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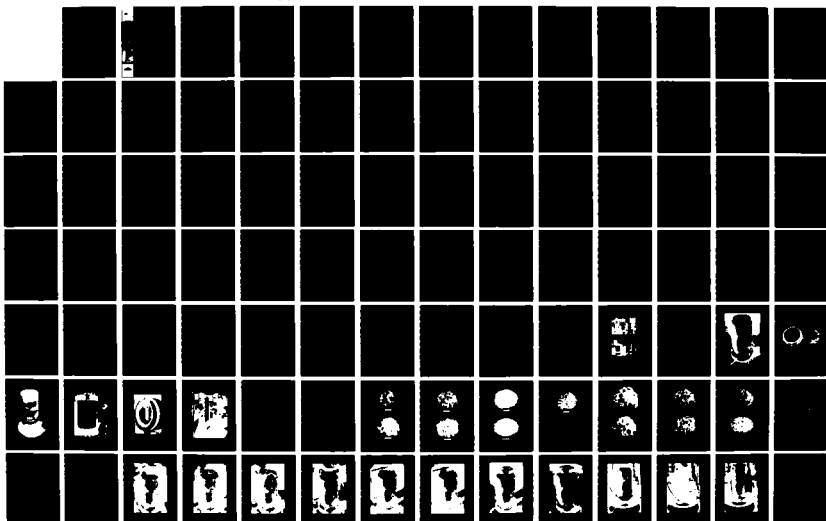
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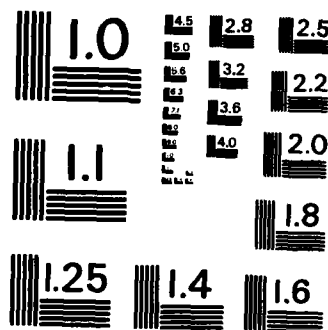
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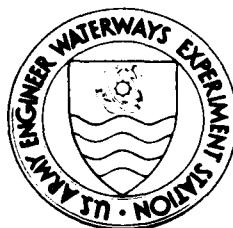
STRENGTH AND DEFORMATION PROPERTIES OF EARTH-ROCK MIXTURES

by

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Geotechnical Laboratory

DEPARTMENT OF THE ARMY
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20. ABSTRACT (Continued).

dictated by the maximum particle size of the full-scale gradation. The objective was achieved by comparing results of Q and $\bar{R}$ triaxial tests performed on 15-in.-diam by 38.5-in.-tall specimens of a full-scale gradation with those obtained from 6-in.-diam by 13.6-in.-tall specimens of scalped/replaced and scalped gradations derived from the full-scale gradation.

Three artificial full-scale earth-rock gradations (3-in. maximum particle size) were created by blending three percentages (20, 40, and 60) by weight of washed gravel (GP), three percentages (55, 35, 15) of mortar sand (SP), and a constant percentage (25) of clay (CL). The three full-scale gradations so created represented a range of earth-rock materials used in constructing many earthen dams. Scalping/replacement (to 3/4-in. maximum particle size) and scalping (minus No. 4 fractions) procedures were then applied to the full-scale gradations to assess effects on strength-deformation characteristics. Tests were performed at confining pressures of 4.22 and 14.06 kg/cm². Effects due to equipment and specimen size were assessed by also testing scalped/replaced specimens 15 in. in diameter. A separate suite of Q triaxial tests was performed on scalped material compacted to the initial conditions of the minus No. 4 fractions of corresponding scalped/replaced specimens to determine the contribution of gravel fractions to strength. Finally, a separate series of $\bar{R}$ triaxial tests was performed on the natural earth-rock gradation (3-in. maximum particle size, 48 percent gravel, 26 percent clay) from DeGray Dam as a check on trends observed for the blended materials. The full-scale and scalped/replaced specimens were compacted to 95 percent of their respective standard effort maximum dry densities at water contents obtained by combining minus No. 4 fractions prepared at their respective standard effort optimum water contents plus 1 percentage point with saturated surface-dry gravel. Scalped specimens were tested at 95 percent of their respective maximum dry densities and at water contents corresponding to their optimums plus 1 percentage point. Data presented include stress-strain and pore pressure-strain curves, effective stress paths, strength envelopes based on total and effective stresses, and comparative trend plots.

For Q and $\bar{R}$ tests, it was concluded that neither scalping/replacing nor scalping procedures yield satisfactory estimates of total stress strength parameters for the parent full-scale gradations. $\bar{R}$ tests on scalped/replaced or scalped gradations may significantly underestimate the strength of the full-scale material, whereas Q tests on the altered gradations may overestimate the true strength. The DeGray Dam material exhibited higher plasticity of the minus No. 40 fraction than that of the blended material (identical PL but PI 10 percentage points higher) and somewhat less difference between full-scale and scalped/replaced strength data. Effect of plasticity of the fines was not a program objective but results suggest additional research to determine if there exists a level of plasticity of fines above which scalped/replaced gradations can be reliable for estimating full-scale gradation strengths. The scalping/replacing procedure is satisfactory for estimating effective stress strength parameters of full-scale materials. Earth-rock mixtures compacted to 95 percent standard effort maximum dry density and near optimum water content can be expected to develop considerable pore pressure during undrained shear which results in total stress angles of internal friction as low as 11 deg. Effects due to differences in sizes of specimens or equipment were not significant.

Additional research to establish effects of plasticity of fines on both triaxial shear parameters and compaction test data between full-scale and scalped/replaced gradations is recommended. Research is also needed to eliminate equipment size effects on earth-rock compaction test results which have been documented by Donaghe and Townsend.

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PREFACE

This investigation was conducted at the US Army Engineer Waterways Experiment Station (WES) as Work Unit 31209 under the Office, Chief of Engineers, Civil Works Investigational Studies (CWIS). The testing was performed during the period October 1977 through December 1981.

The testing was executed by Mr. R. T. Donaghe with the assistance of Mr. C. E. Hills, Soils Research Facility, Soils Research Center, Soil Mechanics Division (SMD), Geotechnical Laboratory (GL), under the technical direction of the principal investigator, Dr. V. H. Torrey III, Research Group, SMD, and the general direction of Mr. J. P. Sale, former Chief, GL; Dr. W. F. Marcuson III, Chief, GL; and Mr. C. L. McAnear, Chief, SMD, GL. OCE Technical Monitor was Mr. R. F. Davidson. Mr. Donaghe and Dr. Torrey were the authors of the report. This report was edited by Ms. Odell F. Allen, Publications and Graphic Arts Division.

COL John L. Cannon, CE; COL Nelson P. Conover, CE; COL Tilford C. Creel, CE; and COL Robert C. Lee, CE, were Commanders and Directors of WES during the conduct of the study. COL Allen F. Grum, USA, was Director of WES during the preparation and publication of this report. Mr. Fred R. Brown and Dr. Robert W. Whalin were Technical Directors.

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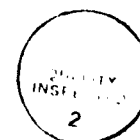
CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
pounds (force) per square inch	8,694.757	pascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2000 pounds, mass) per square foot	9,764.856	kilograms per square metre

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STRENGTH AND DEFORMATION PROPERTIES OF EARTH-ROCK MIXTURES

PART I: INTRODUCTION

Background

1. Over about the last 40 years there has been a considerable increase in usage of earth-rock materials in the fills of high dams and other embankments. Such soils consist of a heterogeneous mixture of particles ranging in size from as coarse as boulders^{*} or gravel to as fine as clay. The relative proportions of fine and coarse fractions, the characteristics of the fine fraction, and the maximum particle size influence the engineering properties and behavior exhibited by a particular total gradation. The presence of a significant fraction of particles larger than 1 in.** in diameter causes extreme difficulty in the assessment of strength and stress-deformation properties by conventional laboratory triaxial tests. The complications arise from the fact that the diameter of laboratory triaxial specimens must exceed the maximum particle size of the material by a sufficient ratio (usually at least 6:1) to yield reliable results. Consequently, unless alternate procedures are employed, large particle sizes dictate specimen proportions which exceed the physical capabilities of most modern soil laboratories as well as making testing economically impractical on a routine basis for those laboratories with the capability. This situation poses two questions: (a) Is it possible to obtain the required knowledge of properties of earth-rock mixtures by laboratory tests on gradations for which the maximum particle size is less than that in the actual fill material? and (b) What method is practical and reliable for

* Terms used in this report relative to classification of soils or fractional components are according to the Unified Soil Classification System (USAEWES Technical Memorandum 3-357, April 1960).

** A table of factors for converting non-SI units of measurement to SI (metric) units is given on page 3.

modeling an earth-rock mixture for laboratory triaxial testing without sacrificing the accuracy of design data as compared to that obtained for the full-scale gradation? Where the US Army Corps of Engineers (USACE) is concerned, alternate procedures to triaxial testing of full-scale earth-rock gradations have been given in EM 1110-2-1906, "Laboratory Soils Testing," (USACE 1970) since its original issue in 1965. Specifically, in Appendix X of EM 1110-2-1906, it is stated that the maximum particle size permitted in any triaxial specimen shall be no greater than one-sixth of the specimen diameter. Therefore, it may be necessary to remove particles larger than a certain size to comply with this requirement if the laboratory does not have equipment permitting testing of the full-scale gradation. The manual further directs that particles considered oversized on the basis of available equipment and the specified minimum ratio of particle size to specimen diameter should be removed (scalped) and, if comprising more than 10 percent by weight of the total sample, be replaced by an equal percentage by weight of material retained on the No. 4 sieve and passing the maximum allowable sieve size. These procedures were based on previous work by various Corps laboratories and others employing large-diameter triaxial specimens. However, these studies were either project related or were research programs not specifically designed to investigate the objectives of the work reported herein.

Overview of Work by Others

General

2. With a period of expansion in the construction of high earth and rock-fill dams beginning in the mid 1930's and the associated increase in use of earth-rock fill materials came the realization that existing laboratory equipment and procedures were not adequate to obtain engineering properties and behavior characteristics of such soils. First, test chambers and loading capacities were not sufficiently large to accept specimens containing larger particles. In addition, much larger specimen confining pressures were dictated to simulate stress

levels imposed by higher embankments. The earliest research in the US directed at developing large-scale triaxial test equipment and procedures was conducted by the USACE South Pacific Division Laboratory (SPDL) (Hall 1951, Leslie 1963) and the US Bureau of Reclamation (USBR) Earth Laboratory Branch (Noell 1953, Holtz and Gibbs 1956). The USACE's large-scale testing was mostly in support of specific project needs while the USBR undertook a broader research program into the physical properties of soils containing gravel. Donaghe and Cohen (1978) summarized results of large-scale triaxial tests on rock-fill materials obtained over several years by the SPDL. Cunney, Strohm, and Hadala (1964) of the USACE Waterways Experiment Station (WES) studied effects of large-size particles on the strength of a clay gravel. By the mid 1960's, the California Department of Water Resources was also testing large-scale specimens of rock-fill material used in constructing Oroville Dam. Hall and Smith (1971) reported special tests performed by the SPDL for the California Department of Transportation, and Prysock (1979) continued large-scale investigations by that organization with its own equipment. Marachi, Chan, and Seed (1972) reported tests on very large specimens of rock-fill and provided a good summary of conclusions reached by other investigators relative to the effects of particle size.

3. Outside the US triaxial testing on large-diameter specimens of a gravel moraine used in the construction of Portage Dam, Canada, was reported by Morgan and Harris (1967). Also in Canada, Insley and Hillis (1965) and Skermer and Hillis (1970) described results from specimens of glacial till used in Mica Dam. In Mexico, the properties of rock-fill material for El Infiernillo and Chicoasen Dams were determined from tests on very large (44.5-in.-diam by 98-in.-high) triaxial specimens (Marsal 1965, 1967; Marsal and Moreno 1979). Charles and Watts (1980) addressed the influence of confining pressure on the shear strength of compacted rock-fill.

4. In overview, the research programs cited above employed specimens ranging in diameter from 6 to 44.5 in., a minimum height-to-diameter ratio of 2, and confining pressures, $\bar{\sigma}_{3c}$, from only 7 psi to

as much as 650 psi. In some of the earliest studies, maximum particle sizes tested were too large compared to specimen diameter to yield reliable results on the basis of the current state-of-the-art. None of the efforts found in the literature (including others not cited above) were specifically designed to determine the feasibility of estimating strength-deformation properties of full-scale gradations of earth-rock mixtures using an alteration of that gradation with smaller maximum particle sizes and, consequently, smaller diameter triaxial specimens. Therefore, there are but a few findings considered especially pertinent to the objectives of the investigations, and these items are reported herein.

Effects of specimen
size on triaxial results

5. In comparing the results of SPDL testing in different types of equipment, primarily on free-draining gravels and boulders, Leslie (1963) found a linear relationship between density and effective angle of internal friction, $\bar{\phi}$. Furthermore, he stated that the different sizes of equipment and/or specimens produced consistent trends for all soils tested and that results could be replicated. In a study of effects of specimen size on measured shear strength, Holtz and Gibbs (1956) concluded that for a material containing 3/4-in. maximum particle size, tests on 6- and 9-in.-diam specimens gave similar results, while 3.25-in. specimens exhibited higher strength and were, therefore, too small. Morgan and Harris (1967) also compared results from 6- and 12-in.-diam specimens of a 5/16-in. (7.94 mm or between the US No. 3 and 3/8-in. sieve sizes) maximum particle size soil and concluded that there were no significant differences attributable to specimen size.

Influence of gradation and maximum
particle size on shear strength

6. Cohesionless gradations. Holtz and Gibbs (1956) found for sand and gravel mixtures that maximum particle size from 3/4 to 3 in. had little effect on shearing resistance because the larger particles were actually so relatively few in number. In addition, they varied gravel content and concluded that the effective angle of internal

friction, $\bar{\phi}$, increased with gravel content up to 50 or 60 percent, depending on maximum particle size. They stated that higher gravel contents produced little or no increase in strength; in fact, some reduction in strength may occur as a result of the material becoming less well graded. Leslie (1963) conducted a somewhat inconclusive study of the influence of gradation and maximum particle size on triaxial results. He reported testing artificially graded materials derived from one parent soil and a naturally graded soil from which oversize particles (+3 in., +1-1/2 in., +1 in.) were successively removed (scalped). One test series reflected increasing strength with increasing coefficient of uniformity, C_u , and density, with maximum strength developed for the 1-in. maximum particle size case. A second series yielded maximum strength for a 1/4-in. maximum particle size. In a third series, Leslie saw little effect from removing oversize particles. Marachi, Chan, and Seed (1972) concluded that the effective angle of internal friction, $\bar{\phi}$, was affected to some extent by the size of particles in the test specimen, i.e., gradation. They stated that at any given confining pressure, the angle of internal friction for 36-in.-diam specimens (6-in. maximum particle size) was about 1 to 1.5 deg lower than that of the 12-in.-diam specimens (2-in. maximum particle size) and 3 to 4 deg lower than that of 2.8-in.-diam specimens (0.45-in. maximum particle size). Therefore, each specimen diameter represented a different gradation of the three materials tested with the coefficient of uniformity increasing with maximum particle size. Gravel content for 2.8-in.-diam specimens of the three materials tested ranged from 27 to 38 percent, for the 12-in.-diam specimens from 70 to 84 percent, and for the 36-in.-diam specimens from 82 to 95 percent gravel and cobbles. Donaghe and Cohen (1978), working with several sand-gravel mixtures up to 60 percent gravel content, reported that strength did not change significantly with increasing maximum particle size up to 3 in. for a constant value of coefficient of uniformity. However, for increasing values of C_u , strength was found to increase over a range of maximum particle size up to almost 1 in. Little increase in strength was noted above 1-in. maximum particle size.

7. Gradations with plastic fines. Such gradations are termed earth-rock mixtures herein. Holtz and Ellis (1961) tested 9-in.-diam specimens of four approximately parallel gradations of clayey sand to clayey gravel to determine the effects of gravel content. All specimens were prepared to optimum water content and compacted to 95 percent of the standard maximum dry density. The specimens were tested at optimum water content, i.e., partially saturated and undrained. Gravel contents tested were zero (minus No. 4 material with about 23 percent plastic fines), 20 percent (3-in. maximum particle size and 21 percent plastic fines), 35 percent (3-in. maximum size and 18 percent fines), 50 percent (3-in. maximum size and 16 percent fines), and 65 percent (3-in. maximum size and 9 percent fines). Even though the presence of such large particles in 9-in.-diam specimens compromises the results, the general conclusion that the effective angle of internal friction, ϕ , increased with gravel content appears valid. In addition, Holtz and Ellis stated that as the gravel content of the clay soil was increased, a rather abrupt change in shear strength occurred when sufficient gravel was added to provide large particle interference. Furthermore, they observed that the friction angle increased and the cohesion decreased only small amounts when 20 to 35 percent gravel was added, indicating very little large particle interference and shear properties similar to the clay matrix. Kawakami and Abe (1970) worked with a sandy clay in which they varied the content of the 2- to 5-mm-size (coarse sand) particles while maintaining constant water content. In terms of effective strength parameters, they found little variation in ϕ up to about 40 percent coarse sand content and marked increases for higher fractions. For the cohesion intercept based on effective stresses, c' , a slightly increasing trend was observed up to 45 percent coarse sand, followed by a rapidly declining trend to zero at 75 percent coarse sand and above. In terms of total stresses, the trend in ϕ was similar to the effective stress case, while the cohesion intercept based on total stresses, c , was observed to increase slightly up to 45 percent coarse sand and markedly increase thereafter. The authors do not state how the total stress parameters were selected from the test data, and it

is possible that high values of the cohesion intercept based on total stresses, c , at higher coarse sand contents reflected negative induced pore water pressure. Kawakami and Abe concluded that the range in coarse sand content reflecting clay matrix property domination was up to 40 percent, followed by a transition toward the sand properties between 40 and 75 percent, and domination of the sand properties above 75 percent. Fedorov and Sergevnina (1973) reported two series of large-scale direct shear tests on coarse-fragmented soils (rubbly, gravelly, sandy) and clay soils with inclusions of coarse-fragmented materials. For the inclusion study, an apparatus with a ring diameter of 50 cm and height of 28.5 cm was used to test inclusions ranging in size from 2 to 80 mm and for rubble-gravel-sand contents of 25, 35, 50, and 70 percent. For the mixed gradation study, a smaller apparatus was employed to test maximum particle sizes from 2 to 10 mm and rubble-gravel contents from 20 to 90 percent. For the inclusions study, the authors concluded that (a) the main factor affecting shear strength was the ratio of content by weight of the rubble-gravel inclusions to the clay filler; (b) the effect of inclusions began at 20 percent rubble-gravel content and the strength rose steadily for higher content; (c) the size of the inclusions did not affect the strength (maximum particle size); and (d) an increase in the Plasticity Index (PI) of the clay filler produced a decrease in the angle of internal friction and the cohesion. For the mixed-gradation study, the authors stated that (a) the angle of internal friction increased and the cohesion decreased with increasing rubble and gravel content, and (b) when the rubble and gravel content was less than 20 percent, the soil properties were those of the clay filler.

Influence of gradation on compaction

8. Cunny, Strohm, and Hadala (1964) reported compaction tests using three sizes of mold (4-in.-, 6-in.-, and 12.25-in.-diam) on gravelly soils with cohesive fines. Seven gradations of clay gravel were tested with maximum particle size up to 3 in. One series was performed to determine the effect of maximum particle size and mold diameter on maximum standard dry density and optimum water content of the minus 3-in. material and of scalped samples without replacement of the

coarse fraction. A second test series was performed to determine the effect of maximum particle size and mold diameter on maximum standard dry density and optimum water content of scalped material with replacement for the coarse fraction. A third series was designed to determine the maximum density and optimum water content of minus 1-1/2-in. material in which portions of the minus No. 4 material had been removed. The authors concluded that mold diameter appeared to have no significant effect, but the writers of this report believe that the test procedures used may have masked the effects of equipment size. Cunny, Strohm, and Hadala detected no effects of maximum particle size on compaction results for specimens containing the same percentages of gravel. They stated that the total density of the specimens increased with gravel content up to the maximum examined of 70 percent. However, the density of the minus No. 4 fraction, as computed using Shockley's equation (1948), decreased steadily as gravel content increased. This would imply that density increase with gravel content is primarily the result of the higher specific gravity of the gravel fraction. Donaghe and Townsend (1973, 1975) reported extensive study of the compaction characteristics of earth-rock mixtures. Gravel contents from 0 to 100 percent and maximum particle size of 3 in. were utilized to compare compaction results on full-scale gradations (18-in.-diam mold) to results from corresponding scalped and replaced (minus 3/4 in.) specimens (6-in.-diam mold). Their conclusions most pertinent to the study reported herein were as follows:

- a. Moisture-density curves varied with the size of compaction equipment used.
- b. The scalping and replacement procedure resulted in a lower maximum dry unit weight and a higher optimum water content than obtained for the full-scale sample.
- c. For tests in which the gravel content was varied while the fines content was maintained constant (the cause for the study reported herein), the optimum gravel content, i.e., the gravel content producing the highest density, was decreased from 40 percent to between 10 and 20 percent by

the scalping and replacement procedure. Thus, in the range of gravel contents from between 10 and 20 percent to 40 percent, the difference between maximum dry unit weights determined for full-scale and for scalped and replaced specimens increased with increasing gravel content.

- d. The influence of fines content on compaction characteristics was found to be much greater than that of gravel content.

Influence of gradation on volumetric strain tendency during shear

9. Cohesionless gradations, i.e., rock-fill. Marachi, Chan, and Seed (1972) described relatively high compressibility and strong contractive volumetric strain during shearing for confining stresses above about 10 tsf. The authors do not provide relative density data for the three rock-fill materials tested, but void ratios prior to isotropic consolidation were all less than 0.45. Donaghe and Cohen (1978), using three rock-fill materials varying in mineral content and particle hardness, reported contractive volumetric strain even for specimens existing at void ratios representing more than 100 percent relative density and over a range of consolidation stresses from about 4.0 to 35 tsf. Other investigators have observed similar trends.

10. Gradations with plastic fines. Holtz and Ellis (1961) observed significant positive pore pressure development in earth-rock mixtures compacted to 95 percent of maximum standard dry density and sheared under initial effective confining stresses above about 3 tsf. Hall and Gordon (1964) also observed large pore pressures for earth-rock mixture specimens compacted to at least 95 percent of maximum standard dry density and subjected to initial effective confining pressures of 4 tsf or higher. Insley and Hillis (1965), working over a range of initial effective confining stress of from 4 to 33 tsf with earth-rock mixtures compacted to a minimum of 96 percent of maximum standard dry density, reported contractive volumetric strain in consolidated drained tests and considerable increase in the Skempton "A" pore pressure parameter in undrained tests with increasing confining stress. The total stress angle

of internal friction, ϕ , obtained from the above-cited studies, ranges from as low as 9 deg to about 18 deg. Such low values over the range of consolidation stresses and gravel contents used clearly indicate the effects of positive pore pressures generated for densities corresponding to about 95 percent of maximum standard dry density.

Laboratory scalping/
replacement procedure

11. The writers of this report have previously stated that no studies were found in the literature specifically directed at assessing the effects on strength-deformation properties of testing smaller maximum particle size gradations derived from full-scale earth-rock gradations. Insofar as current practice in the USACE, EM 1110-2-1906 specifies that for triaxial tests performed on materials containing plus No. 4 sieve sizes, the initial height of the specimen must be no less than 2.25 times the diameter and the maximum particle size shall be no greater than 1/6 the diameter. The manual further states that it may be necessary to remove particles larger than a certain size to comply with specimen dimension requirements. Oversize particles should be removed (scalped) and, if comprising more than 10 percent by weight of the sample, be replaced by an equal percentage by weight of material retained on the No. 4 sieve and passing the maximum allowable sieve size (that sieve size representing particle diameter of 1/6 the specimen diameter). The removal of larger particles resulting in lower measured values of the shear strength is a warning and should be avoided if possible. As far as the writers can determine, this scalping or scalping with replacement procedure was translated to triaxial testing from previous studies of the compaction characteristics of earth-rock mixtures. The evolution of the current manual requirements can be traced through the minutes of the USACE Division Laboratory Conferences. In the conference of June 1950, the subject of large particles in compaction tests was addressed. Mr. T. A. Middlebrooks of the Office, Chief of Engineers, indicated that he believed that it was not necessary to obtain too accurate results in general on materials with larger particles, but that if it

were necessary, larger specimens should be compacted. He cited the large-scale triaxial tests then being performed by the SPDL as an example. In the conference of November 1957, the subject arose again, and it was stated that various Division Laboratories were using a 6-in. compaction mold after removing particles larger than $3/4$ in., i.e., scalping. Most of these laboratories then computed the theoretical density and water content of the total sample accounting for the scalped material. Some laboratories were replacing material larger than $3/4$ in. with material of smaller size, usually No. 4 to $3/4$ in. It was suggested at that conference (1957) that for compaction of cohesive soils containing gravel, a computed maximum density be used to account for scalped gravel if the plus No. 4 fraction did not exceed about 15 percent of the total sample. In 1964, Cunney and Strohm of WES performed a compaction study on Kettle Creek Dam earth-rock mixtures. They concluded that influence of gravel content on compaction results began to occur at about 5 percent. In the Division Soils Laboratories Conference of April 1968, the Office, Chief of Engineers, stated that the problem of scalping and replacing oversize particles in compaction tests would be researched in a new engineering study at WES on earth-rock mixtures. In the interim, the decision was made that if 10 percent or less of the total material was retained on the No. 4 sieve, replacement was not to be considered necessary. The engineering study referred to in the 1968 conference is reported by Donaghe and Townsend (1973, 1975). Their conclusions were previously given in paragraph 8.

Purpose, Scope, and Testing Program

12. The purpose of this study was to obtain information to improve current methods of estimating the strength of earth-rock mixtures and to develop methodology regarding modeling of earth-rock mixtures for laboratory testing. Specifically, the objective was to determine the validity of two widely used modeling techniques (scalping and scalping with replacement) in cases where available testing equipment sizes are

smaller than those dictated by the maximum particle size of the full-scale gradation.

13. The approach to achieving the objective was to compare results of $\bar{R}$ and Q strain-controlled triaxial tests performed on 15-in.-diam by 38.5-in.-tall specimens of a full-scale gradation (a material having 3-in. maximum particle size) with those obtained from 6-in.-diam by 13.6-in.-tall specimens of scalped/replaced and scalped gradations derived from the full-scale gradation. Three artificial full-scale earth-rock gradations were created by blending three percentages by weight of washed gravel (GP), three percentages of mortar sand (SP), and a constant percentage of clay (CL). The three full-scale gradations were representative of a range of earth-rock materials used in the construction of many earthen dams. Scalping/replacement and scalping procedures were then applied to the full-scale gradations to assess effects on strength-deformation characteristics.

14. The full-scale and scalped/replaced specimens were compacted to 95 percent of their respective maximum standard effort dry densities and at water contents obtained by combining the minus No. 4 sieve fractions prepared at optimum plus 1 percent water content (based on standard effort compaction tests performed on the minus No. 4 sieve fractions) with saturated surface-dry gravel. The full-scale specimens were compacted to 95 percent of the maximum dry density because this is a common field compaction standard. The scalped/replaced specimens were compacted to the 95 percent density under the initial assumption that the modeled material would have the same compaction and strength characteristics as the full-scale material. Scalped specimens (minus No. 4 sieve fractions of full-scale gradations) were tested at 95 percent of their respective maximum standard dry densities and at water contents corresponding to their optimum water contents plus 1 percentage point. These compaction standards for the scalped specimens were adopted because this modeling procedure assumes that full-scale characteristics are a function of the minus No. 4 sieve fraction. There is, however, an additional reason for choosing the 95 percent maximum standard dry density condition. Some basis was required for comparing results of

tests on different gradations. For cohesionless soils it is common practice to compare results from specimens prepared to the same relative density. In the case of earth-rock mixtures, it then is logical to compare specimens prepared to equal percent compaction based on the respective maximum dry densities for the tested gradation.

15. Tests on blended materials were performed at confining pressures of 4.22 and 14.06 kg/cm². Two confining pressures were selected to define the materials' behavior over a range while restricting the cost of the program to meet funding limitations. The higher confining pressure was selected to be as high as practicable considering the maximum WES laboratory house pressure of 21.09 kg/cm² and allowing for pressure required for back pressure saturation. A suite of Q triaxial tests was performed on scalped specimens of blended material compacted to the initial conditions of the minus No. 4 sieve fractions of corresponding scalped/replaced specimens to assess the contribution of gravel fractions to unconsolidated undrained strength. Finally, a separate suite of $\bar{R}$ triaxial tests was also performed at confining pressures of 2.11, 4.22, and 8.44 kg/cm² on a natural earth-rock mixture used in the construction of DeGray Dam, Vicksburg District, USACE, to determine the validity of the findings for the artificial blended material. The testing program is outlined in Tables 1 and 2.

PART II: TEST EQUIPMENT

General

16. Photographs of the small- and large-scale testing stations used in the investigation are given in Figure 1. Schematic diagrams of the pressure control and measurement systems are shown in Figures 2 and 3. Detailed description of the testing equipment is given in the following paragraphs.

Large-Scale Equipment

17. The cast-aluminum split mold shown in Figure 4 was used to form specimens 15 in. in diameter by 38.5 in. in height. The inner surface of the mold was formed by a 0.074-in.-thick vulcanized rubber membrane held in place against the mold sides by a vacuum acting through an underlying thin porous plastic liner. The mold was attached to the specimen base platen with bolts.

18. The top and base platens for the full-scale specimens are shown in Figure 5. They were made of steel and equipped with 15-in.-diam aluminum specimen bearing plates which could be replaced as wear or damage dictated. Specimen drainage was provided by 1/8-in.-diam holes in the plates and channels on the backside which guided water to the central drainage ports in the top and base platens. Filter discs of Whatman No. 1 Filter Paper were placed between the specimen and the bearing plates to prevent the movement of fines out of the specimen and into the pore water pressure and volume change measurement system. The top and base platens were provided with ports to which external specimen drainage lines were connected.

19. Full-scale specimens were encased in two membranes. The inner membrane was the 0.074-in.-thick rubber one which formed the inner surface of the split mold. After the specimen was positioned and secured in the triaxial chamber base and the mold removed, an outer membrane of 0.025-in.-thick latex was placed over the specimen using the large

membrane stretcher shown in Figure 6. Inner membranes were sometimes punctured by gravel particles during testing. Rather than attempting to locate and repair the punctures (a lengthy task), an outer, less expensive membrane was used to seal the specimen. (Inner membranes were also purposely punctured in several places to ensure that air was not trapped between the membranes.)

20. The triaxial chamber used to house the full-scale specimens is shown in Figure 7. The chamber was designed for a working pressure of 500 psi and was sealed at the top and bottom with 3/8-in.-diam "O" rings placed in compression using eighteen 1-in.-diam bolts. The piston for applying axial force passed through the chamber base. The piston was thrust upward by a hydraulic actuator centered beneath the triaxial base in a pit below floor level. Figure 8 shows the round steel plate screwed onto the end of the piston upon which the specimen base platen was positioned and secured with bolts. Axial force applied by the actuator was measured by a 100-kip capacity annular-shaped (donut type) electronic load cell mounted on top of the chamber. Axial force was transferred to the load cell by a shaft which passed through the top of the chamber. The shaft was equipped with a tapered foot having a 6-in.-diam flat circular face which, when brought into contact with the flat bottom of the corresponding opposite tapered recess in the specimen top platen, prevented significant tilting of the top platen during loading. This was done to maintain definition of the major principal stresses during shear. The donut load cell was selected because its axial load measurement accuracy was not affected by the presence of any lateral thrust on the shaft.

21. Axial forces were applied by a 270-kip capacity hydraulic actuator controlled by an electrohydraulic closed-loop system (MTS Systems, Inc.). Axial deformations were measured by a linear variable differential transformer (LVDT) attached to the actuator piston. Induced pore water pressures were measured at the bottom of the specimens by means of an electronic differential pressure transducer which sensed the difference between the back pressure required for saturation and the total pore water pressure in the specimen. Chamber pressures were

measured at the base of the chamber using an electronic pressure transducer. The volume by water entering or leaving the specimen during saturation and consolidation phases of the tests was determined by measuring the change in water level in 3-in.-diam aluminum tubes serving as burettes and connected to the specimen by wire-wound high pressure hoses. The changes in water level in the tubes were measured with high-sensitivity electronic differential pressure transducers which detected the difference between the pneumatic pressure (back pressure) acting on the water in the tubes and the total pressure at the bottom of the tubes. The outputs of the electronic sensors were monitored by digital voltmeters and recorded by analog recorders (Hewlett-Packard Model 7046A X-Y recorder and Model 7700, stylus heat, 8-channel strip chart recorder). The electronic sensors were selected on the basis of their calibrated accuracy and range such that values of stress, strain, and pressures cited in this report are accurate to the decimal places given. The data acquisition equipment and MTS System controls for the large- and small-scale testing stations are shown in Figure 9.

Small-Scale Equipment

22. Scalped/replaced and minus No. 4 fraction (scalped) specimens were compacted in an aluminum split mold which provided specimens 6 in. in diameter and 13.6 in. in height. A thin Teflon lining formed the inner surface of the mold. The top and base platens were similar to those for the full-scale specimens except they were made of aluminum and the platens and attached perforated bearing plates were both of the same diameter as the specimen. The base platen was attached to the triaxial chamber base and the split mold fit over the base platen so that after placement of a Whatman No. 1 Filter Paper Disc upon the perforated bearing plate, the specimen could be compacted directly on the base platen. Since the specimens were free-standing after removal of the split mold, they were each encased in three 0.025-in.-thick latex membranes after molding. Three membranes were required to avoid loss of the specimens during loading as a result of puncture of the outermost membrane by

gravel particles in the specimen. As in the case of the full-scale tests, the two innermost membranes were deliberately punctured to prevent entrapment of air between the membranes. The aluminum triaxial chamber was designed for a working pressure of 500 psi. The specimens were axially loaded through the chamber top plate by a piston attached to a 22-kip capacity hydraulic actuator. Equipment used to control the actuator and for measurement of test variables was similar to that used for the full-scale specimens.

PART III: MATERIALS

General

23. The two earth-rock mixtures tested in this investigation were an artificially blended material and a natural material obtained from Borrow Area "F" associated with the construction of DeGray Dam, Caddo River, Ark.

Blended Material

24. Blended samples were prepared by combining a subrounded to subangular mortar sand (SP) with a clay (CL) and subrounded to subangular washed gravel (GP) having a maximum particle size of 3 in. to provide the three full-scale gradations shown in Figure 10. Those gradations were the result of fixing the clay fraction at 25 percent and varying the gravel content from 20 to 40 to 60 percent. Based on the full-scale gradations, the three scalped/replaced gradations also shown in Figure 10 were determined after the procedure given in EM 1110-2-1906, "Laboratory Soils Testing," Appendix VIA, and produced by blending the appropriate fractions. In addition, gradations of the minus No. 4 fractions (scalped gradations, Figure 10) of full-scale gradations were determined and produced by blending appropriate fractions of the sand and clay. Classification data for the clay and sand are given in Figure 11. Gradation curves for the gravel fractions of full-scale and scalped/replaced specimens are given in Figure 12. It is seen in Figure 12 that the gravel fractions of the three full-scale materials conformed to a single gradation while gravel fractions of scalped/replaced specimens varied somewhat as a result of the replacement procedure used (described in Part IV of this report). Atterberg limits for the minus No. 40 fraction of the blended full-scale and scalped/replaced gradations are Liquid Limit (LL) = 27 and Plastic Limit (PL) = 14. Prior to use in the testing program, the sand and clay were spread and dried on a heated floor. After drying, the clay was processed through a hammer mill to

break down any aggregations. The washed gravel fraction was a composite of three materials and was sieved into the following size ranges: plus 3-in. (discarded), 3- to 2-in., 2- to 1-1/2-in., 1-1/2- to 1-in., 1- to 3/4-in., 3/4- to 1/2-in., 1/2- to 3/8-in., and 3/8-in. to No. 4 sieve. The 3- to 2-in. sizes were a California placer (alluvial) gravel; the 2- to 1-1/2-in. sizes were DeGray Dam gravel; and the 1-1/2-in. to No. 4 sieve sizes were a locally obtained gravel. The apparent specific gravity, G_a , of the composite gravels is 2.65, and the bulk specific gravity is 2.58. Photographs of the various washed gravel fractions are shown in Figures 13 through 19.

DeGray Dam Material

25. The DeGray Dam material is classified as a clayey sandy gravel (GC) with maximum particle size of 6 in. Atterberg limits for the minus No. 40 fraction are LL = 37 and PL = 14. The as-tested gradation curves are shown in Figure 20. The material was first spread and dried on a heated floor. After drying, the soil was screened into the following fractions: plus 3-in. (discarded), 3- to 2-in., 2- to 1-1/2-in., 1-1/2- to 1-in., 1- to 3/4-in., 3/4- to 1/2-in., 1/2- to 3/8-in., 3/8-in. to plus No. 4, and minus No. 4. Any aggregations in the dried material were readily broken down by hand during the screening. The plus 3-in. fraction which was discarded represented less than 10 percent by weight of the total material. The plus No. 4 material was washed to remove fines adhering to the larger particles. A negligible quantity of fines was lost in the washing process. The plus No. 4 fraction was composed of subrounded to subangular particles. The apparent specific gravity of the gravel fraction is 2.61, and the bulk specific gravity is 2.50. Photographs of the various gravel fractions are given as Figures 21 through 24.

Compaction Characteristics

26. Standard effort compaction tests were performed on all

full-scale, scalped/replaced, and scalped (minus No. 4 fractions of full-scale) gradations of both blended and DeGray materials in previous investigations of compaction characteristics of earth-rock mixtures reported by Donaghe and Townsend (1973, 1975). Results of these tests are summarized in Table 3. The moisture-density curves are given in Figures 25 and 26.

PART IV: TEST PROCEDURES

Specimen Preparation

27. Both full-scale and small-scale specimens were compacted in seven equal weight layers using sufficient impact effort to achieve the desired density of 95 percent of maximum standard dry density for the given gradation. Overcompaction of the lower layers of the specimens was avoided by deliberately undercompacting lower layers and allowing effort applied to succeeding layers to bring the lower layers to the target density as the specimen preparation was completed. The degree of initial undercompaction was greatest for the first or lowermost layer and was decreased with each succeeding layer. Each layer was batched separately to prevent any variation in gradation among layers. The layer batches were prepared by thoroughly mixing a predetermined amount of sand and clay (minus No. 4 sieve material) with a measured quantity of water sufficient to bring the mixture to its own standard optimum water content plus 1 percentage point and then storing the wetted material in airtight containers for a period of at least 16 hr. The plus No. 4 sieve material (gravel) for each batch was prepared by combining the air-dry portion (by weight) of gravel required for each sieve size and then storing the resulting mixture in containers filled with water. Immediately prior to compaction, the gravel was drained on a No. 4 sieve and patted dry with paper towels. The cured minus No. 4 fraction was thoroughly mixed with the saturated surface-dry aggregate. In strength testing for design of compacted fills, it is customary to compact specimens to 95 percent of maximum standard effort dry density at optimum water content plus 2 percentage points. Therefore, considering the water (absorption) in the aggregate fraction, the minus No. 4 fraction was prepared at optimum water content plus 1 percentage point so that the total combined gradation would not be wetter than optimum plus 2 percentage points. The optimum water content plus 2 percentage points is a reasonable representation of field conditions assuming that fill placement will generally be no wetter, and strength parameters on

the wet side will be lower than on the dry side.

28. Figures 27 through 30 are photographs taken during compaction of a 15-in.-diam full-scale specimen. A filter disc was placed on the perforated bearing plate attached to the specimen base platen. The material for each layer was placed in the mold (Figure 27) and tamped to the desired height using the hand-held rammer shown in Figure 28. The distance from the top of the mold to the soil surface (Figure 29) was periodically measured at several points to determine when the layer was compacted to the desired height. The surface of each completed layer, except the top layer, was scarified before compaction of the subsequent layer. After tamping the top layer of each specimen to within about 1/4 in. above the top of the mold, a steel plate was placed on the soil (Figure 30) and struck vertically with the end surface of a heavy steel bar to bring the soil layer flush with the top of the mold. A bubble level was used during the procedure to assure that the soil surface remained horizontal. Figure 31 shows a specimen after compaction. Following compaction, a filter disc and the top perforated bearing plate were placed on the specimen (Figure 32) and the top platen was lowered into place (Figure 33). The vulcanized rubber membrane lining the mold was then pulled up over the top platen and secured with an "O" ring (Figure 34). Next, the mold and specimen were placed on the actuator piston plate at the base of the triaxial chamber (Figure 35) and a vacuum was applied to the specimen through the base platen. The mold was then removed (Figure 36) and, if new, the rubber membrane was punctured (this did not relieve the vacuum in the specimen significantly). The outer latex membrane (Figure 37) was then placed over the specimen and secured with clamped "O" rings.

29. The procedure for compacting 6-in.-diam specimens was similar to that described above for full-scale specimens except that a 10-lb sliding weight rammer was used for compaction and the top layer was trimmed flush with the top of the mold. Following compaction, the Teflon lined mold was carefully removed and, with the specimen free-standing, three 0.025-in.-thick latex membranes (the inner two punctured) were placed and secured to the specimen top and base platens with

"O" rings. A full (~ 14 psi) vacuum was then applied to the specimen through the top platen.

Specimen Dimension Measurements

30. Specimen dimensions were determined after vacuum within the specimens had stabilized. Specimen height was taken as the average of four measurements at 90-deg intervals around the circumference of the specimen with a metal scale reading to 0.01 in. Diameter measurements were taken at the top, middle, and bottom quarter points of the specimen using a "PI" tape reading to the nearest 0.001 in. A "PI" tape is a thin and narrow flexible metal scale graduated and verniered so that the diameter may be read when the tape is drawn around the circumference of the specimen. The diameter of the specimen was taken as the weighted average of the sum of the top reading, twice the center reading, and the bottom reading. A value of twice the thickness of the membrane was subtracted from each reading before computing the weighted average.

31. After specimen initial dimensions were determined, the tri-axial chamber was assembled and an initial piston LVDT reading taken with the piston in contact with the specimen top platen or, in the case of full-scale specimens, with the specimen top platen in contact with the foot on the load cell shaft. Subsequent LVDT readings made with the piston or load cell shaft in contact with the specimen top platen were used to determine the change in height of the specimen during saturation, consolidation, and axial loading. The chamber was filled with water after the initial LVDT reading was obtained.

R Triaxial Tests

Back pressure saturation

32. Saturation of the specimens was initiated by allowing de-aired water (produced by a Nold deaerator) to percolate slowly from the bottom to the top of the specimen under a differential vacuum head.

During this procedure, the vacuum acting on the bottom of the specimen was regulated so that the differential vacuum between top and bottom of the specimen never exceeded 5.0 psi. When water appeared in the burette through which a high vacuum was being maintained at the top of the specimen, the valve to the bottom of the specimen was closed and the back pressure procedure described below conducted to complete saturation. If movement of water through the specimen was very slow, i.e., if the rate of flow indicated that water would not pass through within about 3 hr, the valve to the bottom of the specimen was closed after about 30 min and back-pressure saturation initiated.

33. After the valve to the bottom of the specimen was closed following the initial saturation phases given above, the valve to the top of the specimen was closed and the burette to the top of the specimen still under full vacuum was filled with de-aired water. The valve to the top of the specimen was then opened and the vacuum acting on the burette was slowly decreased with a simultaneous equivalent increase in chamber pressure. When the vacuum had been decreased to zero, i.e., atmospheric pressure (and the chamber pressure increased to approximately 14 psi), back-pressure saturation was initiated using an automatic electronic servo-control device. The device applied back pressure and chamber pressure concurrently in equal amounts in such a manner that the difference between the back pressure applied at the top of the specimen and the induced pore pressure at the bottom of the specimen never exceeded 5.0 psi. When the back pressure had been increased to a preset value (usually 60 psi), the automatic system maintained the pressures until a check of the Skempton pore pressure "B" parameter could be made. The "B" check was accomplished by closing the valve to the top of the specimen and manually increasing the chamber pressure by 10.0 psi. If the induced pore pressure was 9.5 psi or greater, i.e., a "B" value equal to 0.95 or greater, the specimen was assumed to be saturated. If the "B" value was less than 0.95, the automatic device was reset to again increase back pressure and chamber pressure to a higher preset value and the "B" value again checked. The procedure was continued until the measured "B" value was satisfactory. In most cases,

the saturation procedure required less than 24 hr.

Consolidation

34. Specimens were isotropically consolidated by increasing the chamber pressure, with valves to the top and bottom of the specimen closed, to the desired consolidation stress (the desired difference between chamber and back pressure) and then opening the drainage valves to the top and bottom of the specimen to allow water from the specimen to enter a burette. A plot of the volume of water expelled from the specimen versus the logarithm of time was used to determine the completion of consolidation. In all cases, secondary consolidation was permitted to continue at least 12 hr. Following consolidation, the drainage valves were closed and undrained shear was initiated.

Undrained shear

35. After consolidation, the actuator piston was moved into contact with the 6-in.-diam specimen top platen. In the case of the full-scale specimen, the specimen was moved upward with the actuator until the specimen top platen contacted the foot on the load cell shaft. LVDT readings were taken to obtain the height of the specimen upon which to base axial strain computations during shear. Specimens were then axially loaded at a constant rate of strain of 0.008 percent per minute until the maximum deviator stress was reached or until 1 percent axial strain was developed. The rate of strain was then increased so that 15 percent axial strain could be reached by the end of the work day. Following shear, the entire specimen was oven-dried for a final water content determination.

Q Triaxial Tests

36. After assembling the chamber and obtaining an initial piston LVDT reading, the vacuum acting on the specimen through the top platen was slowly relieved while the chamber pressure was simultaneously increased such that the sum of chamber pressure and internal vacuum remained constant. When the internal vacuum had been totally relieved, the bottom drainage valve was also opened. After allowing approximately

10 min for specimen conditions to stabilize, the specimen was brought into contact with the axial loading system, and an initial piston LVDT reading was taken. The drainage valves were then closed and the chamber pressure was increased to the desired confining pressure. After the chamber pressure was applied, another 10-min period was allowed for the specimen to stabilize. After this period, the specimen was once again brought into contact with the axial loading system, and a piston LVDT reading was taken to determine specimen height for axial strain computation. Specimens were then axially loaded at a constant rate of strain of 0.6 percent per minute until 15 percent or more axial strain was reached. After shear, the entire specimen was oven-dried for a final water content determination.

PART V: TEST RESULTS AND ANALYSES

General

37. Results of the $\bar{R}$ (consolidated, undrained) triaxial compression tests performed on the blended and DeGray gradations are summarized in Tables 4 and 5, respectively. Results of the Q (unconsolidated, undrained) triaxial tests performed on the blended material are summarized in Table 6. The test data are presented in the following paragraphs according to gravel content and confining pressure.

Specimen Conditions

38. Desired and actual specimen conditions are given in Tables 5 and 6. The average values of actual (as compacted) dry unit weights and minus No. 4 fraction water contents for each type of specimen (full-scale, scalped/replaced and scalped specimens) at each gravel content were within 1 pcf and 0.4 percentage points, respectively, of the desired values. For the DeGray material specimens, the initial values were within 1 pcf and 1.5 percentage points of the target dry unit weight and minus No. 4 fraction water content, respectively. The larger difference between actual and desired water contents in the DeGray specimens primarily reflects two specimens where because of batching error, the minus No. 4 water contents failed to match the desired values by more than 3 percentage points. Although these differences were large, the combination of high gravel content (48 percent) and the fact that the specimens were saturated tended to negate possible adverse effects, and it was therefore not deemed necessary to reperform the two tests.

39. The average initial dry unit weight of full-scale specimens of both materials was approximately 4 pcf higher than that of the corresponding scalped/replaced specimens even though specimens of both types were compacted to the same percent (95 percent maximum standard dry

density) compaction. The difference indicates the dissimilarity of the moisture-density curves of the full-scale as compared to scalped/replaced gradations. Relationships between percent compaction of minus No. 4 fractions and gravel content are given in Figure 38. Percent compaction of minus No. 4 fractions were based on maximum standard dry unit weights for each of the minus No. 4 fractions in question (see Table 3). The curves for full-scale and scalped/replaced specimens of blended material show by their relationship in position to the curve for scalped specimens (minus No. 4 fractions) that there was little interference (1 to 2 percent) due to gravel on the compaction of minus No. 4 fractions in full-scale specimens having gravel contents up to 40 percent, whereas there was significant (5 to 20 percent) interference due to gravel on the compaction of minus No. 4 fractions in the scalped/replaced specimens over the full range of gravel contents tested. This is attributed to the more uniform gradation of the gravel fractions in the scalped/replaced specimens. Previous investigations have observed that interference by gravel particles may become evident at gravel contents as low as 10 percent for materials having a 3/4-in. maximum particle size. The lower dry unit weights and associated greater volume change characteristics of scalped/replaced specimens resulted in significant differences in test results as compared to full-scale specimens as is shown in Tables 4, 5, and 6 and will be discussed in later portions of this report.

40. Initial specimen dry unit weights for the scalped (minus No. 4 sieve fractions) gradations of the blended soil decreased with increasing gravel content in the corresponding full-scale and scalped/replaced materials. This reflects the increasing ratio of clay fraction to sand fraction in the minus No. 4 (scalped) gradations as the gravel content in the parent full-scale gradations increased.

41. The initial specimen densities for the DeGray specimens were lower than those of corresponding gradations of the blended material at approximately the same gravel contents partly because of the lower bulk specific gravity, G_m , of the gravel particles in the DeGray materials.

The bulk specific gravity of the DeGray gravel fraction is 2.50 while that of the blended material is 2.58.

R Triaxial Tests

Blended material

42. Deviator stresses. The stress-strain curves of Figures 39 through 41 indicate that all of the specimens exhibited brittle behavior during undrained shear. The curves show that deviator stresses for each type of specimen and confining pressure generally peaked at low axial strains (0.4 to 2.9 percent) and then in many instances, after decreasing, began increasing at strains of 1 to 4 percent to the end of the test. The deviator stress at the initial peak of the stress-strain curve was taken as that at failure. The increases in deviator stress after the initial peak are believed to reflect the inability of the specimens to continue failing along the initially formed shear planes. This occurred because the specimen top platen was restrained from tilting during shear by the equipment configuration. Tilt constraint was decided upon in order to maintain definition of major principal stresses throughout the shear portions of the tests.

43. Relationships between deviator stresses at failure and gravel content given in Figure 42 show that deviator stresses at failure for full-scale specimens were significantly higher than either the corresponding scalped/replaced or scalped specimens at the two lower gravel contents (20 to 40 percent) and approximately equal to those for the corresponding specimens at the 60 percent gravel content. The curves (Figure 42) also show that deviator stresses at failure decreased with increasing gravel content in the full-scale specimens for both confining pressures used (4.22 and 14.06 kg/cm²). For scalped/replaced and scalped gradations, the trends in deviator stress at failure were variable with both increasing gravel content in the parent full-scale gradations and confining pressure. The scalped/replaced specimens exhibited increasing strength with increasing gravel content in the parent

full-scale gradations and confining pressure. The scalped/replaced specimens exhibited increasing strength with increasing gravel content in the parent gradation for the lower confining pressure. However, at the higher confining pressure, strengths of scalped/replaced gradations declined slightly with increasing gravel content up to 40 percent and then increased with further increases in gravel content such that at 60 percent gravel content in the parent gradation the deviator stress at failure was nearly the same as for the 20 percent gravel specimen. The scalped specimens exhibited trends similar to the scalped/replaced specimens except reversed with respect to confining pressure. In addition, at the lower confining pressure and for the lowest percent gravel in the parent gradation (20 percent), the scalped specimen was slightly stronger than the corresponding scalped specimens at both confining pressures and with increasing gravel content in the parent gradations.

44. Differences among deviator stresses at failure among corresponding full-scale, scalped/replaced, and scalped specimens may be partially understood by comparing volumetric strains ($\Delta V/V$, %) during consolidation which, in turn, also reflects initial specimen conditions. Volumetric strains during consolidation were computed as follows:

$$\epsilon_{vc} = \frac{\Delta V_c}{V}$$

where

ϵ_{vc} = the volumetric strain during consolidation

ΔV_c = the change in specimen volume during consolidation
determined from changes in burette readings

V = the specimen volume at the end of back pressure
saturation

If it is assumed that a greater resistance to volume change during consolidation may be indicative of higher strength, then those data should support the trends in deviator stresses at failure (Figure 42). The respective volumetric strains during consolidation are given in Figure 43 which shows the above assumption to have merit. Volumetric strains

during consolidation for full-scale specimens containing 20 and 40 percent gravel were significantly lower than for corresponding scalped/replaced and scalped specimens. Another clear indicator of the comparative trends in deviator stress at failure between full-scale and scalped/replaced specimens is the density of the minus No. 4 fractions. Figure 44 shows the variation in percent compaction after consolidation of the minus No. 4 fractions over the range of gravel contents. It is seen in Figure 44 that for both confining pressures, the percent compactions (and also the absolute dry densities) of the minus No. 4 fractions of full-scale specimens after consolidation were consistently higher than those for companion scalped/replaced specimens. Of course, volumetric strains during consolidation reflect the same comparative differences in minus No. 4 fraction densities as initially compacted. The after-consolidation percent compactions for scalped specimens (minus No. 4 fractions of full-scale gradations) are also shown in Figure 44. However, while scalped/replaced specimens remained earth-rock mixtures, scalped specimens ranged from clayey sand (SC) to sandy clay (CL) as gravel content in the parent full-scale gradations varied from 20 to 60 percent. For this reason, the high percent compactions for scalped specimens shown in Figure 44 are not considered as a basis for comparison to corresponding full-scale and scalped/replaced deviator stresses at failure.

45. The stress-strain data for tests 8, 9, 12, and 13 which were performed on 15-in.-diam scalped/replaced gradations to assess specimen and equipment size effects show that, considering differences in density at the initiation of shear (Table 4), specimen and equipment size effects are probably negligible.

46. Induced pore pressures. Considerable excess pore pressure developed during shear for all gradations tested. However, Figures 39 through 41 show that there was less variation in induced pore pressures than in deviator stresses at failure. Curves of Skempton's "A" pore pressure parameter ($A = \Delta u / (\Delta \sigma_1 - \Delta \sigma_3)$, where Δu is the total induced pore water pressure) versus axial strain given in Figures 45 through 47 reveal a wide variation in A values for gravel contents of 20 and 40

percent, thus indicating large differences in volume change tendencies during undrained shear for the three types of specimens at lower gravel contents. Relationships between A_f , $[A_f = \Delta u_f / (\sigma_1 - \sigma_3)_f]$, and gravel content given in Figure 48 show that the relatively loose structures of scalped/replaced and scalped specimens produced A_f values ranging from 1.1 to 1.5, whereas A_f values for the full-scale material (0.7 to 1.3) did not reach such levels until the gravel content was increased to 60 percent. However, the scalped/replaced specimens exhibited an opposite trend from full-scale specimens in that A_f tended to decrease with increasing gravel content. The density of the minus No. 4 fraction of both scalped/replaced and full-scale specimens declined sharply with increasing gravel content which would suggest increasing A_f values for both cases. The only other difference between these two test series was the gradation of the gravel fractions. It seems logical, therefore, to suspect that the greater uniformity of gravel fractions of scalped/replaced specimens resulted in the decreasing trend in A_f with increasing gravel content above 40 percent.

47. The trend in A_f values for full-scale specimens suggests that if the gravel fraction is the lesser portion of the material, a less contractive structure may be obtained for a given compactive effort. Figure 49 which compares A_f and volumetric strain during consolidation, ϵ_{vc} , supports this supposition in that ϵ_{vc} values and the corresponding A_f values for full-scale specimens were indeed lower than those for the companion scalped/replaced and scalped specimens at the two lower gravel contents tested. As in the case for the deviator stresses at failure, the pore pressure data for 15-in.-diam scalped/replaced specimens indicate that, considering differences in specimen conditions at initiation of shear, specimen and equipment size effects are deemed negligible.

48. Effective stress parameters. Plots of effective principal stress ratio, $\bar{\sigma}_1/\bar{\sigma}_3$, versus axial strain given in Figures 45 through 47 show that full-scale specimens generally developed higher $\bar{\sigma}_1/\bar{\sigma}_3$ ratios, and scalped specimens developed lower. The curves of Figure 50

are based on averaged maximum $\bar{\sigma}_1/\bar{\sigma}_3$ ratios $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ at each gravel content and show that $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ values increased with increasing gravel content for both full-scale and scalped/replaced specimens. An average curve showing a similar trend for scalped specimens is also shown in Figure 50, but it is noted that comparatively large differences in $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ values at each confining pressure occurred. Average values of $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ varied from 3.7 to 4.7 for full-scale specimens while those for scalped/replaced specimens ranged from 3.5 to 4.3, and those for the scalped specimens ranged from 3.3 to 3.8.

49. The effective stress paths shown in Figures 51 through 53 reflect the considerable excess pore pressures developed during undrained shear for all of the gradations tested. Also seen from the effective stress paths is that the scalped specimens at all gravel contents and each of the full-scale and scalped/replaced specimens containing 60 percent gravel tend to dilate toward the end of shear, i.e., approaching 20 percent axial strain. Figures 54 through 56 are plots of maximum effective principal stress ratios, $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$, in q , $\bar{p}$ space where $q = [(\sigma_1 - \sigma_3)/2]$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ and $\bar{p} = [(\bar{\sigma}_1 + \bar{\sigma}_3)/2]$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$. In order to form a basis for comparison, a single $\bar{\alpha}$ -line (where $\tan \bar{\alpha}$ is the slope of the line and $\tan \bar{\alpha} = \sin \bar{\theta}$) was drawn from the origin through the stress points for the scalped/replaced specimens at each gravel content. This procedure implies no effective cohesion intercept, $\bar{c}$, and is seen to be acceptable when the effective stress paths at each confining pressure for each type of specimen are taken together. As may be seen, at each gravel content, the $\bar{\alpha}$ -lines through the scalped/replaced stress points also fit the scalped data fairly well. The full-scale data points, however, lie slightly above the lines. Values of $\bar{\theta}$ versus gravel content are plotted in Figure 57. The individual points in Figure 57 were obtained by calculating $\bar{\theta}$ from the relationship $\sin \bar{\theta} = \tan \bar{\alpha}$ where $\tan \bar{\alpha}$ is the slope of a line from the origin to each of the $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ points in Figures 54 through 56. For the full-scale and scalped/replaced specimens, curves are

drawn in Figure 57 through points at each gravel content at $\bar{\phi}$ representing an $\bar{\alpha}$ -line which best fits the stress points for the two confining pressures. The curves in Figure 57 show that $\bar{\phi}$ values for full-scale and scalped/replaced specimens increased with increasing gravel content and that values for the full-scale materials were consistently about 2 deg higher than for scalped/replaced materials. An average curve for scalped specimens is also shown in Figure 57, but comparatively large differences in $\bar{\phi}$ values occurred between the two confining pressures at each gravel content. It is possible that $\bar{\phi}$ values for the scalped specimens corresponding to 40 and 60 percent gravel content in the parent gradations and tested at $\bar{\sigma}_{3c} = 14.06 \text{ kg/cm}^2$ may have been too high because pore pressures measured at the ends of the specimens did not reflect those in the zone of shear. Strain rates may not have been slow enough to prevent pore pressure gradients with the specimens at the point in the tests when $(\bar{\sigma}_1/\bar{\sigma}_3)_{\text{max}}$ values were developed and pore pressures in the zone of shear where a tendency to dilate was occurring may have been lower than those measured at the ends of the specimens. Values of $\bar{\phi}$ ranged from 34.6 to 41.3 deg for the full-scale gradations and from 32.3 to 38.8 deg for scalped/replaced specimens. Those for scalped specimens ranged from 36.9 to 39.6 deg. There were no apparent effects on $\bar{\phi}$ values for full-scale or scalped/replaced materials due to increasing the confining pressure, i.e., there appears to have been no significant particle breakage. Considering the large excess pore pressures developed in these tests, the effective stresses were apparently low enough such that particle breakage during shear was not significant.

50. Total stress parameters. Relationships between $q = (\sigma_1 - \sigma_3)/2$ and $p = (\sigma_1 + \sigma_3)/2$ at $(\sigma_1 - \sigma_3)_f$ are given in Figures 58 through 60. As was the case for effective stresses, a single α -line was drawn from the origin which fit the total stress points for both scalped/replaced and scalped materials, while another line was required for full-scale specimens. The α -lines of Figures 58 through 60 indicate that

differences in α values (the angle of slope of the α -line) for the full-scale and the scalped/replaced materials diminished with increasing gravel content with the lines for the 60 percent gravel case being only about 1 deg apart for all three gradations. Since all of the α -lines clearly indicate envelopes through the origin, strength parameters are in the normally consolidated range. Curves for the angle of internal friction based on total stresses, ϕ , versus gravel content are given in Figure 57. Values of ϕ were computed using $\sin \phi = \tan \alpha$ where α is the angle of slope of the line from the origin to each of the stress points in Figures 58 through 60. The curves (Figure 57) show that ϕ angles for full-scale specimens ranged up to 7 deg higher than those for the corresponding scalped/replaced and scalped specimens. The greatest difference in ϕ values occurred at the lowest gravel content, and the differences diminished with increasing gravel content as particle interference with the compaction of the minus No. 4 fraction of full-scale specimens produced contractive structures more like those of the scalped/replaced and scalped specimens. The trends suggest that at lower gravel contents the scalping/replacement procedure may produce overly conservative estimates of total stress strength parameters of the full-scale material. The values of ϕ shown are quite low overall, reflecting the considerable excess pore pressures generated during shear. Values ranged from 13.6 to 18.0 deg for the full-scale specimens and from 10.7 to 13.4 deg for the scalped/replaced and scalped specimens. These results are consistent with the findings of Hall and Gordon (1964) who have reported $\phi = 16$ deg for an earth-rock mixture having 40 percent gravel, 31 percent fines (minus No. 200 sieve), and 1-1/2-in. maximum particle size (LL = 30, PI = 12).

DeGray Dam material

51. Deviator stresses. The stress-strain and induced pore pressure-strain curves for $\bar{R}$ tests on DeGray Dam material are given in Figures 61 through 63. The curves show that, as in the case of the blended material, both full-scale and scalped/replaced specimens exhibited brittle behavior during undrained shear with peak deviator stress, i.e., deviator stress at failure, $(\sigma_1 - \sigma_3)_f$, developing at 1 percent

axial strain or less. The stress-strain curves for scalped gradations, however, generally did not peak at low strains, but instead broke over rather sharply at low axial strain and then continued to rise gradually to the end of the test. These patterns were consistent over the three confining pressures used. With the exception of test No. 4 where $(\sigma_1 - \sigma_3)_f$ was taken at 2.9 percent axial strain, failure for scalped specimens was taken at 15 percent axial strain even though the increase in deviator stress occurring after the sharp break may have been a reflection of the restricted top platen (this would have been the practice in the Corps of Engineers routine testing).

52. Relationships between $(\sigma_1 - \sigma_3)_f$ and confining pressure, $\bar{\sigma}_{3c}$, are given in Figure 64. In Figure 64, little difference is seen among the deviator stresses at failure for full-scale as compared to those for scalped/replaced and scalped specimens over the range of confining pressures tested. The exceptions are test No. 2 which was performed to determine effects due to a small increase in density and test No. 7 where there was a somewhat greater increase in deviator stress after the initial break in the stress-strain curve. The small differences for the DeGray material are in contrast to the data for the blended material where similar small differences in deviator stresses at failure were not observed until gravel content was increased to 60 percent. Since the DeGray Dam material had a gravel content of 48 percent, it was expected that effects due to interference of gravel on the compaction of the minus No. 4 fraction of full-scale specimens would not be sufficient to result in a loose enough structure to produce behavior similar to that of scalped/replaced specimens. Figure 65 gives the percent compaction of minus No. 4 fractions of DeGray specimens after consolidation. In order to compare full-scale and scalped/replaced data for DeGray material with that for the blended material, the common confining pressure of 4.2 kg/cm^2 is taken as the basis. From Figure 65 it is seen that at $\bar{\sigma}_{3c} = 4.2 \text{ kg/cm}^2$, the percent compaction of the minus No. 4 fraction of the DeGray material full-scale specimen was about 2 percentage points higher than that of the same fraction of the

scalped/replaced specimen. Figure 44 also shows percent compaction of minus No. 4 fractions versus gravel content for the blended material at $\bar{\sigma}_{3c} = 4.22 \text{ kg/cm}^2$. Observing the implied difference between full-scale and scalped/replaced specimens at 48 percent gravel, i.e., at the gravel content of the DeGray material gradation, it is seen that the percent compaction difference for the blended materials was very nearly the same as for the DeGray material fractions. Therefore, the contrasting behavior of blended versus DeGray material was not explained on this basis. The authors see only one other probable source of the difference which is the plasticity of the minus No. 40 fractions of the two materials. The PI of the DeGray material is 10 percentage points higher than that of the blended material. Furthermore, it is seen from Figures 44 and 65 that at the common confining pressure of 4.22 kg/cm^2 , the percent compactions of the minus No. 4 fractions of the DeGray material full-scale and scalped/replaced specimens were lower than for the corresponding blended specimens. The implication, then, is that increasing plasticity of the fines results in interference at a lower gravel content of the gravel fraction on the compaction of the minus No. 4 fraction. For the blended material, the total stress angle of internal friction was up to 7 deg higher for full-scale specimens than for corresponding scalped/replaced or scalped specimens. For the blended material, it was true that full-scale and scalped/replaced specimens yielded nearly the same value of ϕ at a gravel content of 60 percent for both confining pressures used. However, the trends in ϕ versus gravel content for full-scale and scalped/replaced cases imply an intersection at the 60 percent gravel content rather than convergence. For the DeGray material with a gravel content of 48 percent, close agreement in total stress parameters was seen between full-scale and scalped/replaced specimens over the full range of confining pressure applied. The writers consider this to be coincidental and the result of effects of the higher plasticity of the minus No. 40 fraction of the DeGray material compared to the blended material (identical PL but PI 10 percentage points higher). The

writers are convinced that if a blended series were performed on a material with plasticity identical to the DeGray soil, trends in ϕ versus gravel content similar to those for the blended series reported herein would have been observed except that the intersection of the trends would have occurred near 48 percent gravel content and, perhaps, differences would have been smaller over the range of gravel content. If this were true, the implication is that increasing plasticity of the fines decreases the gravel content at which significant interference of the larger particles on compaction characteristics will occur. However, further research is needed to determine the effect of plasticity of fines on the behavior of earth-rock mixtures. With reference to test No. 2 also shown in Figure 65, it is seen that the $(\sigma_1 - \sigma_3)_f$ value for the full-scale specimen tested at 98 percent of the maximum standard effort density was approximately 20 percent higher than that of the corresponding full-scale specimen test at 95 percent of the maximum density. Since the after-consolidation densities of the two specimens were only 3.3 pcf apart, it appears that small increases in density of earth-rock mixtures may have a significant effect on strength.

53. Because there was so little difference among $(\sigma_1 - \sigma_3)_f$ values for the three gradations of the DeGray Dam material, it is difficult to determine whether the magnitudes of the $(\sigma_1 - \sigma_3)_f$ values were reflective of trends in volume change during consolidation. Further complicating the situation were greater differences between desired and actual initial specimen conditions for the DeGray material tests. The curves in Figure 66 show that at the confining pressure of 4.22 kg/cm^2 the volumetric strain during consolidation, ϵ_{vc} , of both the DeGray material full-scale and scalped/replaced specimens was about 3.8 percent. Referring to Figure 43 for the blended material at 4.22 kg/cm^2 and interpolating for 48 percent gravel content, the full-scale material has an implied volumetric strain during consolidation of about 0.8 percent and the scalped/replaced about 2.6 percent. Comparing the $(\sigma_1 - \sigma_3)_f$ values for the two materials on the same basis as above using Figures 42 and 64, it is seen that the implied $(\sigma_1 - \sigma_3)_f$ for the full-scale blended material is about 3.0 kg/cm^2 while for the DeGray material

it is about 2.0 kg/cm^2 which is consistent with the differences stated above between the volumetric strains during consolidation. Similarly, consistent differences are also seen for scalped/replaced cases. Again, the plasticity of the fines may explain the volumetric strain differences between the two materials.

54. Induced pore pressures. As in the case of the blended material gradations, considerable excess pore pressure developed during shear for each of the DeGray Dam material gradations tested. Curves of induced pore pressure versus axial strain (Figures 61 through 63) indicate that induced pore pressures developed similarly in full-scale and scalped/replaced specimens at each $\bar{\sigma}_{3c}$ value. For scalped specimens, maximum induced pore pressures were comparable to the other types of specimens, but they did not develop as quickly, i.e., at low axial strains. Relationships between A_f and confining pressure $\bar{\sigma}_{3c}$ given in Figure 67 show that if effects of differences in initial specimen conditions for full-scale and scalped/replaced at $\bar{\sigma}_{3c} = 2.11 \text{ kg/cm}^2$ are considered, A_f values for full-scale specimens were approximately equal to those for both scalped/replaced and scalped specimens and did not change appreciably over the range of $\bar{\sigma}_{3c}$ tested. The comparatively large difference between A_f values for the full-scale and scalped/replaced specimens at $\bar{\sigma}_{3c} = 2.11 \text{ kg/cm}^2$ probably reflects the 3 percentage point difference in initial percent compaction between the specimens. If the A_f values for the full-scale specimens tested at $\bar{\sigma}_{3c} = 2.11 \text{ kg/cm}^2$ are not included, A_f values for all of the tests varied only from 1.1 to 1.3. In the case of the blended material, A_f values indicated that specimens of each gradation developed approximately equal volume change tendencies during undrained shear at a gravel content of approximately 60 percent. Since the DeGray Dam material exhibited similar tendencies at the lower gravel content of 48 percent, the higher plasticity of the fines may be the reason. Insofar as the full-scale specimen tested at 98 percent of maximum standard effort dry density and at $\bar{\sigma}_{3c} = 2.11 \text{ kg/cm}^2$, the effects of only 3.3 pcf

increase in density are evident in the decrease in A_f from 0.8 to 0.60.

55. Effective stress parameters. Curves of $\bar{\sigma}_1/\bar{\sigma}_3$ versus axial strain, ϵ_1 , given in Figures 68 through 70 show that specimens of all three gradations of the DeGray material developed approximately equal $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ values at $\bar{\sigma}_{3c} = 2.11$ and 4.22 kg/cm^2 , whereas, at $\bar{\sigma}_{3c} = 8.44 \text{ kg/cm}^2$, the $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ values for the scalped specimens were slightly less than the corresponding full-scale and scalped/replaced specimens. Figure 71 which gives the relationship between $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ and $\bar{\sigma}_{3c}$ shows that $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ values decreased when $\bar{\sigma}_{3c}$ was increased from 2.11 to 4.22 kg/cm^2 and then remained relatively unchanged when the confining pressure was further increased to 8.44 kg/cm^2 . This implies that specimens of each gradation tested at $\bar{\sigma}_{3c} = 2.11 \text{ kg/cm}^2$ were probably overconsolidated. The value of $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ at $\bar{\sigma}_{3c} = 8.44 \text{ kg/cm}^2$ for the full-scale and scalped/replaced specimens was 4.6. The blended material specimens did not develop approximately equal $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ values until the gravel content was increased to 60 percent. The effective stress paths of Figure 72 also reflect the considerable excess pore pressure which can apparently be anticipated from earth-rock mixtures during undrained shear when compacted to 95 percent of standard effort maximum dry density. The stress paths also reveal that specimens of each gradation tended to dilate toward the end of shear at each confining pressure as did the blended material at the 60 percent gravel content.

56. Values of $\bar{p}$ and q taken at $(\bar{\sigma}_1/\bar{\sigma}_3)_{\max}$ are plotted for the DeGray material specimens in Figure 73. A single $\bar{\alpha}$ -line could be drawn from the origin through the stress points for both the full-scale and scalped/replaced specimens. Stress points for the scalped gradation fell slightly below that line. Curves of $\bar{\phi}$ versus $\bar{\sigma}_{3c}$ are given in Figure 74. Values of $\bar{\phi}$ were computed as previously described for blended material. The values of $\bar{\phi}$ for full-scale and scalped/replaced

specimens were nearly equal over the range as $\bar{\sigma}_{3c}$ was increased from 2.11 to 4.22 kg/cm² and remained about the same as $\bar{\sigma}_{3c}$ was further increased to 8.44 kg/cm². The $\bar{\phi}$ values for the scalped specimens were approximately equal to those for the full-scale and scalped/replaced specimens at the two lower levels of $\bar{\sigma}_{3c}$ and about 3 deg lower than the corresponding values at the higher $\bar{\sigma}_{3c}$. Specifically, $\bar{\phi}$ ranged from 40 to 42 deg for full-scale and scalped/replaced specimens and from 37 to 42 deg for the scalped specimens.

57. Total stress parameters. Stress points of q versus p in terms of total stresses and computed at $(\sigma_1 - \sigma_3)_f$ are plotted in Figure 75. For the highest confining pressure (8.44 kg/cm²), a single α -line may be drawn from the origin through the stress points for all three DeGray material gradations. The stress points for full-scale and scalped/replaced specimens fell slightly above the α -line at the lower confining pressure and just below at the intermediate confining pressure. Stress points for scalped specimens were all slightly above the line. Curves of the total stress angle of internal friction, $\bar{\phi}$, versus confining pressure, $\bar{\sigma}_{3c}$, are also shown in Figure 74. It is seen that $\bar{\phi}$ values for full-scale and scalped/replaced specimens were approximately equal over the range of confining pressures and that the $\bar{\phi}$ values for tests at $\bar{\sigma}_{3c} = 2.11$ kg/cm² were about 3 deg higher than those at $\bar{\sigma}_{3c} = 4.22$ and 8.4 kg/cm². There, then, is no indication of particle breakage affecting results at the higher confining pressure. The $\bar{\phi}$ values for the scalped gradation did not change appreciably with increasing confining pressure and were nearly equal to those of the full-scale and scalped/replaced specimens at $\bar{\sigma}_{3c} = 2.11$ kg/cm² and about 2 deg higher at $\bar{\sigma}_{3c} = 4.22$ and 8.44 kg/cm². Values of $\bar{\phi}$ reflected the high induced pore pressures and ranged from 12 to 17 deg for full-scale and scalped/replaced specimens and from 14 to 16 deg for the scalped specimens. Therefore, for the DeGray material, total stress strength parameters obtained from tests on scalped/replaced gradations were equivalent to full-scale parameters while those from scalped

gradations were slightly higher at confining pressures above 4.22 kg/cm^2 .

Q Triaxial Tests on Blended Materials

Deviator stresses

58. The stress-strain curves for the Q triaxial tests on blended material are given in Figures 76 through 81. Test specimen No. 17 developed a leak during application of the confining pressure, and stress-strain data were considered invalid. There was insufficient material for a retest. The curves in Figures 76 through 81 show that full-scale specimens developed deviator stresses during undrained shear lying between those for scalped/replaced and scalped specimens at each gravel content and confining pressure. The curves show that specimens of each type generally exhibited a more plastic behavior during undrained shear in the Q tests than in the $\bar{R}$ tests with deviator stress in most cases still increasing at 15 percent axial strain which was assumed as failure. The curves indicate that the 20 percent gravel content specimens generally developed maximum deviator stresses at lower axial strains than the two higher gravel content specimens of each type.

59. Relationships between deviator stress at failure and gravel content (Figure 82) show a large increase in deviator stress at failure with increasing gravel content for both full-scale and scalped/replaced specimens at each confining pressure. Because of their increasing clay content, deviator stresses at failure for the minus No. 4 fraction specimens decreased when the gravel content in their parent materials was increased from 20 to 40 percent; however, the stresses increased when gravel contents were further increased from 40 to 60 percent. This increase in strength is thought to reflect a lower degree of saturation for the minus No. 4 sieve fraction specimens corresponding to the full-scale and scalped/replaced specimens having 60 percent gravel. Deviator stresses at failure for scalped/replaced specimens were approximately 15 percent higher than those for full-scale specimens over

the range of gravel contents tested at each confining pressure, while those for scalped specimens ranged from 25 to 65 percent lower than those for the corresponding full-scale specimens with the greatest difference occurring at 60 percent gravel content at $\sigma_3 = 14.1 \text{ kg/cm}^2$.

Deviator stresses at failure for the scalped/replaced specimens ranged from 6.4 to 13.4 kg/cm^2 at $\sigma_3 = 4.22 \text{ kg/cm}^2$ and from 19.0 to 37.5 kg/cm^2 at $\sigma_3 = 14.06 \text{ kg/cm}^2$.

60. As in the case of the $\bar{R}$ tests, differences in deviator stress at failure between full-scale, scalped/replaced and scalped specimens may be at least partially explained by resistance to volume change occurring as a result of the application of confining pressure. The volumetric strain values plotted in Figure 83 and tabulated in Table 6 were computed as follows:

$$\epsilon_{vst} = \frac{\Delta V_{st}}{V_o}$$

where

ϵ_{vst} = the volumetric strain during the period allowed for stabilization of the specimen prior to shear

ΔV_{st} = the change in specimen volume during the period of stabilization determined from specimen height changes using the assumption that axial and radial strains are equal

V_o = the measured specimen volume prior to application of the confining pressure

Figure 83 shows that for scalped/replaced specimens, volumetric strains occurring during the portion of the test when the specimens were allowed to reach equilibrium in an undrained state after the application of the desired chamber pressure were lower and corresponding deviator stresses higher than for full-scale specimens. This, of course, reflects the greater resistance to volume change during compaction exhibited by scalped/replaced materials as compared to the full-scale materials. However, considering that the full-scale specimens had comparatively high degrees of saturation after compaction, it was expected that they would have lower volumetric strains during undrained isotropic

compression than corresponding scalped/replaced specimens. Since such was not the case, it is apparent that the more uniform gravel size fraction of the scalped/replaced specimens was responsible for the greater resistance to volume change. Thus, it appears that the more uniform gravel fraction in scalped/replaced specimens hindered volume change during undrained compression in the Q tests and contributed to increased volumetric strain in the consolidation phase of the $\bar{R}$ tests, (see paragraph 44). Since the $\bar{R}$ specimens were saturated prior to the application of confining pressure, it appears that volume changes occurring as a result of gravel particles moving relative to one another are a function of the water content of the minus No. 4 fraction at the time of application of the confining pressure.

61. The contribution of the gravel fractions to strength in Q test specimens was investigated by comparing results of test Nos. 2 and 4, 6 and 8, 10 and 12, 14 and 16, 18 and 20, and 22 and 24 (see Table 6). Test Nos. 4, 8, 12, 16, 20, and 24 were performed on scalped specimens compacted to approximately the same initial dry density as the minus No. 4 fractions of corresponding scalped/replaced specimens. The scalped/replaced specimens were selected for comparison because strength data for those specimens indicated the gravel fractions to have the most pronounced effects. Figure 84 gives the results of the comparisons of the two data sets plotted in terms of percentage of deviator stress at failure contributed by the gravel fractions versus gravel content. As may be seen in Figure 84, the contribution to the deviator stress at failure by gravel fractions varied from 20 to 75 percent as gravel contents were increased from 20 to 60 percent. Therefore, in Q tests, gravel content exerts considerable influence on strength even when as low as 20 percent.

Angles of internal friction

62. Relationships between ϕ (computed in the same manner described in paragraph 57) and gravel content for the Q tests are given in Figure 85. Single lines were drawn to represent the relationships for full-scale and scalped/replaced materials, although ϕ values for

the $\sigma_3 = 4.22 \text{ kg/cm}^2$ tests were slightly higher than those for the $\sigma_3 = 14.06 \text{ kg/cm}^2$ tests in the case of both materials, thus indicating that the strength envelopes for these materials are curved (curved envelopes would be expected because of higher degrees of saturation occurring at higher confining pressure and, therefore, higher pore pressure). Since ϕ values for the scalped materials tested at $\sigma_3 = 14.06 \text{ kg/cm}^2$ were significantly lower than those tested at $\sigma_3 = 4.22 \text{ kg/cm}^2$, separate lines were drawn for each confining pressure to represent the relationships for scalped materials. Based on differences in ϕ values at equal σ_3 values, scalped specimens showed the greatest loss in strength with increasing confining pressure, and full-scale specimens showed the least. The curves in Figure 85 show that ϕ values for full-scale specimens increased from 24 to 33 deg as gravel contents were increased from 20 to 60 percent and were from 0.5 to 4 deg lower than those of the corresponding scalped/replaced specimens. The smallest difference in ϕ values occurred at 20 percent gravel content. The ϕ values for scalped specimens ranged from 1 to 6 deg lower than corresponding values for full-scale specimens at $\sigma_3 = 4.22 \text{ kg/cm}^2$ and from 5 to 14 deg lower at $\sigma_3 = 14.06 \text{ kg/cm}^2$ with the smallest differences between full-scale and scalped/replaced values also occurring at 20 percent gravel content. Since there was more variation in ϕ values for the Q than for the $\bar{R}$ tests, it is apparent that the degree of saturation was an important factor influencing strength. The relationship between ϕ values degree of saturation prior to shear (Figure 86) shows that there was an approximately linear relationship between the degree of saturation and ϕ for all of the tests on the three different types of materials.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

$\bar{R}$ triaxial tests

63. On the basis of results of the programs of $\bar{R}$ triaxial compression tests performed on the artificial blended earth-rock mixtures and the natural earth-rock mixture from DeGray damsite with initial specimen conditions given in this report, the following conclusions are drawn:

- a. The scalping and replacement procedure provides a satisfactory means for determining acceptable effective stress strength parameters for full-scale earth-rock gradations.
- b. The procedure in which tests are performed on minus No. 4 fractions of full-scale gradations yields effective stress parameters which are too conservative. Angles of internal friction based on effective stresses for such scalped materials may be up to 5 deg lower than those for parent full-scale materials.
- c. Earth-rock mixtures may develop considerable pore pressures during undrained shear when compacted to approximately 95 percent of standard effort maximum dry density. As a result, apparent angles of internal friction based on total stresses may be as low as 11 deg. Because of strongly contractive behavior at 95 percent compaction, both materials were in a normally consolidated state at confining pressures of 4.22 kg/cm² and above. However, based on the one higher density test performed in this study and on previous work by WES on earth-rock material associated with a specific project study, it is apparent that small increases in absolute density, 3 to 6 pcf, above that representing 95 percent compaction can result in over-consolidation behavior, i.e., very much reduced

pore pressures and dilatant tendencies.

- d. There was no trend in the results to indicate that angles of internal friction based on either total or effective stresses decreased with increasing confining pressure because of particle breakage. This is attributed to reduced effective stresses generated by considerable excess pore pressure during undrained shear.
- e. Effects due to the differences in the sizes of specimens or testing equipment were not significant.
- f. Neither the scalping and replacement procedure nor the procedure in which tests are performed on minus No. 4 sieve fractions of full-scale gradations provides a satisfactory means for determining strength parameters for earth-rock mixtures based on total stresses.

Q triaxial tests on blended material

64. The following conclusions have been drawn for Q triaxial compression tests performed on blended material specimens:

- a. Neither the scalping and replacement procedure nor the procedure in which tests are performed on scalped minus No. 4 sieve fractions provides a satisfactory means of determining Q strength parameters for earth-rock mixtures. Angles of internal friction for Q tests on scalped/replaced materials may be up to 4 deg higher and those for scalped materials up to 14 deg lower than those for corresponding full-scale materials.
- b. Because of their relatively high degree of saturation prior to shear, full-scale materials having gravel contents of 20 percent may have angles of internal friction as low as 24 deg whereas, because of their relatively low degree of saturation prior to shear, full-scale materials having gravel contents of 60 percent may have angles of internal friction approximating those based on effective stresses, i.e., between 35 and 40 deg.

- c. Because of the higher degrees of saturation prior to shear associated with specimens tested at higher confining pressures, angles of internal friction for full-scale, scalped/replaced and scalped materials decrease with increasing confining pressure. Scalped materials exhibit the greatest decrease in strength with increasing confining pressure while full-scale materials exhibit the least.
- d. Gravel contents as low as 20 percent contribute significantly to the Q strength of scalped/replaced materials with the contribution increasing as the gravel content increases. Results show that approximately 20 percent of the deviator stress at failure for a scalped/replaced material having a gravel content of 20 percent was contributed by the gravel fraction.

Recommendations

65. The findings of this investigation prompt the following recommendations:

- a. It has been shown that ~~an earth-rock mixture~~ ^{triaxial tests} on scalped/replaced gradations of ~~an earth-rock mixture~~ may yield substantially overconservative estimates of the strength of the full-scale material depending on the gravel content and plasticity of the fine fraction. On the other hand, Q tests on scalped/replaced specimens may overestimate the strength of the full-scale gradation. Therefore, it is recommended that triaxial testing of earth-rock mixtures be performed on the full-scale material (large specimens) or at least on the altered gradation obtained by removing (scalping) no more than 10 percent by weight of the full-scale material's largest particles. However, if the material under consideration exhibits gradation and plasticity of the fines (minus

No. 4 fraction) similar to the two earth-rock mixtures tested herein, the design engineer may make use of tests performed on scalped/replaced specimens by adjusting with engineering judgment the strength envelopes obtained according to the findings of this research.

- b. It was not an objective of this investigation to determine the effects on strength-deformation behavior of earth-rock mixtures of plasticity of the fines, (minus No. 40 fraction). However, results of the testing program suggest that plasticity of the fines influences the differences seen between strengths of full-scale material as compared to scalped/replaced material. It is recommended that additional research be conducted to determine these effects and whether or not there exists a range of plasticity of the fine fraction for which tests on scalped/replaced triaxial specimens would satisfactorily estimate the strength-deformation characteristics of the full-scale parent material.

66. Donaghe and Townsend (1973, 1975) reported, but did not research, equipment size effects with respect to compaction tests on earth-rock mixtures. More specifically, if compaction tests are performed on a soil containing 3/4-in. maximum particle size in the 6-in.-diam mold and then the same material is compacted in the earth-rock 12-in.-diam mold at the same effort and according to the procedures and equipment given in EM 1110-2-1906, identical moisture-density curves may not be obtained. Additional research is needed to rectify these equipment and/or procedure problems.

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Table 1
R Triaxial Testing Program

Parameter Investigated	$\bar{\sigma}_{3c}$ kg/cm ²	Specimen Diameter in.	Maximum Particle Size	Gravel Content percent	Type of Specimen
<u>Blended Material</u>					
Gravel content	4.22	15	3-in.	20	Full-scale
				40	
				60	
	14.06			20	
				40	
				60	
Scalping and replacement	4.22	6	3/4-in.	20	Scalped/replaced
				40	
				60	
	14.06			20	
				40	
				60	
Scalping	4.22	6	No. 4 sieve	(20)*	Scalped
				(40)*	
				(60)*	
	14.06			(20)*	
				(40)*	
				(60)*	
Equipment and specimen size	4.22	15	3/4-in.	40	Scalped/replaced
	14.06				
<u>DeGray Dam Material</u>					
Properties of full-scale material	2.11	15	3-in.	48	Full-scale
	4.22				
	8.44				
Scalping and replacement	2.11	6	3/4-in.	48	Scalped/replaced
	4.22				
	8.44				
Scalping	2.11	6	No. 4 sieve	(48)*	Scalped
	4.22				
	8.44				

NOTE: All specimens prepared to 95 percent of the standard effort maximum dry density for the given gradation and at a water content obtained by combining saturated surface dry gravel with the minus No. 4 fraction at its optimum water content plus one percentage point.

* Test performed on minus No. 4 fraction of full-scale material at this gravel content.

Table 2
Q Triaxial Testing Program

Parameter Investigated	σ_3 kg/cm ²	Specimen Diameter in.	Maximum Particle Size	Gravel Content percent	Type of Specimen
<u>Blended Material</u>					
Gravel content	4.22	15	3-in.	20	Full-scale
				40	
				60	
	14.06			20	
				40	
				60	
Scalping and replacement	4.22	6	3/4-in.	20	Scalped/replaced
				40	
				60	
	14.06			20	
				40	
				60	
Scalping	4.22	6	No. 4 sieve	(20)*	Scalped
				(40)*	
				(60)*	
	14.06			(20)*	
				(40)*	
				(60)*	
Scalping	4.22	6	No. 4 sieve	(20)*	Scalped**
				(40)*	
				(60)*	
	14.06			(20)*	
				(40)*	
				(60)*	

* Test performed on minus No. 4 fraction of full-scale material at this gravel content.

** Specimens compacted to initial conditions of minus No. 4 fraction in scalped/replaced specimen.

Table 3

Standard Effort Compaction Data on Blended and DeGray Dam Materials

Type of Specimen	Maximum Particle Size	Gravel Content percent	Compaction Mold Diameter in.	Optimum Water Content percent	Maximum Standard Dry Unit Weight pcf
<u>Blended Material</u>					
Full-scale	3-in.	20	18	6.1	136.1
		40		5.9	138.0
		60		5.2	134.9
Scalped/replaced	3/4-in.	20	6	7.9	131.8
		40		7.9	132.0
		60		9.2	128.5
Scalped	No. 4 sieve	* (20)* (40)* (60)	4	8.6 9.8 11.9	131.0 127.0 119.3
<u>DeGray Dam Material</u>					
Full-scale	3-in.	48	18	8.7	127.9
Scalped/replaced	3/4-in.	48	6	10.8	122.4
Scalped	No. 4 sieve	(48)*	4	14.2	117.0

* Refers to the gravel content of the parent full-scale gradation.

Table 4
Results of $\bar{A}$ Triaxial Tests Performed on Blended and DeGray Dam Materials
Specimen Conditions

Test No.	Type of Specimen	Maximum Particle Size, in.	Gravel Content, Percent	Desired Initial Conditions				Actual Initial Conditions				Consolidation				Conditions After Consolidation			
				Total Specimen Weight, g	Minus No. 4 Fraction Weight, g	Water Content, Percent	pcf	Total Specimen Weight, g	Minus No. 4 Fraction Weight, g	Water Content, Percent	pcf	Stress, kg/cm ²	Strain, %	Consolidation, %	pcf	Total Specimen Weight, g	Minus No. 4 Fraction Weight, g	Water Content, Percent	pcf
Blended Material																			
1	Full-scale	3-in.	20	126.3	9.6	123.2	8.1	0.275	78.2	130.2	9.6	124.3	9.9	131.4	135.6				
2	Scalped/ replaced	3/4-in.	6	125.2	9.6	118.6	8.2	0.329	66.2	124.9	9.7	118.3	11.0	125.7	123.6				
3	Scalped	No. 4 sieve	6	124.5	9.6	124.5	9.7	0.333	77.2	124.5	--	--	10.9	128.5	--				
4	Full-scale	3-in.	20	129.3	9.6	123.2	7.2	0.267	71.9	131.1	8.5	125.3	8.7	134.8	129.5				
5	Scalped/ replaced	3/4-in.	6	125.2	9.6	118.6	8.0	0.326	65.3	125.2	9.5	118.6	10.7	132.5	126.9				
6	Scalped	No. 4 sieve	6	124.5	9.6	124.5	9.8	0.332	78.7	124.6	--	--	10.4	132.4	--				
7	Full-scale	3-in.	40	131.1	10.8	116.7	6.6	0.268	65.2	130.9	9.6	116.4	10.0	131.0	116.6				
8	Scalped/ replaced	3/4-in.	6	125.4	10.8	109.3	7.3	0.317	61.6	126.0	10.9	110.1	10.7	129.3	115.6				
9	Scalped	3/4-in.	15	125.4	10.8	109.3	7.3	0.320	60.6	125.7	10.8	109.7	11.0	128.4	113.1				
10	Scalped	No. 4 sieve	6	120.7	10.8	120.7	10.9	0.374	77.5	120.8	--	--	12.6	125.8	--				
11	Full-scale	3-in.	40	131.1	10.8	116.7	5.0	0.231	52.5	132.7	9.1	118.7	7.9	132.1	124.8				
12	Scalped/ replaced	3/4-in.	6	125.4	10.8	109.3	7.5	0.309	64.1	126.8	11.1	111.1	9.4	132.9	119.1				
13	Scalped	3/4-in.	15	125.4	10.8	109.3	6.1	0.308	53.0	126.9	8.9	111.1	9.2	133.3	119.6				
14	Scalped	No. 4 sieve	6	120.7	10.8	120.7	10.8	0.380	75.7	120.4	--	--	11.1	128.1	--				
15	Full-scale	3-in.	60	128.2	12.9	98.2	6.4	0.286	59.2	129.0	12.9	99.4	1.83	131.4	103.0				
16	Scalped/ replaced	3/4-in.	6	122.1	12.9	89.6	6.0	0.358	44.6	122.3	12.0	89.9	2.78	125.8	94.8				
17	Scalped	No. 4 sieve	6	113.3	12.9	113.3	13.3	0.455	77.8	114.1	--	--	15.2	118.3	--				
18	Full-scale	3-in.	60	128.2	12.9	98.2	4.5	0.284	42.2	129.3	9.8	99.8	8.1	136.6	111.3				
19	Scalped/ replaced	3/4-in.	6	122.1	12.9	89.6	6.5	0.351	49.3	122.9	13.2	90.7	5.02	129.4	99.9				
20	Scalped	No. 4 sieve	6	113.3	12.9	113.3	13.0	0.462	74.8	113.5	--	--	13.6	121.9	--				
DeGray Dam Material																			
1	Full-scale	3-in.	48	121.5	15.2	100.9	10.1	0.357	75.4	122.9	17.8	102.0	0.41	123.4	103.0				
2	Full-scale	3-in.	15	125.0	15.2	105.6	8.9	0.327	72.5	125.6	15.5	106.5	0.87	126.7	108.0				
3	Scalped/ replaced	3/4-in.	6	117.4	15.2	95.6	10.7	0.445	64.0	115.1	17.0	92.7	11.8	118.5	97.0				
4	Scalped	No. 4 sieve	6	111.2	15.2	111.2	15.3	0.512	79.8	110.2	--	--	18.1	112.3	--				
5	Full-scale	3-in.	48	121.5	15.2	100.9	9.6	0.396	64.6	119.3	16.9	98.0	3.79	124.0	104.3				
6	Scalped/ replaced	3/4-in.	6	117.4	15.2	95.6	10.0	0.421	61.6	117.3	17.7	95.4	3.85	122.0	101.6				
7	Scalped	No. 4 sieve	6	111.2	15.2	111.2	14.5	0.500	77.2	111.1	--	--	17.4	113.7	--				
8	Full-scale	3-in.	48	121.5	15.2	100.9	10.9	0.380	76.4	120.6	19.3	100.0	6.86	125.2	105.9				
9	Scalped/ replaced	3/4-in.	6	117.4	15.2	95.6	10.5	0.441	63.5	115.6	18.6	93.3	6.02	123.0	102.9				
10	Scalped	No. 4 sieve	6	111.2	15.2	111.2	15.6	0.509	81.7	110.4	--	--	15.8	117.2	--				

Notes: Tests 9 and 13 on the blended material were performed to study effects of equipment and specimen size.
Test 2 on the DeGray Dam material was performed to study effect of higher initial specimen density.

Table 5
Results of Triaxial Tests Performed on Blended and DeGray Dam Material

Test No.			Type of Specimen			Particle Size			Diameter, in.			Gravel Content, percent			Consolidation Stress, σ_{1c} , kg/cm ²			Shear Data at Max. $\bar{\sigma}_1/\bar{\sigma}_3$					Shear Data at $(\sigma_1 - \sigma_3)/2$					θ , Deg.			$A_f \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 + \sigma_3)}$			Percent																																																																																																																																																																																																													
																		Blended Material					DeGray Dam Material																																																																																																																																																																																																																								
																		$\sigma_1 - \sigma_3$, kg/cm ²	$\sigma_1 + \sigma_3$, kg/cm ²	σ_3 , kg/cm ²	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$										$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$

Notes: Tests 9 and 13 on the blended material were performed to study effects of equipment and specimen size.

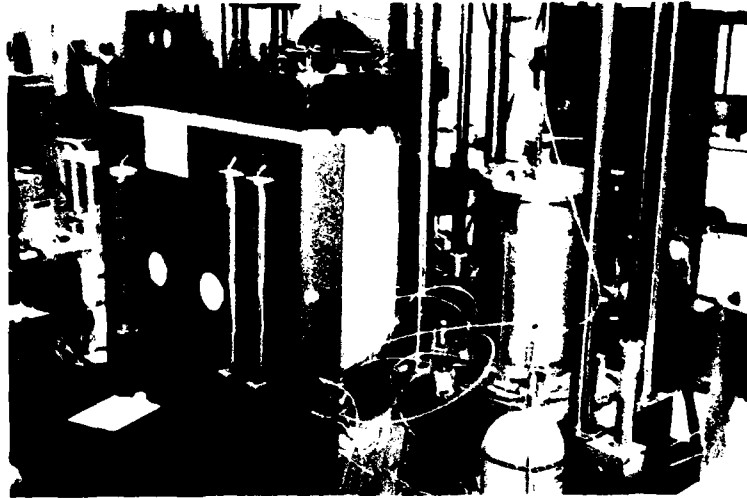
Test 2 on the DeGray Dam material was performed to study effect of higher initial specimen density.

Table 6
Results of Q Triaxial Tests Performed on Blended Material

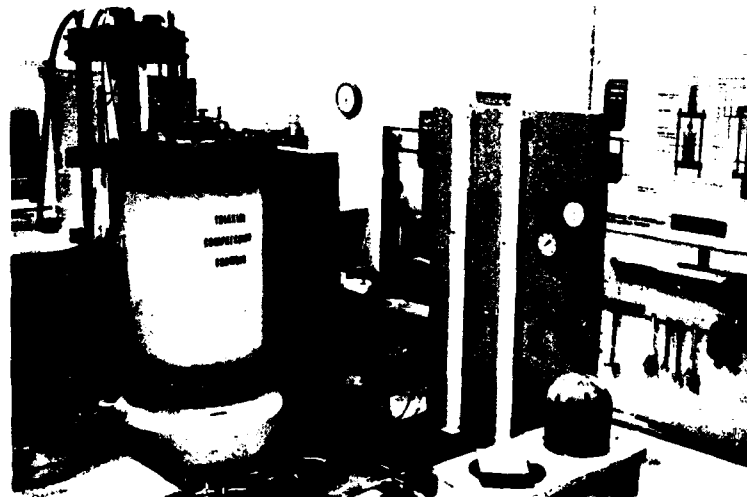
Test No.	Type of Specimen	Maximum Particle Size	Specimen Diameter, in.	Desired Initial Specimen Conditions				Actual Initial Specimen Conditions				Principal Stresses				Volumetric Strains				Shear Data at Failure				Percent Saturation at End of Stabilization Period Prior to Shear, %
				Total Specimen Weight, lb.	Minus No. 5 Fraction, lb.	Water Content, PERCENT	Dry Unit Weight, PCF	Total Specimen Weight, lb.	Minus No. 5 Fraction, lb.	Water Content, PERCENT	Dry Unit Weight, PCF	Principal Stress, lb./sq. in.	Intermediate Stress, lb./sq. in.	Minor Stress, lb./sq. in.	Volume Change, %	Volume Change, %	Volume Change, %	Volume Change, %	$\sigma_1 - \sigma_3$, lb./sq. in.	σ_1 / σ_3 , PERCENT	ϕ , DEGREES	θ , DEGREES		
1	Full-scale	3-in.	15	129.2	9.6	123.2	9.6	7.5	0.287	68.8	130.0	9.3	124.0	9.3	4.22	3.45	13.4	24.4	76.1	5.96	13.4	24.4	24.4	76.1
2	Scalped/replaced	3/4-in.	6	125.2	9.6	118.6	8.1	0.332	61.0	122.8	9.0	115.9	9.0	4.22	3.00	12.9	25.4	68.8	6.42	12.9	25.4	25.4	68.8	
3	Scalped	No. 4 sieve	6	118.5	9.6	123.5	9.7	0.344	75.0	123.5	9.0	115.9	9.0	4.22	3.00	10.0	20.2	61.0	5.60	20.0	23.5	23.5	61.0	
4	Scalped	No. 4 sieve	6	118.5	9.6	116.7	9.6	9.7	0.405	61.9	116.1	9.0	115.9	9.0	4.22	6.18	16.0	20.2	61.0	4.47	16.0	20.2	20.2	61.0
5	Full-scale	3-in.	15	131.1	10.8	116.6	7.4	0.261	75.5	132.6	12.3	118.6	12.3	3.68	2.18	8.1	24.2	76.6	8.66	8.1	24.2	24.2	76.6	
6	Scalped/replaced	3/4-in.	6	125.4	10.8	106.5	7.4	0.261	75.5	132.6	12.3	118.6	12.3	3.68	2.18	8.1	24.2	76.6	8.66	8.1	24.2	24.2	76.6	
7	Scalped	No. 4 sieve	6	120.7	10.8	108.5	10.8	10.8	0.406	70.9	122.3	11.2	105.4	11.2	3.08	6.41	20.0	21.4	69.8	4.64	20.0	21.4	21.4	69.8
8	Scalped	No. 4 sieve	6	107.1	10.8	99.0	10.8	10.8	0.550	52.0	107.1	13.4	99.2	13.4	3.93	9.21	20.0	13.5	69.8	3.08	19.0	13.5	13.5	69.8
9	Full-scale	3-in.	15	128.2	12.9	89.3	5.4	0.298	68.8	128.6	9.3	124.0	9.3	4.22	3.45	13.4	24.4	76.1	10.80	17.7	24.1	24.1	76.1	
10	Scalped/replaced	3/4-in.	6	127.1	12.9	89.3	5.4	0.305	63.4	121.6	12.0	108.9	12.0	3.68	1.83	17.9	24.4	76.1	13.44	20.0	27.9	27.9	76.1	
11	Scalped	No. 4 sieve	6	115.3	12.9	111.6	12.9	0.407	65.7	111.6	9.0	115.9	9.0	4.22	3.66	18.0	26.4	76.4	4.03	20.0	26.4	26.4	76.4	
12	Scalped	No. 4 sieve	6	93.7	12.9	93.0	12.9	0.784	43.6	93.0	9.0	115.9	9.0	4.22	13.65	18.0	18.0	67.5	6.01	18.0	18.0	18.0	67.5	
13	Full-scale	3-in.	15	129.2	9.6	121.2	7.6	0.293	71.6	129.4	9.3	124.3	9.3	4.22	3.50	18.05	13.5	23.0	18.05	13.5	23.0	23.0	23.0	23.0
14	Scalped/replaced	3/4-in.	6	125.2	9.6	118.6	7.6	0.293	71.6	129.4	9.3	124.3	9.3	4.22	3.50	18.05	13.5	23.0	18.05	13.5	23.0	23.0	23.0	23.0
15	Scalped	No. 4 sieve	6	124.5	9.6	118.6	9.7	0.337	76.8	124.1	9.7	117.7	9.7	4.22	6.08	14.06	13.0	19.5	14.06	13.0	19.5	19.5	19.5	19.5
16	Scalped	No. 4 sieve	6	120.5	9.6	119.9	9.7	0.384	67.4	119.9	12.4	110.2	12.4	3.68	6.18	15.99	7.3	21.3	15.99	7.3	21.3	21.3	21.3	21.3
17	Full-scale	3-in.	15	131.1	10.8	116.6	7.4	0.234	77.6	132.3	12.4	118.2	12.4	3.68	---	---	---	---	---	---	---	---	---	---
18	Scalped/replaced	3/4-in.	6	125.4	10.8	106.5	7.4	0.245	57.3	122.4	11.0	106.8	11.0	4.66	6.66	25.88	16.4	28.6	6.66	25.88	16.4	28.6	28.6	28.6
19	Scalped	No. 4 sieve	6	120.7	10.8	119.8	10.8	0.395	76.8	119.8	11.0	106.8	11.0	4.66	6.66	25.88	16.4	28.6	6.66	25.88	16.4	28.6	28.6	28.6
20	Scalped	No. 4 sieve	6	110.5	10.8	96.1	10.8	0.497	56.1	110.5	9.0	115.9	9.0	4.22	11.79	8.92	3.5	13.9	8.92	3.5	13.9	13.9	13.9	13.9
21	Full-scale	3-in.	15	128.2	12.9	98.1	5.6	0.313	67.7	127.3	13.0	112.3	13.0	3.68	3.89	32.45	14.9	28.4	3.89	32.45	14.9	28.4	28.4	28.4
22	Scalped/replaced	3/4-in.	6	129.2	12.9	89.3	5.6	0.313	67.7	127.3	13.0	112.3	13.0	3.68	3.89	32.45	14.9	28.4	3.89	32.45	14.9	28.4	28.4	28.4
23	Scalped	No. 4 sieve	6	113.3	12.9	89.3	13.0	0.479	72.1	112.2	13.0	107.5	13.0	3.68	7.19	11.35	20.0	18.7	7.19	11.35	20.0	18.7	18.7	18.7
24	Scalped	No. 4 sieve	6	92.2	12.9	92.4	12.7	0.797	42.5	92.4	12.7	87.5	12.7	3.68	26.63	9.53	18.6	14.7	26.63	9.53	18.6	14.7	14.7	14.7

Notes: Scalped test specimens 4, 8, 12, 16, 20, and 24 were compacted to approximately the dry unit weight of minus No. 4 fraction of corresponding scalped/replaced specimens to study contribution of gravel fraction to strength.

Test results on specimen 17 were invalid because a leak developed during application of the confining pressure. Insufficient material was available for a repeat.



a. Small-scale testing station



b. Large-scale testing station

Figure 1. Small- and large-scale testing stations

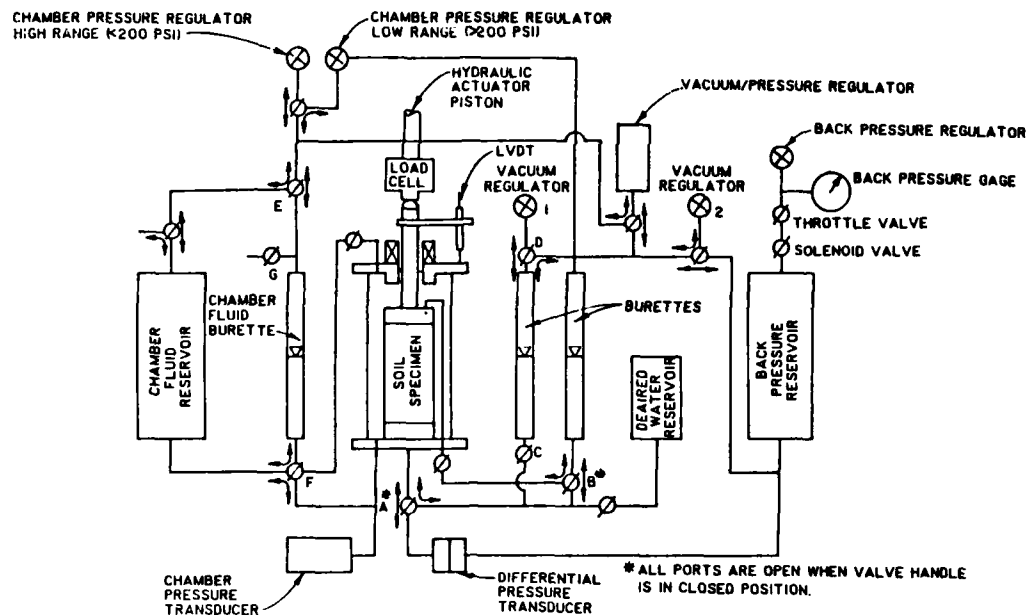


Figure 2. Schematic diagram of pressure control and measurement systems, small-scale testing equipment

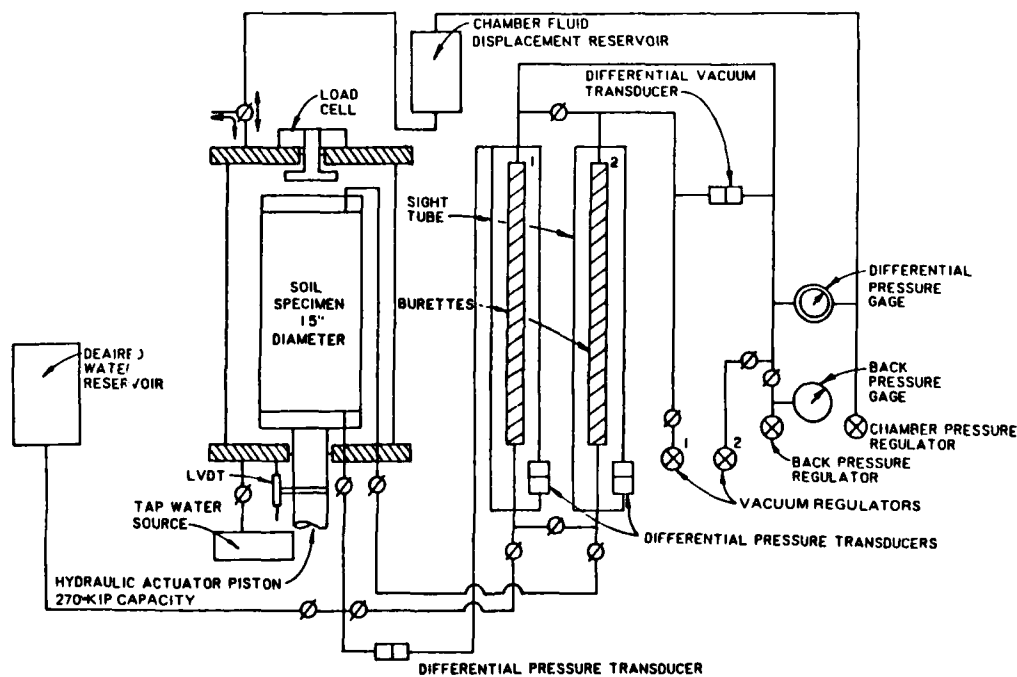


Figure 3. Schematic diagram of pressure control and measurement systems, large-scale testing equipment



Figure 4. Cast aluminum split mold for 15-in.-diam
reconstituted triaxial specimens

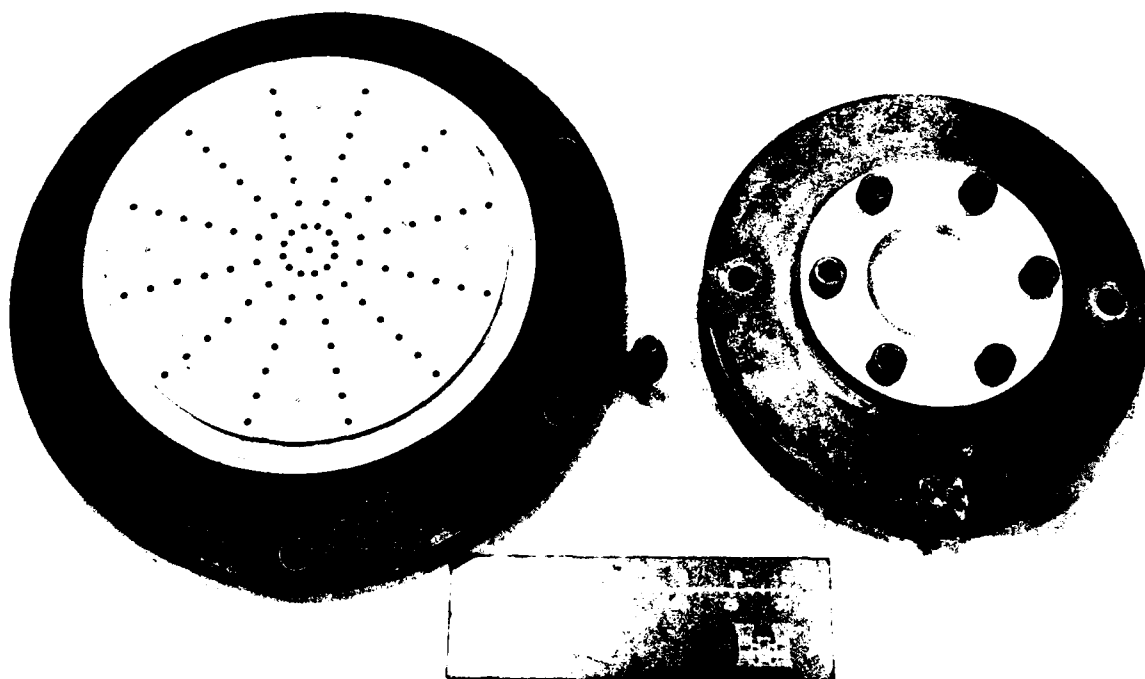


Figure 5. Base and top platens, 15-in.-diam
triaxial specimens



Figure 6. Membrane stretcher, 15-in.-diam triaxial specimens



Figure 7. Triaxial chamber, 15-in.-diam triaxial tests

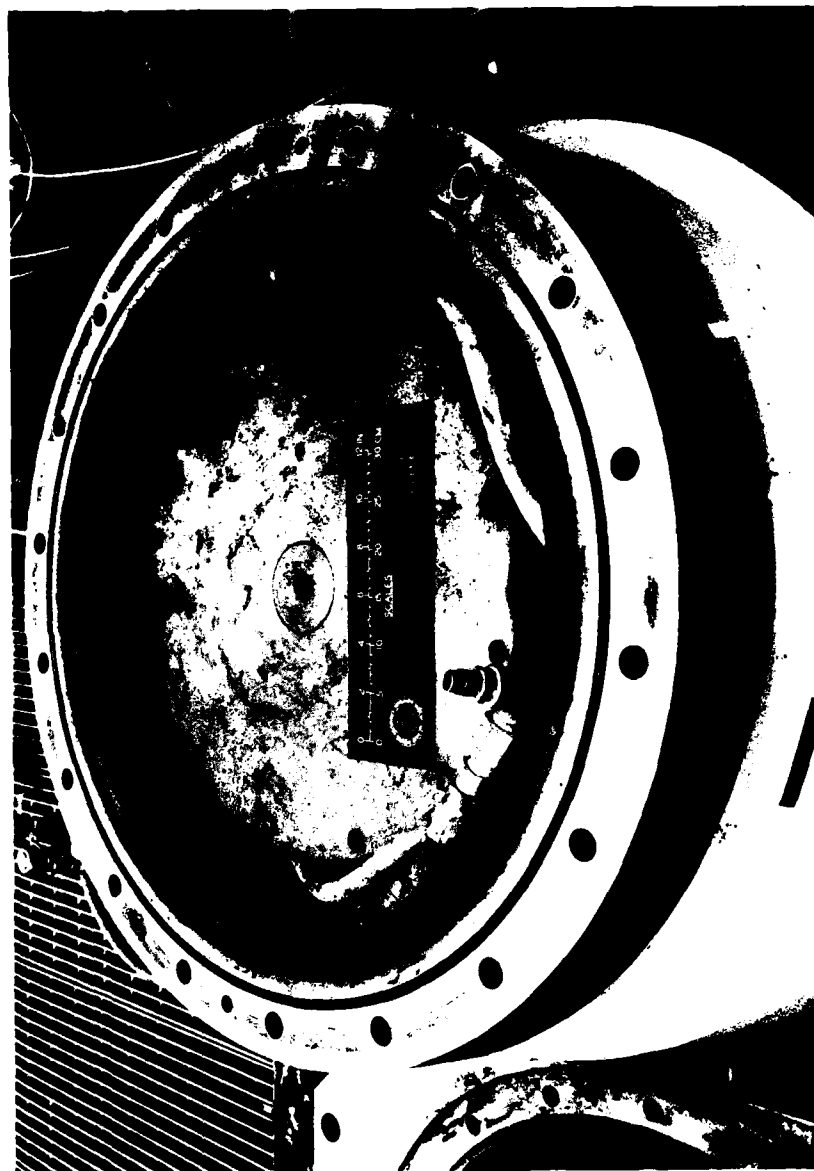


Figure 8. Base loading piston, 15-in.-diam triaxial tests



Figure 9. Instrumentation and data acquisition panel,
small- and large-scale triaxial tests

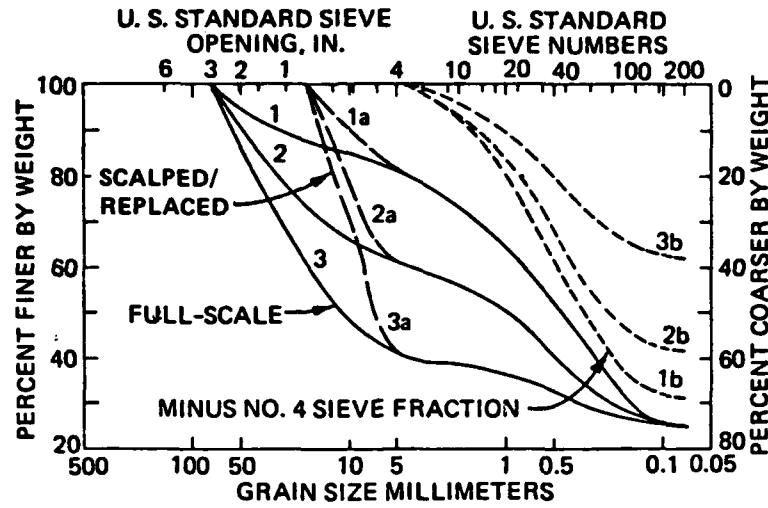


Figure 10. Test specimen gradations, blended material

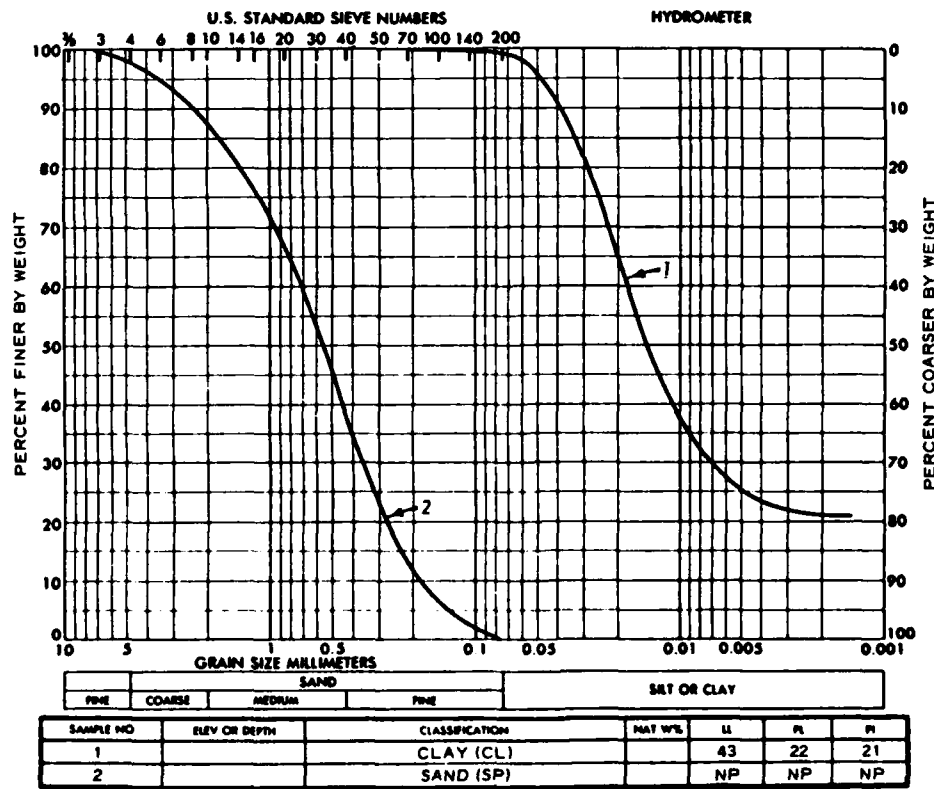


Figure 11. Gradation curves and classification data, clay and sand used for blended material specimens

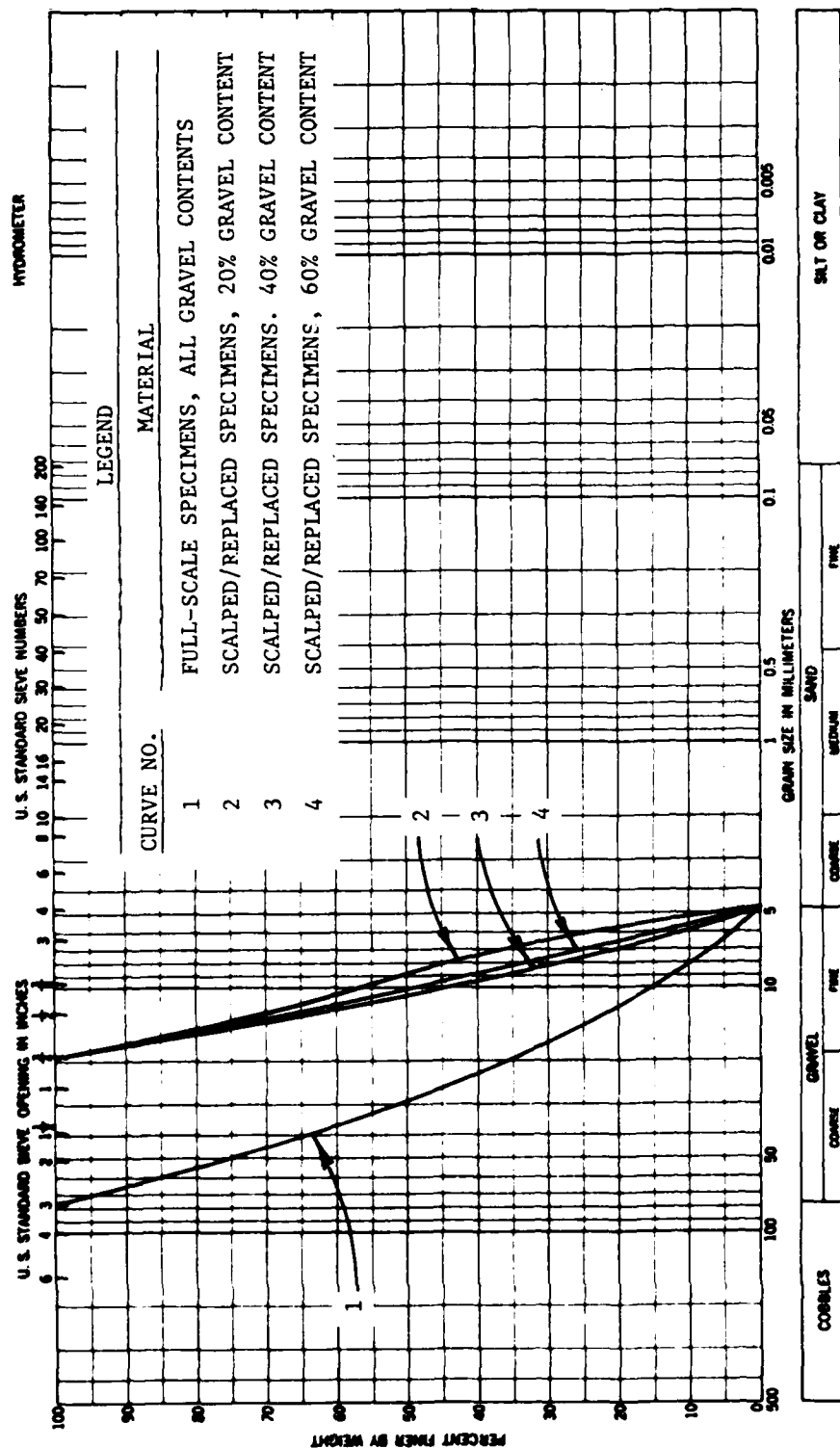


Figure 12. Gradations of gravel fractions, blended material specimens



3"-2" fraction

Figure 13. Blended material, 3- to 2-in. sizes,
California placer gravel



2"-1 1/2" fraction

Figure 14. Blended material, 2- to 1-1/2-in. sizes,
DeGray Dam gravel



Figure 15. Blended material, 1-1/2- to 1-in. sizes, local gravel

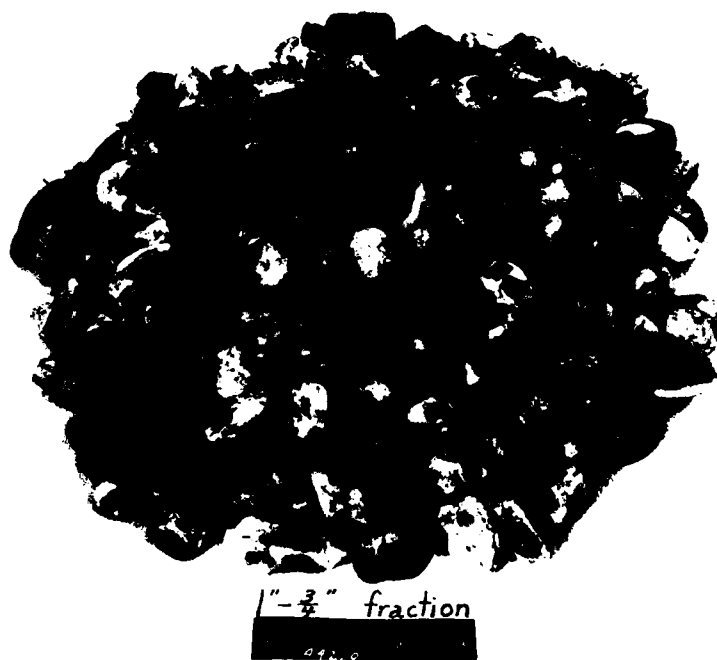


Figure 16. Blended material, 1- to 3/4-in. sizes, local gravel

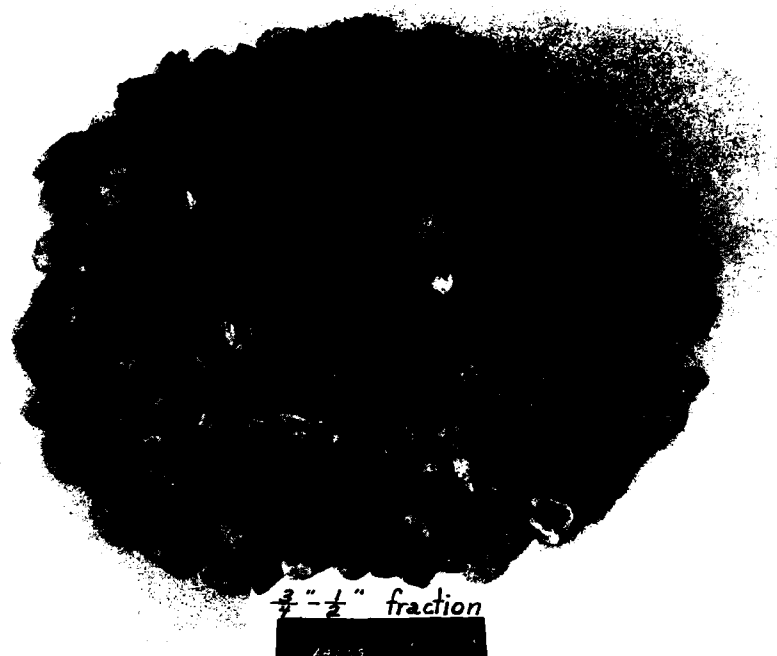
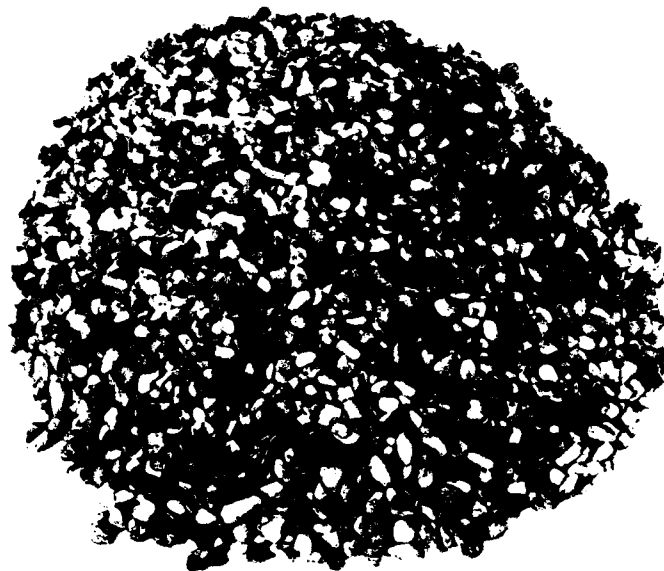


Figure 17. Blended material, 3/4- to 1/2-in. sizes,
local gravel



Figure 18. Blended material, 1/2- to 3/8-in. sizes,
local gravel



3/8" - #4 fraction

4436

Figure 19. Blended material, 3/8-in. to No. 4 sizes, local gravel

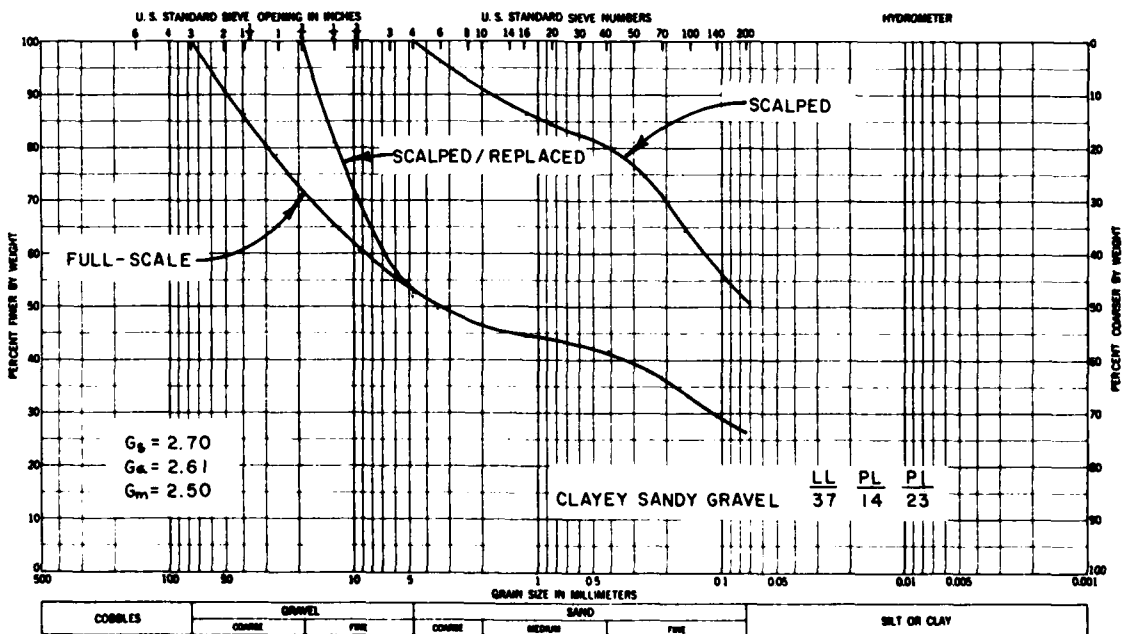
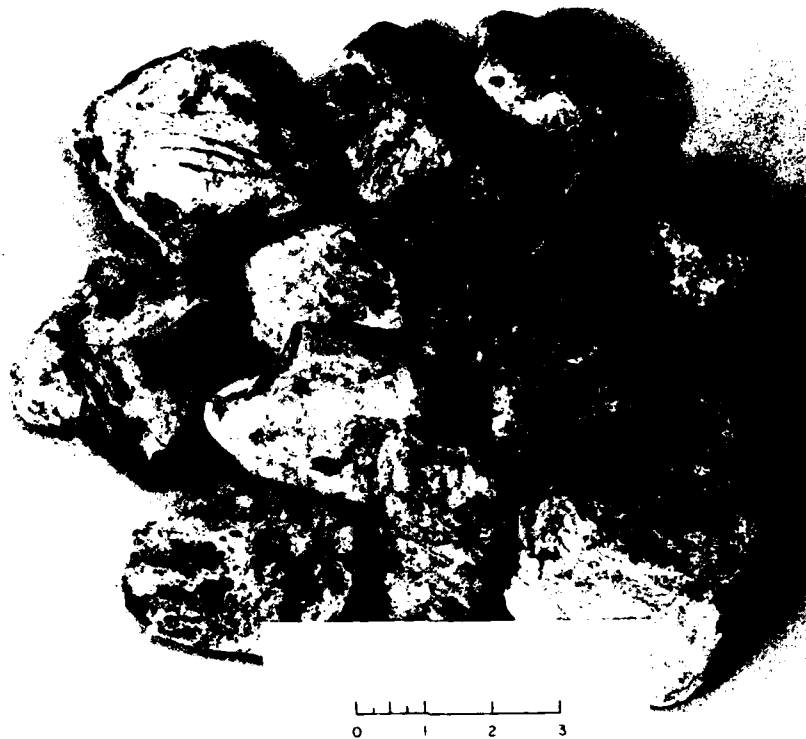


Figure 20. Test specimen gradations, DeGray Dam material

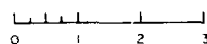


a. 3- to 2-in. sizes

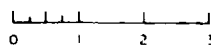


b. 2- to 1-1/2-in. sizes

Figure 21. DeGray Dam material, 3- to 2-in. and 2- to 1-1/2-in. sizes

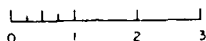
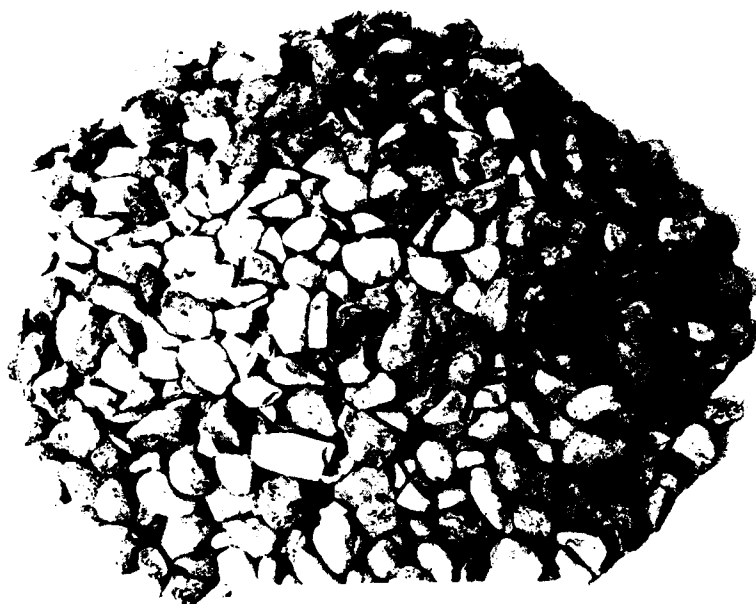


a. 1-1/2- to 1-in. sizes

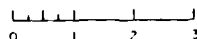


b. 1- to 3/4-in. sizes

Figure 22. DeGray Dam material, 1-1/2 to 1-in. and 1- to 3/4-in. sizes



a. 3/4- to 1/2-in. sizes



b. 1/2- to 3/8-in. sizes

Figure 23. DeGray Dam material, 3/4- to 1/2-in. and 1/2- to 3/8-in. sizes

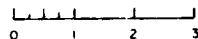
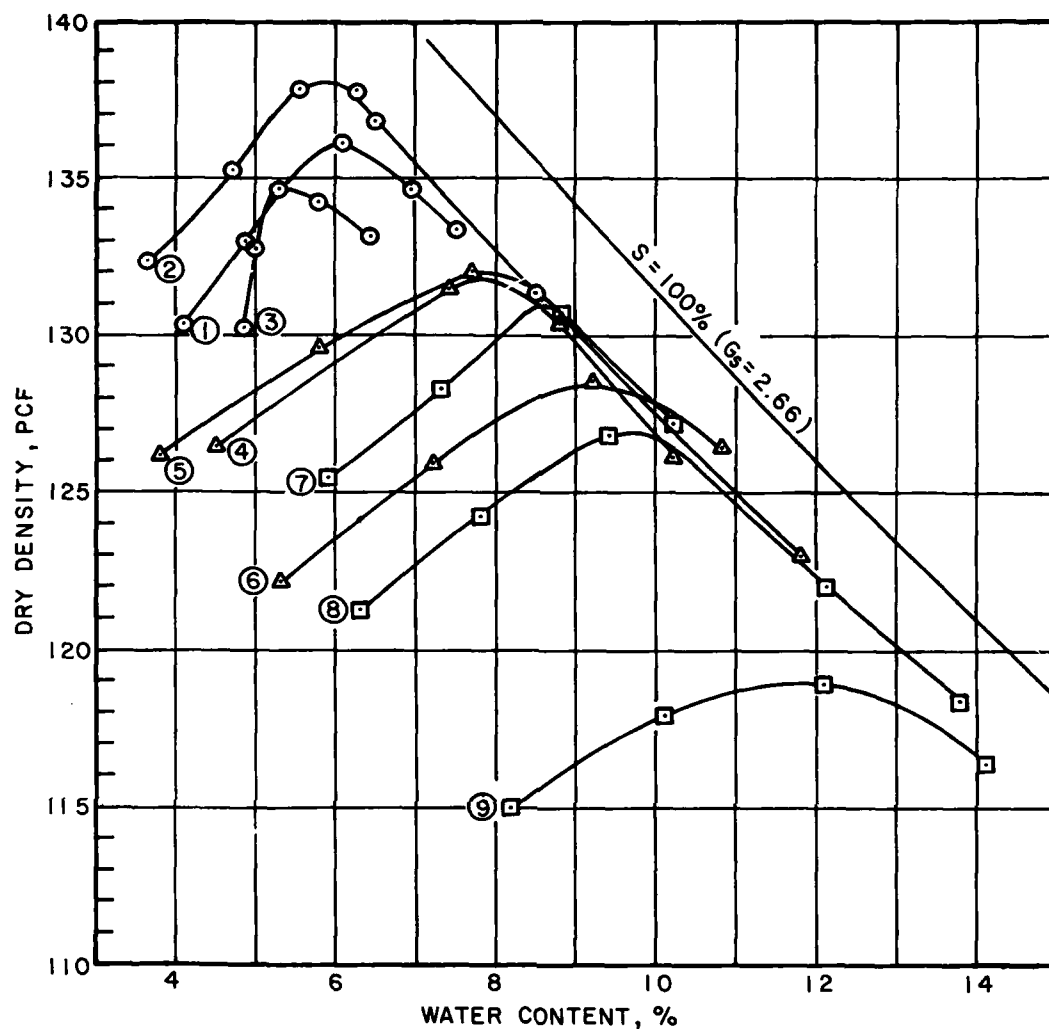


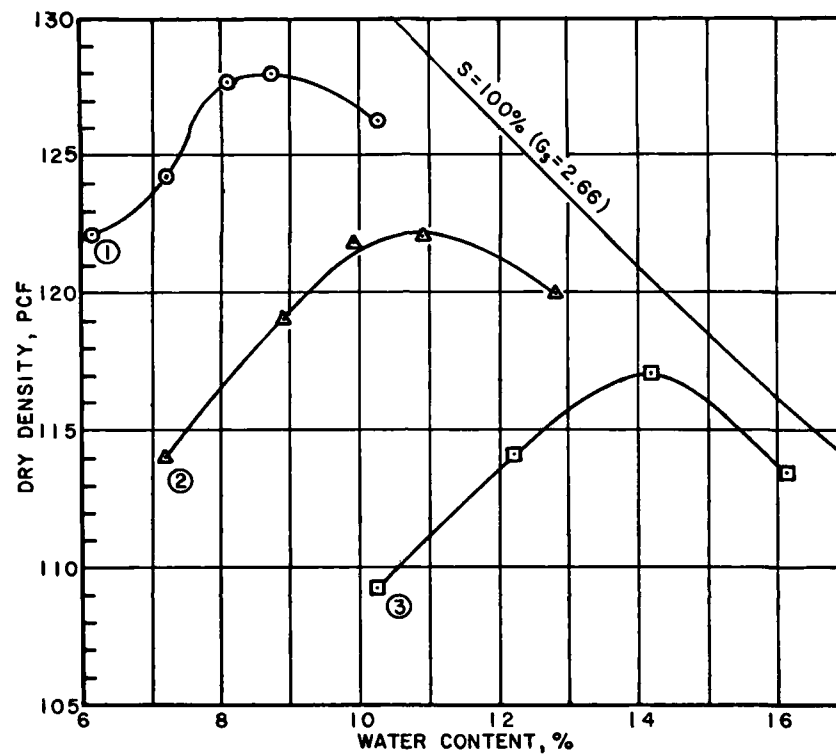
Figure 24. DeGray Dam material, 3/8-in.
to No. 4 sieve sizes



CURVE NO.	SYMBOL	GRAVEL CONTENT %	TYPE OF SPECIMEN	MAXIMUM PARTICLE SIZE IN.
①	○	20	FULL - SCALE	3
②	○	40		
③	○	60		
④	△	20	SCALPED / REPLACED	3/4
⑤	△	40		
⑥	△	60		
⑦	□	(20)*	SCALPED	NO.4 SIEVE
⑧	□	(40)*		
⑨	□	(60)*		

* REFERS TO GRAVEL CONTENT IN PARENT FULL-SCALE GRADATION

Figure 25. Standard effort compaction curves, blended material



CURVE NO.	SYMBOL	GRAVEL CONTENT %	TYPE OF SPECIMEN	MAXIMUM PARTICLE SIZE IN.
①	○	48	FULL-SCALE	3
②	△	48	SCALPED / REPLACED	3/4
③	□	(48)*	SCALPED	NO.4 SIEVE

* REFERS TO GRAVEL CONTENT IN PARENT FULL-SCALE GRADATION

Figure 26. Standard effort compaction curves, DeGray Dam material



Figure 27. Placement of a material layer in the 15-in.-diam mold



Figure 28. Compacting a layer of a large-scale specimen



Figure 29. Checking the height of a layer of a large-scale specimen



Figure 30. Placing the steel plate upon the final layer of a large-scale specimen



Figure 31. A large-scale specimen after compaction



Figure 32. Placement of a filter disc and the top perforated bearing plate upon a compacted large-scale specimen



Figure 33. Placement of the top plate upon a compacted large-scale specimen



Figure 34. Securing the inner vulcanized rubber membrane to the top platen of a large-scale specimen

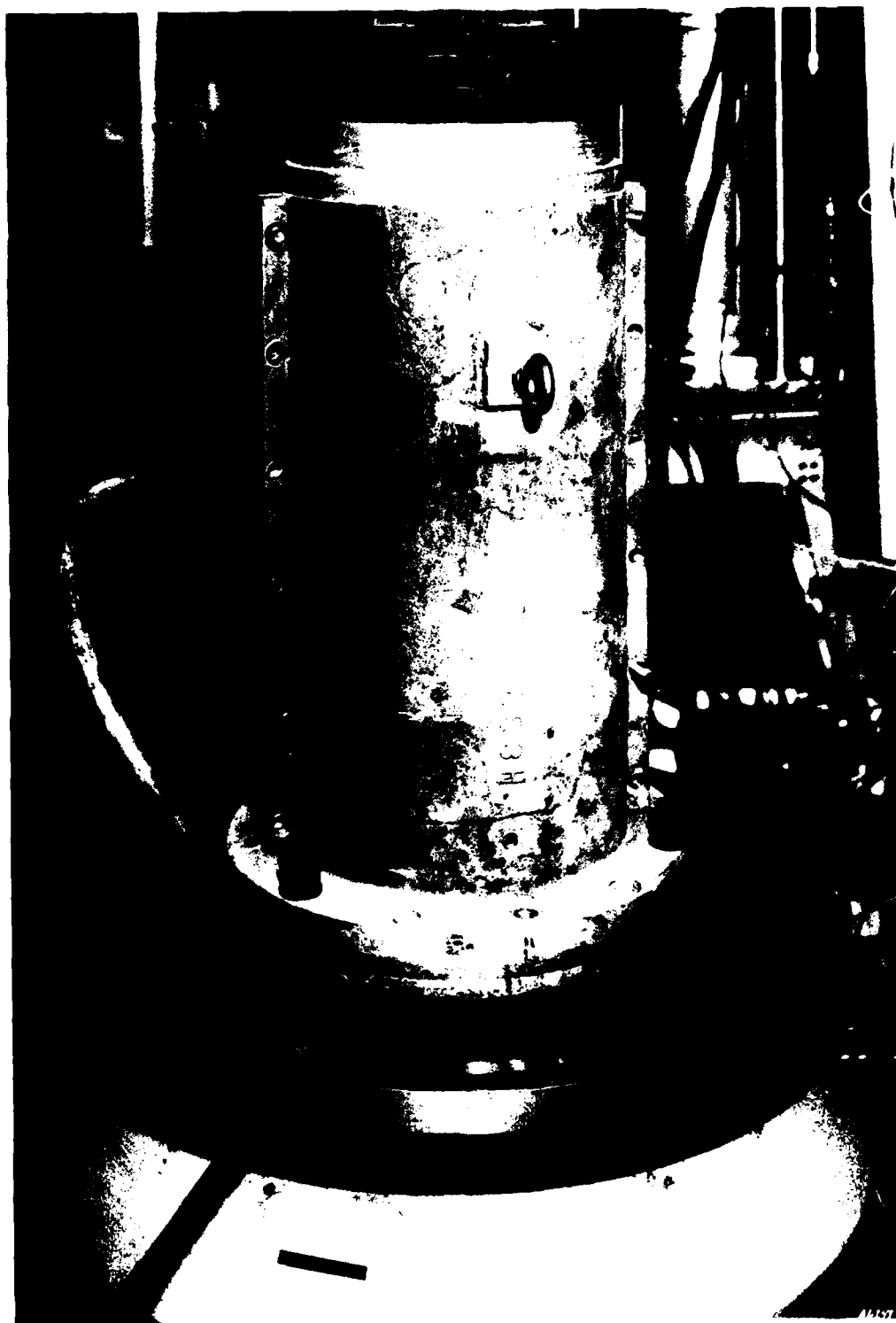


Figure 35. Placement of the mold and large-scale specimen on the actuator piston plate

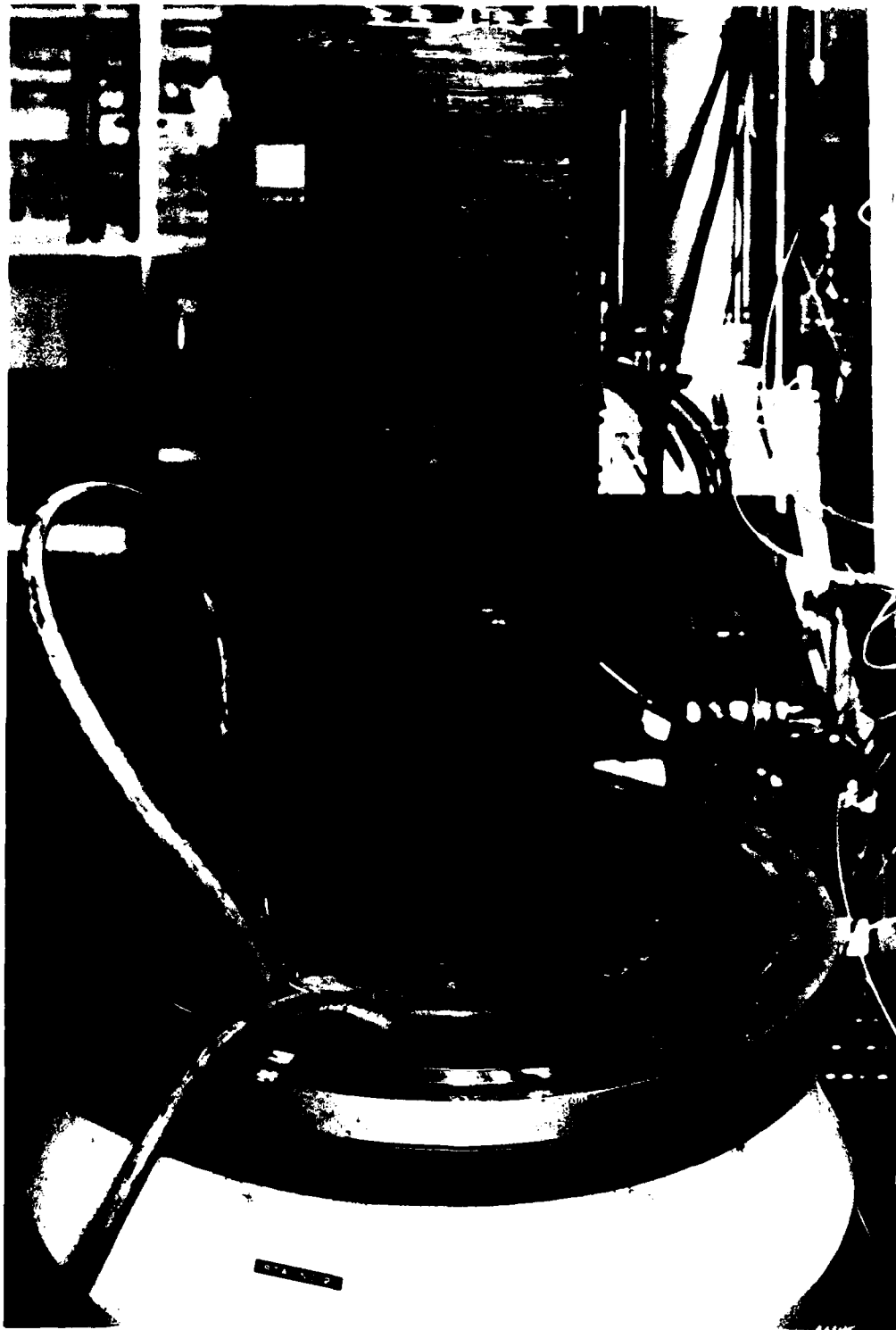


Figure 36. Large-scale specimen standing under an internal vacuum after removal of the mold

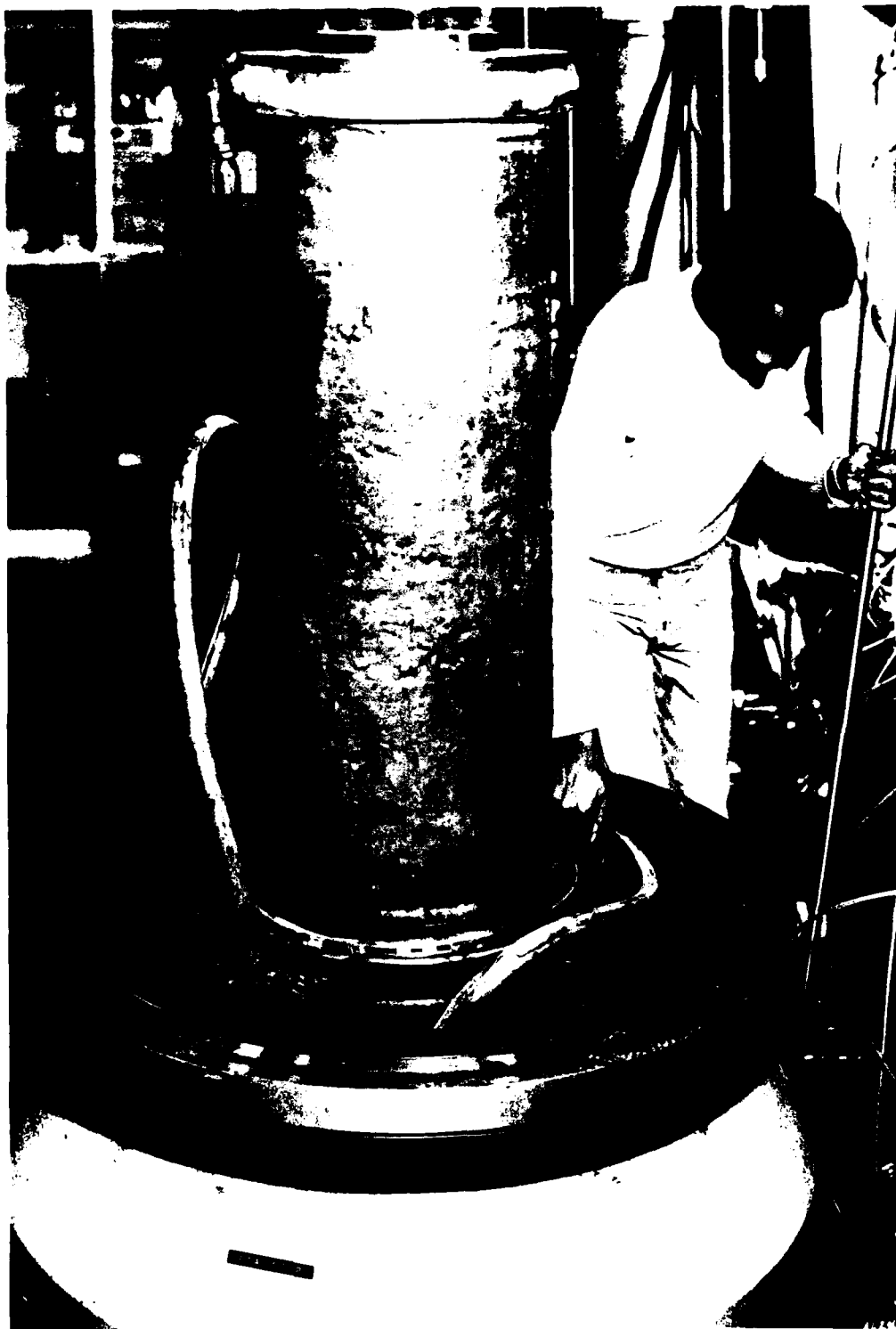


Figure 37. Large-scale specimen after placement of
outer latex membrane

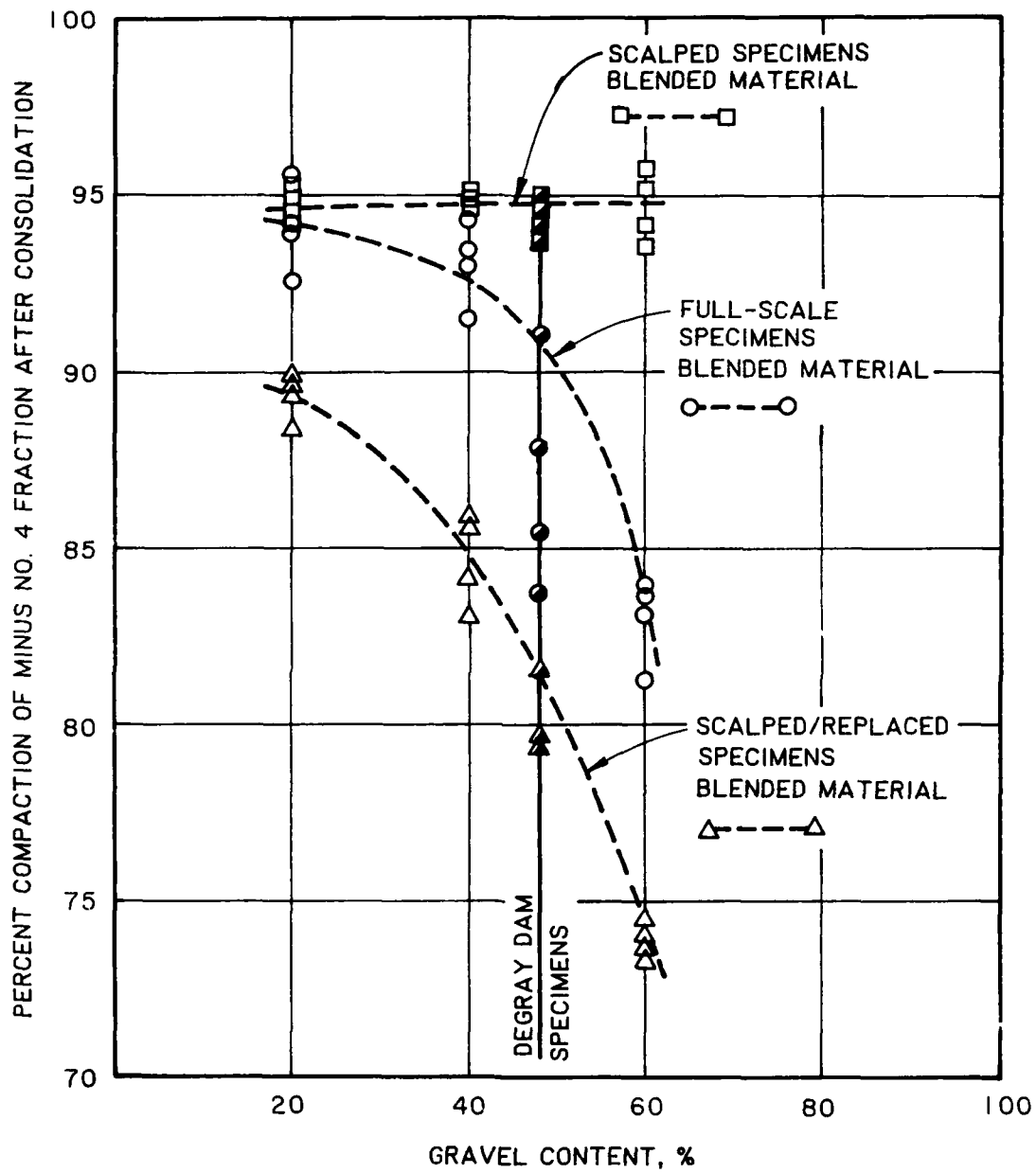


Figure 38. After compaction percent compaction of minus No. 4 fractions versus gravel content

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STRENGTH AND DEFORMATION PROPERTIES OF EARTH-ROCK
MIXTURES(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG MS GEOTECHNICAL LAB R T DONAGHE ET AL.

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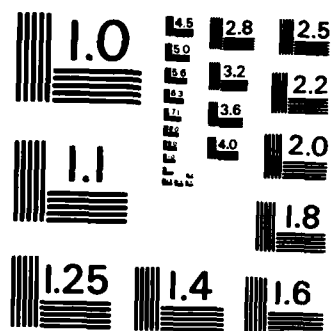
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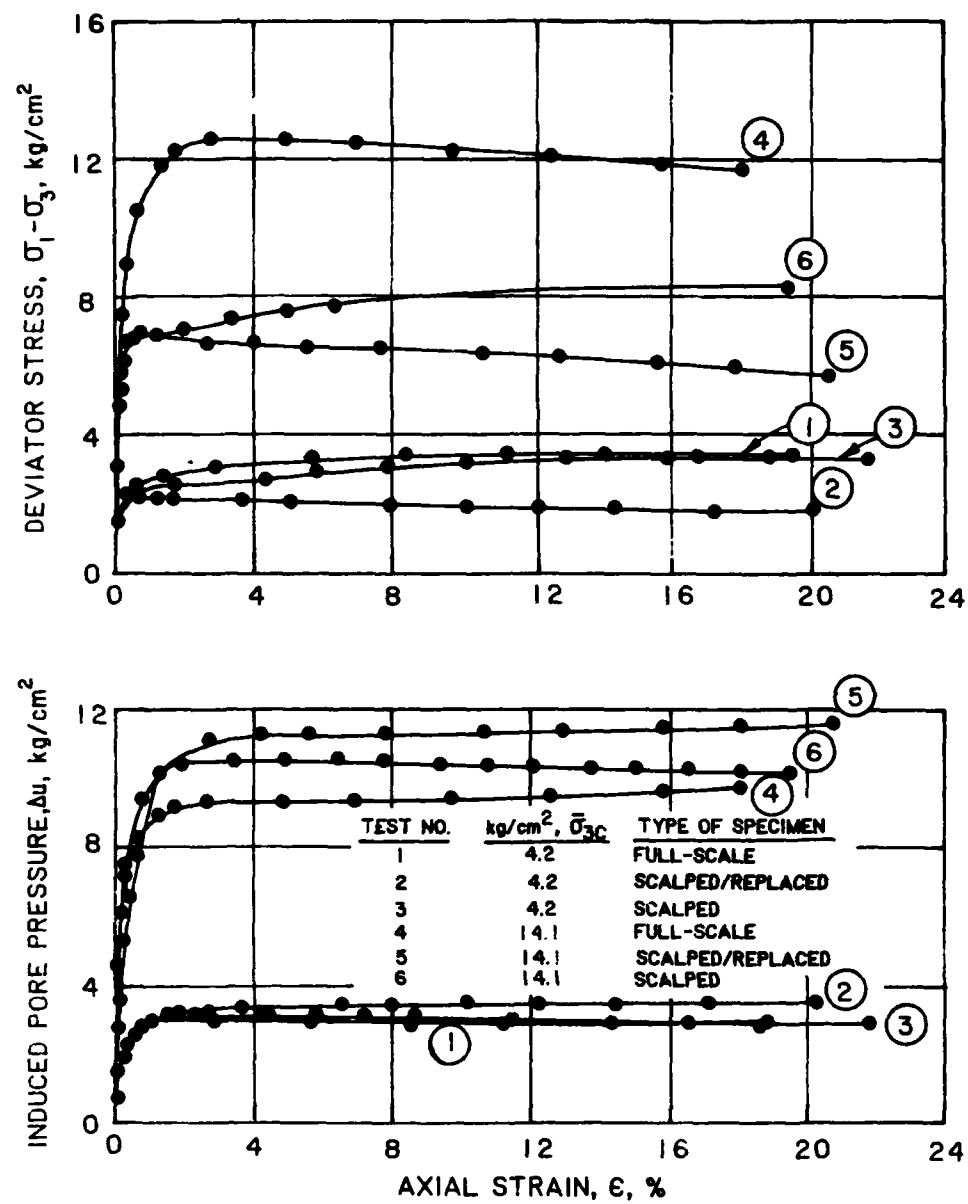


Figure 39. Deviator stress and induced pore pressure versus strain
blended material, 20 percent gravel

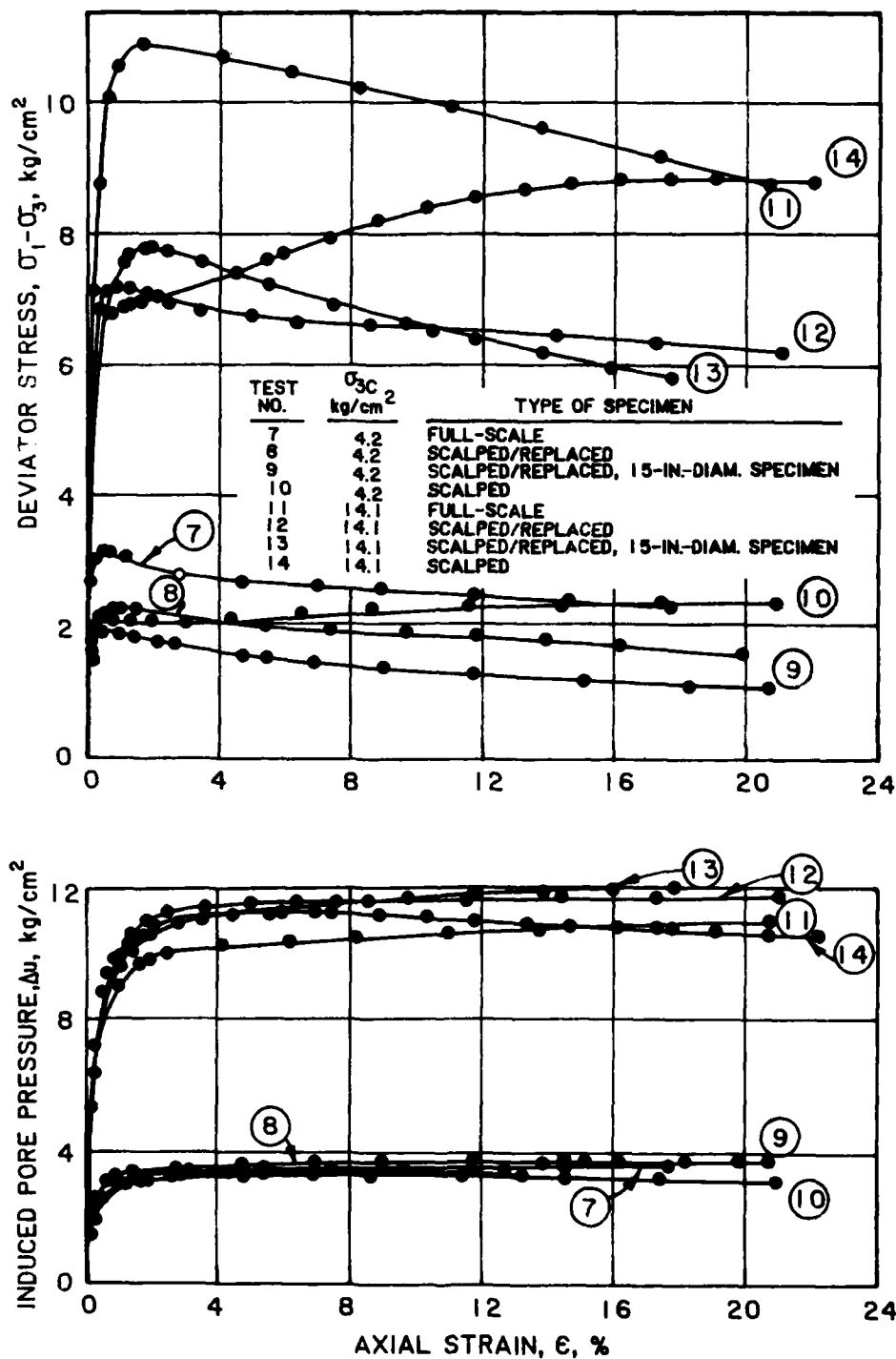


Figure 40. Deviator stress and induced pore pressure versus axial strain, blended material, 40 percent gravel

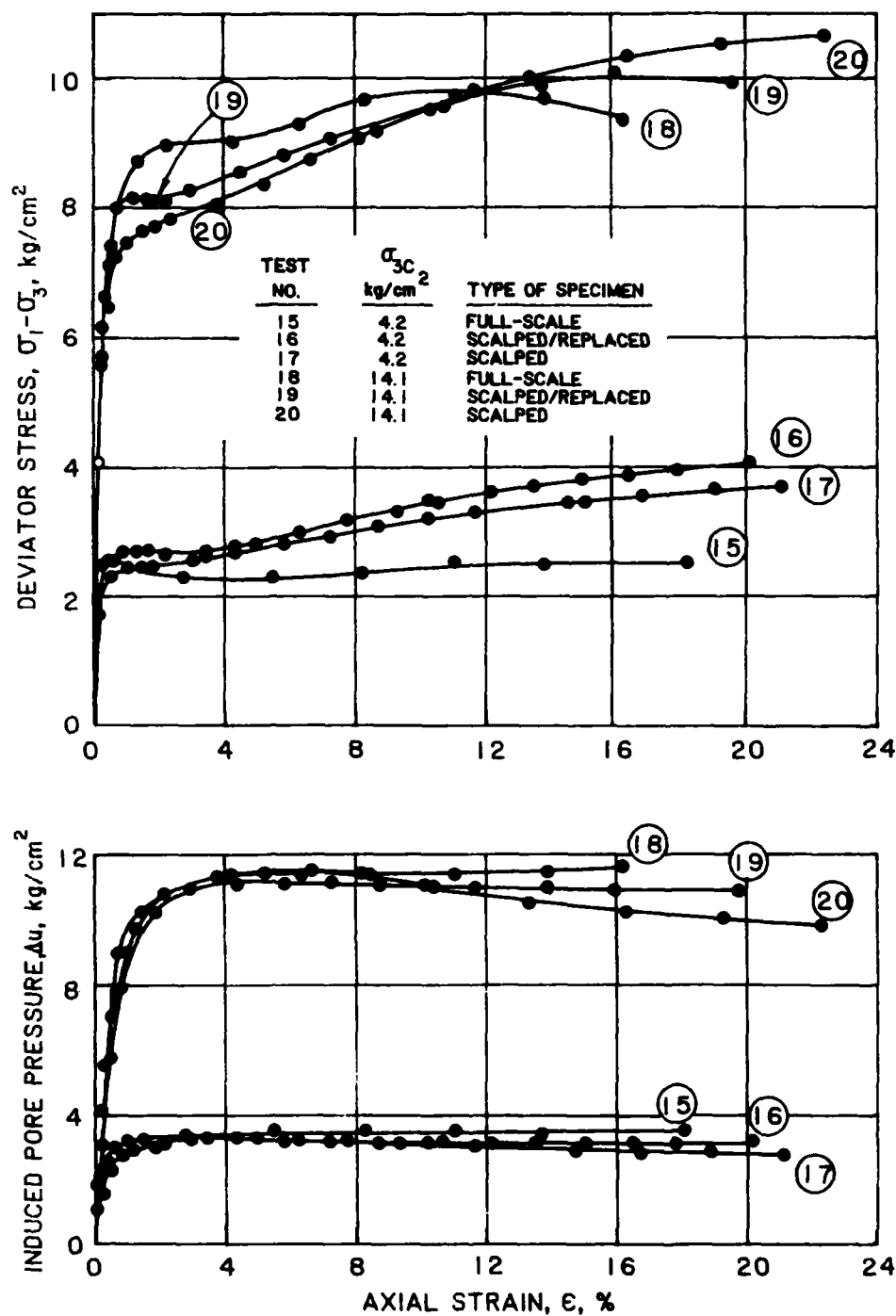


Figure 41. Deviator stress and induced pore pressure versus axial strain, blended material, 60 percent gravel

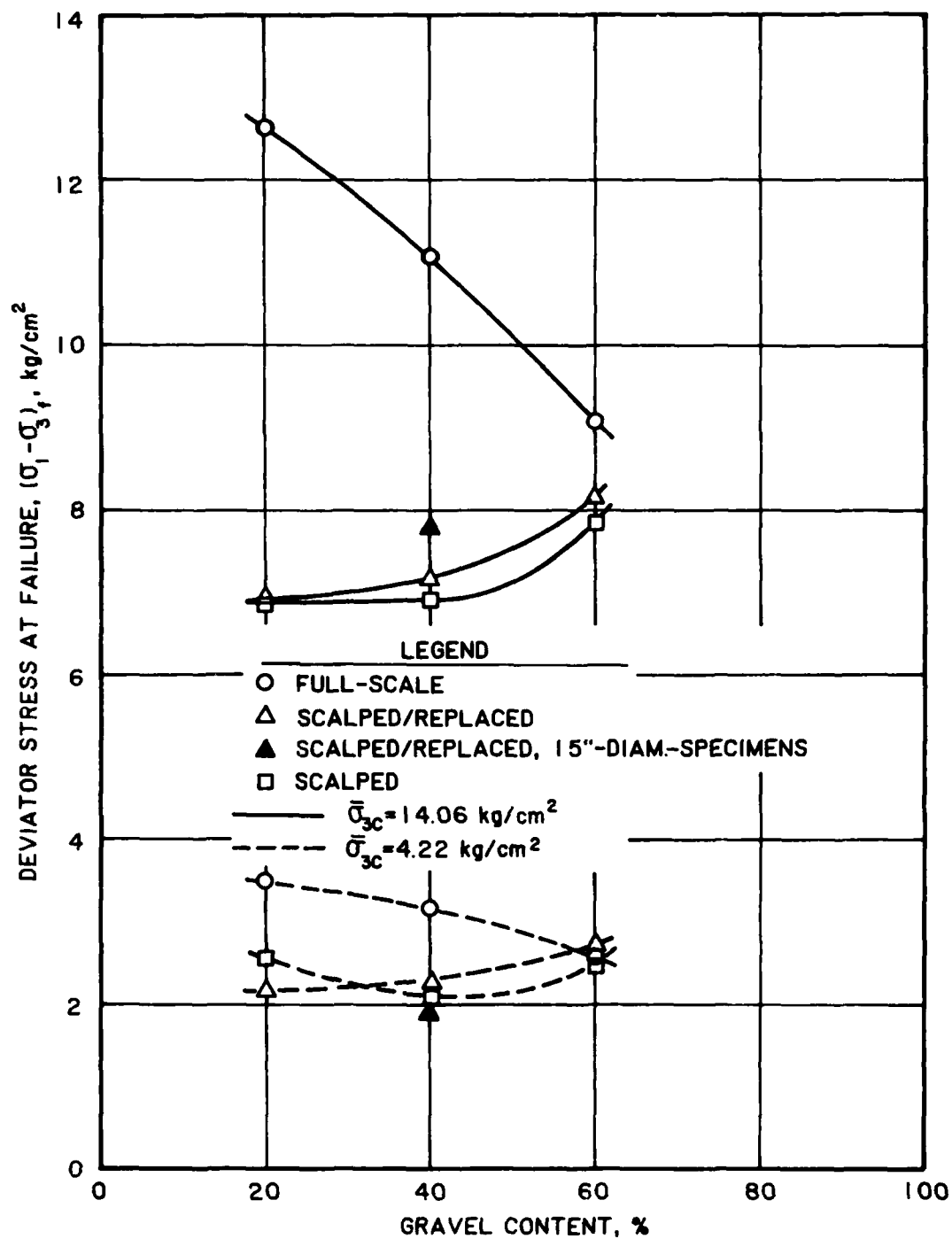


Figure 42. Deviator stress at failure, $(\sigma_1 - \sigma_3)_f$, versus gravel content, blended material

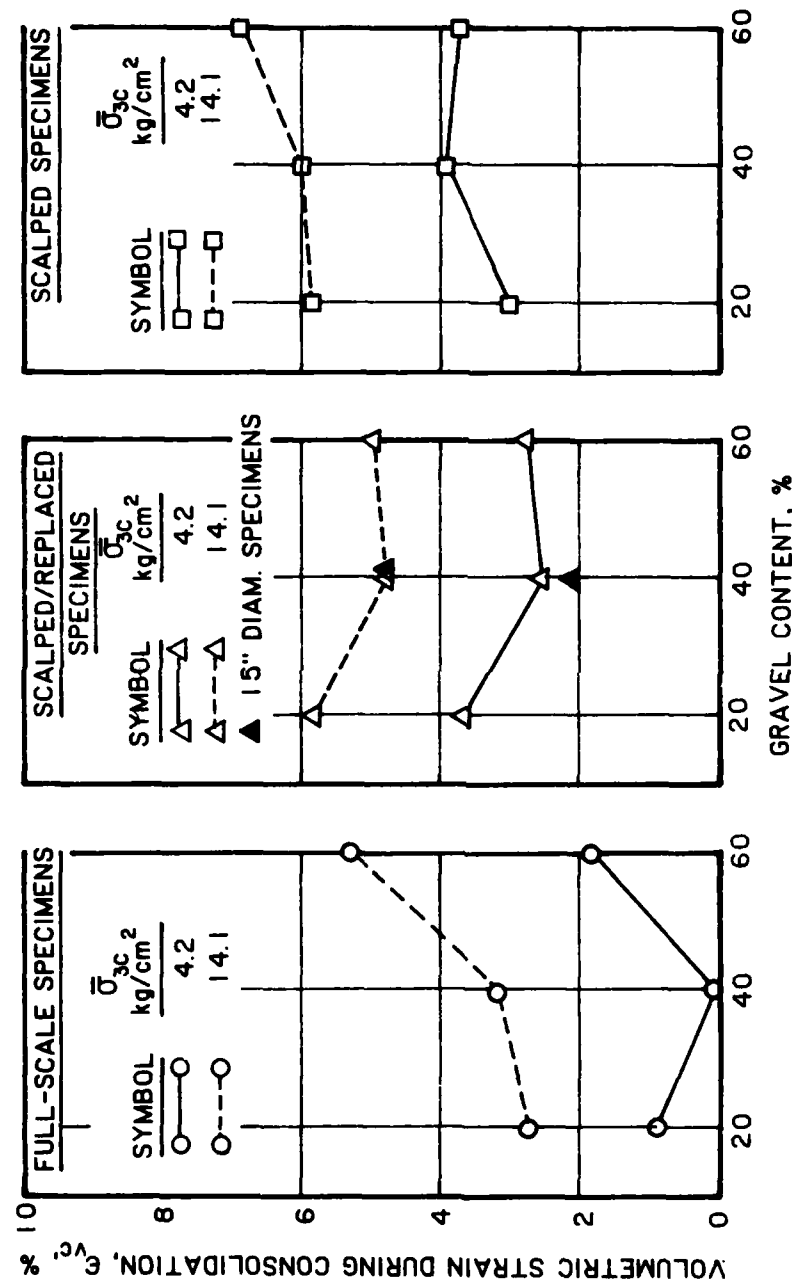


Figure 43. Volumetric strain during consolidation versus gravel content, blended material

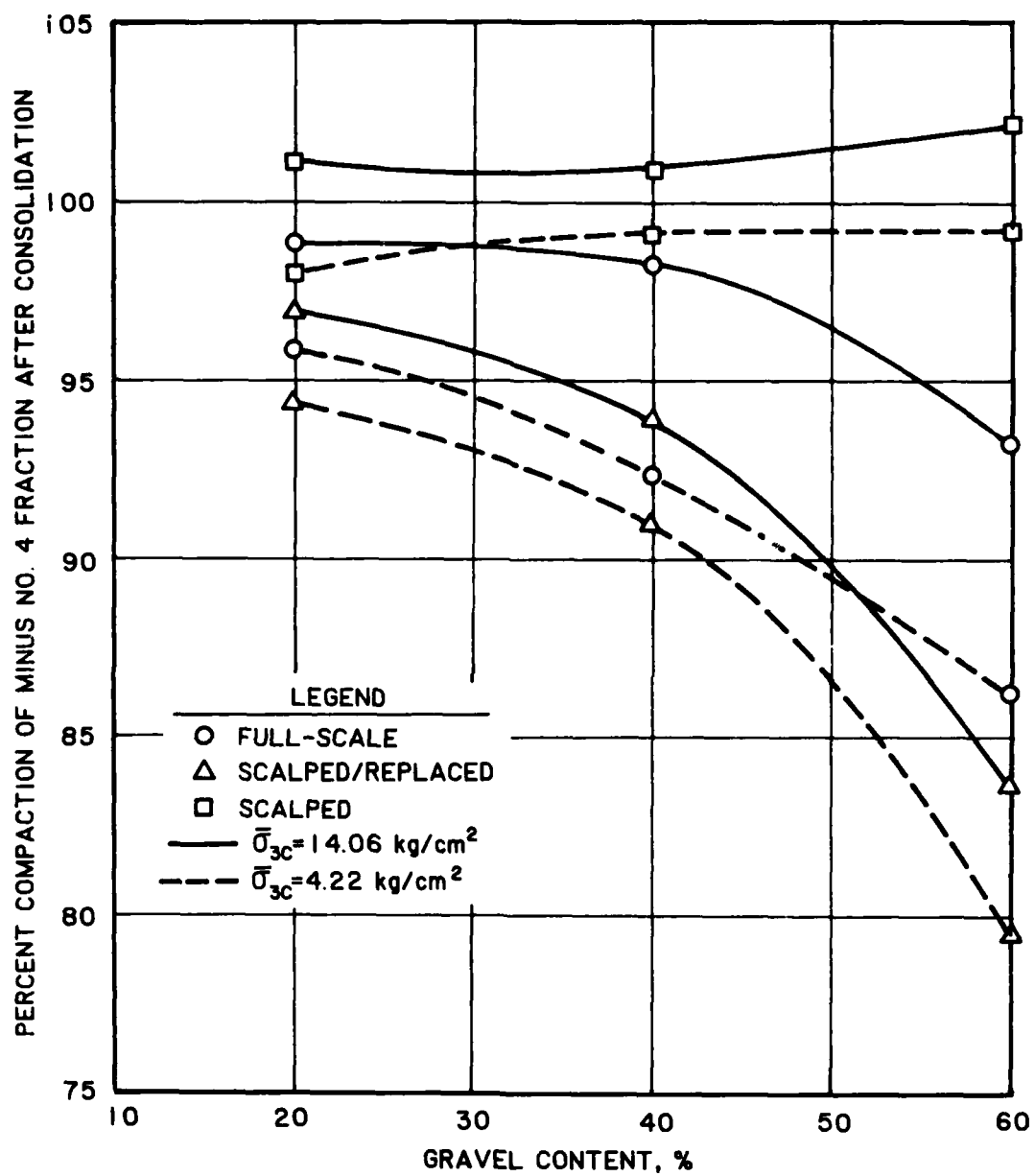


Figure 44. Percent compaction of minus No. 4 fractions after consolidation versus gravel content, blended material

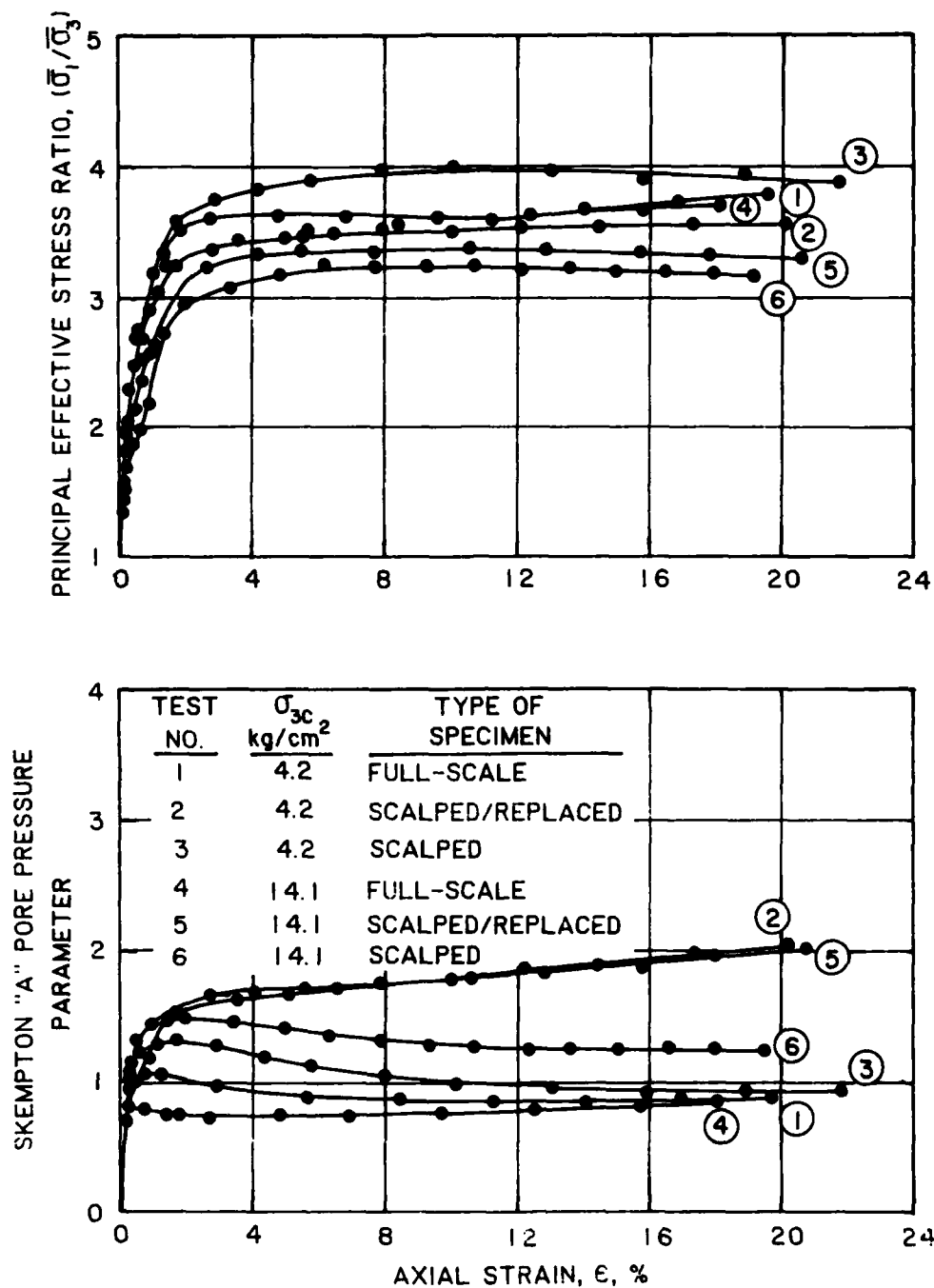


Figure 45. Principal effective stress ratio and Skempton "A" pore pressure parameter versus axial strain, blended material, 20 percent gravel

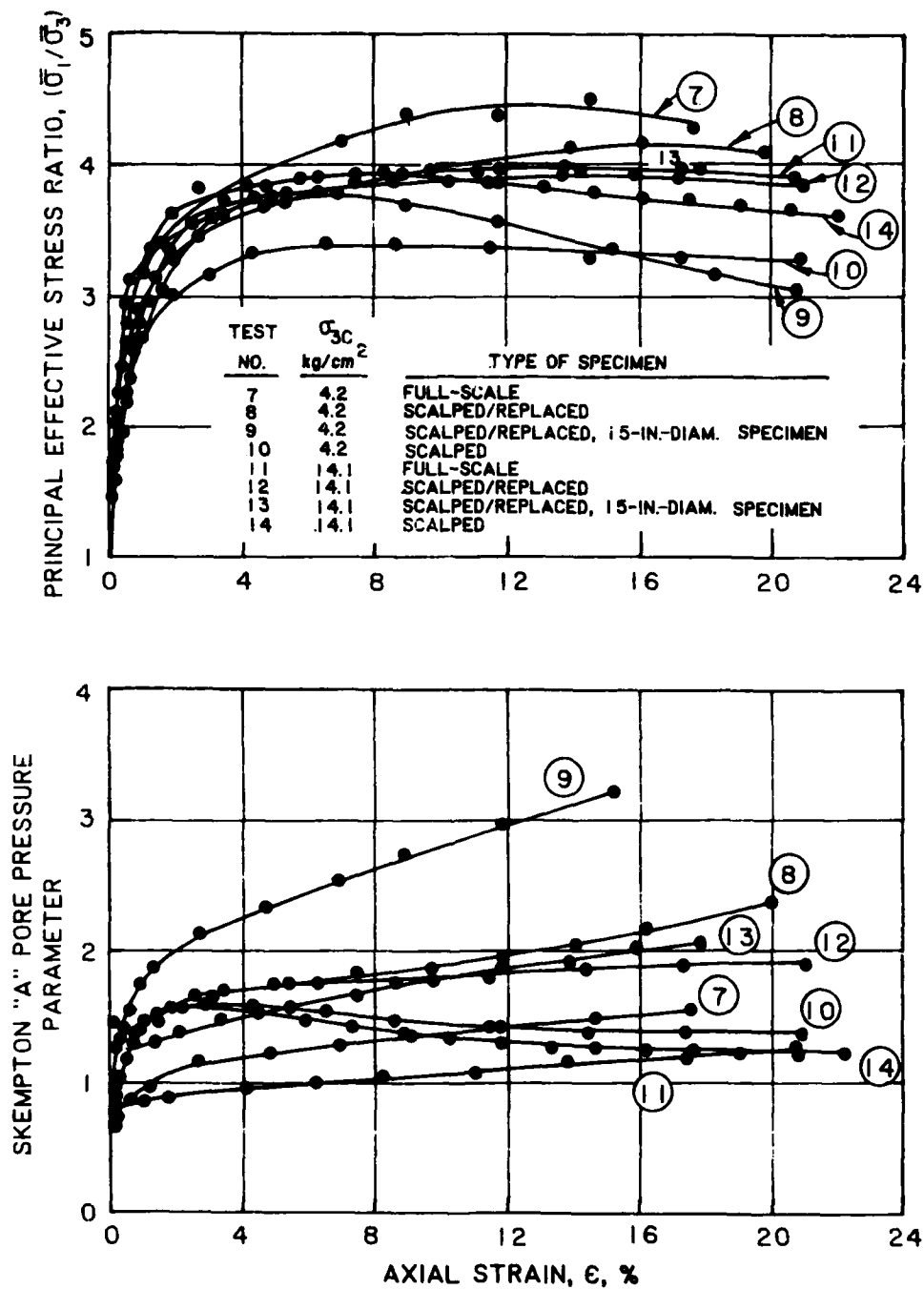


Figure 46. Principal effective stress ratio and Skempton "A" pore pressure parameter versus axial strain, blended material 40 percent gravel

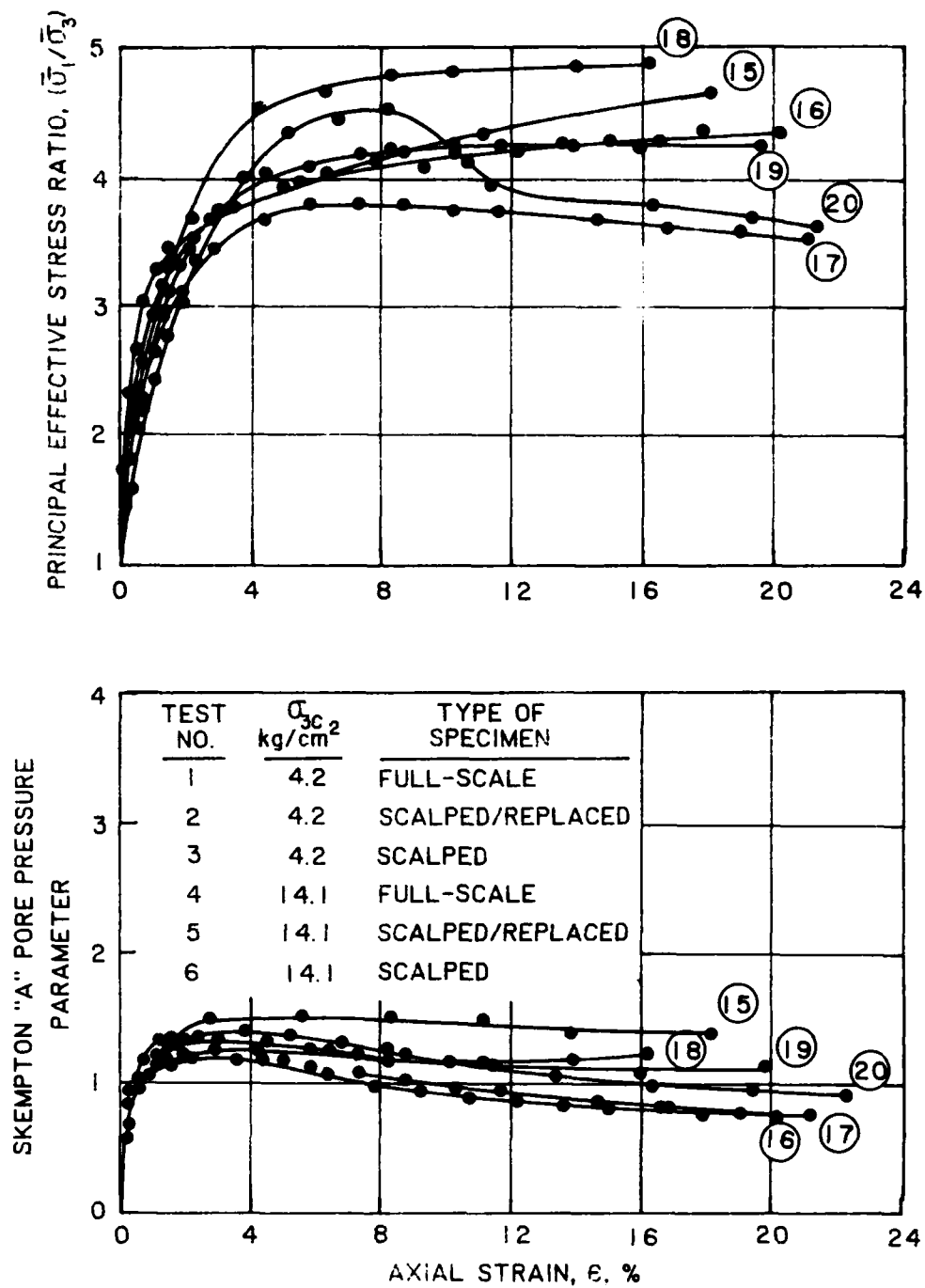


Figure 47. Principal effective stress ratio and Skempton "A" pore pressure parameter versus axial strain, blended material, 60 percent

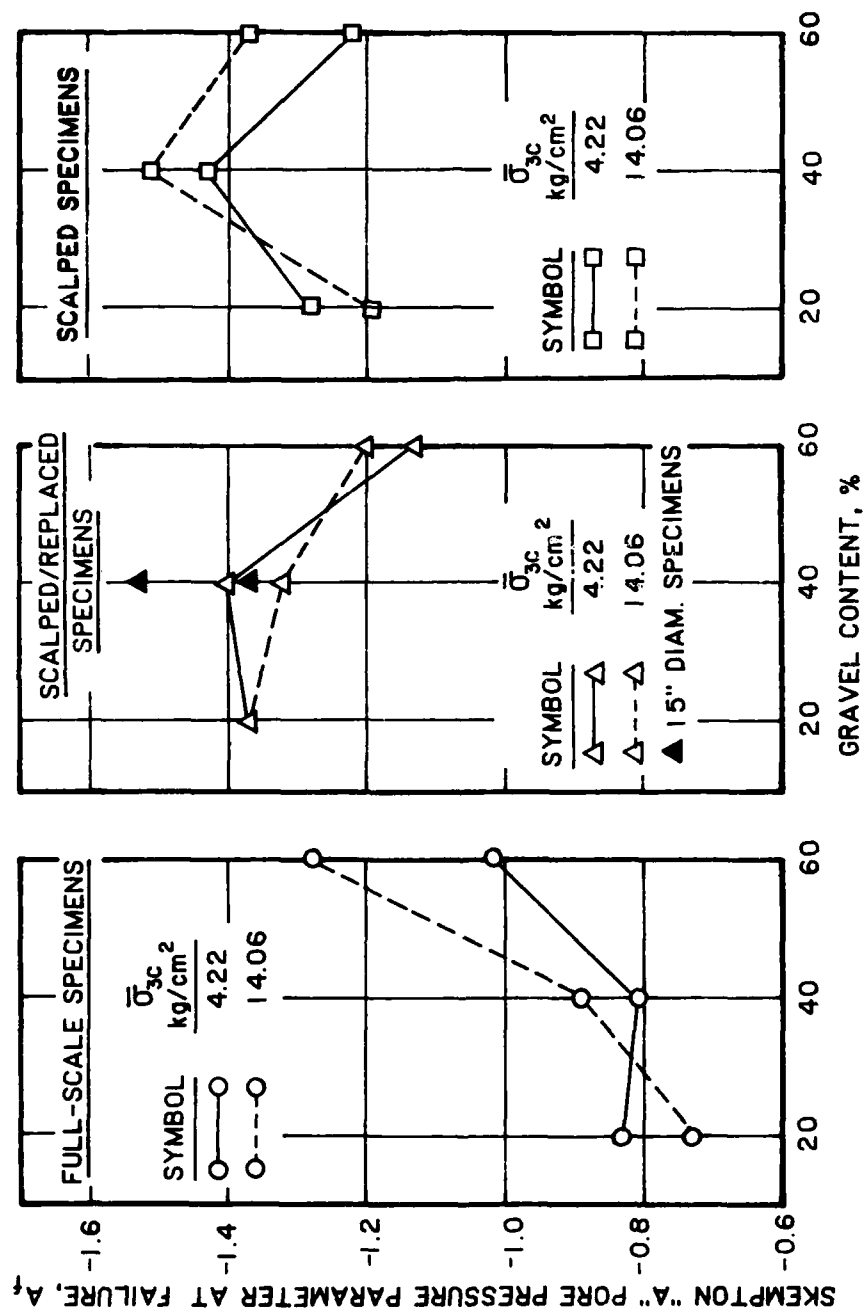


Figure 48. A_f versus gravel content, blended material

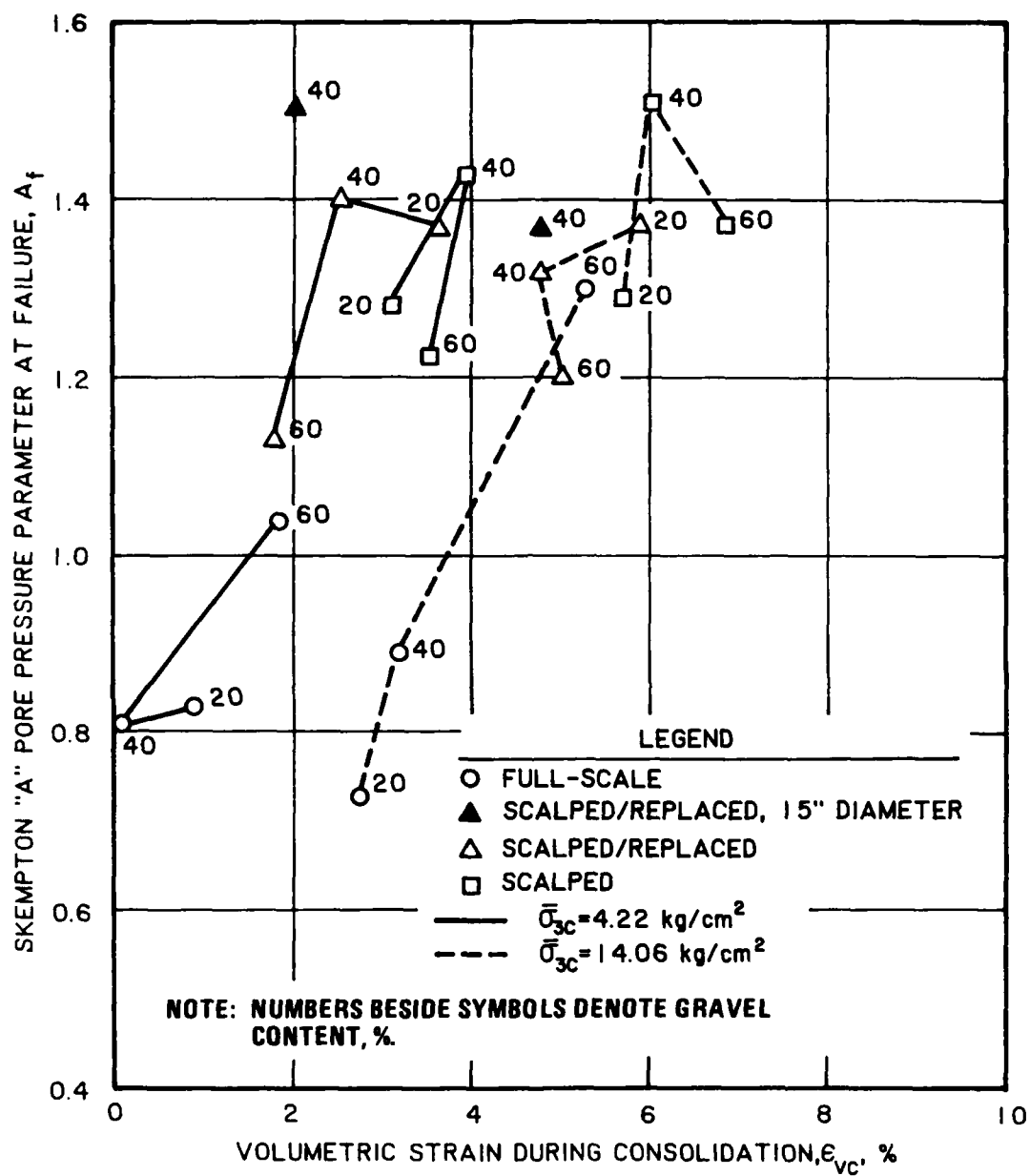


Figure 49. Skempton "A" pore pressure parameter at failure, A_f , versus volumetric strain, ϵ_v , during consolidation, blended material

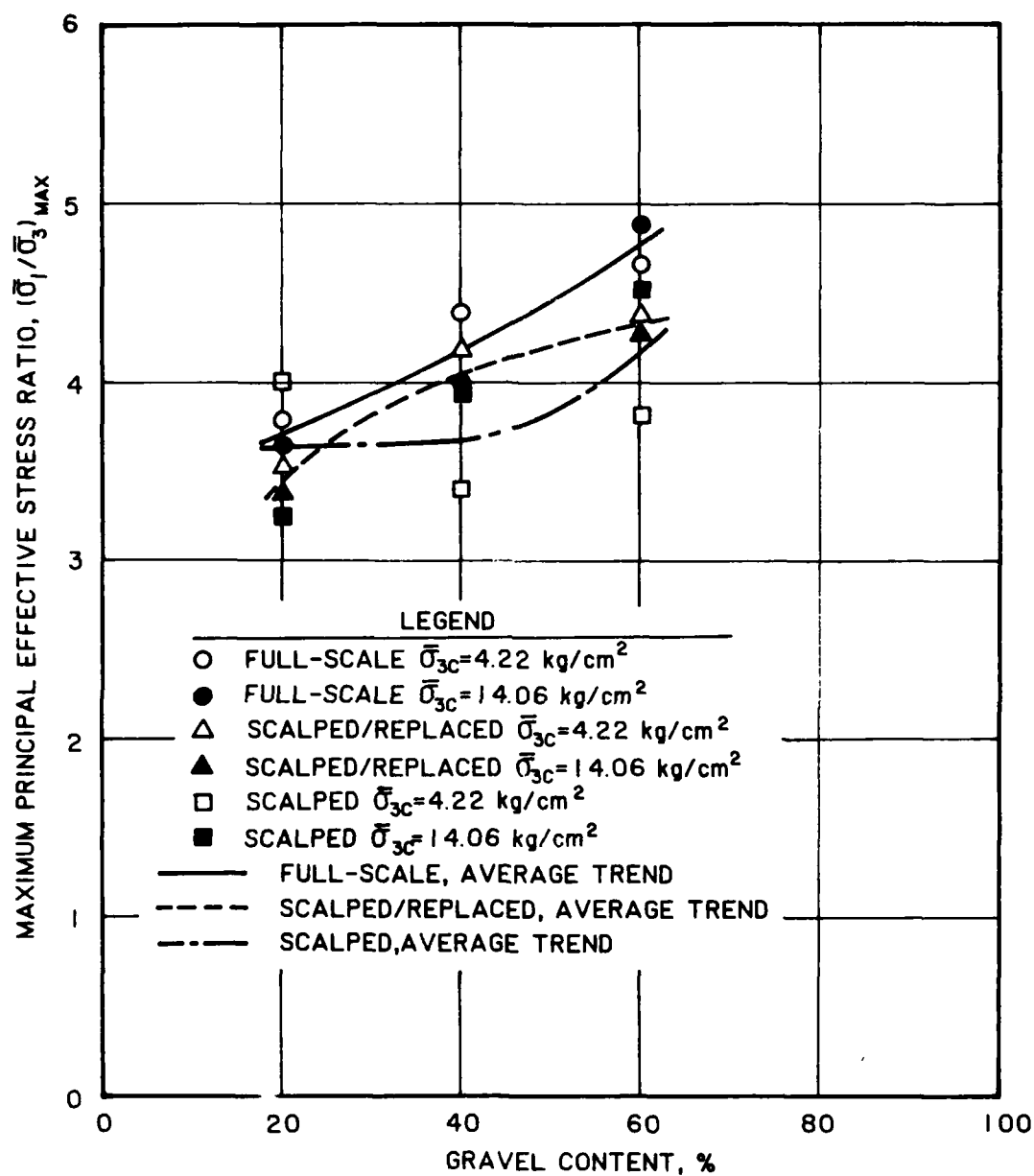


Figure 50. Maximum principal effective stress ratios versus gravel contents, blended material

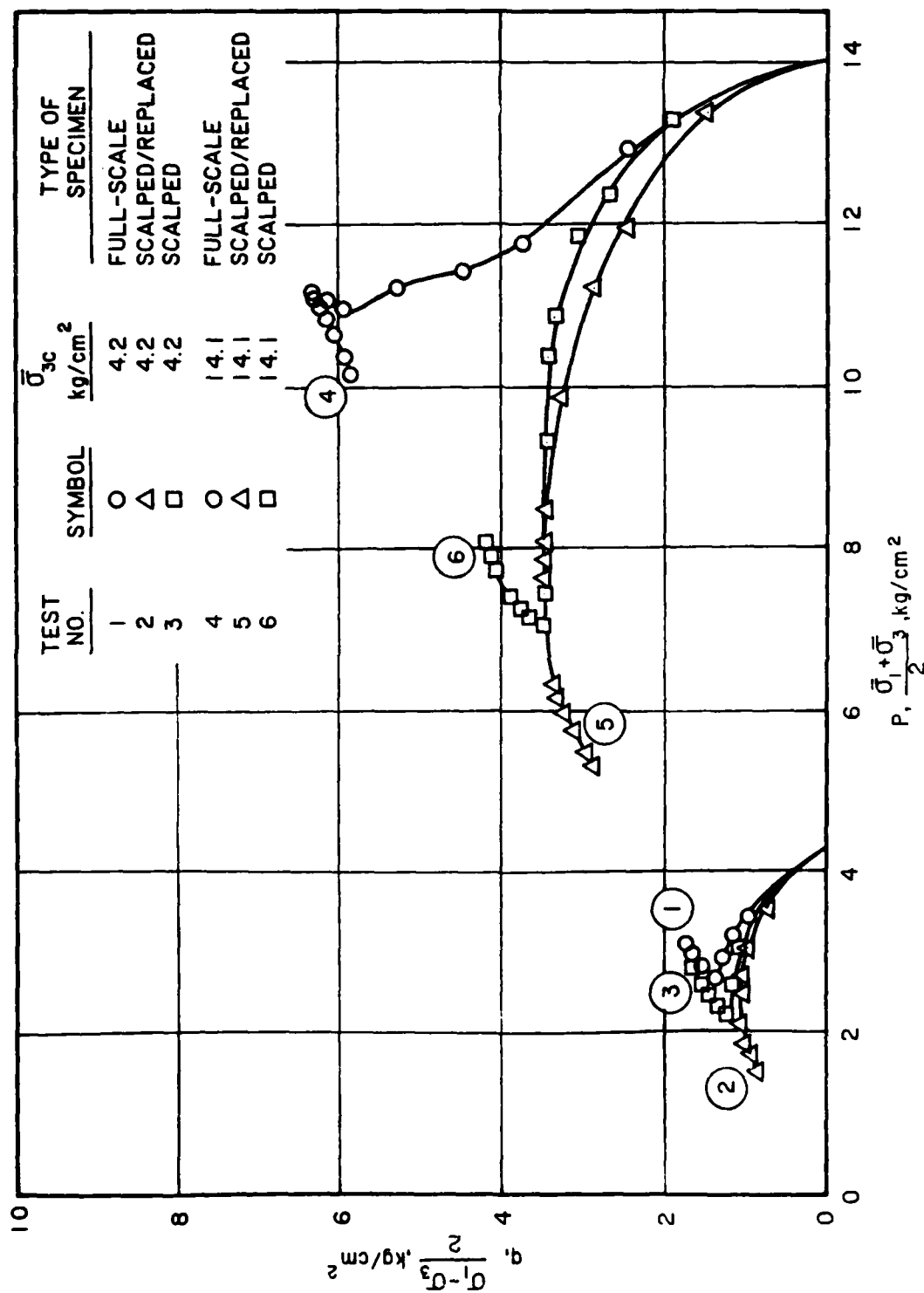


Figure 51. Effective stress paths, blended material, 20 percent gravel

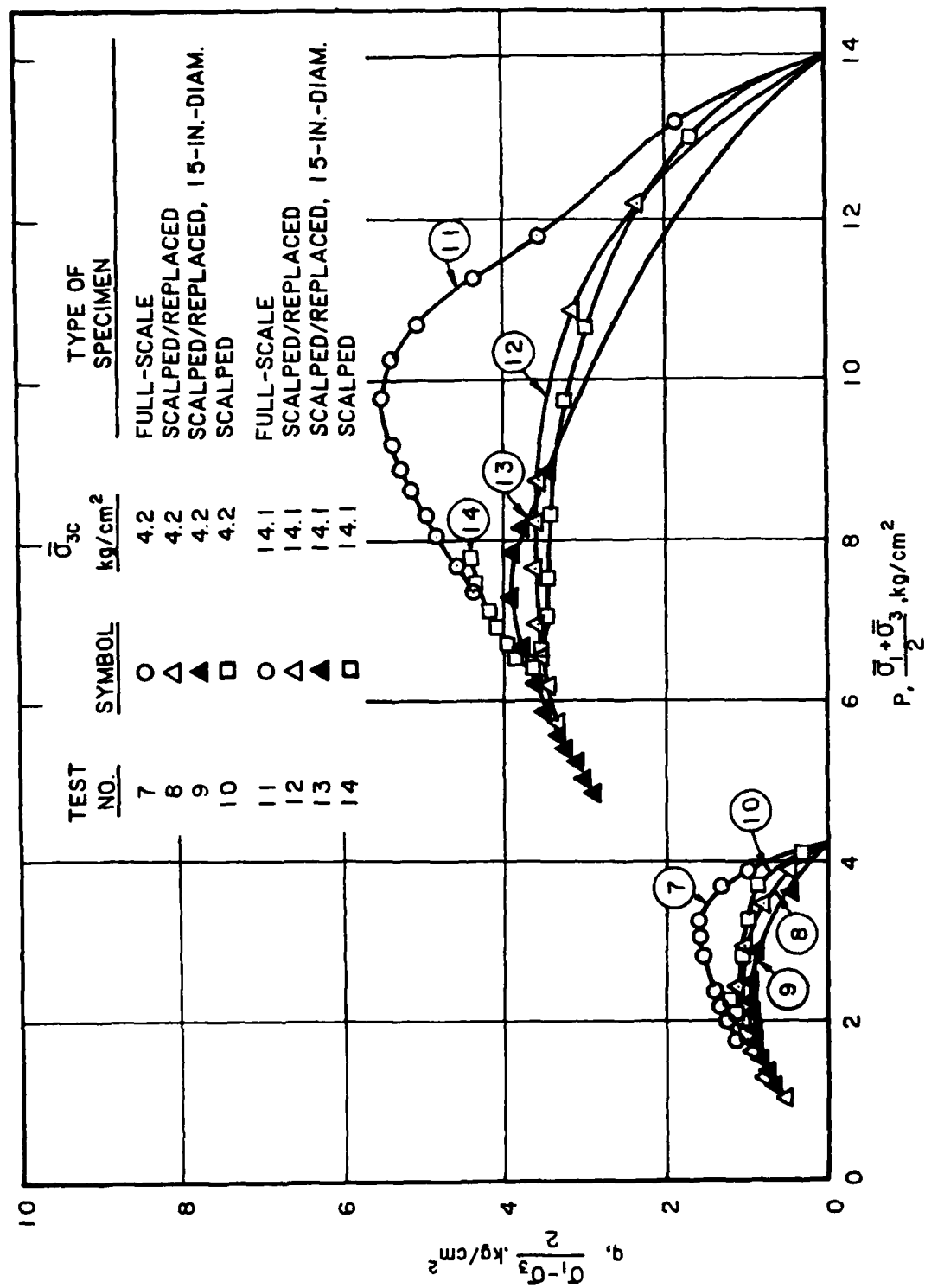


Figure 52. Effective stress paths, blended material, 40 percent gravel

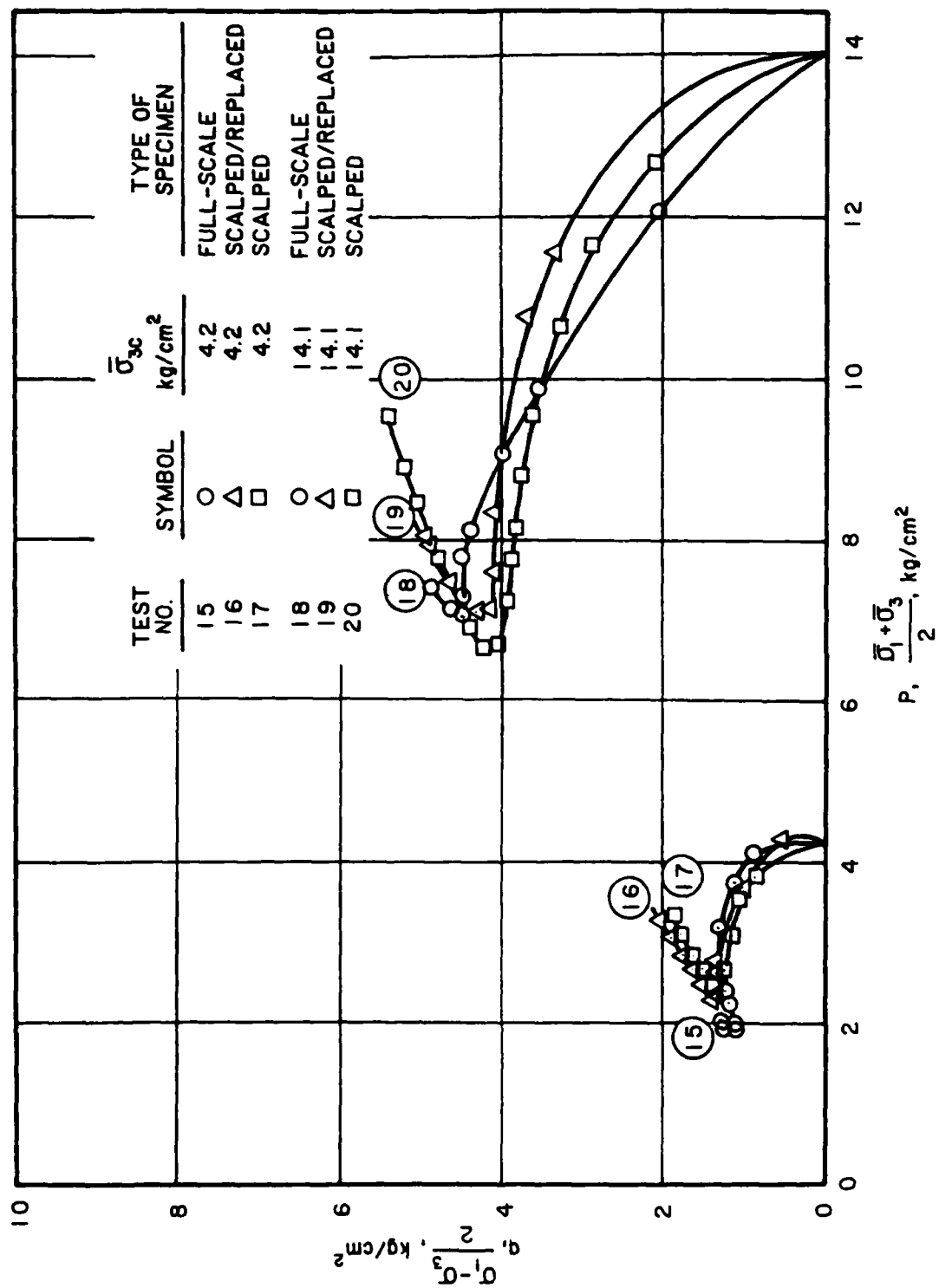


Figure 53. Effective stress paths, blended material, 60 percent gravel

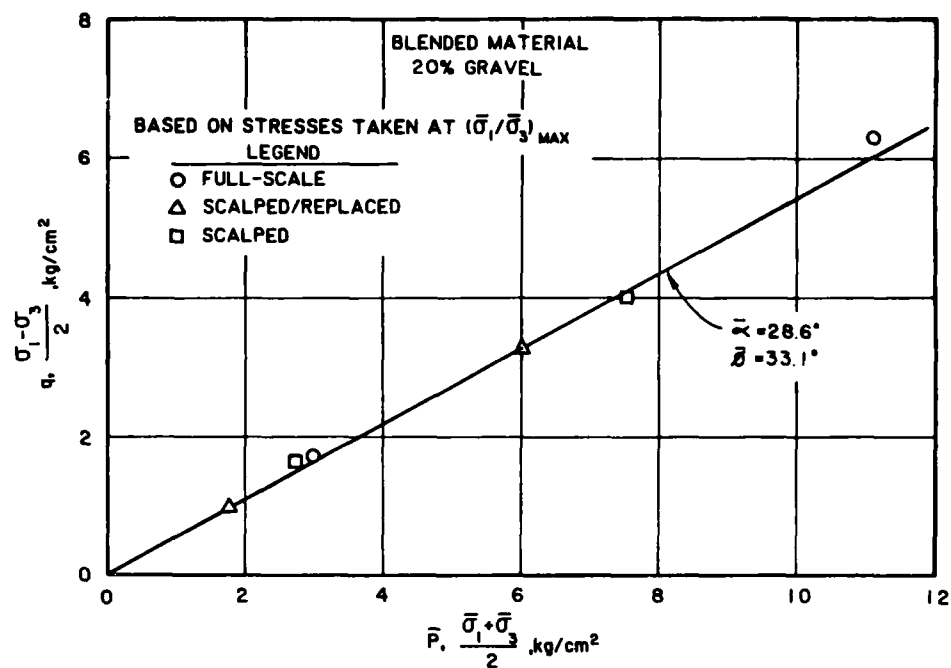


Figure 54. Maximum principal effective stress ratios plotted in terms of q and p , blended material, 20 percent gravel

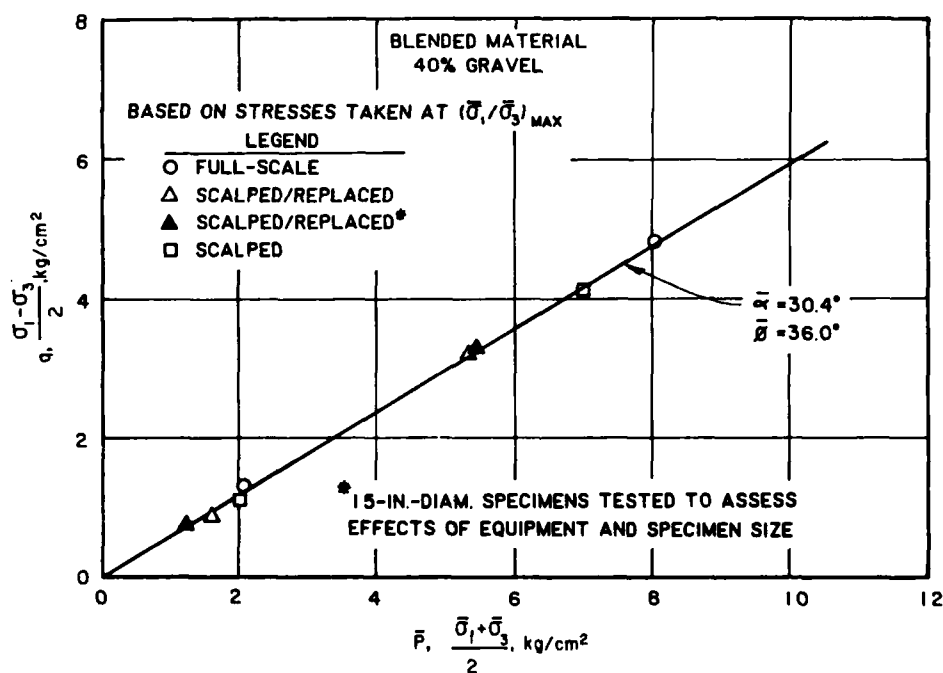


Figure 55. Maximum principal effective stress ratios plotted in terms of q and p , blended material, 40 percent gravel

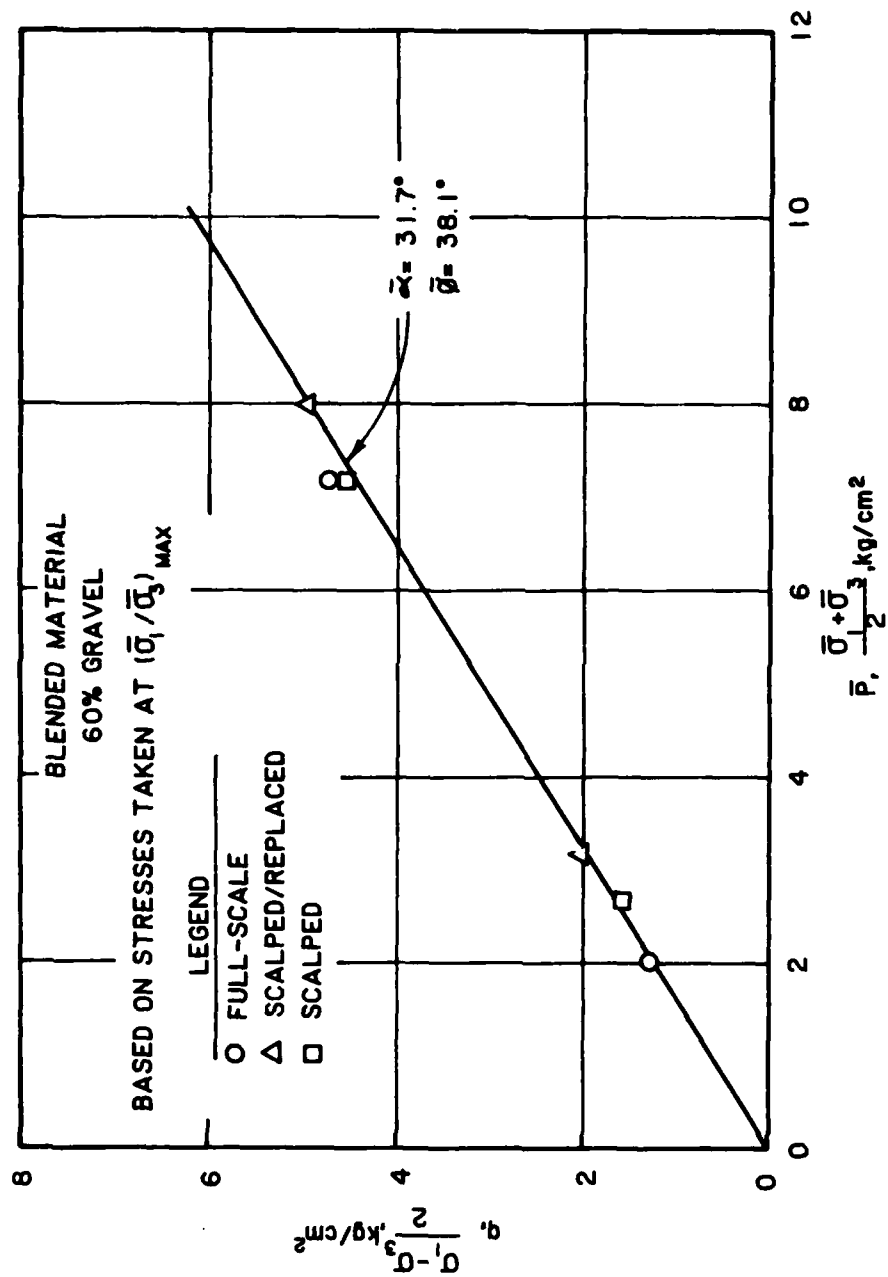


Figure 56. Maximum principal effective stress ratios plotted in terms of $\bar{q}$ and $\bar{p}$, blended material, 60 percent gravel

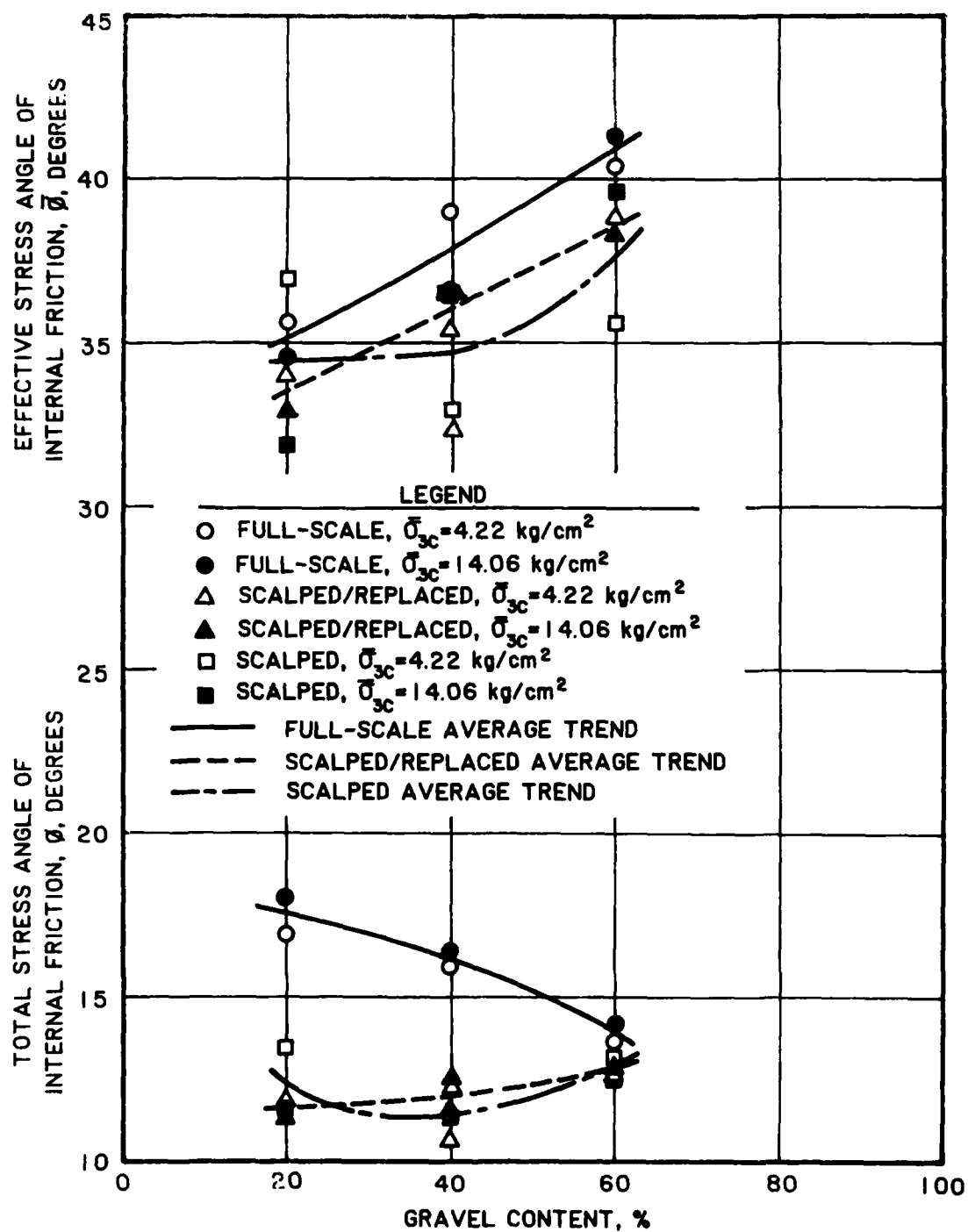


Figure 57. Total and effective angles of internal friction versus gravel content, blended material

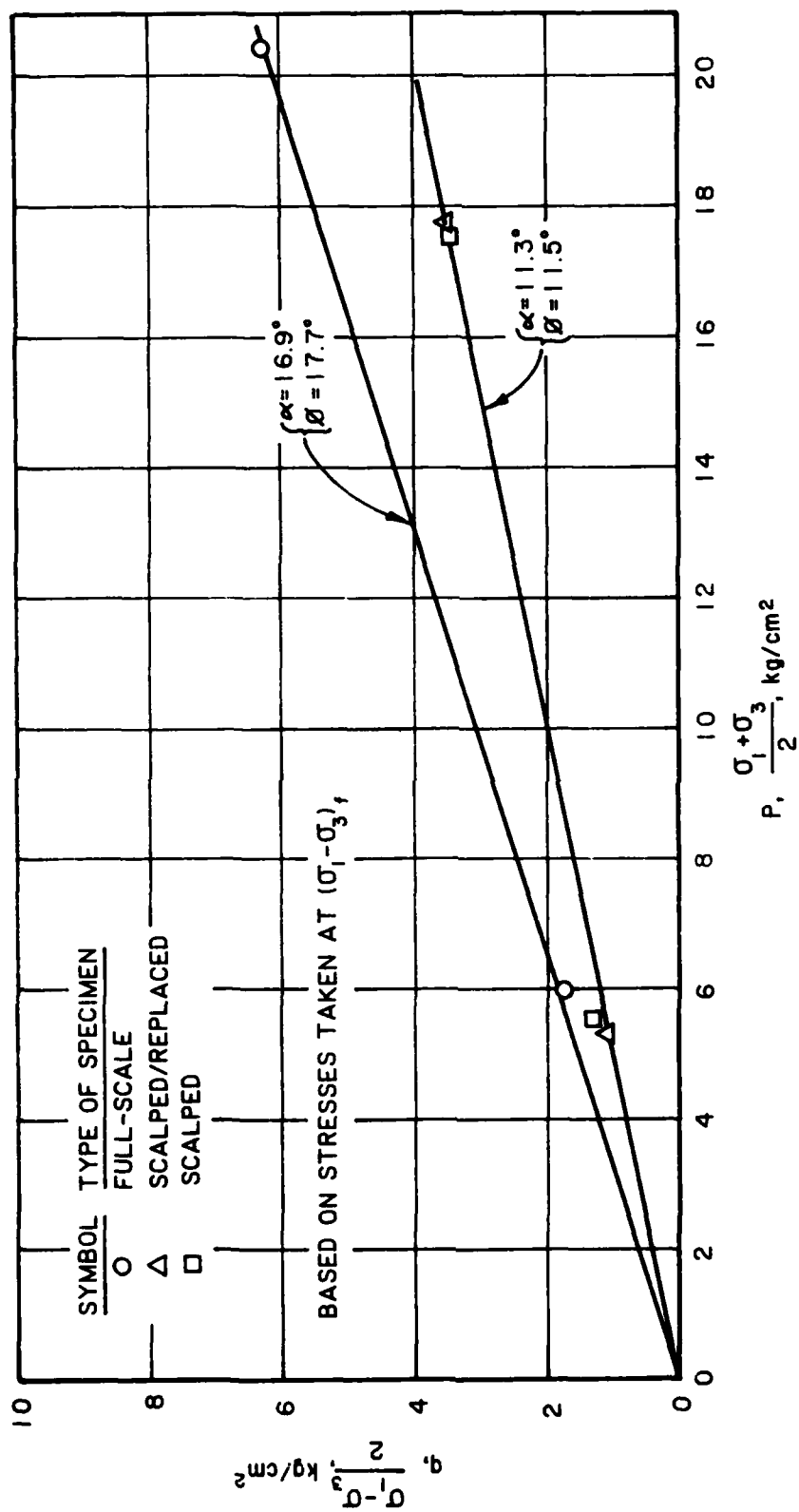


Figure 58. Deviator stresses at failure plotted in terms of q and p , blended material, 20 percent gravel

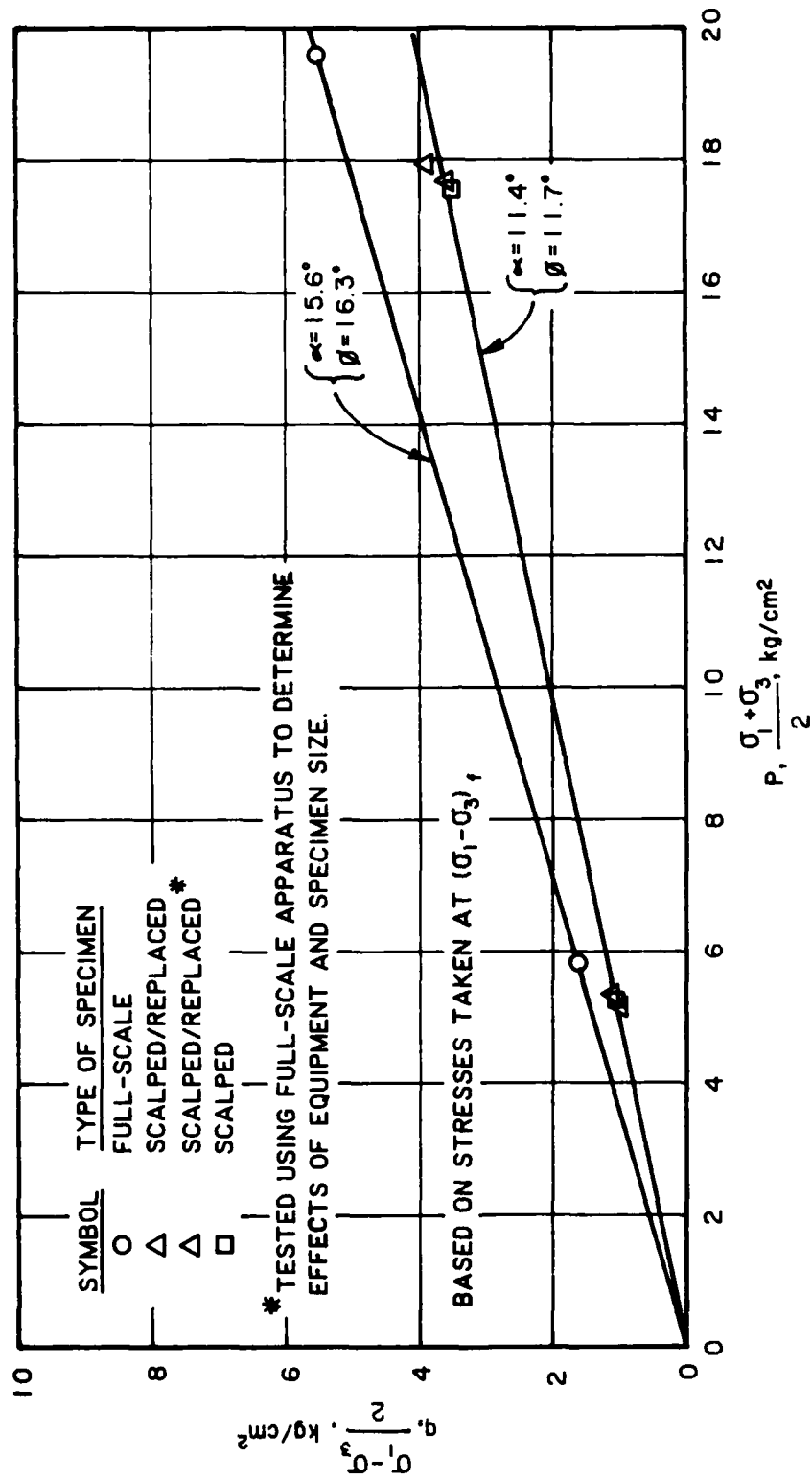


Figure 59. Deviator stresses at failure plotted in terms of q and p , blended material, 40 percent gravel

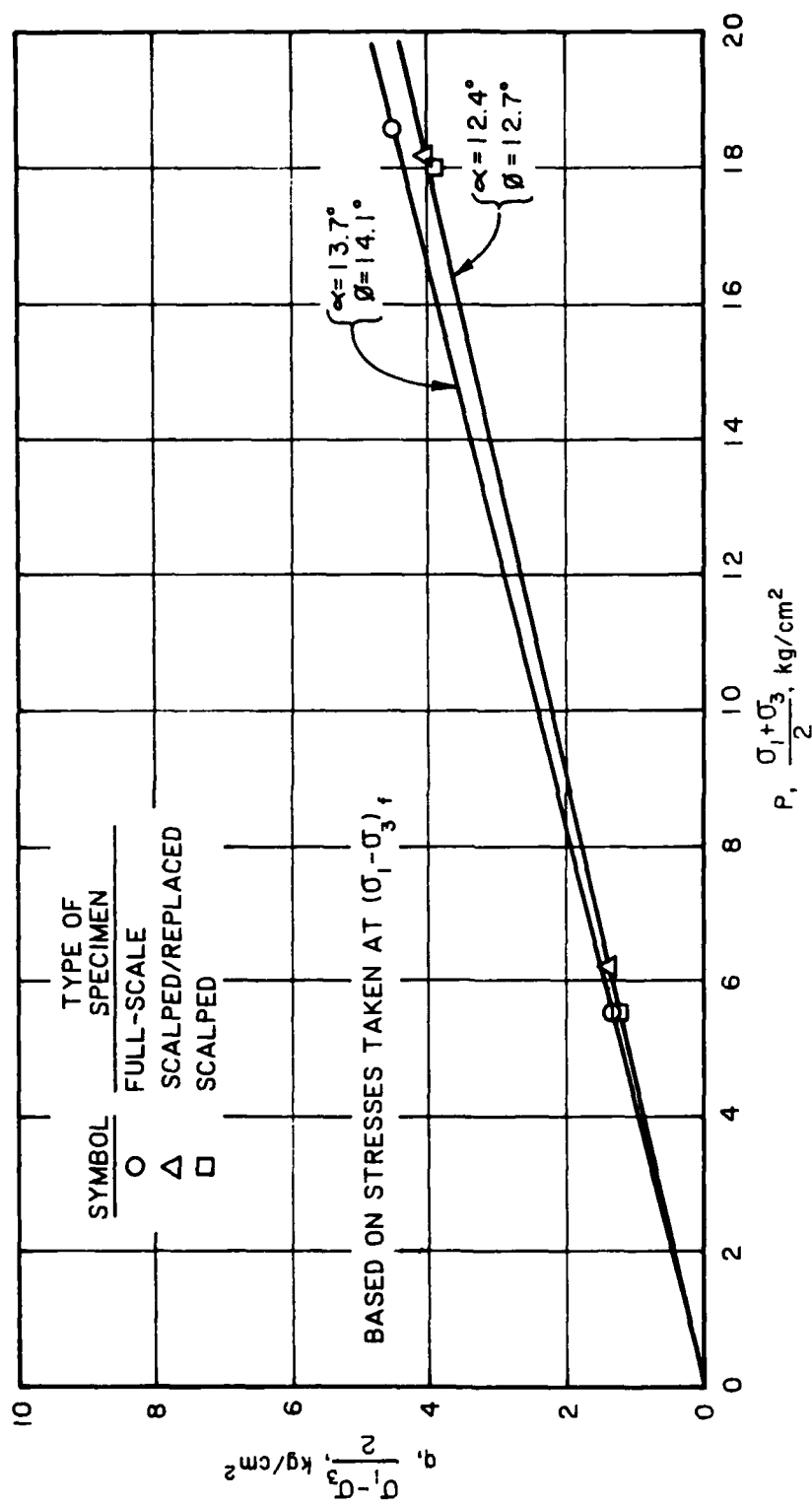


Figure 60. Deviator stresses at failure plotted in terms of q and p , blended material, 60 percent gravel

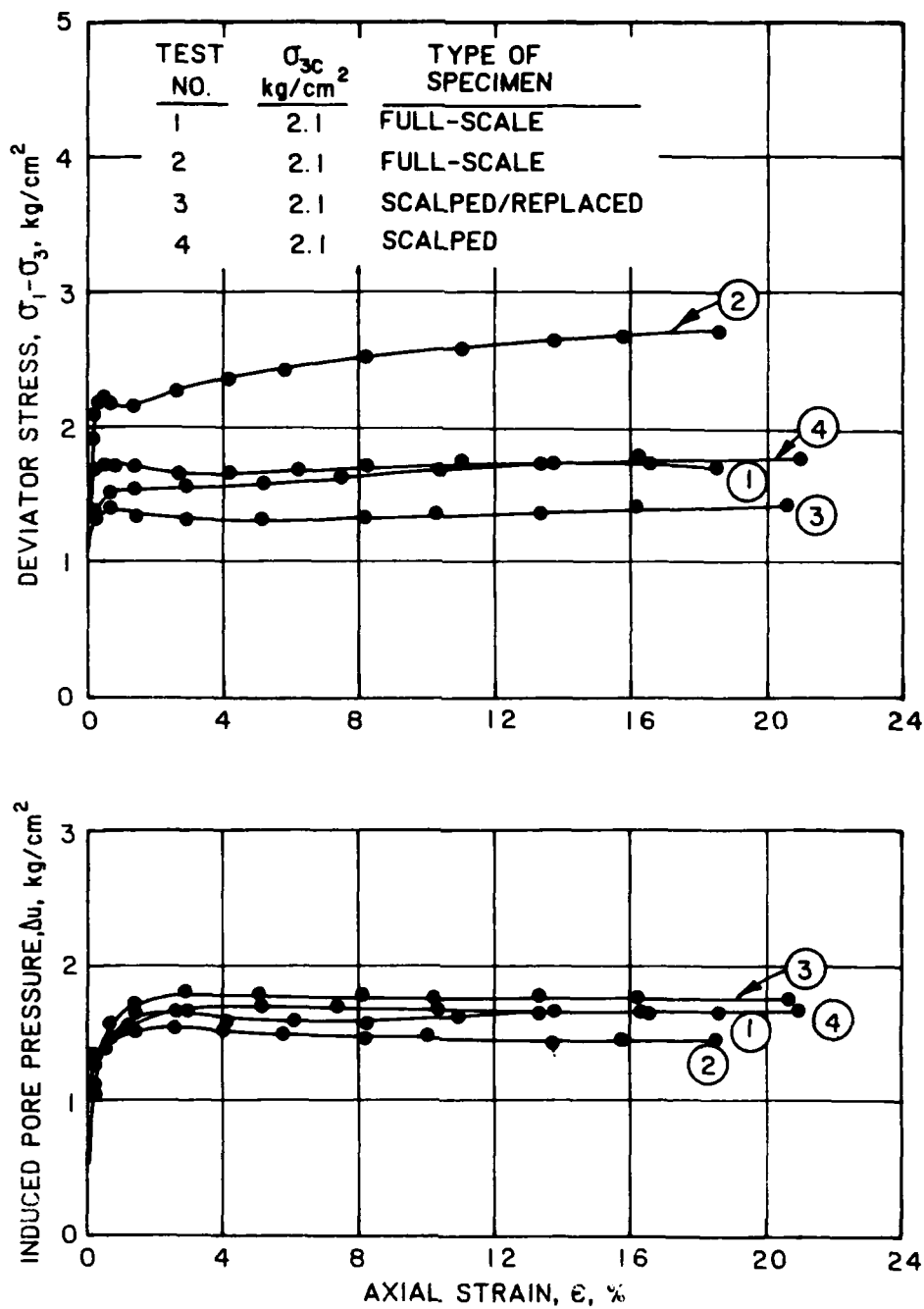


Figure 61. Deviator stress and induced pore pressure versus axial strain, DeGray Dam material, $\sigma_{3c} = 2.1$ kg/cm²

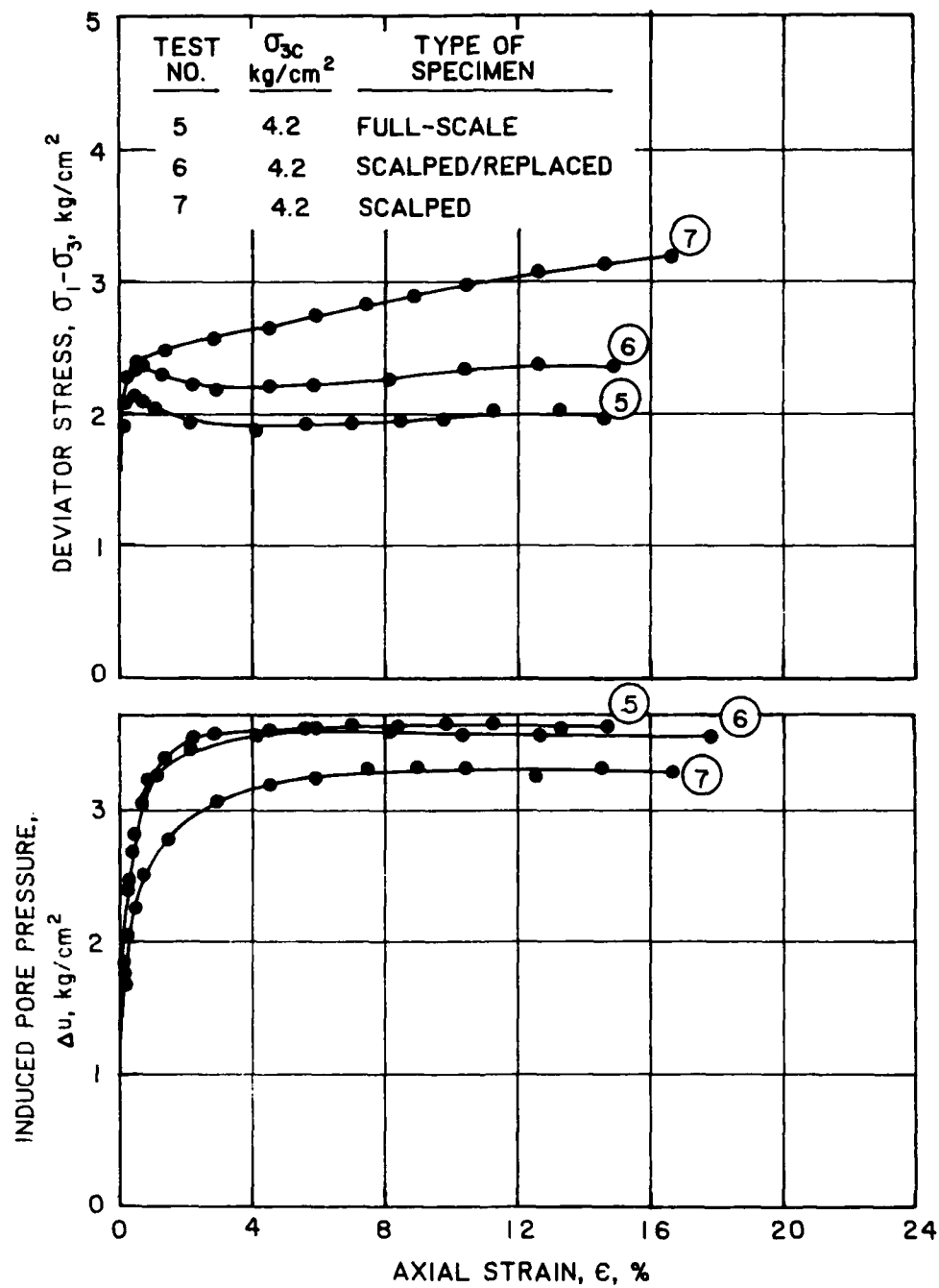


Figure 62. Deviator stress and induced pore pressure versus axial strain, DeGray Dam material, $\bar{\sigma}_{3c} = 4.2 \text{ kg/cm}^2$

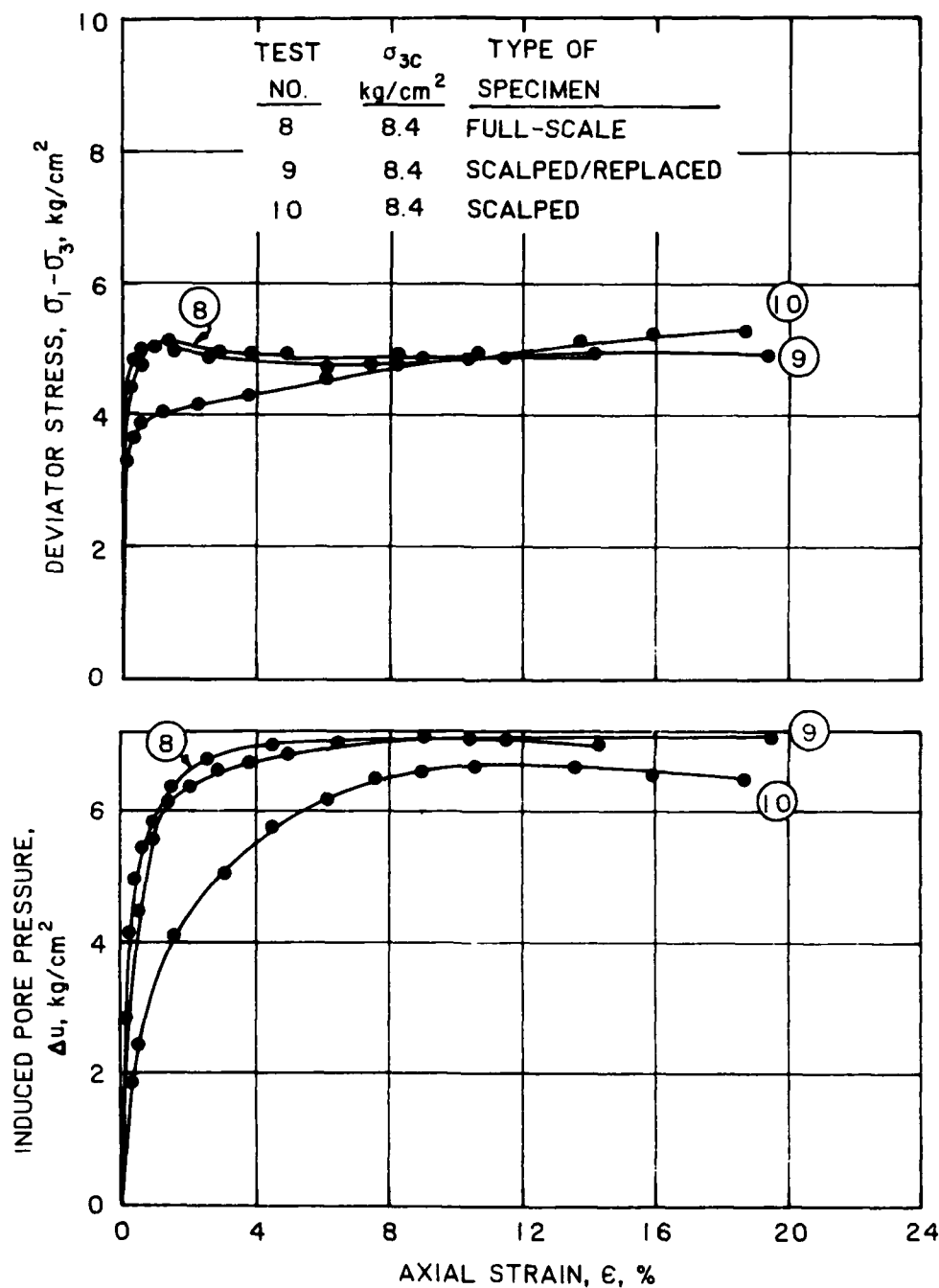


Figure 63. Deviator stress and induced pore pressure versus axial strain, DeGray Dam material, $\bar{\sigma}_{3c} = 8.4$ kg/cm²

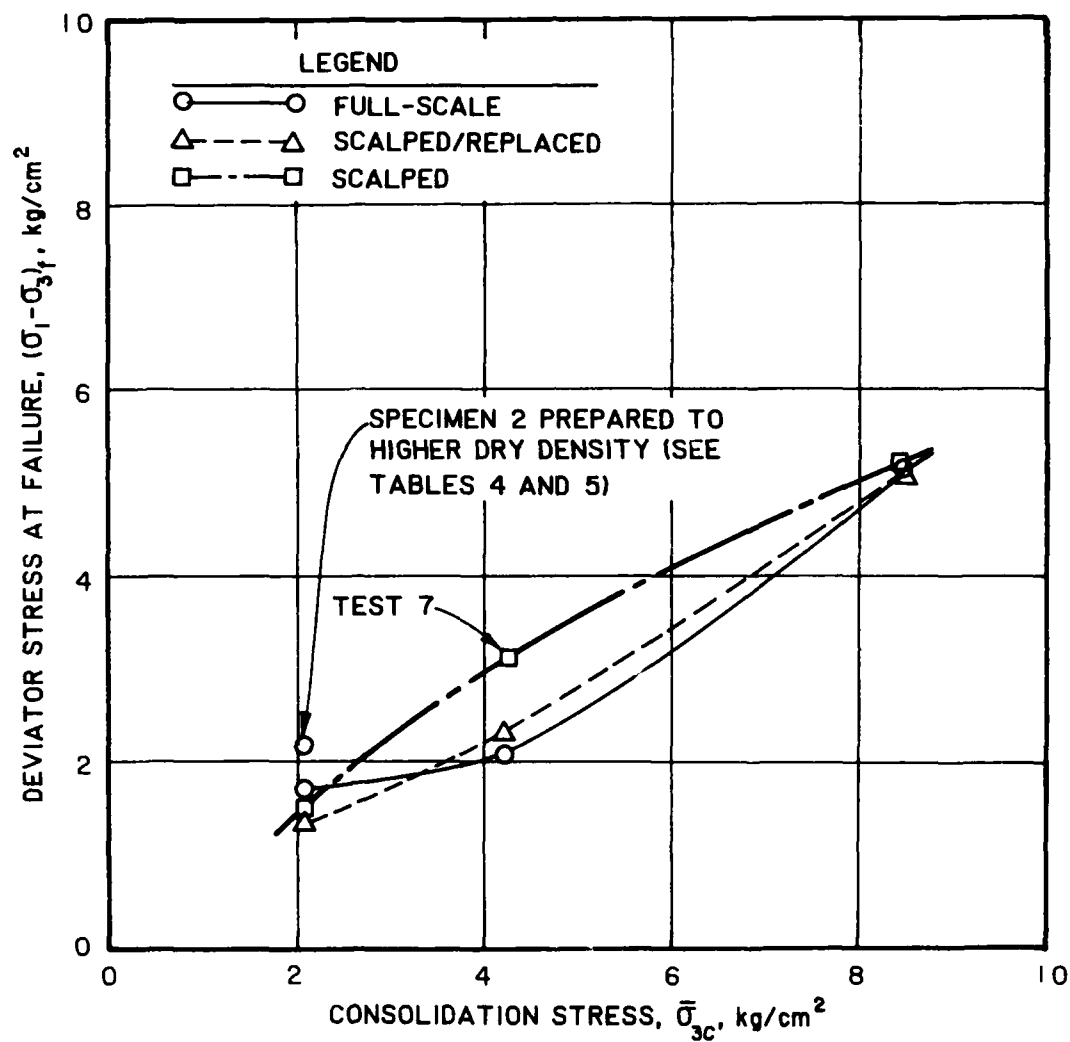


Figure 64. Deviator stresses at failure versus confining pressure, DeGray Dam material

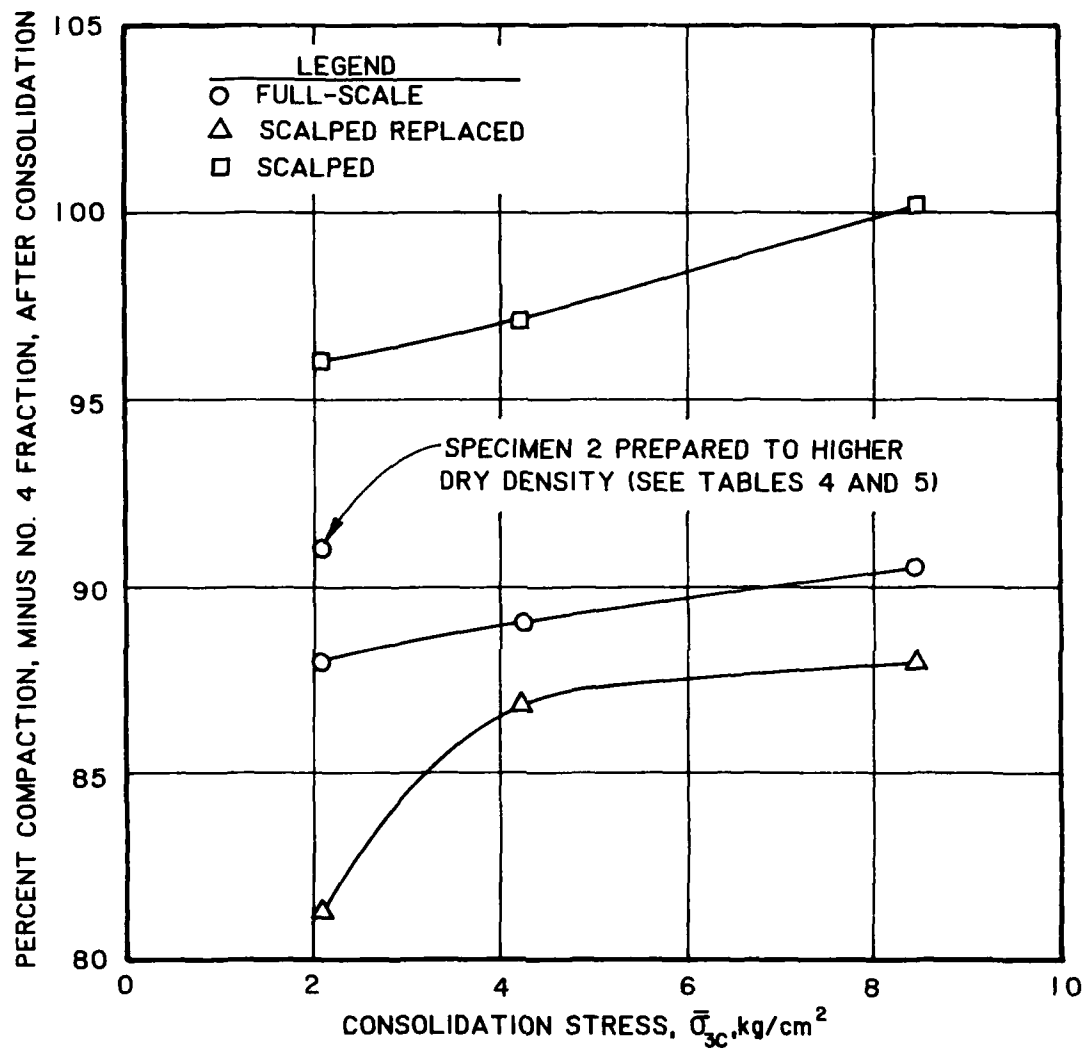


Figure 65. Percent compaction of minus No. 4 fractions versus confining pressure, DeGray Dam material

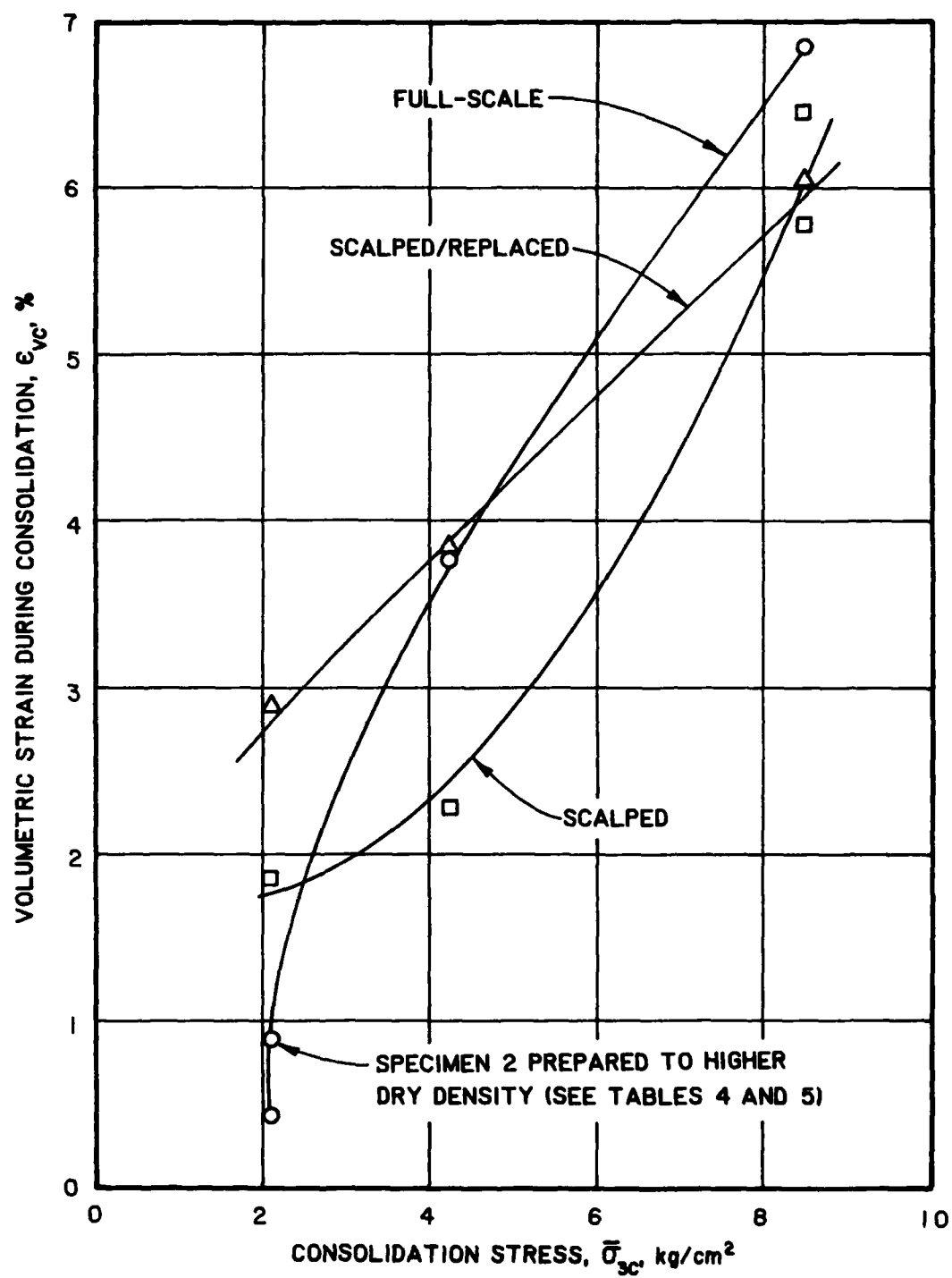


Figure 66. Volumetric strain during consolidation versus confining pressure, DeGray Dam material

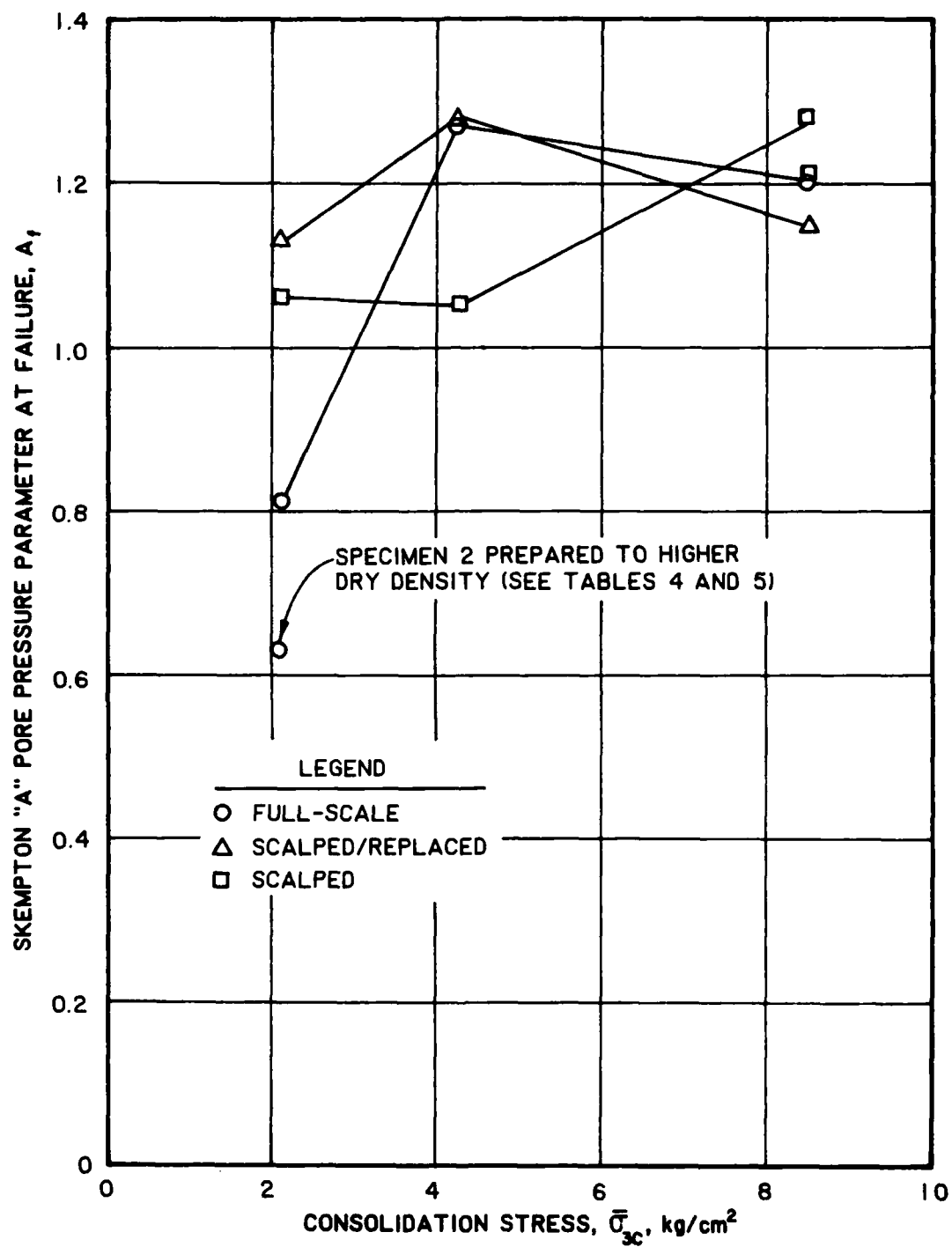


Figure 67. Skempton "A" pore pressure parameter at failure, A_f , versus confining pressure, DeGray Dam material

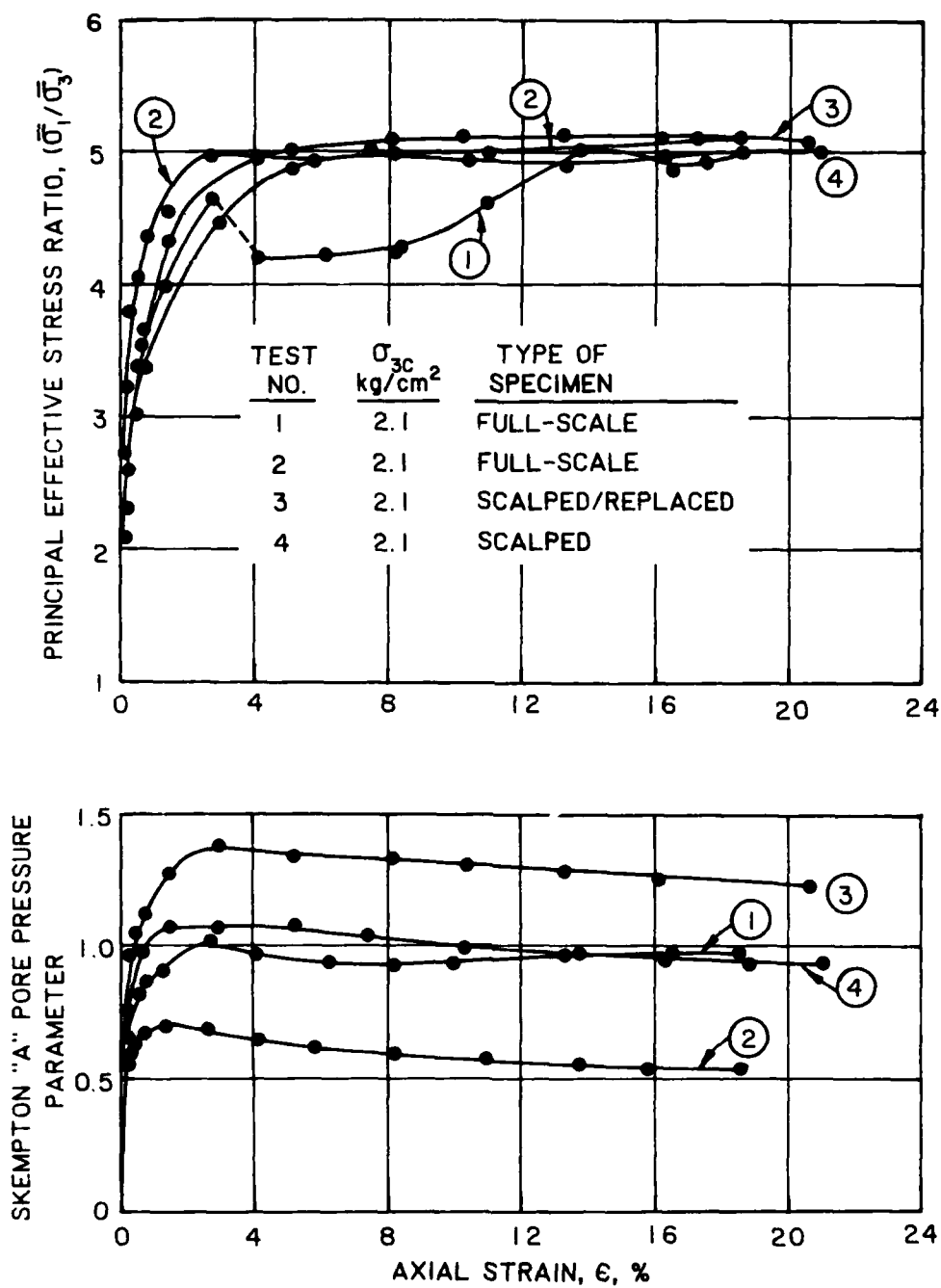


Figure 68. Principal effective stress ratios and Skempton "A" pore pressure parameters versus axial strain, DeGray Dam material, $\sigma_{3c} = 2.11 \text{ kg/cm}^2$

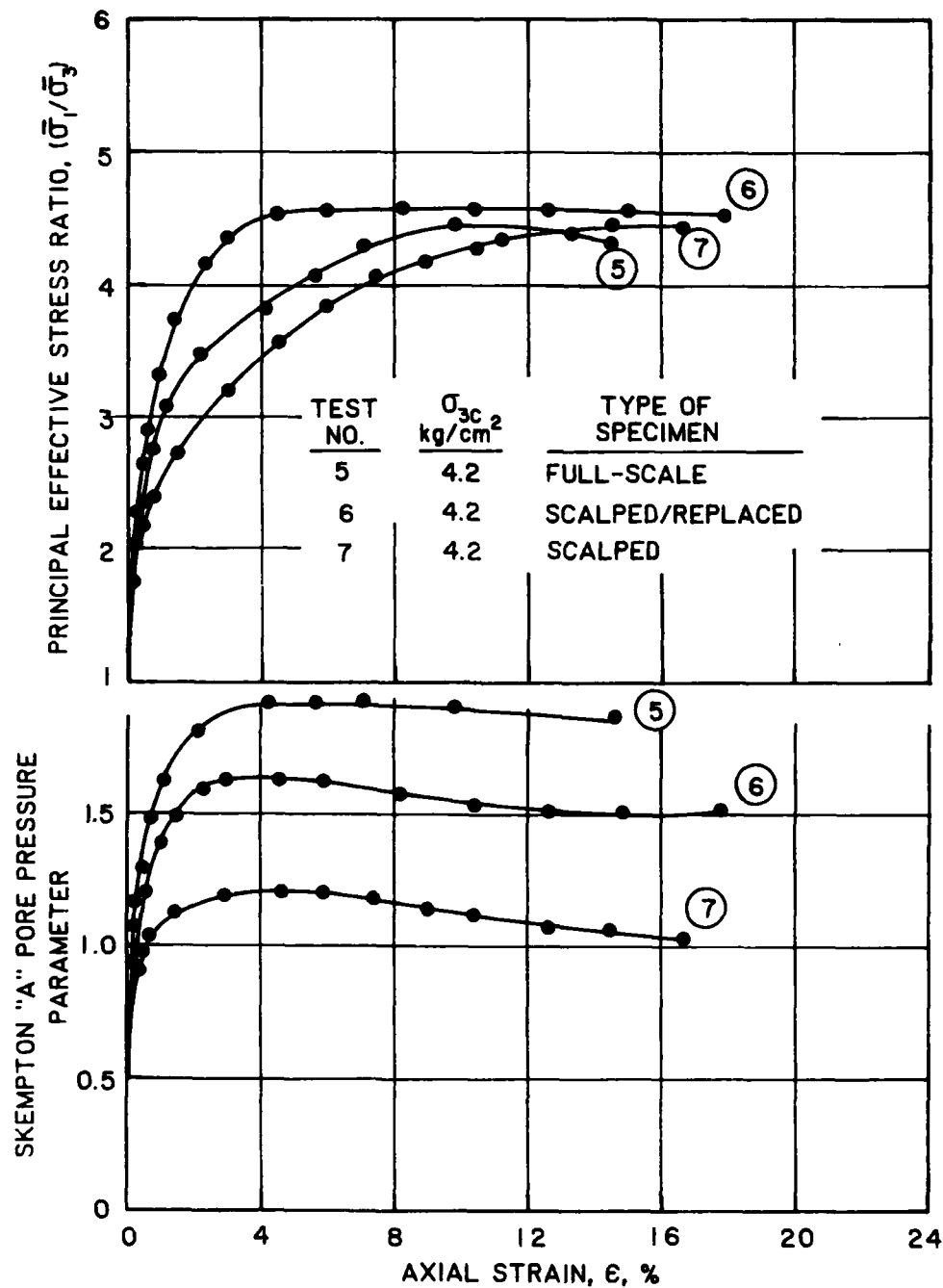


Figure 69. Principal effective stress ratios and Skempton "A" pore pressure parameters versus axial strain, DeGray Dam material, $\bar{\sigma}_{3c} = 4.22 \text{ kg/cm}^2$

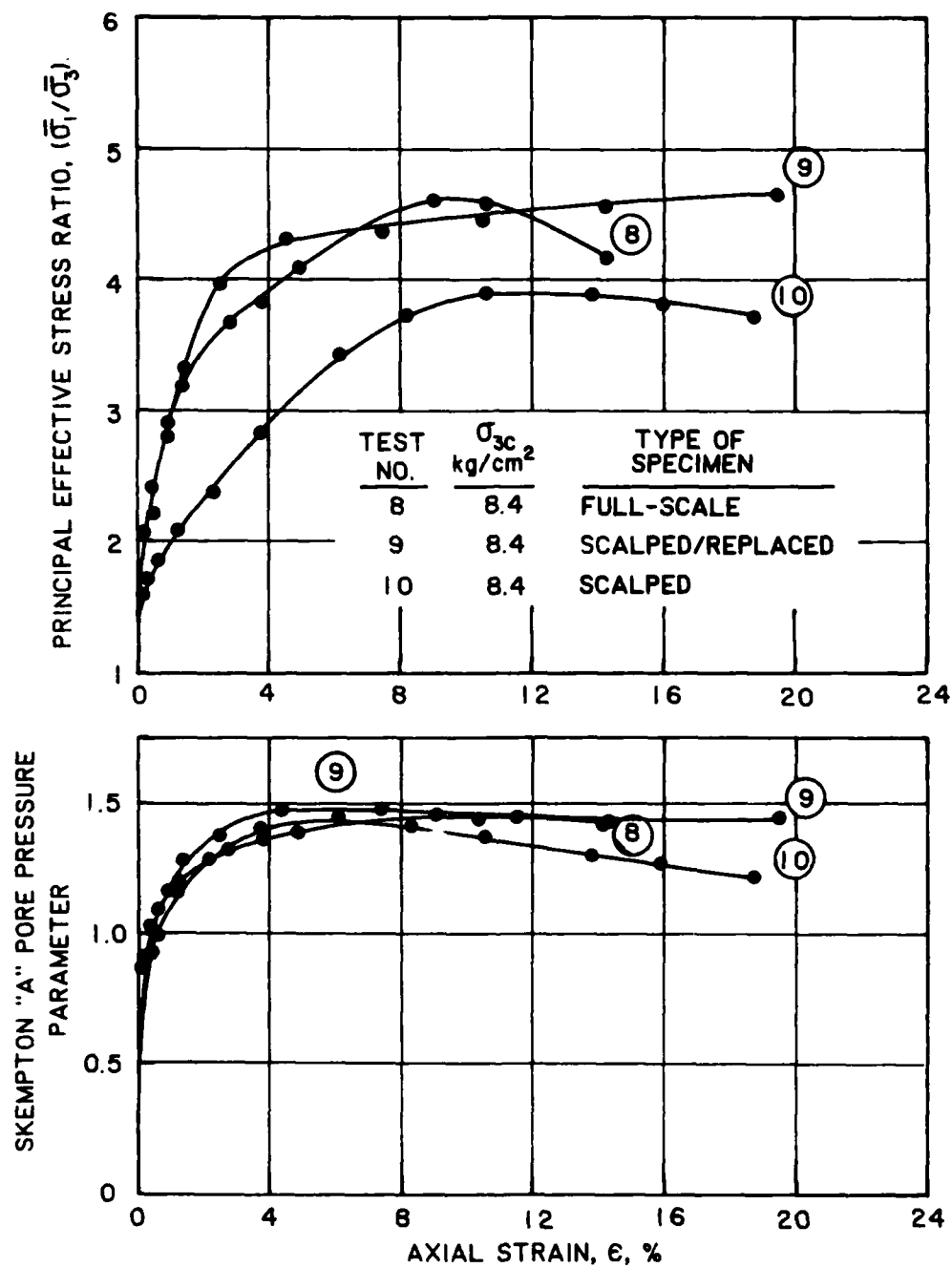


Figure 70. Principal effective stress ratios and Skempton "A" pore pressure parameters versus axial strain, DeGray Dam material, $\bar{\sigma}_{3c} = 8.44 \text{ kg/cm}^2$

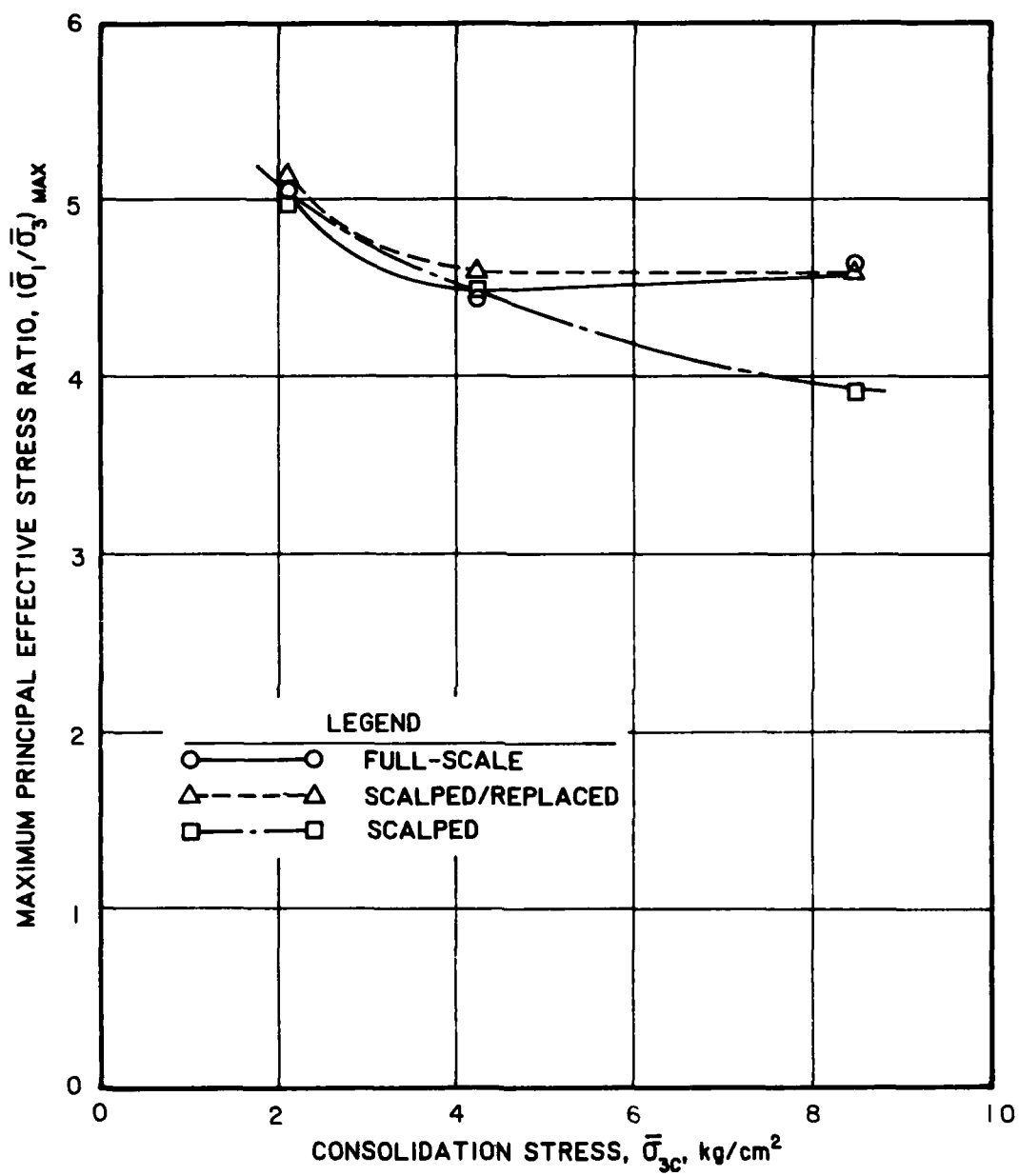


Figure 71. Maximum principal effective stress ratios versus confining pressure, DeGray Dam material

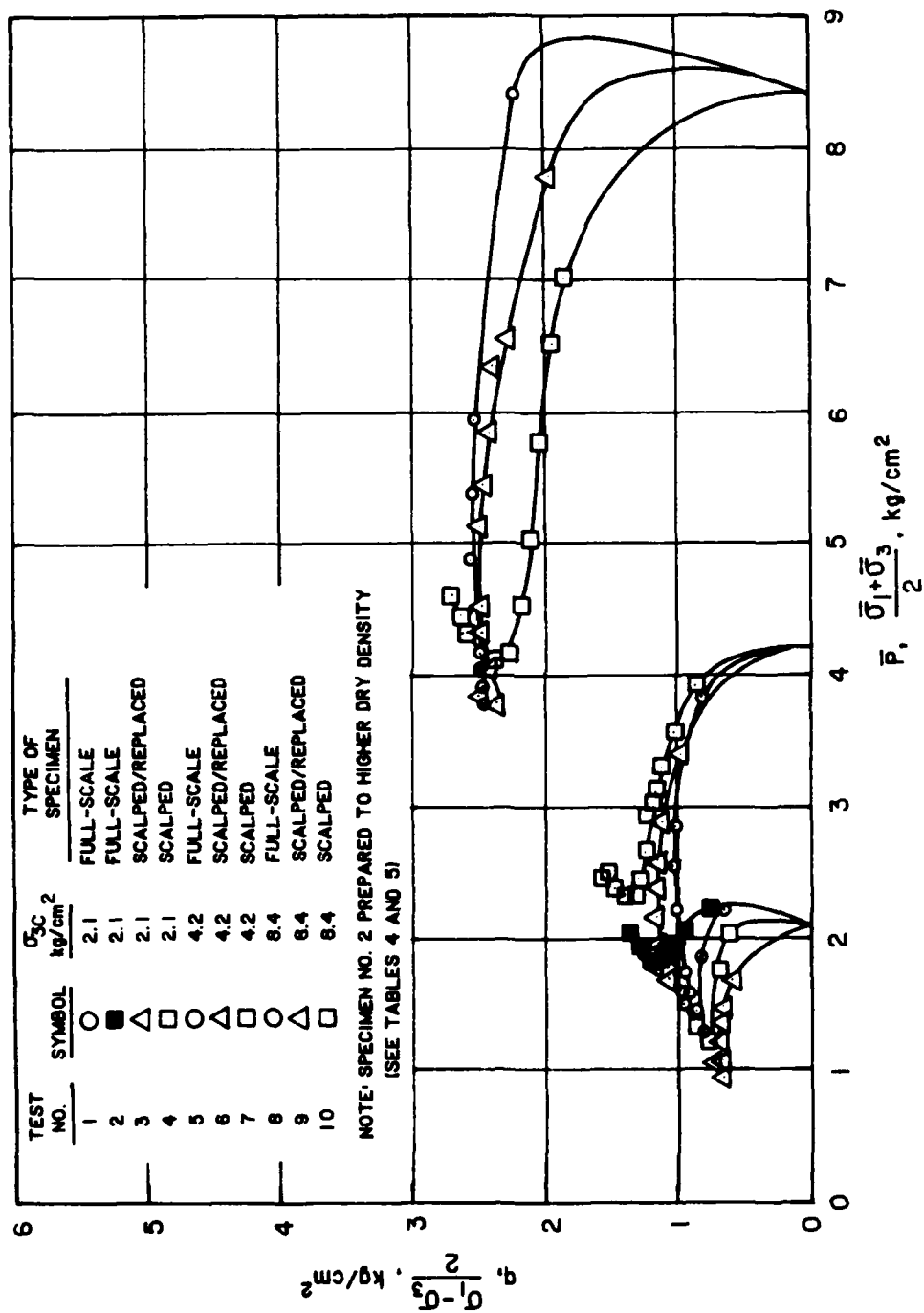


Figure 72. Effective stress paths, DeGray Dam material

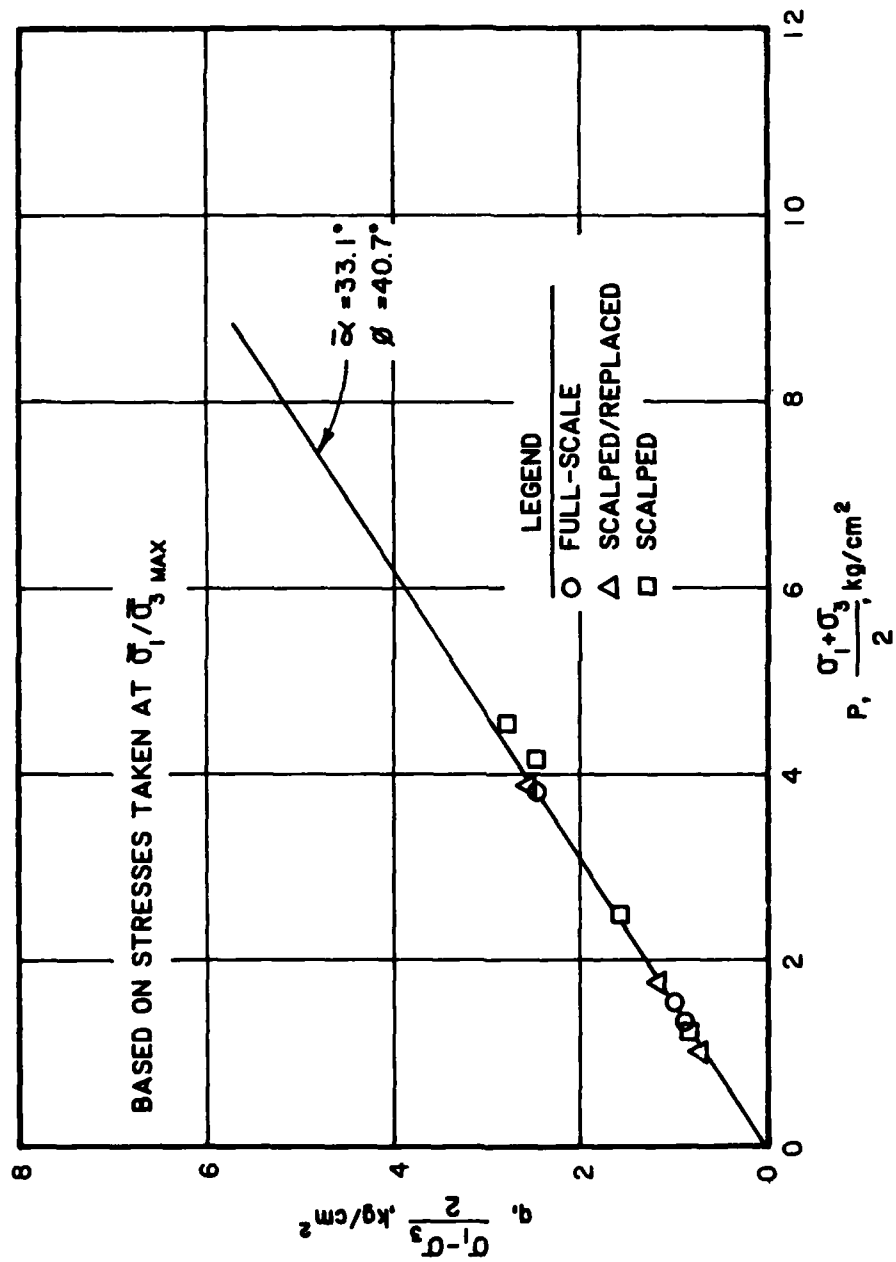


Figure 73. Maximum principal effective stress ratios in terms of $\bar{q}$ and $\bar{p}$, DeGray Dam material

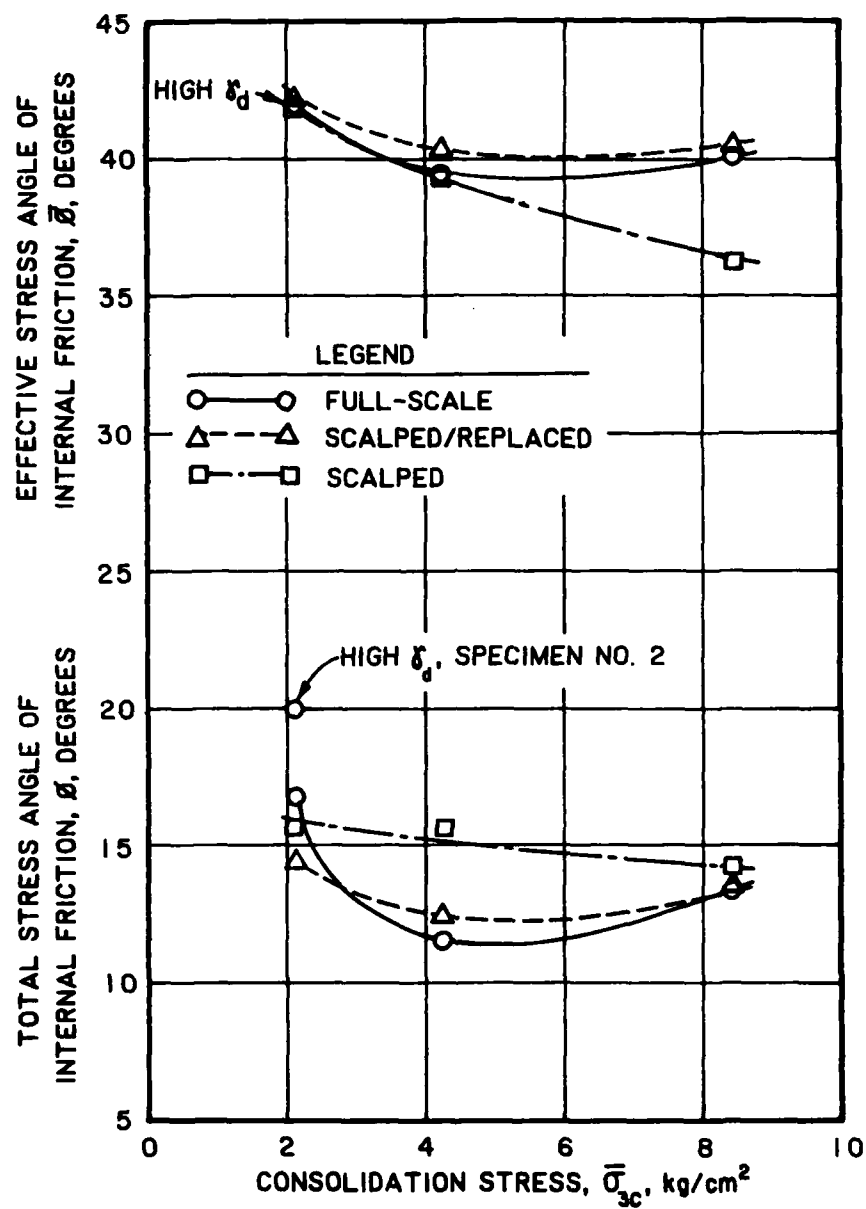


Figure 74. Total and effective angles of internal friction versus confining pressure, DeGray Dam material

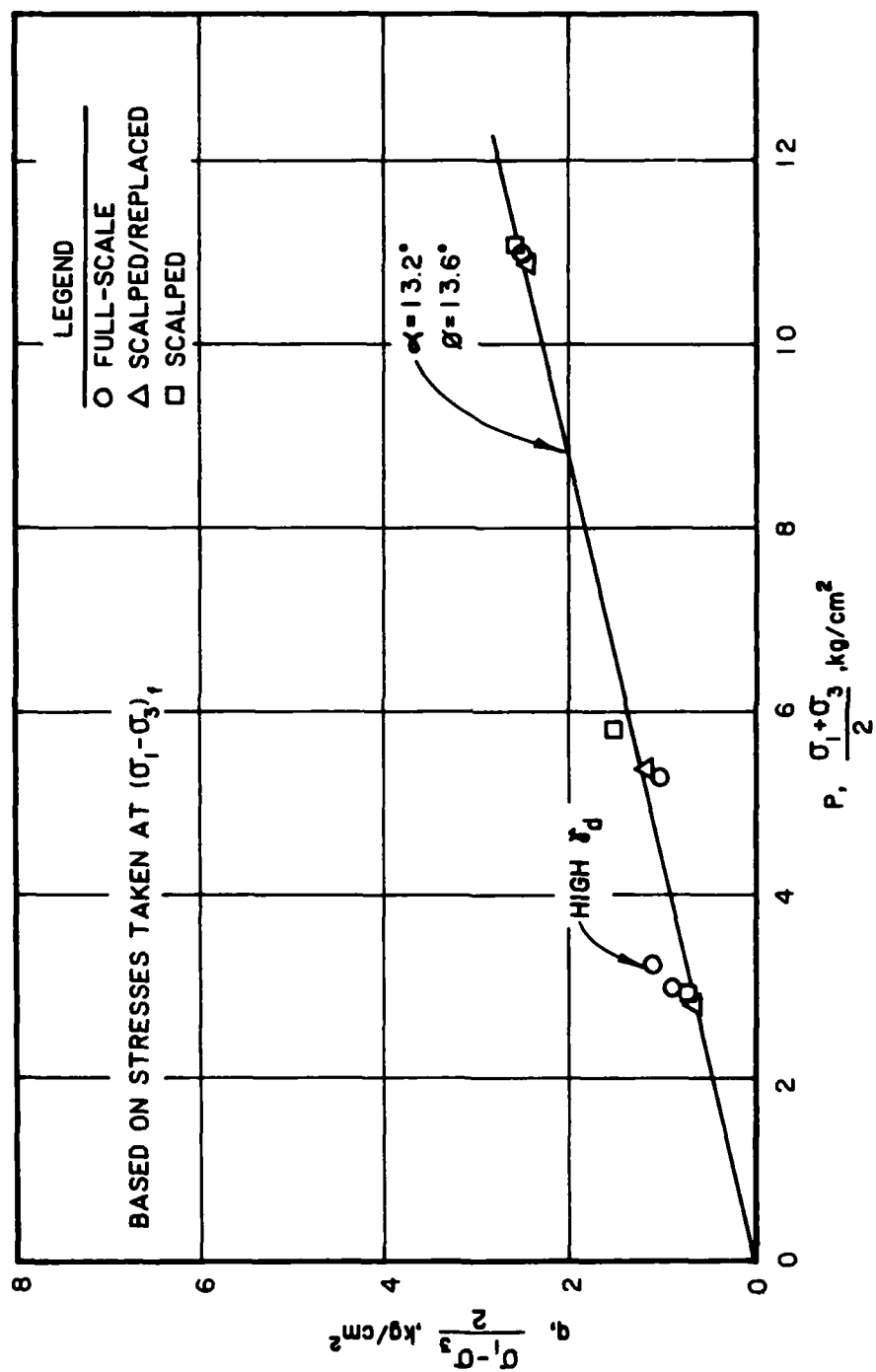


Figure 75. Deviator stresses at failure plotted in terms of p and q , DeGray Dam material

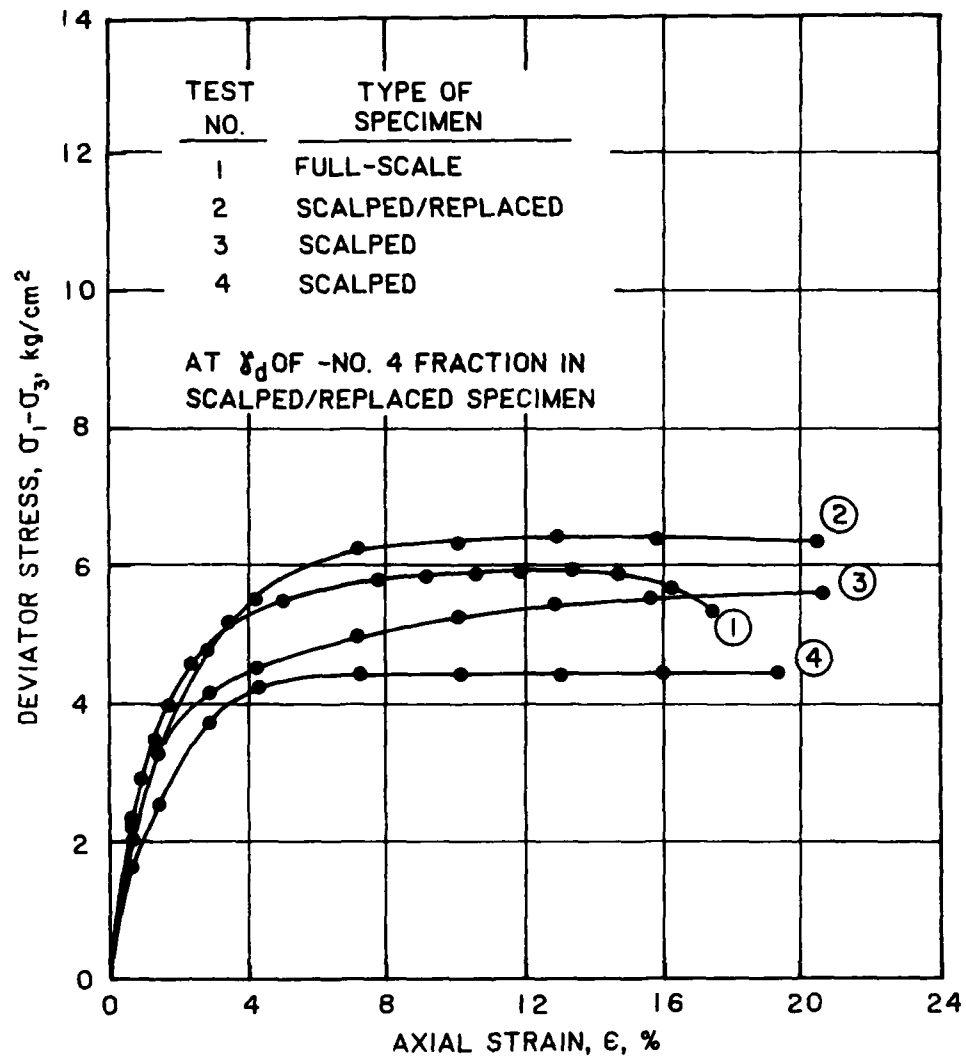


Figure 76. Stress-strain curves, Q tests, blended material, $\sigma_3 = 4.22 \text{ kg/cm}^2$, 20 percent gravel

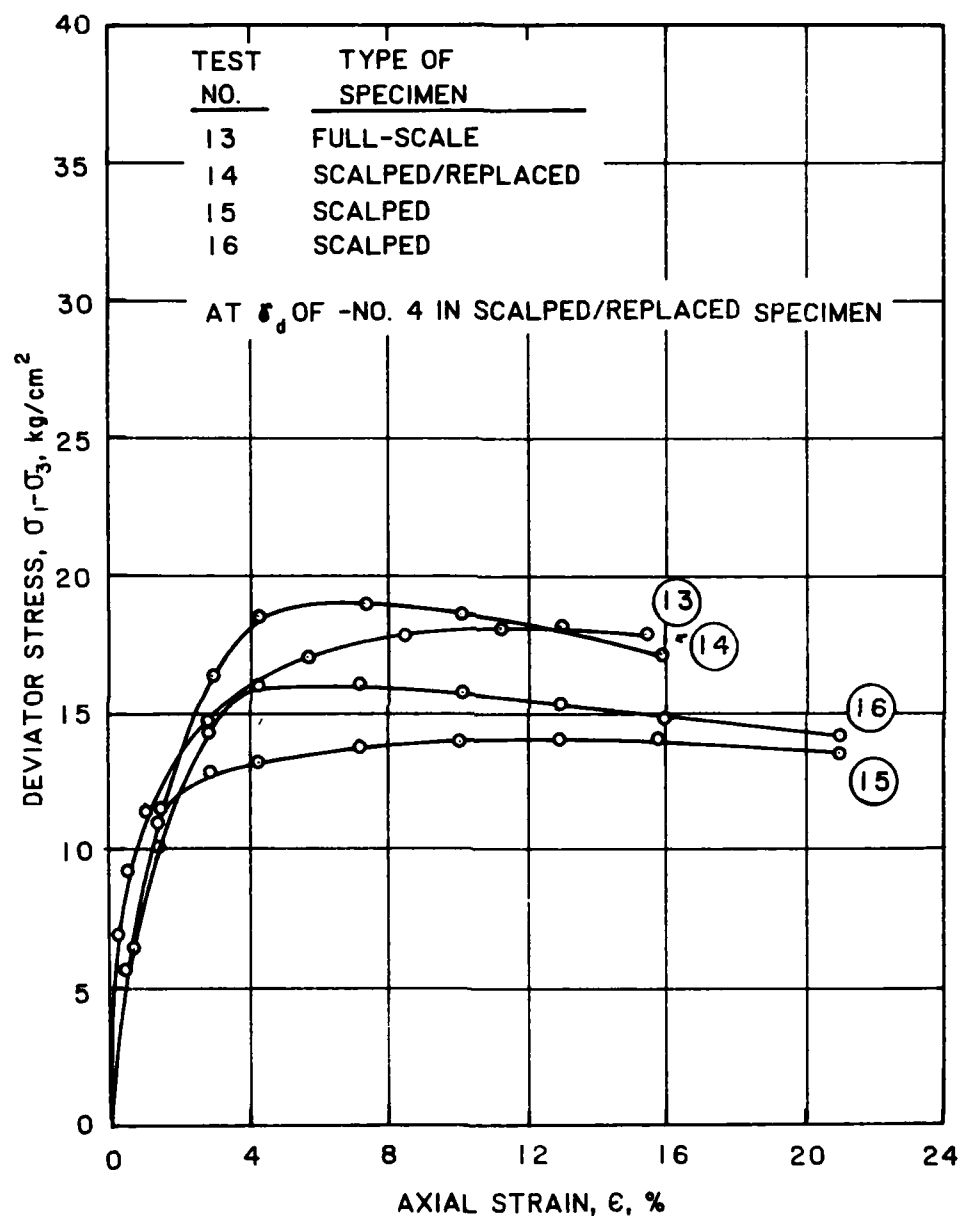


Figure 77. Stress-strain curves, Q tests, blended material,
 $\sigma_3 = 14.06 \text{ kg/cm}^2$, 20 percent gravel

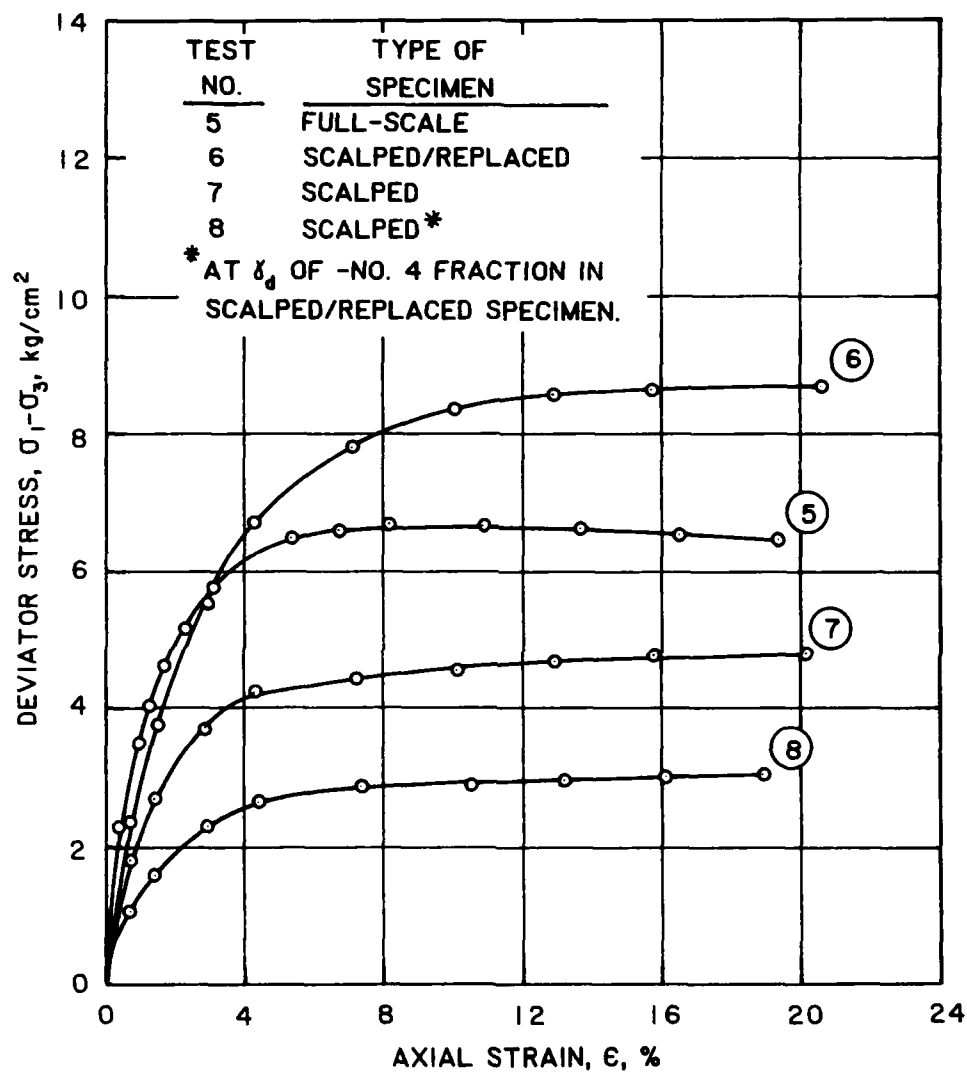


Figure 78. Stress-strain curves, Q tests, blended material,
 $\sigma_3 = 4.22 \text{ kg/cm}^2$, 40 percent gravel

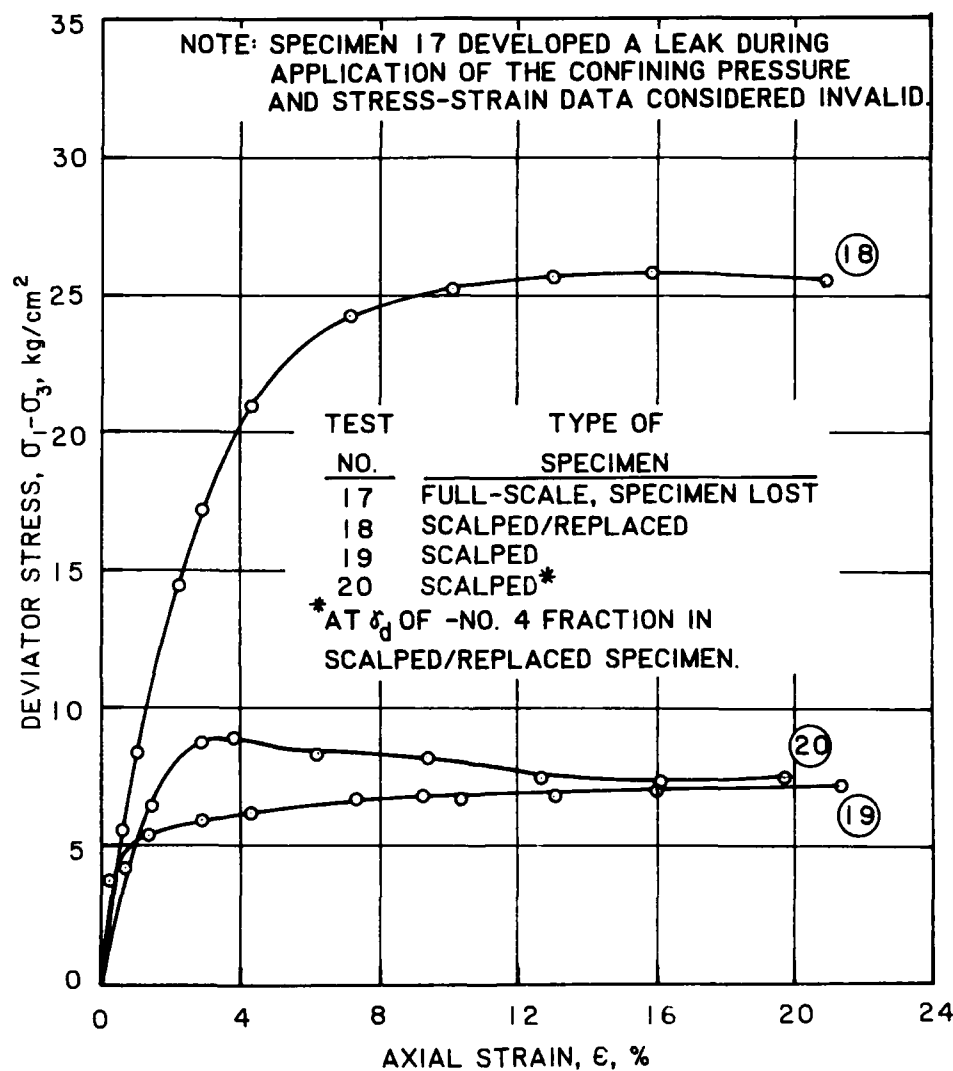


Figure 79. Stress-strain curves, Q tests, blended material,
 $\sigma_3 = 14.06 \text{ kg/cm}^2$, 40 percent gravel

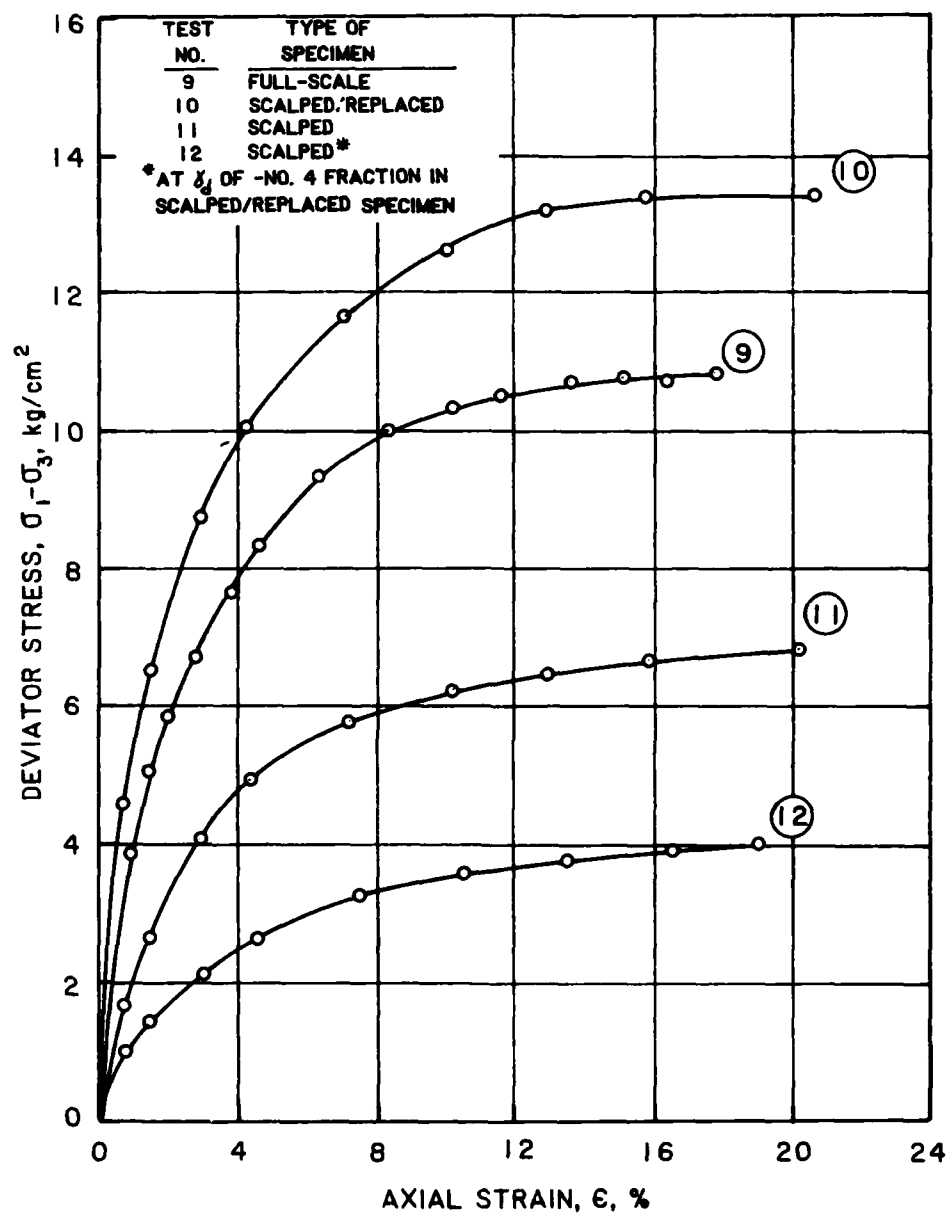


Figure 80. Stress-strain curves, Q tests, blended material,
 $\sigma_3 = 4.22 \text{ kg/cm}^2$, 60 percent gravel

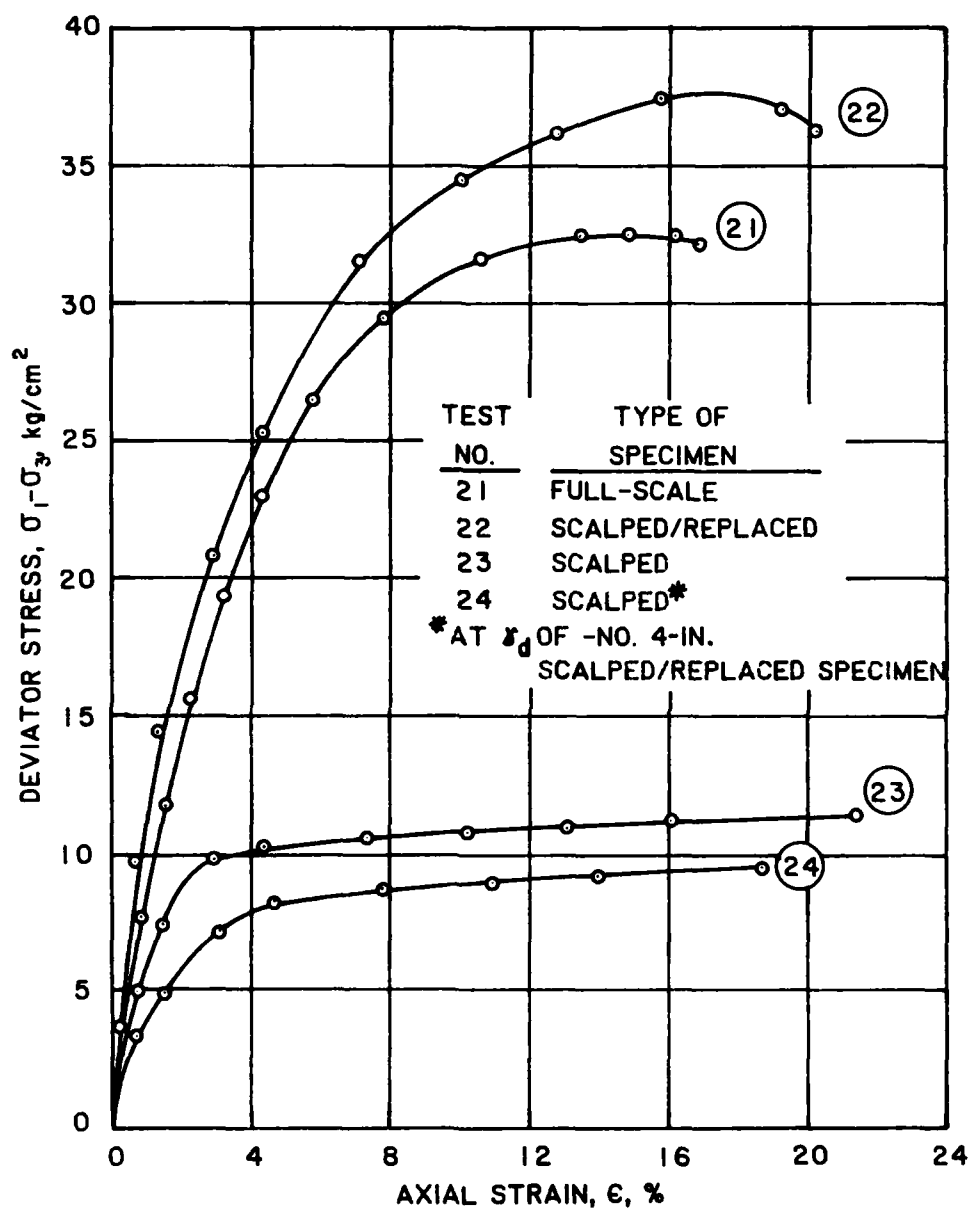


Figure 81. Stress-strain curves, Q tests, blended material,
 $\sigma_3 = 14.06 \text{ kg/cm}^2$, 60 percent gravel

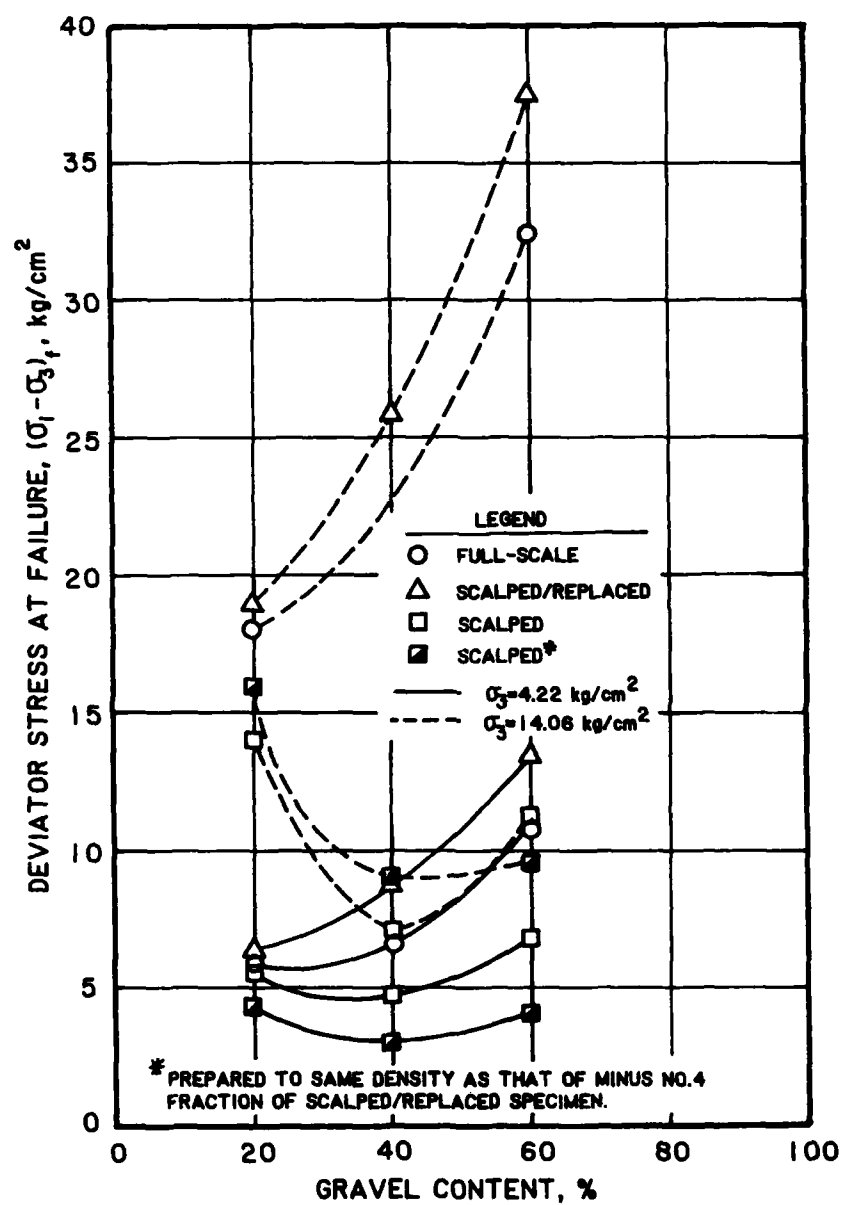


Figure 82. Deviator stresses at failure versus gravel content, Q tests, blended material

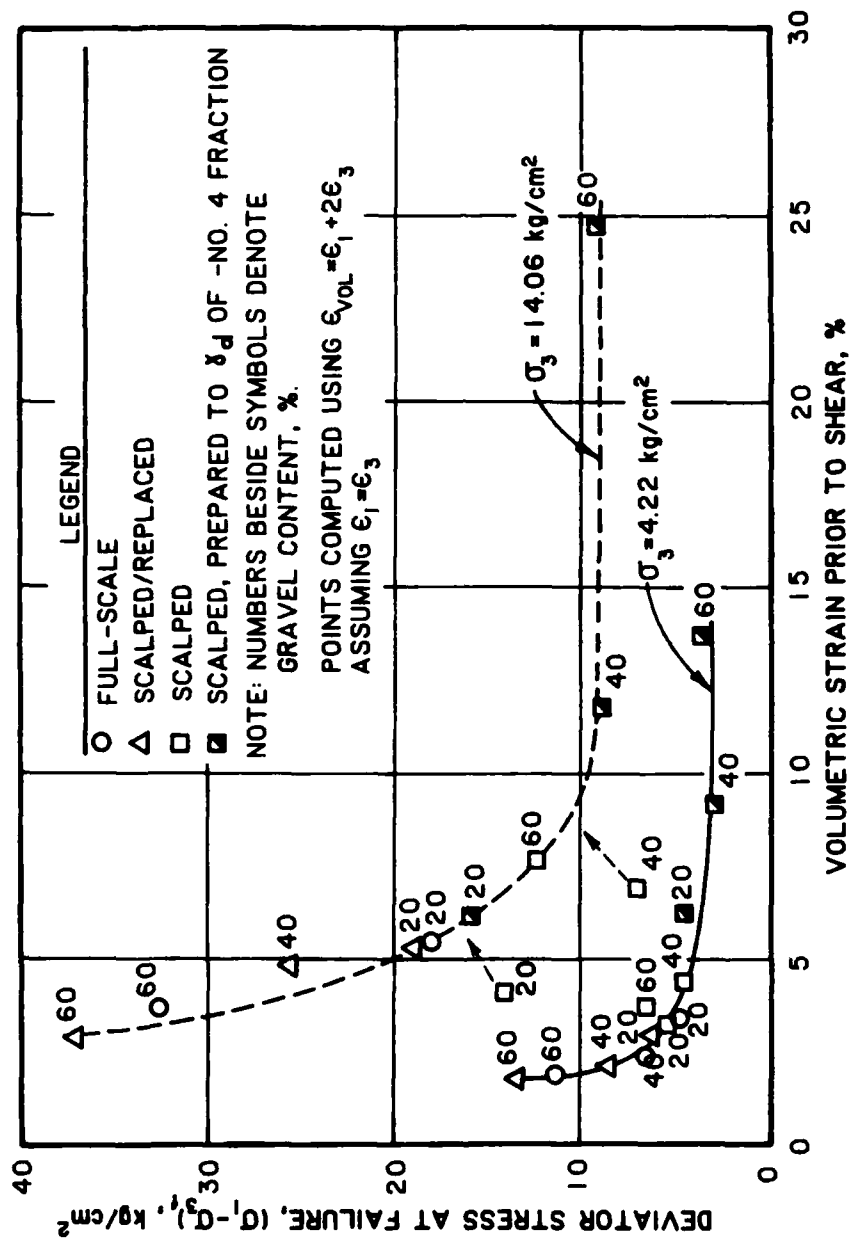


Figure 83. Deviator stresses at failure versus volumetric strain prior to shear, Q tests, blended material

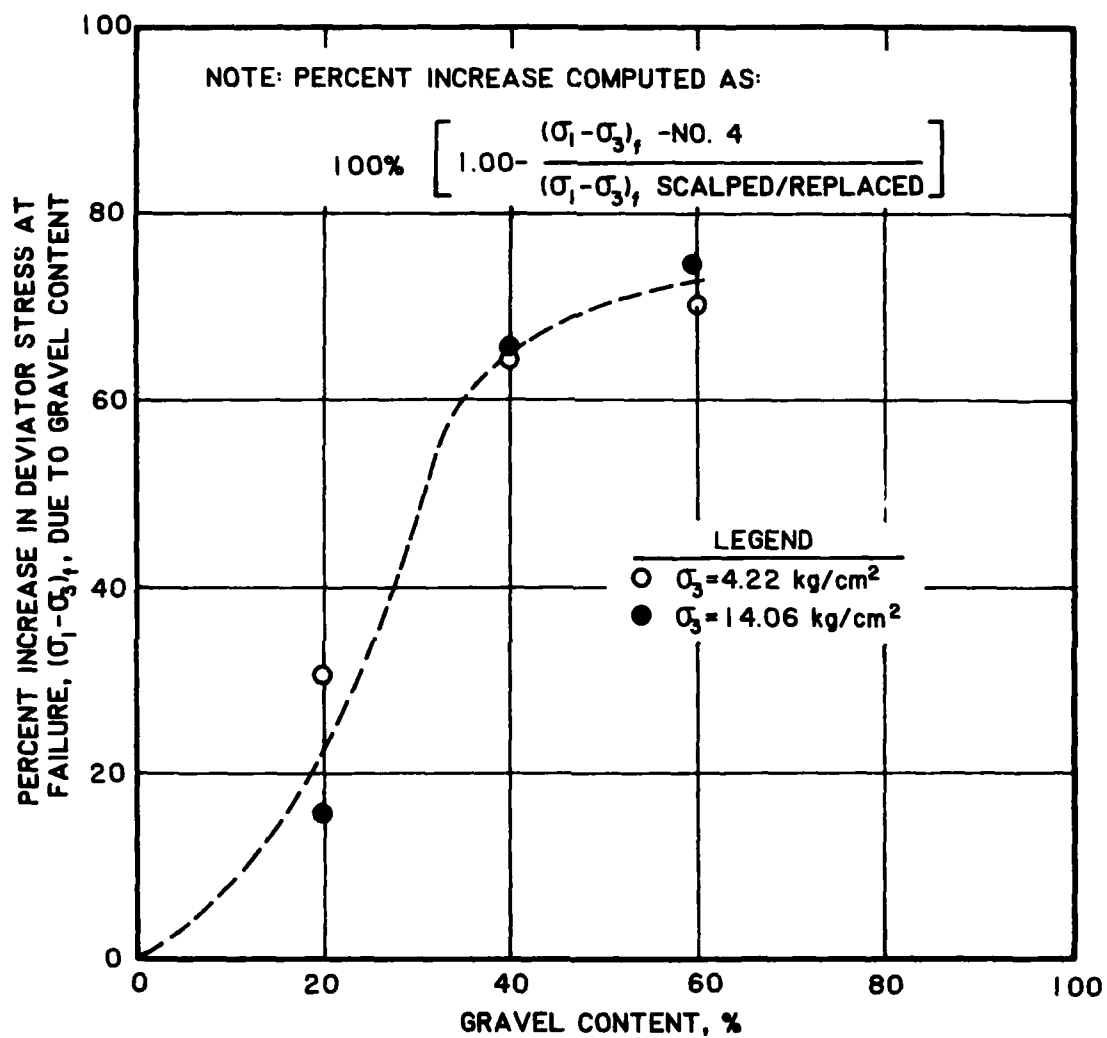


Figure 84. Proportion of shear strength contributed by gravel fraction, Q tests, blended material

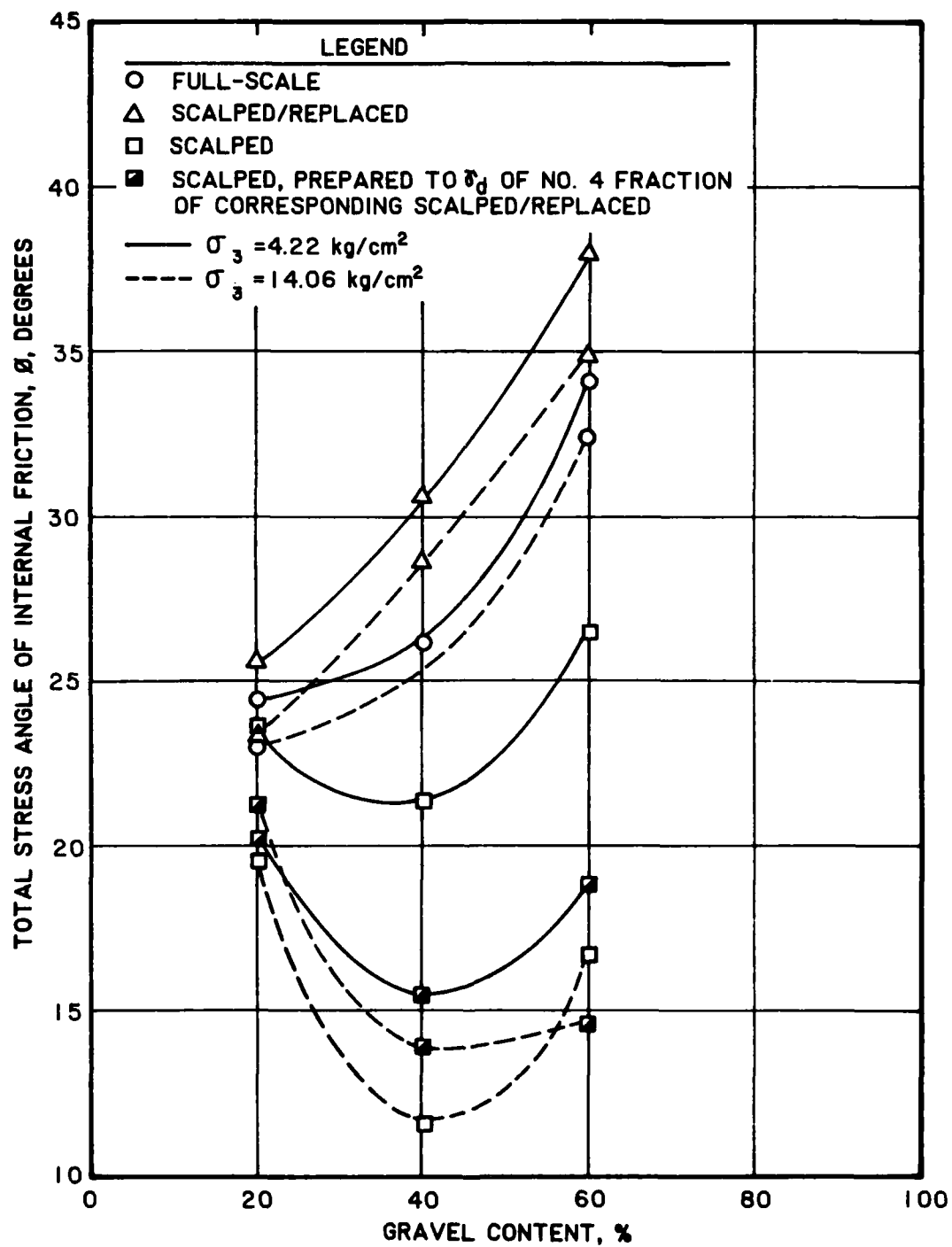


Figure 85. Total stress angle of internal friction versus gravel content, Q tests, blended material

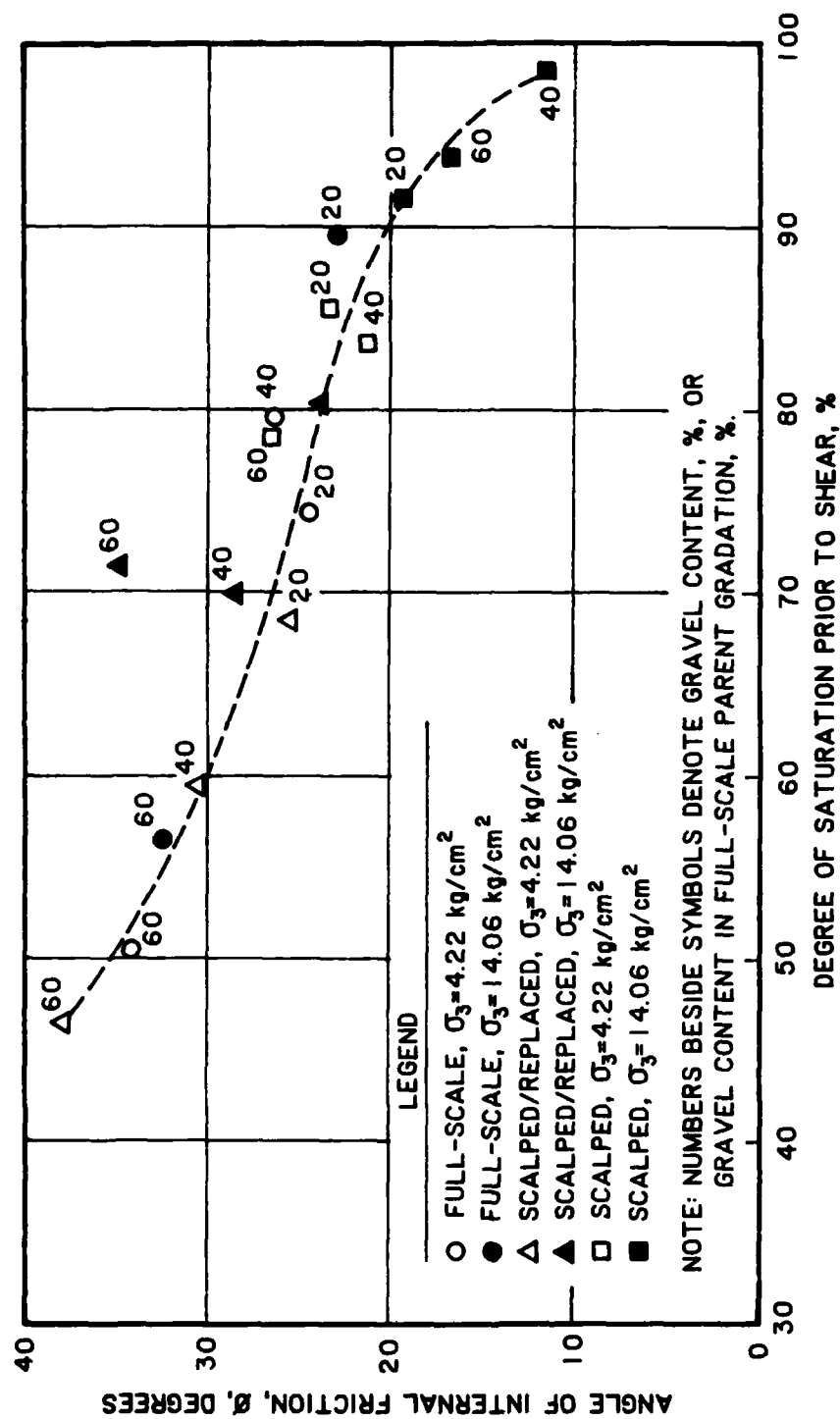


Figure 86. Total stress angle of internal friction versus degree of saturation prior to shear, Q tests, blended material

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