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Factors affecting post-liquefaction strength assessment

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ABSTRACT: Several factors potentially affecting evaluation of post-liquefaction "steady state" shear strength based on laboratory testing are investigated, including soil fabric or method of sample preparation, initial stress anisotropy, rate of strain, sample handling methods, and membrane compliance effects. Suggestions are made regarding (a) the potential significance of each of these factors, and (b) the possible ramifications with respect to the use of steady state methods as a basis for post-liquefaction stability analyses.

1 INTRODUCTION

The "steady state" method for analysis of post-liquefaction stability proposed by Poulos et al. (1984) has been increasingly applied to analysis of dams and structural foundations since its initial development in the late 1960s. This method of liquefaction stability analysis is based on the premise that post-liquefaction shear deformations will occur under "steady state" conditions. The concept of steady state deformation is closely analogous to the critical state concept, and is defined as a state of plastic flow at constant volume, constant normal effective stress, constant shear stress (constant undrained residual strength), and constant rate of shear strain. If the driving shear stresses within a soil mass are less than the undrained steady state shear strength, then the soil mass is considered not to be susceptible to liquefaction failure with associated large deformations.

At the heart of the steady state concept, as it is currently applied to liquefaction analyses (Poulos et al. 1984; Castro et al. 1984), are the assumptions that (a) steady state residual strength and effective stress conditions are reasonably unique functions of initial void ratio, and (b) their relationship with initial void ratio can be evaluated based on controlled-rate-of-strain IC-U triaxial testing.

This paper presents the results of an investigation of these assumptions. Several factors which may affect steady state residual strength and effective stress conditions and/or their evaluation based on laboratory testing are considered, and these are: (a) soil fabric or method of sample preparation, (b) re-use of bulk samples, (c) rate of strain, (d) initial stress anisotropy, and (e) membrane compliance effects.

1.1 The steady state analysis method

The steady state analysis method proposed by Poulos et al. (1984) consists of five steps: (1) determination of in-situ void ratio (e), (2) determination of steady state strength as a function of void ratio based on controlled-rate-of-strain IC-U tests of compacted bulk samples, (3) IC-U testing to determine steady state strengths of "undisturbed" samples, (4) correction of these "undisturbed" test results for void ratio changes which occur during sampling and subsequent test preparation to arrive at estimates of in-situ steady state strengths, and (5) determination of in-situ driving shear stresses. The correction in step 4 is accomplished by plotting the $\log_{10}$ of either steady state strength ($\tau_{f,ss}$) or effective confining stress ($\sigma'_{3,c}$) vs. void ratio from step 2 on the same plot with $\tau_{f,ss}$ or $\sigma'_{3,c}$ values from step 3, and then assuming the samples tested in step 3 can be "corrected" for void ratio changes by assuming their $\tau_{f,ss}$ or $\sigma'_{3,c}$ vs. void ratio relationships are "parallel" in semi-log space to those of the bulk samples tested in step 2. This is a potentially significant assumption, as relatively small void ratio changes may result in large corrections of $\tau_{f,ss}$ or $\sigma'_{3,c}$.

2 EFFECTS OF SOIL FABRIC OR METHOD OF SAMPLE PREPARATION

Implicit in the correction method employed in step 4 is the assumption that the semi-log slope of the e vs. $\tau_{f,ss}$ or e vs. $\sigma'_{3,c}$ relationship is independent of soil fabric or method of sample preparation, so that "compacted" bulk samples can be used as a basis for correction of "undisturbed" test results.

A number of controlled-rate-of-strain IC-U triaxial tests were performed on 2.8-in. diameter samples of Sacramento River sand to investigate this assumption. Sacramento River sand is a clean, fine, uniformly graded sand (as shown in Fig. 1), and is primarily quartzitic with subangular to subrounded grains. This sand was selected for its lack of unusual characteristics and because it is fine enough to preclude significant detrimental membrane compliance effects in testing of samples of this scale.

Samples were prepared by two different techniques: (a) dry pluviation and (b) moist tamping in layers. Following vacuum/backpressure saturation to B-values of greater than 0.985, samples prepared by

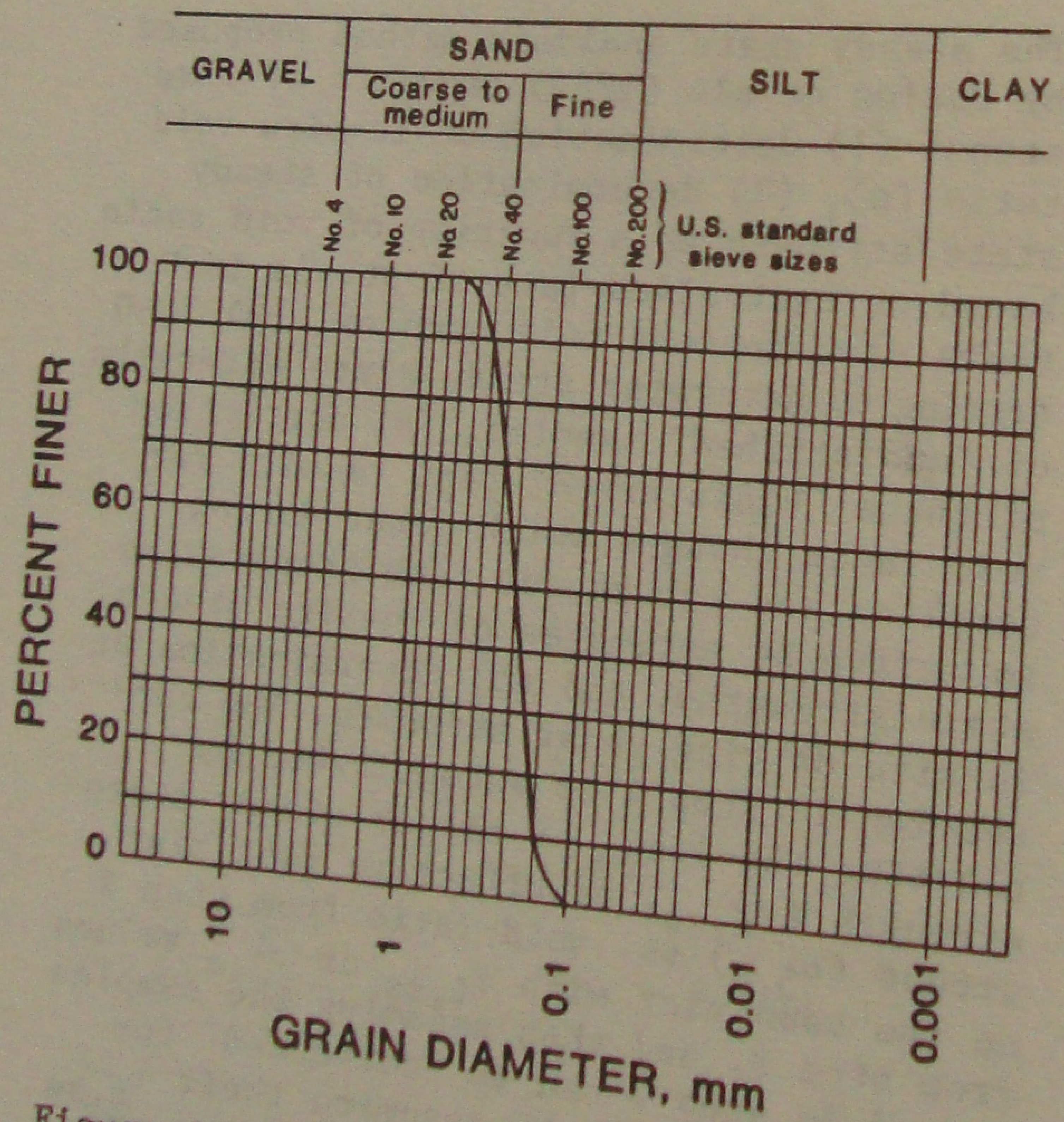

Figure 1. Gradation curve for Sacramento River sand.

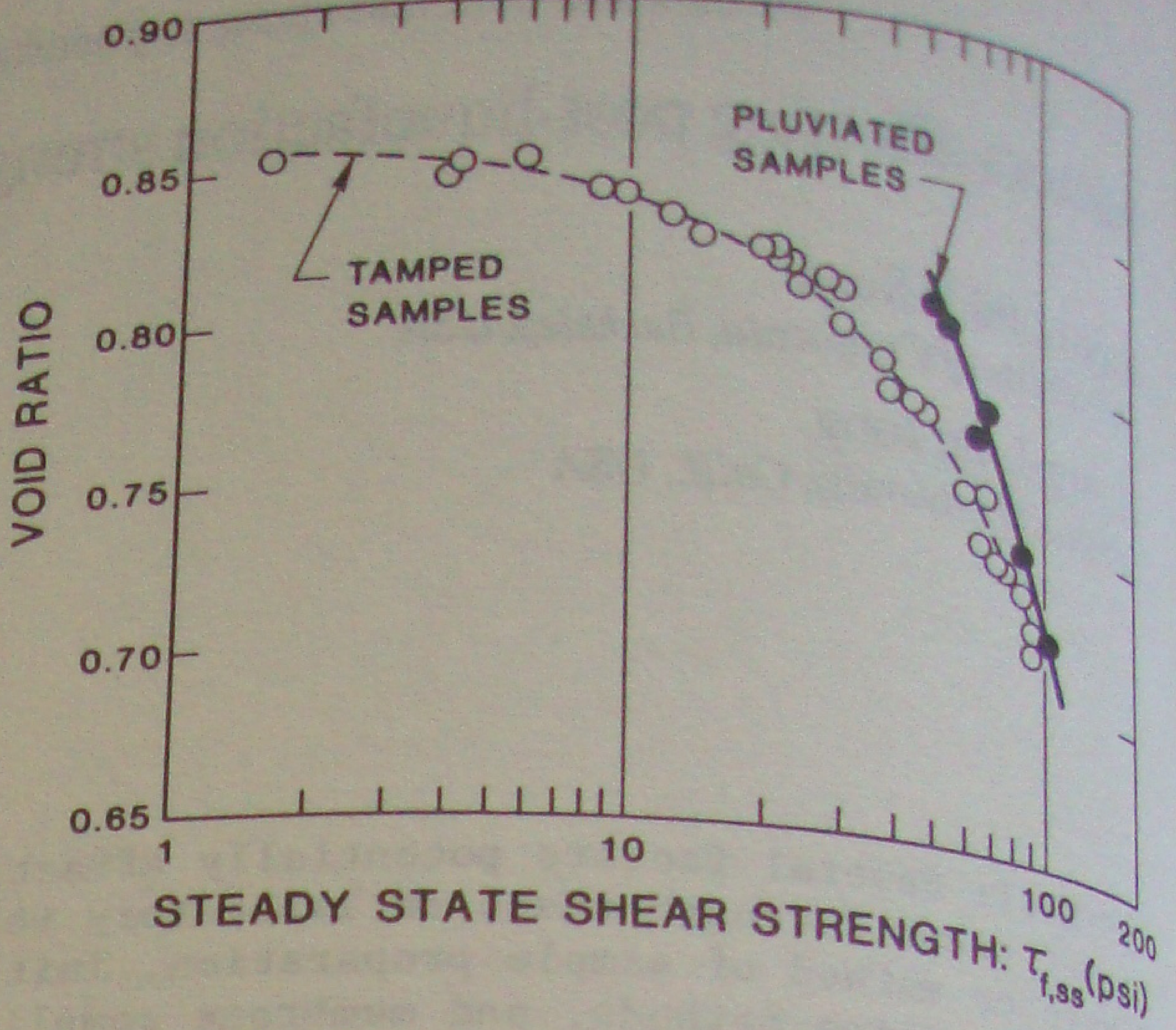

Figure 2. Steady state strengths of pluviated vs. moist tamped samples of Sacramento River sand.

both methods were isotropically consolidated to a "contractive" state prior to undrained shearing. Figure 2 shows the results of this test series. As shown in this figure, the two different sample preparation methods resulted in significantly different "steady state lines" with different slopes. This suggests that tests based on tamped samples would not represent a viable basis for correction of tests on "undisturbed" samples of Sacramento River sand if the in-situ soil fabric conditions resulted from a pluviation depositional process.

It should be noted that not all soils appear to have steady state lines whose slopes are significantly affected by method of sample preparation. Additional tests of samples of non-plastic sandy silt from the downstream hydraulic fill section of Lower San Fernando Dam were also performed as part of these studies (Seed et al. 1987). Samples were formed by both (a) moist tamping, and (b) hydraulic pluviation and subsequent consolidation from near slurry-like conditions, and the steady state lines resulting from tests by both sample preparation methods were essentially identical.

It would thus appear that soil fabric, or method of sample preparation, is a factor which may significantly influence critical state soil behavior, and that this should be investigated for individual soils as a part of the steady state analysis and testing process.

3 EFFECTS OF RE-USE OF BULK SAMPLES

For this investigation, a virtually inexhaustible supply of Sacramento River sand was available. This is not always the case when dealing with bulk samples obtained from the field, however, raising the question as to whether or not samples can be fabricated (or reconstituted), re-using soil from earlier tests on reconstituted bulk samples, for purposes of determining the steady state line as called for in step 2 of the steady state analysis method.

Soil from IC-U tests on samples of Sacramento River sand was saved, and subsequently used in an additional series of IC-U tests. These "re-used" soil samples were again prepared by moist tamping. Figure 3 shows the results of these tests on re-used soil, along with the results of tests of fresh or "virgin" soil samples prepared by tamping from Fig. 2. The steady state line for the re-used soil differs significantly from that of the virgin soil.

Figure 4 shows gradation curves for both the fresh and virgin soils. The re-used soil shows a nominally increased fines content of from zero to two percent. This fines content is not sufficient to provide meaningful "lubrication" between the sand grains. It is also not sufficient that its weight contribution explains the offset in void ratios for any given value of $\tau_{f,ss}$ as shown in Fig. 3; and in any case, this offset is greatest at low void ratios. It is tentatively hypothesized that the significant effect of sample re-use on critical state behavior is due to grain particle damage (grain breakage and removal of sharp corners). As these effects might be virtually undetectable by means of gradation analyses, it is recommended that bulk sample material not be re-used in testing for evaluation of steady state strength conditions.

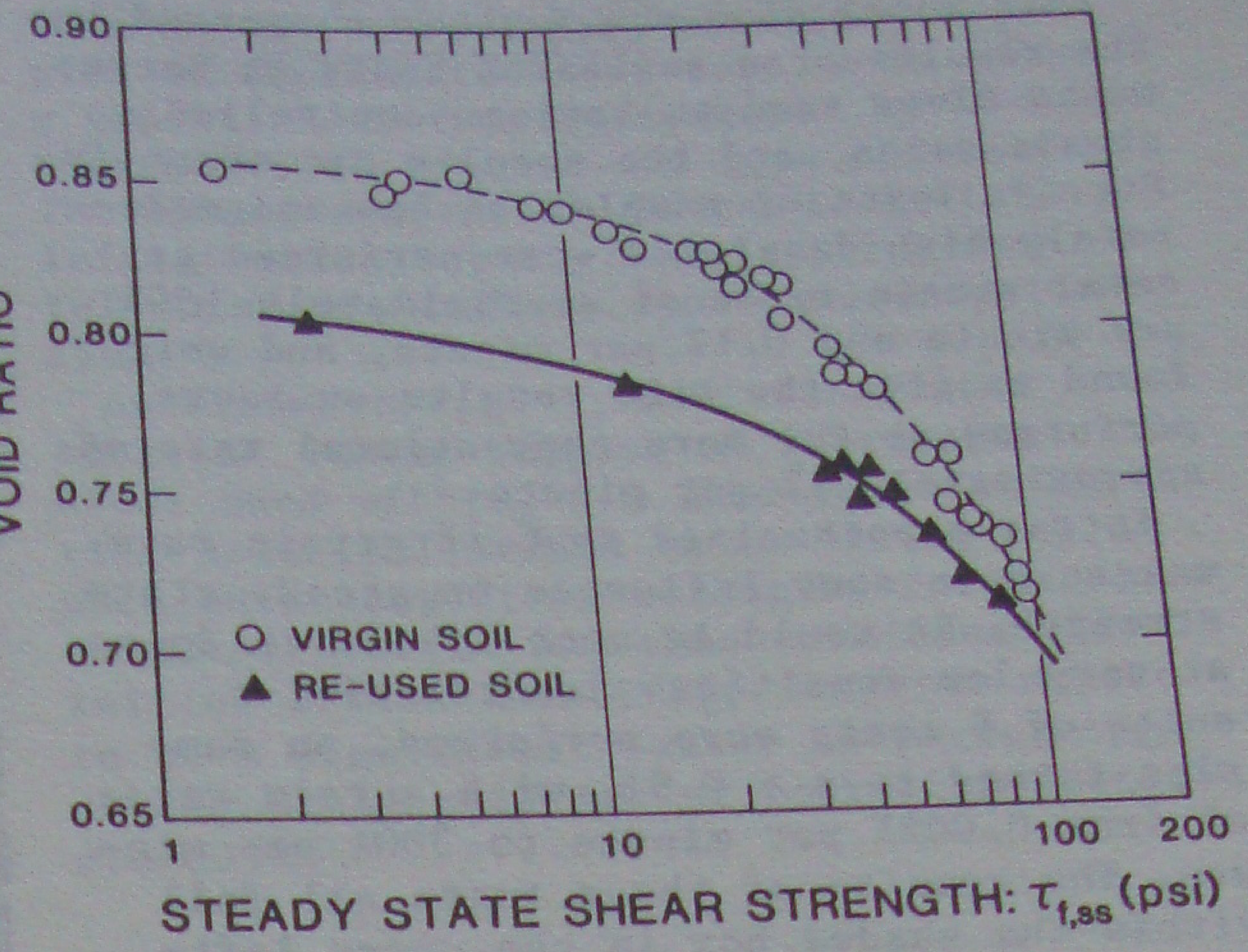

Figure 4. Steady state strengths of virgin vs. re-used samples of Sacramento River sand.

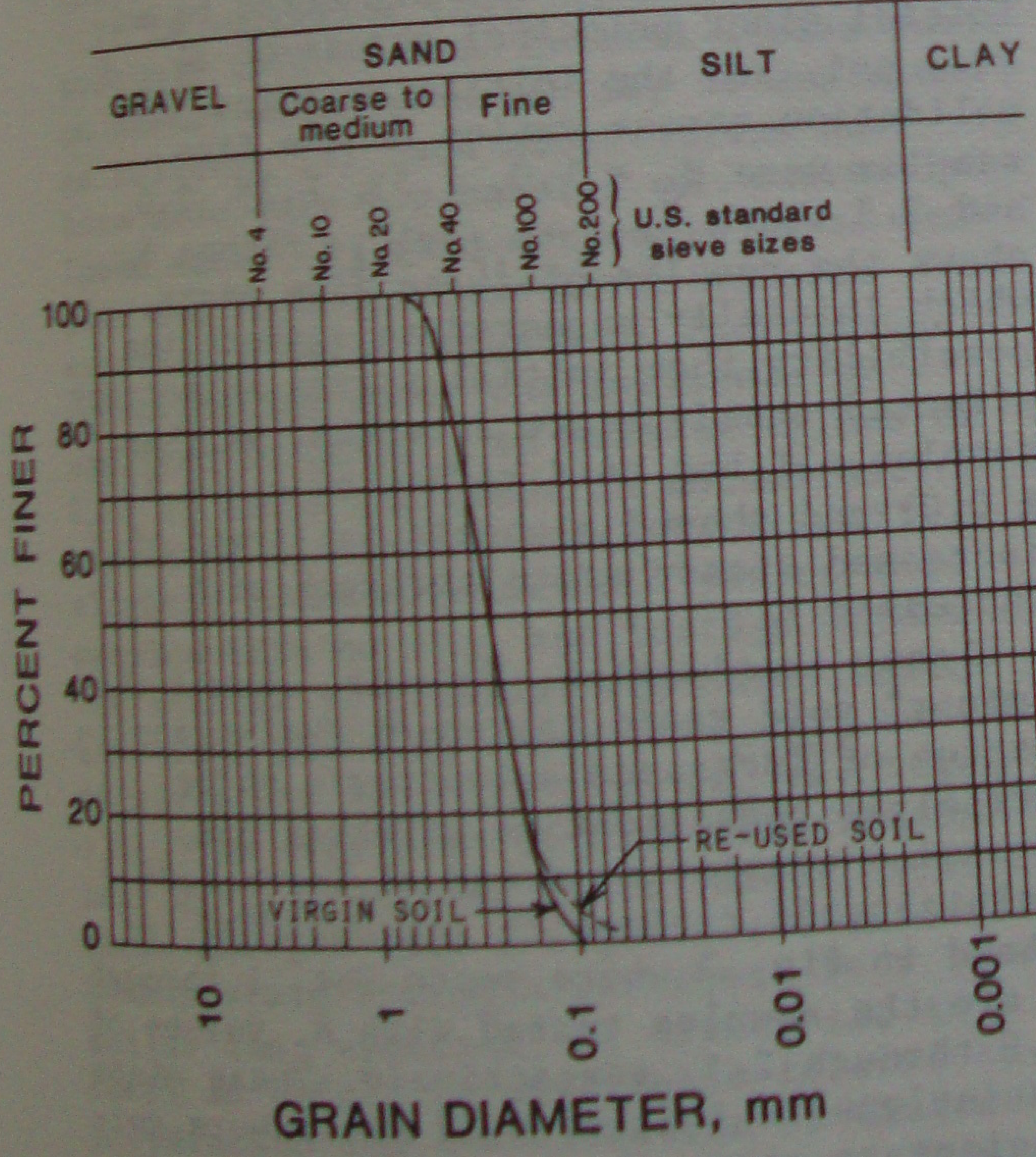

Figure 3. Gradation curves of virgin and re-used Sacramento River sand.

4 EFFECTS OF STRAIN RATE

The principal difference between the definitions of "steady state" conditions (Poulos 1981) and "critical state" conditions (Roscoe et al. 1958; Schofield & Wroth 1968) lies in the steady state specification of constant velocity. Sladen et al. (1985) suggest that this distinction is significant only for clays, whereas the terms steady state and critical state may be used interchangeably for sandy soils, though they observed that little data is available to confirm this. They performed a number of load-controlled and strain-controlled IC-U triaxial tests on fine to medium sands, and found tests of both types yielded the same steady state results. Castro et al. (1982) also performed a few stress- and strain-controlled tests on sands with similar results. Schimming et al. (1966) and Hungr and Morgenstern (1984) also presented data suggesting that the steady state strength of sands is independent of strain rate.

This conclusion was further supported by the results of a series of tests on Sacramento River sand at various controlled strain rates, and the results are shown in Fig. 5. Tests of samples at low and moderately high densities were performed at axial strain rates of approximately 150% per minute and 0.1% per minute, and were found to give the same results as tests performed at the more conventional rate of approximately 1% per minute.

It was hypothesized that if strain rate was to have some influence on steady state strength, it would be most likely to do so at very low densities. Accordingly, a suite of 8 tests were performed, on samples formed to e ≈ 0.85, with strain rates of from 0.002% per minute to 300% per minute. The results of these tests all fall within the shaded box in the upper left-hand corner of Fig. 5, and it was concluded that strain rate had no discernible effect on these results.

5 EFFECTS OF INITIAL STRESS ANISOTROPY

Unpublished research at the University of California at Berkeley involving AC-U triaxial testing of extremely loose sands suggested the possibility that the degree of initial sample stress anisotropy might influence the steady state conditions for very loose sandy soils. Accordingly, AC-U triaxial tests were performed on tamped samples of Sacramento River sand to investigate this possibility.

Five "extremely loose" samples were prepared, with void ratios of approximately e = 0.85, and were consolidated to initial

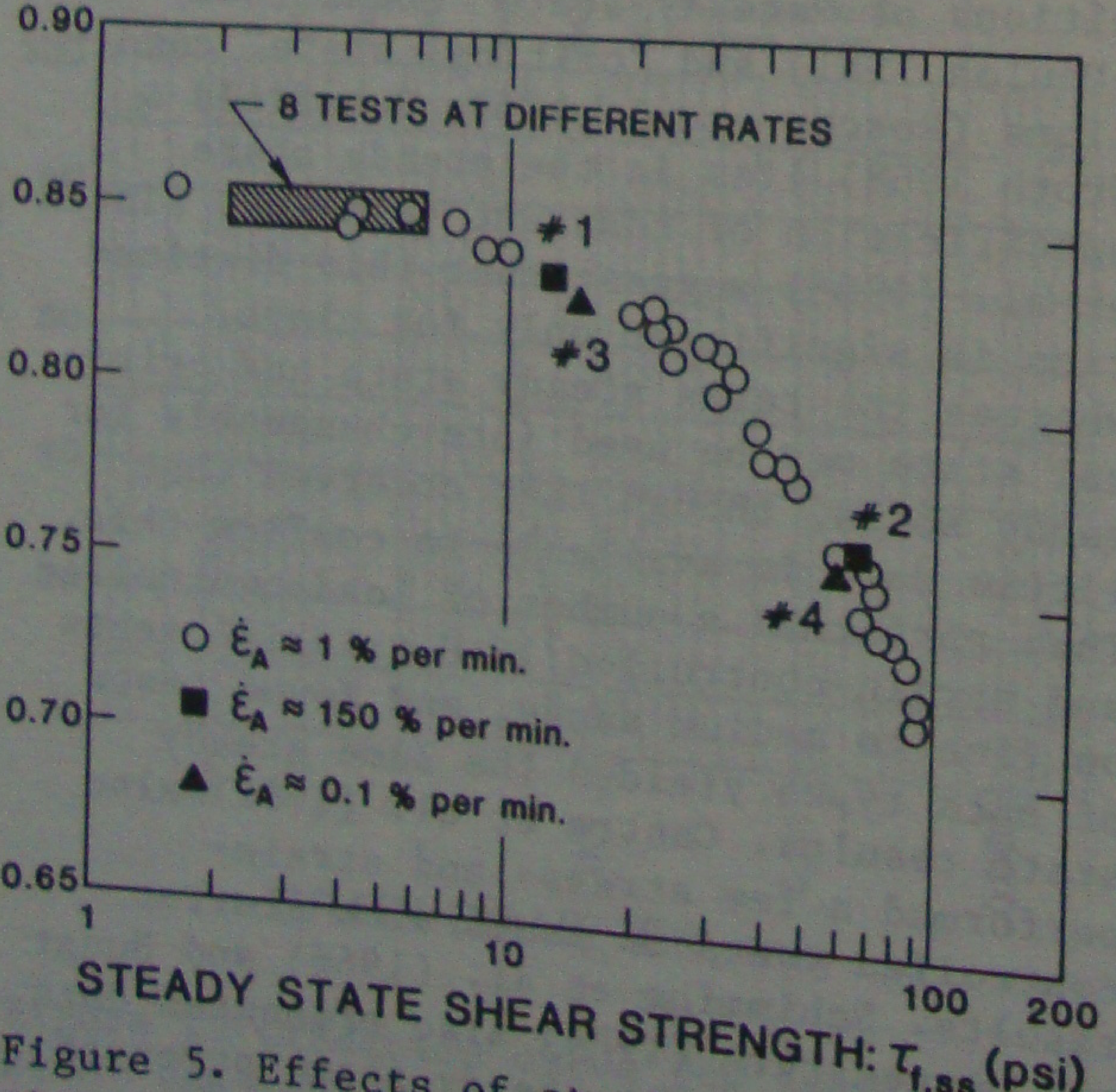

Figure 5. Effects of strain rate on steady state strength of Sacramento River sand.

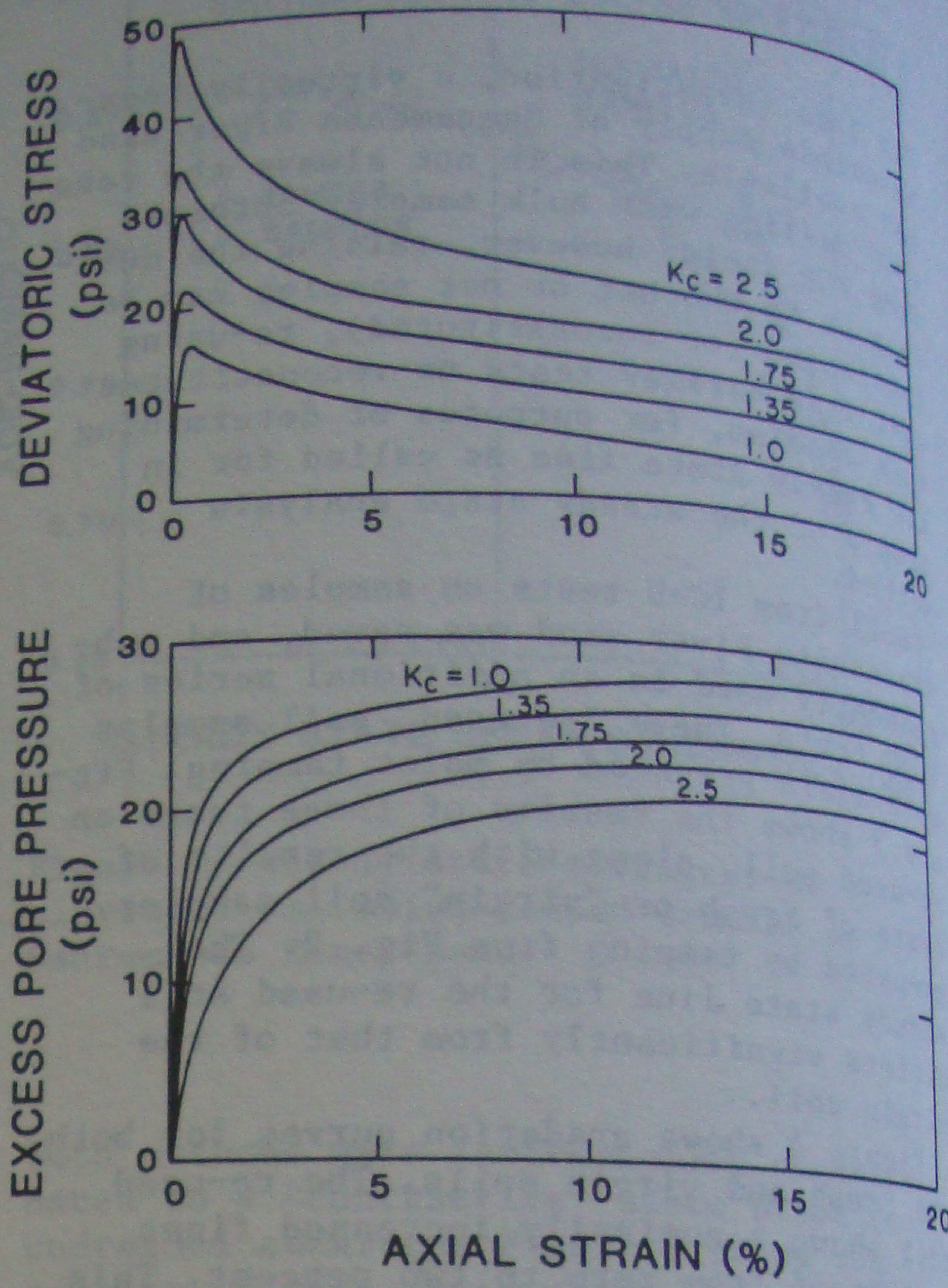

Figure 6. "Apparent" influence of initial stress anisotropy on steady state strength of Sacramento River sand.

effective confining stress conditions with varying degrees of initial stress anisotropy. All samples were subjected to an initial minor principal stress of $\sigma'_{3,i}$ = 29.4 psi, and the initial principal consolidation stress ratios of the five samples were K_c = 1.0, 1.35, 1.75, 2.0, and 2.5 where $K_c = \sigma'_{1,i}/\sigma'_{3,i}$. Figure 6 shows the results of these five tests, which initially appeared to support the possibility that initial stress anisotropy affected steady state strength, as these samples all had void ratios of e = 0.85 ± 0.01 and showed a consistent trend of increased steady state strength with increased K_c.

Closer examination, however, showed that "minor" void ratio increases during application of increased anisotropic major principal consolidation stresses had caused a consistent trend of increased initial void ratio with increased K_c, as shown in Fig. 7 where tests Nos. 1 through 5 are the samples tested with K_c values of 1.0 through 2.5, respectively. These minor variations in initial void ratio were sufficient to cause the significant and consistent variational trend in steady state strength values observed because at these

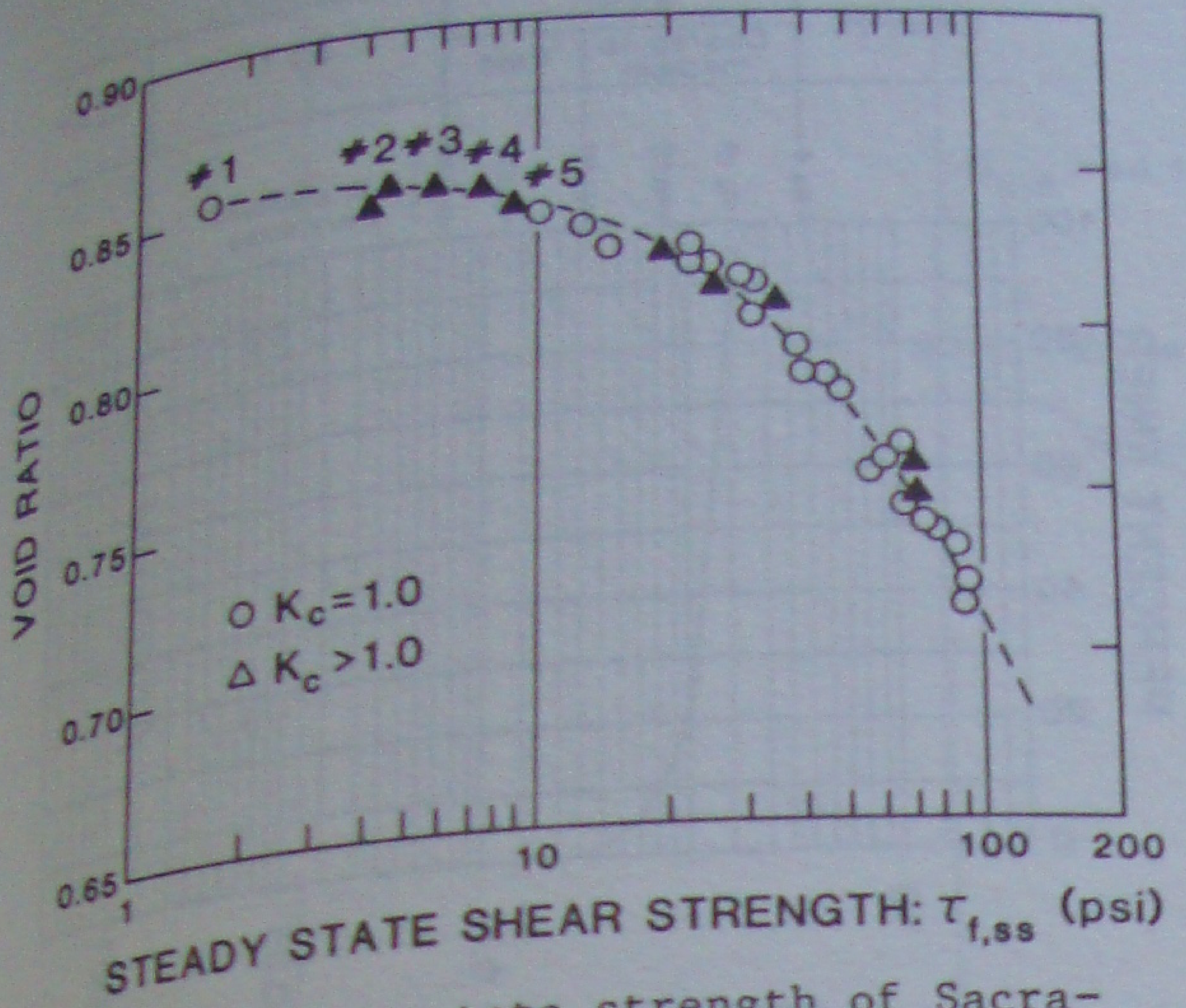

Figure 7. Steady state strength of Sacramento River sand vs. void ratio based on IC-U and AC-U tests.

high void ratios (corresponding to "extremely" loose conditions), the slope of the steady state strength vs. void ratio relationship becomes very flat.

There is, in fact, a limiting value of critical maximum void ratio ($e_{c,max}$) above which Sacramento River sand samples collapse completely and exhibit no measureable steady state strength under undrained loading. The value of $e_{c,max}$ for Sacramento River sand is approximately 0.86.

It is suggested that observations of possible influence of initial stress ratio on steady state strength may be explained as follows: (1) Such observations have been made only for samples of "extremely" loose sand. (2) When extremely loose samples are formed, these loose samples experience small amounts of incremental consolidation under the influence of application of $\sigma'_{1,i}$ to achieve values of $K_c > 1$; these increases in consolidation resulting in samples of (statistically) slightly lower initial void ratios than samples with no $\sigma'_{1,i}$ application beyond the initial $\sigma'_{3,i}$ value. (3) Sandy soils have some limiting value of $e_{c,max}$ and the steady state strength line (e_c vs. $\log_{10} \tau_{f,ss}$) is very flat at high void ratios near $e_{c,max}$, so that these "minor" void ratio changes have a significant effect on steady state strength for these very loose soils. The "apparent" K_c effect is, in fact, an effect on void ratio related to preparation and consolidation of extremely loose samples; K_c itself has no influence on the e vs. $\tau_{f,ss}$ relationship. As a practical matter, it was not possible to form and anisotropically consolidate samples of Sacramento River sand to as loose a condition as that represented by IC-U test No. 1 in Figs. 6 and 7 as a result of the slight densification due to drained initial anisotropic stress application following preliminary isotropic consolidation.

As shown in Fig. 7, additional AC-U tests of denser samples of Sacramento River sand with $K_c > 1$ show that the initial principal stress ratio has no influence on steady state strengths for denser sands either. It was thus concluded that initial stress anisotropy does not appear to be a significant factor in performing steady state analyses of post-liquefaction stability.

6 EFFECTS OF MEMBRANE PENETRATION

Since the early work of Newland and Allely (1959) it has been known that during undrained testing of saturated soils, the variation of the penetration of the rubber membrane used to confine a triaxial sample into the peripheral sample voids (membrane compliance) due to variation in σ'_3 results in sample volume changes which violate the assumed "undrained" testing conditions. The potential significance of this is a function of sample grain size distribution and sample scale. The use of larger samples which have a larger ratio of total volume to peripheral surface area reduces the impact of membrane compliance on pore pressures. Typical triaxial tests using samples two to three inches in diameter provide good results, with minimal influence of membrane compliance effects, for silts and fine sands (Seed & Anwar 1986). In most conventional triaxial test apparatus, however, membrane compliance effects can be significant for medium and coarse sands.

A number of approaches have been employed in attempting to mitigate the effects of membrane compliance during undrained testing (e.g., Moussa 1973; Pickering 1973; Kiekbusch & Shuppener 1977; Lade & Hernandez 1977; Raju & Venkatamura 1980; Wong 1983; Seed & Anwar 1986). Many of the techniques employed have been at least partially successful in eliminating membrane penetration effects, but none prior to 1986 had been fully successful without introducing new testing problems. There are, in addition, a number of theoretial techniques available for post-testing correction of undrained test results to account for the effects of membrane compliance (e.g., Martin et al.

1978; Raju & Venkatamura 1980; Vaid & Negussey 1982; Baldi & Nova 1984). In the absence of an effective method of mitigating membrane compliance effects during testing, however, it has not previously been possible to perform tests on identical samples (with and without mitigation of compliance effects) and so it has not been possible to verify the accuracy of these theoretical post-testing correction methods and/or their ability to predict compliance effects on steady state strength evaluation.

A recently developed method for continuous computer-controlled compensation for membrane compliance-induced volumetric error appears to represent an accurate and reliable method for mitigation of compliance effects during undrained testing (Seed & Anwar 1986). IC-U triaxial tests, performed with and without implementation of this compliance compensation method, serve to illustrate the potential influence of membrane compliance on steady state strength evaluation.

The compliance mitigation methodology developed represents a modification of an approach originally proposed by Ramana and Raju (1981), and consists of (a) first pre-determining the volumetric magnitude of membrane compliance for a given soil of given density as a function of variations in minor principal effective stress (σ_3'), and (b) using a computer-controlled process to continuously inject or remove water from the sample during "undrained" testing in order to exactly offset the volumetric error induced by membrane compliance (Seed & Anwar 1986). It has been demonstrated that volumetric compliance is a direct and repeatable function of variation in σ_3', and that this relationship is not affected by soil fabric changes during testing, so that monitoring of variation of σ_3' represents a suitable basis for control of injection/removal compensation during testing.

The soil selected for illustration of the potential influence of membrane compliance effects on steady state strength evaluation was Monterey 16 sand, a uniformly graded, clean medium beach sand, primarily quartzitic, with subangular to subrounded particles. Figure 8 presents a gradation curve for this material. Over the range of densities tested, the normalized unit volumetric membrane compliance (S; compliance-induced volume change per unit membrane area per $\log_{10}$ change in σ_3') is approximately five to six times that of the finer Sacramento River sand, so that this medium sand represents a soil with potentially significant influence of compliance on steady state conditions,

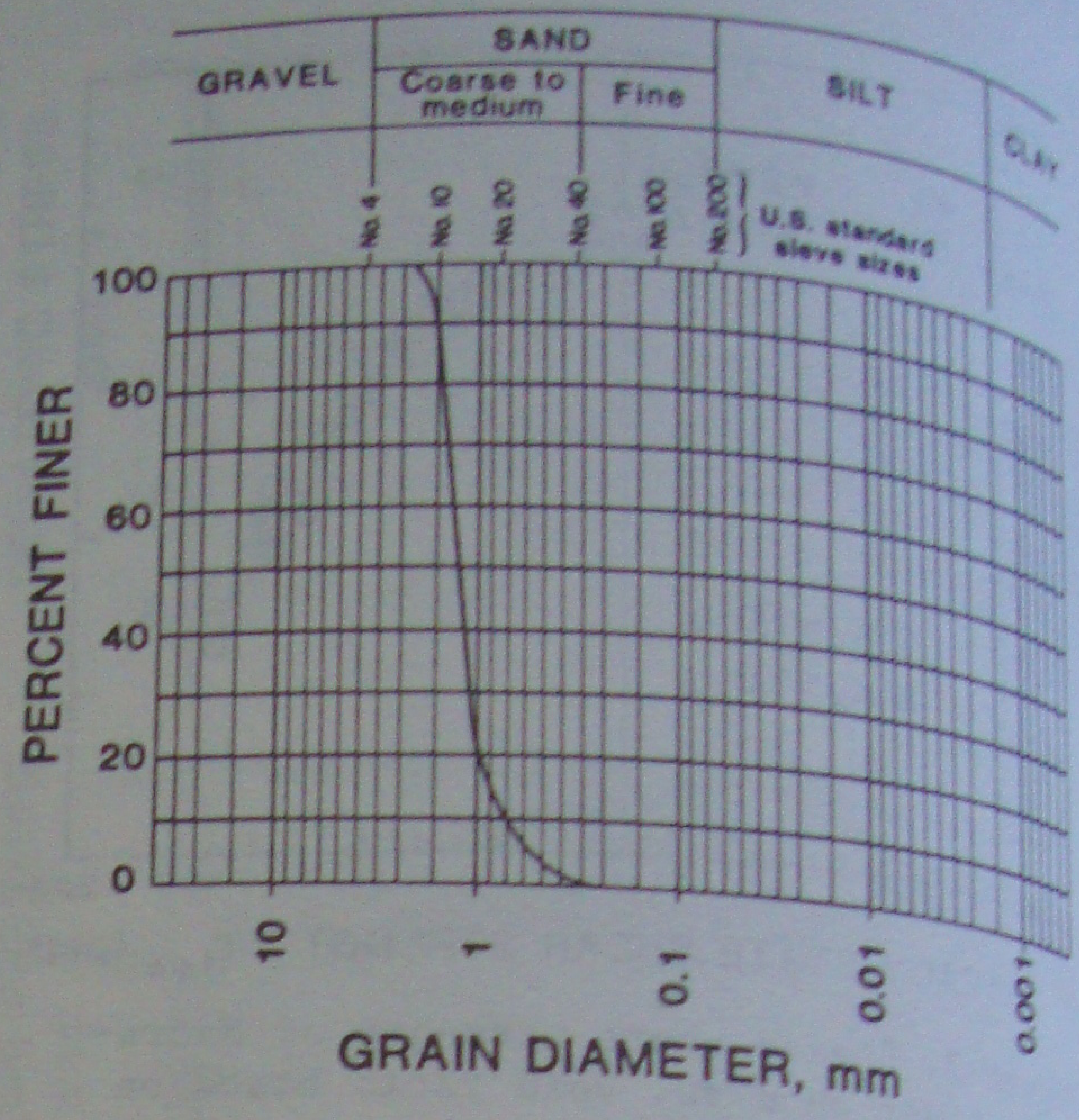

Figure 8. Gradation curve for Monterey 16 sand.

while the fine sand is not significantly affected by membrane compliance effects.

Five pairs of IC-U triaxial tests were performed on 2.8-in. diameter samples of Monterey 16 sand, one pair each at void ratios of e ≈ 0.664, 0.700, 0.724, 0.731, and 0.743. One test in each pair was a "conventional" undrained test, and in the second test of each pair computer-controlled compliance compensation was employed. All samples were isotropically consolidated to $\sigma_{3,i}' = 44.1$ psi prior to undrained testing.

Figure 9 shows the results of these IC-U tests with the "conventional" tests interpreted as having represented undrained loading tests. This figure shows both the initial ($\sigma_{3,i}'$) and final or critical state effective confining stress ($\sigma_{3,c}'$) for each test. The ratio of steady state strength vs. σ_3' is constant as the residual friction angle ϕ_r' does not vary over the range of σ_3' tested.

As shown in this figure, the tests performed without mitigation of compliance effects lead to a significantly different evaluation of steady state or critical state conditions than do the tests performed with mitigation. This is because compliance-induced volumetric changes result in underestimation of both positive and negative pore pressure development. Figures 10 and 11 show stress-strain and pore-pressure-strain curves for two of these pairs of tests: one on loose or

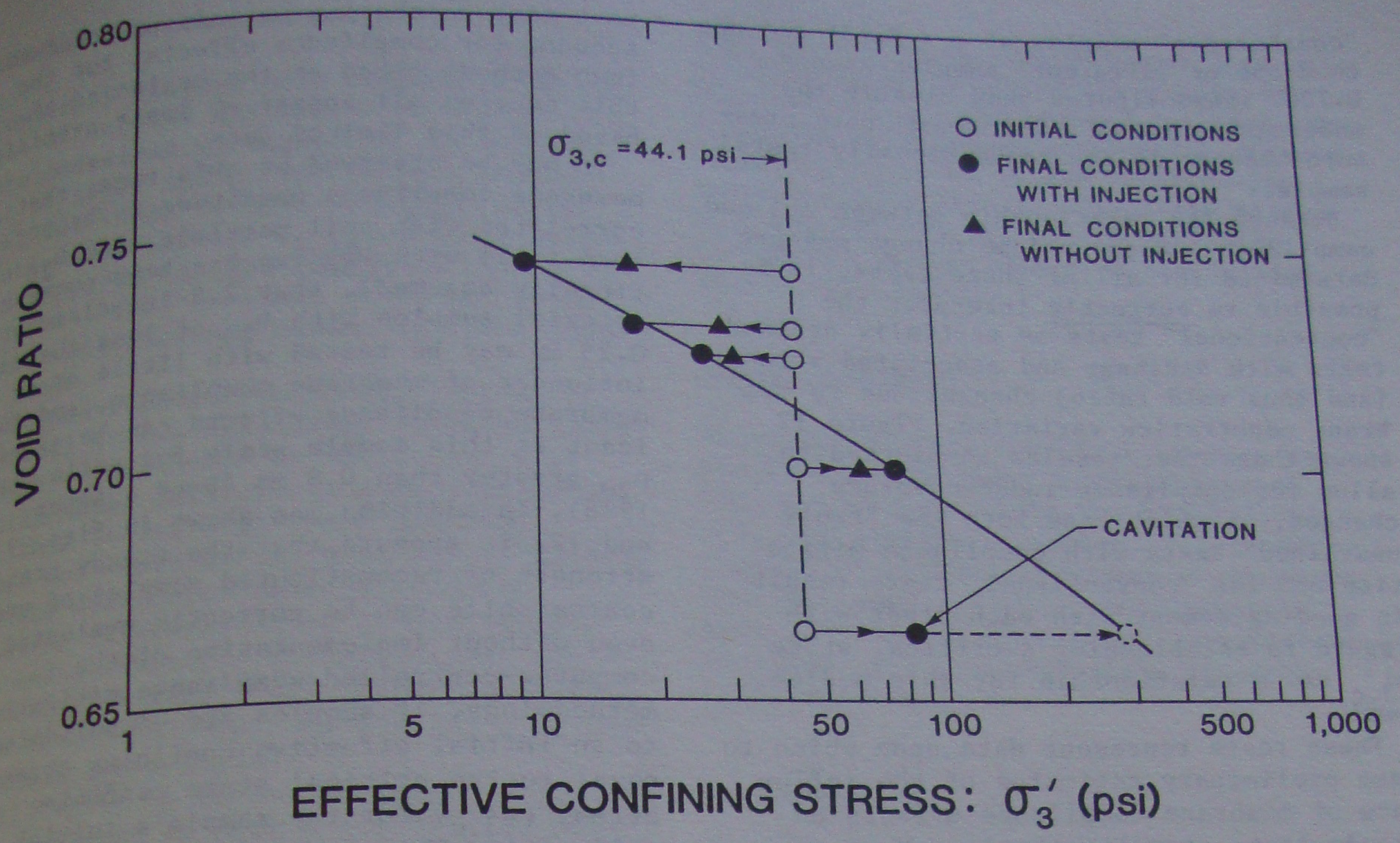

Figure 9. Steady state strength of Monterey 16 sand tested with and without mitigation of membrane compliance effects.

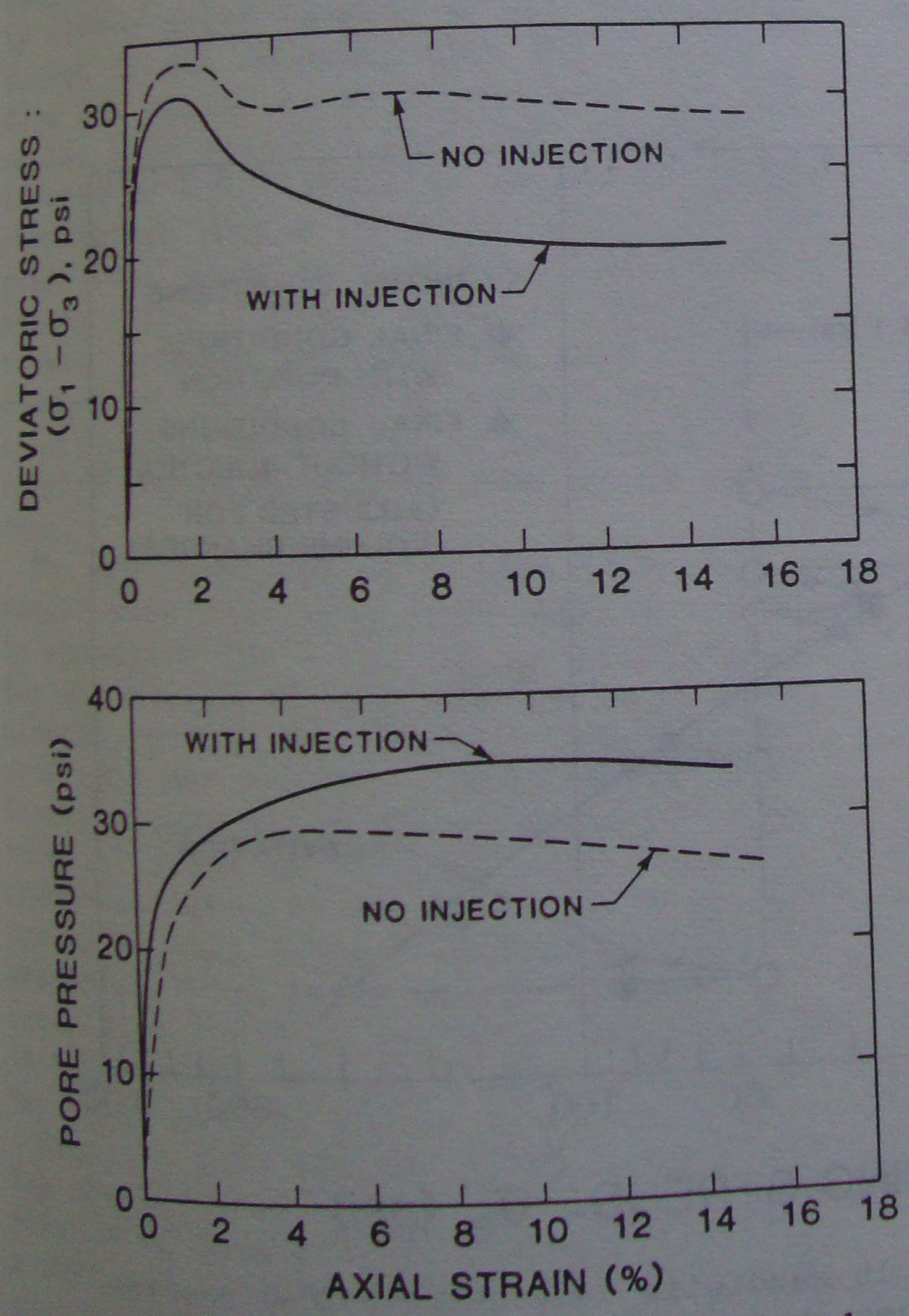

Figure 10. IC-U tests of Monterey 16 sand at e = 0.731.

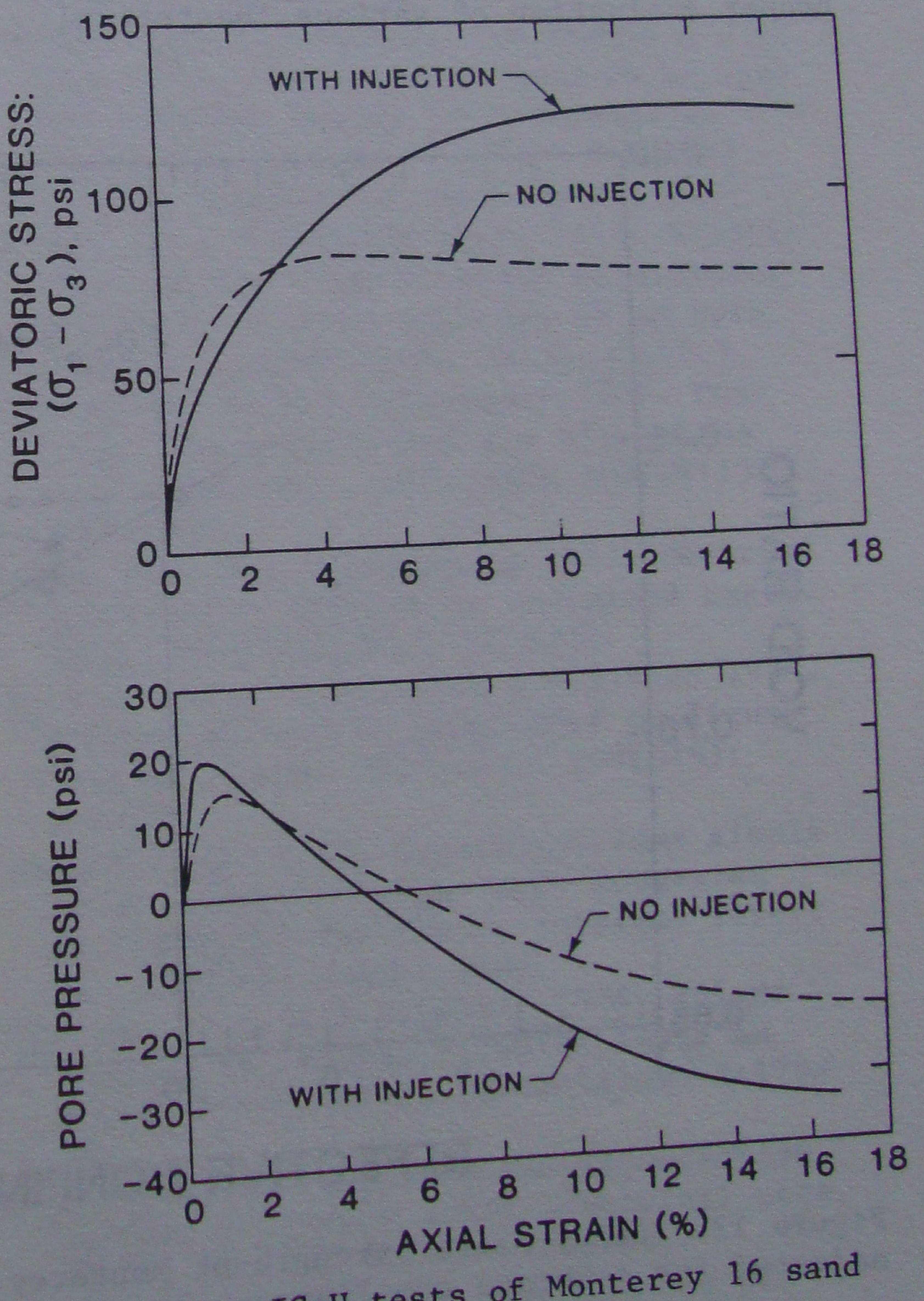

Figure 11. IC-U tests of Monterey 16 sand at e = 0.700.

"contractive" samples at e ≈ 0.731 and one on dense or "dilatent" samples at e ≈ 0.700. These figures show clearly the underestimation of "undrained" pore pressure changes in the conventionally tested samples.

Because the relationship between $\Delta\sigma_3'$ and compliance-induced volume change was predetermined for all of these tests, it is possible to correctly interpret the "conventional" tests as partially drained tests with drainage and associated volume (and thus void ratio) changes due to membrane penetration variation. Figure 12 shows these test results re-plotted to allow for compliance-induced volume changes, in which case both the "truly undrained" tests with compliance mitigation and the "conventional" tests result in good agreement with each other with regard to establishing a critical state $\sigma_{3,c}'$ vs. e relationship for this medium sand.

These tests represent data upon which to base preliminary estimates of the influence of membrane compliance effects on steady state strength testing. More comparative tests of this type are necessary to provide an adequate data base for proper evaluation of various theoretical post-test correction methods proposed to account for compliance effects, but the four methods cited at the beginning of this section all appear at least promising based on this limited data.

It may be observed at this time, that membrane compliance magnitude is better correlated with soil particle size D_{20} than it is with D_{50} (as has been conventionally assumed), that 2.8-in. diameter triaxial samples with D_{20} of less than 0.25 mm may be tested with little or no influence of membrane compliance, and that membrane compliance effects can be significant at this sample scale for soils with D_{20} greater than 0.8 mm (Seed & Anwar 1986). In addition, as shown in Figs. 9 and 12, it appears that the steady state strength of reconstituted samples of even coarse soils can be correctly evaluated, even without implementation of the computer-controlled compliance mitigation methodology, if samples are consolidated to an initial effective confining stress equal to the critical state confining stress ($\sigma_{3,c}'$) for the sample's initial void ratio. This would, however, require tedious iterative testing in practice.

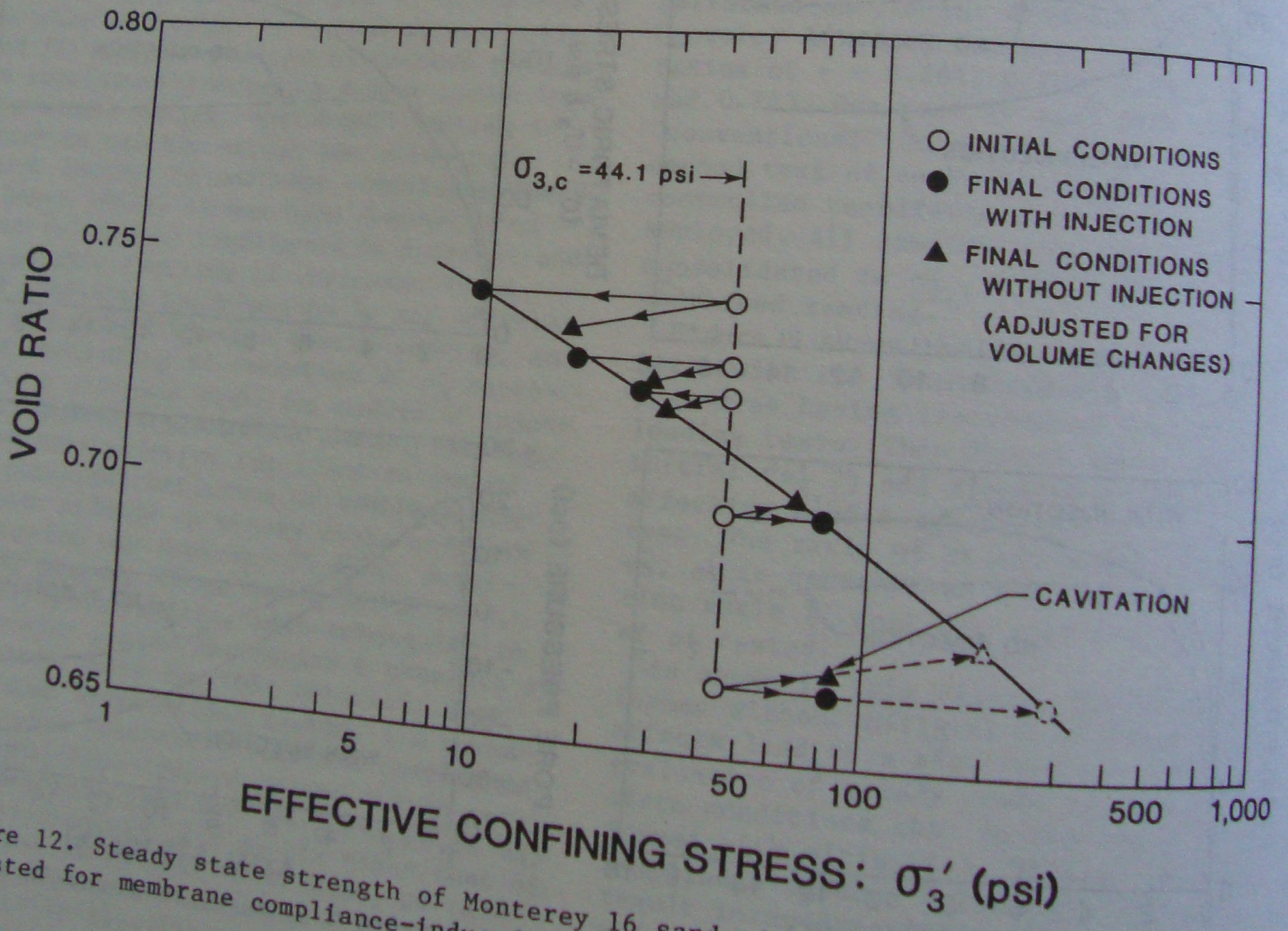

Figure 12. Steady state strength of Monterey 16 sand with conventional IC-U results adjusted for membrane compliance-induced volume changes.

7 SUMMARY AND CONCLUSIONS

A number of factors were considered which might affect the assessment of steady state undrained strength ($\tau_{f,ss}$) assessment of sandy soils. One factor, strain rate, was found to have no effect on the relationship between $\tau_{f,ss}$ and void ratio. This supports the contention of Sladen et al. (1985) that critical state and steady state conditions are essentially the same for sands.

A second factor considered, initial principal stress anisotropy was also found to have no influence on the e vs. $\tau_{f,ss}$ relationship. Earlier evidence suggesting that this factor might have had some effect was explained as possibly resulting from the difficulty associated with achieving extremely loose sample fabrication and anisotropic consolidation, and the fact that very small changes in void ratio result in large changes in $\tau_{f,ss}$ for extremely loose soils.

Soil fabric or method of sample preparation was found to have a potentially significant effect on the relationship between e and $\tau_{f,ss}$, as shown by tests on samples of Sacrmento River sand prepared by either moist tamping or dry pluviation, though it was observed that this factor does not appear to be significant for all soils. This suggests that the potential effects of soil fabric or method of sample preparation should be investigated for any given soil prior to basing correction of steady state strength tests of "undisturbed" samples for sampling void ratio changes on "parallelism" with e vs. $\tau_{f,ss}$ relationships developed by testing bulk samples reconstituted by some given procedure.

The ramifications of re-use of bulk sample material saved from earlier tests to establish the e vs. $\tau_{f,ss}$ relationship for reconstituted bulk samples were considered, and it was demonstrated that re-used soil may exhibit significantly different steady state characteristics than those of fresh or "virgin" soil. This change in behavior was shown to be accompanied by only nominal changes in soil gradation which might not be reliably detectable based on gradation analyses. It is recommended that bulk sample material not be re-used in steady state testing.

Finally, data was presented demonstrating the potntial effects of membrane compliance on steady state strength tests of a uniformly graded medium sand. It is suggested that (a) membrane compliance effects are small for undrained testing of 2.8-in. diameter triaxial samples of soils with $D_{20} < 0.25$ mm, (b) these effects can be significant in undrained testing of soils with $D_{20} > 0.8$ mm, and (c) that the influence of membrane penetration effects is also a function of the relationship between the initial consolidation stress $\sigma'_{3,i}$ and the critical state confining stress $\sigma'_{3,c}$ associated with the initial sample void ratio.

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