INCLUDES PLOTTER PROGRAM FOR STRESS STRAIN &
1PORE PRESSURE STRAIN CURVES.
 DIMENSION SDIA(25),SLENG(25),NUMREA(25),WW(25,3),ALDR(25,30),WD(25
1,3),DDR(25,30),POKGCM(25,30),SIG3KG(25,30),POLBIN(25,30),WC(25,3),
1CM(25,3),EV(25,3),SG(25,3),Y1(25,30),Y2(25,30),X1(25,30)
 CALL PENPOS ('JACKSON, ALAN T.',16,1)
 CALL NEWPLT (3.0,2.5,10.0)
 CALL ORIGIN (0.0,0.0)
 CALL XSCALE (0.0,20.0,5.0)
 CALL YSCALE (0.0,40.0,6.0)
 CALL XAXIS(5.0)
 CALL YAXIS(10.0)
 CALL SYM (1.60,-0.5,0.14,'PERCENT STRAIN',0.0,14)
 CALL SYM (-0.6,0.6,0.14,'PORE PRESSURE(PSI)    )DEVIATOR STRESS(PSI
1)',90.0,42)
 CALL NUM (2.3,-.2,0.14,10.0,0.0,1)
 CALL NUM (4.8,-.2,0.14,20.0,0.0,1)          (KG'S QCM)
 CALL NUM (-0.5,2.9,0.14,20.0,0.0,1)          KGRSQCM
 CALL NUM (-0.5,5.9,0.14,40.0,0.0-1)
 READ(1,1)NUMSAM
 READ(1,4) (NUMREA(I),I=1,NUMSAM)
 READ(1,3) (SDIA(I),SLENG(I),I=1,NUMSAM)
 DO 33 J=1,NUMSAM
 WRITE(3,131)J
 WRITE(3,97)
 WRITE(3,999)
 NUMBER=NUMREA(J)
 READ(1,5)(ALDR(J,L),DDR(J,L),POKGCM(J,L),SIG3KG(J,L),L=1,NUMBER)
 X1(1,J)=0  Percent
 Y1(1,J)=0  P.P.
 Y2(1,J)=0  DRP.
 USTRES=0.0
 PERCEN=0.0
 DELTAU=0.0
 SIG3LB=SIG3KG(J,1)*14.22
 POLBIN(J,1)=POKGCM(J,1)*14.22
 SIG3EF=SIG3LB-POLBIN(J,1)
 SIG1EF=SIG3EF
 STSRAT=SIG1EF/SIG3EF
 ACOEF=0.0
 WRITE(3,96)USTRES,PERCEN,POLBIN(J,1),DELTAU,SIG3EF,SIG1EF,STSRAT,
1ACOEF
96 FORMAT(/6X,F6.2,3X,F6.2,3X,F6.2,3X,F6.2,3X,F6.2,3X,F6.2,3X,F6.2,3X
1,F5.2)
 DO 10 I=2,NUMBER
 SAREAI=(3.1415*(SDIA(J)/2.)**2)/6.45
 POLBIN(J,I)=POKGCM(J,I)*14.22
 DELTAU=POLBIN(J,I)-POLBIN(J,1)
 USTRN=DDR(J,I)/(SLENG(J)/2.54)
 CORARE=SAREAI/(1.-USTRN)
```

```
      PROVING RING NO. 63260
  26 TOTLD=(.504*ALDR(J,I))
  11 USTRES=TOTLD/CORARE
      ACOEF=DELTAU/USTRES
      SIG3LB=SIG3KG(J,I)*14.22
      SIG3EF=SIG3LB-POLBIN(J,I)
      SIG1EF=SIG3EF+USTRES
      PERCEN=(DDR(J,I)/(SLENG(J)/2.54))*100.
      STSRAT=SIG1EF/SIG3EF
      WRITE(3,98)USTRES,PERCEN,POLBIN(J,I),DELTAU,SIG3EF,SIG1EF,STSRAT,
     1ACOEF
      X1(I,J)=PERCEN
      Y1(I,J)=USTRES
      Y2(I,J)=DELTAU
  10 CONTINUE
      DO 36 KK=1,3
      READ(1,5)(WW(J,KK),WD(J,KK),WC(J,KK),SG(J,KK))
      CM(J,KK)=(WW(J,KK)-WD(J,KK))/(WD(J,KK)-WC(J,KK))
      EV(J,KK)=SG(J,KK)*CM(J,KK)
  36 CONTINUE
      WRITE(3,60)
  60 FORMAT(////6X,'FINAL VOID RATIOS')
      WRITE(3,61)(EV(J,KK),KK=1,3)
  61 FORMAT(11X,F5.2,/11X,F5.2,/11X,F5.2)
  33 CONTINUE
   1 FORMAT(I10)
   3 FORMAT(2F10.0)
   4 FORMAT(7I10)
   5 FORMAT(4F10.0)
  97 FORMAT(/6X,'USTRES',3X,'PERCEN',3X,'POLBIN',3X,'DELTAU',3X,'SIG3EF
     1'3X'SIG1EF',3X,'STSRAT',3X,'ACOEF')
  98 FORMAT(6X,F6.2,3X,F6.2,3X,F6.2,3X,F6.2,3X,F6.2,3X,F6.2,3X,F6.2,3X,
     1F5.2)
 131 FORMAT('1',6X,'RESULTS OF R-BAR TRIAXIAL TEST PERFORMED ON 100% SA
     1T SAMP. NO.',I3)
 999 FORMAT(7X,'(PSI)',4X,'(%)',6X,'(PSI)',4X,'(PSI)',4X,'(PSI)',4X,'(P
     1SI)')
      DO 1000 J=1,NUMSAM
      CALL XYPLT(X1(1,J),Y1(1,J),NUMREA(J),1,2)
1000 CALL XYPLT(X1(1,J),Y2(1,J),NUMREA(J),1,0)
      CALL ENDPLT
      CALL LSTPLT
      STOP
      END
```

```
      TRIAXIAL SHEAR TEST, PLOTTER PROGRAM FOR EFFECTIVE STRESS CIRCLES.
      DIMENSION SIG3EF(10),SIG1EF(10),Y(32),X(32)
      CALL PENPOS ('JACKSON, ALAN T.',16,1)
      CALL NEWPLT (3.0,3.0,10.0)
      CALL ORIGIN (0.0,0.0)
      CALL XSCALE (0.0,50.0,5.0)
      CALL YSCALE (0.0,25.0,2.5)
      CALL XAXIS(5.0)
      CALL YAXIS(5.0)
      CALL SYM(0.50,-0.5,0.14,'EFFECTIVE NORMAL STRESS(PSI)',0.0,28)
      CALL SYM(-.6,0.30,0.14,'SHEAR STRESS(PSI)',90.0,17)
      CALL NUM(2.3,-0.2,0.14,25.0,0.0,1)
      CALL NUM(4.6,-0.2,0.14,50.0,0.0,1)
      CALL NUM(-0.5,1.0,0.14,10.0,0.0,1)
      CALL NUM(-0.5,2.4,0.14,25.0,0.0,1)
      READ(1,1)NUMSAM
      READ(1,2)(SIG3EF(L),SIG1EF(L),L=1,NUMSAM)
      DO 10 I=1,NUMSAM
      C=(SIG1EF(I)+SIG3EF(I))/2
      RA=(SIG1EF(I)-SIG3EF(I))/2
      AIN=3.141593/30.0
      THA=0.0
      DO 1000 J=1,31
      X(J)=C+COS(THA)*RA
      Y(J)=RA*SIN(THA)
      THA=THA+AIN
 1000 CONTINUE
   10 CALL XYPLT(X,Y,31,1,-1)
    1 FORMAT(I10)
    2 FORMAT(2F10.0)
      CALL ENDPLT
      CALL LSTPLT
      STOP
      END
```

VITA

Alan T. Jackson was born on September 9, 1933, at Saginaw, Michigan. He is the son of a Naval Officer. He graduated from Woodstock High School, Mussoorie, United Provinces, India in October, 1951. He received a Bachelor of Science Degree in Civil Engineering from Virginia Military Institute in 1956 and then entered active military service as a Lieutenant in the Corps of Engineers.

Duty assignments have included Fort Belvoir, Virginia, Japan, Korea, West Germany, Vietnam, with temporary duty in England, Holland, and Russia. He was assigned to the University of Missouri at Rolla in January, 1967, to obtain his Master of Science Degree.

Major Jackson is married to the former Mary Josephine Parker of Raleigh, North Carolina. They have two daughters and a son, Jennifer, Marianne, and Mark Wendell.