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EARTHQUAKE ENGINEERING RESEARCH CENTER

STRENGTH EVALUATION OF COARSE-GRAINED SOILS

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A report on research sponsored by
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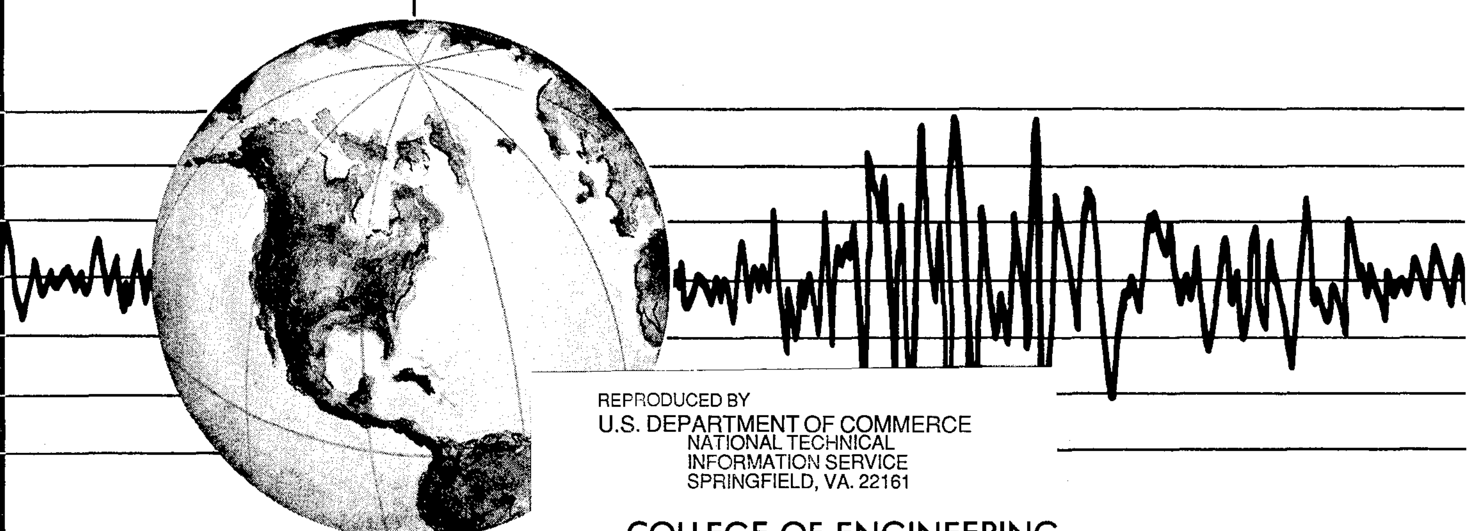

COLLEGE OF ENGINEERING

UNIVERSITY OF CALIFORNIA · Berkeley, California

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STRENGTH EVALUATION OF COARSE-GRAINED SOILS

By Farhat H. Siddiqi,[1] Raymond B. Seed,[2] Clarence K. Chan,[3]
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INTRODUCTION

Due to the need for large quantities of fill materials which are used in the construction of large earth and rock-fill dams, increasing numbers of dams are now being built utilizing very coarse-grained materials, including gravel-boulder mixtures. The large size of the particles often causes problems in the determination of the strength parameters for these materials. Many particle sizes for coarse-grained fills are too large to be tested with conventional laboratory equipment. In order to determine the relevant properties of the prototype (total) materials, laboratory specimens have to be prepared with a maximum particle size which is considerably smaller than the maximum field particle size.

The maximum grain size that can be tested in the laboratory is determined by the maximum size of the soil specimen that can be accommodated in a given triaxial testing apparatus. The term "oversize," therefore, is often used to refer to the constraint imposed on a strength testing program for a given material due to the availability of only a limited size of soil testing equipment.

There are at least three procedures commonly used to overcome the problem of "oversize" particles. These procedures include: (1) removing

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all oversized particles and preparing the test specimen from the remaining material; (2) replacing all oversized particles with an equal dry weight of relatively large particles which fall within the maximum allowable grain size limit; and (3) fabricating a new soil specimen with a grain size distribution parallel to that of the prototype material. Which of the methods should be chosen for a given prototype material depends on the state of the oversize particles. While in some prototype materials there may be particle to particle contact within the oversize particles, in others the quantity of the oversize grains may be so small that they have almost no contact with each other. In such a state, these oversized grains are described as "floating" in the finer soil matrix.

In addition to the selection of a suitable grain size distribution for the soil used in the laboratory tests, the proper choice of soil density is also necessary for the correct prediction of the strength properties of the prototype material. The laboratory specimen can be prepared either at the same dry density as the soil matrix of the prototype material, at a dry density which corresponds to the same relative density as that of the overall prototype material, or at some other condition. The optimal criteria for the selection of grain size distribution and density may also be different for the determination of the static and the cyclic strength parameters.

Improper modeling of the laboratory test specimens could lead to erroneous estimates of both static strength and cyclic loading resistance of the prototype material. Whereas errors on the safe side could lead to significant increases in cost of constructing large dams, errors on the unsafe side could lead to undesirable or potentially unsafe performance. It is, therefore, essential that estimates of the strength behavior of earth dam fill material be made as accurately as possible. The following investigation was conducted towards this objective.

BACKGROUND AND REVIEW OF LITERATURE

Cyclic Strength Evaluation

The last two decades have seen great progress towards the evaluation of soil response to seismic loading conditions. With the development of the cyclic triaxial test, the era of the quantitative analysis of seismically induced liquefaction or cyclic mobility began. Analytical procedures were developed for evaluating the seismic response of soil deposits and the stability of earth dams. These analytical procedures were used as a guide to understanding and explaining soil behavior during past earthquakes. From these early beginnings, knowledge of soil liquefaction potential and cyclic mobility expanded rapidly. Extensive reviews of many of these developments have been presented by Seed (1976, 1979a, 1979b), Yoshimi et al. (1977), Finn (1981) and the Committee on Earthquake Engineering of the NRC (1985). Most investigations related to soil behavior under cyclic loading conditions, however, have been performed on sandy soils. Test data for gravelly soils, on the other hand, are very limited. This is probably due to two reasons: (1) gravelly soils have generally performed much better than sands during past earthquakes, and (2) most soil laboratories are not equipped with the large-scale apparatus required for testing gravelly soils. Thus, the only available data for cyclic load tests on gravelly soils seems to be that presented by Lee and Fitton (1969), Wong et al. (1974), Banerjee et al. (1979) and Ishihara (1985). Lee and Fitton tested 2.8-inch diameter soil specimens containing gravel up to 3/4-inch in size in cyclic loading triaxial tests. They found that for a given confining pressure and relative density, the stress required to cause a pore pressure ratio of 100 percent in gravelly soils was almost twice that required to cause the same condition in fine sands under undrained cyclic loading conditions. Later investigations by Wong et al. (1974), however, showed that due to the small sample size of the test specimens (2.8-inch diameter) compared with the particle size of the soil investigated (up to 3/4 inch), and system compliance problems, their findings were, at the best, inconclusive.

Wong, Seed, and Chan (1974) reported that the cyclic stresses required to cause a given strain in gravelly soils were apparently somewhat higher than those needed to cause a similar strain in sandy soils. It was explained, however, that the difference could have been due entirely or in part to membrane penetration effects. The materials tested by Wong et al. (1974) were uniformly graded and, therefore, these tests also were affected to a significant degree by membrane compliance causing the cyclic stresses required to produce different strain amplitudes to be erroneous. The test results were considered to be somewhat more reliable than those presented by Lee and Fitton (1969) however because, by using a sample diameter of 12 inches with a maximum particle size of 1.5 inches, a ratio between the specimen diameter and the maximum grain size of approximately 8:1 (recommended for uniformly graded soils) was maintained.

Dense gravel specimens tested by Banerjee et al. (1979) exhibited many similarities in behavior to dense sand specimens under cyclic loading conditions, but the shape of the pore water pressure ratio versus the cycle ratio curve for the medium to very dense gravel was very different than that for sand. Unlike many sands, the rate of pore water pressure increase was very rapid during the first few load cycles for the Oroville gravel tested by Banerjee et al. (1979), followed by a very slow rate of increase thereafter. This rapid initial pore pressure generaiton followed by slow pore pressure generation at later cycles of loading may have resulted from the strongly dilatant behavior of this dense gravel as well as from membrane compliance effects. Banerjee et al. (1979) also found that:

1. The method of sample preparation had relatively little influence on the cyclic loading resistance of dense, well graded Oroville gravel compared to that for sand. At higher confining pressures, the influence was further reduced.
2. The dense Oroville gravel exhibited a significant increase in resistance to cyclic loading when kept under sustained pressure for a relatively short period of 10 weeks.
3. The effect of membrane penetration into the peripheral voids of the dense Oroville gravel on cyclic stress ratio was found to have a significant effect on the test data. It was concluded that the stress ratios required to cause a pore water pressure ratio of 100 percent should be uniformly reduced by approximately 10 percent to allow for the membrane penetration effects.

Ishihara (1985) reported tests on sand-gravel mixtures showing that the presence of more than 30% of gravel sizes may increase the liquefaction resistance of such mixtures. However the results were inconclusive because of variations in relative densities as well as gravel contents in the samples tested.

Thus, the limited data available indicate many similarities but some significant differences in the cyclic behavior of gravelly and sandy soils. In general, the test data indicate that under cyclic loading conditions, soil specimens containing gravels are somewhat more resistant to the development of high pore-water pressure than specimens containing sandy soils and considerably more resistant to the development of large axial strains. However, the experimentally observed differences in cyclic resistance of gravelly and sandy soils could be primarily due to membrane compliance effects. If due allowance is made for membrane compliance effects on the reported results of laboratory tests, it could be concluded that under truly undrained conditions the gravelly soils are no more resistant to undrained cyclic loading than sandy soils. Martin, Finn, and Seed (1978) demonstrated this conclusion by correcting tests conducted by Lee and Fitton (1969) on 2.8-inch diameter samples and those performed by Wong et al. (1974) on 12-inch-diameter specimens, to allow for membrane penetration effects. When the data were corrected in this way the results (shown in Fig. 1) suggest that the differences in cyclic loading resistance with grain size and specimen diameter observed in the laboratory test results could be mainly due to membrane compliance effects.

Most of the tests reported above (Lee and Fitton, 1969; Wong et al., 1974) were performed on uniformly graded soils. No suggestions were made by these investigators regarding the problems of handling the oversized particles in coarse grained soils or well-graded soils containing coarse particles.

Static Strength Evaluation

During the mid-1950's, the need for static stability analysis of rock-fill dams was recognized and methods to determine the strength and deformation characteristics of rock-fill materials were investigated. Zeller and Wullimann (1957) were among the first to propose a method for estimating the field properties of rock-fill material from laboratory

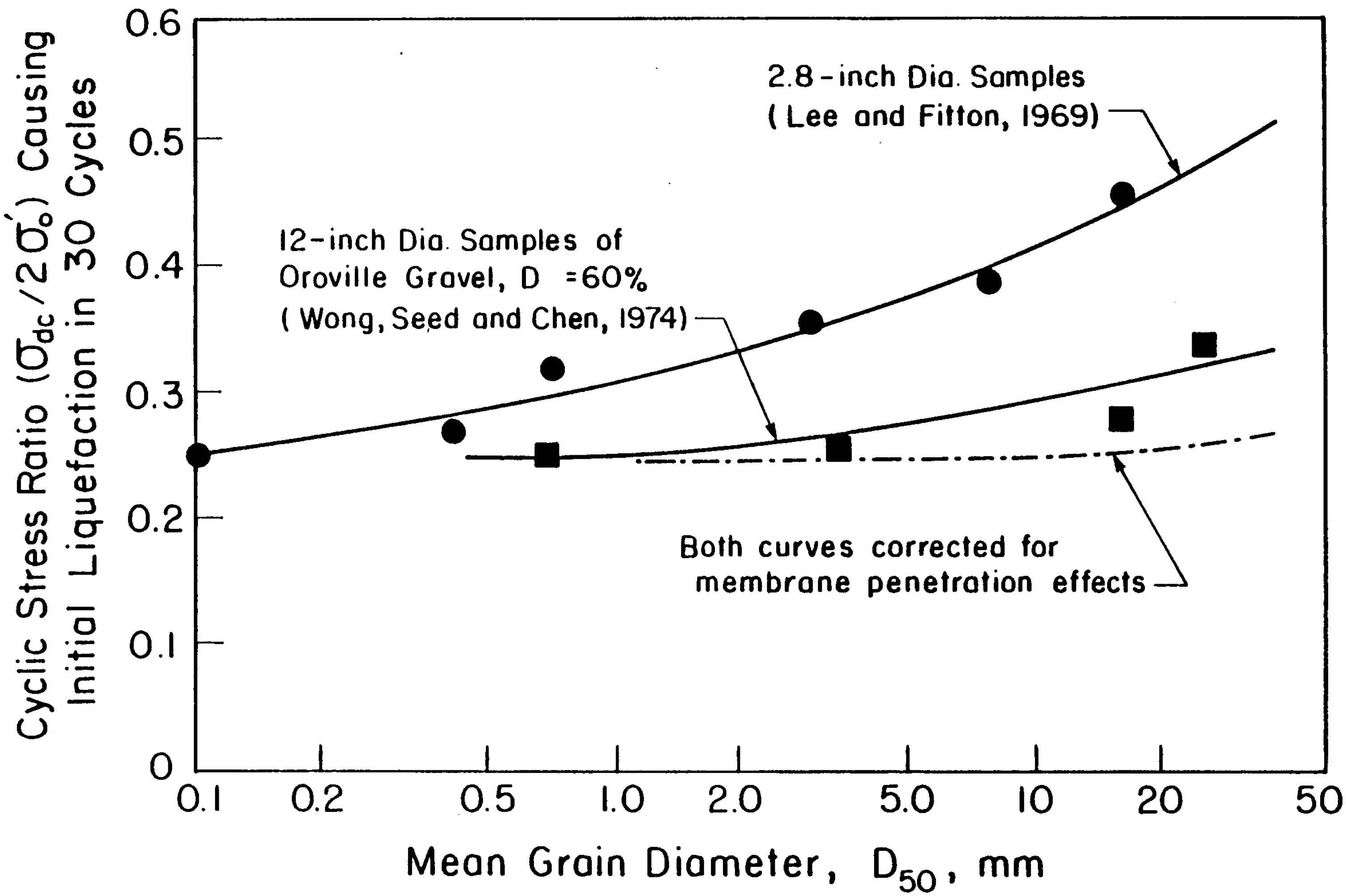

Fig. 1 CYCLIC TRIAXIAL TEST RESULTS CORRECTED FOR MEMBRANE PENETRATION EFFECTS
(After Martin et al., 1978)

tests. The coarse fractions from the rock-fill material to be used in the shell zone of Goscheonalp Dam in Switzerland were removed to obtain a series of materials for testing in the laboratory. Triaxial tests were then performed on each of these materials to establish a relationship between the strength and porosity for each material. Specimen diameters of 8 to 50 cm were used for these tests. By cross-plotting, a series of curves of strength versus maximum grain size for different porosities were obtained. Estimates of the shear strength of the field gradation were then made by extrapolation.

Another method for estimating the shear strength of coarse shell material from laboratory tests performed on small size specimens was proposed by Lowe (1964) during the construction of Shihmen Dam in Taiwan. Whereas the maximum particle size to be used in the shell of the dam was 12 inches, laboratory triaxial shear tests were conducted on specimens 6 inches in diameter and prepared from a modeled material having a grain-size distribution curve parallel to that of the prototype material (Fig. 2). It was assumed that, "...in models of coarse fractions where the only difference between prototype and model samples is the difference in the size of particles, the model samples should closely duplicate the behavior of the prototype in shear." There are some inherent problems with the concept of using parallel gradation. Since the surface roughness of the particles affects the angle of internal friction (Vallerga et al., 1957), the surface roughness of the particles in the prototype material should also be properly modeled. The crushing of particles is also known to have an influence on the observed angle of internal friction (Marsal, 1967). Therefore, the effect of the crushability of particles on the angle of internal friction should also be taken into account in the modeling

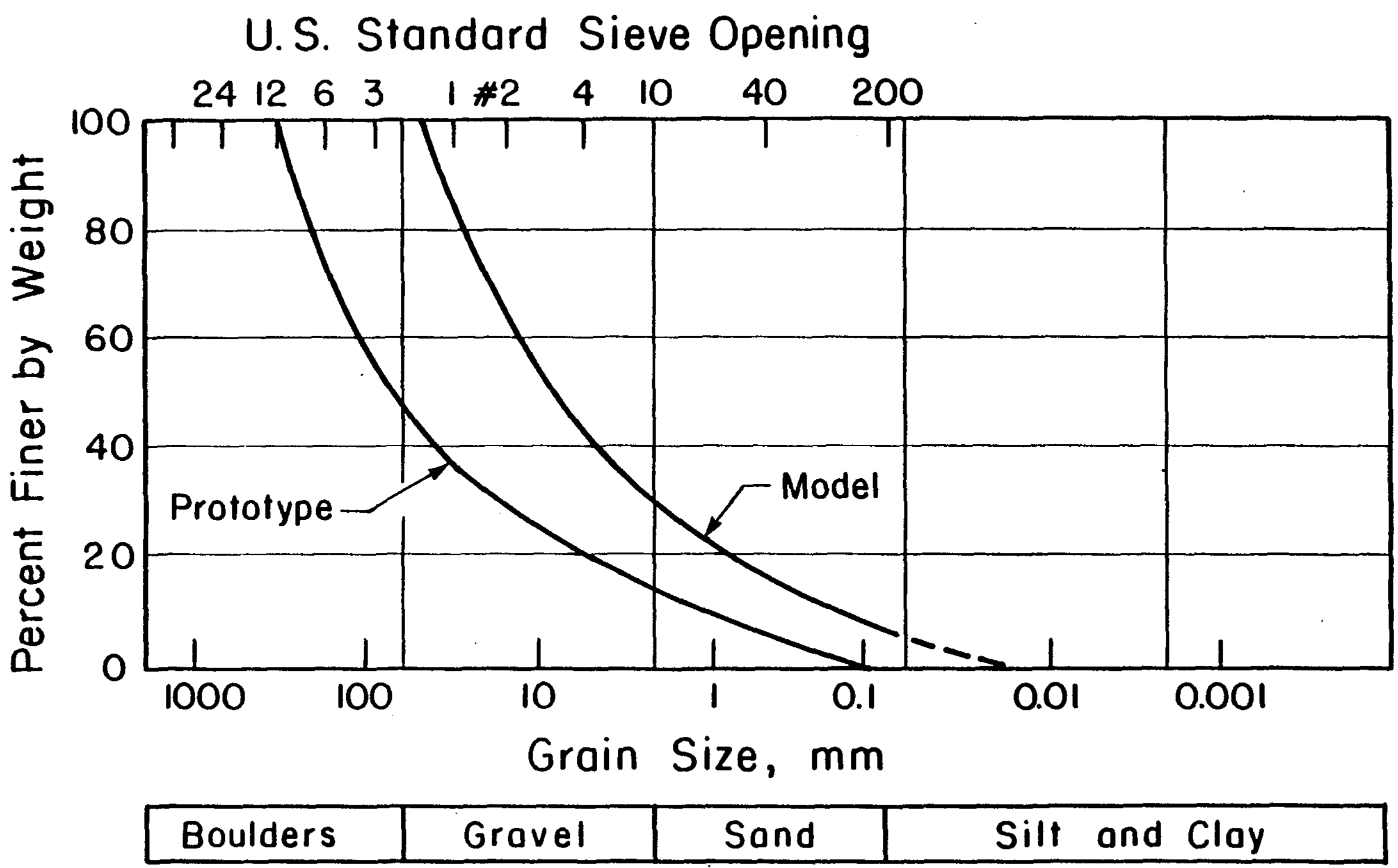

Fig. 2 MODELLING OF ROCKFILL MATERIALS (After Lowe, 1964)

process. Finally in some cases, the adoption of a parallel gradation introduces an undesirably high fraction of fines.

Marachi et al. (1969) investigated the possibility of predicting the angle of internal friction of actual rock-fill material from the results of tests performed on modeled small sized specimens. They used the modeling criteria proposed by Lowe (1964) in which laboratory specimens are prepared with a grain size distribution parallel to that of the field material. Tests were performed on a variety of types of rockfill. For comparison purposes, these authors tested large-scale rock-fill specimens (36-inch diameter), intermediate size specimens (12-inch diameter), and small conventional sized specimens (2.8-inch diameter), all with parallel particle size distributions and the same material. The grain size curves of the Oroville Dam material tested are shown in Fig. 3. The results of drained triaxial compression tests indicated that the angle of internal friction of the samples containing large sized particles (up to 6 inches in maximum dimension) in the 36-inch-diameter specimens was 3 to 4 degrees lower than that of the small-scale samples of 2.8-inch diameter, regardless of the confining pressure or material type. It was also noted that the volume changes during drained triaxial compression tests were least compressive for the small (2.8-inch diameter) specimens for all the materials tested.

More recently, Donaghe and Torrey (1979) of the U.S. Army Waterways Experiment Station in Vicksburg, Mississippi studied the effects of scalping and replacement of oversized particles on the undrained strength parameters of earth-rock mixtures. Based on the results of consolidated undrained triaxial compression tests performed on test specimens compacted

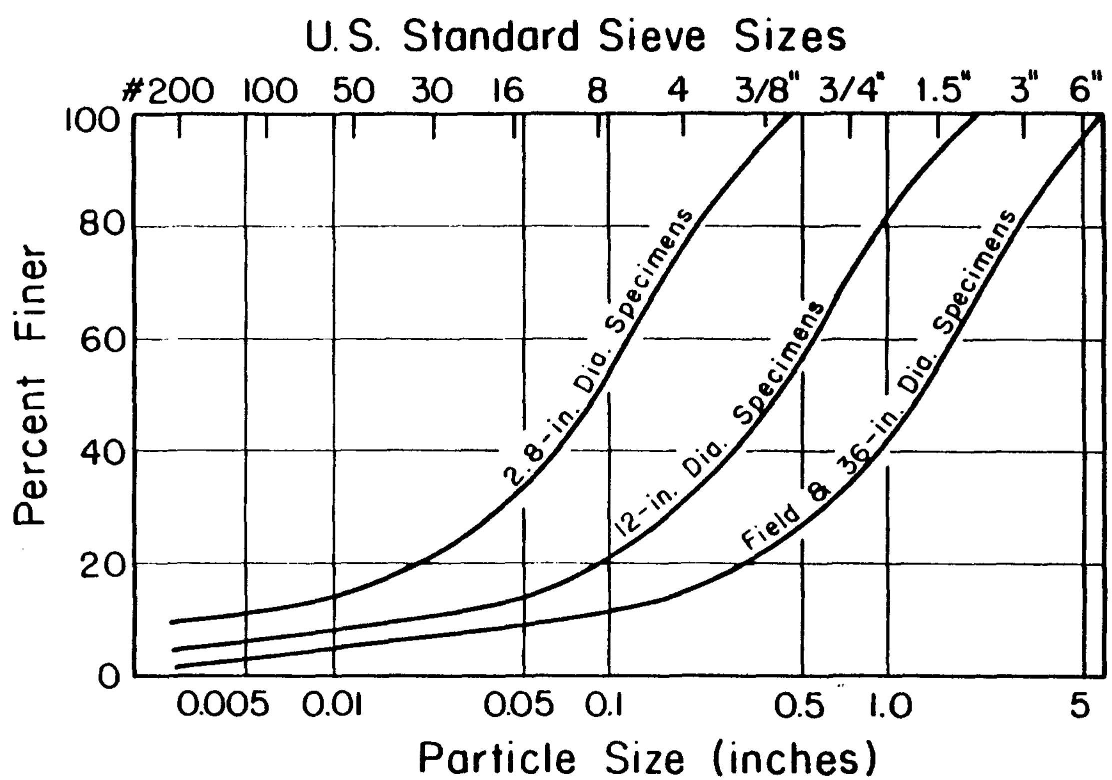

Fig. 3 GRAIN SIZE DISTRIBUTION CURVES FOR MODELLED OROVILLE DAM MATERIAL (After Marachi et al., 1969)

to a degree of compaction of 95 percent (based on the Standard AASHTO Compaction Test), Donaghe and Torrey concluded that:

1. The earth-rock mixtures developed large pore-water pressures during undrained loading. Consequently, angles of internal friction based on total stresses for these materials may be as low as 11 degrees (for test specimens compacted to 95 percent of the standard effort maximum density).
2. The results of the tests performed on minus U.S. Standard No. 4 sieve fractions of the earth-rock mixtures were in a reasonably good agreement with the results of full-scale tests performed on the total material in terms of angles of internal friction based on total stresses.
3. The scalping and replacement procedures provided conservative strength parameters for earth-rock mixtures based on effective stresses.
4. The test results showed no trend indicating that the angle of internal friction based on either total or effective stresses decreases with increasing confining pressures over the test confining pressure range of 60 to 200 psi.
5. The undrained strengths in the large-scale tests performed on the earth-rock mixtures (total material) were considerably larger than the undrained strengths of the minus U.S. Standard No. 4 sieve fractions, particularly at low strains, though this may have been due largely to membrane compliance effects.

The studies performed by Marachi et al. (1969) and Donaghe and Torrey (1979) were a significant step towards understanding the drained and undrained strength behavior of earth-rock mixtures. No conclusions, however, were derived regarding the selection of the grain size distribution and density of the modeled specimens with small sized particles which would predict the drained and undrained strength behavior of the earth-rock mixtures with large sized particles. The same is true for studies performed at the Corps of Engineers, South Pacific Division in San Francisco, California. Based on the results of tests conducted on 6- and 12-inch-diameter specimens with maximum particle sizes of 1.5 and 3 inches (Leslie, 1963), the findings with respect to the effects of grain size distribution and the effect of removing oversize particles were largely inconclusive.

COMPOSITION OF COARSE-GRAINED SOILS WITH FLOATING LARGE PARTICLES

In many cases the composition of a coarse-grained soil is such that the over-size particles are in fact floating in a matrix of finer particles as shown in Fig. 4, and under these conditions it is to be expected that the behavior of the soil as a whole will be controlled by the properties of the matrix. It is never-the-less important to understand in some detail the nature of the matrix in order to evaluate correctly the properties of the overall soil. A schematic representation of the arrangement of a series of large particles in a matrix of smaller particles is shown in Fig. 5 and it should be noted that the relatively large flat surfaces of the large particles leads to the formation of a series of relatively large void spaces where the smaller particles are in contact with the very large particles. These spaces results from the fact that the contact surface with the large particles does not provide the same smaller curvature as exists where smaller particles are in contact with other smaller particles.

Thus each large particle can be considered to introduce into the soil a layer of extra large voids or increased volume of void space that is

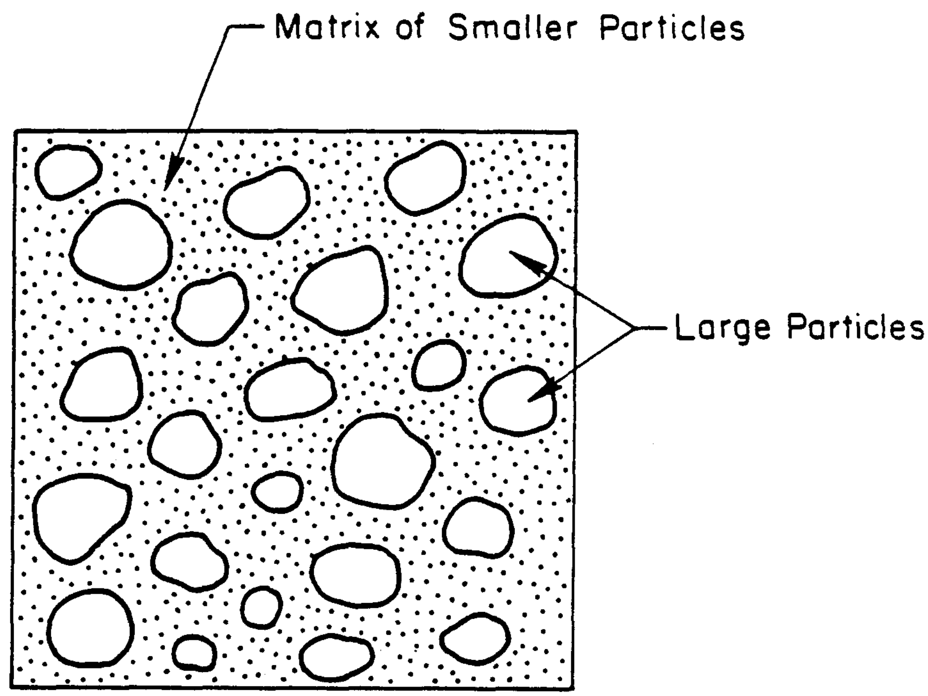

Fig. 4 SCHEMATIC COMPOSITION OF COARSE-GRAINED SOIL WITH LARGER PARTICLES FLOATING IN A MATRIX OF FINER-GRAINED SOIL

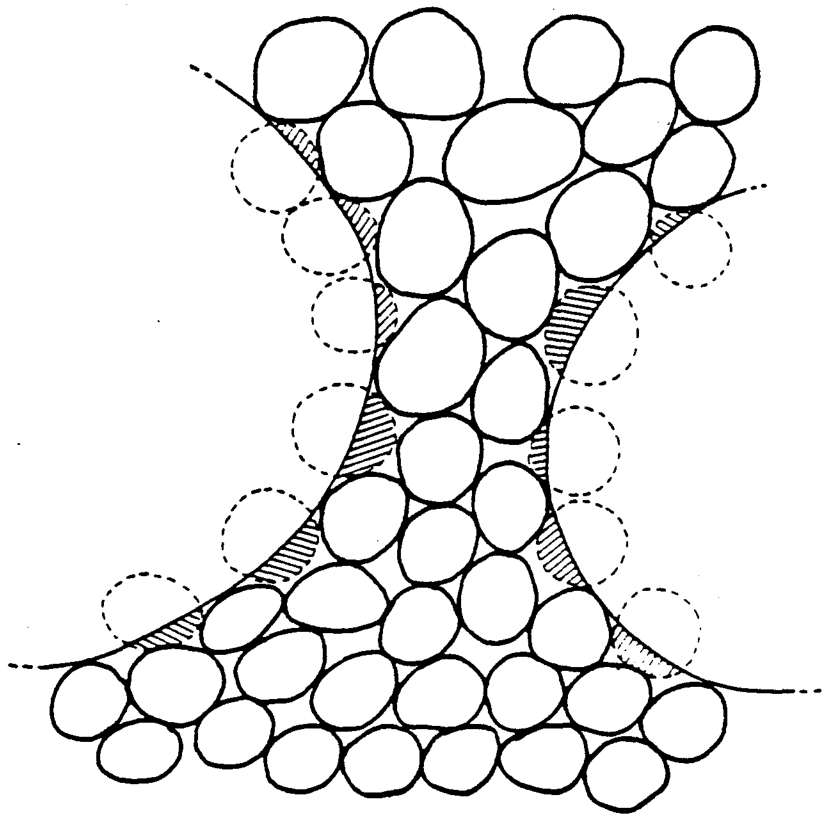

Fig. 5 SCHEMATIC ILLUSTRATION OF THE LARGER VOID SPACES AT THE CONTACT INTERFACES BETWEEN LARGE PARTICLES AND FINER-GRAINED PARTICLES

present only because of the presence of the interfaces between small and large particles. If the large particles were removed from the soil, this associated void space would also be removed automatically and it should therefore be considered separately from the remainder of the void space. In the following discussion the void space associated with the surface of the large particles will be designated V_{valp} (volume of voids associated with large particles). It should also be noted that for the conditions shown in Fig. 5, the contact surface between the large particles and the smaller particles of the matrix has certain special characteristics, in that relative shear deformation is possible without any significant volume change; in effect the smaller particles move freely over a large relatively flat surface and there is little or no tendency for volume change during such movement. Thus the voids associated with the large particles make no contribution to either volumetric compression, which would affect pore pressure build-up during cyclic loading, or to dilation, which would affect the effective strength of the soil during static shear.

It is not unreasonable to hypothesize therefore that:

1. The voids associated with the surface of the floating large particles, V_{valp}, make no contribution to volume change under any type of loading and thus they do not affect the pore-pressures which would be generated by undrained cyclic loading. These pore pressures will thus be due only to the properties of the main body of the matrix in the zone removed from the surfaces of the large particles.
2. In a soil with a dilatant matrix under static loading, the fact that some surfaces exist within the soil mass where shear deformations can take place without dilation will provide a

series of zones of weaknesses and it is likely that failure surfaces will be attracted to these nondilatant zones of contact between larger particles and the soil matrix.

In effect, then, the void ratio of the matrix can be considered to involve two zones--a near field zone close to the large particles where the void space will involve the presence of V_{valp}, and a far-field zone where the presence of the void spaces associated with the large particles have no effect. Under cyclic loading conditions the pore pressure generation characteristics of the entire soil mass are likely to be dominated by the condition of the far-field matrix alone, but under static loading conditions the contact surfaces between large and smaller particles will play a significant role in determining the shear resistance of the entire soil.

These considerations have important implications in determining the behavior of soils containing floating large particles under both static and cyclic loading conditions. In order to express these effects quantitatively it is convenient to consider the entire soil mass illustrated in Fig. 4 to consist of four components:

1. The volume of solids represented by the finer particles comprising the matrix, designed V_{sm}.
2. The volume of solids represented by the large particles, designated V_{slp}.
3. The volume of voids between the particles forming the main portion of the matrix, V_{vm}.

and 4. The volume of extra void space associated with the relatively flat contact surface between the larger particles and the finer particles constituting the matrix, V_{valp}.

Thus the composition can be depicted schematically as shown in Fig. 6(a). If the weight and volume of the large particles are subtracted from this composition, the remaining volume of soil (which comprises the matrix) will consist of V_{sm}, V_{vm} and V_{valp}. The average void ratio of the matrix in the soil sample when the large particles are removed in this way, which might be termed theoretical scalping, is

$$e_{ma} = \frac{V_{vm} + V_{valp}}{V_{sm}}$$

If, for the same volume of solids in the matrix, the same particles were placed in their densest and loosest conditions, the relative volumes of solids and voids would be as shown in Figs. 6(b) and 6(c) and the relative density of the matrix in the soil sample would be expressed by the ratio

$$D_{rm} = \frac{e_{max} - e_{ma}}{e_{max} - e_{min}}$$

$$D_{rm} = \frac{(V_{vm})_{max} - (V_{vm} + V_{valp})}{(V_{vm})_{max} - (V_{vm})_{min}}$$

$$= \frac{C}{B} \quad \text{(in Fig. 6).}$$

In fact, however, if the large particles are removed, then the volume of voids associated with the large particles, V_{vlp}, is also removed and the volume of voids in the main part of the matrix becomes V_{vm}. Thus the relative density of the main part of the matrix, if the extra void space associated with the presence of the large particles is not included, becomes:

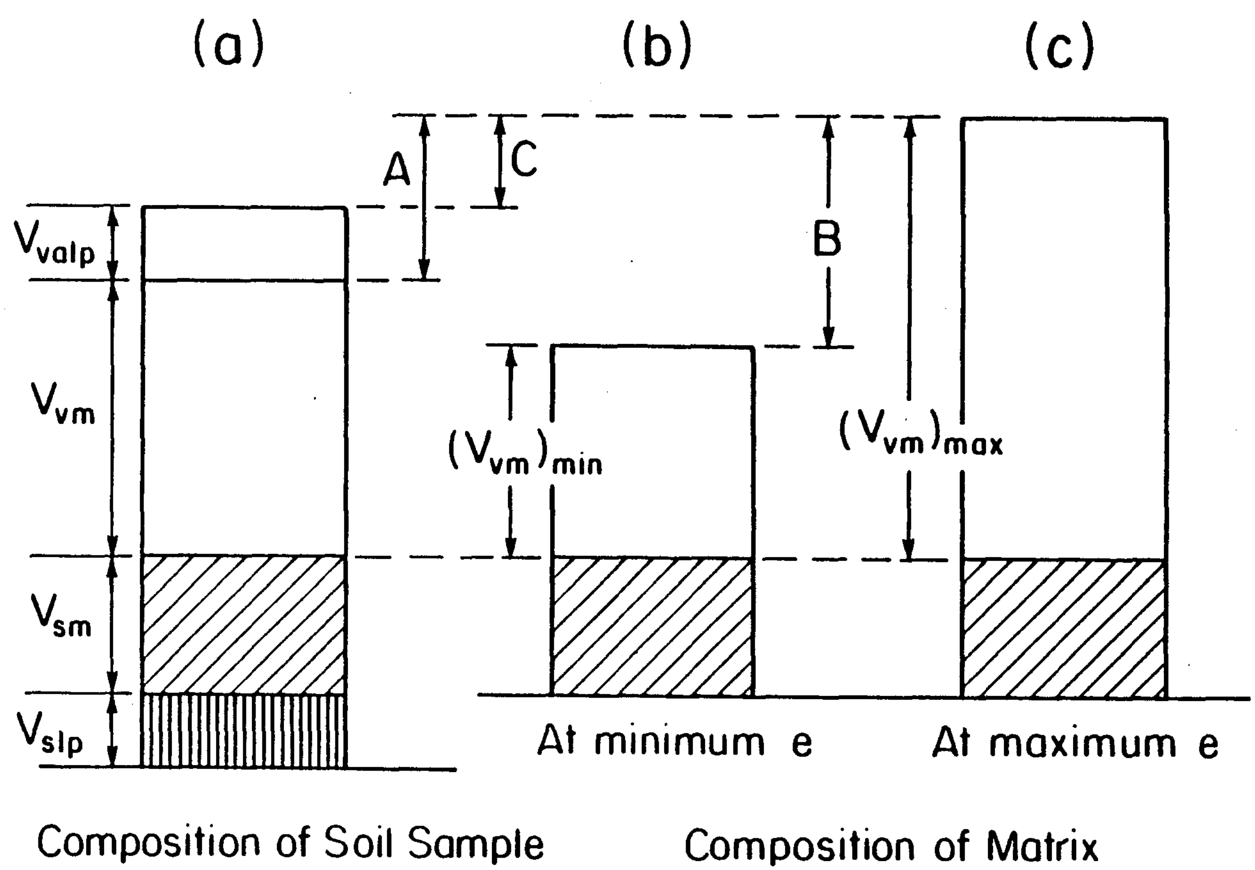

Fig. 6 SCHEMATIC REPRESENTATION OF COMPONENTS OF COARSE-GRAINED SOIL

$$D_{rm} = \frac{(V_{vm})_{max} - V_{vm}}{(V_{vm})_{max} - (V_{vm})_{mm}}$$

$$= \frac{A}{B} \quad \text{(in Fig. 6).}$$

This ratio describes the relative density of the main portion of the matrix, i.e., the far-field matrix. For some soils the difference between the ratios C/B and A/B may be very large. Both terms describe a property of the matrix and it is necessary to decide which one, if either, defines a relative density which controls any given behavioral property of the matrix.

It is interesting to note that if maximum and minimum density tests were performed on the total soil sample, the large particles would be present in the soil and the volumes of voids determined in such tests for the matrix alone would be increased by an amount closely equal to V_{valp}, as depicted in Fig. 7. In this case the relative density of the whole soil sample, D_{rs}, would be

$$D_{rs} = \frac{[(V_{vm})_{max} + V_{valp}] - [V_{vm} + V_{valp}]}{[(V_{vm})_{max} + V_{valp}] - [(V_{vm})_{min} + V_{valp}]}$$

$$= \frac{A'}{B'}$$

where A' and B' are shown in Fig. 7. It is interesting to note that the ratio:

$$\frac{A'}{B'} = \frac{A}{B}$$

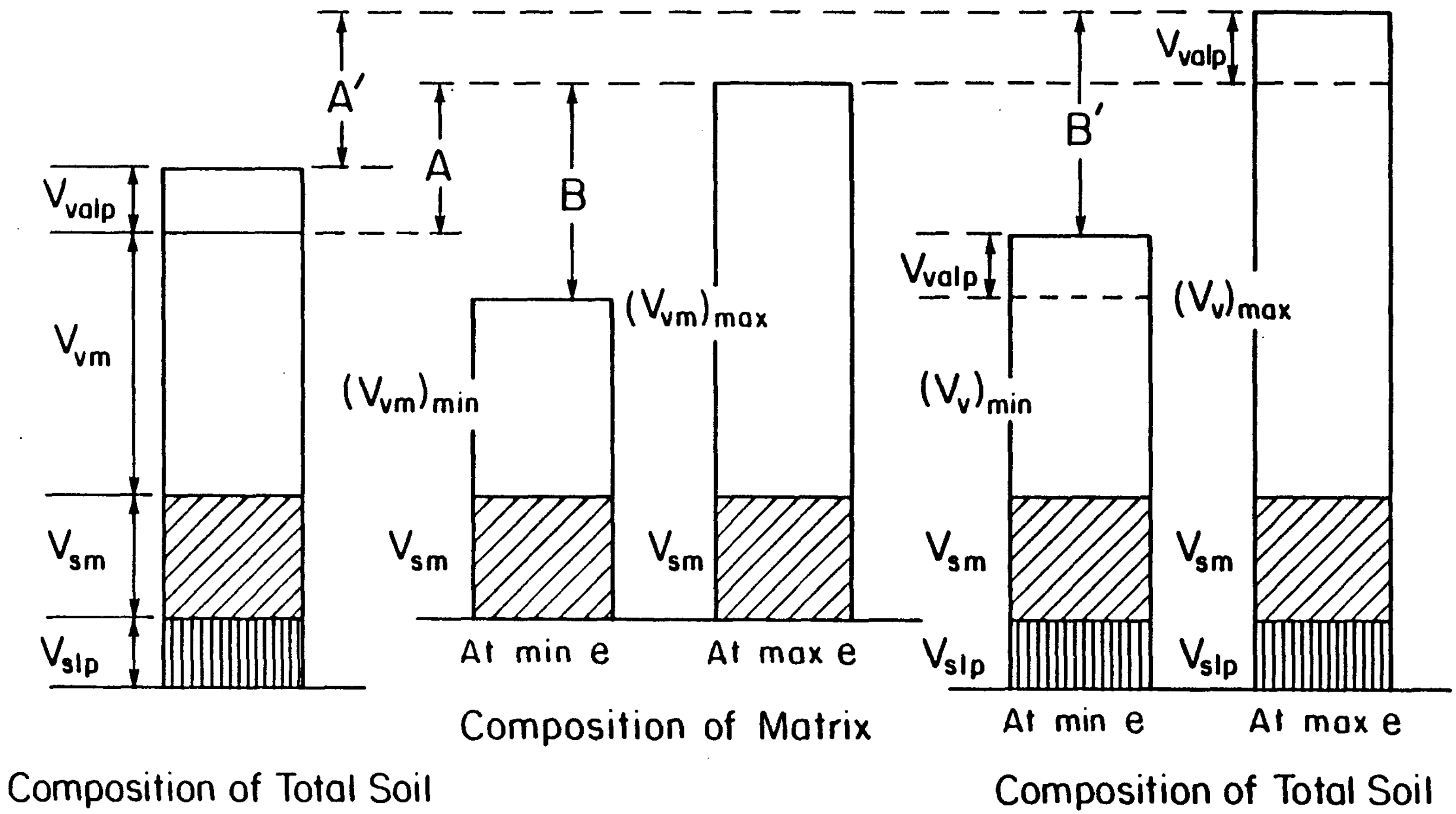

Fig. 7 SCHEMATIC REPRESENTATION OF COMPONENTS OF COARSE-GRAINED SOIL

and thus the relative density of the original soil sample is numerically equal to the relative density of the matrix if the voids associated with the large particles are excluded from the matrix composition, i.e. the far-field matrix.

It is also interesting to note that if tests are made to determine the composition of the sample in its natural state and at its maximum and minimum void ratios, and then the weight and volume of large particles is theoretically removed from each of the compositions so determined to establish the average composition of the matrix for the soil in its natural state, maximum density, and minimum density, the relative density of the matrix determined in this way would also be equal to the ratio of A'/B' or A/B; thus it would be equal to the relative density of the far-field matrix and not the average relative density of the matrix.

Thus the relative density of the matrix, excluding the effects of the voids associated with the presence of the large particles, V_{valp}, can be determined in different ways and found to be equal to the relative density of the original sample of soil.

Investigation of Matrix Properties for a Soil

The significance of the results discussed above depends of course, on the magnitude of the term V_{valp} in the various formulas presented. Clearly the value of this term will vary significantly from one soil to another, but the possible effects and significance of this term can be illustrated by its evaluation for a problem involving the composition of a deposit of streambed alluvium which had a maximum particle size of the order of 50 mm (2 inches). As much as 40% by weight was coarser than 10 mm (3/8 inch), the largest particle size which could reasonably be included in the 61 mm

(2.4 inch) diameter laboratory test specimens--the largest samples which could be tested with the testing equipment available for the project. However inspection of test trenches that had been dug in the field showed that the plus 3/8" size particles appeared to be dispersed or floating in a matrix of the minus 3/8" material. This observation was also confirmed by analytical studies of the grain-size distribution of the soil. Thus the composite soil in the field had the schematic structure shown in Fig. 4.

For such a sample of composite soil, containing large particles of soil floating in a matrix of finer grained soil, it is reasonable to expect that failure will occur in the matrix and that failure planes will not pass through the larger-size particles. Thus it might well be expected that the strength of the sample will be controlled by the matrix, and that it could be measured by testing a sample composed of matrix material alone at, say, the average void ratio of the matrix.

In the case of the stream-bed alluvium, therefore, in-situ dry densities of the alluvium were measured by the sand cone method in accordance with ASTM Test D1556. It was found that the in-situ dry density of the soil as a whole was 126.0 pcf. Maximum and minimum density tests were also performed on the soil as a whole with the following results:

Maximum dry density of soil (ASTM D 2049) = 134.9 pcf.
Minimum dry density of soil (ASTM D 2049) = 111.8 pcf.

From this data it may readily be determined that the relative density of the in-situ deposit is about 64%.

The entire material that was obtained from each test hole was then separated on a 3/8" screen and the weight of the plus 3/8" material was determined. Knowing the specific gravity of this material, its volume could be computed and subtracted from the total volume of the test hole; thus the dry density of the minus 3/8" material could be computed. It was found that this material had a dry density of 106 pcf. It may be noted that this measure of the dry density of the minus 3/8" material presumes that it includes all the voids in the total sample--as would be expected if the plus 3/8" particles were floating in the minus 3/8" matrix material.

In accordance with the preceding concepts, therefore, a testing program was initiated to determine the strength characteristics of samples of the minus 3/8 matrix at a dry density of 106 pcf. At the outset of this program, for the sake of completeness, it was also decided to determine the maximum and minimum densities of the minus 3/8" fraction of the soil. These tests led to the following results:

Maximum dry density of matrix (ASTM D 2049) = 123 pcf.
Minimum dry density of matrix (ASTM D 2049) = 106 pcf.

Since the in-situ dry density of the matrix (the minus 3/8" fraction of the natural deposit) was measured to be 106 pcf, this would indicate that it had a relative density of 0 percent in the ground. Thus the measured properties of this soil were as follows:

Relative density of entire soil $\approx$ 64 percent.
Relative density of matrix of soil $\approx$ 0 percent.

If indeed the matrix does control the strength, then the cyclic loading resistance or the residual strength of the matrix (and therefore of the soil) at a relative density of 0 percent would apparently be negligible. On the other hand, it is not likely that the strength of a soil having a relative density of 60 percent is negligible. In fact it would be expected to be large because at this relative density the soil as a whole would be expected to be dilatant under monotonic loading.

These computations lead to an interesting dilemma in our understanding of the behavior of such soils. The compositional studies described above led to the following conflicting conclusions:

1. The undrained strength of the soil as a whole is controlled by the characteristics of the matrix since the larger particles are floating in the matrix. The matrix has a very low relative density, it is compressive, and it has a very low undrained strength. Therefore the soil as a whole must have a very low undrained strength.
2. The relative density of the soil as a whole is about 64 percent. At this relative density the soil is dilatant and it has a very considerable undrained strength.

One might opt for one or the other of these conclusions on some other basis--for example that the soil appeared to be very stable, that alluvial deposits are hardly ever found to have a relative density less than 30 percent, or that the material involved in this case had existed for many years during which time it had been subjected to numerous earthquake shocks--an effect which would inevitably lead to some degree of densification. Thus the possibility of the matrix having zero percent relative density is nonexistent and there must be something wrong with the test data leading to this conclusion. Yet in fact a careful check on all the test data showed no errors in the results obtained. The problem apparently lies not in the accuracy of the data but in the interpretation of the results.

In an attempt to throw some light on this subject a more detailed series of studies was initiated to explore the maximum and minimum densities which would develop in the minus 3/8 in. fraction of the soil if it were compacted using the standard ASTM Test D 2049 in the presence of various proportions of plus 3/8 in. material. The plus 3/8 in. material added for this purpose ranged in size between 3/8 in. and 1-1/2 in. and the dry densities of the various soil compositions determined in this test program are shown in Fig. 8. Also shown in Fig. 8 are the theoretical values of maximum and minimum density which would be computed by replacing various proportions of the minus 3/8 in. material with solid particles (as floating particles) and assuming that this replacement does not alter the packing of the minus 3/8 in. material from its condition when tested alone. It is apparent from a comparison of the test results and the theoretical values computed as discussed above that in the presence of over-size or plus 3/8 in. particles, the ability of the minus 3/8 in. fraction to compact under the influence of the ASTM D 2049 compactive effort is considerably reduced, particularly for mixtures containing over 15 percent of particles larger than 3/8 in. size.

From the test data shown in Fig. 8 for the entire soil, it is a relatively simple matter to compute the densities of the minus 3/8 in. fraction in the different soil mixtures corresponding to the measured values of maximum and minimum dry density shown in Fig. 8. These results are shown in Fig. 9. It may be seen that if the dry density of the minus 3/8 in. fraction of the in-situ deposit (106 lb/cu ft.) is compared with the maximum and minimum densities of the minus 3/8 in. fraction tested alone, a situation in which $V_{valp} = 0$, its relative density is 0. This is the average relative density of the matrix. However, when it is compared with the maximum and minimum dry densities of the minus 3/8 in. fraction compacted by the ASTM D 2049 test in the presence of 40 percent of plus

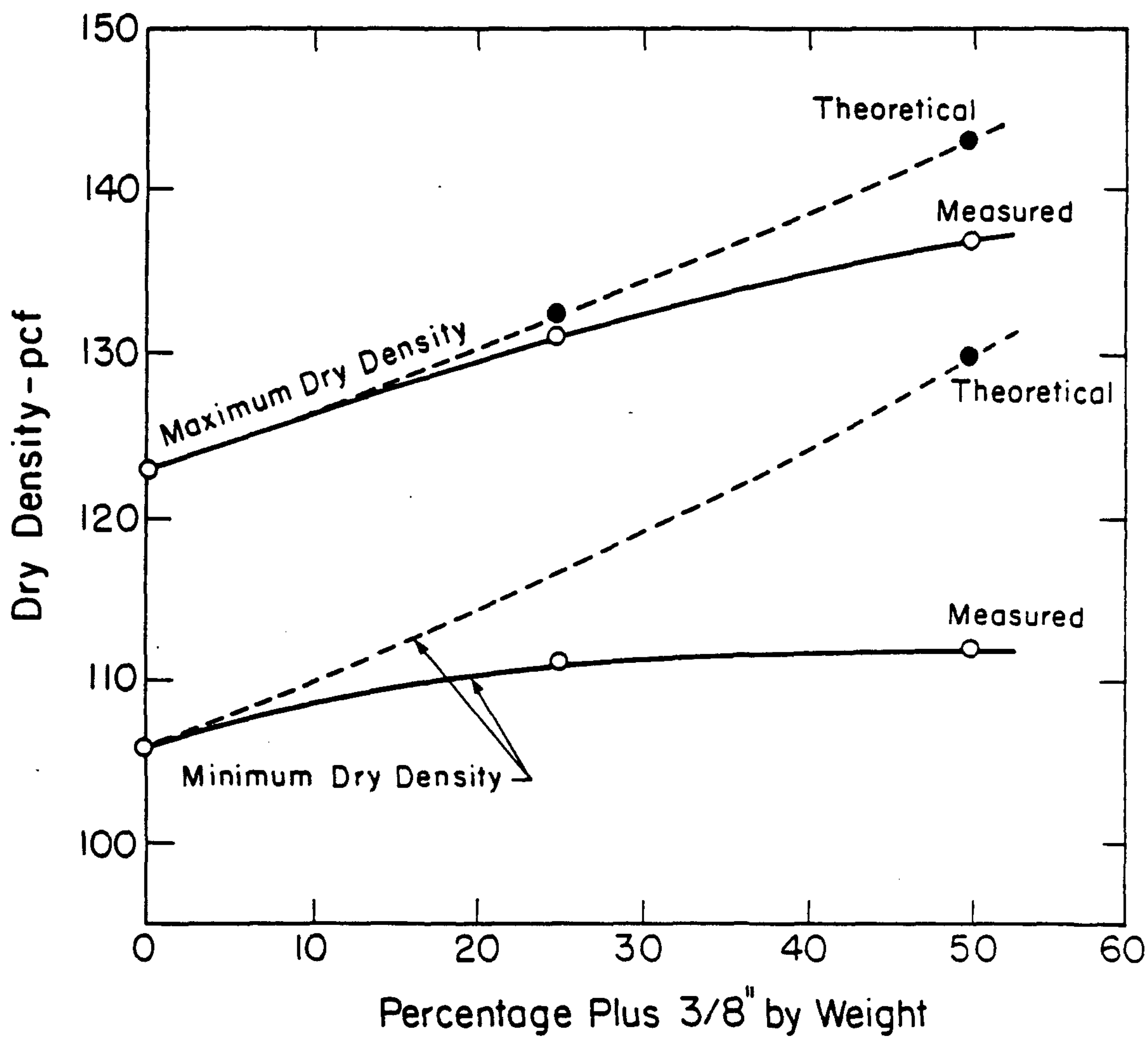

Fig. 8 EFFECT OF GRAVEL CONTENT ON MAXIMUM AND MINIMUM DENSITIES OF COARSE ALLUVIUM

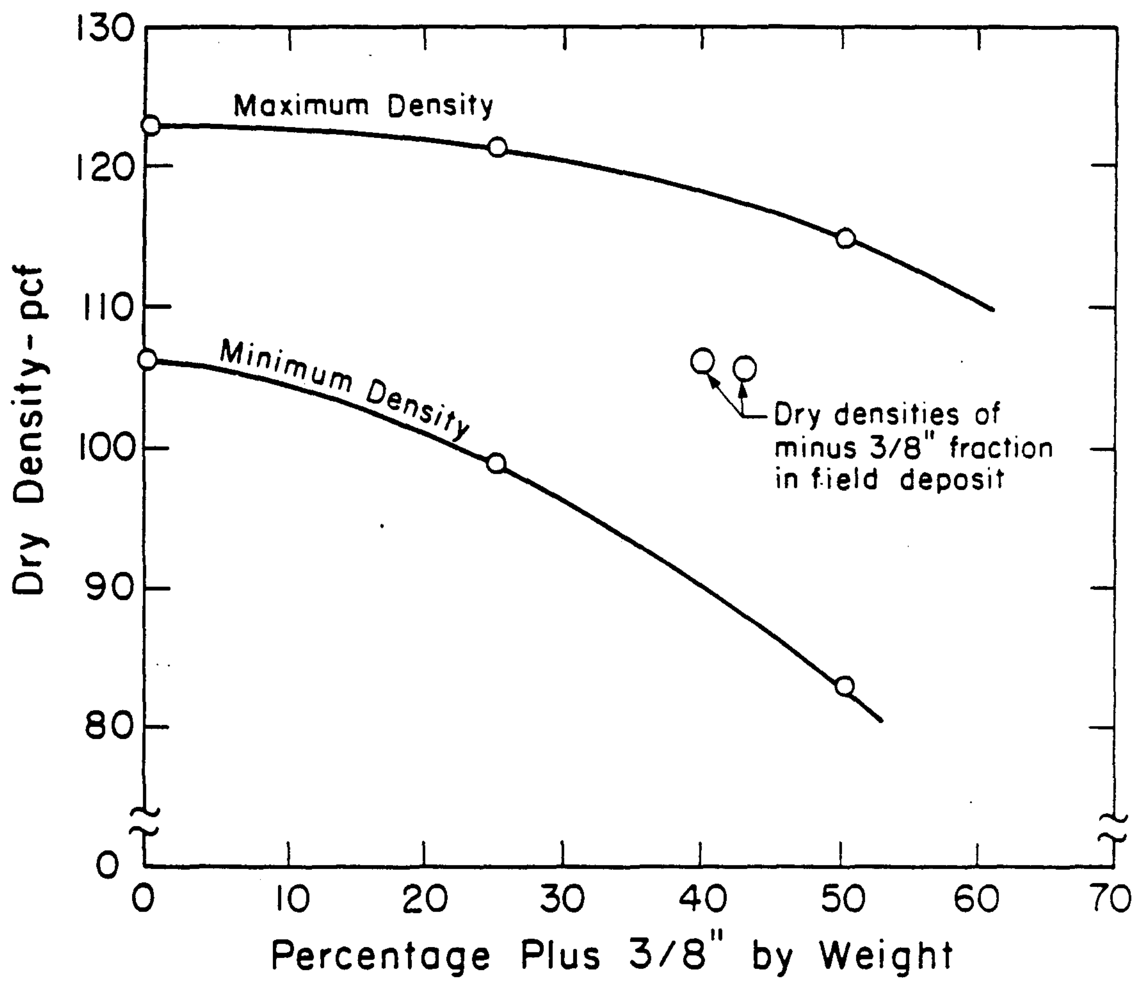

Fig. 9 MAXIMUM AND MINIMUM DENSITIES OF MINUS 3/8-INCH FRACTION OF COARSE ALLUVIUM

3/8 in. particles (as was the case for the in-situ deposit) its relative density is about 60 percent. In effect, this is the relative density of the far-field matrix since the effects of V_{valp} have been eliminated in the testing and data interpretation process (because V_{valp} has approximately the same values and thus the same influence on the maximum, minimum and in-situ density determinations).

This example provides a clear illustration of the possible effects of the quantity V_{valp} on the interpretation of test data for soils containing floating large particles and of the need to consider carefully the effects of oversize material on the properties of soils whose strength characteristics are clearly controlled by a matrix of finer-grained material. The particular soil discussed above seems to be an extreme case since other soils do not show such large effects of V_{valp}. However studies by other investigators have led to a similar general conclusion and it is particularly clearly illustrated by the case study described above. Undrained tests on samples of matrix material at its in-situ dry density of 106 pcf would have shown it to have a very low strength. Yet the in-situ deposit, whose strength was apparently controlled by this same matrix at this density, was clearly a stable material by all conventional criteria.

INVESTIGATION OF STRENGTH CHARACTERISTICS OF SOILS CONTAINING FLOATING LARGE PARTICLES

The above example seems to provide confirmation of the concept of near field and far-field matrix conditions on a compositional basis but it does not provide direct confirmation of the associated concepts that:

1. Relative movement in shear between large particles and smaller particles will not produce any significant tendency for volume

change and thus it is to be expected that the pore pressures generated in a soil containing floating large particles will be controlled primarily by the relative density of the far-field matrix, other factors such as fabric, etc. being the same. This relative density will be the same as that of the original total soil.

2. If the matrix is dilative, then under static loading conditions the surface of greatest weakness will probably be one passing through some parts of the matrix and then along the surfaces of large particles since no dilation (leading to increased strength) would be able to develop along these zones.

To explore these concepts, therefore, tests were performed on two distinctly different types of gravelly soils, one from Lake Valley Dam, a rockfill dam located in the Sierra Nevada mountains, approximately 50 miles northwest of Lake Tahoe, and the other from Oroville Dam in northern California. For both materials, tests were performed first on prototype material with 2-inch maximum size particles using 12-inch diameter samples. The gradations were such that the particles greater than 1/2-inch in size were floating in the minus 1/2-inch matrix material.

Having thus established the properties of the prototype material, the matrix material (the minus 1/2-inch size particles) was separated out and tested at the same relative density as the prototype material to permit a comparison of the cyclic loading resistance of the prototype and matrix materials.

Static load tests were also performed on prototype and matrix materials to compare the relative densities at which these materials had the same undrained strengths.

The results of these tests are presented in the following sections of this report.

Materials Tested

The gravel material from Lake Valley Dam was obtained from the same general location as the borrow area used during the buttressing operations of the dam. The material as obtained from the field was comprised of gravel, sand, and small amounts of silt and some cobbles. The gravel particles were rounded to subrounded with specific gravities ranging from a low of 2.70 for particles larger than 2-inch size to a high value of 2.76 for particles smaller than 1/2-inch size. The minus No. 4 sieve material had an unusually high specific gravity of 2.92.

Since laboratory cyclic testing of the fill material as obtained directly from the field was not feasible with the equipment available, the gravel material with a maximum particle size of 2 inches was designated as the prototype material for this study. The cyclic triaxial testing equipment available was capable of testing 12-inch diameter specimens. Therefore, with the maximum particle size of 2 inches, a satisfactory ratio of specimen diameter to maximum particle size of 6 to 1 could be maintained.

The smaller scale cyclic triaxial tests were performed on 2.8-inch diameter specimens. To maintain the specimen diameter to maximum particle size ratio of at least 6 to 1, the 1/2 inch sieve size was selected as the maximum particle size for the matrix material; this material was obtained simply by scalping the oversized (1/2-inch to 2-inch) material, in accordance with the concepts previously discussed. The grain size distributions of the assumed prototype and matrix materials are presented in Fig. 10.

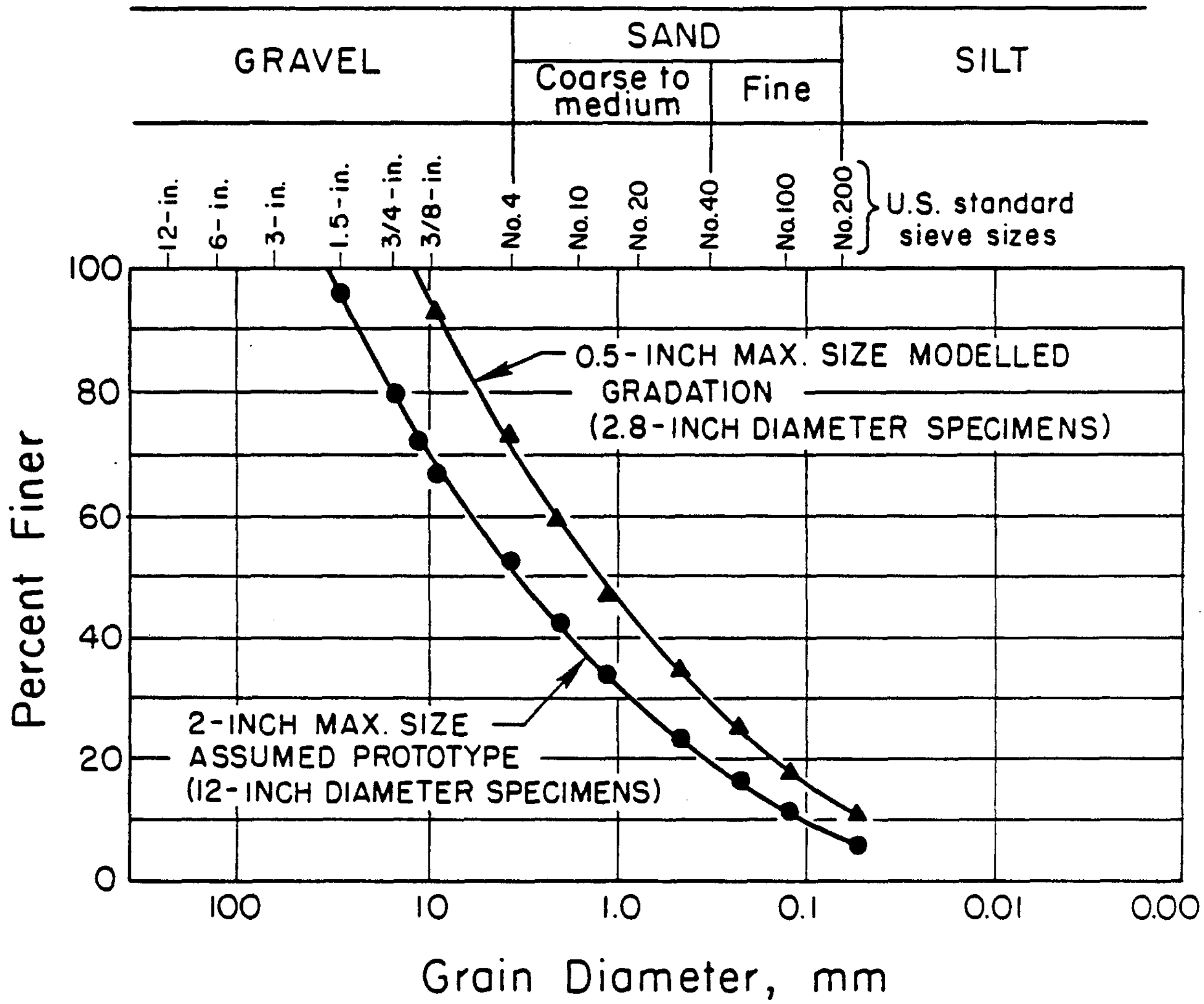

Fig. 10 GRAIN SIZE DISTRIBUTION CURVES OF THE PROTOTYPE AND THE MODELLED SPECIMENS, LAKE VALLEY DAM MATERIAL

Table 1 summarizes some important physical characteristics of the Lake Valley Dam material chosen for this study.

The gravel material from Oroville Dam consisting of cobbles, gravel, and sand left by earlier gold dredging operations was obtained from the same general location as the borrow area utilized for the construction of the dam. The gravel size particles were rounded to subrounded with rock particles being mostly amphibolite. Additional physical characteristics of the Oroville Dam gravel used in this study are presented in Table 2. The maximum particle size limitations for the assumed prototype and matrix materials which were applied to the Lake Valley gravel were also applied to Oroville gravel. The grain size distribution curves for the prototype (total) material and the modeled or matrix material specimens of Oroville gravel are presented in Fig. 11.

Test Programs

A. Cyclic Load Tests

Three series of cyclic load tests were performed in this study, as described below:

Series 1

a. 12-inch-diameter specimens of Lake Valley gravel with 2-inch maximum particle size were tested at 60 percent relative density.

b. 2.8-inch-diameter specimens of Lake Valley matrix material with 1/2-inch-maximum size were tested at 60 percent relative density.

Series 2

a. 12-inch-diameter specimens of Lake Valley gravel with 2-inch maximum particle size were tested at 40 percent relative density.

Table 1

Physical Characteristics of Gravel Material from Lake Valley Dam

A. Assumed Prototype, 2-inch Maximum Particle Size Material

1. Type of material: river gravel, rounded to subrounded

2. Unified soil classification: GW-GM

3. Mean grain diameter (D_{50})(mm): 3.8

4. Uniformity coefficient (C_u): 62.5

5. Coefficient of curvature (C_c): 1.0

6. Specific gravity:

Minus No. 4 sieve	2.92
No. 4 sieve to 1/2-inch maximum size	2.76
1/2-inch to 2-inch maximum size	2.74
Average	2.81

7. Plasticity index: Minus No. 40 material is nonplastic

8. Maximum and minimum dry densities as obtained from the ASTM D-2049 testing procedures:

Maximum dry density	139 pcf
Minimum dry density	114 pcf

9. Void ratio:

e_{max}	0.538
e_{min}	0.261

10. Moisture-density relationship as obtained from the ASTM D-1557-78 testing procedures:

Maximum dry density	137 pcf
Optimum water content	9.1%

11. Gradation:

U.S. Standard Sieve Size	Percent Passing
2-inch	100.0
1-1/2-inch	93.4
3/4-inch	79.3
3/8-inch	65.5
No. 4	53.1

Table 1 - contd.

B. Modeled Samples, 1/2inch Maximum Particle Size Material

1. Unified soil classification: SW-SM

2. Mean grain diameter (D_{50})(mm): 1.4

3. Uniformity coefficient (C_u): 29.3

4. Coefficient of curvature (C_c): 1.07

5. Specific gravity:

Minus No. 4 sieve	2.92
No. 4 sieve to 1/2-inch maximum size	2.76
Average	2.84

6. Plasticity index: Minus No. 40 material is nonplastic

7. Maximum and minimum dry densities as obtained from the ASTM D-2049 testing procedures:

Maximum dry density	132 pcf
Minimum dry density	105 pcf

8. Void ratio:

e_{max}	0.688
e_{min}	0.343

9. Moisture-density relationship as obtained from the ASTM D-1557-78 testing procedures:

Maximum dry density	132 pcf
Optimum water content	10.2%

10. Gradation:

U.S. Standard Sieve Size	Percent Passing
1/2-inch	100.0
3/8-inch	91.9
No. 4	74.5

C. Miscellaneous Information

1. Amount of oversize particles (1/2-inch to 2-inch) present in the assumed prototype: = 29 percent of the total (prototype) material based on dry weight.

2. Minimum dry density as obtained from the ASTM D-2049 testing procedure of 1/2-inch to 2-inch maximum size material: 94 pcf.

Table 2

Physical Characteristics of Gravel Material from Oroville Dam

A. Assumed Prototype, 2-inch Maximum Particle Size Material

1. Type of material: river gravel, rounded to subrounded

2. Unified soil classification: GW-GM

3. Mean grain diameter (D_{50})(mm): 9.53

4. Uniformity coefficient (C_u): 47.0

5. Coefficient of curvature (C_c): 3.85

6. Specific gravity:
    - Coarse fraction (2"-3/16") 2.92
    - Fine fraction (-3/16") 2.78
    - Average 2.85

7. Atterberg's Limits:
    - Liquid Limit (LL)(%) 15.5-17.0
    - Plastic limit (PL)(%) 15.0-17.0
    - Plasticity index (PI)(%) 0.5-0.0

8. Maximum and minimum dry densities as obtained from the ASTM D-2049 testing procedures:
    - Maximum dry density 153.6 pcf
    - Minimum dry density 125.0 pcf

9. Void ratio:
    - e_{max} 0.440
    - e_{min} 0.165

10. Gradation:

U.S. Standard Sieve Size	Percent Passing
2-inch	100.0
1-1/2-inch	92.5
3/4-inch	73.0
1.2-inch	59.0
3/8-inch	49.0
No. 4	31.0

Table 2 - contd.

B. Modeled Samples, 1/2inch Maximum Particle Size Material

1. Unified soil classification:	GW-GM
2. Mean grain diameter (D_{50})(mm):	4.0
3. Uniformity coefficient (C_u):	38.7
4. Coefficient of curvature (C_c):	4.83
5. Specific gravity:	
Coarse fraction (2"-3/16")	2.92
Fine fraction (-3/16")	2.78
Average	2.85

6. Plasticity index: Minus No. 40 material is nonplastic to slightly plastic

7. Maximum and minimum dry densities as obtained from the ASTM D-2049 testing procedures:

Maximum dry density	139.9 pcf
Minimum dry density	107.3 pcf

8. Void ratio:

e_{max}	0.617
e_{min}	0.240

10. Gradation:

U.S. Standard Sieve Size	Percent Passing
1/2-inch	100.0
3/8-inch	83.05
No. 4	52.54

C. Miscellaneous Information

1. Amount of oversize particles (1/2-inch to 2-inch) present in the assumed prototype: = 41 percent of the total (prototype) material based on dry weight.

2. Minimum dry density as obtained from the ASTM D-2049 testing procedure of 1/2-inch to 2-inch maximum size material: 103 pcf.

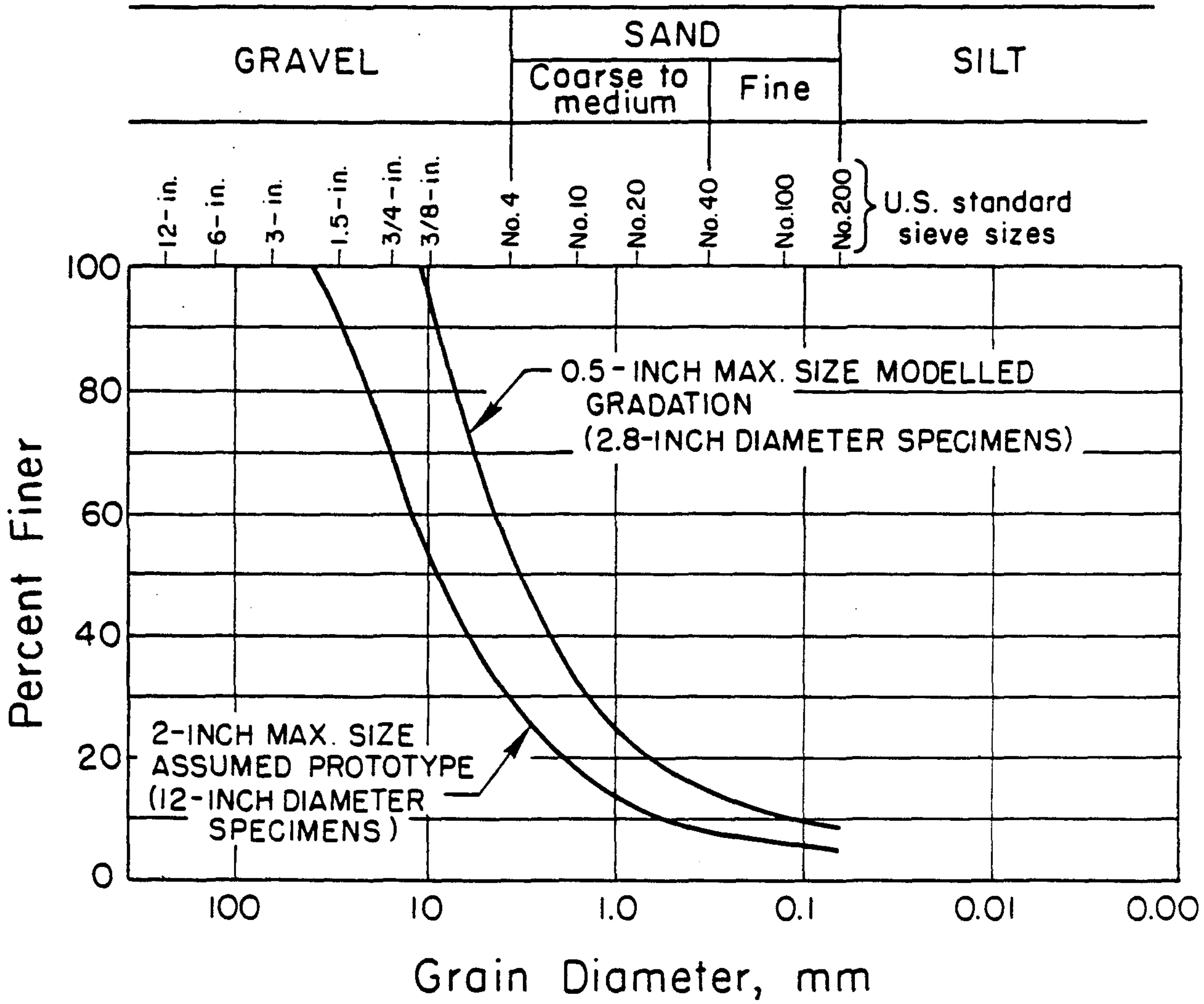

Fig. 11 GRAIN SIZE DISTRIBUTION CURVES OF THE PROTOTYPE AND THE MODELLED SPECIMENS, OROVILLE DAM MATERIAL

b. 2.8-inch-diameter specimens of Lake Valley gravel matrix material with 1/2-inch-maximum size were tested at 40 percent relative density.

Series 3

a. 12-inch-diameter specimens of Oroville gravel with 2-inch maximum particle size were tested at 84 percent relative density. This test series was performed by Banerjee (1979).

b. 2.8-inch-diameter specimens of Oroville gravel matrix material with 1/2-inch-maximum size were tested at 84 percent relative density.

For each group of specimens, a series of consolidated-undrained cyclic triaxial tests were performed with different cyclic deviator stresses. Each test was continued until the sample developed at least 10 percent double amplitude strain in 4 to 50 cycles. In each test, the development of pore water pressures and axial strains with increasing numbers of stress cycles was recorded on either a Sanborn or MFE four-channel recorder. All the tests were performed with an initial isotropic effective confining pressure of 2 ksc.

B. Static Tests

The Lake Valley gravel, the physical characteristics of which are presented in Table 1, was selected for the study on static strength testing. An initial relative density of 55 percent was selected for the large-scale (12-inch diameter) specimens of the assumed prototype material with maximum grain size of 2-inch. The test samples were consolidated under a confining pressure of 2 ksc. The relative density following completion of primary consolidation was 58 percent. For the prototype relative density of 58 percent, the computed average relative density of the soil matrix was 44 percent. Since the relative density of the "far-field" matrix is higher than the "average" soil matrix density, nine consolidated undrained triaxial compression tests with pore-pressure measurements were performed on 2.8-inch diameter specimens of the soil matrix with maximum particle size of 1/2-inch, prepared at relative densities after consolidation ranging from 44 to 57 percent.

The results of these different test programs are presented below.

A. Cyclic Tests

The results of the cyclic tests performed on 12-inch and 2.8-inch diameter specimens of Lake Valley and Oroville gravels are presented in Figs. 12 through 16 in the form of plots of stress ratio, $\sigma_{dc}/2\sigma_{3c}$, versus the number of stress cycles, N_c, required to cause a residual pore pressure ratio of 100 percent or different levels of double amplitude axial strain. The test data presented include all corrections (area, membrane strength, and system compliance). In general, for dense specimens of Oroville gravel (D_r = 84%) a pore pressure ratio of 100 percent was developed in the cyclic axial strain range of 2.75 to 3.5 percent. The corresponding strain levels for samples of Lake Valley gravel prepared at a relative density of 60 percent were 3 to 5 percent. Pore water pressure generation curves were prepared by plotting the pore water pressure ratio developed (Δr_u) vs. the cycle ratio (N/N_ℓ) where $r_u = \Delta u/\sigma'_{3,i}$, N = the current cycle number, and N_ℓ = the number of cycles to full initial liquefaction. The initial portion of the pore water pressure generation curves was markedly steeper for test specimens of higher relative density.

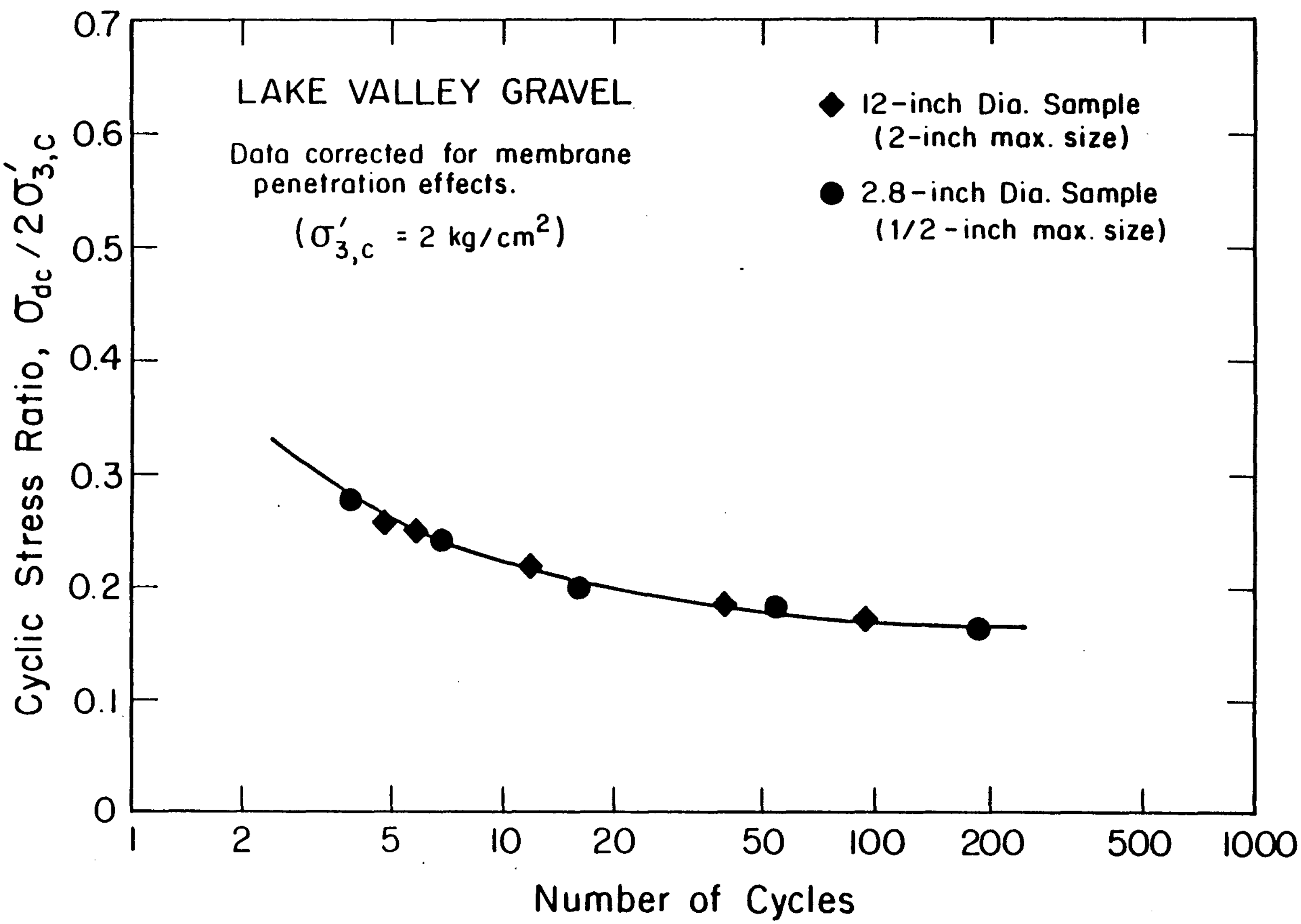

Fig. 12 CYCLIC STRESS RATIOS CAUSING RESIDUAL PORE PRESSURE RATIO OF 100% IN DIFFERENT NUMBERS OF CYCLES FOR 12-INCH AND 2.8-INCH DIAMETER SPECIMENS: LAKE VALLEY GRAVEL AT A RELATIVE DENSITY OF 40%

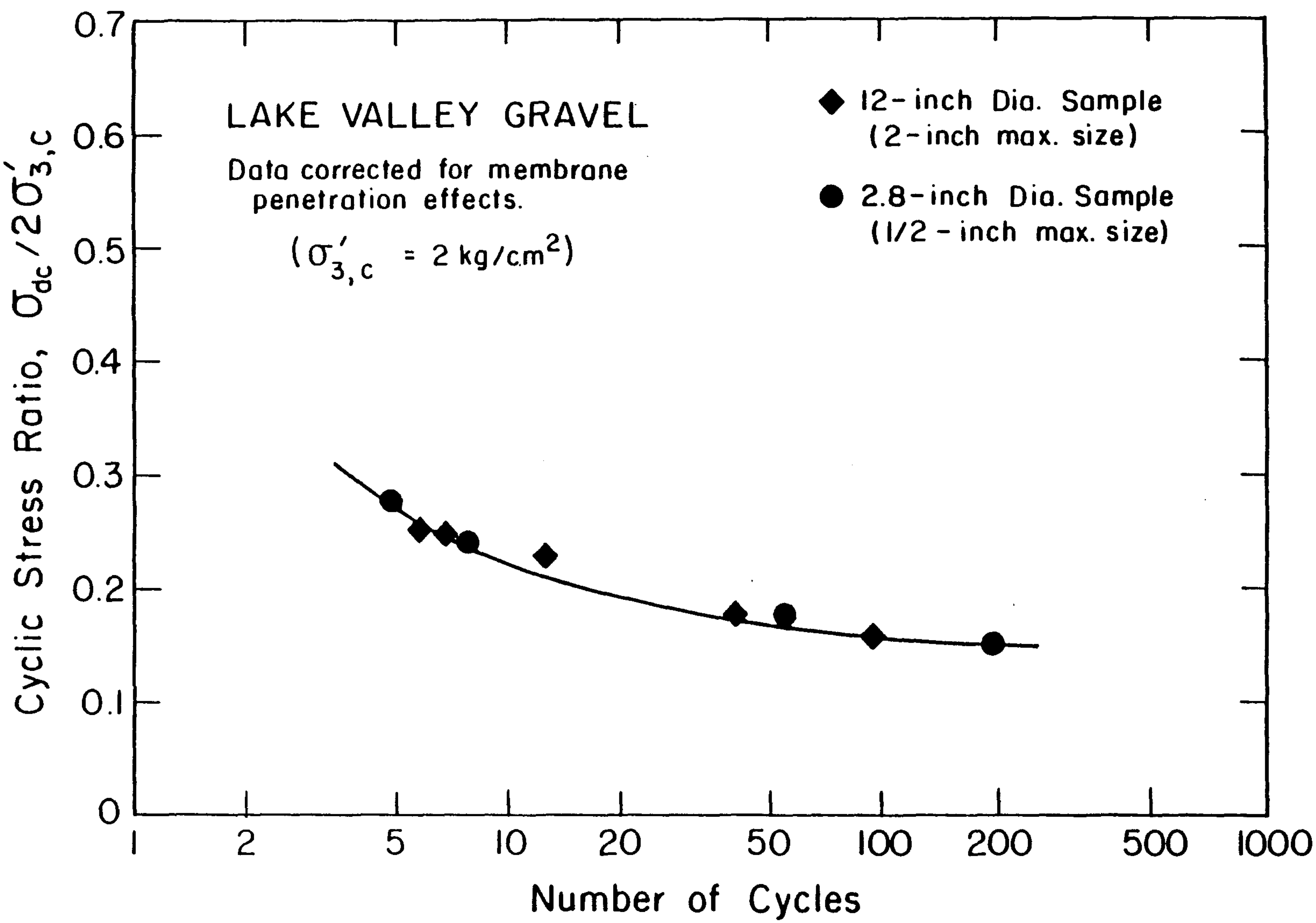

Fig. 13 CYCLIC STRESS RATIOS CAUSING 10% DOUBLE AMPLITUDE AXIAL STRAIN IN DIFFERENT NUMBERS OF CYCLES FOR 12-INCH AND 2.8-INCH DIAMETER SPECIMENS: LAKE VALLEY GRAVEL AT A RELATIVE DENSITY OF 40%

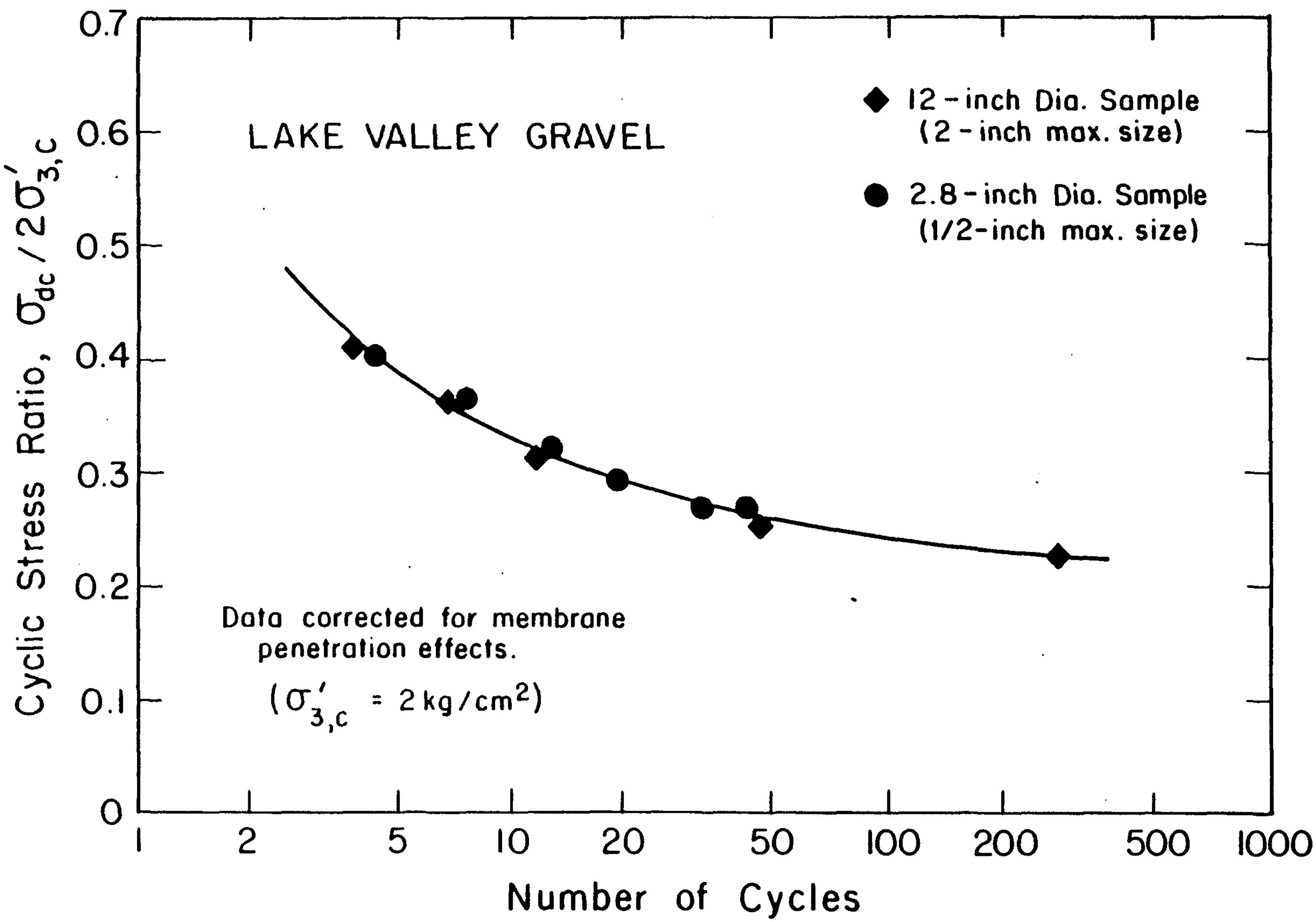

Fig. 14 CYCLIC STRESS RATIOS CAUSING RESIDUAL PORE PRESSURE RATIO OF 100% IN DIFFERENT NUMBERS OF CYCLES FOR 12-INCH AND 2.8-INCH DIAMETER SPECIMENS: LAKE VALLEY GRAVEL AT A RELATIVE DENSITY OF 60%

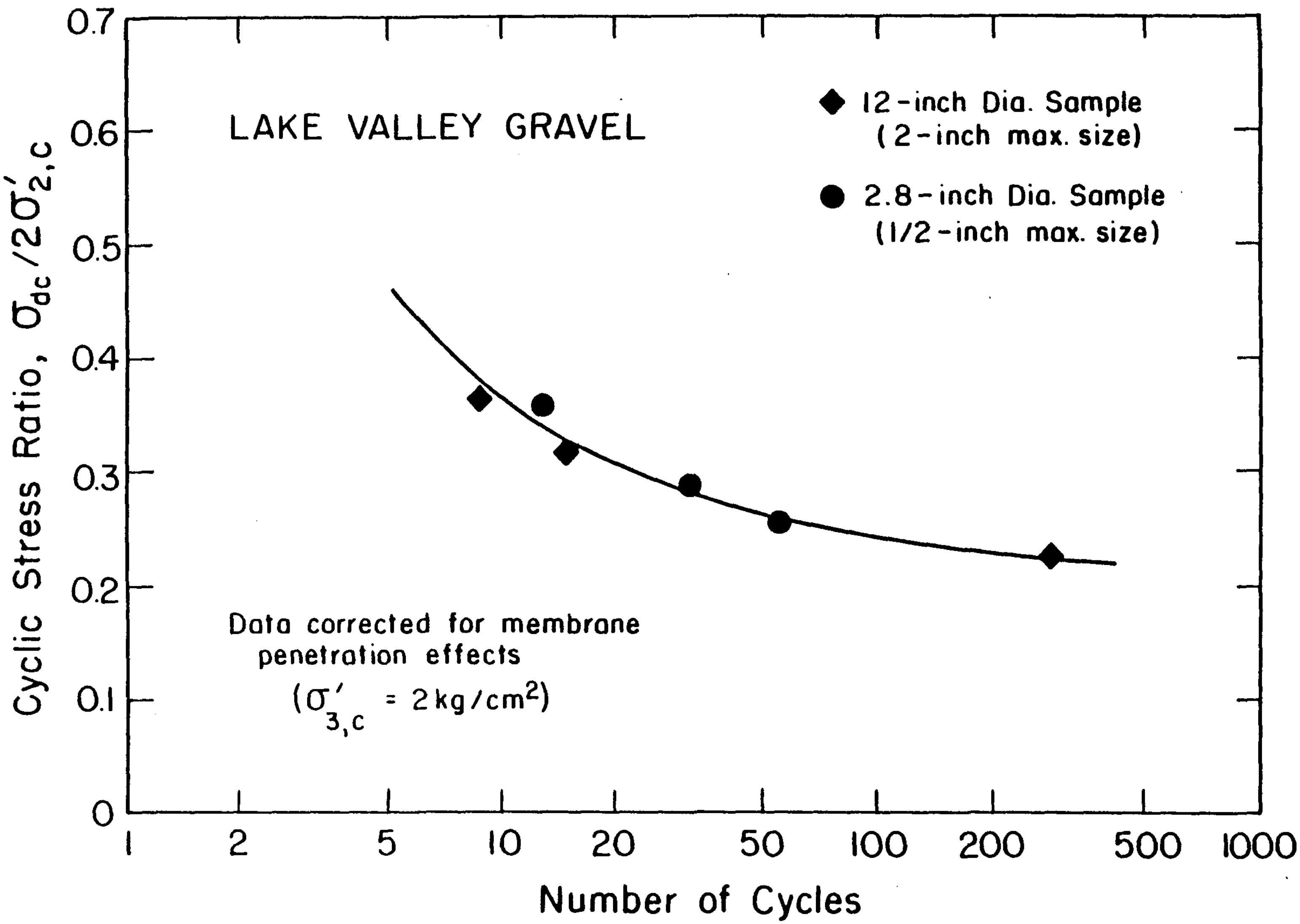

Fig. 15 CYCLIC STRESS RATIOS CAUSING 10% DOUBLE AMPLITUDE AXIAL STRAIN IN DIFFERENT NUMBERS OF CYCLES FOR 12-INCH AND 2.8-INCH DIAMETER SPECIMENS: LAKE VALLEY GRAVEL AT A RELATIVE DENSITY OF 60%

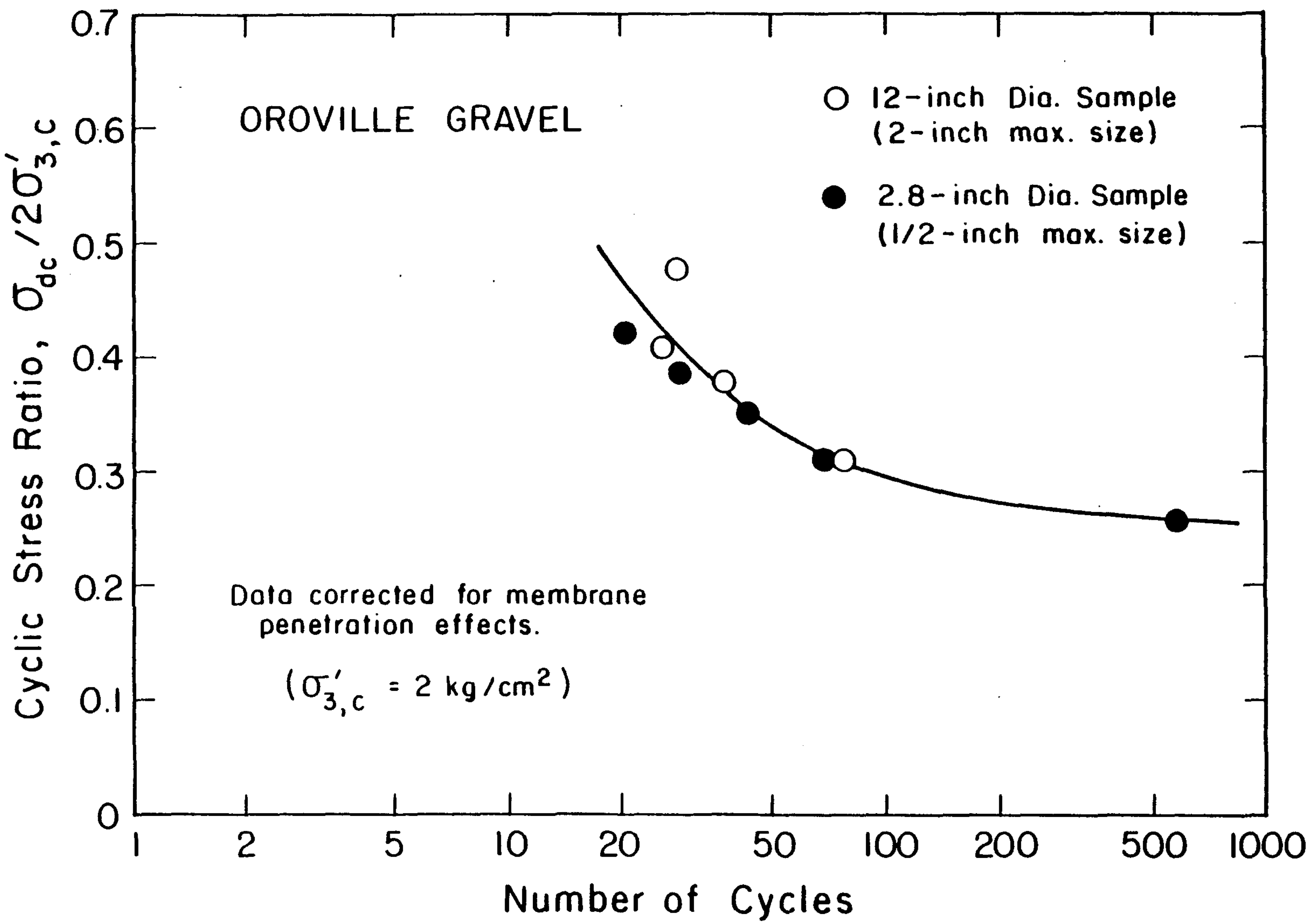

Fig. 16 CYCLIC STRESS RATIOS CAUSING RESIDUAL PORE PRESSURE RATIO OF 100% IN DIFFERENT NUMBERS OF CYCLES FOR 12-INCH AND 2.8-INCH DIAMETER SPECIMENS: OROVILLE GRAVEL AT A RELATIVE DENSITY OF 84%

It may be seen from the data presented in Figs. 12 to 16 that there was excellent agreement between the data obtained from tests on the prototype and matrix materials when they were tested at the same relative densities. Although tests were not performed to verify the results, it seems apparent that very different results would have been obtained if the cyclic loading resistance of the prototype material had been compared with that of the matrix tested at the average relative density of the matrix material in the prototype soils, even though the cyclic loading resistance of the prototype material is controlled completely by the matrix material. This result would seem to confirm the previous discussion of the need to consider carefully the test density for the matrix material in order to obtain meaningful results in test programs of this type.

B. Static Test Results

The results of the tests performed on 12-inch diameter specimens of the assumed Lake Valley gravel prototype material and 2.8-inch diameter specimens of the soil matrix are summarized in Table 3. The undrained strength, S_u, of the matrix samples is plotted against the relative density of the matrix in Fig. 17. It may be seen that the soil matrix specimen with a relative density of about 51 percent (almost halfway between the computed average soil matrix relative density of 44 percent and the relative density of the total material of 58 percent), has about the same undrained strength as that of the total material, indicating again the need for careful consideration of the condition of the matrix material in determining the strength of prototype materials from tests on matrix materials.

Table 3

Summary of Static Test Results on 12-in. Diameter Samples of Lake Valley Gravel and 2.8-in. Diameter Samples of Lake Valley Gravel Matrix Material

Test No.	Sample Diameter in.	Relative Density %	Dry Density pcf	S_u ksc	Average S_u ksc	σ'_{3f} ksc	Average σ'_{3f} ksc	ϕ' Degrees
1	12.0	58	127	5.2		1.45		39.8
2	12.0	58	127	5.3	5.6	1.60	1.61	38.4
3	12.0	58	127	6.3		1.80		39.5
1	2.8	44.*	115.4	2.9	2.9	0.92	0.92	37.7
2	2.8	47.2	116.2	3.9		1.20		38.3
3	2.8	47.2	116.2	4.1	4.1	1.44	1.31	36.0
4	2.8	47.3	116.2	4.2		1.30		38.1
5	2.8	48.2	116.5	4.5	4.5	1.40	1.40	38.1
6	2.8	49.8	116.9	5.2	5.2	1.60	1.60	38.3
7	2.8	53.6	117.5	6.8	6.8	1.86	1.86	40.3
8	2.8	57.2	118.9	8.1	7.8	2.18	2.18	40.6
9	2.8	57.2	118.9	7.5		2.17		39.2

*Average relative density of the soil matrix (obtained by computations based on mass-density relationships).

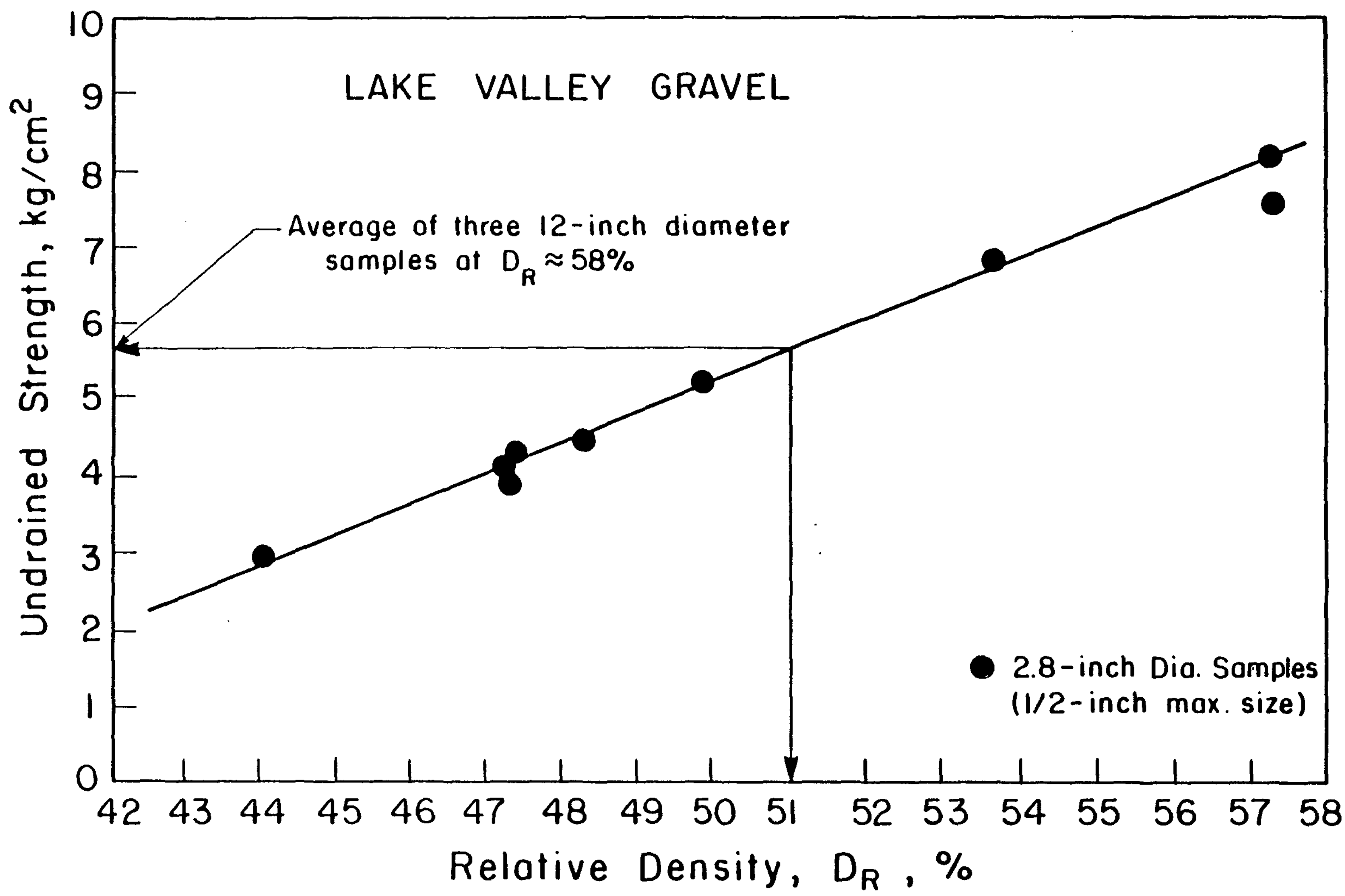

Fig. 17 UNDRAINED STRENGTH VS. RELATIVE DENSITY FOR 2.8-INCH DIAMETER SPECIMENS OF LAKE VALLEY GRAVEL MATRIX MATERIAL

CONCLUSIONS

A. Cyclic Strength Prediction

1. It was hypothesized that in order to predict the cyclic behavior of a prototype (total) material with "floating" oversized grains, laboratory tests should be performed on the soil matrix alone at the same relative density as that of the total prototype material. Based on the test results obtained during this study for both Oroville and Lake Valley gravels, it was concluded that this hypothesis appears to be substantiated.
2. Because of the range of the relative densities (40 to 84 percent) chosen for tests on two distinctly different gravelly materials, it was concluded that the above-mentioned hypothesis appears to be applicable to widely different coarse-grained cohesionless materials with "floating" oversized grains.
3. The generation of pore water pressure during undrained cyclic loading and the deformation characteristics of the prototype and matrix materials prepared to the same relative density were similar in nature. This agreement was better when the results were compared on the basis of the stress ratio required to cause a pore pressure ratio of 100% than for comparisons based on the stress ratio required to produce higher double amplitude axial strains in the range of 10 percent.
4. The maximum particle size limit of 2-inches used in this study was based on necessity, since with the equipment available, the maximum particle size which could be accommodated in a 12-inch diameter specimen was 2-inches. The modeling criteria, however,

should be equally applicable to other prototype materials with maximum particle sizes larger than 2-inches.

5. The errors in cyclic stress ratio required to cause 100 percent pore water pressure ratio or produce a given level of double amplitude axial strain, introduced by the membrane penetration effects, were significant; in this study, the correction in cyclic stress ratio due to membrane penetration effects was approximately 10 percent for both the specimens of Lake Valley gravel and the Oroville gravel.

B. Static Strength Prediction

It was hypothesized that in order to predict the static strength behavior of the prototype (total) material, tests should be performed on the soil matrix alone (the modeled specimen) at a relative density less than that of the prototype material or the "far-field" soil matrix as it exists within the total material.

Based on the test results obtained in this study, it would appear that the test relative density for the matrix giving the same undrained strength as that of the prototype is about half way between the average relative density of the matrix in the prototype material and the relative density of the total prototype material. This result is consistent with the concepts of relative density distribution in the matrix of a soil containing floating oversize particles presented in this report. However since tests were performed on only one type of gravel (Lake Valley gravel) and the prototype samples were prepared at only one relative density (58 percent), this conclusion can only be considered tentative in nature and should be investigated further by tests on gravels with different gradations tested at different relative densities.

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