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MEASUREMENT AND ELIMINATION OF MEMBRANE COMPLIANCE EFFECTS IN UNDRAINED TRIAXIAL TESTING

by

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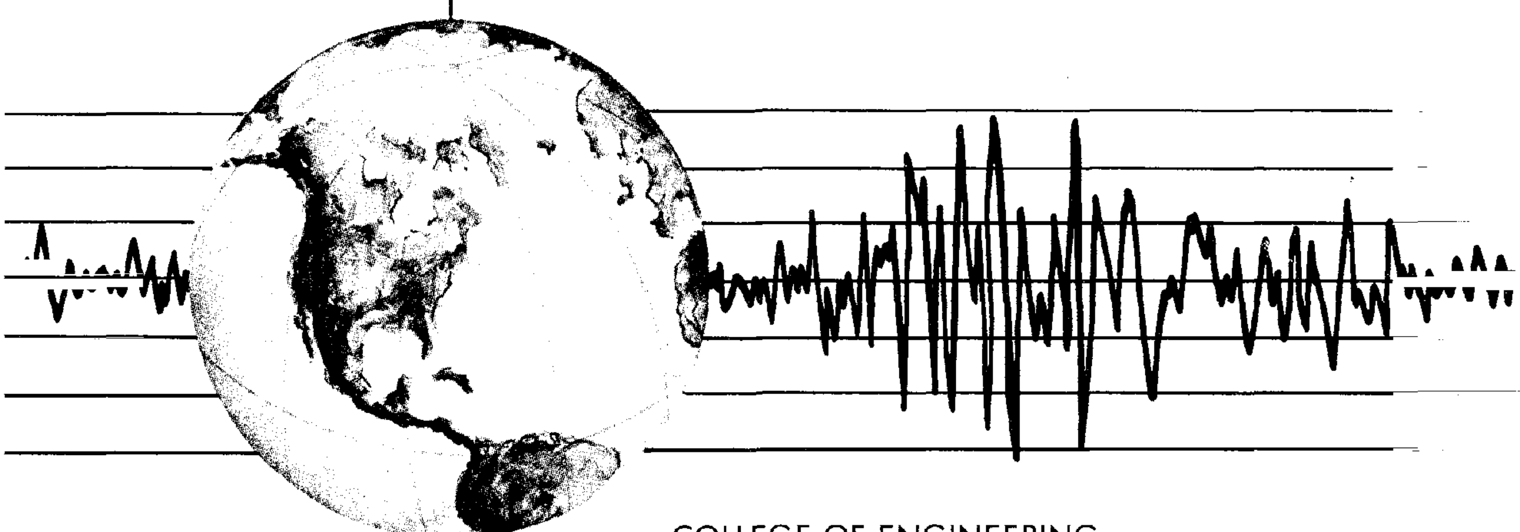
Report to the National Science Foundation



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An improved testing procedure for the elimination of compliance effects during testing, recently developed for conventional scale (2.8-inch diameter) samples of sand, was verified by comparison with results of large-scale (12-inch diameter) tests of similar materials, for which compliance effects were negligible. The technique was then further developed and implemented for large-scale testing of gravelly soils.

The compliance mitigation procedure used in these studies involved predetermining the magnitude of volumetric compliance as a function of effective confining stress and soil parameters, and then using computer-controlled injection or removal of water to continuously eliminate membrane compliance effects. Monotonic and cyclic load tests were performed on uniformly-graded gravel with and without implementation of the computer-controlled compliance mitigation system. These tests are the first performed on gravels for which membrane compliance effects have been completely mitigated, and the results support the hypothesis that such soils are much more susceptible to liquefaction than had previously been thought.

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by

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REPORT NO. UCB/EERC-89/10

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ABSTRACT

Undrained loading tests are widely used to investigate the susceptibility of soils to liquefaction. Changes in effective confining stress cause variations in the degrees of penetration of the confining membrane into peripheral sample voids. This phenomenon, known as membrane compliance, invalidates the fundamental assumption of constant volume during undrained testing. Membrane compliance can have a serious detrimental effect on the accuracy and validity of such tests, particularly for medium to coarse sands and gravels.

Historically, gravelly soils have typically been assumed to be "safe" from liquefaction failures. However, recent improvements in understanding of the effects of membrane compliance on triaxial test results, along with documentation of a number of field cases in which gravels have liquefied, has led to the realization that such soils can liquefy. As a result, the importance of developing a method to correctly assess the dynamic strength of these coarser soils has become apparent. Previous attempts at mitigating membrane compliance effects during testing, or making post-test corrections to conventional undrained test results, have not been fully successful in providing verifiably correct test results. An improved testing procedure for the elimination of compliance effects during testing, recently developed for conventional scale (2.8-inch diameter) samples of sand, was verified by comparison to results of large-scale (12-inch diameter) tests of similar materials, for which compliance effects were negligible. The technique was then further developed and implemented for large-scale testing of gravelly soils.

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or removal of water to continuously eliminate membrane compliance effects. Monotonic and cyclic load tests were performed on uniformly-graded gravel with and without implementation of the computer-controlled compliance mitigation system. These tests are the first performed on gravels for which membrane compliance effects have been completely mitigated, and the results support the hypothesis that such soils are much more susceptible to liquefaction than had previously been thought. In addition, these tests, performed with and without mitigation, also allow a check of the validity and accuracy of theoretical post-test corrections suggested for gravelly soils.

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NOTATIONS

A_m	=	Membrane surface area.
$\overline{\text{AC-U}}$	=	Anisotropically consolidated-undrained test with pore pressure measurements.
D	=	Sample diameter.
D_R	=	Relative density (%).
D_{20}	=	The soil particle size such that 20% (by dry weight) of the soil is finer.
D_{50}	=	The mean soil particle size.
d_g	=	Grain size (mm).
E	=	Young's modulus.
E_m	=	Young's modulus of confining membrane.
H	=	Sample height.
$\overline{\text{IC-U}}$	=	Isotropically consolidated-undrained test with pore pressure measurements.
K_c	=	$\frac{\sigma'_{1,i}}{\sigma'_{3,i}}$
K_o	=	Coefficient of lateral earth pressure at-rest.
P	=	Applied vertical load.
S	=	δV_m per logarithmic cycle change in σ'_3 (δV_m per order of magnitude change in σ'_3).
t_m	=	Membrane thickness.
u	=	Pore pressure.
V_o	=	Initial volume; or initial sample volume.
V_T	=	Volume of a soil sample.

Δu	=	Change in pore pressure.
ΔV	=	Change in volume.
ΔV_c	=	Volume change due to compression of the soil (sample skeletal compression).
ΔV_m	=	Volume change due to membrane compliance.
ΔV_t	=	Total volume change.
δV_m	=	Membrane compliance induced volume change unit area of membrane surface.
ϵ_a	=	Axial strain.
ϵ_r	=	Radial strain.
ϵ_v	=	Volumetric strain.
$\epsilon_{v,m}$	=	Volumetric strain due to membrane compliance.
$\epsilon_{v,s}$	=	Volumetric strain of a soil sample.
σ'_1	=	Major principal effective stress.
$\sigma'_{1,i}$	=	Initial major principal effective stress.
σ'_3	=	Effective confining stress; effective minor principal stress.
$\sigma'_{3,i}$	=	Initial effective confining stress.

CHAPTER ONE

INTRODUCTION - IDENTIFICATION OF THE PROBLEM

1.1 Evaluation of the Liquefaction Potential of Soils

Liquefaction can be one of the most dramatic and devastating types of soil failures that can occur due to earthquake shaking. Since the mechanics of this type of soil failure were first understood in the early 1960's, there has been a greatly increased interest in evaluating the resistance of soils comprising particular sites and earth structures to liquefaction as a result of dynamic loadings. Over the last 30 years there has been much research and investigation into developing accurate methods to estimate or evaluate the in situ dynamic strength and liquefaction resistance of soils.

Two general types of testing methods are used to evaluate the liquefaction potential of soils. One of these is in-situ testing, including most prominently field penetration testing wherein measurements are taken of the resistance of the soil to penetration by standardized testing equipment. The standard penetration test (SPT) has become an industry standard to evaluate the in situ liquefaction resistance of sandy and silty soils. Samples can be obtained at intervals during the "drilling" of each test hole in order to give a cross check of the penetration results and can be used to perform further investigations of the soils found at different depths. It has been demonstrated through numerous investigations and correlations that SPT test results can give reasonably accurate evaluations of liquefaction potential for these "finer" grained soils. More recently, the static cone penetration test (CPT) has become an increasingly attractive alternative to the SPT as it has a number of advantages including: (1) a rapid and continuous log of the soil resistance to penetration by the cone, (2) measurement of both point resistance (normal load) and shear resistance along the side shaft of the instrument, and (3) improved standardization and repeatability relative to the SPT. These advantages are offset, however, by the smaller field case study data base upon which to base empirical correlations between CPT penetration resistance and in situ liquefaction resistance.

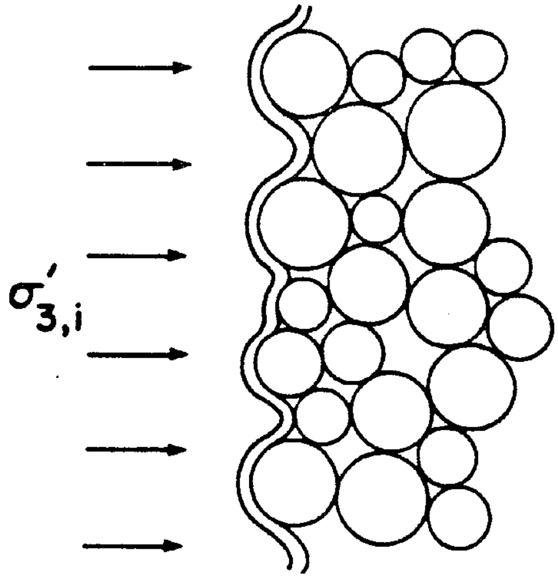
Unfortunately, while these field penetration tests provide reasonably accurate and reliable evaluations of liquefaction potential for most silty and sandy soils, they cannot be used to reliably perform the same kind of strength evaluations for coarser gravelly materials. In coarser soils, the particle sizes are either too large for the standard types of equipment to penetrate, or else unreasonably high and very unrepresentative results are obtained from tests of these soils. A recent investigation (Harder and Seed, 1986) suggests that a large-scale penetration test may be possible using the Becker Hammer for evaluating the liquefaction potential of coarse gravelly soils. This type of application has great promise for the future if a significant database can be generated with which to correlate large-scale penetration test results.

The second type of method used to evaluate soil liquefaction potential is laboratory testing of "representative" samples. This approach is not generally viable for evaluation of the in situ undrained loading characteristics of sandy soils as a result of "sampling disturbance" effects, but evidence available to date suggests that reasonably representative results might be obtained for coarser gravelly soils if the deleterious effects of membrane compliance can be eliminated. Several types of undrained loading tests (e.g. triaxial, simple shear, and torsional shear) are used to assess the susceptibility of soils to liquefaction. The implicit assumption of any of these undrained tests is that no sample volume change occurs during undrained loading except for a nominal compression of pore water resulting from increased pore pressure. A phenomenon known as membrane compliance, described in the next section, invalidates this important assumption for testing of coarse, granular soils.

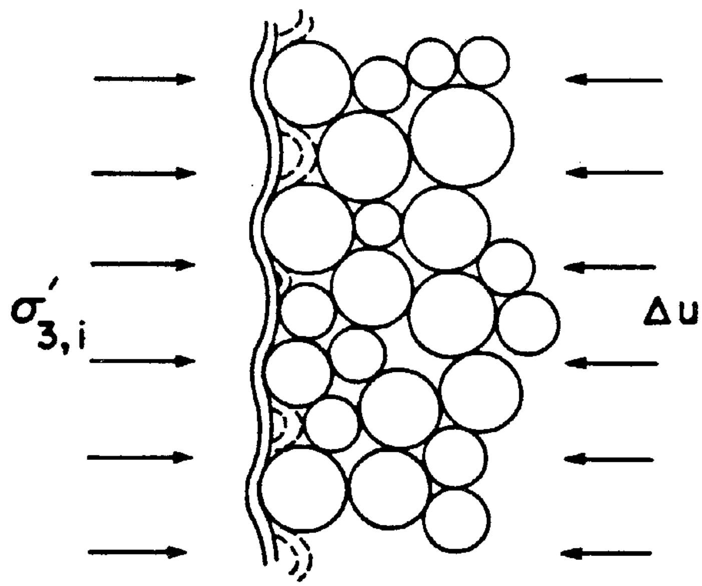
1.2 Effects of Membrane Compliance

When loading stresses are applied to relatively loose samples under undrained conditions, there is a resulting progressive increase in pore pressure within the sample, which reduces the effective confining stress and weakens the sample until strain amplitudes become large signaling the onset of liquefaction or cyclic mobility. In the laboratory, saturated soil samples are typically confined by a thin rubber membrane and subjected to cyclic or monotonic loading under undrained conditions. As a confining stress is applied to a sample, the confining membrane conforms to the shape of the sample surface and penetrates any perpheral surface voids in the sample. This effect is termed "membrane penetration". During undrained loading of relatively loose samples, the increase of pore pressure within the sample tends to push the membrane out of the interstitial voids at the sample edges. The changes in degree of penetration that develop due to changes in confining stress during undrained testing is called "membrane compliance".

Figure 1.1(a) illustrates the phenomenon of membrane penetration acting on a sample under an initial confining stress. Figure 1.1(b) schematically illustrates the effects of an increase in sample pore pressure resulting in a decrease in sample effective confining stress, which in turn results in a decrease in the degree of membrane penetration into peripheral sample voids. Due to compliance of the membrane with changes in effective confining stresses, the assumption of constant volume in an undrained test is invalidated. This change in shape of the confining



a) Initial Conditions



b) Conditions After Pore Pressure Increase

Figure 1.1: <u>Schematic Representation of Membrane Penetration and Membrane Compliance</u>

membrane that can introduce significant testing error in undrained tests. Membrane compliance allows pore water to migrate from within the sample center towards the vacated edge voids. The result is that internal pore pressures are relaxed, which then gives the sample an incorrect, and in most critical cases a higher (fictitious) strength.

The degree to which membrane compliance may affect the results of an undrained test is a function of the soil grain size and overall geometry of the test specimen. With fine sands and silts tested in conventional 2.8-inch diameter samples, membrane compliance effects may be negligible since even very thin membranes cannot penetrate significantly into the small surficial voids (Martin et al., 1978; Ramana and Raju, 1982). For medium and coarse sands, however, membrane compliance effects may have a significant influence on test results. Figure 1.2 illustrates a comparison between undrained monotonic loading tests for a fine Sacramento river sand and a coarse uniformly graded sand, both at essentially the same initial relative density and subjected to the same loading conditions. In spite of the initial low relative density, the coarse sand shows no reduction in strength or other evidence of liquefaction with increasing strain, although it would be expected to do so under truly undrained conditions in the field. This is due primarily to the effects of membrane compliance on the laboratory test results. Likewise, for undrained cyclic tests, membrane compliance has the potential to result in considerable overestimation of the resistance of samples to liquefaction under dynamic loadings.

1.3 Liquefaction Potential of Gravelly Soils

Laboratory techniques used for testing sands to obtain estimates of their static and dynamic strength characteristics have been well established by a

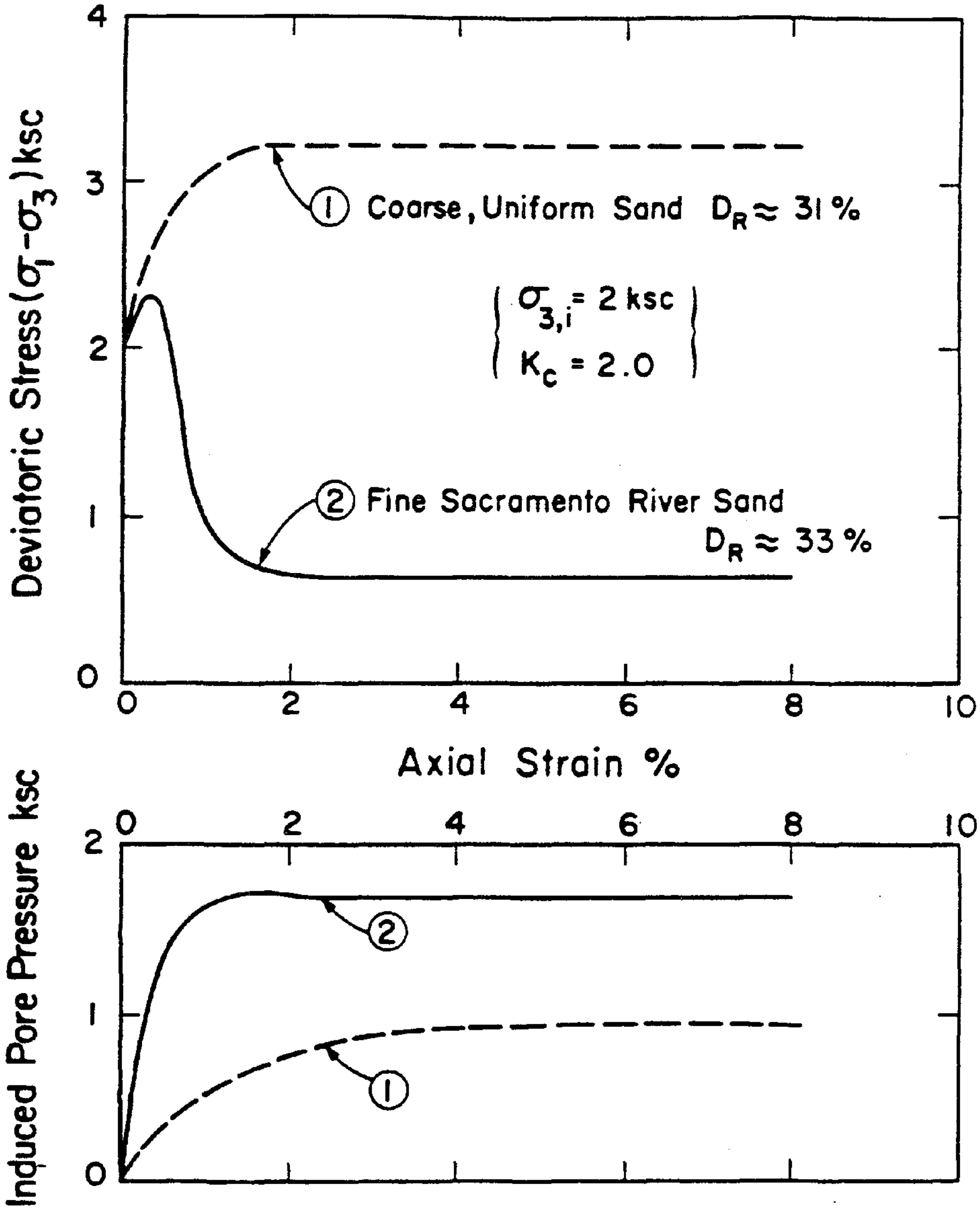


Figure 1.2: <u>Typical $\overline{\text{AC-U}}$ Triaxial Test Results for Coarse and Fine Sands at Low Initial Relative Densities</u>

multitude of investigators over the years. Extensive studies have been made in attempts to account for the problems that have been encountered in the testing of sand under undrained conditions in the laboratory. Much less research has been conducted to explain and mitigate the problems associated with obtaining "truly undrained" representative test results for coarser, gravelly soils. The greatest problems encountered in obtaining correct laboratory test results for these coarser soils are those associated with membrane compliance. Membrane compliance effects, and methods to reduce or mitigate those effects, have been studied primarily for soils no coarser than coarse sand. The effects of membrane compliance are greatly increased for gravelly soils due to the larger interstitial spaces between the larger grains. This allows for a greater amount of volumetric compliance error in undrained tests for these coarser grained materials. For loose gravels subjected to undrained testing, the development of large excess pore-pressures that would cause liquefaction failures is inhibited due to compliance effects, and in some cases sufficiently high pore pressures may never be attained. This is an unsafe error that can potentially lead to failure of earth structures or foundations that had been considered to be "safe" according to conventional test results.

One of the main reasons that the problems of conducting undrained tests on gravels have not been studied to a greater extent is that until recently gravels had not generally been considered to be liquefiable for several reasons including:

(a) The capacity of gravelly soils to dissipate induced pore pressure changes is, in many field cases, sufficiently large enough that initial liquefaction cannot be attained.

(b) Few cyclic laboratory tests have been conducted that provide evidence that gravels are liquefiable.

(c) There are very few case histories in which gravels have been conclusively shown to have liquefied in the field.

(d) Some studies have speculated that gravels may be inherently more resistant to liquefaction than sand.

In response to each of the aforementioned reasons why gravels have not been a major liquefaction concern are the following arguments:

(a) If the drainage from gravelly soils is sufficiently impeded, either by finer surrounding soils or by finer grains infilling the pores between the gravelly soil particles, then an undrained or partially undrained condition may exist in the field, whereby pore pressures may be able to build up to the point of causing initial liquefaction.

(b) The reason that most laboratory tests have shown gravelly soils to be much less susceptible to liquefaction than sands is most likely largely due to the effects of membrane compliance. A recent study by Evans and Seed (1987) used a modified version of the analytical correction procedure proposed by Martin, Finn, and Seed (1978) to correct the data of Lee and Fitton (1969) and Wong, Seed, and Chan (1975). It was concluded that the difference in liquefaction resistance between gravels and sands previously reported could be attributed to membrane compliance effects, and further concluded that sand and gravel specimens at the same relative density under the same test conditions had essentially the same resistance to liquefaction failure.

(c) Although there have, until recently, been very few conclusive field case histories where gravels have been shown to have liquefied in comparison to the abundance of cases involving the liquefaction of sands, a number of noteworthy cases have recently been reported wherein the liquefaction of gravels has occurred. These include: (1) The liquefaction of a gravelly-sand alluvial fan deposit in the in the 1948 Fukui Earthquake (Ishihara, 1985), (2) The liquefaction induced flow slide at Valdez occurring in gravelly sand and sandy gravel in the 1964 Alaska Earthquake (Coulter and Migliaccio, 1966), (3) The slide of the upstream sandy gravel slope of the Baihe Dam in the 1974 Tangshan Earthquake (Chang, 1978; Wang, 1984), and (4) The liquefaction of gravelly soils during the 1983 Mount Borah Earthquake (Youd et al., 1985; Harder and Seed, 1986). A much older report from the 1906 San Francisco earthquake (Lawson, 1908) gives an undeniably clear description of the liquefaction of coarse river gravels that were brought up to the surface from a "blow-hole", although at the time of the report the investigators had little idea of the mechanics behind what they had observed. In the wake of the Mt. Borah Earthquake in Idaho, a drilling investigation discovered that the soil horizon that had liquefied beneath the observed sand boils was in actuality a coarse sandy-gravel. It was suggested that the resulting sand boils revealed only the finer and lighter sand inclusions of the liquefied layer, or that they consisted of material from the dense sandy layer overlying the liquefied zone that had been washed to the surface as water was ejected from the liquefied zone. This finding raises the question as to how many other occurrences of reported sand boils may have obscured the finding of more field cases in which gravels had liquefied. In retrospect, it seems quite possible that in fact there may have been many more cases of gravel liquefaction that were not recognized by surficial investigations and therefore remained unnoticed.

As a result of these recent findings, new concerns are developing regarding the liquefaction potential of gravelly soils. At the recent 50th Anniversary Conference of the International Soil Mechanics and Foundation Engineering held in San Francisco, Dr. Ishihara of Tokyo University, in his state of the art address on the subject of liquefaction, declared that development of methods for evaluation of the liquefaction potential of gravelly soils was one of the most important and urgent problems in this important field of soil mechanics. In a recent state of the art summary report by the National Research Council Committee on Earthquake Engineering (1985), it was concluded that "...our understanding of the dynamic strength of gravels and gravelly soils is not complete, and that these soils can be susceptible to liquefaction."

1.4 Scope of Research Performed

The scope of this research investigation was concentrated in four basic steps: (1) to examine and evaluate the available methods to measure and characterize membrane compliance, and to review the methods previously proposed to mitigate the problems associated with membrane compliance effects, (2) to develop an improved understanding of the factors affecting membrane compliance for a range of soil types, including a wide range of gradation types and grain sizes from silts through gravels, and provide an updated correlation for estimating membrane compliance characteristics for these soils, (3) to verify the use of a computer-controlled injection-correction technique for mitigation of membrane compliance effects in small-scale undrained testing, and (4) to develop and implement a compliance mitigation technique for undrained testing of coarse gravelly soils.

These studies represent the second and final phase of a two-stage effort. The first stage, as reported in detail by Anwar et al. (1989) consisted of developing and implementing a computer-controlled injection/mitigation procedure for use with conventional (2.8-inch diameter) triaxial testing systems. The two basic objectives of this second stage were: (1) to verify the accuracy and reliability of this "small-scale" injection/mitigation procedure, and (2) to develop and undrained testing of "large-scale" (12-inch diameter) samples of coarse, gravelly soils.

The membrane compliance mitigation methodology used in this study involved first pre-determining the volumetric magnitude of membrane compliance for a given soil under a given set of testing conditions as a function of effective confining stress, and then using a computer-controlled process to continuously compensate for calculated volumetric errors by injecting or removing water from the sample, based on monitored changes in effective confining stresses. In order to implement the use of this method, it was necessary to demonstrate that volumetric compliance could be accurately and reliably pre-determined prior to testing, and that it could be reliably characterized in such a manner that the computer-controlled injection/removal process could be based on monitored changes in sample effective confining stresses. In addition, the compensation process had to be continuous so as not to introduce new errors to the test results.

All of the basic goals of the research investigation were achieved. Compliance measurement methods were demonstrated to be accurate and reliable, and volumetric errors induced by membrane compliance were shown to be direct and repeatable functions of effective confining stress, indicating that monitoring the changes in effective stress was a suitable measure on which to base computer-controlled injection/correction during undrained testing. The volumetric compliance characteristics were evaluated for a significant number of sandy and gravelly materials, which when added to the existing database enabled the development of an updated correlation relating measured compliance magnitudes to "representative" material grain sizes. A computer-controlled injection/correction technique implemented for testing of conventional "small-scale" (2.8-inch diameter) samples, as reported by Anwar et al. (1989), was verified by laboratory diameter) samples, as reported by Anwar et al. (1989), was verified by laboratory techniques employing "large-scale" (12-inch diameter samples) triaxial testing equipment.

A large-scale injection/mitigation system was then developed and used to perform undrained monotonic and cyclic triaxial loading tests on samples of a uniformly-graded medium gravel, with and without the use of the computer-controlled membrane compliance mitigation system, in order to provide a basis for evaluating the effectiveness of the compliance mitigation procedures. All undrained loading tests performed as a part of this study used a large-scale testing apparatus with samples of approximately 12 inches in diameter. The test results provide support for the effectiveness of the membrane compliance mitigation procedure, and show the first test results for gravelly soils tested under undrained conditions where the effects of membrane compliance have been completely eliminated.

Chapter 2 presents a review of the extensive previous research performed regarding: (a) measurement and characterization of membrane compliance, and (b) methods to mitigate of compensate for membrane compliance effects in undrained testing. Chapter 3 presents a brief review of previous research performed involving testing of gravelly soils for both static and dynamic strength evaluations. Chapter 4 describes the process by which a continuous computer-controlled injection/correction system developed for testing of conventional sized (2.8-inch diameter) samples was verified by comparison of the test results obtained in that earlier study to test results utilizing large-scale equipment. This step provided the necessary proof that such a mitigation system provided reliable and accurate results which could be suitably adapted for use with large-scale undrained testing of coarse, gravelly soils in specimens of 12-inch diameter or greater.

Chapter 5 describes the development of the computer-controlled process for mitigation of membrane compliance effects in large-scale testing of coarse, gravelly soils. Also included in this chapter is an overview of the studies performed to develop and verify the accuracy and reliability of techniques to pre-determine membrane compliance characteristics. Chapter 6 describes and presents the results of tests performed on a uniformly-graded medium gravel with and without the implementation of the computer-controlled membrane compliance mitigation system developed for large-scale testing, and examines the effectiveness of the mitigation method based on examination of the test results. Chapter 7 provides a summary of the investigations performed in this study and presents conclusions as to the accuracy, reliability and usefulness of the testing correction procedures developed and evaluated in these studies.

CHAPTER 2

SUMMARY OF PREVIOUS INVESTIGATIONS

2.1 Introduction

The problems arising as a result of membrane penetration were first presented and reported by Newland and Allely (1957, 1959). Since that time, a number of investigations have been conducted in order to develop techniques for evaluation and/or mitigation of the effects of membrane compliance in undrained tests in order to develop reliable procedures by which truly representative test results could be obtained.

A number of methods have been proposed in attempts to test cohesionless soils without the effects of membrane compliance by physically mitigating compliance during testing. Learning from these previous investigations, it was concluded that the technology has advanced to the point where it now appears possible to develop and implement a technique to obtain truly representative undrained test results for granular materials. While many of the previously proposed methods have reduced the effects of compliance, it was not until recently that research had advanced to the point of being potentially able to completely mitigate membrane compliance during testing without introducing additional problems of load corrections or the use of unverified assumptions. A review of previous investigations leading up to the current research to physically mitigate membrane compliance effects in undrained tests is presented in Section 2.3.1.

While many were attempting to discover a method to directly eliminate the effects of membrane compliance during undrained testing by physical mitigation, other investigators conducted studies to devise theoretical post-test corrections. Ideally, these theoretical corrections could be used to correct not only future tests, but also any conventional tests that had been previously performed, where membrane compliance may have given erroneous results. Until recently, it had not been possible to verify the validity of these analytical corrections, as no method had been devised to give correct, truly undrained results with which to compare the theoretical analyses. A summary of some of these theoretical post-testing correction methods is presented in Section 2.3.4.

In spite of the progress and advances made in testing and measurement techniques, there still existed several areas in which the problems associated with membrane compliance had not been resolved. These include: (a) the lack of any reliable procedures for correction of conventional test results in order to compensate for compliance effects; (b) the inability to obtain representative undrained testing of saturated soils coarser than coarse sands to fine gravels as a result of compliance effects, even when employing available large-scale testing apparatus; (c) the lack of verification of the efficacy of recently developed (and promising) techniques for mitigation of membrane compliance effects, and (d) the inability of large-scale triaxial testing systems to provide representative undrained strength and liquefaction evaluations for most gravelly soils and rockfill.

This chapter presents a brief summary of previous research investigations that have been performed leading to development of methods to evaluate and mitigate membrane compliance effects in undrained testing of saturated granular soils.

2.2 Evaluation of Membrane Compliance Effects

2.2.1 General:

Over the past 30 years there has been considerable progress made in developing methods to accurately evaluate the magnitude of membrane compliance effects. Accurate evaluation of these effects is vital, and represents a necessary first step in developing procedures to mitigate the effects of membrane compliance in undrained testing. All of the methods that have been proposed for evaluation of membrane compliance effects incorporate the use of fully drained tests in order to determine the volumetric magnitude of membrane penetration changes as a result of changes in the effective confining stress on a sample.

As the effective confining stress applied to a sample under drained conditions is changed, the total sample volume contained within the sample membrane is changed. This change in sample volume (ΔV_T) is conventionally measured by means of a calibrated device which measures the volume of water either expelled or drawn into the sample. This volume of water is actually the sum of two volume change components. One component is the "true" or skeletal volume change of the soil sample, which is equal to the "true" sample volumetric strain ($\epsilon_{v,s}$) multiplied by the overall sample volume (V_T). The second component of the total measured volume change is due to the variation in the amount of membrane penetration (membrane compliance). This second component of volume change can be expressed as the compliance-induced volume change per unit area of the membrane (δV_m) multiplied by the total area of the membrane (A_m). The total measured volume change can then be taken as the sum of these components as:

$$\Delta V_T = [\epsilon_{v,s} * V_T] + [\delta V_m * A_m] \qquad \text{[Eq. 2.1]}$$

The different methods developed to evaluate the magnitude of volumetric membrane compliance have been based primarily on developing different techniques to differentiate between the two volume change components ($\epsilon_{v,s}$ and δV_m). For large-scale triaxial samples (diameters ≥ 12 inches) this is not such a difficult problem as both axial and radial strains can be measured directly in order to evaluate the "true" sample skeletal volume change. Radial strains for such large-scale samples can be measured with the use of so called "girth-belts" which can accurately measure changes in sample circumference during the application of changes in effective sample confining stress. A illustration of the use of girth belts is shown in Figure 2.1. When dealing with smaller-scale samples (diameters < 6 inches), however, it is exceedingly difficult to measure radial strains with sufficient accuracy by this method, and other techniques must be employed to differentiate between the volume change components due to sample skeletal volume change and membrane compliance.

2.2.2 Measurement Methods to Evaluate Membrane Compliance

Newland and Allely (1957, 1959) were the first investigators to propose a method for evaluation of volumetric membrane compliance due to changes in applied effective confining pressures in triaxial testing. Assuming isotropic compression and rebound of triaxial specimens under varying hydrostatic loadings, they calculated volumetric membrane compliance as the difference between total volumetric strain (ϵ_v) and three times the measured axial strain ($3\epsilon_a$) induced by application of an isotropic stress increase in drained tests on saturated specimens. This procedure implicitly assumes that sample radial strains will be equal to the axial strain; an assumption subsequently found to be untrue. Nonetheless, this early work by Newland and Allely established important precedents and was the forerunner for numerous subsequent investigations of membrane compliance.

Since this early work, considerable progress has been made by various investigators in developing more accurate and reliable techniques for determining the magnitude of membrane compliance. A common feature of all the methods developed thus far is the use of fully drained tests to determine the volumetric magnitude of membrane penetration as a result of variations in effective confining

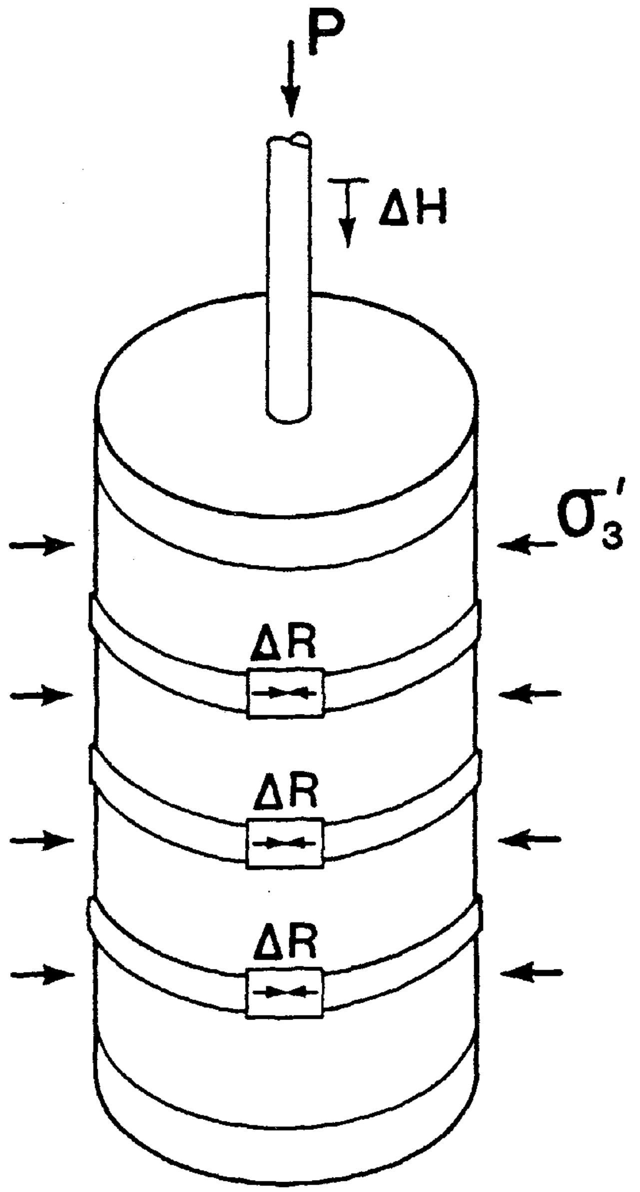
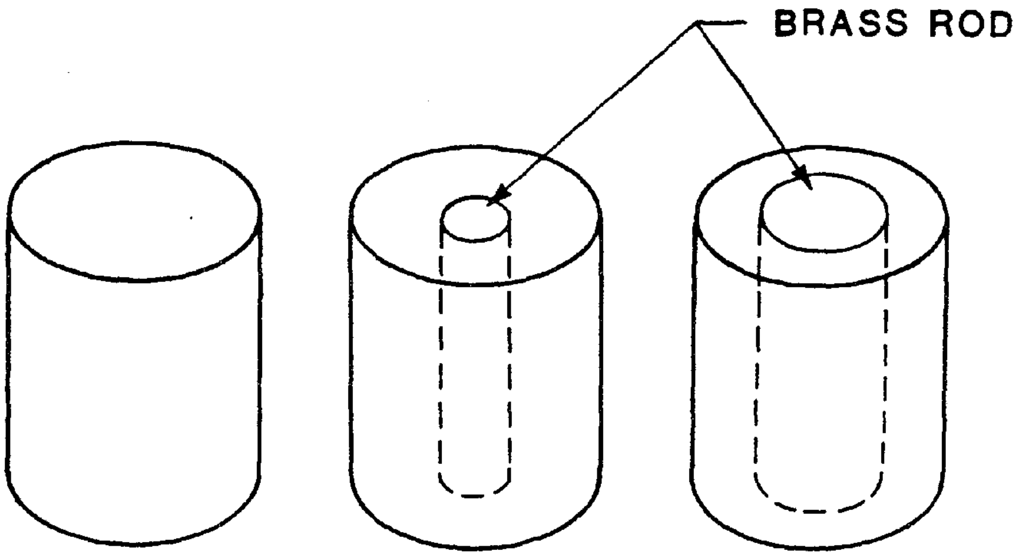


Figure 2.1: Schematic Illustration of the Use of Girth Belts to Measure Radial Strains

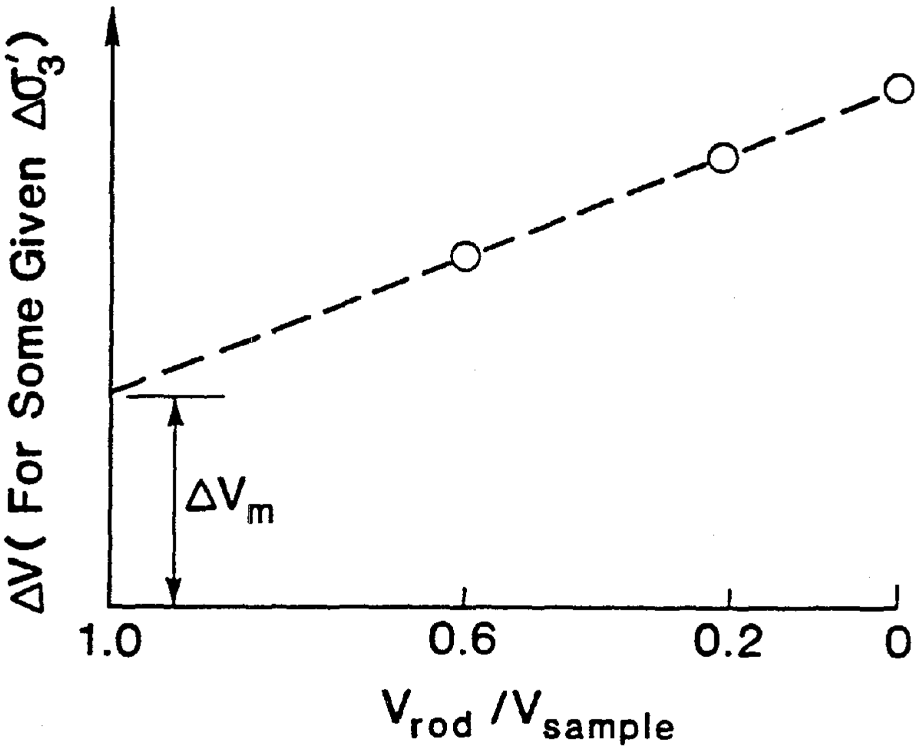
stress. Roscoe et al. (1963) proposed two methods for evaluating membrane penetration effects. The first method was similar to that of Newland and Allely and thus over-estimated membrane penetration. The error in the above method stems from the fact that hydrostatic loading of granular soils leads to anisotropic deformations, with greater radial than axial strains (Vaid & Negussey, 1984). Thus membrane compliance is overestimated due to the assumption that skeletal volumetric strain is equal to three times the skeletal axial strain.

Noting correctly that the assumption of isotropic behavior is unrealistic, Roscoe proposed a second method for evaluating compliance by placing brass rods coaxially in 1.5 inch diameter triaxial specimens as illustrated schematically in Figure 2.2(a). The rod heights were equal to the heights of the specimens, but two different rod diameters (0.25 and 1.37 inches) were used. As the diameter of the rods increased, the remaining sample soil volume decreased while membrane surface area remained constant. A sample with no rod inclusion was also tested. Volumetric membrane compliance was estimated by plotting measured volume changes for a given change in applied effective confining stress versus rod diameter. Using linear extrapolation, membrane penetration was assumed to be the total remaining measured volume change at a projected rod diameter equal to the diameter of the sample (at which point skeletal volume change would be equal to zero, so that all remaining volume change would be due the membrane compliance.)

El-Sohby (1964) and Lee (1966) also used the central rod method with minor variations in rod sizes to evaluate compliance effects. Thurairaja and Roscoe (1965) performed a subsequent study and concluded that the central rod method suffered from a number of drawbacks and in fact was not markedly better than Newland and Allely's original method based on assumed sample isotropy.



(a) Samples with Central Rods of Various Diameters



(b) Plot of Δ V vs. Sample Soil Volume (or the Ratio of Rod Volume: Soil Volume) for Some $\Delta\sigma_3$ '

Figure 2.2: Schematic Illustration of the Use of Central Rods in Triaxial Samples to Evaluate Membrane Compliance

Steinbach (1967) concurred and used the original Newland and Allely method in a parametric study of compliance effects and concluded that the major factor influencing membrane penetration is grain size, with gradation, particle shape and density exerting minor effects. Raju and Sadasivian (1974) noted that the main drawbacks of the central rod method were twofold: (a) the rod itself caused stress concentrations within the sample due to axial rigidity; and (b) the assumption of a linear relationship between rod diameter and total volume change was incorrect. They developed a modified ("flexible") top platen which theoretically provided an improved stress field within the sample, and they demonstrated that a linear relationship existed between total volume change under some incremental load increase and actual sample volume, not rod diameter, as illustrated schematically in Figure 2.2(b).

Roscoe's central rod method was conceptually correct except for the assumption that the change in sample volume was linearly related to rod diameter. Raju and Sadasivian, having recognized this point, corrected it but failed to significantly reduce the other error introduced in the experimental method by employing a "flexible" top platen, as this did not prevent stress concentrations and non-uniformity of stress fields within the samples. This conclusion is drawn from comparing Roscoe's results with those of Raju and Sadasivian. After correcting Roscoe's original results using the correct relationship between skeletal volume change and soil volume (instead of rod diameter), no significant difference is detected between the two sets of results. The problem with both approaches is that the internal stress and strain fields within the samples vary considerably as a function of rod diameter (as a result of stress concentrations), so that exact linearity of the relationship between measured volume change and soil volume is not likely. Also, sample quality in terms of reproducible homogeneity and density control is poor due to difficulties in sample preparation with the obstruction presented by the central rod. Nonetheless, this method can provide good results when soil particles are "large" relative to sample diameter, so that volume changes due to membrane compliance are large relative to volume changes due to sample skeletal compression or rebound.

Frydman, et al. (1973), assumed that isotropic loading on various samples having similar densities yields identical volumetric strains, and developed a method for evaluating volumetric membrane compliance using hollow cylindrical samples as shown in Figure 2.3. The samples, formed from monosized glass beads, had central cylindrical voids of varying diameters that eliminated the rod stiffness and rod shearing effects (stress concentrations) inherent in the use of the "central rod method", while the central cylindrical voids caused sample volume and surface area (and the ratio between them) to vary. The pressure within the central void was varied in conjunction with the confining pressure applied on the outer face of each sample. For a given variation in applied effective confining stress, the total volumetric strain (ϵ_v) was plotted versus the ratio between membrane surface area and initial sample volume (A_m/V_o) for samples prepared with various internal void diameters, as shown in Figure 2.4. The points were found to plot linearly and intercepted the ϵ_v axis at the true sample volumetric strain. The slope was equal to the unit volumetric membrane penetration (volumetric membrane compliance per unit membrane area; δV_m). By plotting unit membrane compliance against the log of effective confining pressure, Frydman obtained linear relationships with slope S (normalized membrane penetration) which, when plotted against the log of mean particle size (D_{50}), resulted in a general relationship (see Figure 2.5) that was subsequently used to back-calculate membrane compliance corrections for his tests.

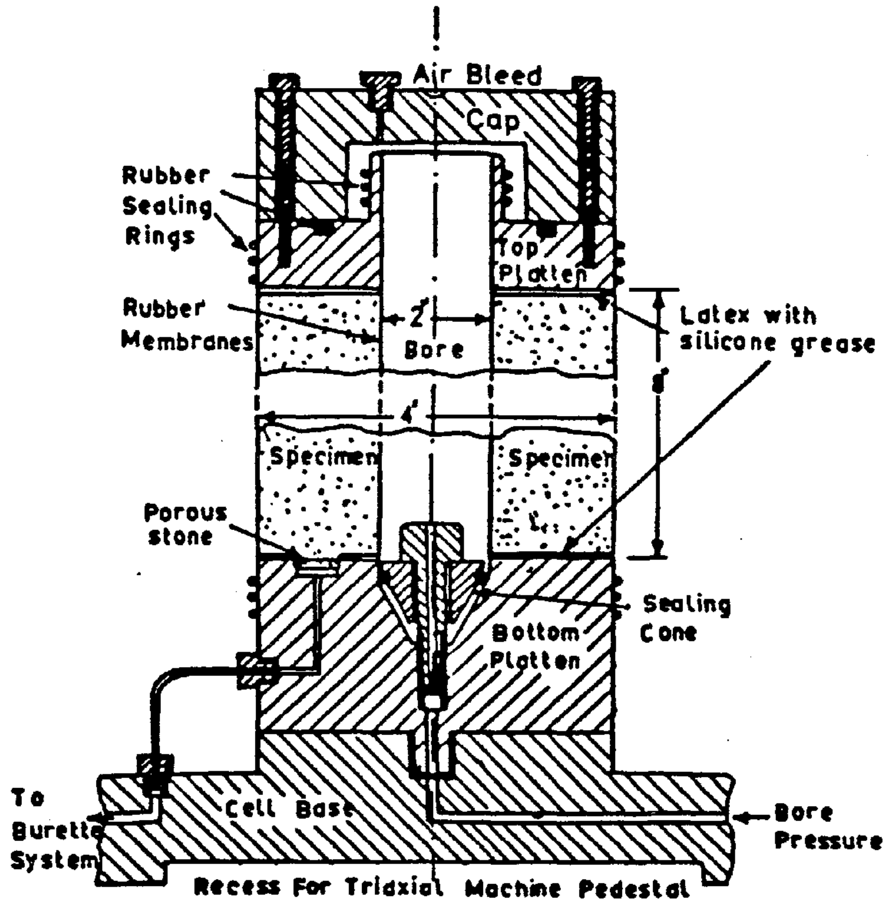


Figure 2.3: Schematic Illustration of Hollow Cylinder Samples Used to Evaluate Membrane Compliance: (Frydman et al., 1973)

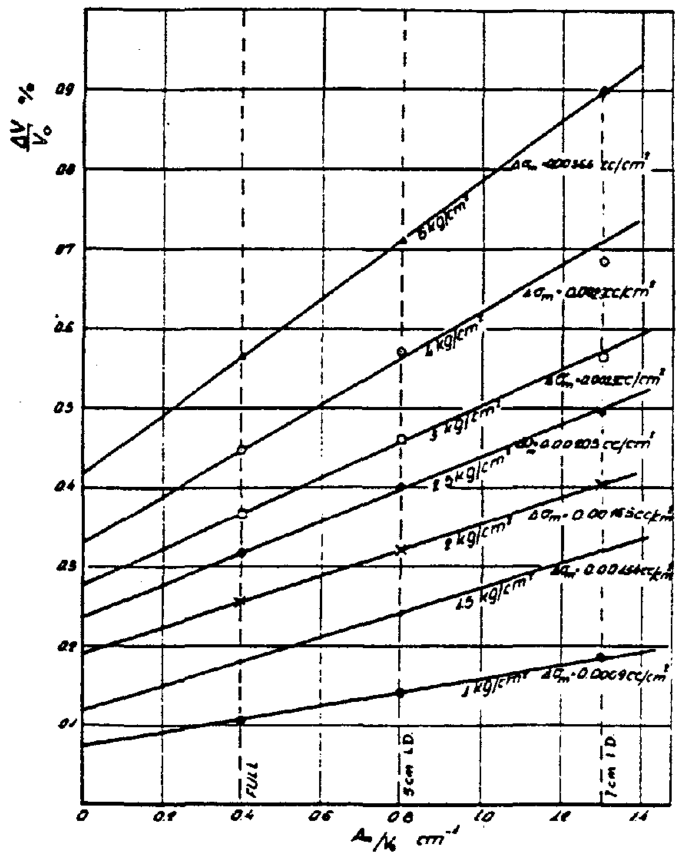


Figure 2.4: Plot of Volumetric Strain vs. A_m/V_0 for a Given Change in Applied Effective Confining Stress:(Frydman et al., 1973)

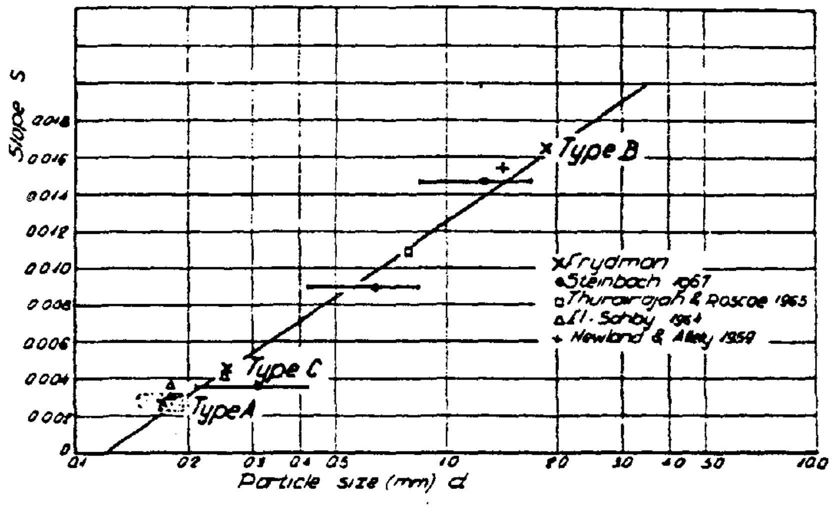
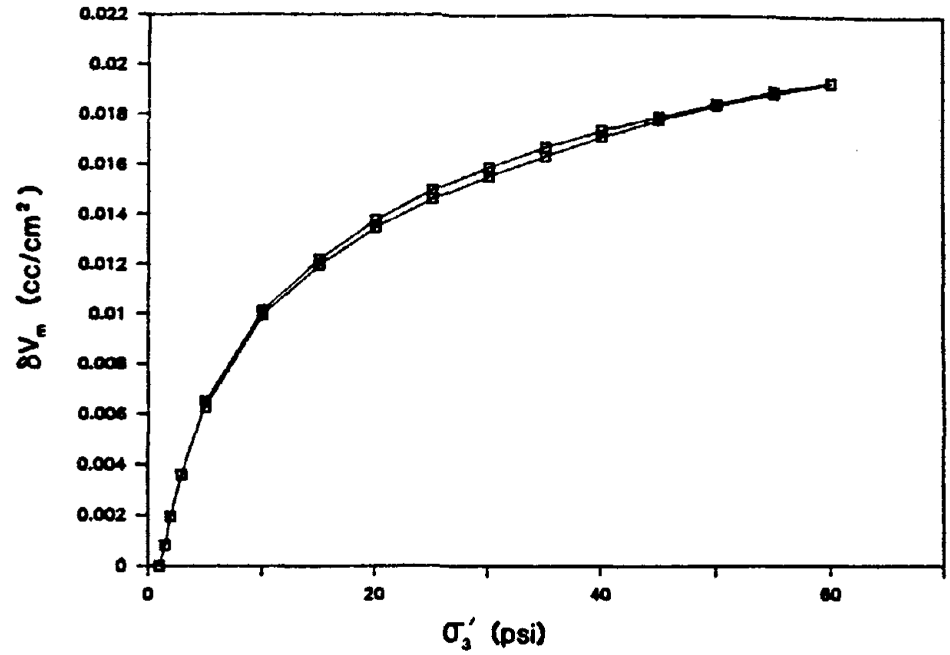


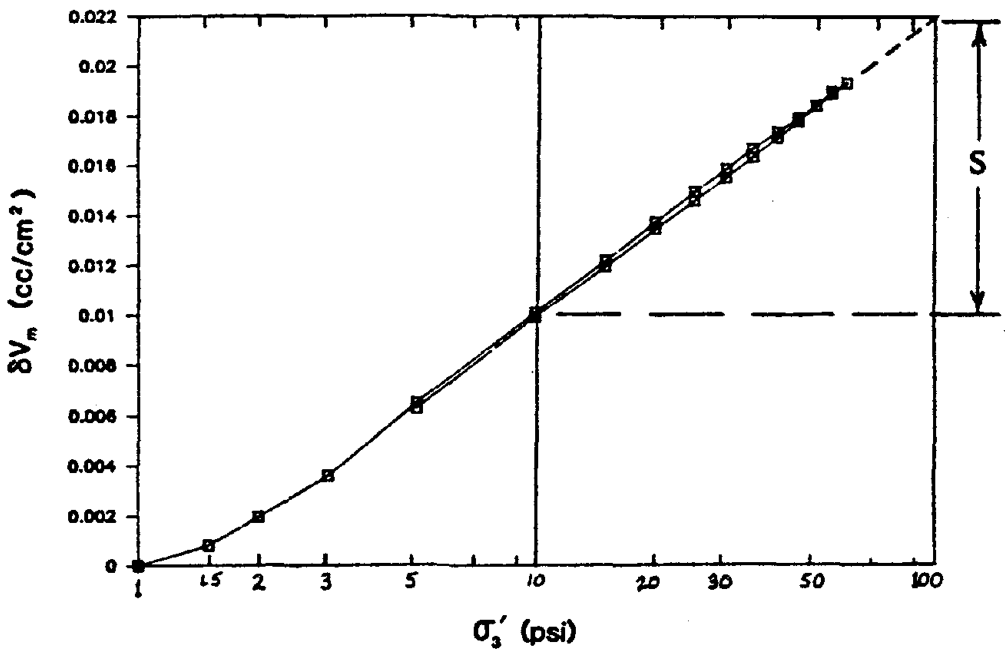
Figure 2.5: Proposed Relationship Between Mean Grain Size (D_{50}) and Normalized Penetration (S): (Frydman et al., 1973)

Frydman's method of membrane correction satisfies the basic assumptions of uniform stresses and strains within and between specimens, and yields more reliable estimates of volumetric membrane compliance than previous techniques. However, hollow samples are extremely difficult to prepare and duplicate with the precise duplication of sample density required; thus a simpler technique for evaluation of membrane compliance, based on simpler and more readily reproducible sample preparation techniques is desirable. The concept of unit membrane compliance introduced in his work represents a significant contribution as it is useful as a normalization tool for comparing compliance for different situations and materials. It is now well-established that the nonlinear relationship between membrane-induced volume change and applied effective stress, as illustrated in Figure 2.6(a), becomes essentially linear when plotted on a semi-logarithmic scale as shown in Figure 2.6(b). The slope of the relationship between unit membrane compliance (δV_m: change in volume per unit membrane area) and log (σ_3') will hereafter be referred to as the normalized penetration (S). S will thus be formally defined as δV_m per log-cycle change in σ_3'.

Keikbusch and Schuppener (1977) developed a procedure in which a saturated soil sample was placed in a shallow well in the base of a triaxial cell. The top surface of the sample was flush with the base of the cell, and was covered with a sheet membrane. Pressure was applied to the top of the sample, and a sensitive deflection guage measured the resulting vertical deflection of a tripod mounted on the membrane covering the sample, allowing differentiation between volume changes due to membrane penetration and those due to sample compression. Their investigation examined specimens ranging from poorly graded to very well graded sandy and silty soils and yielded an interesting nonlinear relationship for normalized membrane penetration (S) versus mean grain size (D_{50}). This



(a) Unit Membrane Compliance (δV_m) vs. Effective Stress (σ_3')



(b) δV_m vs. log (σ_3'); Defined Normalized Penetration (S)

Figure 2.6: Typical Plots of Unit Membrane Compliance vs. Effective Confining Stress (Monterey #16 Sand at $D_R \approx 60\%$)

relationship, shown in Figure 2.7, will be discussed later. Unfortunately, most of the soils tested had very high fines contents so that membrane compliance was either negligible or was dominated by consolidation effects in the sample which, due to the choice of measurement method, could not be determined with the level of accuracy desirable. A further problem with this method was the potential for edge effects during sample loading which could also introduce errors in the results. This method, which is essentially a one-dimensional version of Roscoe's approach, may potentially yield good results if soil volume were minimized so that changes in volume would be principally due to membrane compliance, and provides good results when compliance-induced volume changes are large (when soils are relatively coarse).

Vaid and Negussey (1984) examined the fundamental assumptions involved in the assessment of sample volume changes due to membrane compliance in triaxial tests on granular soils. They concluded that the necessary assumptions render invalid methods that (a) use dummy rod inclusions or (b) assume isotropic sample behavior. They noted that if, for a given change in confining pressure, the total resulting volume change is measured, then this volume change is the sum of volume changes due to (a) membrane penetration effects and (b) soil deformation effects, and can be expressed as:

$$\Delta V_t = \Delta V_m + \Delta V_c \qquad \text{[Eq. 2.2]}$$

where ΔV_t = total volume change,

ΔV_m = volume change caused by membrane penetration, and

ΔV_c = volume change due to soil skeletal deformation.

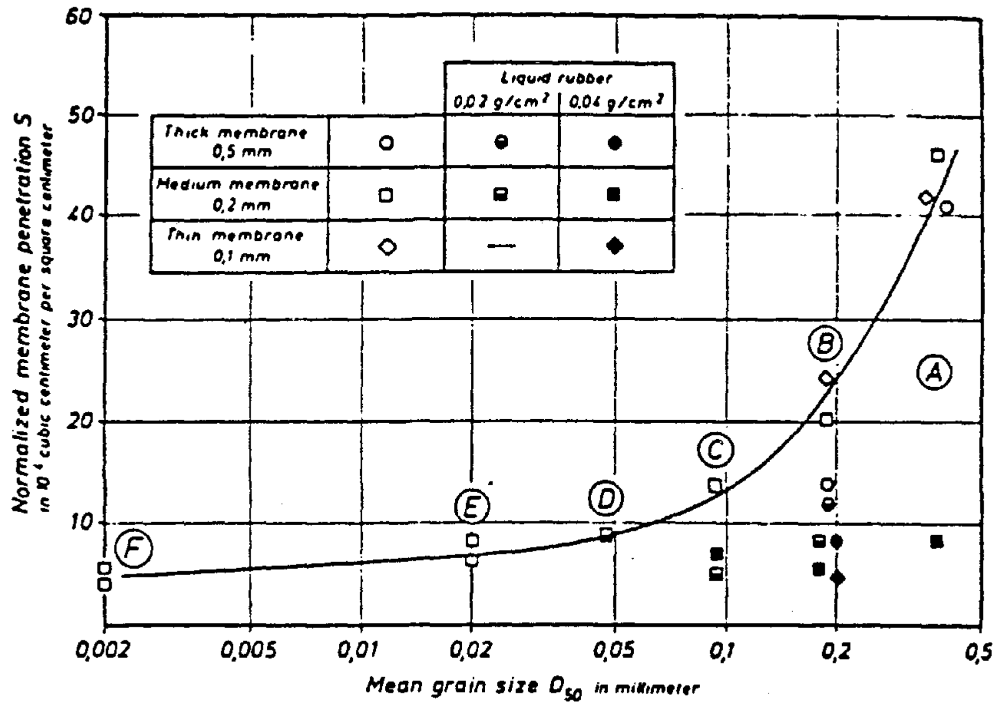


Figure 2.7: Proposed Relationship Between Mean Grain Size (D_{50}) and Normalized Penetration (S): (Keikbusch and Schuppener, 1977)

If δV_m is membrane penetration per unit membrane surface area, and ϵ_{vs} is soil volumetric strain, Eq. 2.2 can be rewritten as shown earlier in Eq. 2.1.

Vaid and Negussey argued that Newland and Allely's method overestimates the membrane effects since experimental results suggest hydrostatic loading of sand is associated with anisotropic strains, with larger radial than axial strains, and not with isotropic behavior as originally assumed. For the central rod method the assumption that sample volume is linearly related to rod diameter was shown to be erroneous by Raju and Sadasivan (1974), but the modification they suggested (which was aimed at ensuring that under an incremental change in confining pressure the sample would be subjected to a state of hydrostatic compression) was not fully successful. However, the assumption of equal soil volumetric strain for a given change in confining pressure among specimens with different rod diameters is still central to the interpretation of test results.

Vaid and Negussey suggested that reliable membrane compliance determinations can be made by either (a) Performing multiple isotropic tests on specimens having different diameters, then solving for the component of volume change due to membrane compliance using Equations 2.1 and 2.2 (a method also employed by Wong, 1983), or (b) Performing single tests on triaxial specimens subjected to triaxial unloading ("isotropic rebound"), in which case the assumption that volumetric sample strain is equal to three times the axial strain is nearly valid. They performed tests of both types on samples of Ottawa sand, a uniformly graded medium sand at a relative density of approximately $D_R \approx 50\%$, and found that both methods yielded similar results.

In considering Vaid and Negussey's alternate proposed technique, based on the assumption of isotropic strain behavior during unloading, however, it should be noted that for the Ottawa sand tested, the magnitude of the volume changes due to membrane compliance was significantly greater than that due to changes in volume of the soil itself. Errors introduced due to the assumption of isotropic rebound were therefore not as significant as would be expected for finer-grained soils. This "isotropic rebound" assumption thus appears to provide good results when membrane compliance effects are large relative to sample volume, but may be suspect when compliance-related volume changes are small. A more refined examination of this isotropic rebound assumption (Anwar et al., 1989) showed that sample strains in rebound are indeed more nearly isotropic than in initial compression, but that they are not fully isotropic. Moreover, the relationship between axial strain and radial strain (ϵ_a/ϵ_r) in rebound was found to be a function of sample density. Assumption of isotropic rebound behavior can introduce significant error in the measurement of membrane compliance for soils (and sample sizes) in which the volume changes due to membrane penetration variation are not large relative to sample skeletal volume changes resulting from variation of effective sample confining stress.

The use of a pair of samples of different diameter represents an excellent technique so long as both samples are prepared to exactly the same density (so that skeletal volumetric strains will be the same under the same applied effective stress changes.) This approach can be further improved by preparing the two samples with different diameters, but with <u>identical height:diameter ratios</u>, so that internal stress and strain fields will be similar, ensuring appropriate scaling of sample skeletal volume changes.

Kramer and Sivaneswaran (1988) carried this type of measurement technique one step further by devising a membrane penetration test frame with a thin sheet membrane stretched over a 7 by 9 inch opening between 3/4 inch thick aluminum plates. Theoretically, a single layer of particles could be placed beneath the membrane in specified packing arrangements so that there could be no collapse of the soil structure adding to volumetric strains. An advantage of this newer testing apparatus was that the material below the membrane could be vacuum saturated while a regulated vacuum pressure was applied below and above the membrane. This way no excess pressure would be applied to the membrane at the start of the test. Both uniform steel spheres and actual soil samples were tested in this apparatus. Assuming that the layer of spheres or soil grains and the testing frame were virtually incompressible, the measured volume change resulted solely from membrane penetration. Another variation of the "shallow well" or "thin" sample method to determine membrane penetration errors was performed by Lo et al. (1989), in which a special cell called the "error cell" was developed and used for the tests. A schematic of the error cell is shown in Figure 2.8. This measurement technique was used to more accurately determine the effectiveness of using membranes coated with liquid rubber, as suggested by Keikbusch and Schuppener (1977), to reduce the effects of membrane compliance from undrained triaxial tests. This method is discussed in more detail in Section 2.3.3.

Kramer and Sivaneswaran (1988) also conducted a second set of membrane penetration measurements in order to develop analytical expressions for membrane penetration behavior. The second set of tests were performed on a number of coarse-grained soils prepared in conventional triaxial samples employing a two membrane, "non-destructive" method. The argument for using this method was that every sample constructed has a slightly different amount of compliance, so that the actual amount of volumetric compliance should be measured for each individual sample to be tested. Most methods for measuring volumetric compliance result in an appreciable amount of sample disturbance due to loading and unloading of the sample. In that study, a conventional triaxial

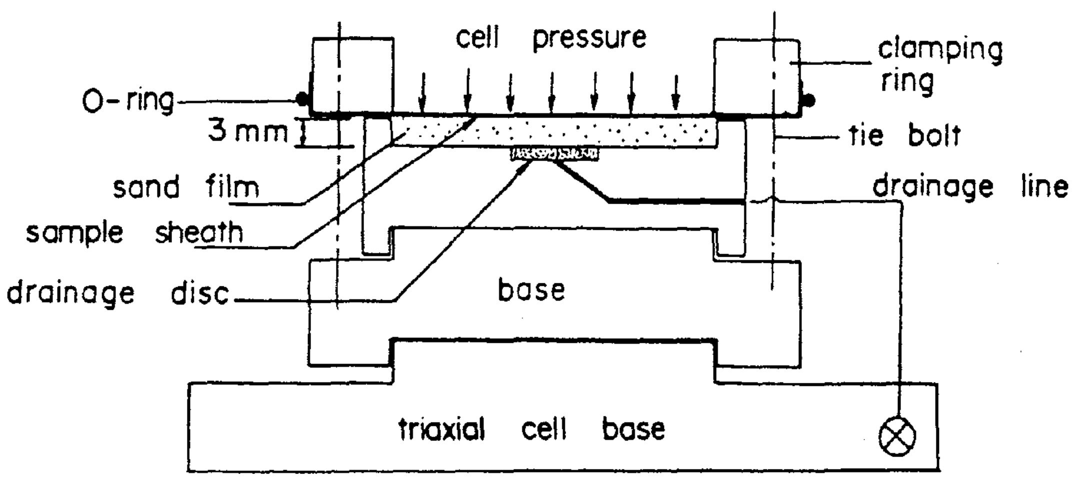


Figure 2.8: Schematic Illustration of the "Error Cell" Used to Measure Membrane Penetration:(Lo et al., 1989)

sample was constructed with a single membrane. A second membrane with a separate drain line connected to the annular space between the two membranes was placed outside of the first and the full consolidation stress was applied. Then without causing any changes of the stresses on the sample or the inner membrane, the pressures inside and outside of the outer membrane were adjusted so that the effective stress on the outer membrane was allowed to be incrementally lowered from the full confining stress to a nominal effective stress. As the outer membrane was relaxed, recordings were made of the volume of water drawn into the annular void thus representing the volumetric error induced by membrane compliance over the range of effective stresses of concern for the test.

2.2.3 Summary of Compliance Measurement Research

A comprehensive study of early investigations of membrane compliance was summarized by Ramana and Raju (1982). It was concluded that the most dominant factors affecting volumetric membrane compliance were: (a) mean grain size, (b) effective confining stress, and to a lesser extent, (c) sample density. Empirical equations have been proposed to estimate the volume changes that may be expected due to membrane penetration as a function of these parameters. It should be noted that most empirical equations are applicable only to uniformly graded materials, and would not necessarily hold for other types of soil gradations.

Seed and Anwar (1986) conducted a more complete investigation into the factors affecting volumetric membrane compliance. They proposed that the use of grain size indice D_{20} gave a significantly better correlation to volumetric compliance measurements than D_{50}, which has been most commonly used in empirical relationships. The effect of different types of soil gradations on compliance values was also investigated by Seed and Anwar, but at this point it has not been proven conclusively that the effects of specific gradations can be quantitatively accounted for. It does appear, however, from the limited amount of data available, that gap-graded soils and well graded soils that contain a high percentage of fines, tend to exhibit a somewhat lower amount of compliance than would be predicted from the D_{20} relationship. Factors that appeared to have little or no significant contribution to membrane compliance included: (a) sample particle angularity, (b) sample fabric or method of sample preparation (also reported from the investigations made by Banerjee et al., 1979), (c) sample density, and (d) membrane thickness or stiffness (within reasonable limits). While sample density has been shown to have minor effects, studies have demonstrated that only some minor adjustments would have to be made for very loose or very dense soils. An example of the difference in unit membrane compliance for a soil over a range of relative densities can be seen in Figure 2.9.

The effect of membrane thickness or stiffness has been studied by several investigators including Keikbusch and Schuppener (1977), Ramana and Raju (1982), Seed and Anwar (1986), Evans and Seed (1987), and Kramer and Sivaneswaran (1988). While most researchers have suggested that membranes of different thicknesses had only a nominal effect on values of unit membrane compliance, some others have described appreciable variations in compliance values with varying thicknesses or numbers of membranes, especially when the ratio of mean particle size to total membrane thickness becomes large (Kramer and Sivaneswaran, 1988; Evans and Seed, 1987). Theoretical studies that demonstrated insignificant effects of different membrane thicknesses on unit membrane compliance were performed by Molenkamp and Luger (1981) and Baldi and Nova (1982).

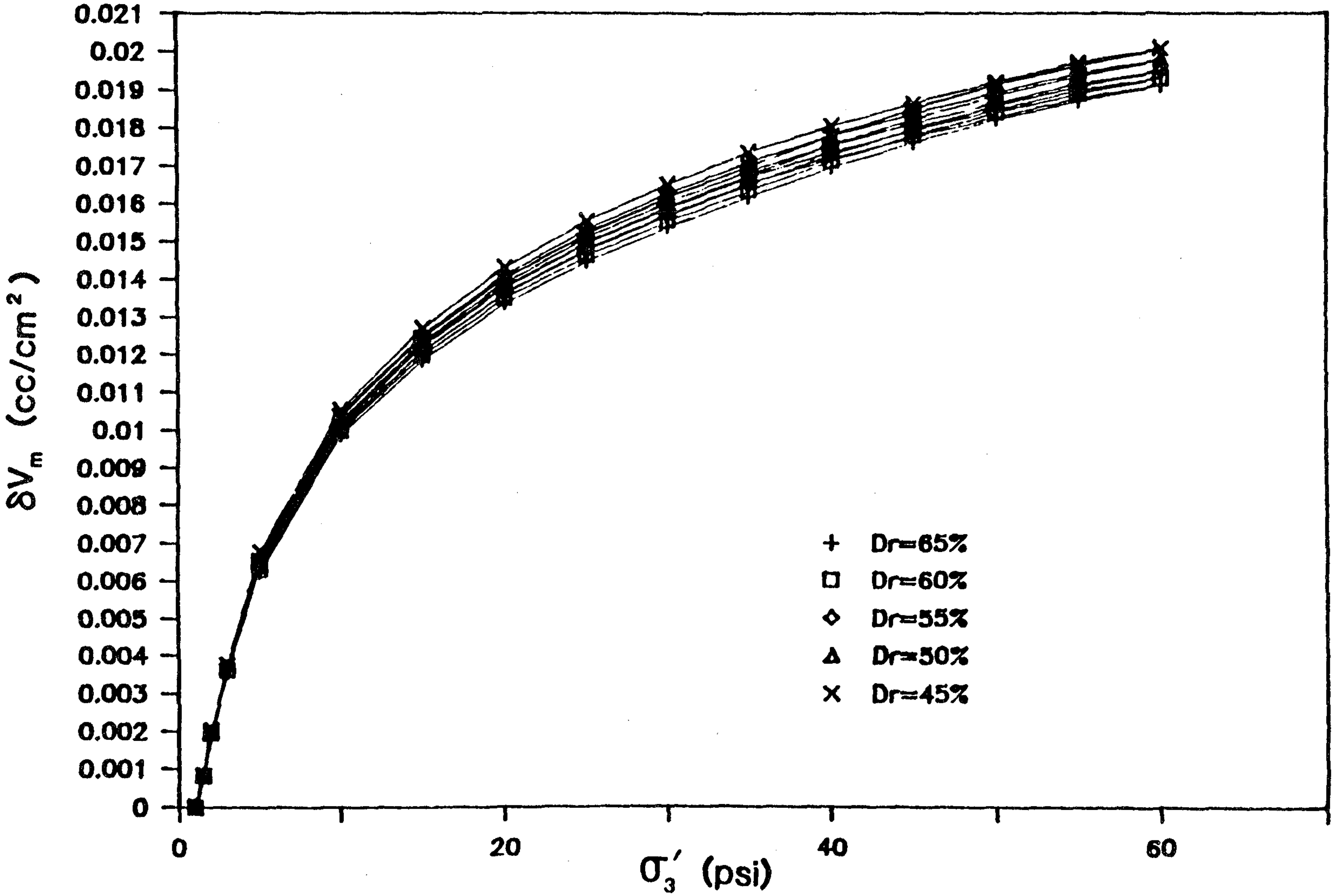


Figure 2.9: Unit Membrane Compliance vs. Effective Confining Stress for Monterey 16 Sand Over a Range of Relative Densities

It was noted by Molenkamp and Luger (1981), Seed and Anwar (1986), and Evans and Seed (1987), that some creep effects and strain softening of the latex membranes is likely to affect membrane compliance, and so it is suggested that samples should be left standing under the initial effective confining stress for approximately one hour before taking volumetric compliance measurements.

Evans and Seed (1987) performed an investigation to account for the possible effects of using varied numbers of membranes to confine 12-inch samples. Their investigation showed that the difference between using a single membrane as opposed to using two membranes to confine a sample, made a difference in measured volumetric membrane compliance of nearly two times. However, very little difference was noted between using two and four membranes, suggesting that this compliance effect had to do with the use of multiple membranes, and not necessarily the total thickness of the membrane(s). The effect of using multiple membranes was attributed to an adhesion between membranes which results in a non-recoverable residual penetration of the membranes into the external sample voids. Results of the investigation by Evans and Seed of using different numbers of confining membranes is shown in Figure 2.10. Investigations made as a part of this study did not fully support the findings of Evans and Seed, but instead concluded that the difference between confining the 12-inch samples with one or two membranes had a differential effect on the amount of membrane compliance of less than 15%.

With all of these possible affecting influences duly accounted for, it is apparent that volumetric membrane compliance is a direct and repeatable function that can be accurately and reliably pre-determined for a soil at a given density prior to testing.

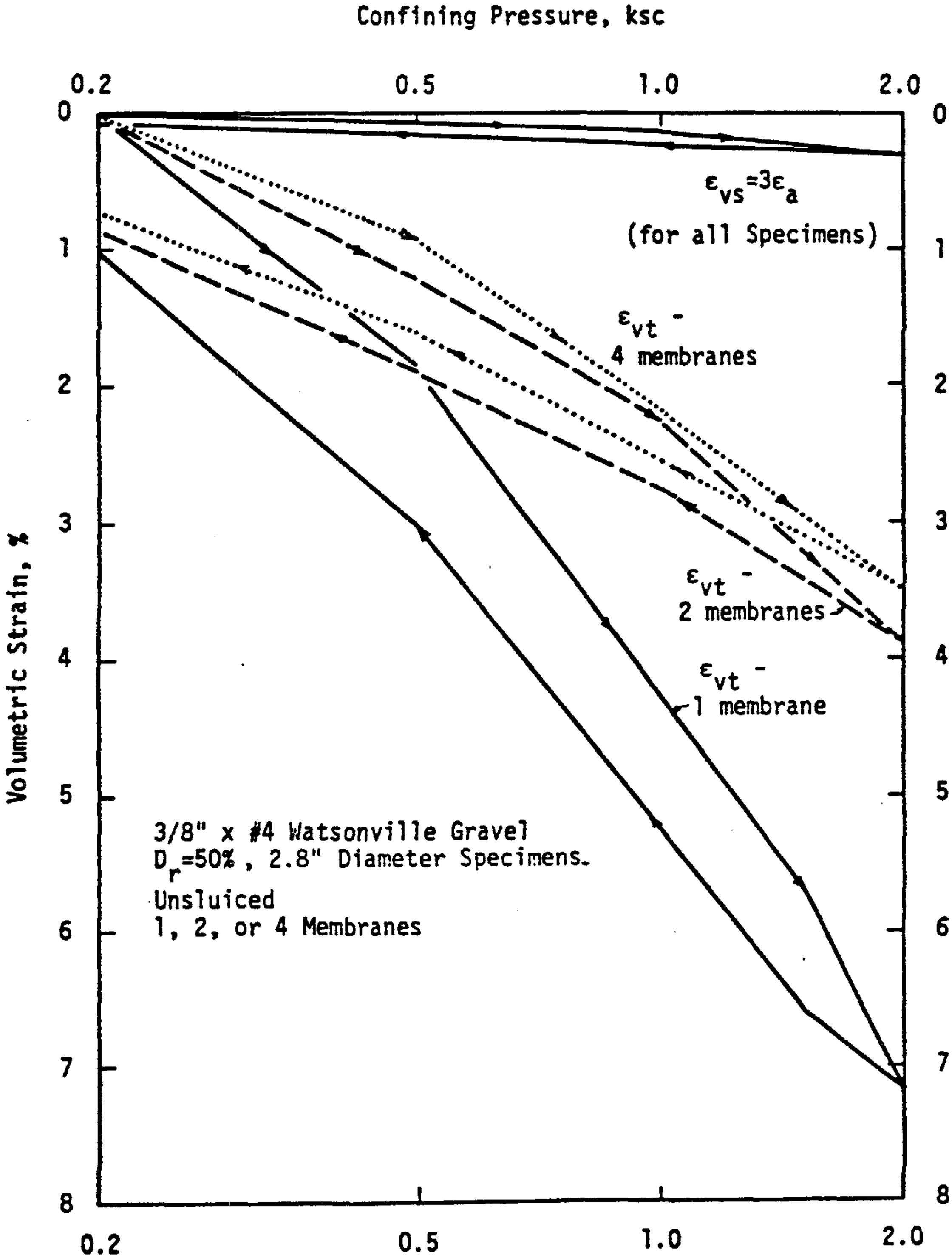


Figure 2.10: Total and Skeletal Volumetric Strains in 2.8-Inch Diameter Specimens Confined With 1, 2, or 4 Membranes: (Evans and Seed, 1987)

2.2.4 Mathematical Modelling of Membrane Compliance

A number of studies have been made including those by Flavigny and Darve (1977), Molenkamp and Luger (1981), and Kramer and Sivaneswaran (1989) to devise analytical or "theoretical" models in attempts to simulate membrane compliance behavior. Although these models may be of interest as they suggest possible mechanisms that may control aspects of compliance behavior, they do not yet appear to be able to provide accurate and reliable representations of compliance magnitudes for different soils and situations as they have been measured by the most sophisticated laboratory tests. Furthermore, it is interesting to note that for these "theoretical" models, expressions that may at first appear to give erroneous results could be reasonably "fit" to compliance measurement test data by manipulating some of the modelling parameters, such as membrane thickness and Young's modulus (stiffness), which have been shown to have minimal effect on membrane compliance values.

A model has been developed by Kramer and Sivaneswaran (1989), that assumes a more correct deformed membrane shape than the more simplified shapes assumed by previous studies. Figure 2.11 shows a comparison between the membrane deformation models used by Molenkamp and Luger (1981), Baldi and Nova (1984), and that used by Kramer and Sivaneswaran. The model used by Molenkamp and Luger assumed a simple three-dimensional indentation mechanism for the membrane, shown in Figure 2.11(a). The model described by Baldi and Nova (1984) proposed that a better representation of the deformed membrane would be as shown in Figure 2.11(b). The results of the modelling performed using the assumed deformation configuration suggested by Kramer and Sivaneswaran (1989), shown in Figure 2.11(c), compared favorably with the experimental results performed on a sample of coarse sand, and appears to give a

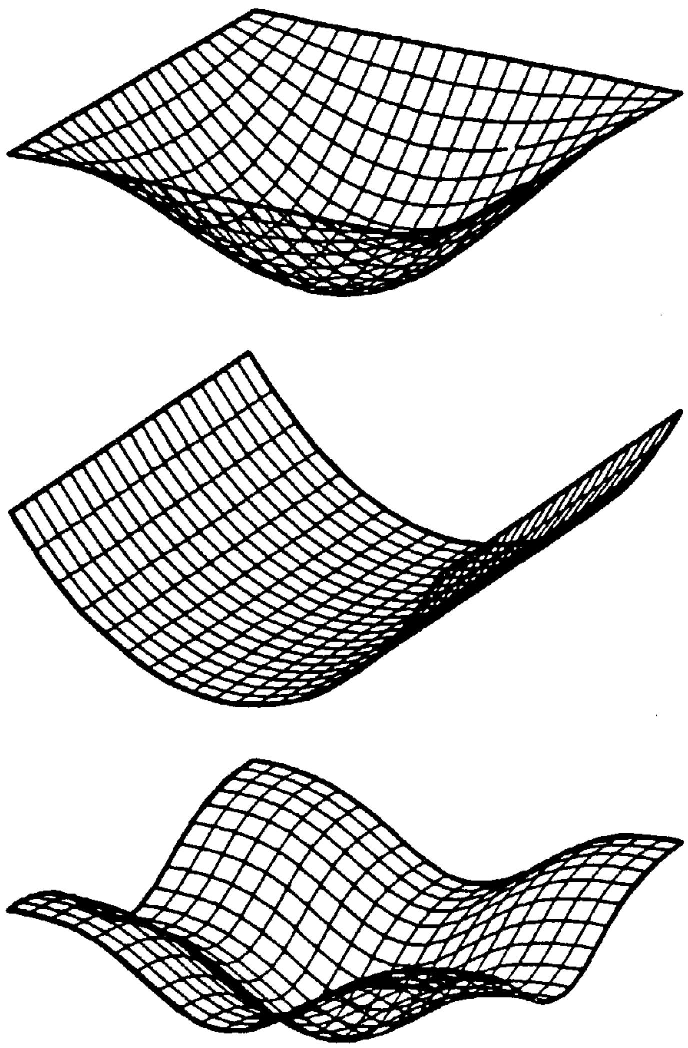


Figure 2.11: Assumed Deformed Shapes of Membranes in Unit Cell by (a) Molemkamp and Luger (1981); (b) Baldi and Nova (1984); and (c) Kramer et al. (1989)

good numerical solution to predicting membrane penetration volume change behavior. Figure 2.12 shows a comparison of the observed and analytically predicted membrane penetration behavior for coarse sand using the different membrane deformation models.

Empirical techniques and correlations have been developed by a number of investigators for evaluation of membrane compliance. Frydman et al. (1973) proposed one of the earliest empirical correlations between normalized membrane penetration (S) and mean particle size (D_{50}), previously shown in Figure 2.5.

Keikbusch and Schuppener (1977) presented a summary of their test results with a correlation between S and D_{50} in the form of a plot shown earlier as Figure 2.7. It is interesting to note that the relationship given by Keikbusch and Schuppener depicted a nonlinear slope between the normalized compliance (S) and $\log_{10}$ of soil particle size, with the values of normalized compliance increasing at an increasing rate with larger particle sizes. This trend of nonlinear correlation has since been concurred with by a number of subsequent investigations.

Ramana and Raju (1982) proposed an empirical equation for the estimation of unit membrane compliance (δV_m) as a function of mean particle size (D_{50}) based on the summarized data of previous investigators. This equation, applicable only to uniform soils whose D_{50} is between 0.08mm and 2.0mm, is given as:

$$\delta V_m = (D_{50}/80) \log(\sigma_3'/\sigma_{3,i}') \qquad \text{[Eq. 2.6]}$$

where:

δV_m = unit membrane penetration ($\Delta V_m / A_m$) in mm,

D_{50} = mean particle size (mm),

$\sigma_{3,i}'$ = initial effective confining stress (kN/m^2),

σ_3' = current effective confining stress (kN/m^2).

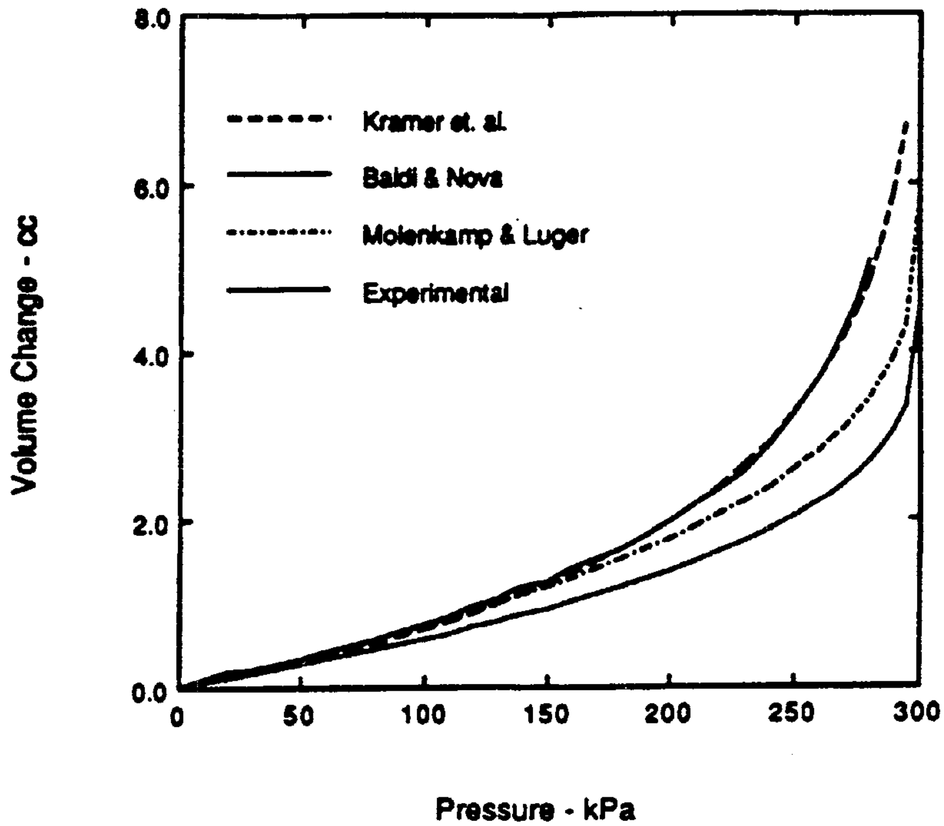


Figure 2.12: Comparison of Observed and Analytically Predicted Membrane Penetration Behavior for Coarse Sand:(Kramer and Sivaneswaran, 1989)

Baldi and Nova (1984) combined an analytical model with empiricle findings based on the data provided by previous investigations, including in their equation parameters pertaining to membrane thickness and geometric shape of material particles. The major drawback of that equation is that it is applicable only to uniform soils whose mean grain size is less than 0.5 mm and for effective confining stresses less than 400 KPa. The equation is given as:

$$\Delta V_m = 0.5(d_g / D) V_o [(\sigma_3{}' * d_g)/(E_m * t_m)]^{1/3} \qquad \text{[Eq. 2.7]}$$

where:

ΔV_m = membrane penetration volume change (cc),

d_g = mean grain size (mm),

D = sample diameter (cm),

V_o = initial sample volume (cc),

$\sigma_3{}'$ = effective confining pressure (kPa),

Em = Young's modulus of the membrane (kPa), and

t_m = thickness of the membrane (mm).

These investigators deduced from their studies and this equation that membrane compliance and its associated effects on undrained tests could be reduced by increasing sample diameter, a suggestion that has been confirmed by a number of investigators. They also proposed a non-linear relationship between normalized penetration (S) as a function of the $\log_{10}$ of mean particle size of a material by applying the above equation (Equation 2.7) to a conventional sample size and typical testing membrane. That relationship is shown in Figure 2.13, and is similar to that given by Keikbusch and Schuppener (1977). Baldi and Nova argued that their mathematical correlation fit well with the available data of previous investigators, suggesting that the scatter of some data points may be the result of

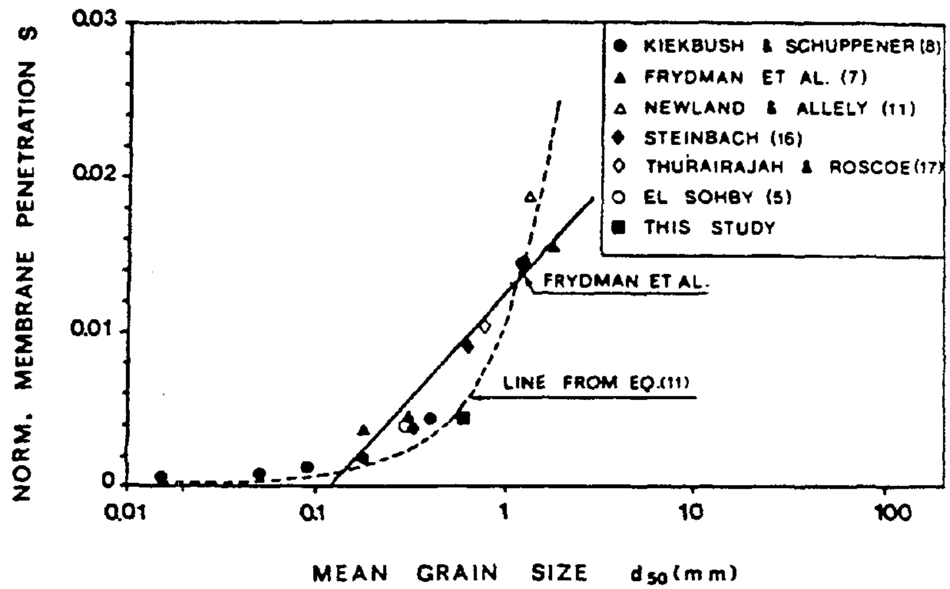


Figure 2.13: Proposed Relationship Between Mean Grain Size (D_{50}) and Normalized Penetration (S): (Baldi & Nova, 1984)

varied experimental testing techniques. It is important to note that for almost all of the investigations made to evaluate membrane compliance values up to that time, only uniformly graded materials had been tested. The few more broadly graded materials for which compliance measurements had been made did not fit their correlation nearly as well. Since that time, many more soil gradation types have been tested for volumetric compliance, showing that indeed, the relationships based on the data of uniformly graded soils did not hold for some of these other soil gradations.

In studies conducted as preliminary phases of this research investigation, reported by Anwar, Nicholson, and Seed (1989), a number of sandy soils with a wide range of grain sizes, from silty sands to coarse sands, and gradation types, including well graded and gap graded soils, were tested in order to evaluate their volumetric membrane compliance. Brief summary descriptions and gradation curves, as well as compliance measurement data for those soils tested are presented in Chapter 5. The compliance measurement tests performed on those soils was based on the "two sample scale model" method described in Section 2.2.2. The results of these compliance evaluations were combined with the results from other investigators using compliance measurement techniques judged likely to provide reasonably accurate results in order to derive a more widely applicable relationship between normalized compliance (S) and particle grain size. It was found that unit membrane compliance magnitude was much better correlated with smaller particle sizes (D_{20}) than with the mean grain size (D_{50}). This is not surprising, as membrane penetration is a phenomenon associated primarily with the interstitial voids between soil grains, and studies of both flow and "soil filter" characteristics have long recognized that soil particles finer than the mean grain size control the characteristics of these inter-particle voids.

It was also found that when soils were characterized by D_{20} grain size, sample gradation exerted a relatively minor influence on the value of normalized membrane compliance. Figures 2.14 and 2.15 show plots of the collective compliance measurement data using "representative" grain sizes of D_{50} and D_{20} respectively for silty and sandy soils. As can be seen in the relationship between S and D_{50}, there is a significant scatter of data points prevalent in the plot, which include the data points for most of the non-uniformly graded soils. Figure 2.15 shows the relationship between S and D_{20} for all of the same soils as in Figure 2.14. Clearly, the relationship is much more robust for a wide range of soil grain sizes and gradation types, and the use of D_{20} as a representative grain size for the characterization of compliance appears to remove much of the scatter from the relationship. Other representative or "characteristic" particle size indices were also tried (eg. D_{10}, D_{15}, D_{25}, etc.), but the correlation of S with D_{20} was found to be the strongest.

An equation was derived to "fit" the relationship between S and D_{20}, as illustrated in Figure 2.15, and is expressed as:

$$S = 0.0009\,[\log_{10}(D_{20}) + 2.1]^{3.57} \qquad \text{[Eq. 2.8]}$$

where:

S = membrane-compliance-induced volume change (cm^3) per square centimeter of membrane surface area per order of magnitude (per log-cycle) change in effective confining stress (σ_3'), and

D_{20} = the soil particle size such that 20% of the soil is "finer" (mm).

This empirically derived relationship provided a significantly improved basis for estimating normalized compliance values than had previously been proposed by

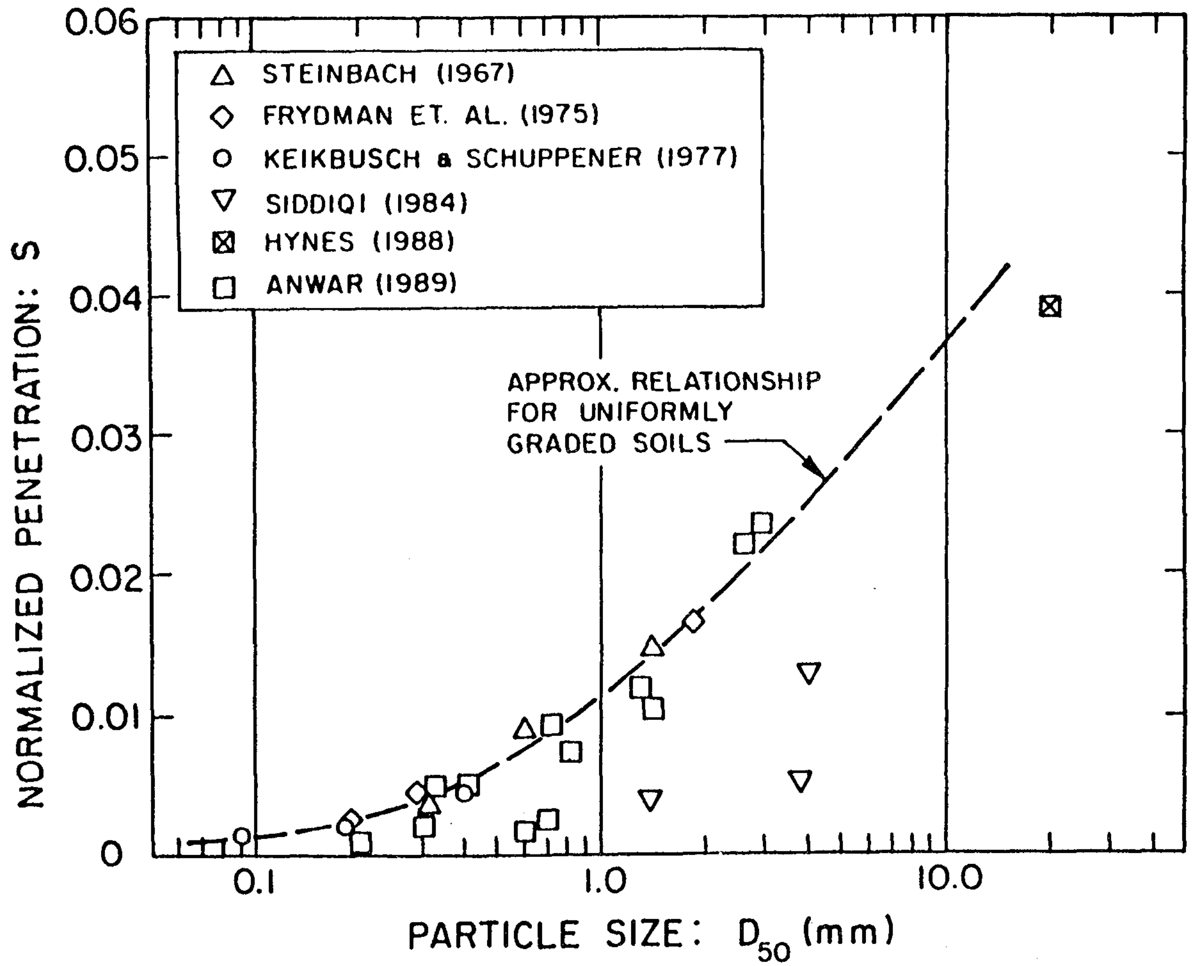


Figure 2.14: Relationship Between Normalized Unit Membrane Penetration (S) and D_{50}

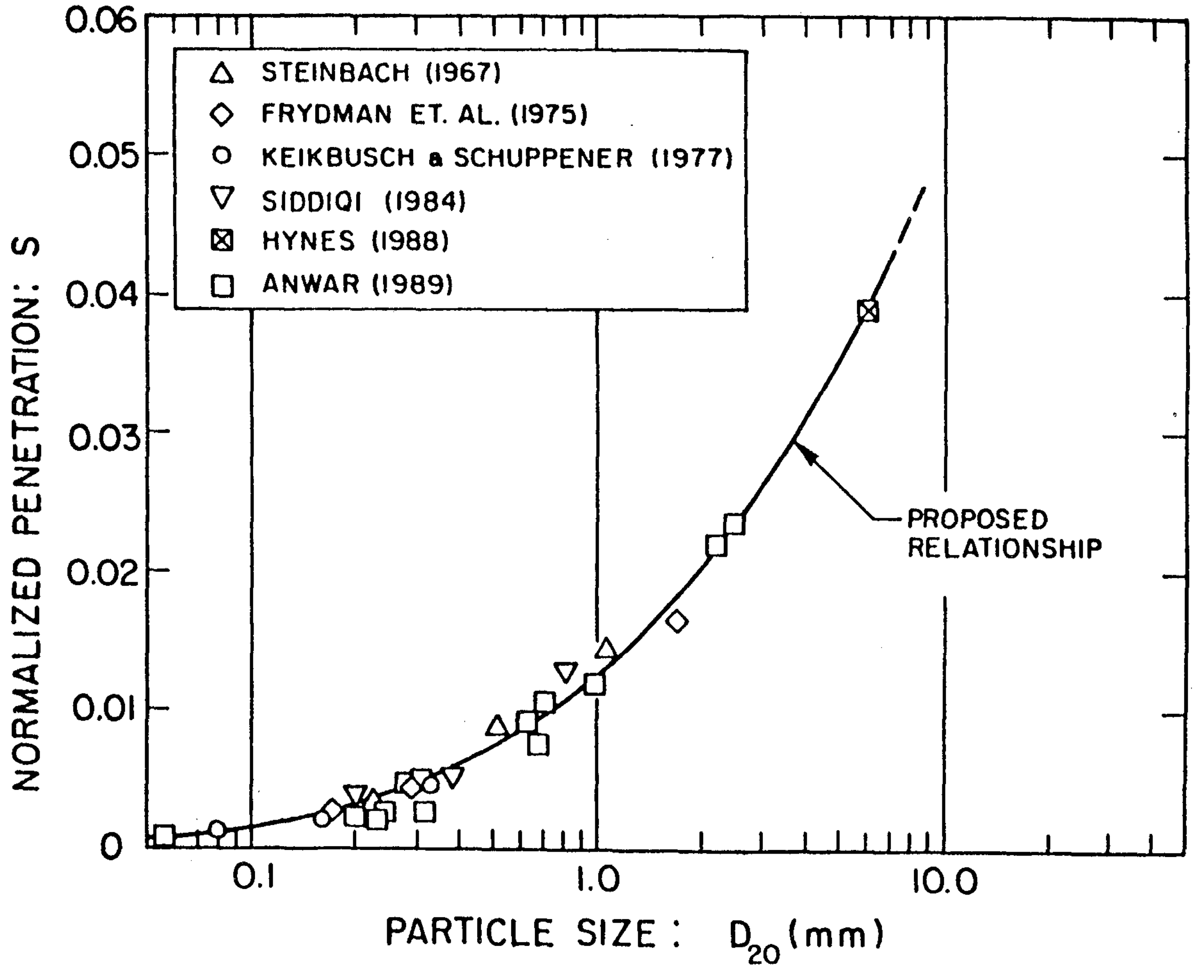


Figure 2.15: Relationship Between Normalized Unit Membrane Penetration (S) and D_{20}

correlations which were based on D_{50} grain size, which neglected the potential influence of non-uniform sample gradations.

Figure 2.16 schematically illustrates how the smaller particles in a soil gradation have a controlling influence on membrane compliance characteristics of more broadly graded soils. In Figure 2.16(a), a uniformly graded soil exibits considerably more penetration than the more broadly graded soil of Figure 2.16(b), where "finer" particles partially fill the peripheral sample voids between the larger soil grains.

2.3 Methods of Mitigating Compliance Effects

2.3.1 General

In order to obtain meaningful representative results for undrained tests of saturated granular materials, it would be most desirable to mitigate the adverse effects of membrane compliance from the tests. Numerous attempts at achieving this goal have been investigated over the last 25 years. The different types of methods that have been developed for mitigation of, or compensation for, membrane compliance effects can be grouped into three categories:

1. Use of larger sample sizes in order to reduce the the proportional impact of compliance effects.
2. Physical mitigation of compliance effects during actual undrained testing.
3. Post-testing correction of test results in order to compensate for compliance effects.

Each of these types of approaches that have been proposed, and the variations of each, will be reviewed in the following sections.

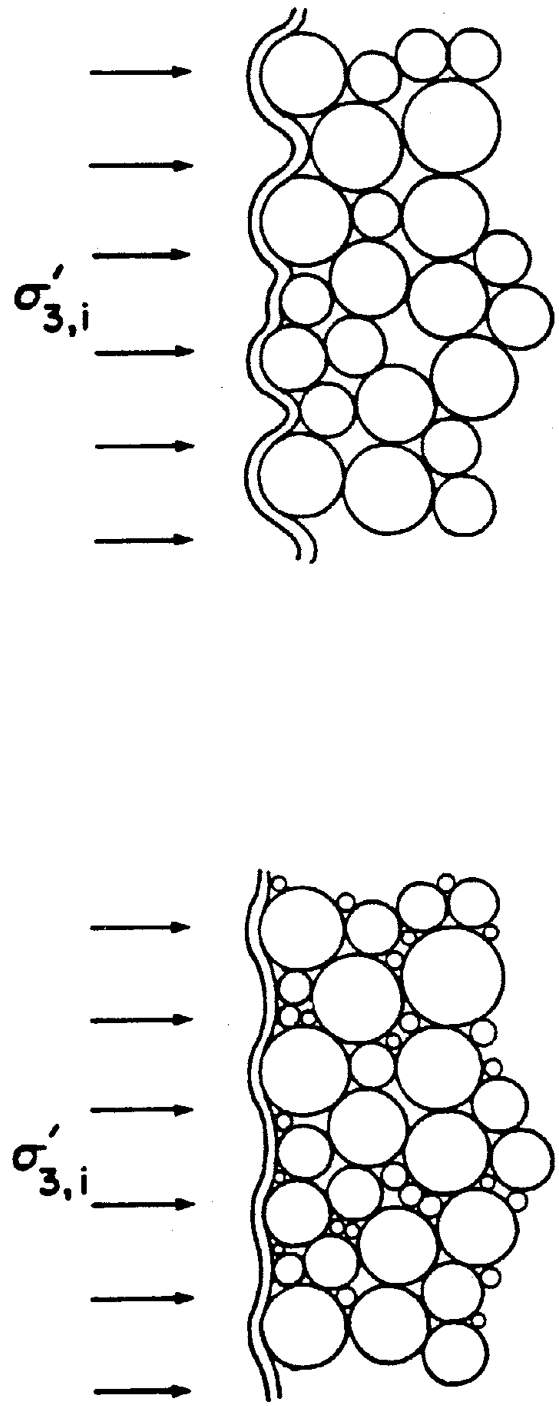


Figure 2.16: Schematic Illustration of the Controlling Influence of "Finer" Particles on the Membrane Compliance Characteristics of Broadly-Graded Soils

2.3.2 Use of Larger Sample Sizes

Conventional triaxial tests are typically performed on samples two to three inches in diameter, and provide good results for tests of silts and fine sands which experience minimal effects due to membrane compliance. For soils coarser than fine sands, the effects of membrane compliance are significant for these typical sample sizes. This results in unreliable and often unconservative test results for these coarser soils.

Newland and Allely (1959) first suggested that the effect of membrane compliance could be significantly reduced by testing larger diameter samples, and/or by using thicker (stiffer) membranes. These insightful suggestions were later tested by several investigators implementing a variety of methods.

The idea behind testing samples with larger diameters is to reduce the ratio of membrane surface area to sample volume, which in turn reduces the effects of compliance, as compliance is a function of the amount of membrane area with respect to the overall volume of the sample. As sample size is increased, the sample volume increases with the third power of sample diameter, while the sample membrane surface increases only with the square of the sample diameter. This geometric scale effect is demonstrated schematically in Figure 2.17. The practical applicability of this procedure was verified by a number of investigators including Wong, Seed and Chan (1975), Martin, Finn and Seed (1978), Chan (1978), and Seed, Anwar, and Nicholson (1989). The use of large scale testing apparatus (with sample diameters $\approx$ 12 inches), although too expensive for most applications, is one method to achieve representative tests for soils whose grain sizes do not exceed medium to coarse sand. However, for soils coarser than fine gravels, even these large scale testing facilities do not reduce the effects of membrane compliance enough to avoid significant errors in strength evaluations.

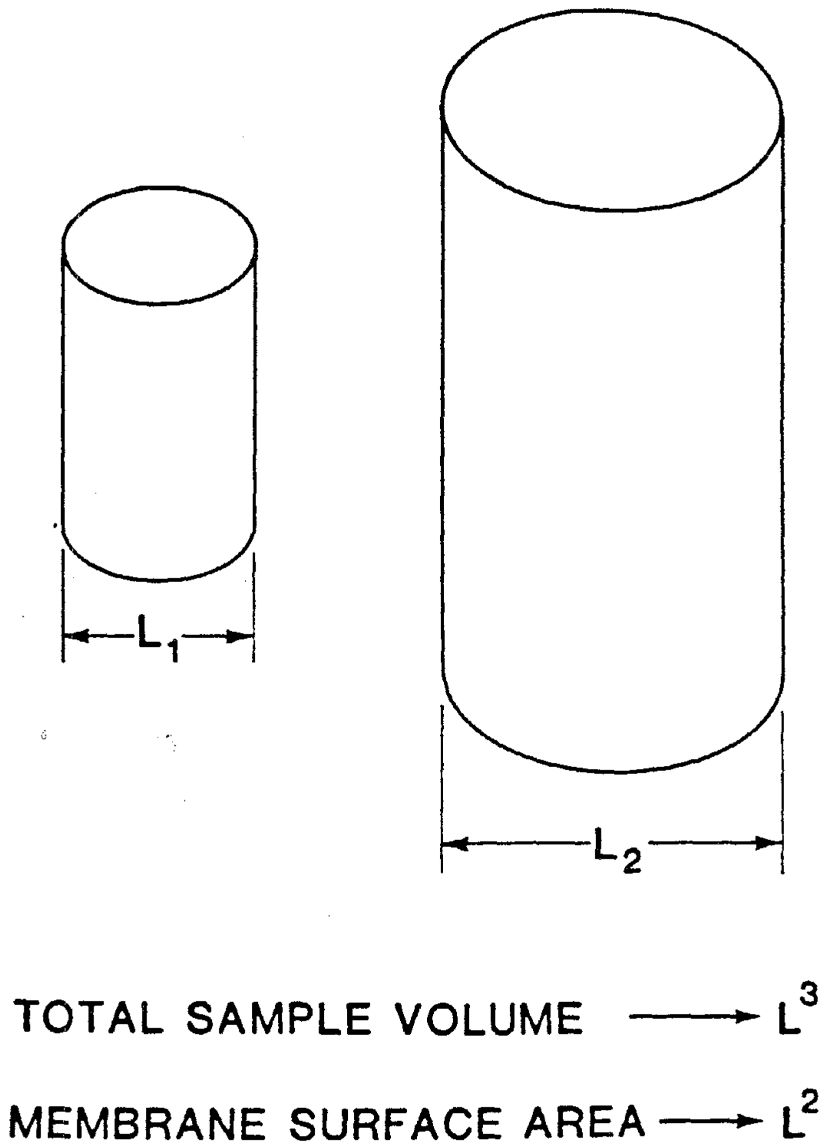


Figure 2.17: Scale Effects: Influence of Sample Size on the Ratio of Sample Volume to Membrane Surface Area

2.3.3 Physical Mitigation of Membrane Compliance During Testing

A number of approaches have been taken to achieve physical mitigation of membrane compliance effects during actual undrained testing. The effectiveness or usefulness of each method varies. The types of methods that have been attempted may be grouped into six basic categories as outlined in Table 2.1. This section presents a brief discussion of each of these methods.

(1) Use of protective plates adjacent to the rubber membrane used to confine the soil sample

A number of studies have been conducted in an attempt to mitigate membrane compliance effects by the use of a variety of protective plates placed between the membrane and soil sample. Chan (1972) experimented with a special membrane originally conceived for the purpose of preventing membrane rupture when testing rockfill at high confining stresses. The procedure involved covering the confined sample with high density polyethylene plates which would bridge over the external sample voids. It was noted that this method would also significantly reduce the effects of membrane compliance. Unfortunately, the polyethylene plates deformed plastically when significant confining stress was applied, and formed a stiff shell that caused serious errors in the measurement of axial load applied to the sample. Figure 2.18 illustrates this method.

Lade and Hernandez (1977) employed a system of overlapping brass plates placed between two membranes confining the soil sample, as illustrated in Figure 2.19(a). These brass plates were slightly curved in order to conform to the cylindrical shape of the sample. This procedure successfully mitigated much of the membrane compliance, but the amount of friction between the overlapping plates again caused the "armored membrane" to carry a great deal of the axial load

Table 2-1: <u>Methods for Mitigation of Membrane Compliance Effects During Undrained Testing</u>

1. Use of protective plates between the rubber membrane used to confine a soil sample and the sample soil grains.
2. Infilling of external peripheral voids in the membrane face at the condition of maximum membrane penetration prior to testing.
3. Filling of internal sample voids directly adjacent to the membrane prior to testing.
4. "Constant-volume" fully drained simple shear testing.
5. Maintaining a controlled confining cell volume outside of, and surrounding, a triaxial sample in order to preclude variation in membrane penetration.
6. Injection of water into otherwise "undrained" samples to offset pre-determined volume changes due to membrane compliance.

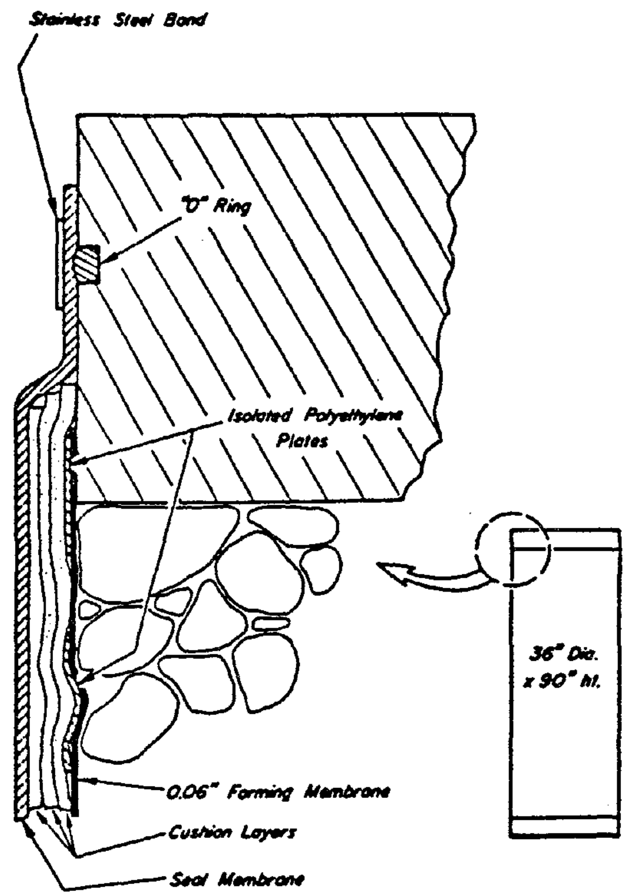
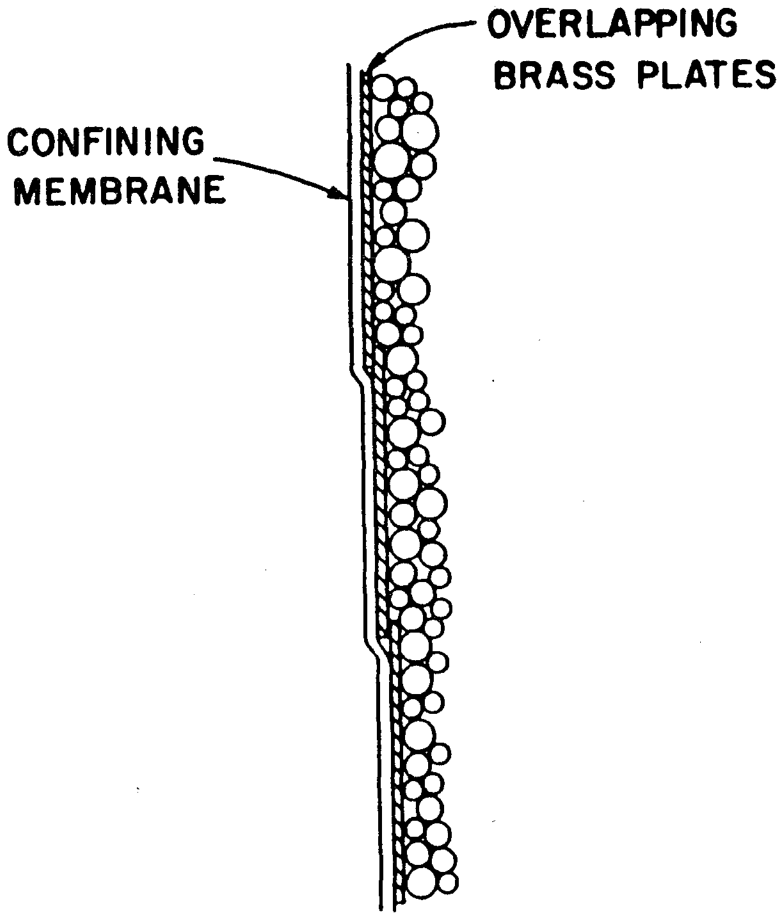
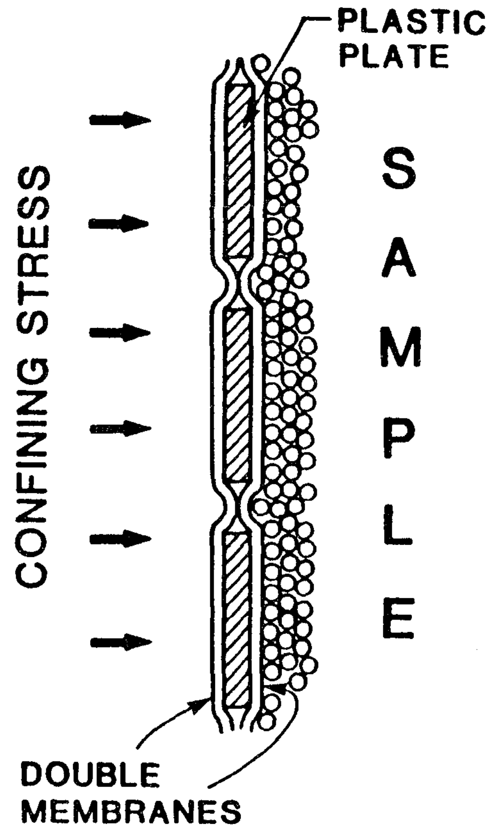


Figure 2.18: Special Membrane Developed for Testing Rockfills: (Chan, 1972)



(a) Overlapping Protective Plates



(b) Segmentally-Armored Membrane

Figure 2.19: <u>Schematic Representation of the Use of Overlapping or Segmented Plates to Mitigate Membrane Compliance Effects</u>

applied to the sample, which precluded the usefulness of this procedure. Raju and Venkataramana (1980) used polythene strips with silicone grease between the membrane and the sample in an attempt to overcome the frictional resistance encountered in previous investigations, but the results of their tests showed that membrane stiffness continued to contribute unacceptable bias to the test results.

Other investigators devised a variety of other plate configurations such as using segmentally armored-plates, as illustrated in Figure 2.19(b), to overcome the problems encountered with frictional forces. Unfortunately, while these configurations were found to greatly reduce the amount of membrane penetration where the plates were positioned, it was also found that they added an indeterminate ficticious contribution to the apparent strength and stiffness of the samples.

(2) Infilling of External Membrane Voids

Another method used in attempting to negate membrane compliance errors involved infilling the external peripheral voids in the membrane face at the condition of maximum membrane penetration. This method is schematically illustrated in Figure 2.20(a). Chan (1982) attempted to minimize the effects of membrane compliance by coating the outside of the membrane with liquid latex rubber. It was noted that during cyclic loading of 2.8-inch diameter samples, pore pressures were built up much faster and to higher pore pressure ratios than for specimens tested without the coated membranes. However, the effects that the coating had on membrane compliance could not be accurately quantified due to the wide scatter of test results. The scatter of data was attributed to an inconsistent thickness of the rubber coating which was not closely monitored.

Similar testing was performed by Raju and Venkataramana (1980), Wong (1983), and Torres (1983), in which some quantified results were obtained. These

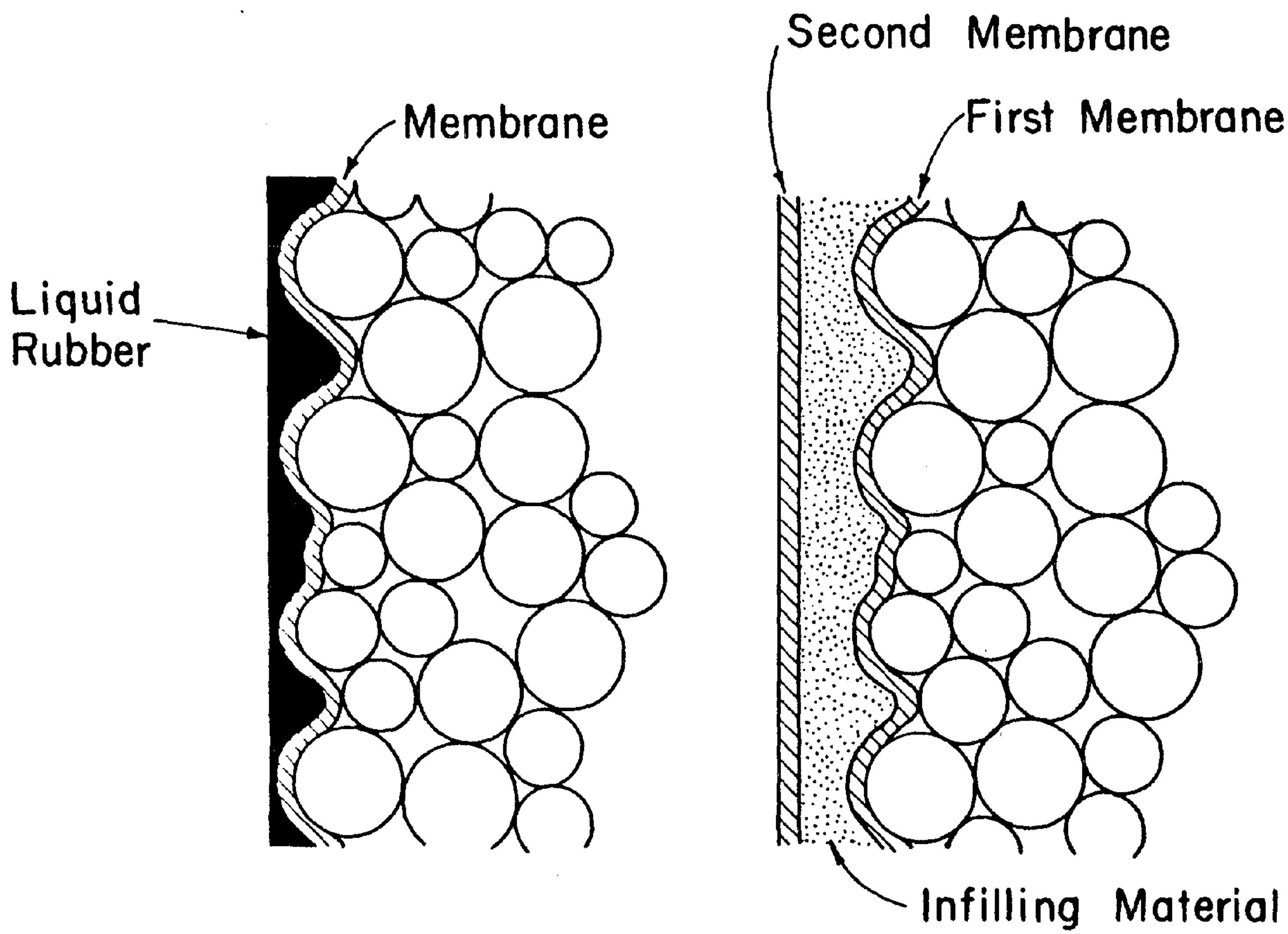


Figure 2.20: Infilling of External Voids in the Membrane Prior to Undrained Testing

tests showed that although the use of external rubber coating reduced membrane compliance effects significantly, corresponding calculated correction factors were much lower than those that had been predicted by Martin et al. (1978). This was an indication that the rubber coating did not completely eliminate the effects of membrane compliance and/or that the formation of a thicker, stiffer confining membrane contributes an indeterminate fictitious strength and stiffness to the sample.

Attempts to mitigate membrane compliance by using a layer of clay or sand between two confining membranes were made by Evans and Seed (1987) and Hynes (1988), as well as a number of unpublished efforts, but the results of these tests showed that this method would not prove to be useful in reducing the problem, as it was again accompanied by an added undesireable "ficticious" strength. This method is illustrated in Figure 2.20(b). Additional unpublished efforts have also been made that involved filling external sample membrane voids with other infilling materials between two membranes, and applying a separate back-pressure to the infilling material in order to minimize the effective confining stress applied to the material. But this did not appear to eliminate the problem, and in addition, sample preparation and testing procedures were excessively complicated and difficult.

(3) Filling of Internal Peripheral Sample Voids

Keikbusch and Schuppener (1977) experimented with coating the inside surface of confining membranes with liquid rubber so that it would penetrate the peripheral sample voids when confining pressure was applied. The liquid rubber was allowed to set under the initial confining stress before testing. The results of this method were to reduce the volumetric compliance to only 15% of the corresponding value for those samples tested without rubber coating. The effects of the additional thickness and strength added to the membrane on axial load measurements were not determined for these tests, which may invalidate this type of test for use in measuring actual sample strength. The problems arising from thickening the membrane with irregular amounts of rubber would be even more of a problem for gravelly soil specimens since the amount of rubber coating may have to be over an inch thick depending on the grain size distribution of the soil. Figure 2.21(a) illustrates the filling of internal peripheral sample voids with liquid rubber.

Raju and Venkataramana (1980) also tried a membrane system that involved filling of internal sample peripheral voids. In this study, a thin film of polyurethane was spread on the inside of the membrane prior to constructing the sample. They reported that by using coated membranes, compliance values were reduced to between 15% and 25% of the values measured when using uncoated membranes.

Some additional unpublished investigations have been performed at the University of California at Berkeley and at the U.S. Army Corps of Engineers Waterways Experiment Station in which soils have been used as an infilling material between the sample soil particles and the confining membrane, as illustrated in Figure 2.21(b). Once again it was found that a "ficticious" strength and stiffness was added to the sample, and furthermore, the infilling materials reportedly tended to "pump" into the interior of the samples during undrained loading, so that compliance mitigation became less effective as testing progressed.

In a recent study, Evans and Seed (1987) investigated the use of sluicing gravels with sand. This study had multiple purposes, including an attempt to mitigate membrane compliance effects. Sluicing of gravels with sand is not a new concept in itself, as it was conducted as part of the construction procedures for the Malpaso Canyon Dam in Peru and the Aswan High Dam in Egypt. However,

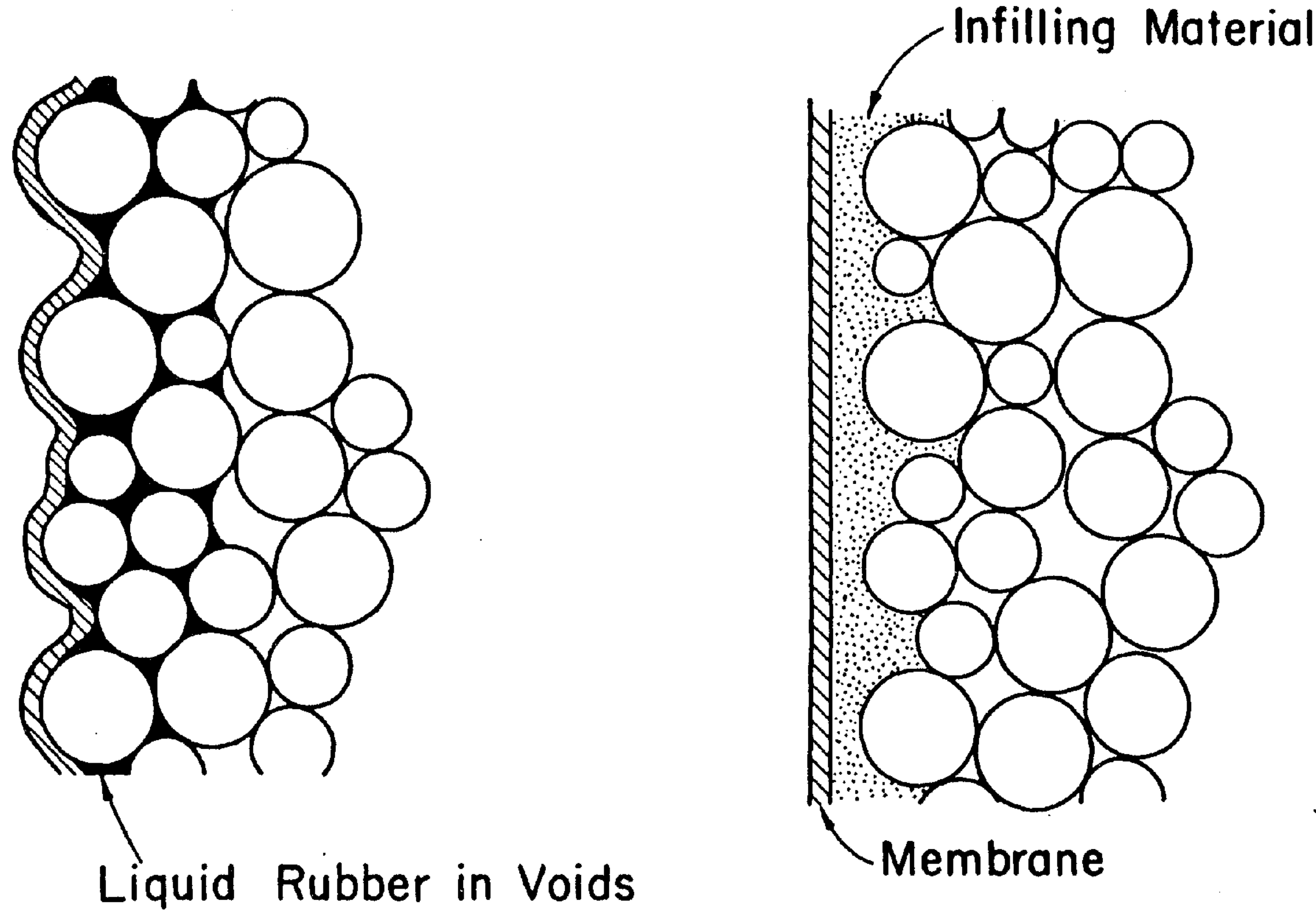


Figure 2.21: <u>Filling of Internal Peripheral Sample Voids</u>

sluicing had not previously been explored as a tool to mitigate compliance effects in undrained testing of soils. The idea behind this investigation was that by sluicing sand into the voids within gravel specimens, the sample/membrane contact would be much smoother, and therefore would exhibit greatly reduced membrane compliance during undrained cyclic loading. Clearly, this method is not applicable to testing of finer grained soils, but many of the methods described previously could not be realistically adapted to the testing of gravels. It was believed that the sluicing material was "loose" enough so as not to add significant strength to the samples and since the sluicing material was nearly continuous throughout the sample, there would be no tendency for the material to "pump" towards the interior of the sample as had been noted in earlier experiments.

Sluicing was accomplished by first constructing the complete gravel sample to a desired density and structure, and then replacing the water in the voids with the sluicing material. It is suggested that by constructing the samples in this manner, the individual gravel particles form a continuous, load carrying structure, while the sluiced material merely fills the void spaces in a loose state without adding to the strength of the sample.

Undrained cyclic triaxial tests were performed on 2.8-inch and 12-inch diameter samples of sluiced and unsluiced uniformly-graded gravel samples at various densities. From the results of these tests, it was concluded that sluicing of gravel specimens was a viable method of significantly reducing the effects of membrane compliance, although it did not completely mitigate compliance. It was noted that the sluicing sand may have had an effect on increasing cyclic strength by prohibiting grain rearrangement. But, despite the claim of the authors that those effects had only a secondary effect, this hypothesis has not been proven conclusively. Figure 2.22 shows some of the results obtained by using sluicing sand to reduce the effects of membrane compliance for gravel specimens.

(4) Constant Volume Fully-Drained Simple Shear Testing

Pickering (1973) and Moussa (1973, 1975) suggested the use of drained constant volume cyclic simple shear tests to obtain correct cyclic strength evaluations by eliminating the pore pressure generation which encompasses the errors induced by membrane compliance. This procedure, which is similar to the one first used by Bjerrum and Landua (1966) to investigate the behavior of quick clays, involves performing drained cyclic simple shear tests on samples that are kept at constant volume by locking the load ram once the initial vertical load has been applied. As the sample densifies during cyclic shearing, the sample tends to "drop away" from the locked top cap, thus reducing effective stresses. These changes in effective stresses are assumed to represent reductions in effective stresses that would occur as a result of pore pressure increases during undrained tests. A schematic representation of this testing setup is illustrated in Figure 2.23.

This technique of obtaining "representative" test results was examined by Finn and Vaid (1977) by comparing the results of conventional undrained cyclic simple shear tests to drained constant volume tests on similar sand. The result of the investigation was that liquefaction resistance was lower for the drained constant volume tests, indicating that compliance effects may have been successfully mitigated. Although the evidence to support this conclusion is very limited, this method may potentially have a future in mitigating membrane compliance effects once the procedure is verified. Unfortunately, like several of the other possible mitigation methods, this procedure is not likely to be adapted to performing representative tests on coarse grained soils due to the size limitations of the testing equipment.

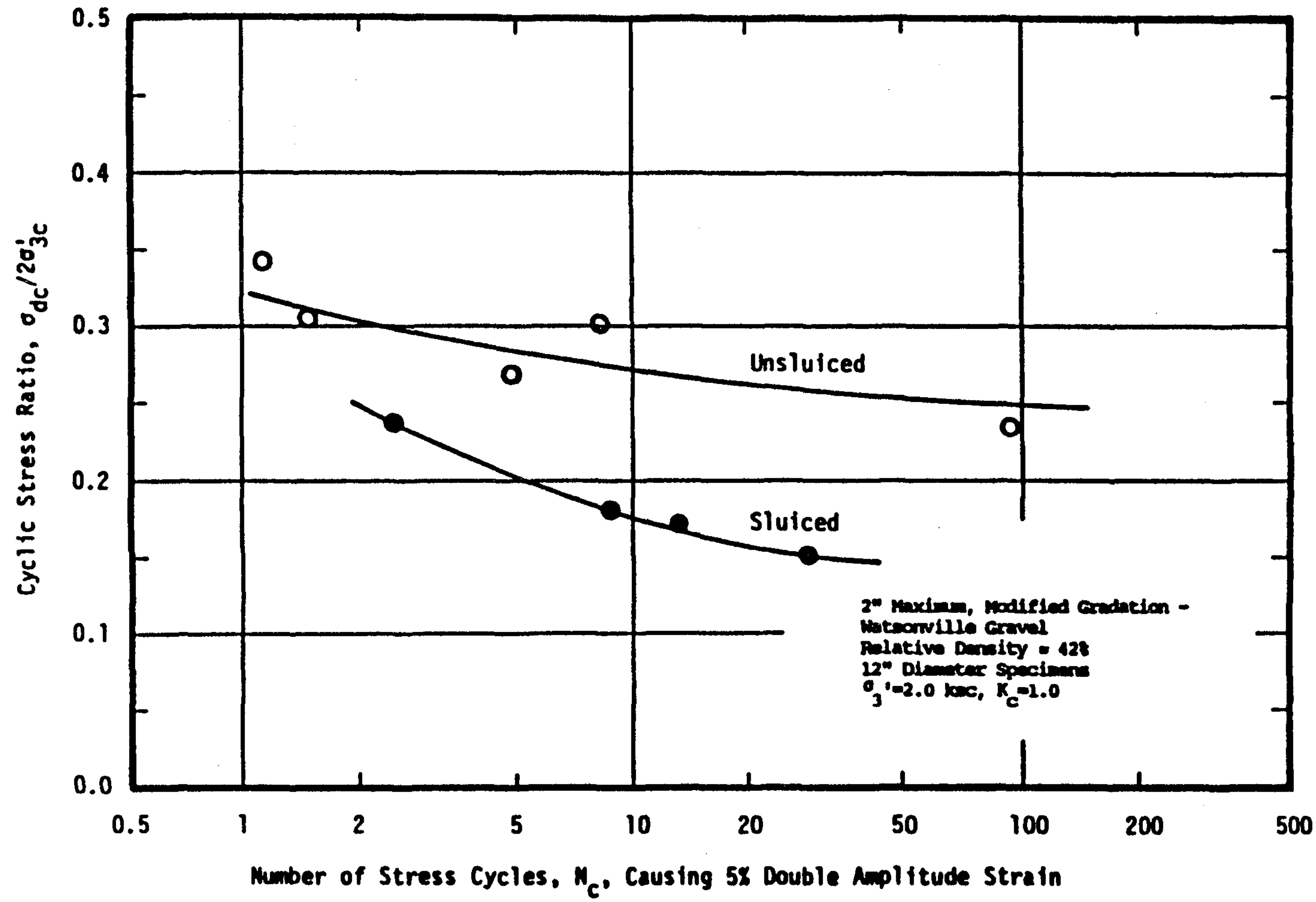


Figure 2.22: Comparison of Relationship Between Cyclic Stress Ratio and Number of Cycles Causing 5% Double Amplitude Strain For Sluiced and Unsluiced Samples:(Evans and Seed, 1987)

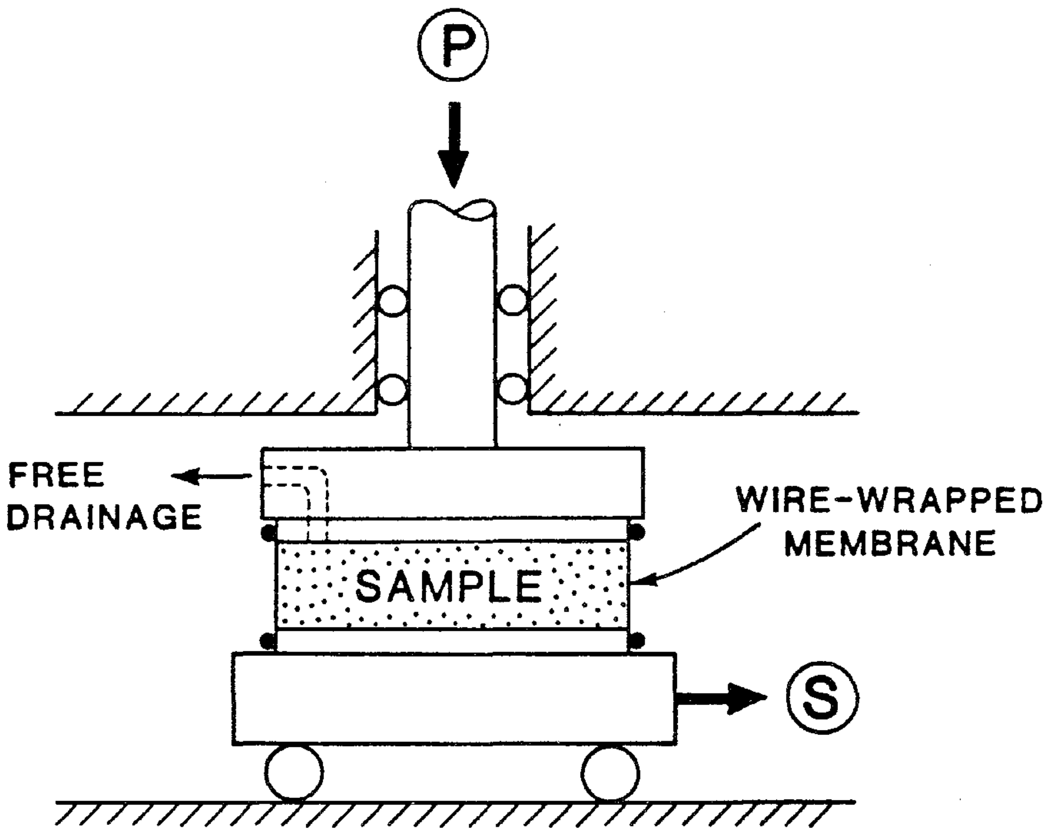


Figure 2.23: Constant-Volume Fully Drained Simple Shear Testing (Schematic)

(5) Maintaining a Controlled "Constant" Cell Volume

Some recently completed unpublished investigations, including one at the University of California at Berkeley, have investigated the possibility of preventing membrane compliance effects by controlling the confining cell volume that surrounds the soil sample being tested. This procedure is schematically illustrated in Figure 2.24. The idea behind this method is to offset the volume change in the cell due to the ingress or egress of the loading ram, by adding or removing an equal volume of water from the cell. If this is accomplished, then the volume of the sample itself undergoes no change, resulting in a true constant-volume test.

The studies employing this method have shown that, due to several technical difficulties, the necessary accuracy of controlling the required cell fluid volume cannot be accomplished, thus prohibiting the use of this method as a viable alternative for performing representative non-compliant tests.

(6) Compliance Mitigation/Compensation by Injection

A relatively new approach to mitigating membrane compliance, involves compensating for the compliance-induced sample volume changes by injecting and/or removing a volume of water equal to that amount previously pre-determined as a volumetric error for the current effective confining stress.

Compensated undrained tests were first performed by Raju and Venkataramana (1980) to assess the magnitude of pore pressures that would develop if the volumetric compliance was eliminated. Compensation was accomplished by manual injection of a volume of water equal to the amount of pre-determined volumetric compliance. The amount of volumetric compliance was determined using the modified central rod method as described by Raju and Sadasivian (1974). The additional fluid added to the system changes the effective confining pressure which causes additional volumetric compliance which , in turn,

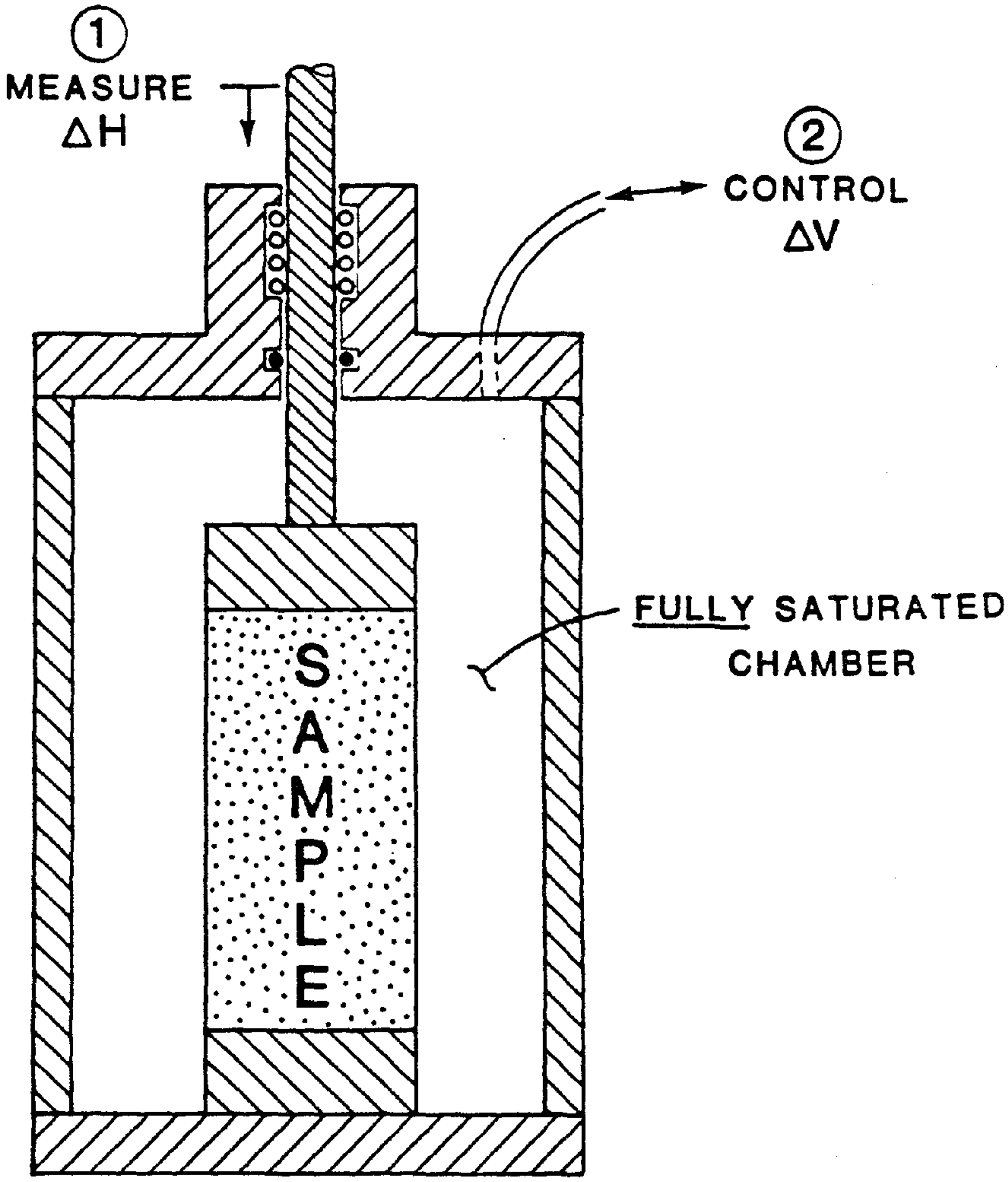
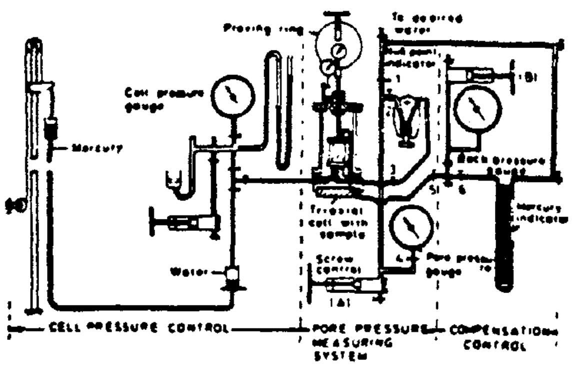


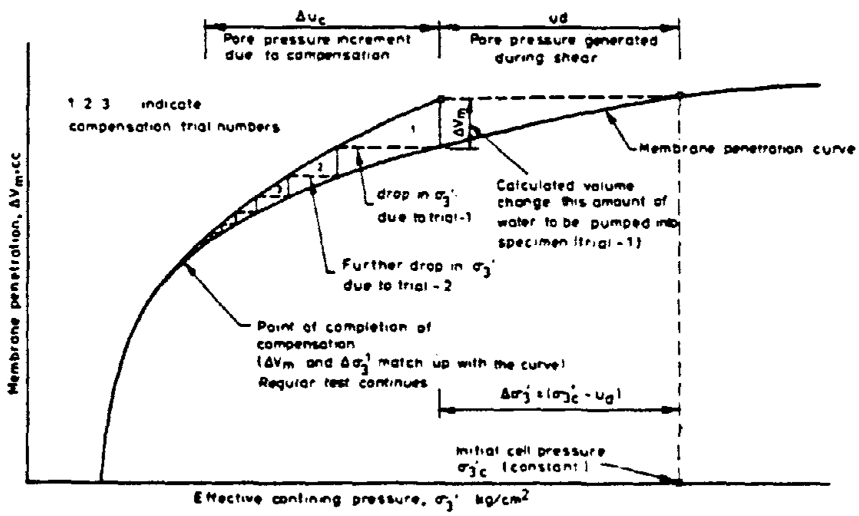
Figure 2.24: Schematic Illustration of Membrane Compliance Prevention by Controlling Confining Cell Volume

must be corrected for. Figure 2.25(b) illustrates the iterative correction process. This process continues until the pore pressure reaches a constant value which requires no further correction. This step-by-step procedure proved to be effective for obtaining more accurate test results, in which membrane compliance effects were significantly reduced. Unfortunately, the time involved to make the corrections for each increase in stress increment, as well as the lack of continuity between increments, was a serious drawback to the procedure.

The manual injection-correction method was further investigated by Ramana and Raju (1981) who applied it to monotonic loading and unloadings, and to cyclic tests, performing the cyclic tests at a rate of one cycle per minute in order to allow enough time for the manual corrections. Figure 2.25(a) shows the test apparatus used and Figure 2.25(b) illustrates the procedure used to compensate for compliance effects at each step of the test. Figures 2.26 and 2.27 show test results for undrained monotonic and cyclic tests performed on uniformly graded medium sand with and without implementation of the manual injection-correction procedure. Even though the tests were run slowly, the time required for the tedious stepwise manual injection process limited the number of iterations that could be performed during each step of the test. The iterative correction process was completed only two or three times during each monotonic loading test, and two to three times during each cycle of cyclic tests, so that compliance corrections were probably not complete. Further drawbacks to the manual injection-correction process were that: (a) The injection-correction only occurs at irregular and widely spaced intervals during testing so that sample volume is "correct" only at certain intervals during each test, (b) The periodic injection process necessitates injection of relatively large water volumes causing relatively large and sudden increases in pore pressure which may represent a significant loading mechanism whose effect



a) Test Set-Up



b) Stepwise Injection Process

Figure 2.25: Ramana and Raju's Manual Injection-Correction Procedure

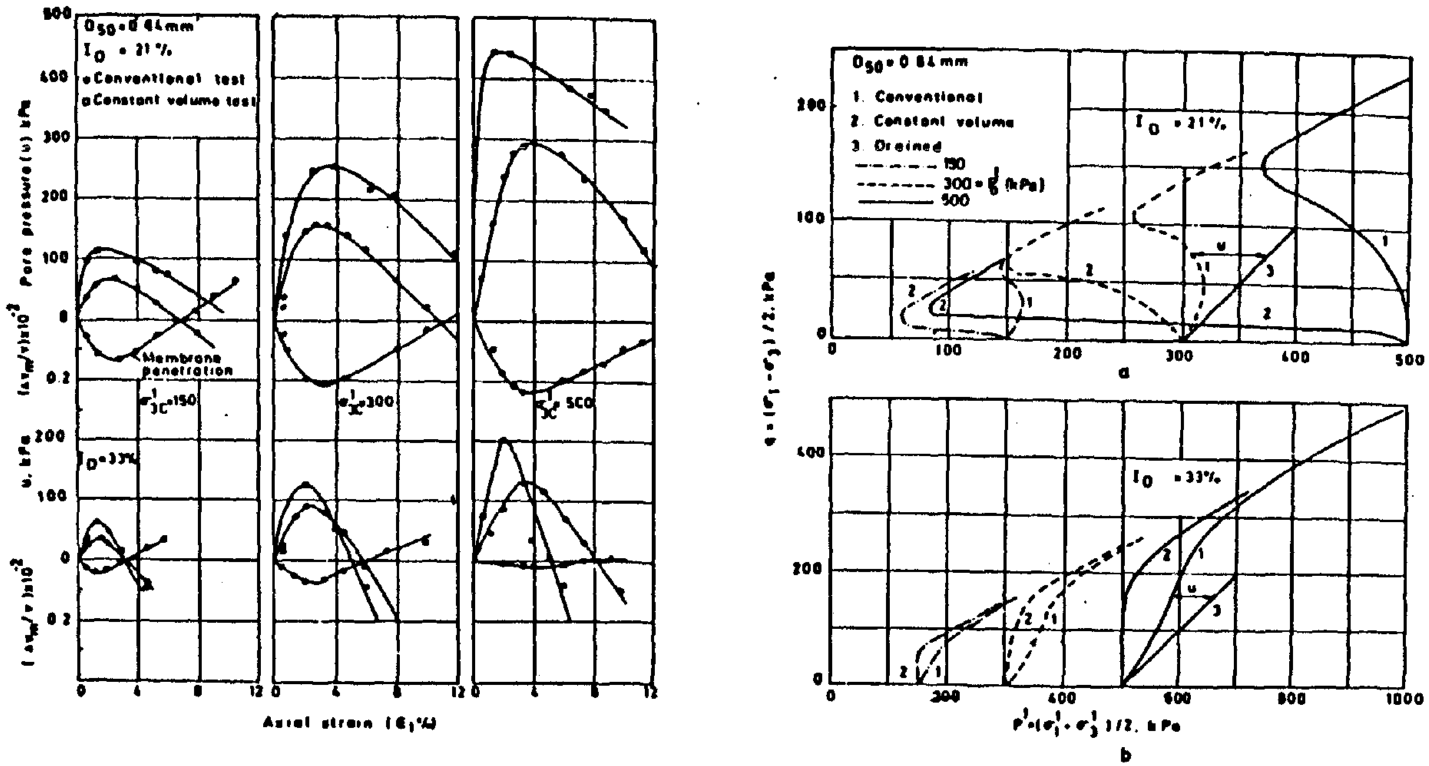


Figure 2.26: Undrained Monotonic Triaxial Test Results With and Without Manual Injection-Correction:(Ramana & Raju, 1981)

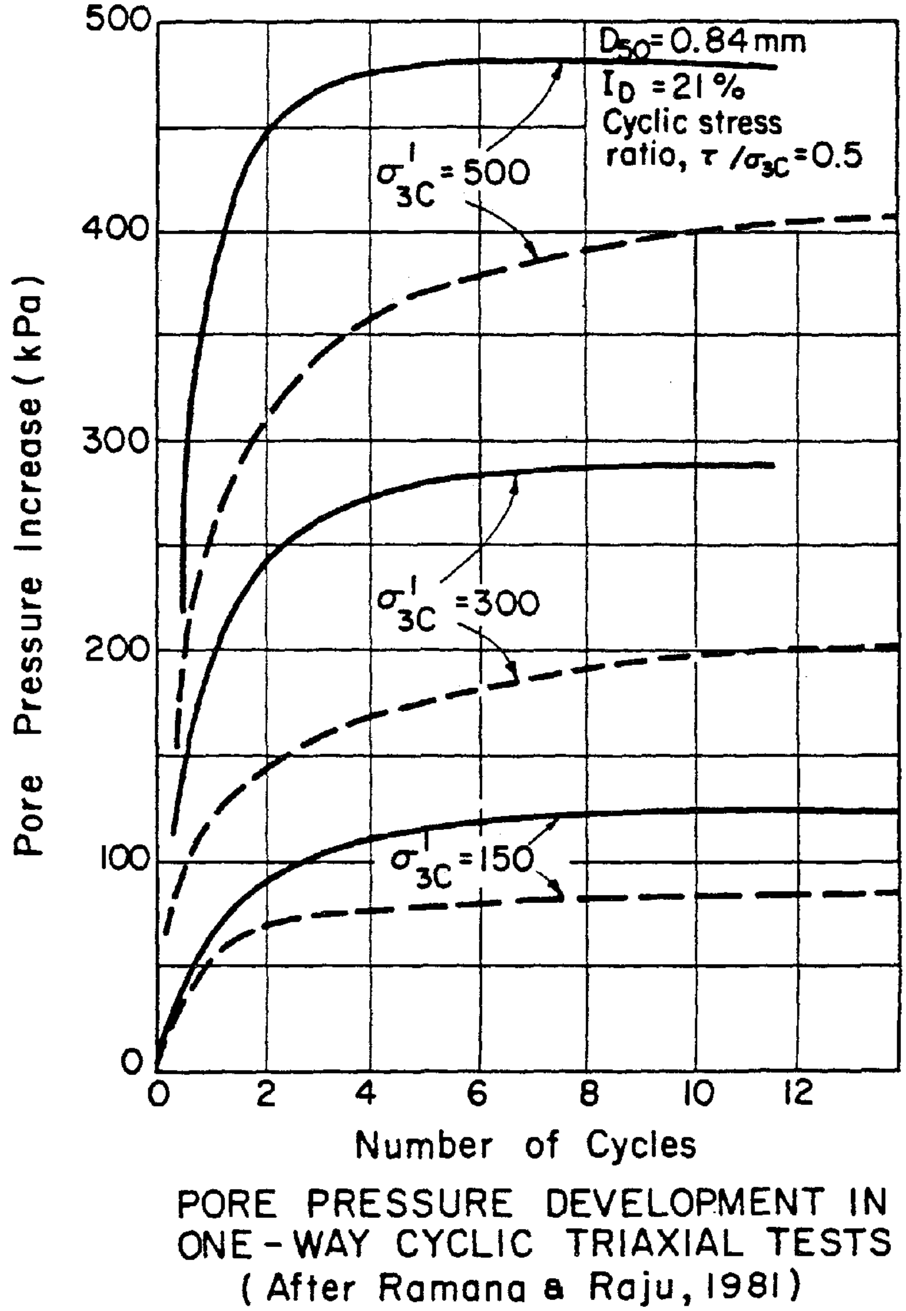


Figure 2.27: Undrained Cyclic Triaxial Test Results With and Without Manual Injection-Correction:(after Ramana & Raju, 1981)

on sample behavior is unknown, and (c) due in part to the aforementioned problems, Ramana and Raju were not able to quantitatively evaluate the effectiveness of the overall mitigation process. With these considerations in mind, the test results still showed that the procedure greatly increased the rate of pore pressure generation and apparently reduced the resistance to cyclic loading, which indicated that this procedure was a promising development in the research of physical mitigation of membrane compliance during undrained testing.

Laboratory techniques for performing continuous computer-controlled injection/removal corrections for conventional triaxial tests were developed by Seed and Anwar (1986), and Tokimatsu and Nakamura (1986). The pre-determination of volumetric membrane compliance was demonstrated to be reliably repeatable so that it could be characterized in such a manner that computer-controlled injection/removal could be performed based on monitored changes in the effective confining stress on the sample.

The computer-controlled membrane compensation system devised by Tokimatsu and Nakamura (1986) is illustrated schematically in Figure 2.28. Compensation was accomplished by adjusting the measured specimen volume by pneumatic pressure control of the monitored volumes in burettes, based on membrane compliance error measurements performed prior to undrained testing. Compliance measurements for use with these compensation tests were performed by the single sample unloading method described by Vaid and Negussey (1984).

The computer-controlled system used by Seed and Anwar (1986) consisted of an IBM PC-AT microcomputer and a GDS digital pressure/volume controller (injection piston) which was modified to bypass its internal circuitry in favor of direct control of the injection system by the microcomputer. A schematic

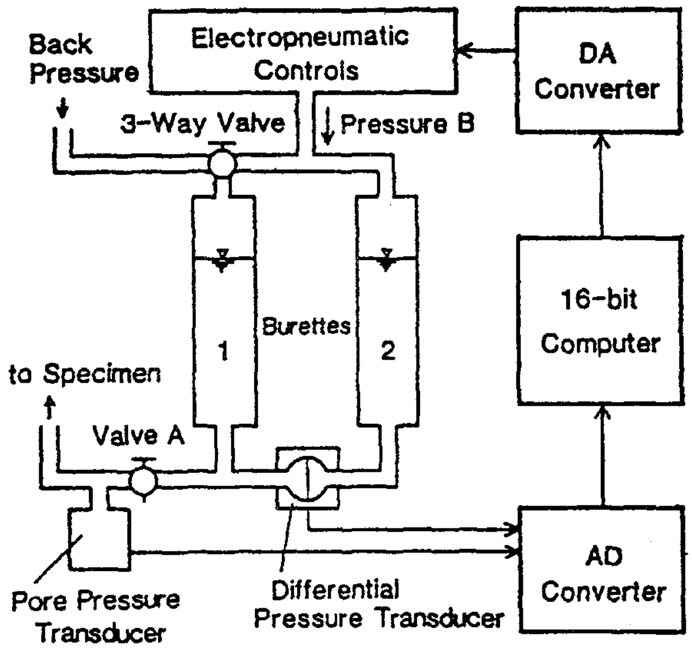


Figure 2.28: Schematic Diagram of the Pneumatic Membrane Compensation System: (Tokimatsu and Nakamura, 1986)

illustration of the computer-controlled injection system developed by Seed and Anwar is presented in Figure 2.29.

Implementation of the injection-mitigation system used by Seed and Anwar for undrained triaxial tests first involved pre-determining the unit membrane compliance for a given soil at a given density, for which the two sample scale model method (described in Section 2.2.2) was employed. The normalized unit membrane compliance S (cm^3 per cm^2 of membrane area per log cycle change in σ_3') was derived from the compliance measurements, and, along with sample dimensions, was entered into a computer program which would calculate the appropriate amount of water to inject to or remove from the sample for monitored changes in effective confining stress during undrained loading. Both monotonic and cylic tests were performed with and without employment of the computer-controlled injection-correction system on samples of two uniformly graded sands. Plots of the test results for those tests are shown in Figures 2.30 through 2.34. Individual test results for the tests performed on Monterey 16 sand are presented in Chapter 4.

Mathematical corrections of volumetric error based on the unit membrane compliance curve were used to theoretically "correct" the unmitigated monotonic test results (critical state conditions), giving support to the correctness of the test results obtained for those tests performed with compliance-mitigation. A plot of the monotonic test results performed with injection-mitigation as compared to the mathematical corrections of the unmitigated tests for Monterey 16 sand is shown in Figure 2.34.

2.3.4 Theoretical Post-Test Corrections

Post-testing corrections began as empirical corrections based on relationships developed to compare various sample characteristics and the amount

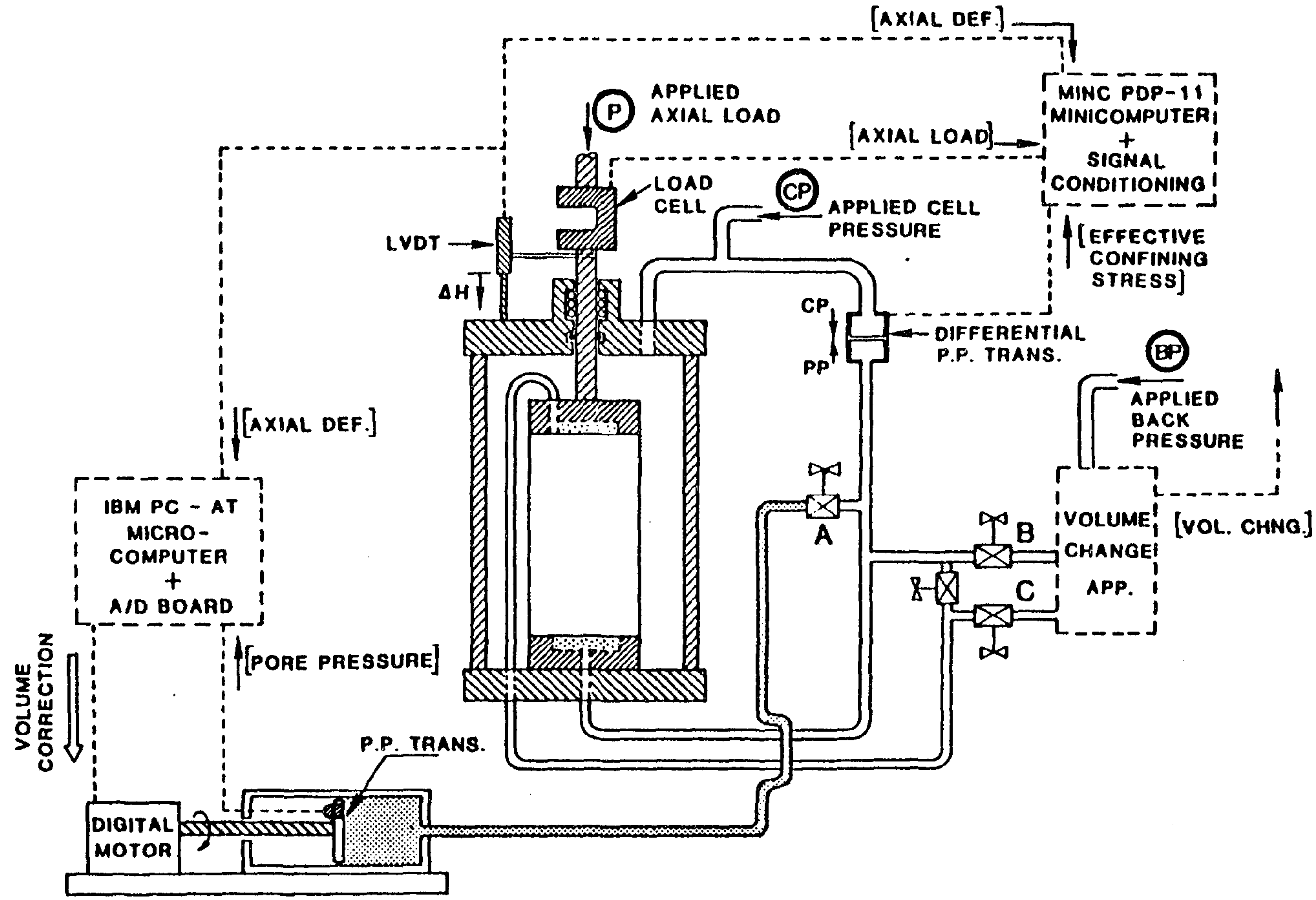


Figure 2.29: Schematic Illustration of Computer-Controlled Injection/Removal System for Membrane Compliance Mitigation During Undrained Triaxial Testing

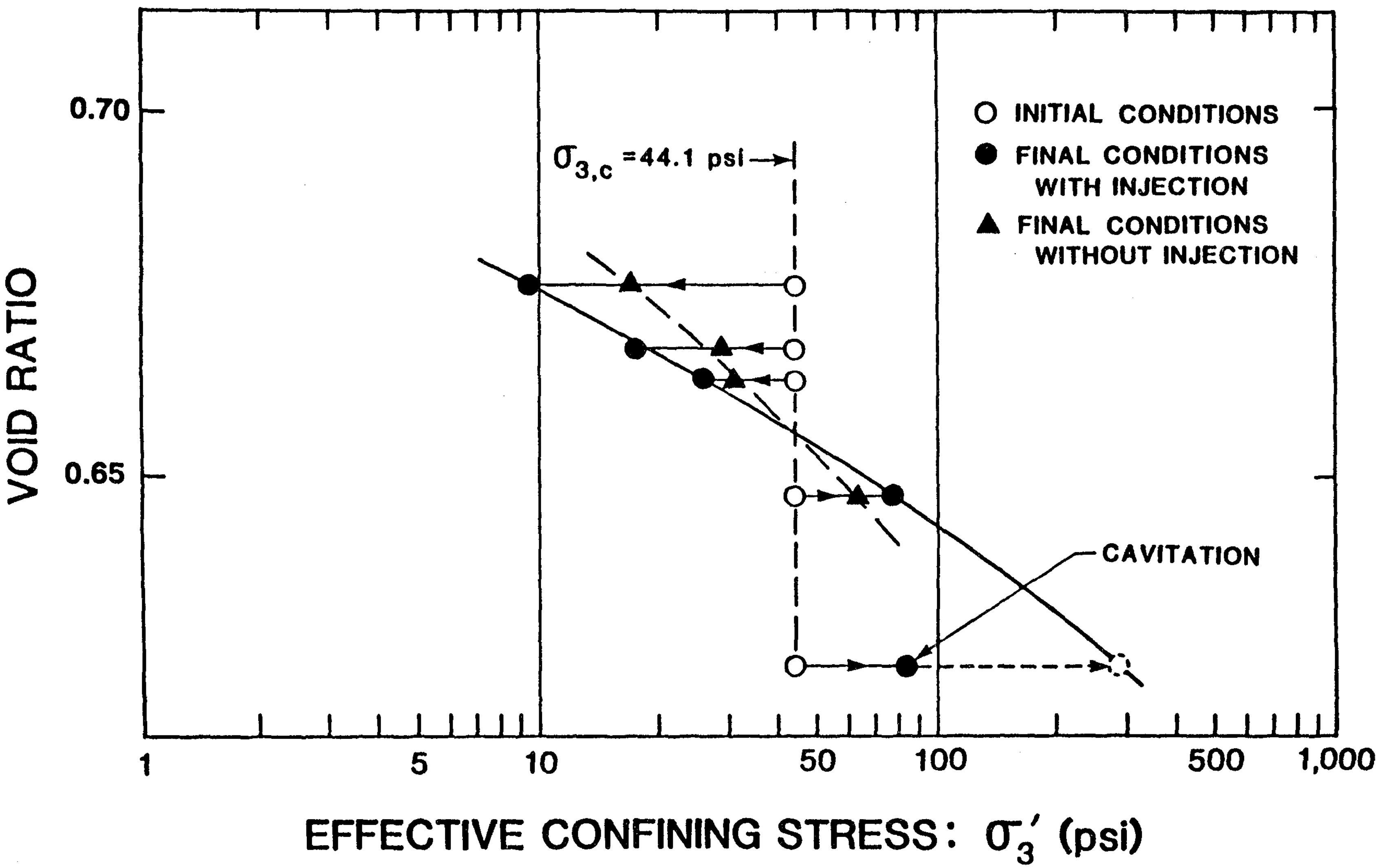


Figure 2.30: Critical-State Plot for $\overline{\text{IC}}$-U Triaxial Tests on Monterey 16 Sand With and Without Membrane Compliance Mitigation

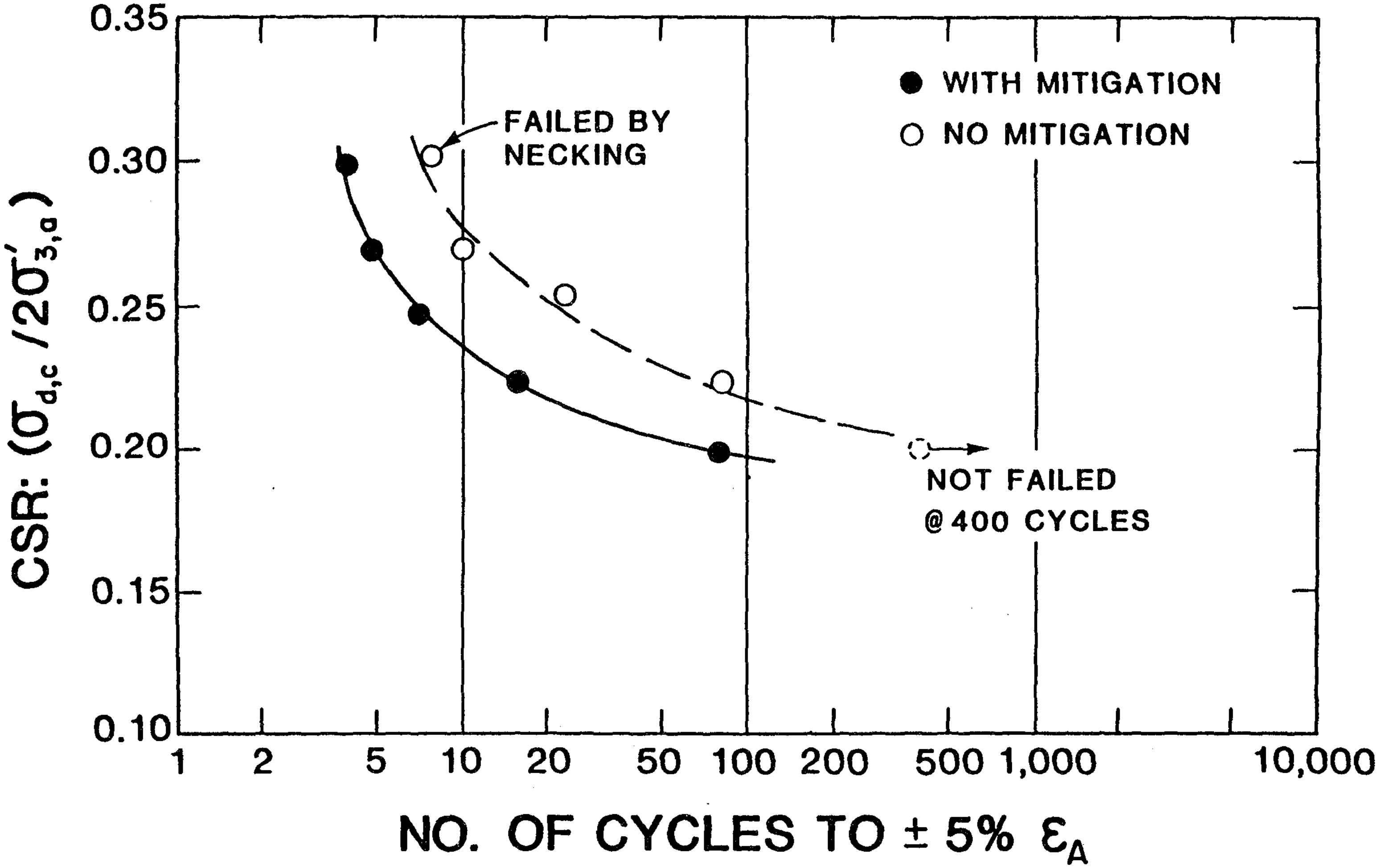


Figure 2.31: <u>Results of Isotropically Consolidated-Undrained Cyclic Triaxial Tests on Samples of Monterey 16 Sand at $D_R \approx 55\%$ (With and Without Mitigation of Membrane Compliance Effects)</u>

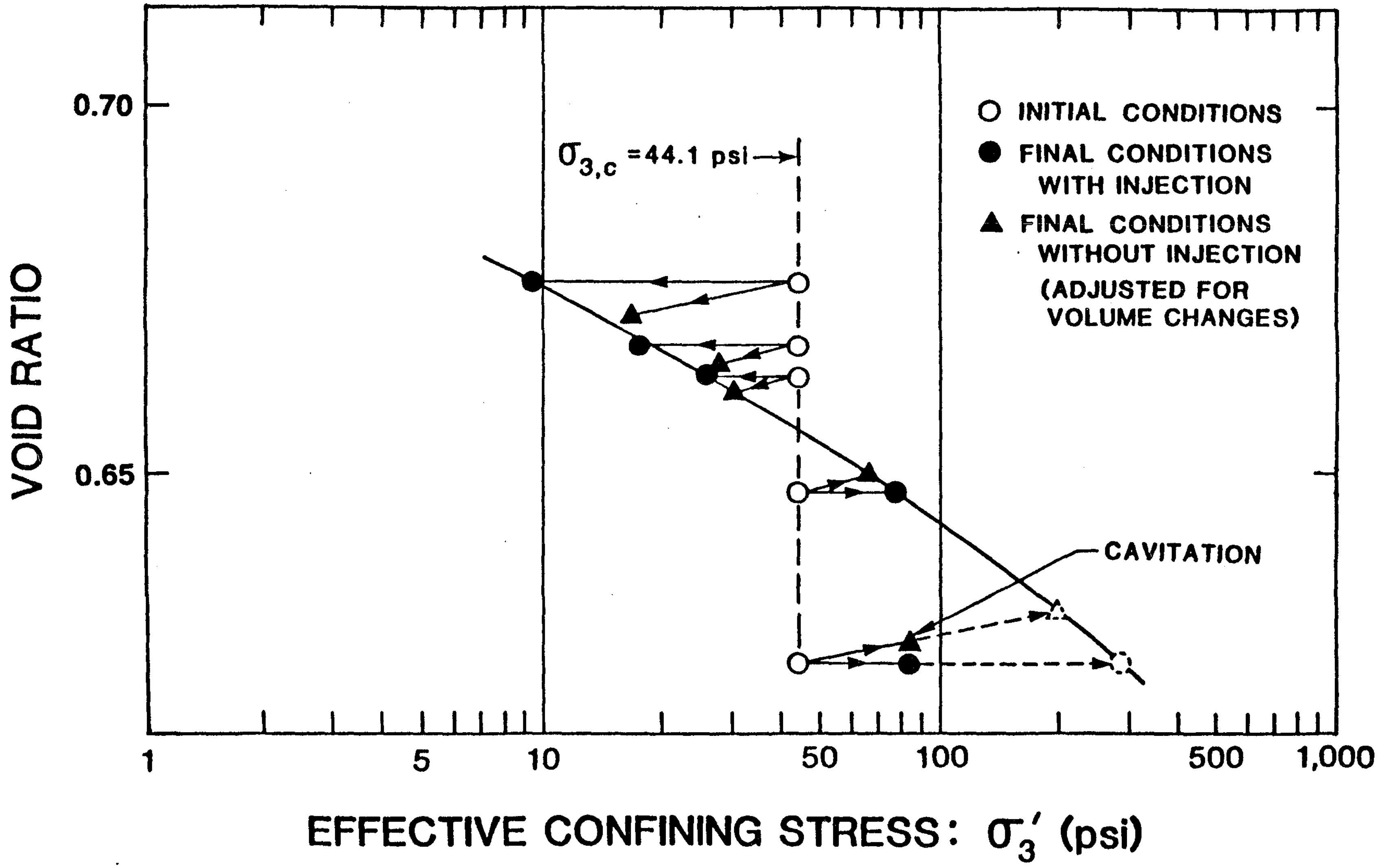


Figure 2.34: Critical-State Plot for $\overline{\text{IC-U}}$ Triaxial Tests on Monterey 16 Sand With and Without Membrane Compliance Mitigation (With Correction for Membrane Compliance-Induced Volume Changes)

of membrane compliance that could be expected for those characteristics at different effective confining stresses. Empirical equations for expected compliance magnitudes as functions of soil characteristics have been made by several investigators including Steinbach (1967), Ramana and Raju (1982), and Seed et al. (1989), and are continually being modified and refined as more updated testing data becomes available.

An early theoretical membrane compliance correction developed by DeAlba, Chan, and Seed (1975), was based on the analytical model of soil behavior proposed by Martin, Finn, and Seed (1975). This correction method involves the relationships between shear modulus, sample volumetric strain and shear strain from equivalent drained tests.

Assessment of the correction factor was determined by performing hydrostatic rebound tests on large-scale shake table samples of different heights. Volumetric strain due to membrane compliance was determined by extrapolating sample height to zero, at which point all of the change was due to membrane compliance.

A theoretical procedure for correcting conventional test results subsequent to the completion of the tests was developed by Martin, Finn, and Seed (1978). The procedure, which proposes a theoretical stress ratio correction for cyclic simple shear tests that can be applied to conventional triaxial tests, is based on the fundamental model for pore pressure generation devised by Martin et al. (1975). A membrane compliance ratio, equal to the ratio of the average slope of the rebound curve of the sample skeleton to that of the membrane penetration volume change curve, was computed for 1.4-inch diameter samples, and the results were then used to construct curves for 2.8-inch and 12-inch diameter samples by reducing the error in proportion to the inverse of the sample diameter. This correction procedure has since been modified to reflect the findings of more recent test data, as will be discussed in Chapter 3.

Baldi and Nova (1984) proposed a theoretical post-testing correction procedure similar to that of Martin et al. (1978) except that it was applicable only to single cycle tests, and was based on estimation of sample compressibility as a function of sample stress state.

Raju and Venkataramana (1980) proposed a post-testing correction procedure based on a simplified version of the theoretical model for compliance effects on pore pressure during undrained testing developed by Lade and Hernandez (1977).

A study by Raines et al. (1988), modelled the results of undrained monotonic and cyclic triaxial tests with and without the effects of membrane compliance, by numerical analyses. Test results for the "true undrained" tests was achieved by the computer-controlled method described by Seed and Anwar (1986). Initially, a constitutive model capable of modeling stress-strain and pore pressure development for actual uncorrected tests was developed, and appropriate parameters based on the uncorrected data were derived. After the compliance induced volumetric error was evaluated over the range of effective confining stresses for the tests, "corrected" or "true undrained" behavior could be represented by incrementally adjusting the pore pressures of the model, derived from the relationship between changes in pore pressure and void ratio.

The Modified Cam Clay constitutive model (Borja and Kavazanjian, 1985) coupled with a pore pressure analysis algorithm (Borja, 1986) was used to model the pore pressure generation in monotonic tests. Figure 2.35 shows a comparison of the stress vs. axial strain and pore pressure vs. axial strain for one set of modelled and actual test data, on a transformed plane appropriate to the

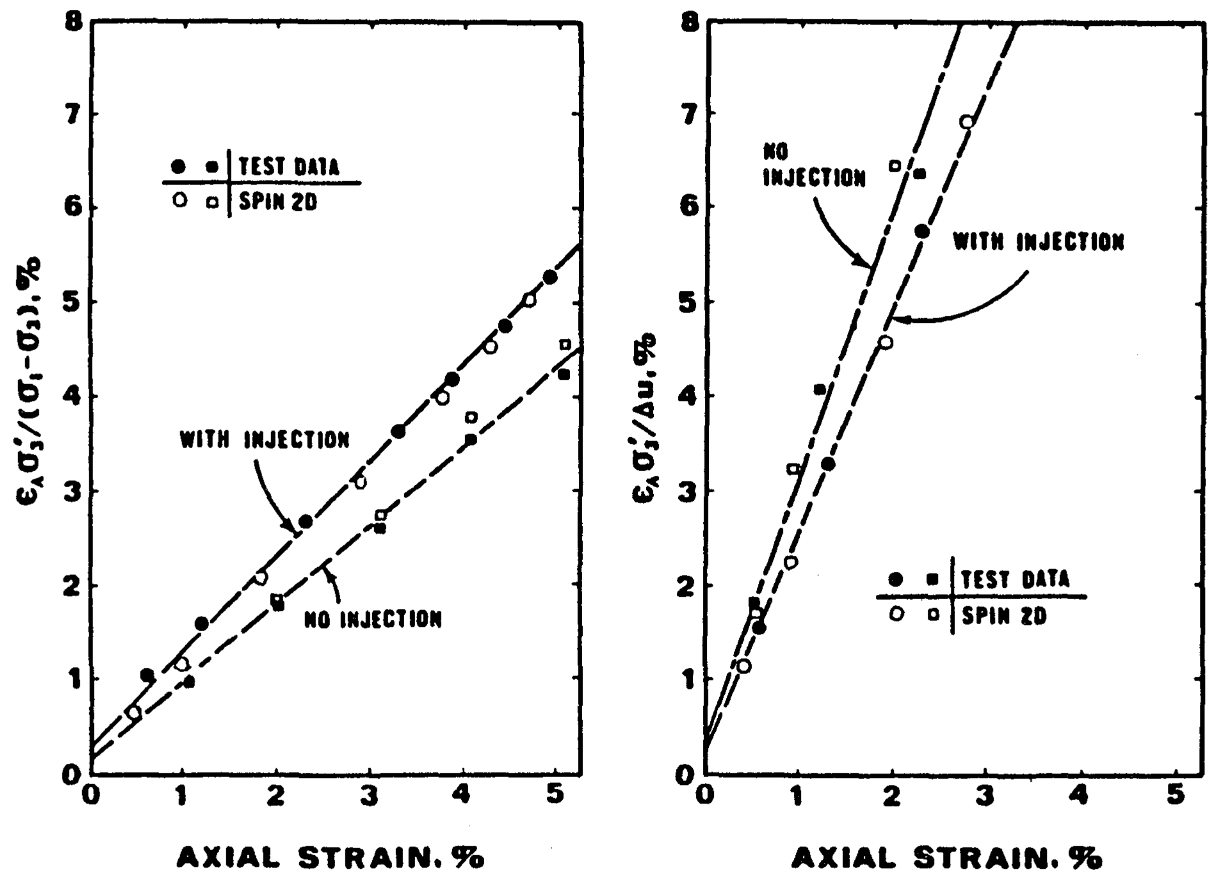


Figure 2.35: Numerically Modelled Stress-Strain and Pore Pressure-Strain vs. Test Results With and Without Membrane Compliance Mitigation: (Raines et al., 1987)

hyperbolic relationships implicitly assumed by the constitutive model. Similar agreement between modelled and actual data was also noted for the other sets of data studied.

The correction procedure developed by Martin, Finn, and Seed (1978) was modified by Evans and Seed (1987) to incorporate their test results of 12-inch diameter tests. This correction, given as a correction of cyclic stress ratios for given tests, was further modified by Hynes (1988) by correlating errors in stress ratios to D_{20} rather than D_{50} as suggested by Seed and Anwar (1986) to give better correlation with currently available penetration data. This updated model of the original procedure appeared to work well for the test results obtained by Hynes (1988).

A correction method has recently been proposed by Kramer and Sivaneswaran (1989) which incorporates a more accurate uniform assumption of the deformed shape of a unit cell of membrane into a constitutive bounding-surface plasticity model. The results of corrections to conventional "uncorrected" tests compared favorably to static tests conducted as a part of that same investigation in which manual injection was used to offset pre-determined volumetric errors. A drawback to that correction method is that the parameters that are necessary for the constitutive model must be obtained from a number of laboratory tests. This precludes the use of this type of correction method for application to previously performed tests from which the necessary constitutive model parameters may not be obtainable. It may also be recognized that all of the evidence supporting the results of the correction method come from tests on one type of uniformly graded sand and one confining membrane type. Whether or not this method will also work for other configurations of materials, membrane types, and sample sizes has not been shown conclusively.

Before tests are performed in which membrane compliance effects are completely and conclusively mitigated, the accuracy of theoretical corrections cannot be proved. Evidence produced as a part of this study and reported by Seed, Anwar, and Nicholson (1989), is described in Chapter 4 to show that the method of continuous computer-controlled injection/removal of water to offset the volumetric error induced by membrane compliance, appears to completely mitigate the deleterious effects from tests conducted on compliant materials. From the test results reported by Seed, Anwar, and Nicholson, and the additional data from tests carried out as part of this study, the accuracy of previous and future theoretical and analytical corrections may be able to be more accurately evaluated.

CHAPTER 3

PREVIOUS TESTING OF GRAVELS

3.1 Introduction

Some of the reasons why there have been so few tests performed on gravelly soils have been explained in Chapter 1. These include the fact that until recently gravels had not generally been considered to be liquefiable, and that many of the undrained static strength evaluations implied that gravels were inherently very strong. It is now understood that these strength evaluations may have been unconservatively erroneous due to apparent errors encountered as a result of membrane compliance during undrained tests. Another reason that testing of gravelly soils has not been extensively researched is the necessary sample sizes that are required to avoid stress concentration problems as pointed out in studies by Holtz and Gibbs (1956), Leslie (1963), Wong, Seed, and Chan (1975), and others. These previous investigations have suggested that the ratio of sample diameter to maximum particle size should be on the order of 6 to 8, depending on the gradation of the soil. For many gravels commonly used for construction or used as naturally occurring foundations, the maximum grain size dictates the use of 12-inch diameter or larger samples in order to avoid stress concentration problems in triaxial tests. The number of facilities which are capable of conducting tests on samples of these sizes are few, and the cost of performing the tests is usually prohibitive except for research.

3.2 Static Strength Evaluations of Gravelly Soils

The amount of test data available to date for strength evaluations of gravelly soils is very limited. This is primarily due to the limiting size of available testing equipment, and further complicated by testing problems involved when attempting to determine reasonable strength parameters for these materials. One possible solution to the problems encountered in testing gravelly soils was to make correlations with tests performed on finer grained material. Among the first to propose a method for estimating the static strength of oversized rockfill from laboratory tests of finer grained material were Zeller and Wullimann (1957). They proposed that by scalping the large particles from the material, a series of tests could be made on remaining graded fragments with different maximum particle sizes, and that the static strength of the total material could then be extrapolated to the maximum particle size of the rockfill gradation.

Lowe (1964) conducted tests on 6-inch diameter specimens with a maximum particle size of 1.5 inches employing the assumption that a parallel gradation to the prototype material (with 12-inch maximum particle size) should give similar shear strength results.

Marachi et al. (1969) further investigated the use of parallel gradations suggested by Lowe to predict friction angles of rockfill material from tests performed on small-scale specimens. They conducted tests on a wide range of parallel gradations by using sample diameters of 36 inches, 12 inches, and 2.8 inches, with maximum particle sizes of 6 inches, 2 inches, and 0.5 inches respectively. Their findings were that the different sample sizes gave similar strengths based on friction angles, with variations between different sample sizes amounting to less than 10%.

Siddiqi (1984) noted some inherent problems with estimating the strengths of gravelly soils by testing parallel gradations. Among them was the fact that parallel gradations tend to introduce undesirably high fractions of fines to the tested material which can have significant effects on soil strengths, especially in evaluating the resistance of soils to cyclic loading.

Torrey and Donaghe (1985) proposed that a better method of preparing a modelled gradation for laboratory testing was by scalping and replacement of the oversized particles. From the undrained triaxial compression tests performed as part of those studies, it was concluded that the scalp and replacement procedure provided conservative strength parameters for their earth-rock prototype material based on total stresses. The undrained strengths of the total earth-rock material were considerably larger than the undrained strengths from tests made of the minus No. 4 sieve fraction of the soil gradation, suggesting that scalping may be over-conservative.

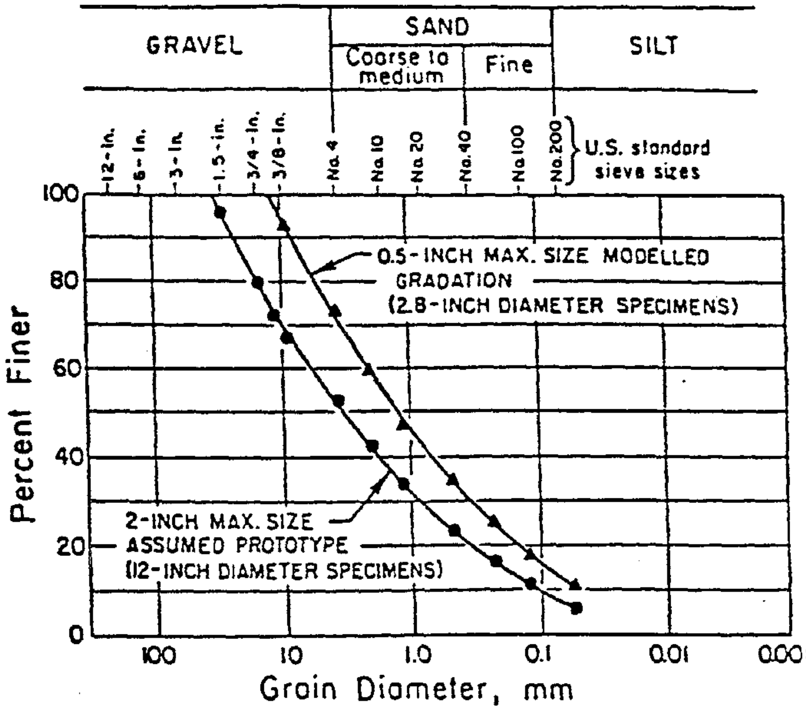
3.3 Cyclic Testing of Gravelly Soils

The amount of cyclic testing that has been performed on gravelly soils prior to this study has been even more limited than undrained static testing since the resistance of gravelly soils had until recently not been a major concern. Among the researchers that have contributed to the small database available are Lee and Fitton (1969), Wong, Seed, and Chan (1974), Banerjee, Seed, and Chan (1979), Siddiqi et al. (1987), Evans and Seed (1987), and Hynes (1988).

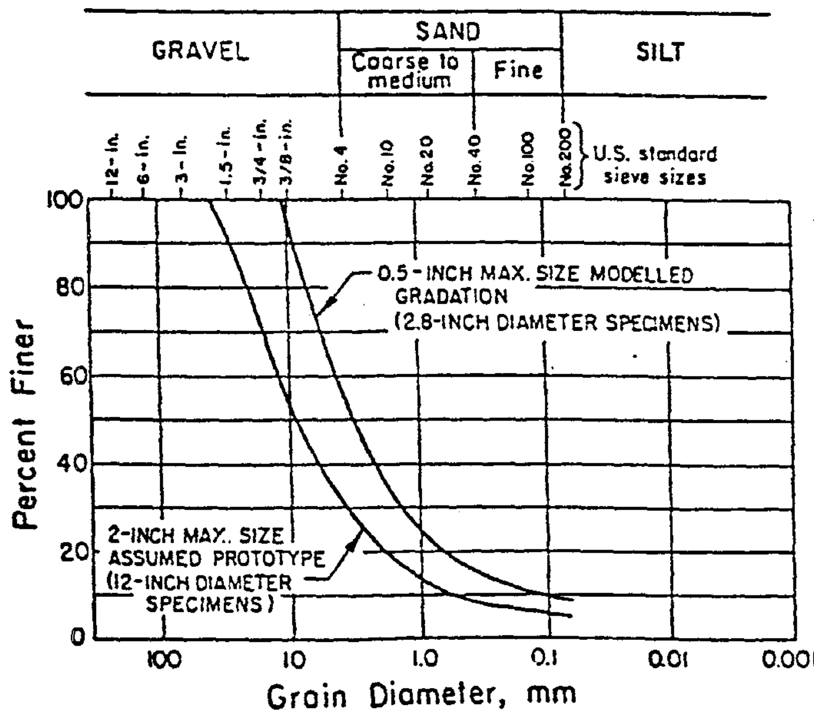
Lee and Fitton (1969) performed tests on 2.8-inch diameter samples including uniformly-graded gravels with a maximum particle size of 3/4 inch. It was reported that with all other test conditions held constant, the cyclic shear stress required to cause initial liquefaction in the gravel specimens was nearly twice that needed for similar specimens of sand. These erroneous results have since been attributed to the adverse effects of membrane compliance and stress concentration problems associated with the large ratio of particle size to sample diameters encountered in those specimens.

Wong, Seed, and Chan (1974) performed tests on sands and gravels using both 2.8-inch and 12-inch diameter samples. They reported that the gravel specimens required somewhat higher cyclic stresses to cause given strains than for similar samples of sand at the same relative densities. This result led the investigators to speculate that gravels may be inherently more resistant to liquefaction failure than sands. It was also reported in that study that lower stress ratios were required to cause given strains for well-graded specimens than for uniformly graded specimens. The investigators recognized that the noted phenomenon was an indication of the effects of membrane compliance but did not attempt to make compliance corrections. An attempt was made by Seed and Anwar (1986) to make numerical corrections to volumetric compliance magnitudes for non-uniform soil gradations, but presently there is not enough of a database to make a realistic evaluation of the accuracy of such quantitative corrections.

Banerjee et al. (1979) reported the results of large scale (12-inch diameter) and small scale (2.8-inch diameter) cyclic tests on dense, well-graded gravels with maximum particle sizes of 2 inches and 0.5 inches respectively. Banerjee used the method of testing modelled parallel gradations to evaluate the cyclic strengths of coarse gravelly materials from Oroville and Lake Valley Dams. The modelled gradations used for the testing of 2.8-inch and 12-inch diameter samples are shown in Figure 3.1. The cyclic test results presented in Figure 3.2 appear to support the use of parallel gradations for these types of tests. Among the parameters investigated by Banerjee were the effects of sample preparation, and sustained high confining pressure. The method of sample preparation appeared to be insignificant while the sustained confining load reportedly increased the cyclic loading resistance of the gravel specimens. The increase in cyclic strength or cyclic resistance due to a sustained confining load found by Banerjee is shown in Figure 3.3. These results are combined with results from previous investigations (Figure 3.4) to suggest an increase in strength for a wide range of times of sustained pressures on granular soils.

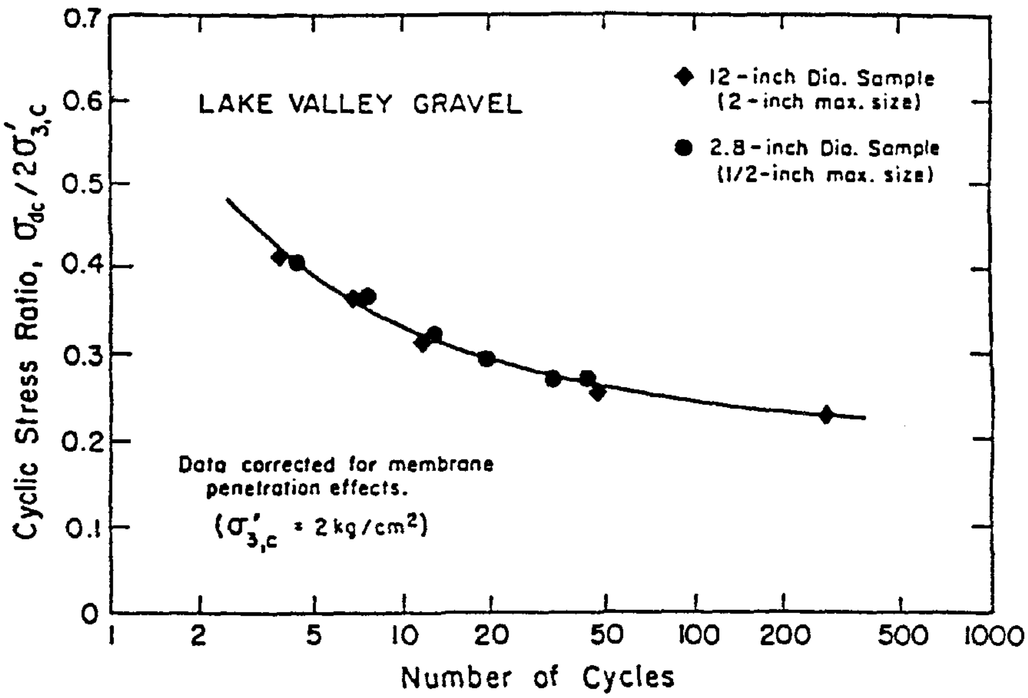


a) Lake Valley Dam Material

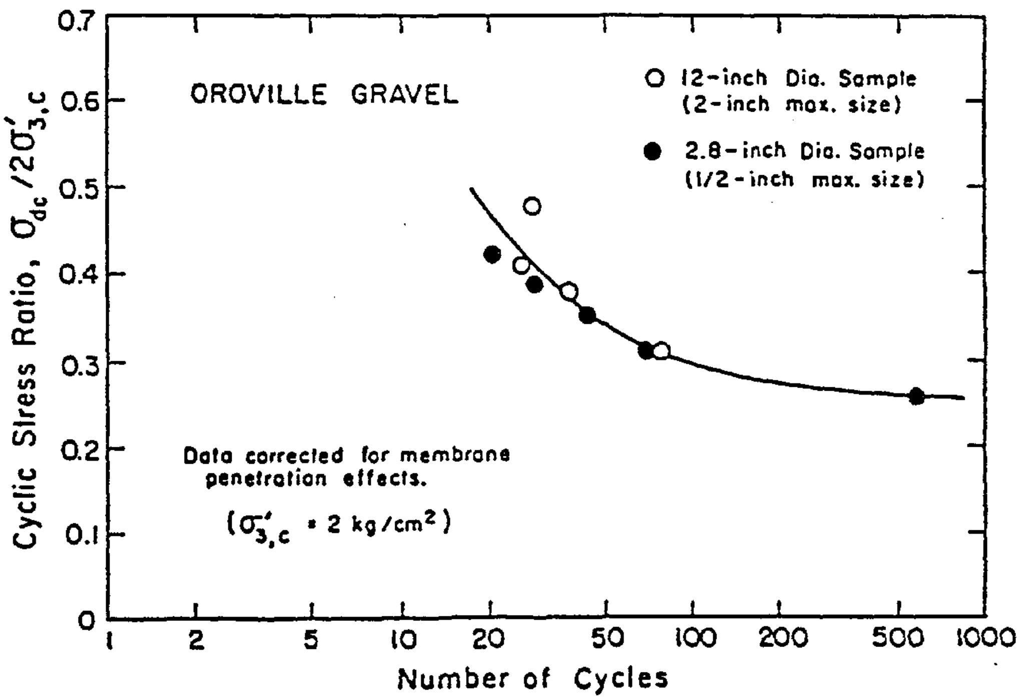


b) Orovolle Dam Material

Figure 3.1: Grain Size Distribution Curves of the Prototype and Modelled Specimens: (Banerjee et al., 1979)



a) Lake Valley Gravel at $D_R = 60\%$



a) Oroville Gravel at $D_R = 84\%$

Figure 3.2: Cyclic Strength Curves for 12-Inch and 2.8-Inch Diameter Samples of Prototype and Modelled Gradations: (Banerjee et al., 1979)

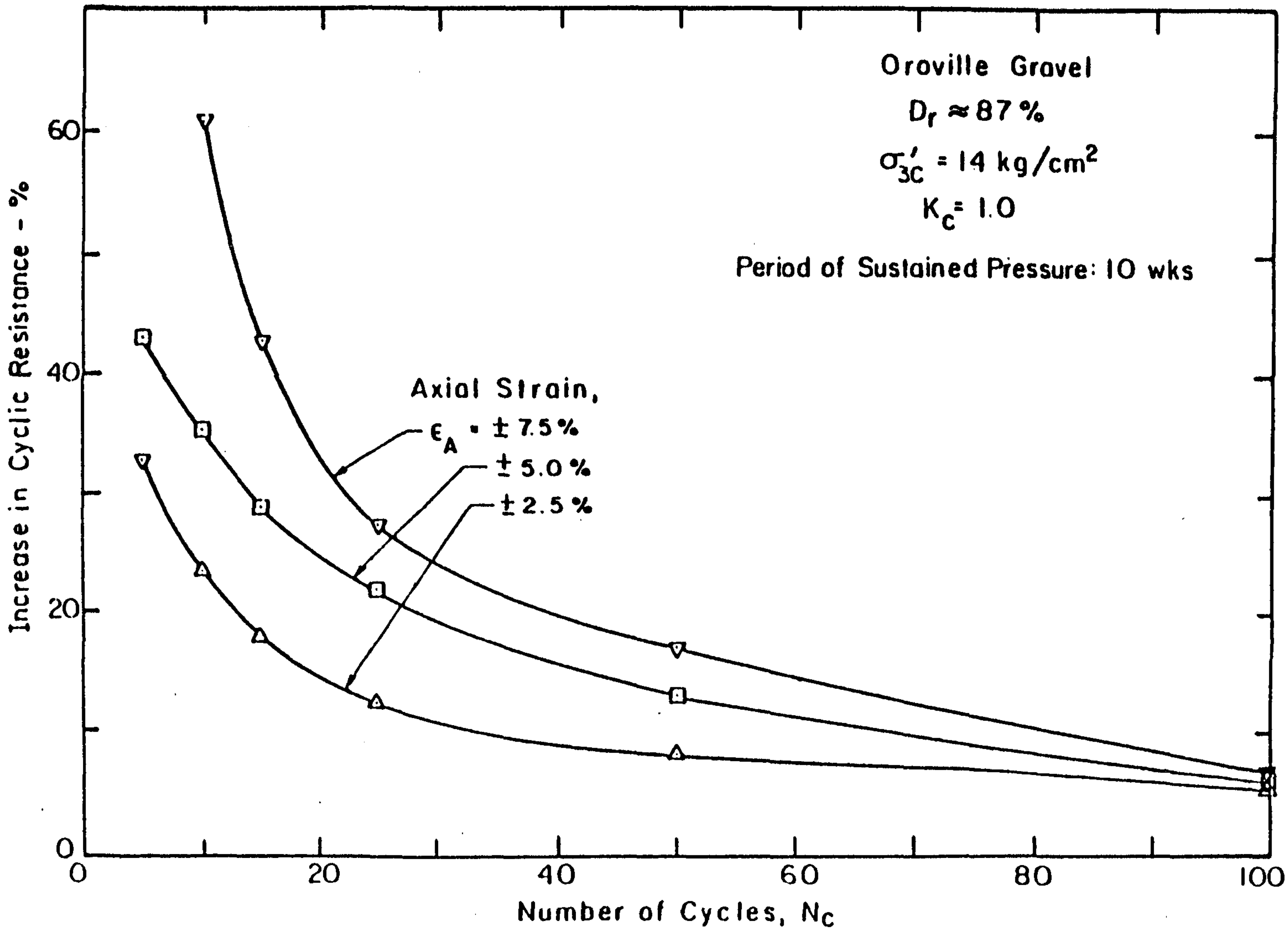


Figure 3.3: Increase in Cyclic Resistance Due to Sustained Pressure Effects: (Banerjee et al., 1979)

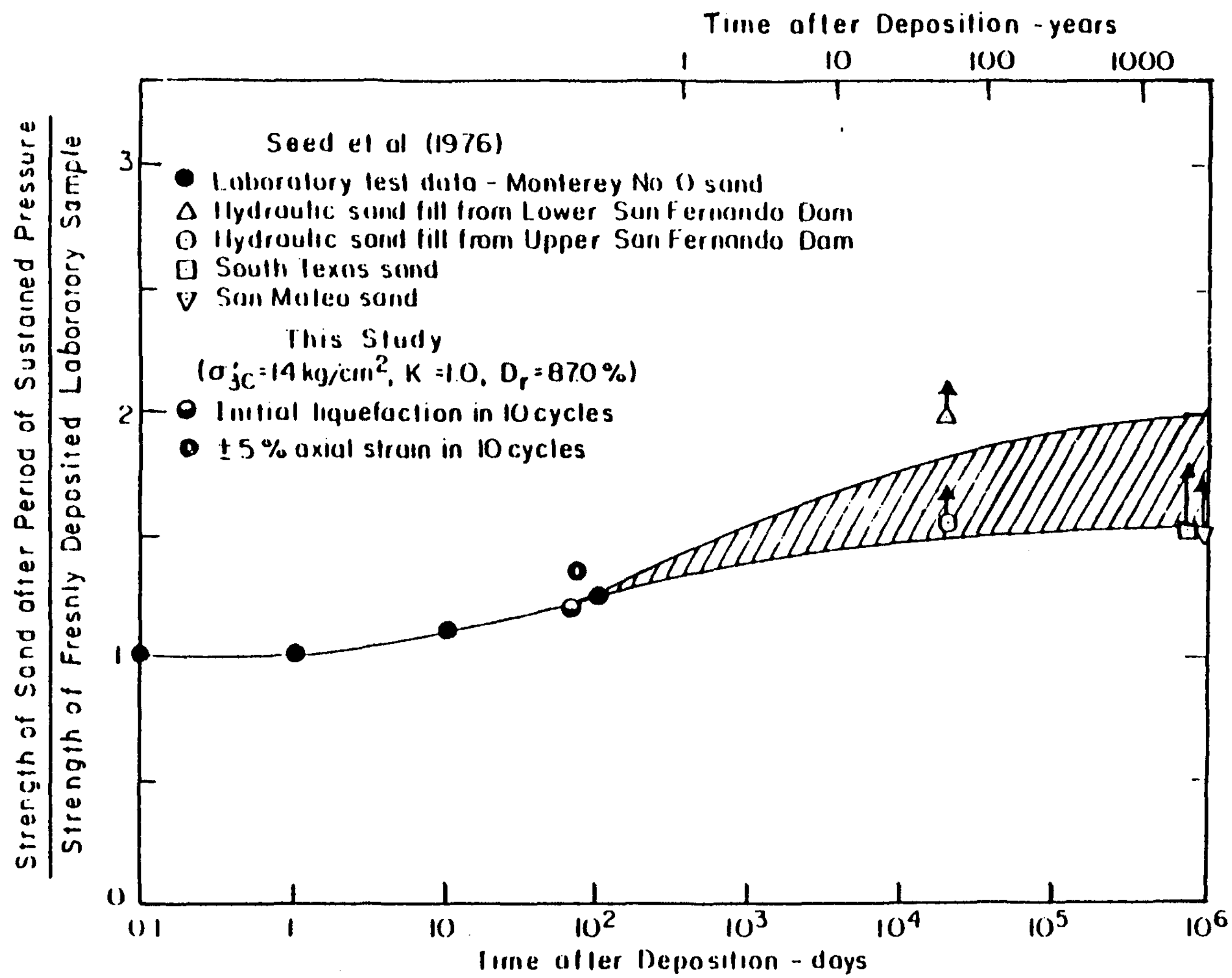


Figure 3.4: Influence of Period of Sustained Pressure on Stress Ratio Required to Cause Pore-Pressure Ratio of r_u = 100% or ± 2.5% to 5.0% Strain in 10 Cycles: (Banerjee et al., 1979)

Banerjee also developed a laboratory procedure for measuring the membrane compliance variations over a range of confining stresses. This procedure which incorporates the use of so-called "girth belts" discussed in section 2.2, allows the measurement of both axial and radial deformations of sample skeletons, from which accurate values of membrane compliance can be calculated. A detailed discussion of the equipment and calculations involved in making the radial strain measurements can be found in the appendix of the report by Banerjee et al. (1979).

Siddiqi et al. (1987) investigated methods to determine representative laboratory gradations for actual field gradations which contained particles too large to be tested with conventional laboratory equipment. Siddiqi made several conclusions pertaining to the scalping of oversized "floating" particles and the densities at which such scalped materials should be tested, in order to obtain representative results from the total prototype materials. An illustration of the added void space due to the inclusion of "oversized" large particles in a "matrix" of smaller particles is given in Figure 3.5. The components of the total volume occupied by such a material is demonstrated schematically in Figure 3.6, demonstrating the need to adjust the densities of materials to be tested so as to properly represent in-situ field densities. Figure 3.7 shows a proposed relationship between theoretical and measured dry densities as functions of percent of gravel contained in the material.

In order to account for, or to offset membrane penetration effects for cyclic triaxial tests performed on gravelly soils, Banerjee applied a simplified correction of adding 10 percent to the cyclic stress ratios actually applied during the tests.

Evans and Seed (1987) attempted to evaluate "correct" strengths for gravels and mitigate compliance effects by sluicing the gravel specimens with loose sand. The results of Evans' tests led the investigators to conclude that the correction

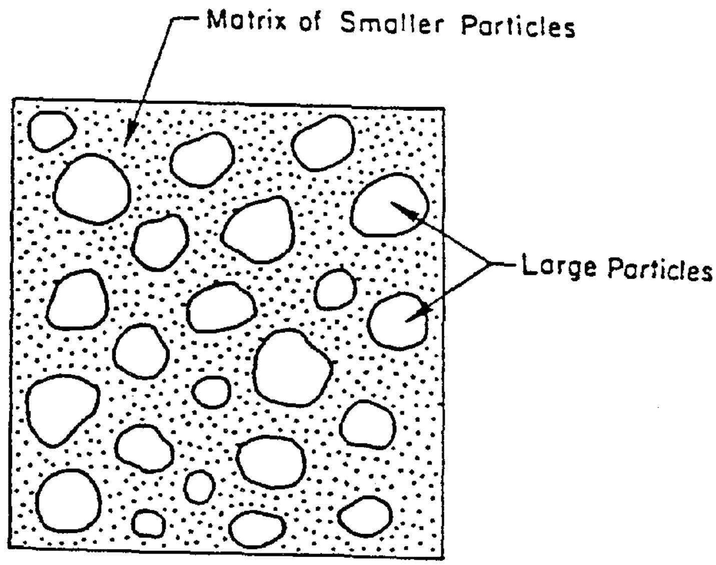


Figure 3.5(a) Illustration of Larger Particles Floating in a Matrix of Finer-Grained Soil

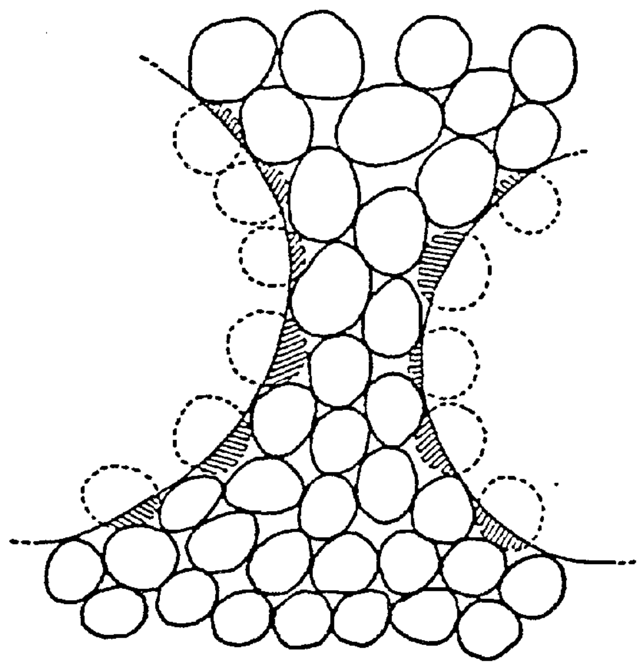


Figure 3.5(b) Illustration of Larger Void Spaces at the Contact Interfaces Between Large Particles and Finer-Grained Particles: (Siddiqi et al., 1987)

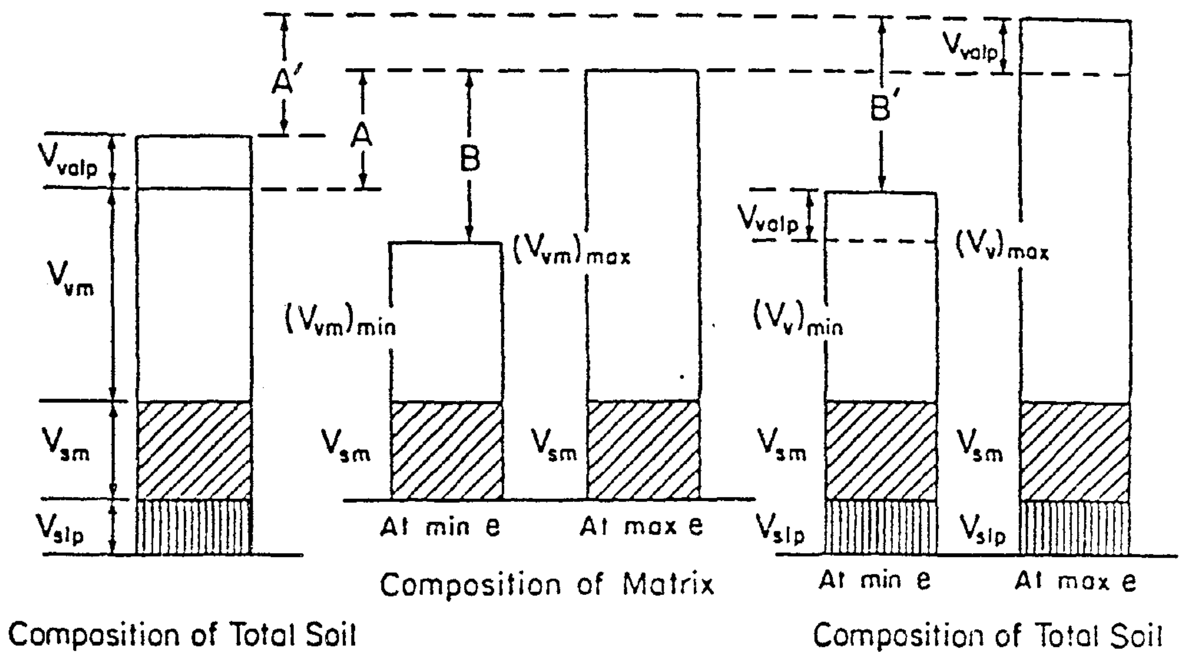
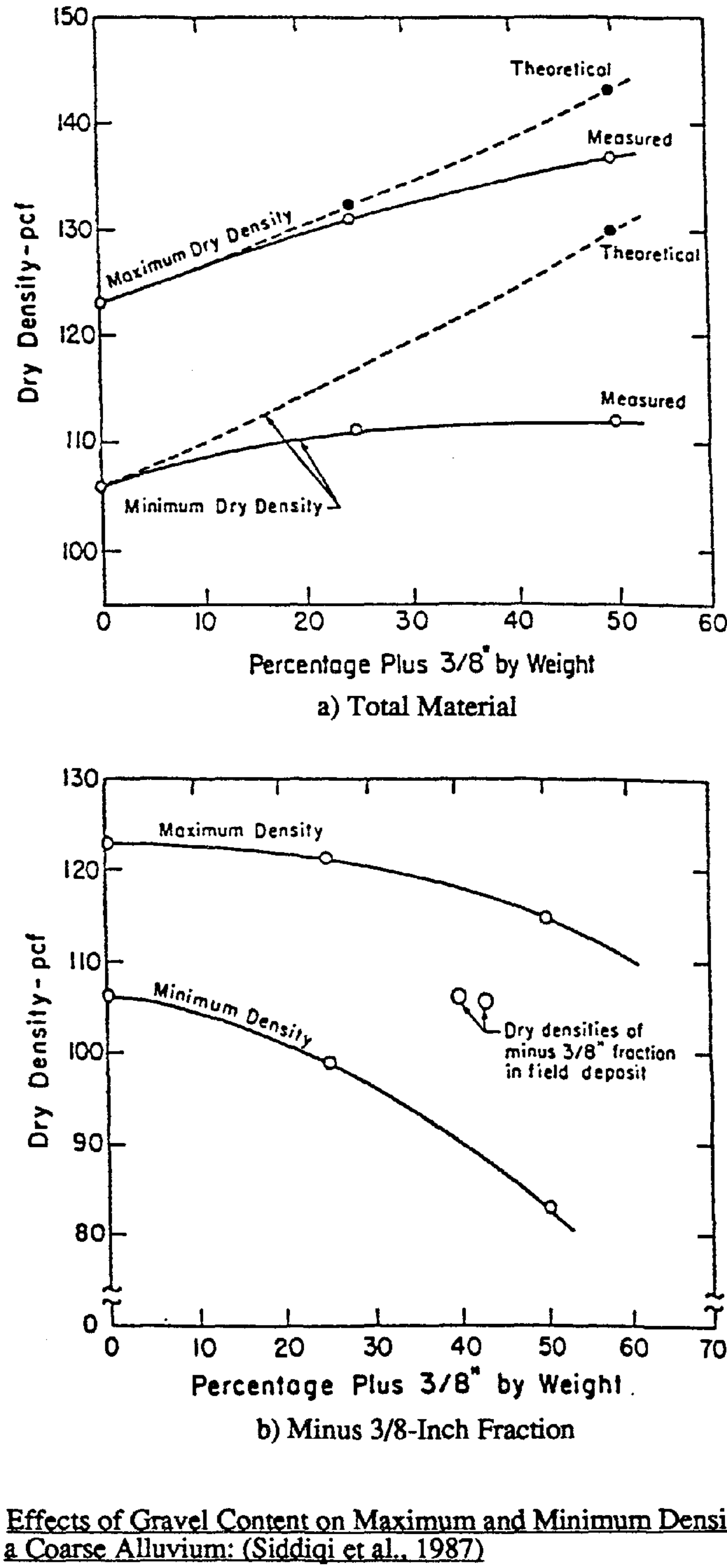


Figure 3.6: Schematic Representation of Components of Coarse-Grained Soil: (Siddiqi et al., 1987)



a) Total Material

b) Minus 3/8-Inch Fraction

Figure 3.7: Effects of Gravel Content on Maximum and Minimum Densities of a Coarse Alluvium: (Siddiqi et al., 1987)

procedure proposed by Martin et al. (1978) should be modified for 12-inch diameter specimens to incorporate a slightly larger compliance induced error. A typical set of results from cyclic tests performed on samples of sluiced and unsluiced gravel are presented in Figure 3.8, showing the reduction in cyclic strength by the sluicing method. Figure 3.9 depicts how the results generated by Evans on sluiced samples of uniformly graded gravel compare with the hypothesized corrections made by Martin et al. (1978). While compliance was significantly reduced by the sluicing method, it was not completely mitigated by employing this procedure. Because of this, the Martin, Finn, and Seed correction procedure may still need to be further modified to give more accurate predictions of what the true corrections should be, once the database becomes more complete.

Hynes (1988) investigated using the threshold strain approach to evaluate the liquefaction potential of gravels. The concept of the threshold strain theory, is that for each soil type there is a certain shear strain at which pore pressures begin to develop. It was suggested by Seed (1979) that threshold strain levels appear to be independent of most of the major factors that affect cyclic strength. Therefore only the threshold strain of the material, and the expected possible strains at a given site, need to be known in order to provide a primary screening of liquefaction potential. The idea being that if the necessary strains needed to cause pore pressure generation are not expected, then liquefaction of the material can be ruled out, and further cyclic testing of the material for liquefaction potential would be unnecessary. Additional objectives of that study included providing pore pressure generation data and characteristics for well-graded gravels at loose and moderately dense relative densities. Membrane compliance measurements were made on at least one of the gravel types tested in that study, and those determinations were used to correct cyclic strength evaluations using the procedure developed by Martin et al. (1978), and modified by Evans and Seed (1987).

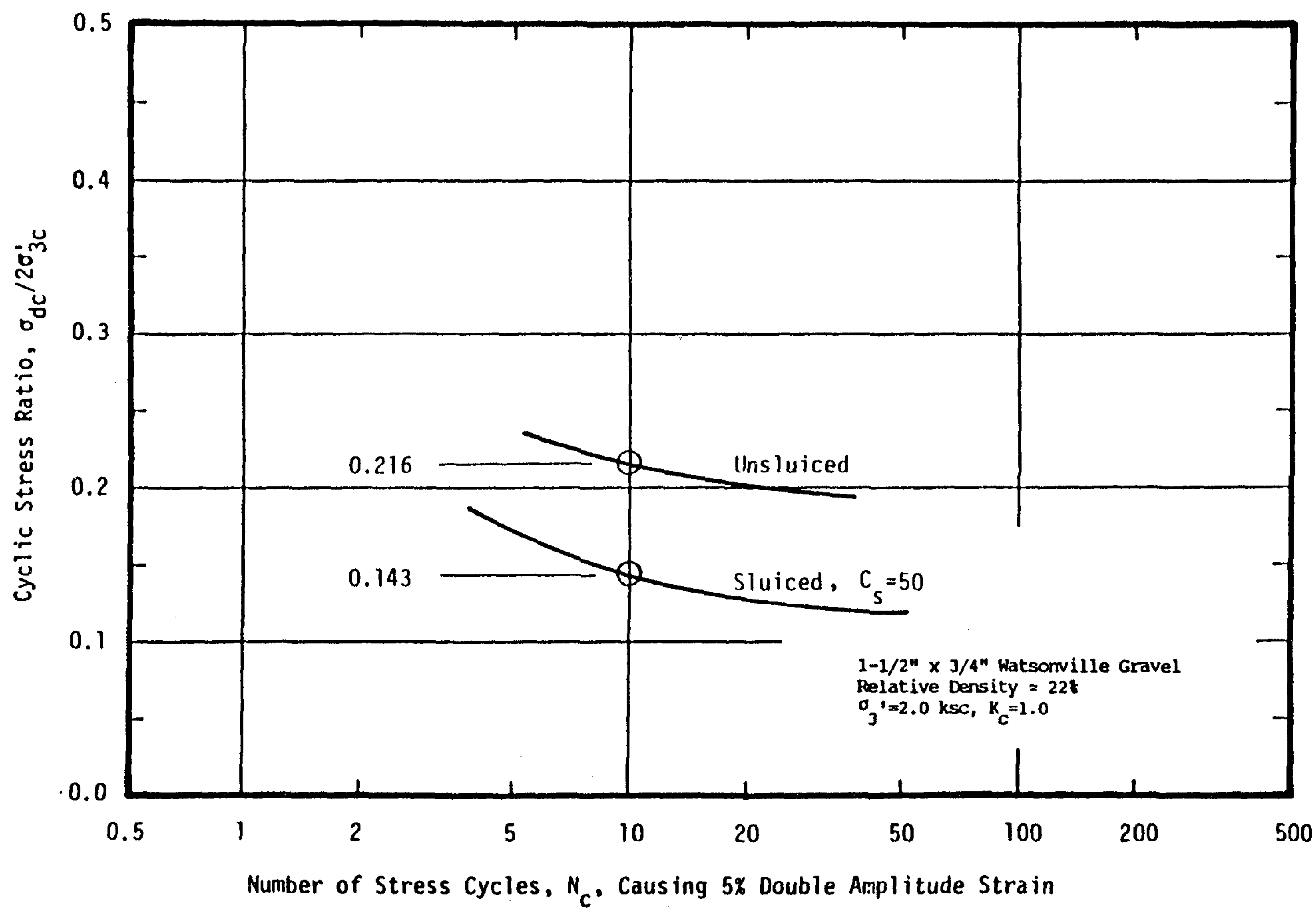


Figure 3.8: Cyclic Strength Curve Showing Effect of Sluicing on 12-Inch Diameter Samples: (Evans and Seed, 1987)

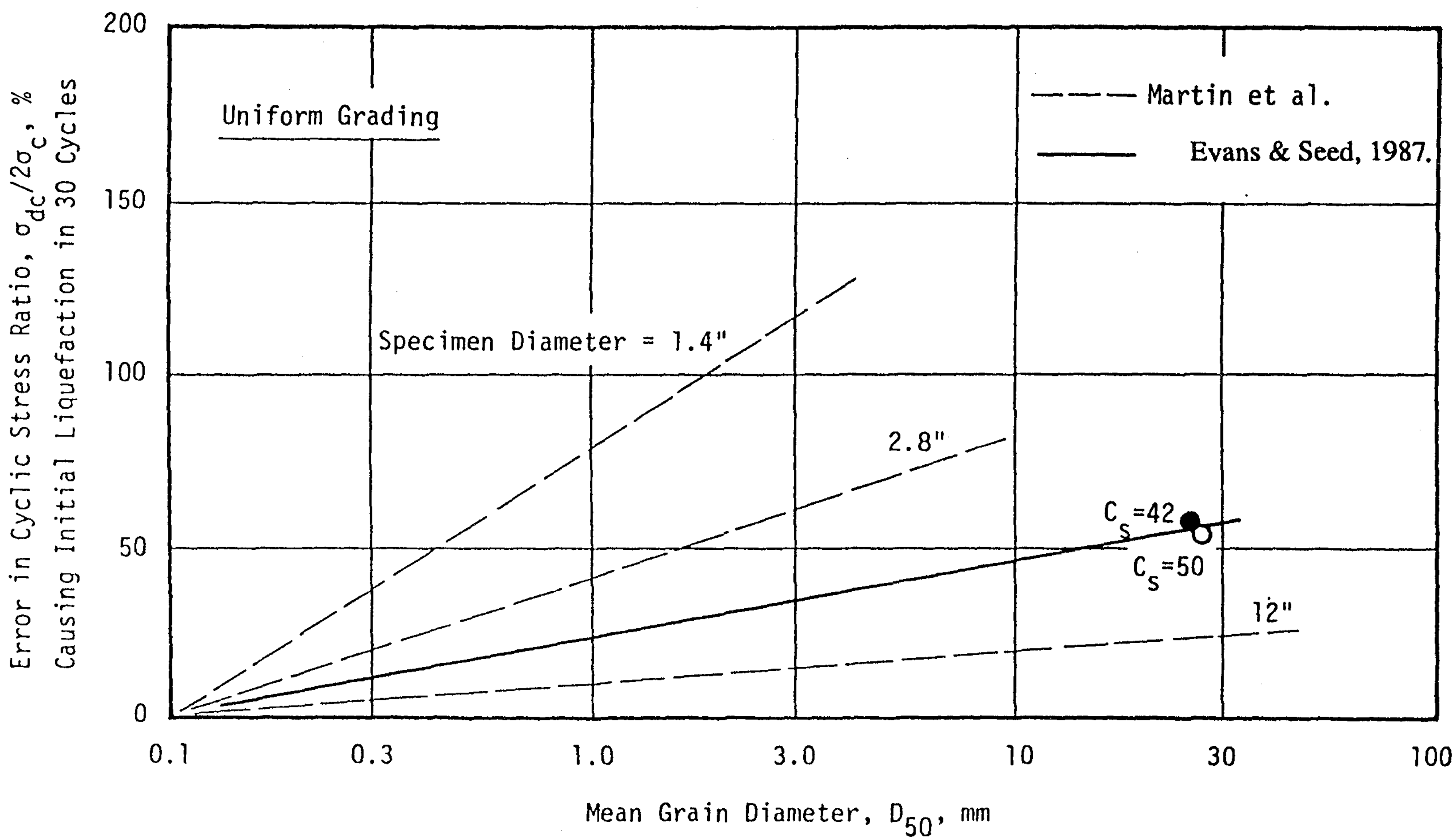


Figure 3.9: Error in Cyclic Stress Ratio Due to Membrane Compliance vs. Mean Grain Size for Various Specimen Diameters: (after Martin et al., 1978; Evans and Seed, 1987)

CHAPTER 4

VERIFICATION OF A COMPUTER-CONTROLLED INJECTION/REMOVAL SYSTEM FOR MITIGATION OF COMPLIANCE EFFECTS

4.1 Introduction

Until tests are performed in which membrane compliance effects are completely and conclusively mitigated, the accuracy of theoretical corrections or compliance mitigation techniques can not be proved. In order to investigate whether or not the proposed method of continuous computer-controlled injection/removal of water to offset the volumetric error induced by membrane compliance completely mitigates the deleterious effects of compliance on undrained tests, a number of monotonic and cyclic triaxial tests performed with the use of the injection-mitigation system devised by Seed and Anwar (1986) on small scale (2.8-inch diameter) samples of Monterey 16 sand were duplicated using large scale (12-inch diameter) samples of the same material for which no corrections were necessary. The results of these large-scale tests were then compared to the results of the earlier small-scale tests. Based on the test results reported in this chapter, the "correctness" and accuracy of such a compliance mitigation system was verified.

4.2 Verification of a Computer-Controlled Injection/Removal System for Mitigation of Compliance Effects

The first phase of implementing a computer-controlled injection-correction system was to verify the accuracy and validity of such a system to eliminate the adverse affects of membrane compliance. The compliance mitigation system implemented by Seed and Anwar (1986) for use with "conventional" sized (2.8-inch diameter) samples appeared to give excellent results for eliminating membrane compliance effects as compared with the results of tests "corrected" for volumetric compliance errors by analytical methods. But the validity of these analytical models themselves could not be verified. Therefore in order to validate the use of the computer-controlled injection-correction system used in the investigation by Seed and Anwar, a physical testing approach was used, repeating the earlier tests with 12-inch diameter samples. As described in Section 2.3.2, the effects of membrane compliance can be eliminated by using a sufficiently large sample size such that volumetric membrane compliance is negligible for the material being tested. The material used by Seed and Anwar for a majority of their tests on 2.8-inch diameter samples performed with and without the implementation of their computer-controlled injection-correction system, was a uniformly graded Monterey 16 sand. A gradation curve for this material is given in Figure 4.1. The grain sizes of this material were sufficiently small as to preclude any "significant volume" of penetration of the large-scale membrane into peripheral sample voids, so that the expected amount of membrane compliance was negligible when 12-inch diameter samples were prepared. As explained in Section 2.3.2, this is due to the very small volume of edge voids with respect to overall sample volume of the "large-scale" samples.

A quantity of the same material that was used for the comparative tests by Seed and Anwar (1986) was obtained for testing in "large-scale" 12-inch diameter samples. Gradation, as well as maximum and minimum densities were checked to ensure that the material was indeed similar to that previously tested. 12-inch diameter samples were prepared by the same method (moist tamping in controlled-volume layers) and at the same relative density for cyclic tests as the 2.8-inch diameter samples had been in the earlier study. For monotonic (static) tests, samples were prepared at a variety of densities as described in Section 4.3.3.

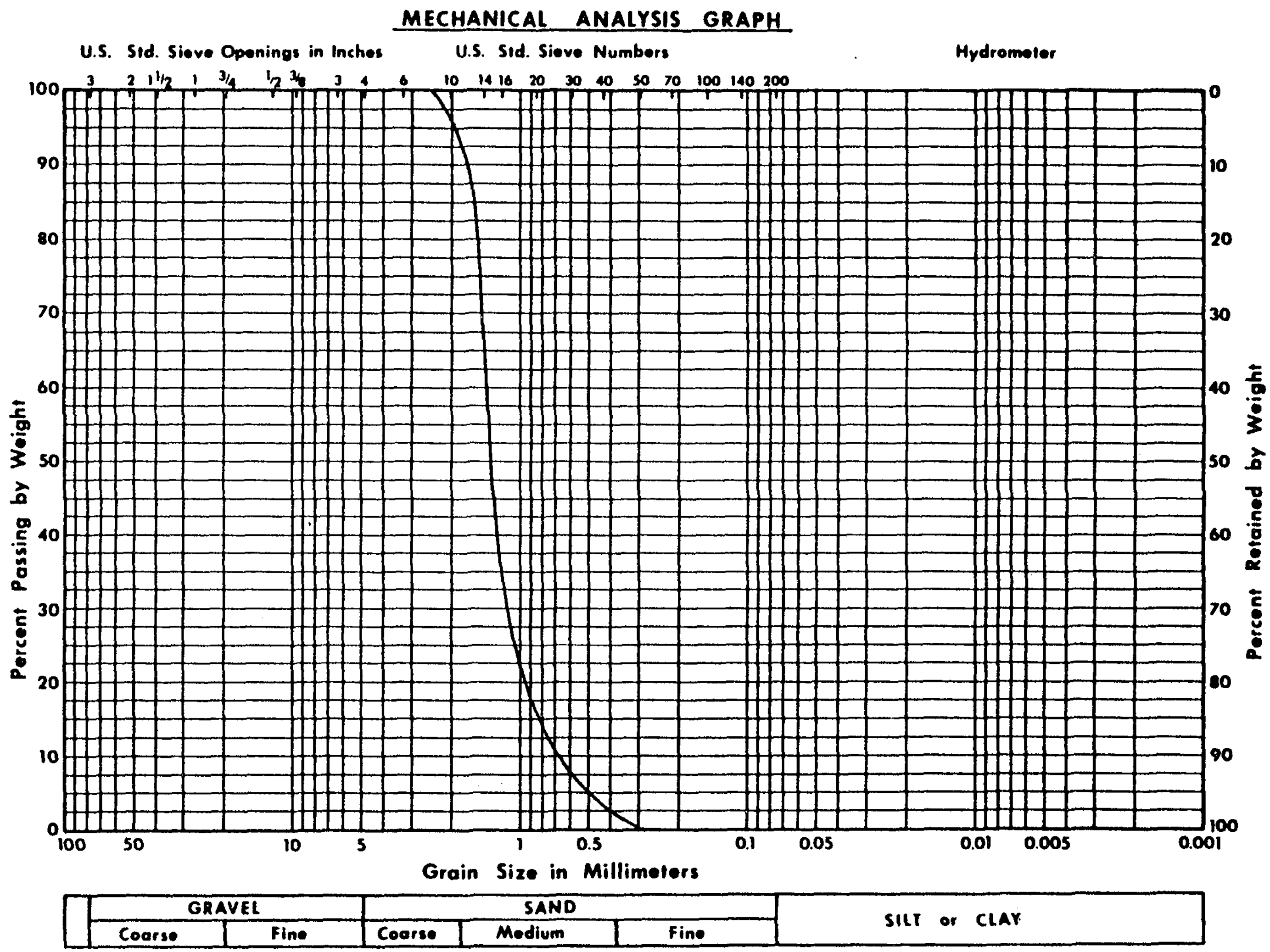


Figure 4.1: Gradation for Monterey 16 Sand

Both static and dynamic (cyclic) undrained load tests were performed on the large-scale samples, employing conventional testing methods. This provided test results which could be taken as "correct" (without the deleterious effects of membrane compliance). These "correct" large-scale test results were then compared to those results presented by Seed and Anwar for the earlier small-scale tests of the same material under similar conditions with the exception that a computer-controlled injection-correction system was employed for the smaller samples (which exhibited considerable compliance in the small-scale tests) to offset the pre-determined volumetric errors expected for those samples. The results of the comparative tests performed by Seed and Anwar on the material with and without the use of the injection-correction system are presented in conjunction with the results of the new tests performed using 12-inch diameter samples in Figures 4.2 and 4.3. It can be seen from these results that the tests performed on the 2.8-inch diameter samples with the use of the computer-controlled injection system appear to be in excellent agreement with "correct" large-scale test results, and that the small-scale samples tested without computer-controlled injection/correction agreed very poorly with the large-scale test results. Accordingly, it can be concluded that the computer-controlled injection system accurately and reliably eliminated the effects of membrane compliance for both the monotonic and cyclic load tests. The initial test conditions and principal results for all of the tests plotted in Figures 4.2 and 4.3 are tabulated in Tables 4.1 and 4.2. Individual test results for each of the tests are presented in Figures 4.4 through 4.26.

Having thus physically verified the accuracy and reliability of the system used by Seed and Anwar, the mitigation system devised for this study was modelled using the same general techniques, but with some obvious modifications necessary

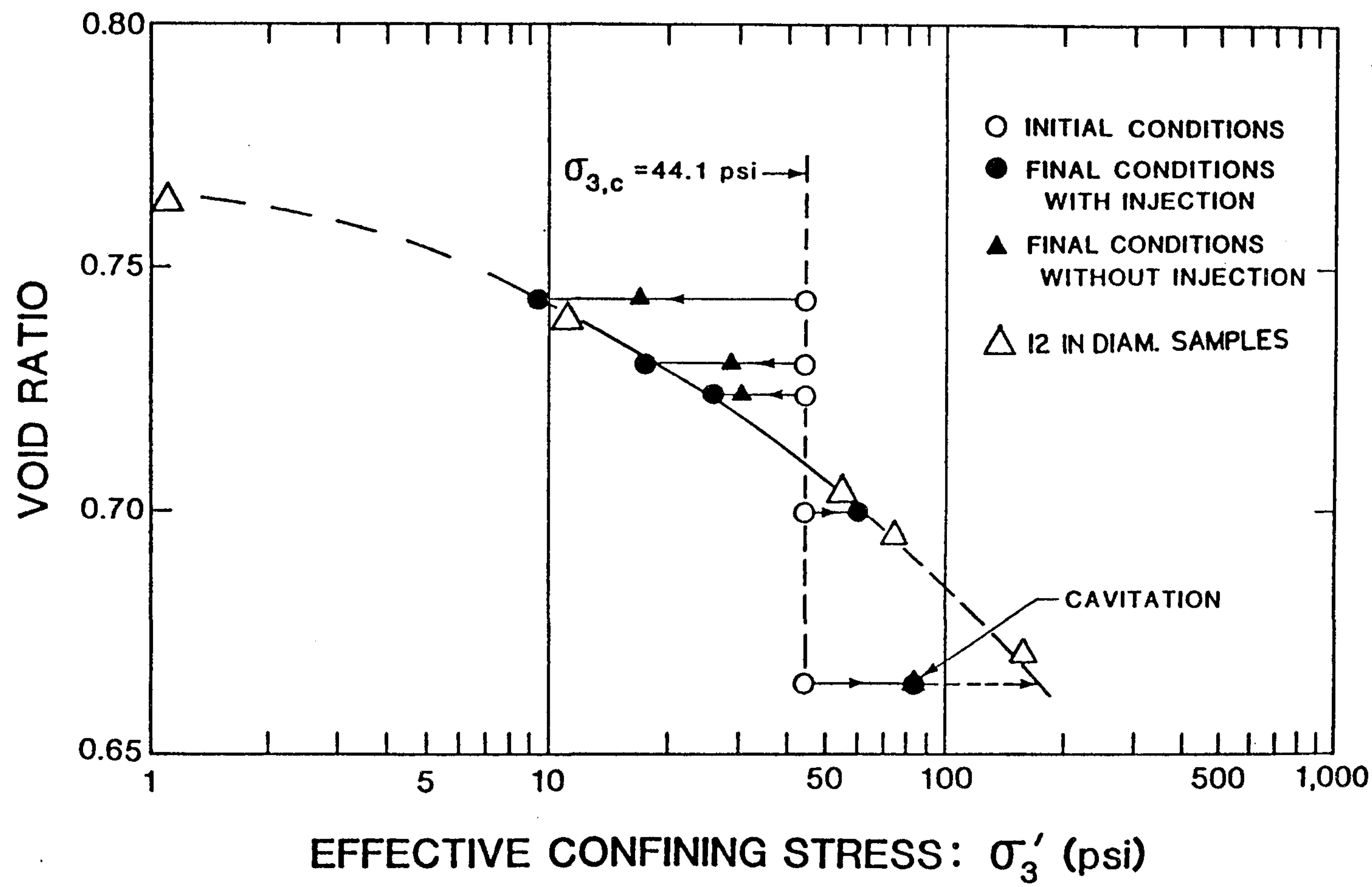


Figure 4.2: Critical-State Plot for $\overline{\text{IC-U}}$ Triaxial Tests on Monterey 16 Sand for 2.8-Inch Diameter Samples With and Without Membrane Compliance Mitigation and for 12-Inch Diameter Samples

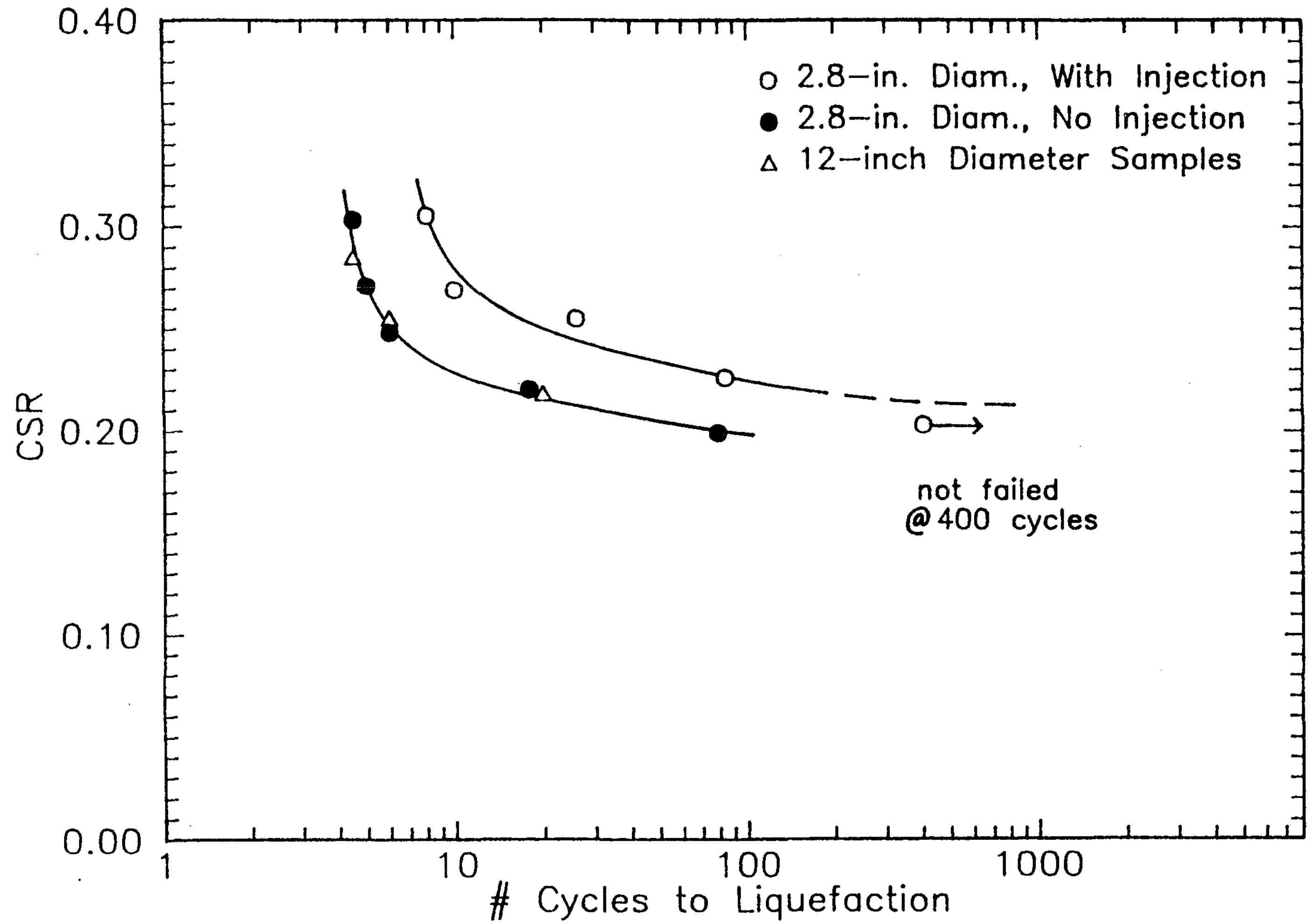


Figure 4.3: Results of Isotropically Consolidated-Undrained Cyclic Triaxial Tests on Monterey 16 Sand at $D_R \approx 55\%$ for 2.8-Inch Diameter Samples With and Without Membrane Compliance Mitigation and for 12-Inch Diameter Samples

Table 4.1: Testing Conditions: $\overline{\text{IC}}$-U Triaxial Tests on Monterey 16 Sand With and Without Membrane Compliance Mitigation

(a) 2.8-inch diameter samples:

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,c}$ (psi)	B-Value
1A	15	Yes	44.1	0.991
1B	15	No	44.1	0.993
2A	19	Yes	44.1	0.987
2B	19	No	44.1	0.990
3A	22	Yes	44.1	0.988
3B	22	No	44.1	0.989
4A	29	Yes	44.1	0.983
4B	29	No	44.1	0.990
5A	40	Yes	44.1	0.987
5B	40	No	44.1	0.981

(b) 12-inch diameter samples:

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,c}$ (psi)	B-Value
PT-4	31	No	44.1	0.975
PT-5	27	No	44.1	0.970
PT-6	10	No	44.1	0.970
PT-7	16	No	44.1	0.980
PT-8	37	No	44.1	0.980

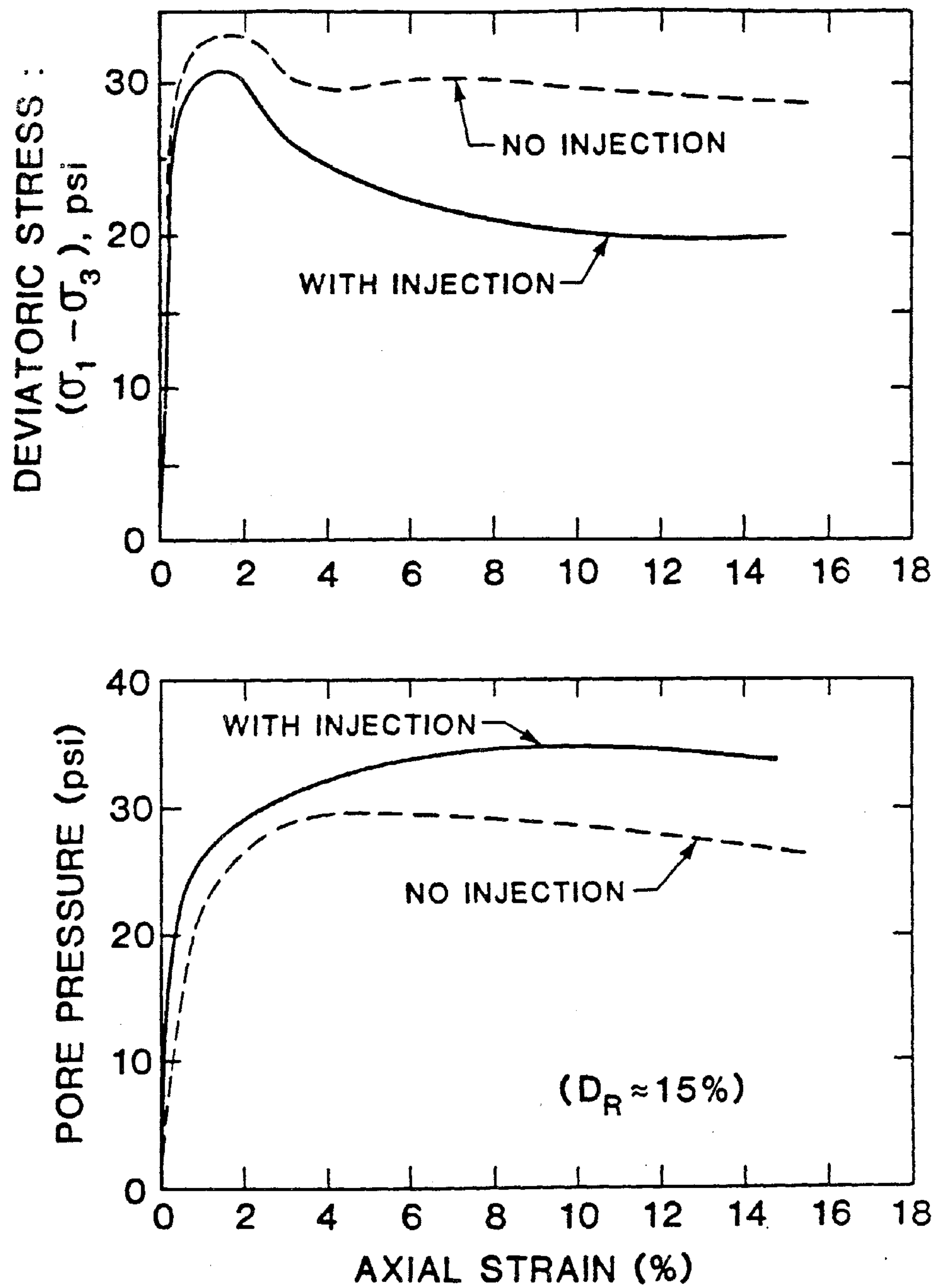


Figure 4.4: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 15\%$)

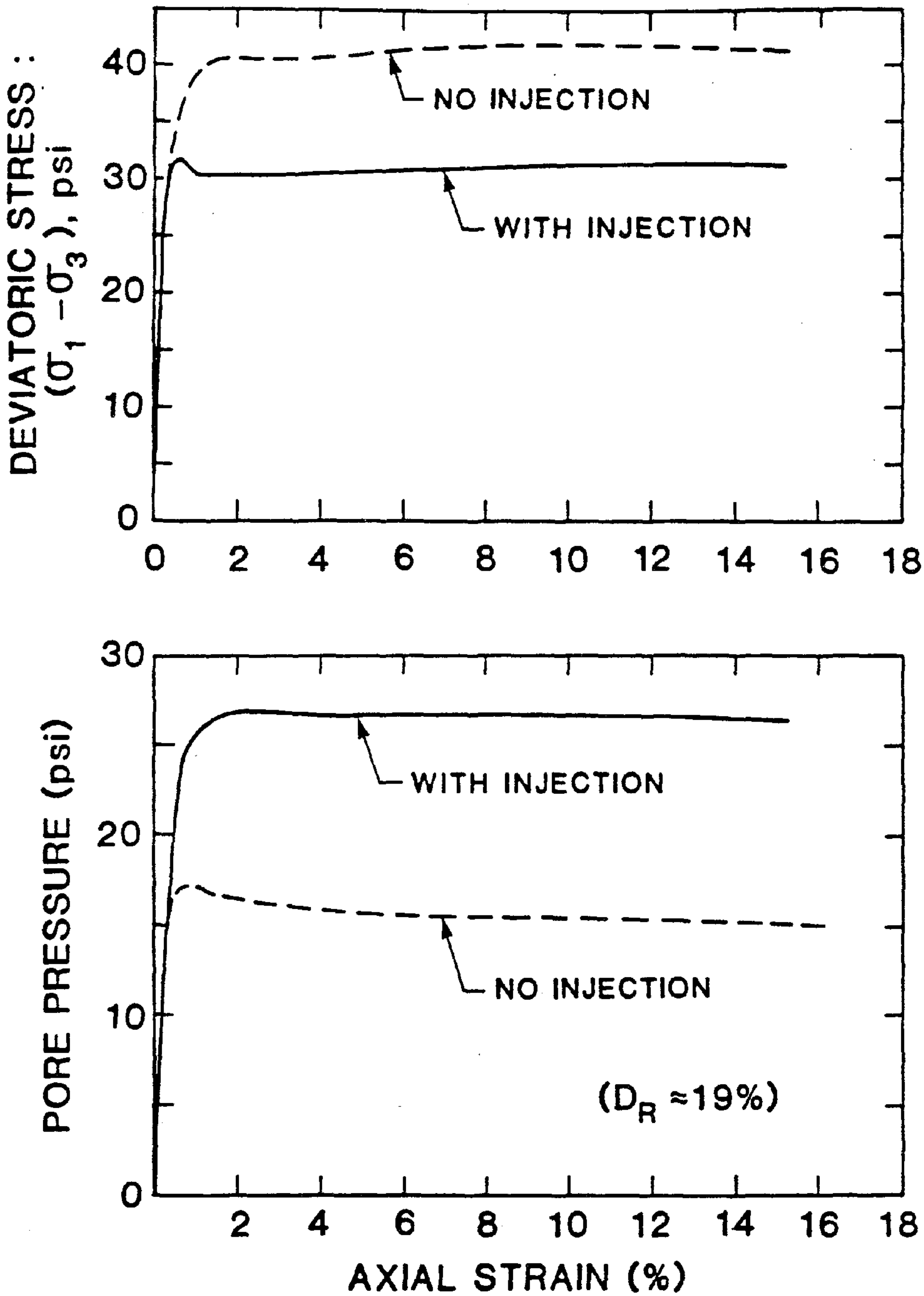


Figure 4.5: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx$ 19%)

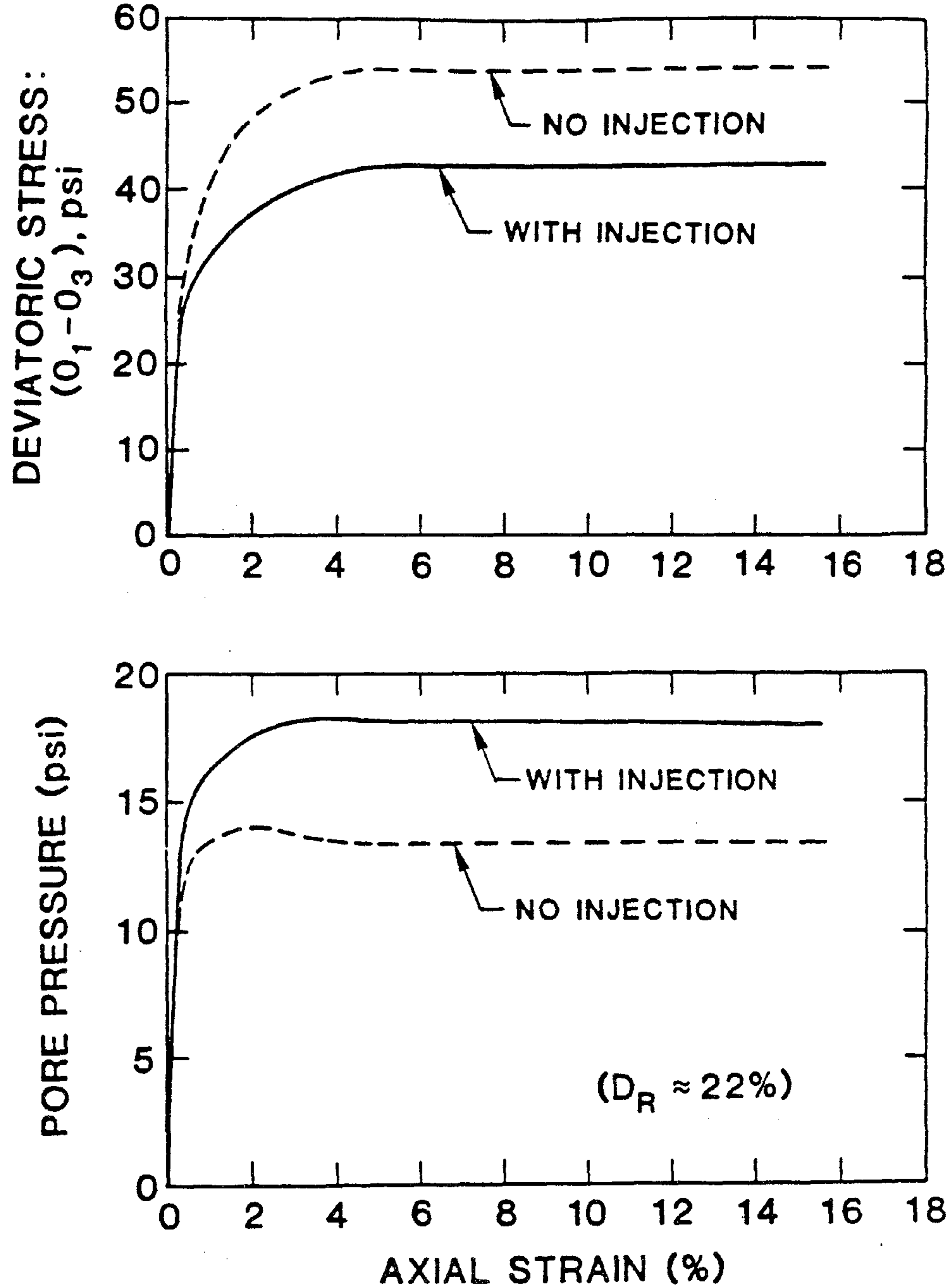


Figure 4.6: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 22\%$)

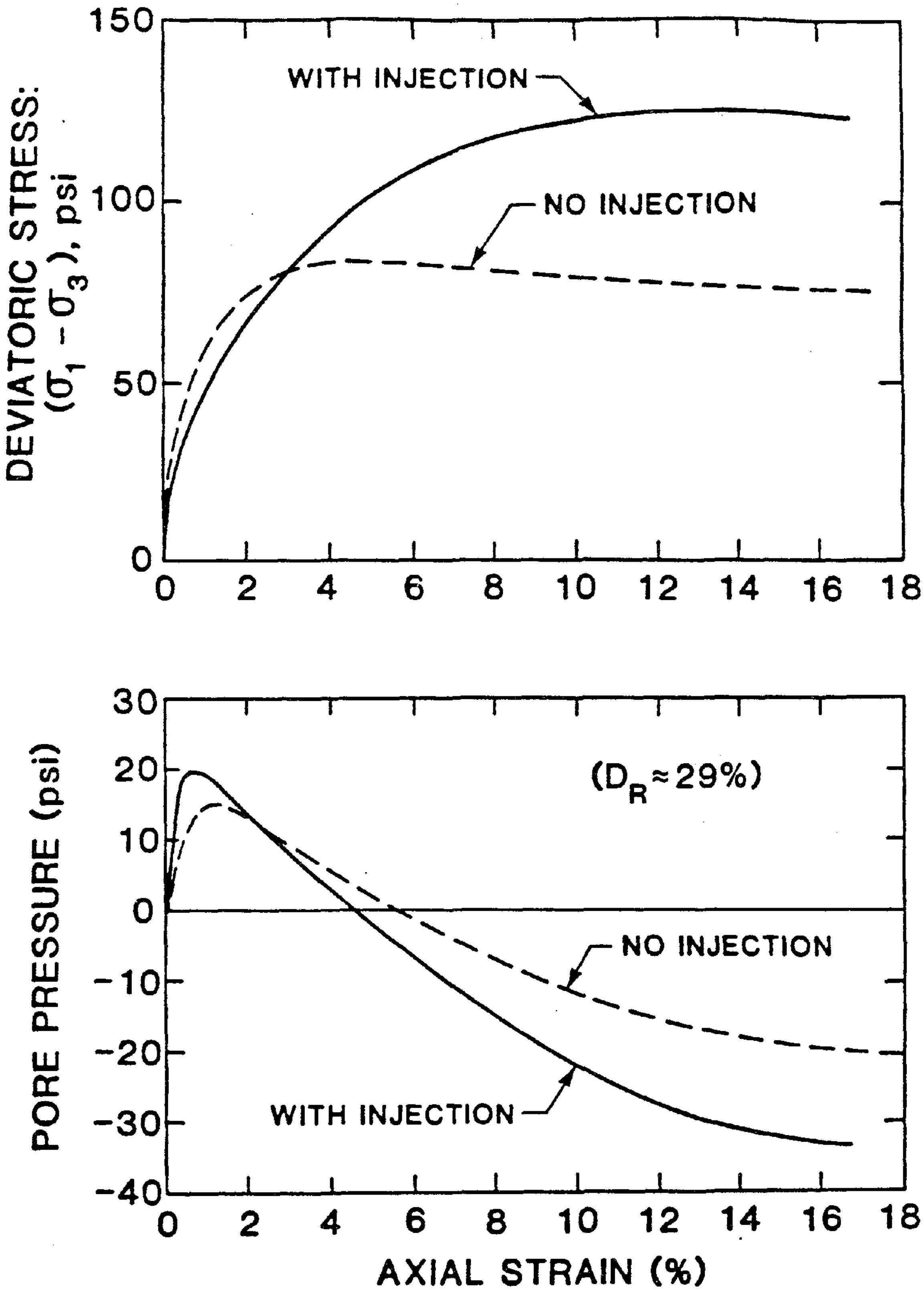


Figure 4.7: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 29\%$)

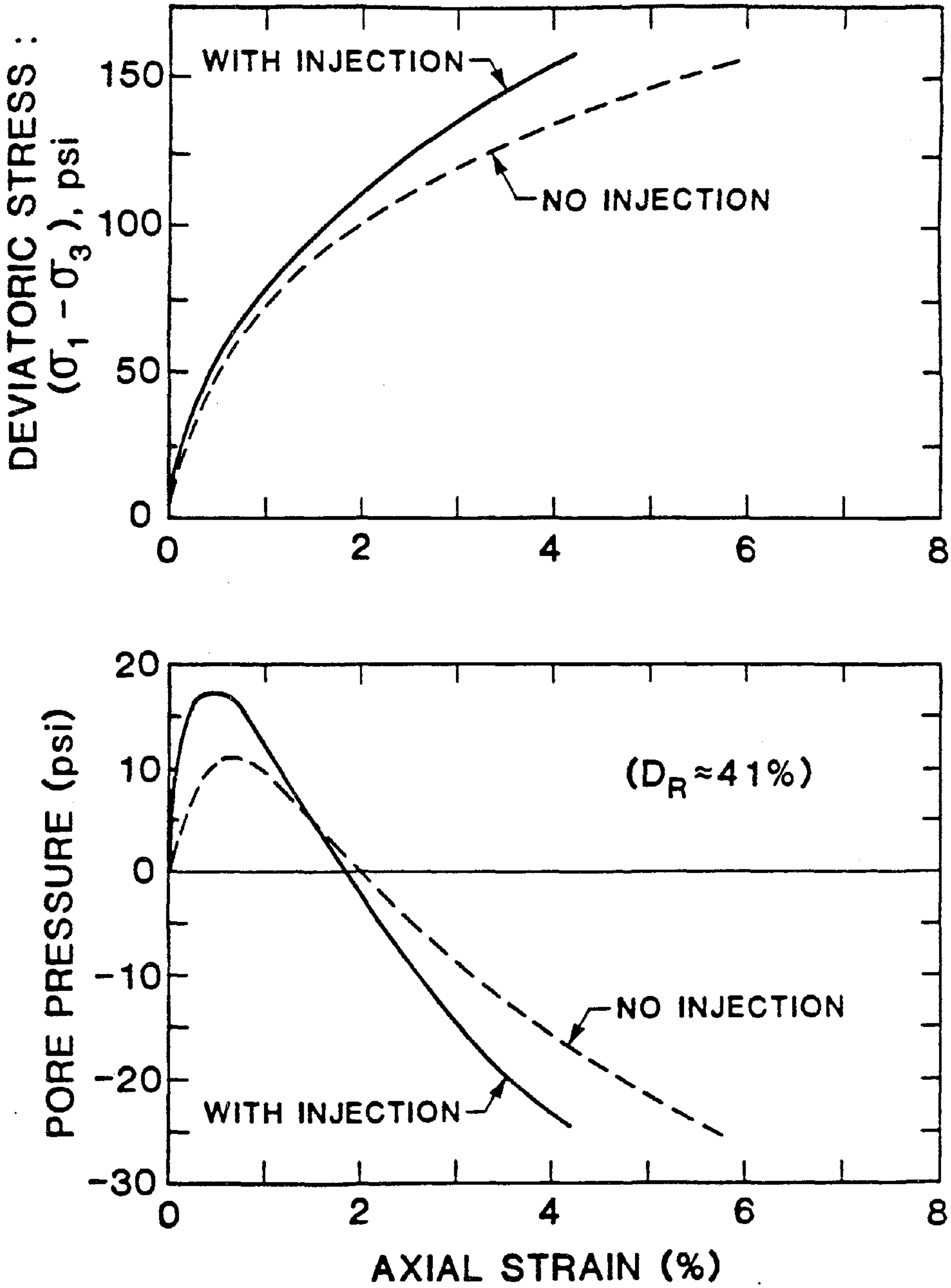


Figure 4.8: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 41\%$)

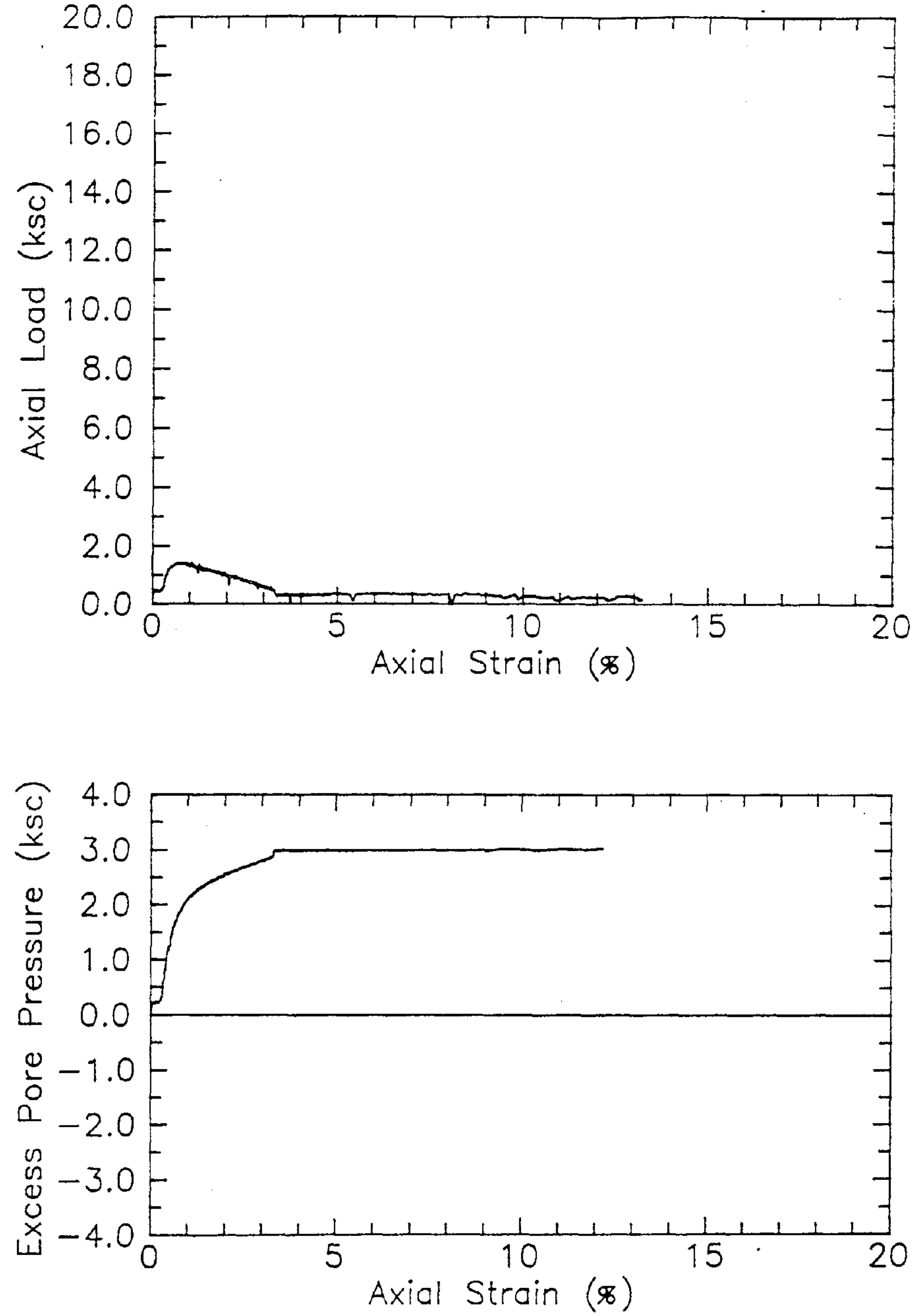


Figure 4.9: IC-U Triaxial Test No. PT-6 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 10\%$)

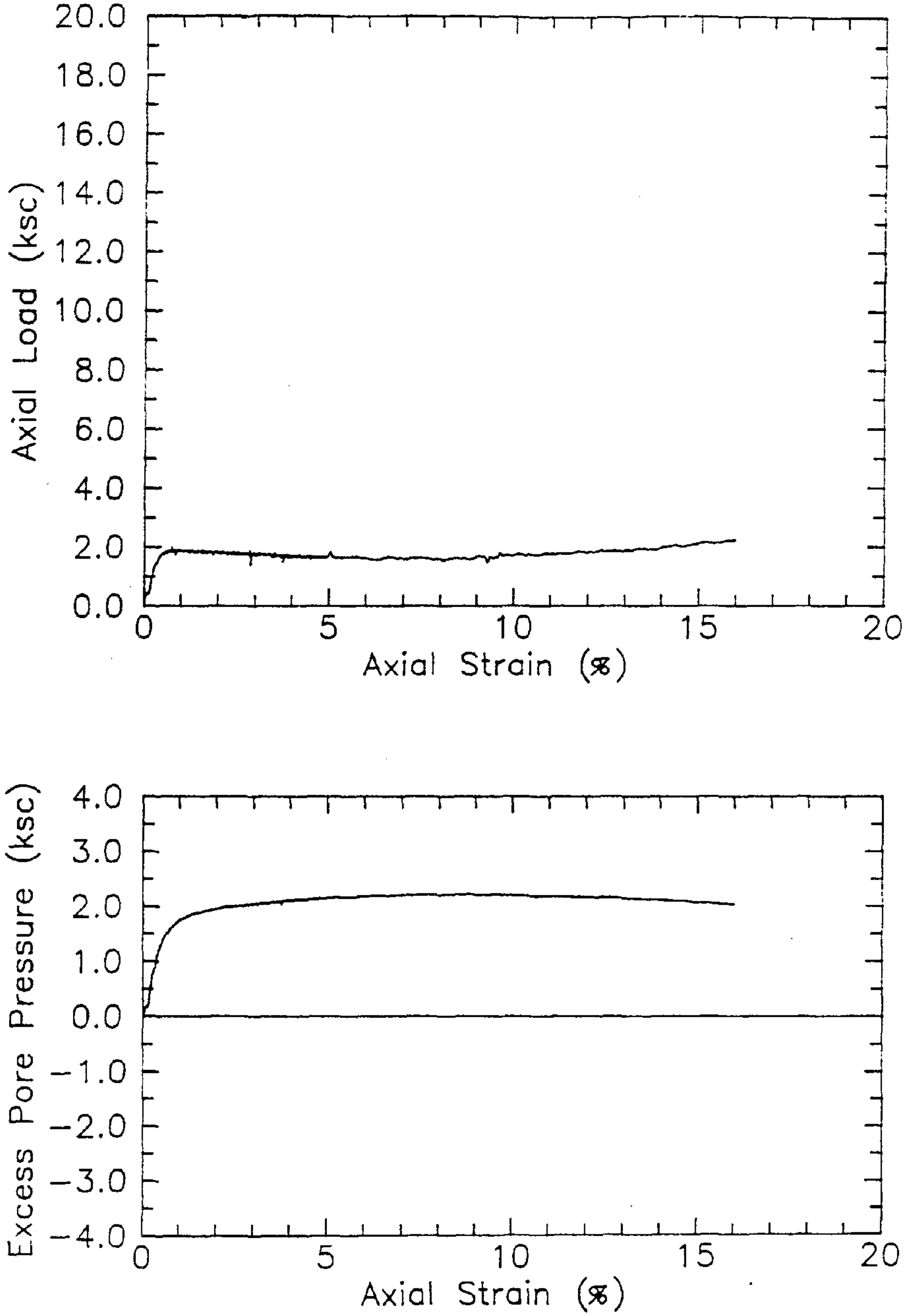


Figure 4.10: $\overline{\text{IC}}$-U Triaxial Test No. PT-7 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 16\%$)

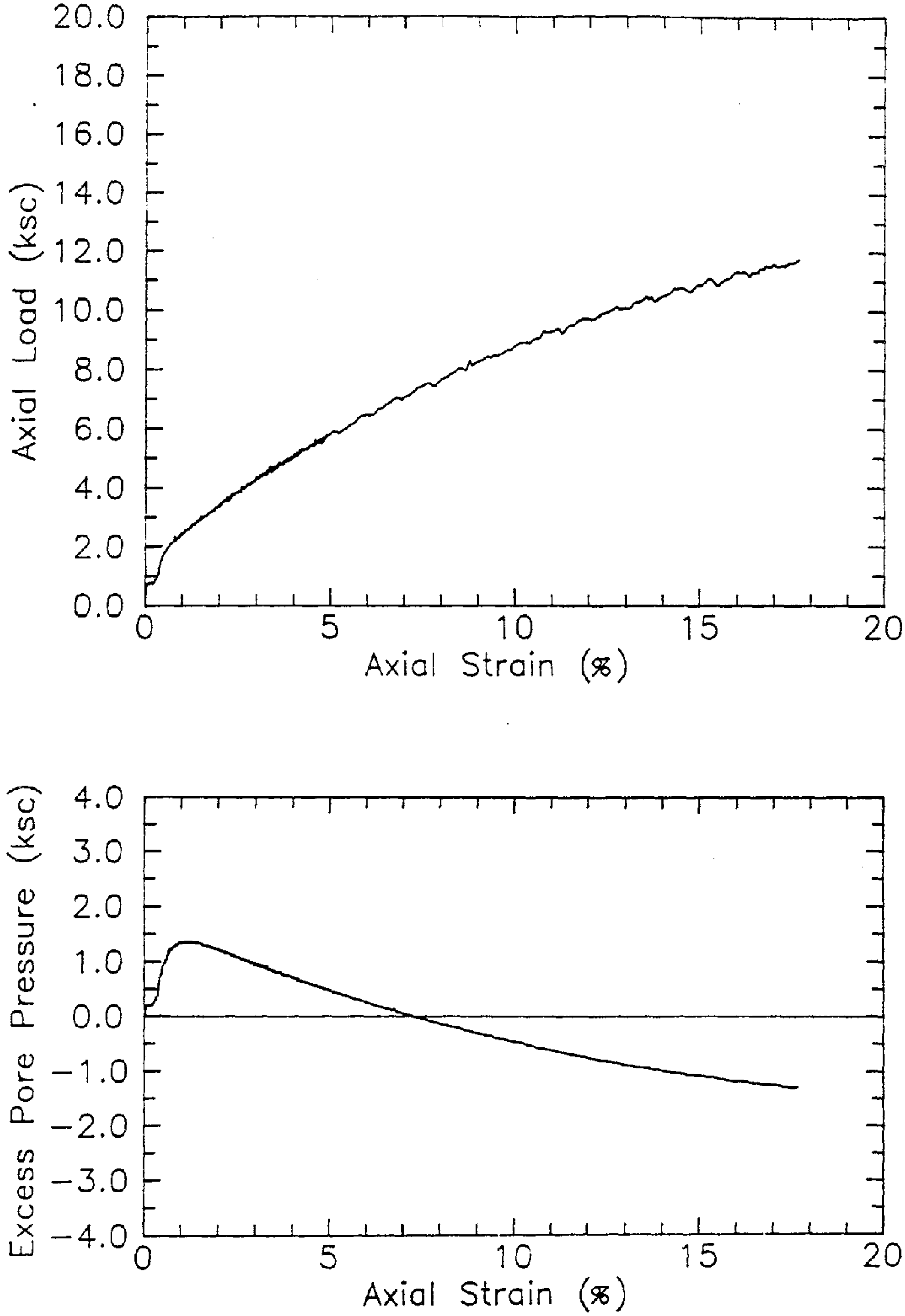


Figure 4.11: $\overline{\text{IC-U}}$ Triaxial Test No. PT-5 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 27\%$)

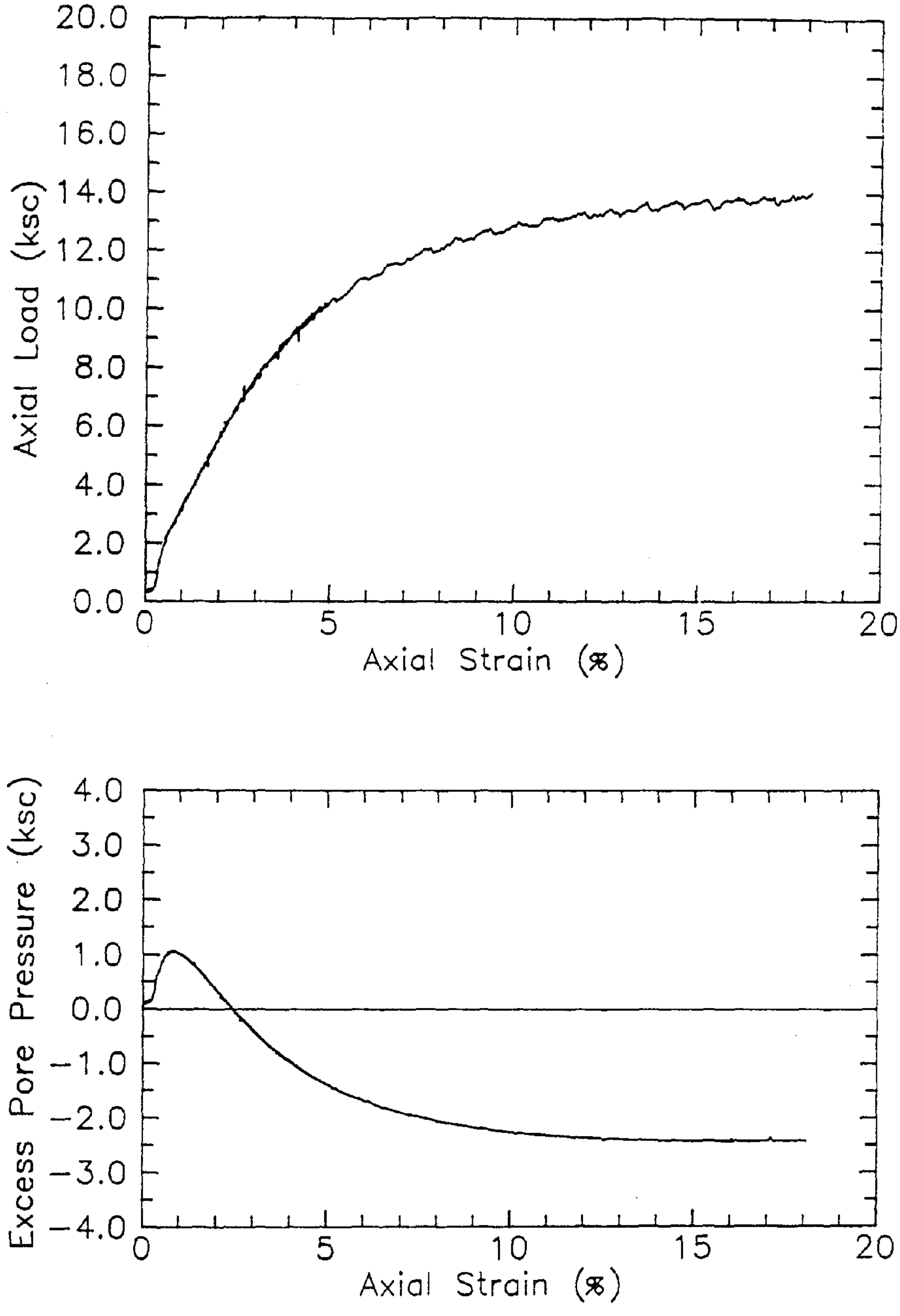
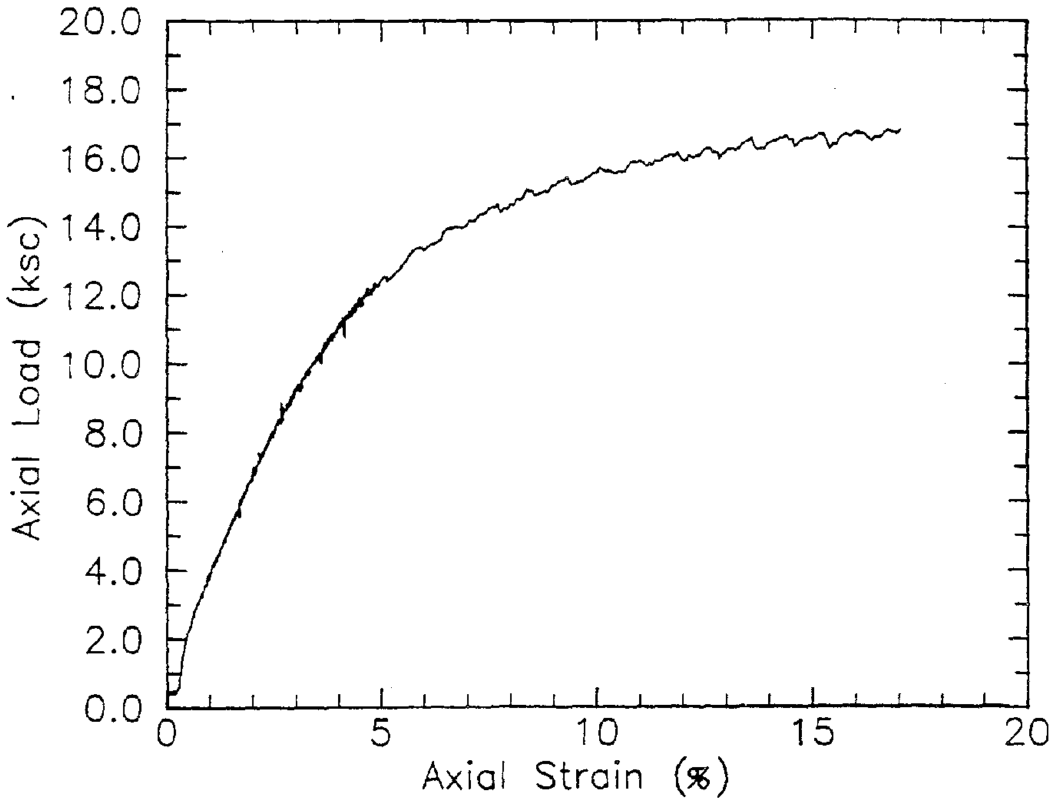


Figure 4.12: IC-U Triaxial Test No. PT-4 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 31\%$)



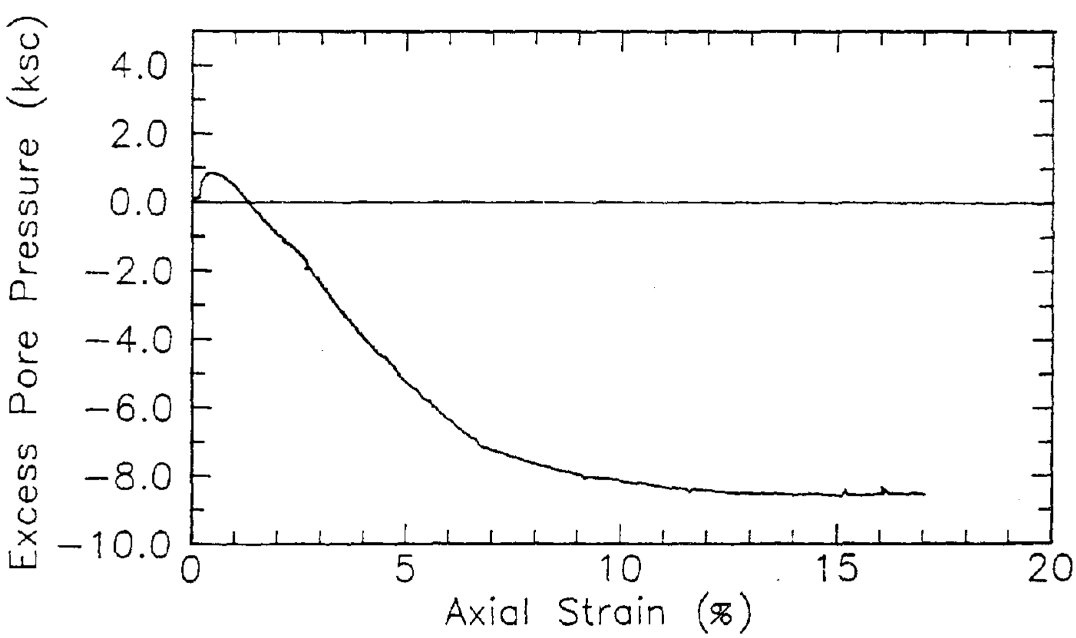


Figure 4.13: $\overline{\text{IC-U}}$ Triaxial Test No. PT-8 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 37\%$)

Table 4.2: Isotropically Consolidated Undrained Cyclic Triaxial Tests on Monterey 16 Sand With and Without Membrane Compliance Mitigation

(a) 2.8-inch diameter samples:

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,i}$ (psi)	B-Value	CSR ($\sigma_{d,c}/2\sigma'_a$)	No. of Cycles to ± 5% ϵ_A
1A	55.5	Yes	29.4	0.993	0.303	5
2A	55.7	Yes	29.4	0.987	0.248	6
3A	54.8	Yes	29.4	0.993	0.199	79
4A	55.2	Yes	29.4	0.989	0.271	5
5A	55.9	Yes	29.4	0.992	0.220	18
1B	55.0	No	29.4	0.992	0.305	8
2B	56.0	No	29.4	0.996	0.255	26
3B	55.8	No	29.4	0.988	0.203	See Note
4B	54.1	No	29.4	0.992	0.226	84
5B	55.7	No	29.4	0.984	0.269	10

Note: Test No. 3B was stopped at 400 cycles with pore pressure ratio $r_u \approx 0.6$.

(b) 12-inch diameter samples:

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,i}$ (psi)	B-Value	CSR ($\sigma_{d,c}/2\sigma'_a$)	No. of Cycles to ± 5% ϵ_A
PT-11	55.1	No	29.4	0.979	0.255	6
PT-12	56.3	No	29.4	0.980	0.218	18
PT-14	55.3	No	29.4	0.975	0.283	4

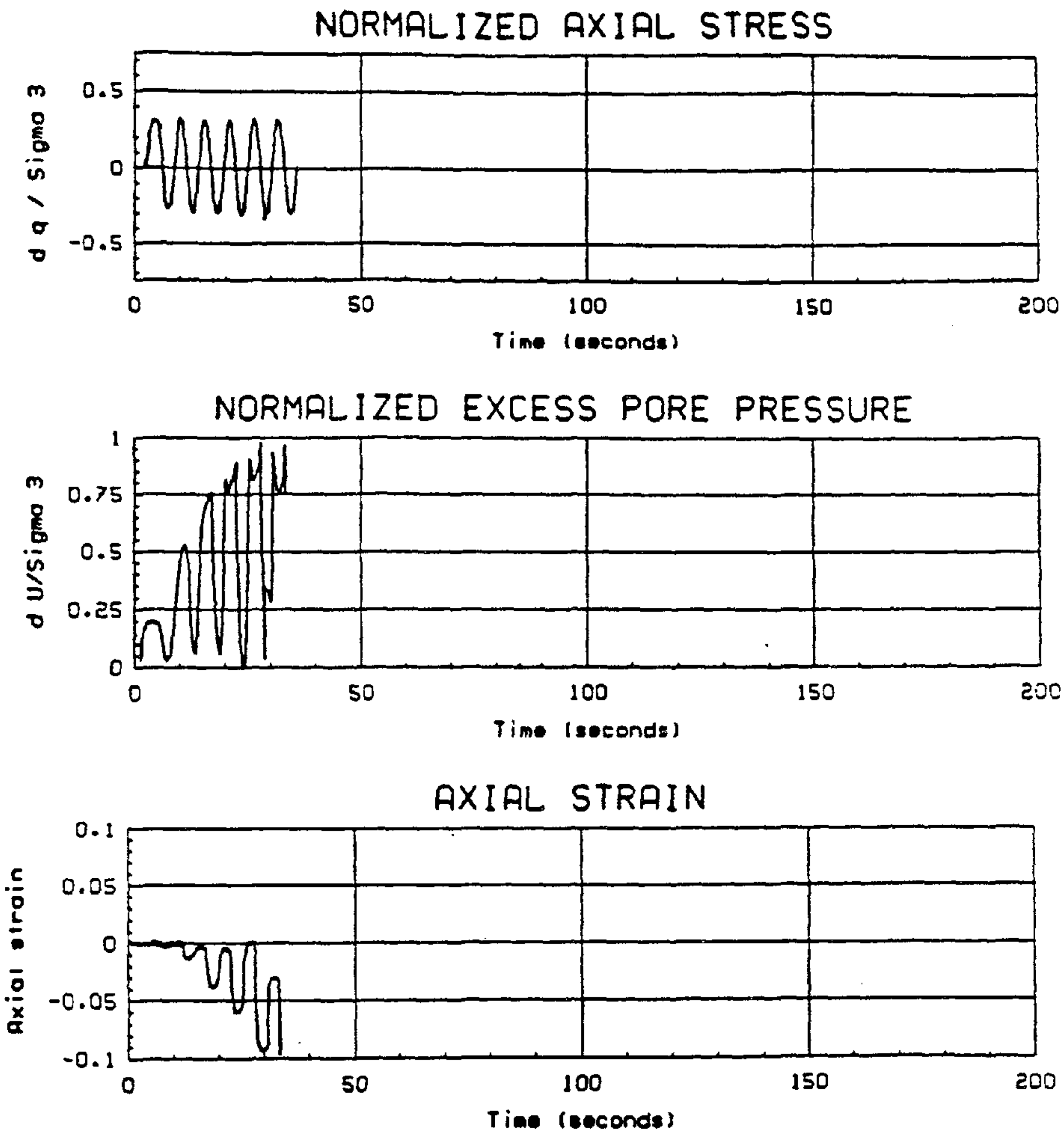


Figure 4.14: Undrained Cyclic Triaxial Test No. 1A (Monterey 16 Sand)

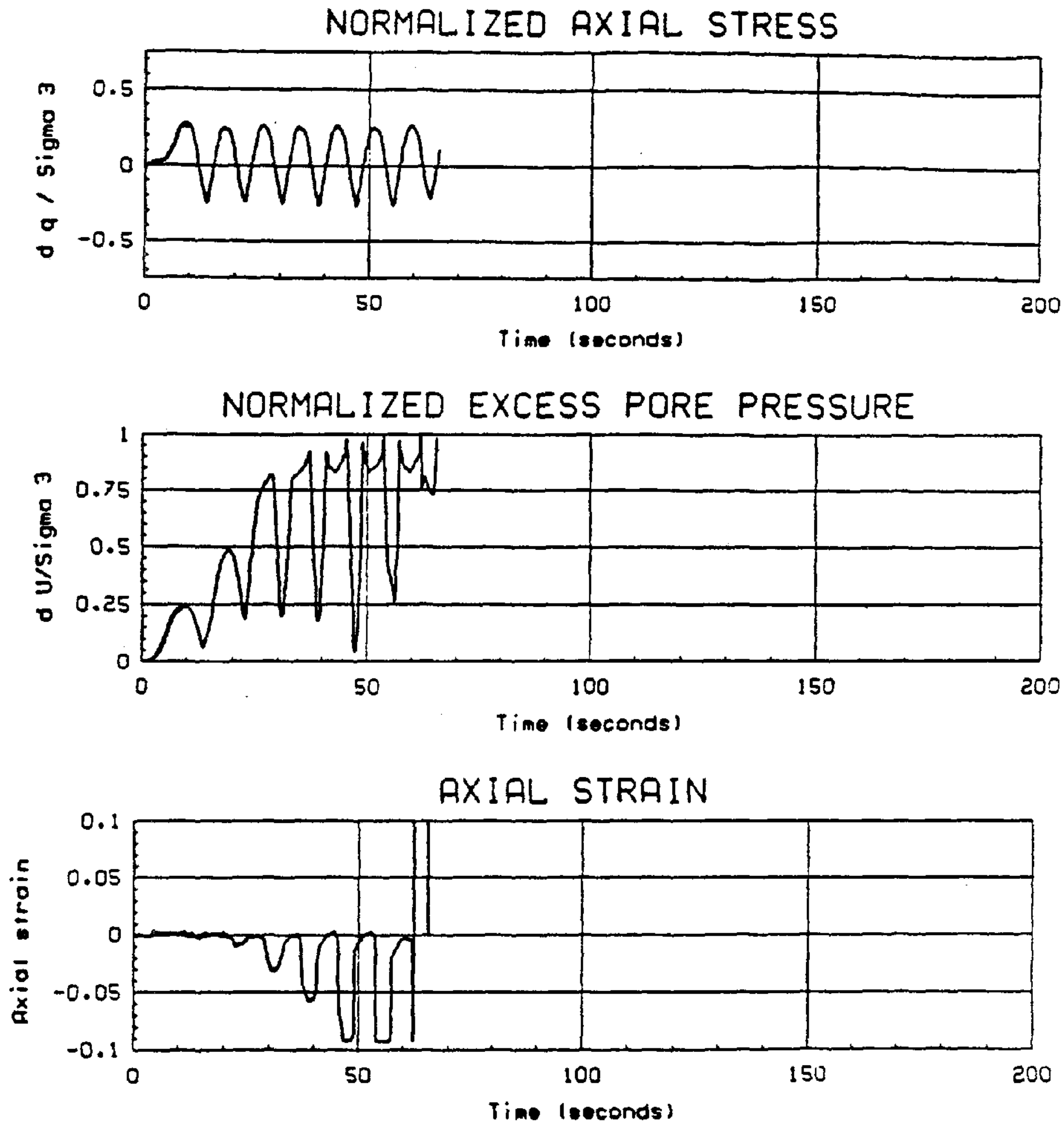


Figure 4.15: Undrained Cyclic Triaxial Test No. 2A (Monterey 16 Sand)

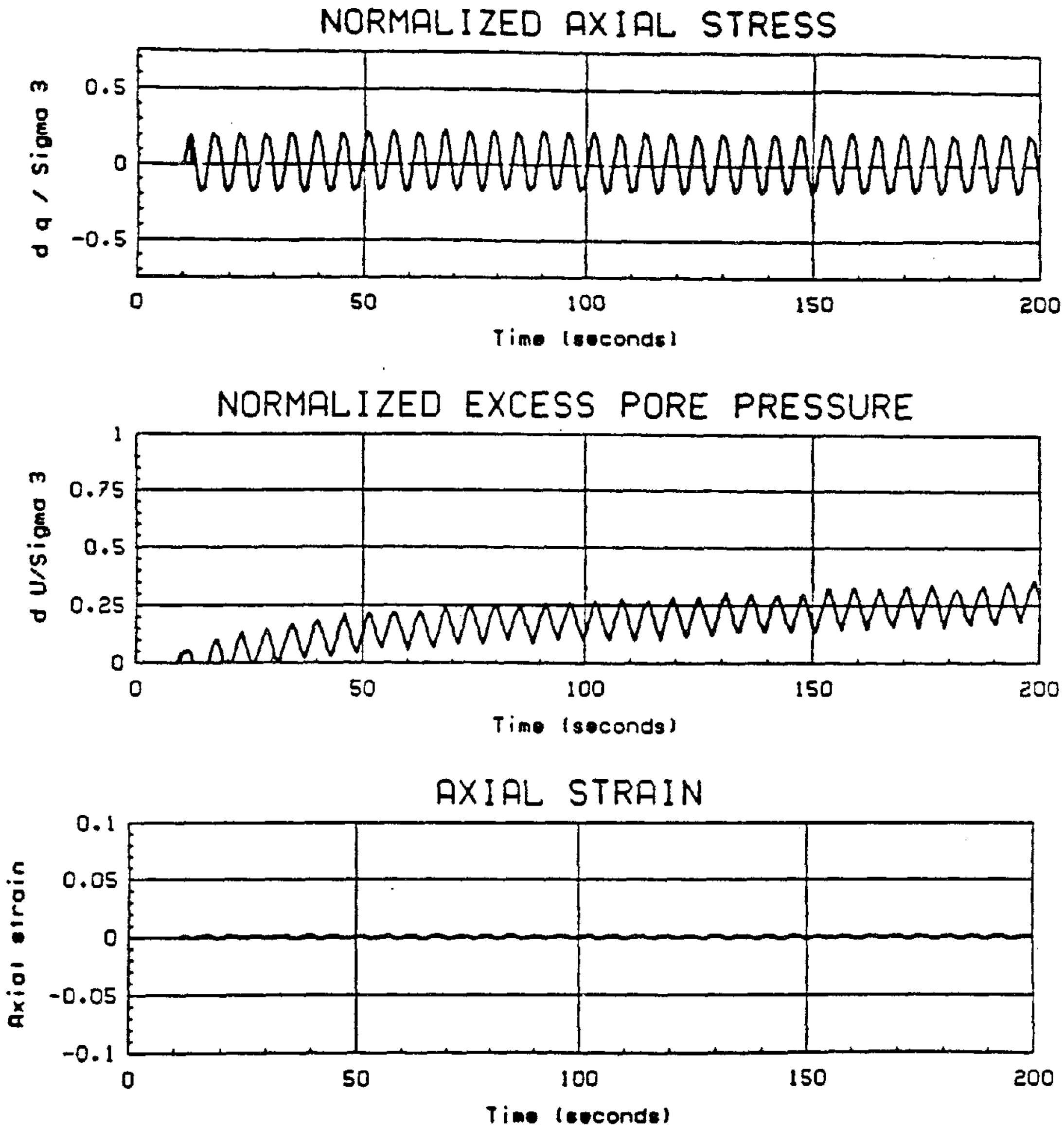


Figure 4.16: Undrained Cyclic Triaxial Test No. 3A (Monterey 16 Sand)

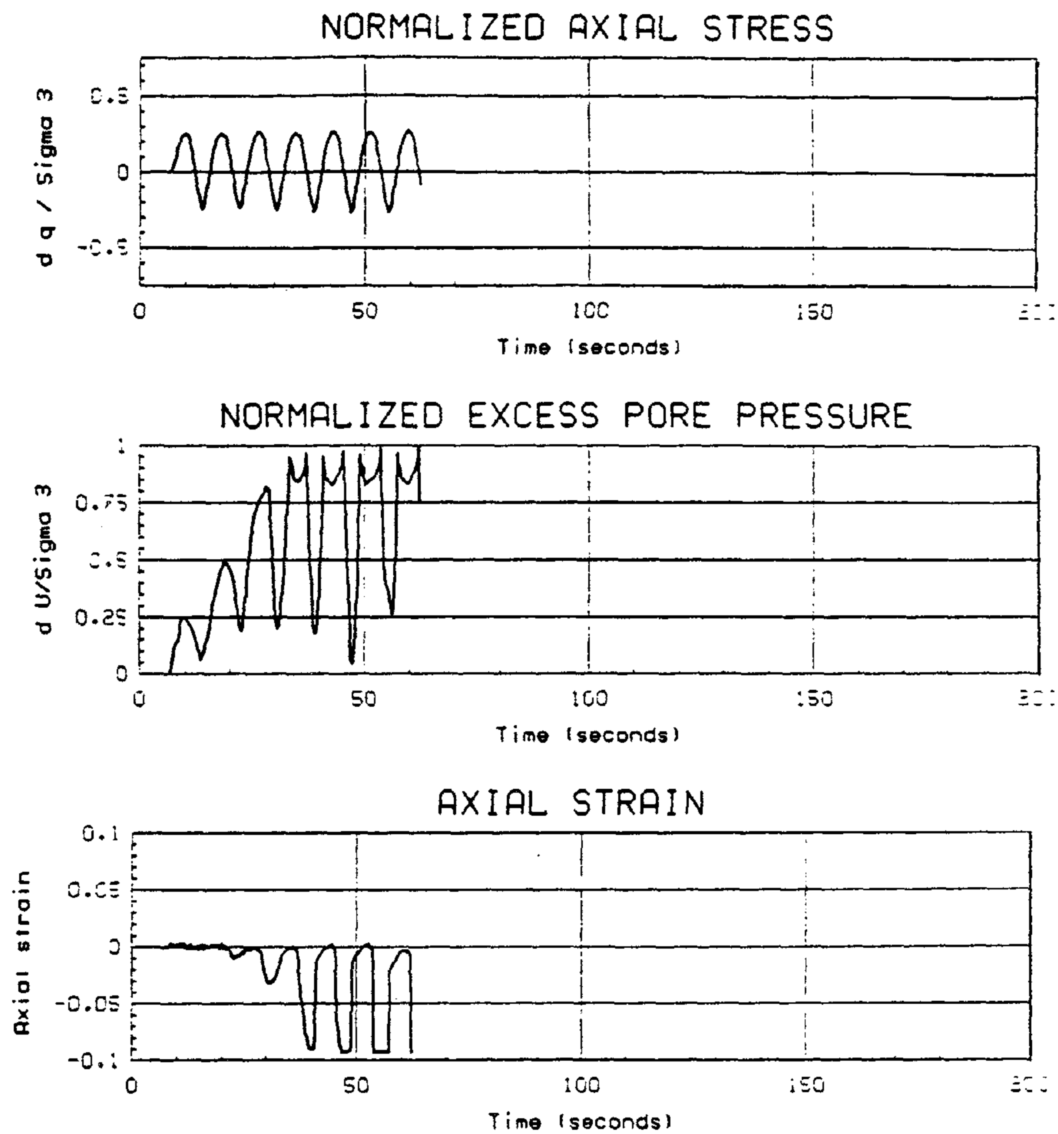


Figure 4.17: Undrained Cyclic Triaxial Test No. 4A (Monterey 16 Sand)

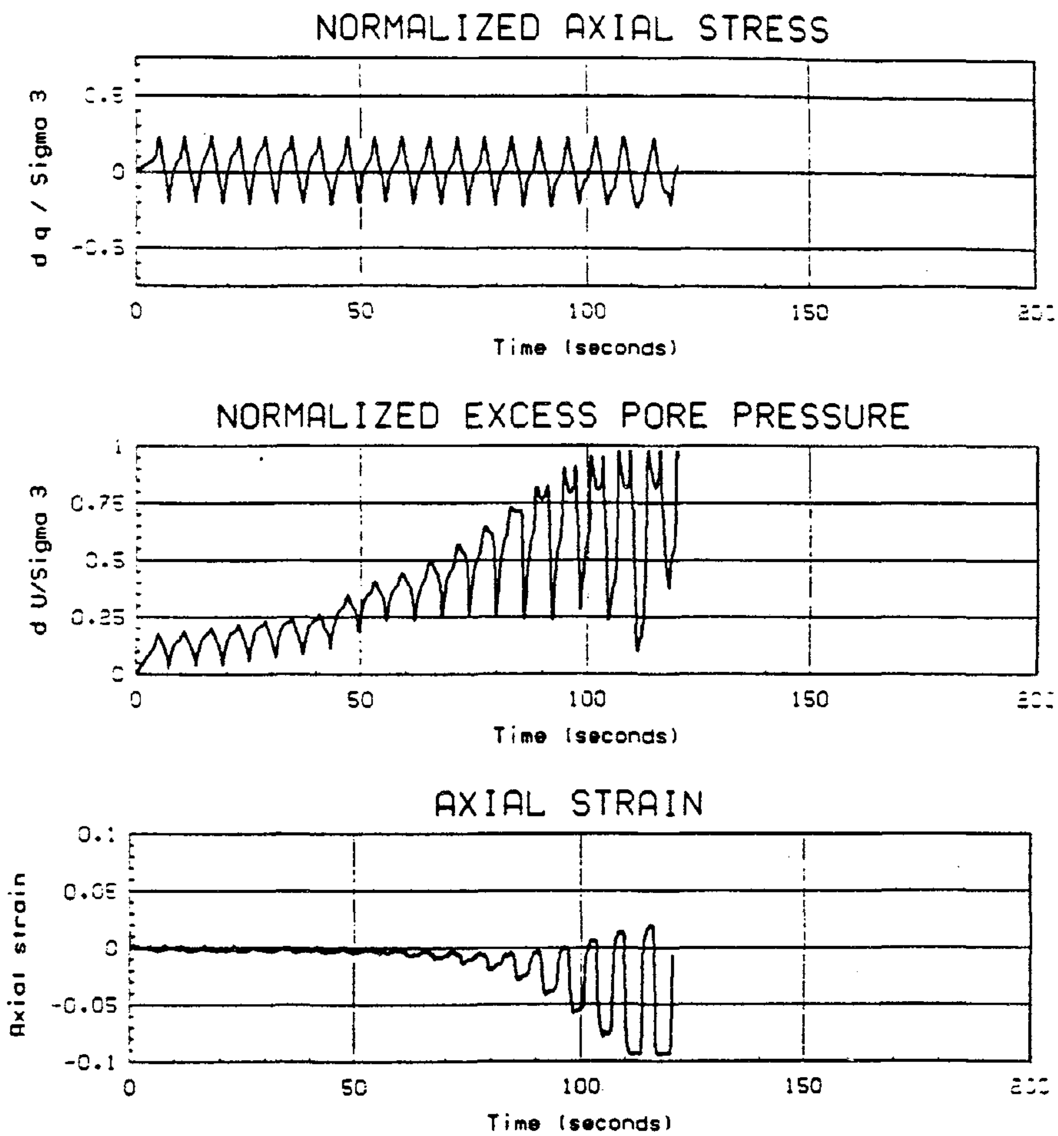


Figure 4.18: Undrained Cyclic Triaxial Test No. 5A (Monterey 16 Sand)

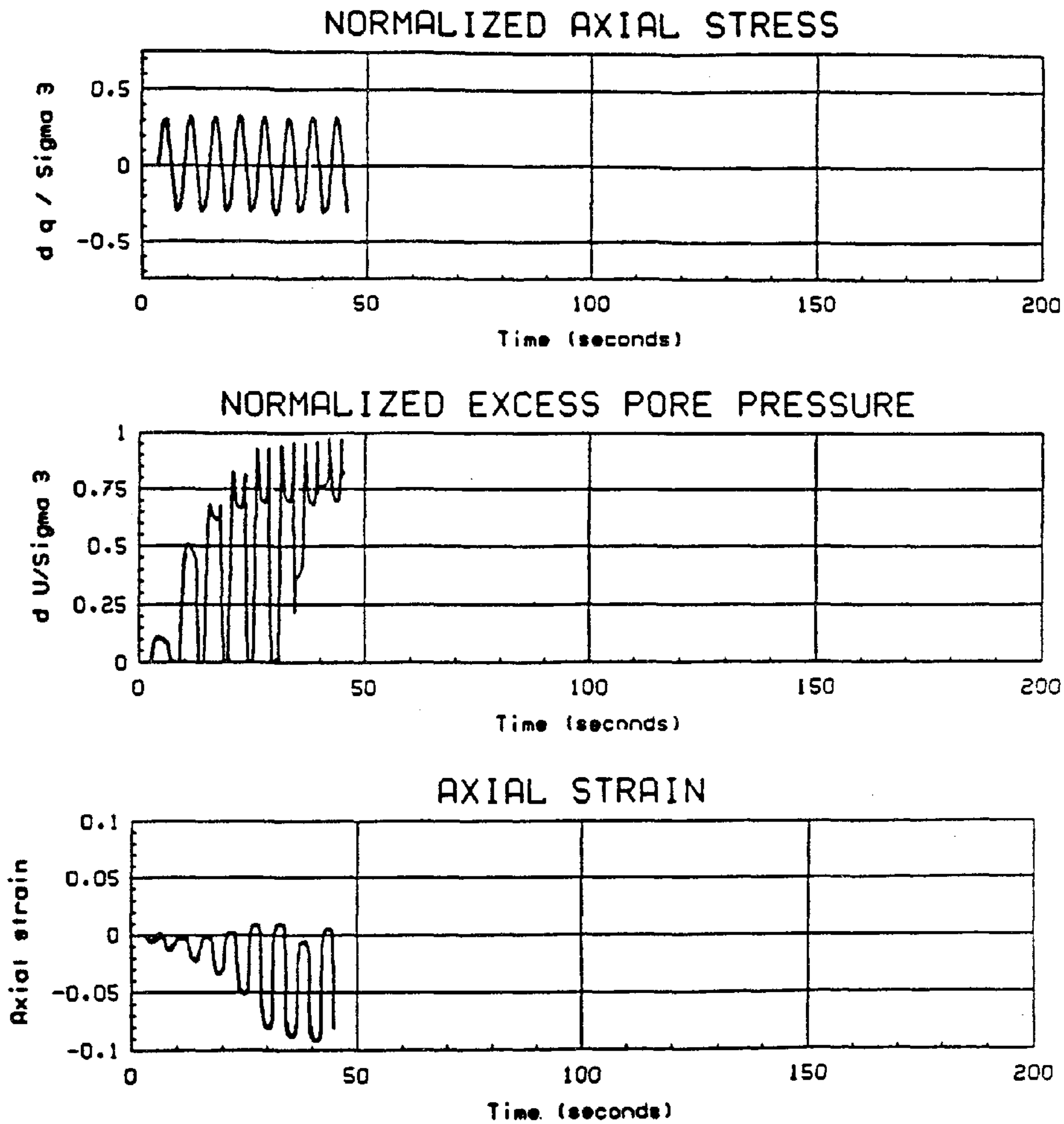


Figure 4.19: <u>Undrained Cyclic Triaxial Test No. 1B (Monterey 16 Sand)</u>

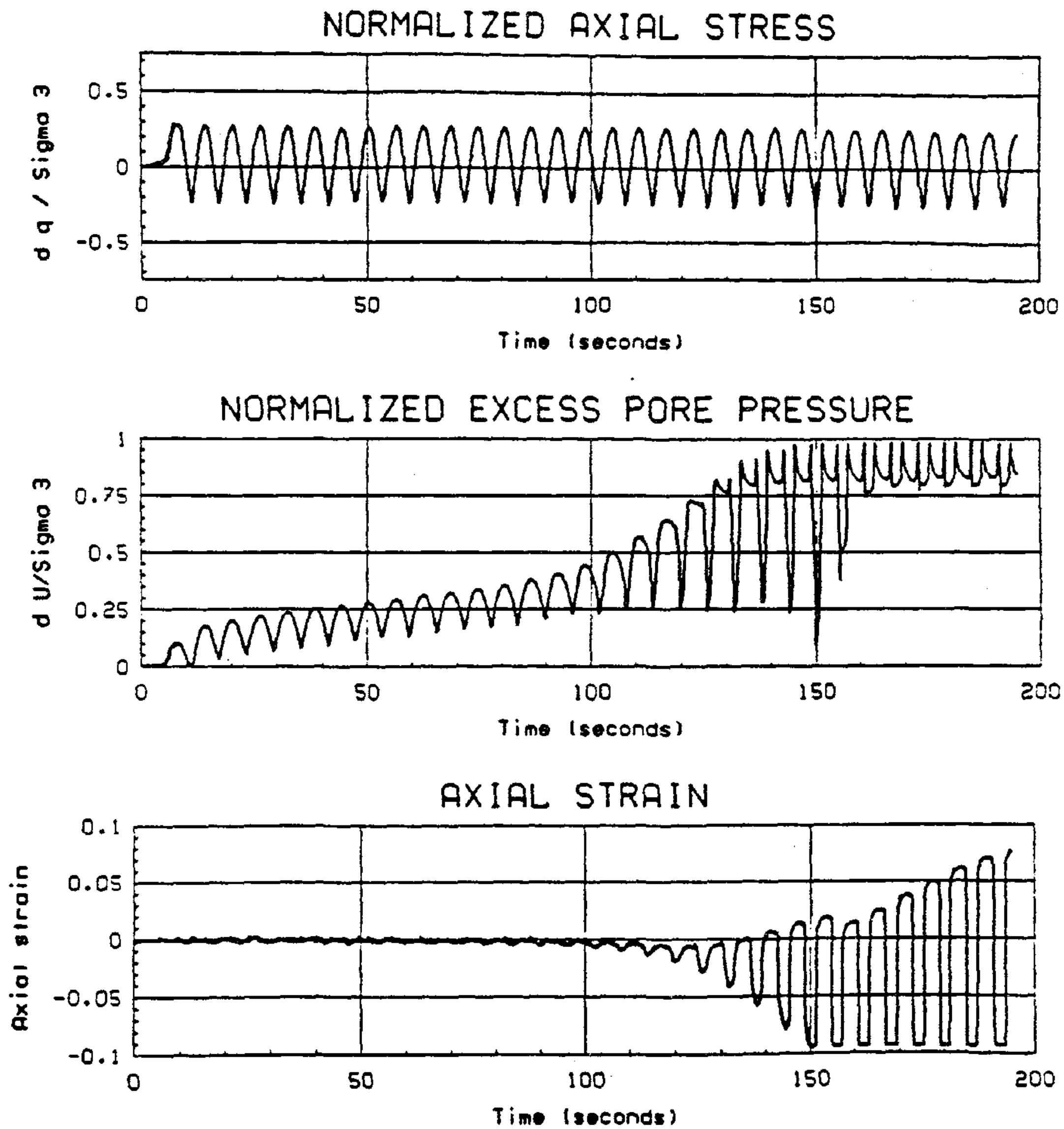


Figure 4.20: Undrained Cyclic Triaxial Test No. 2B (Monterey 16 Sand)

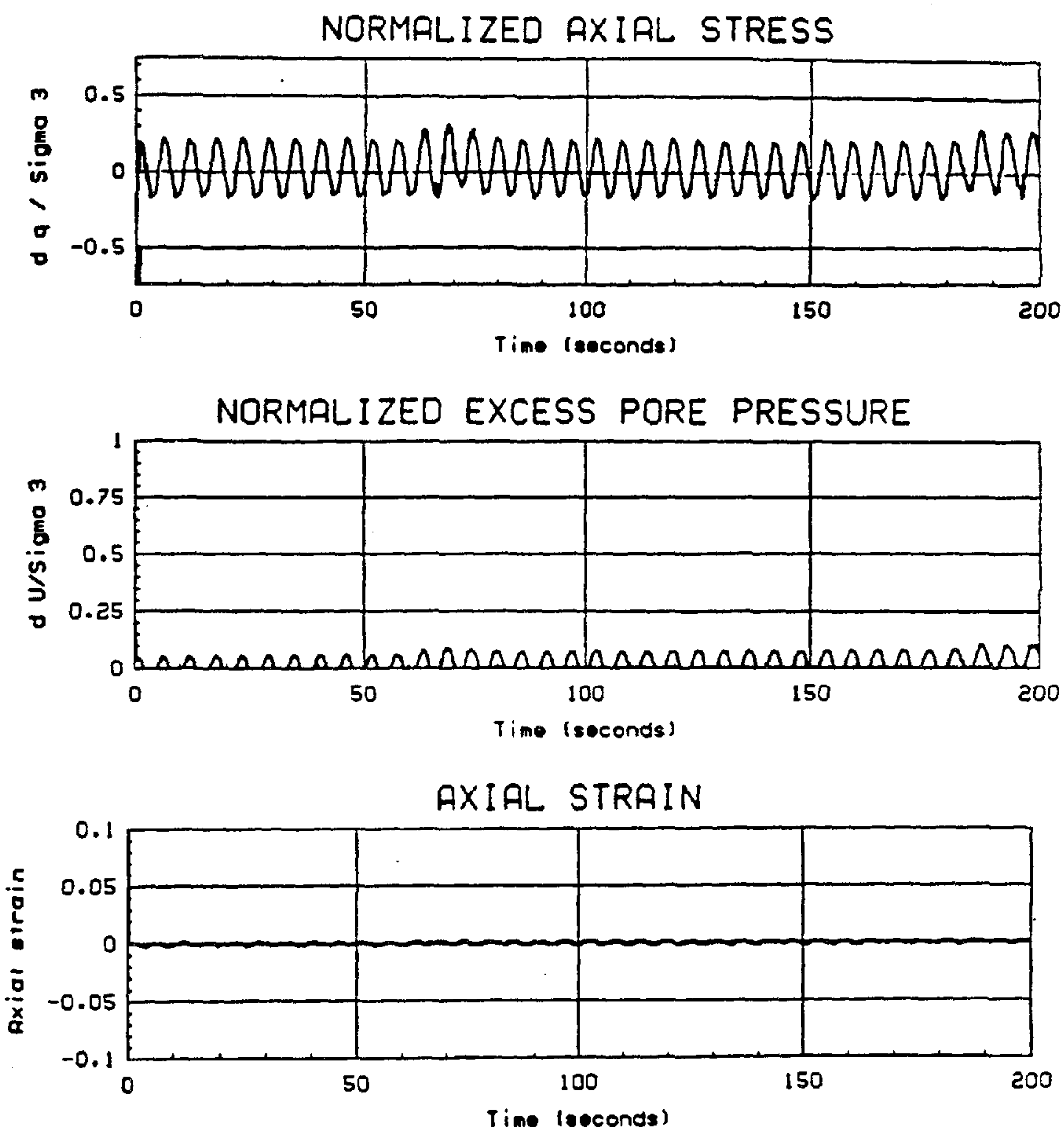


Figure 4.21: <u>Undrained Cyclic Triaxial Test No. 3B (Monterey 16 Sand)</u>

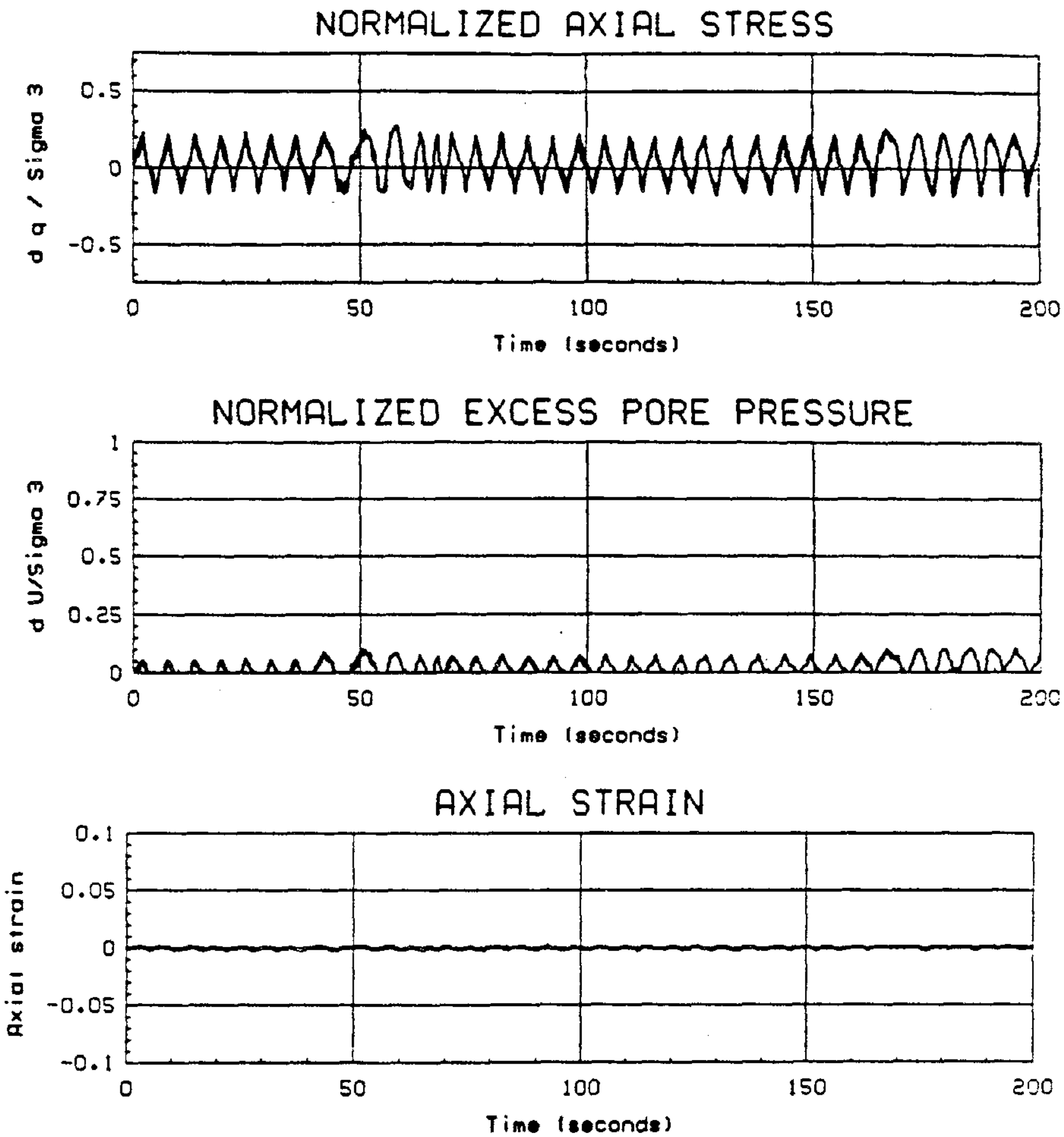


Figure 4.22: Undrained Cyclic Triaxial Test No. 4B (Monterey 16 Sand)

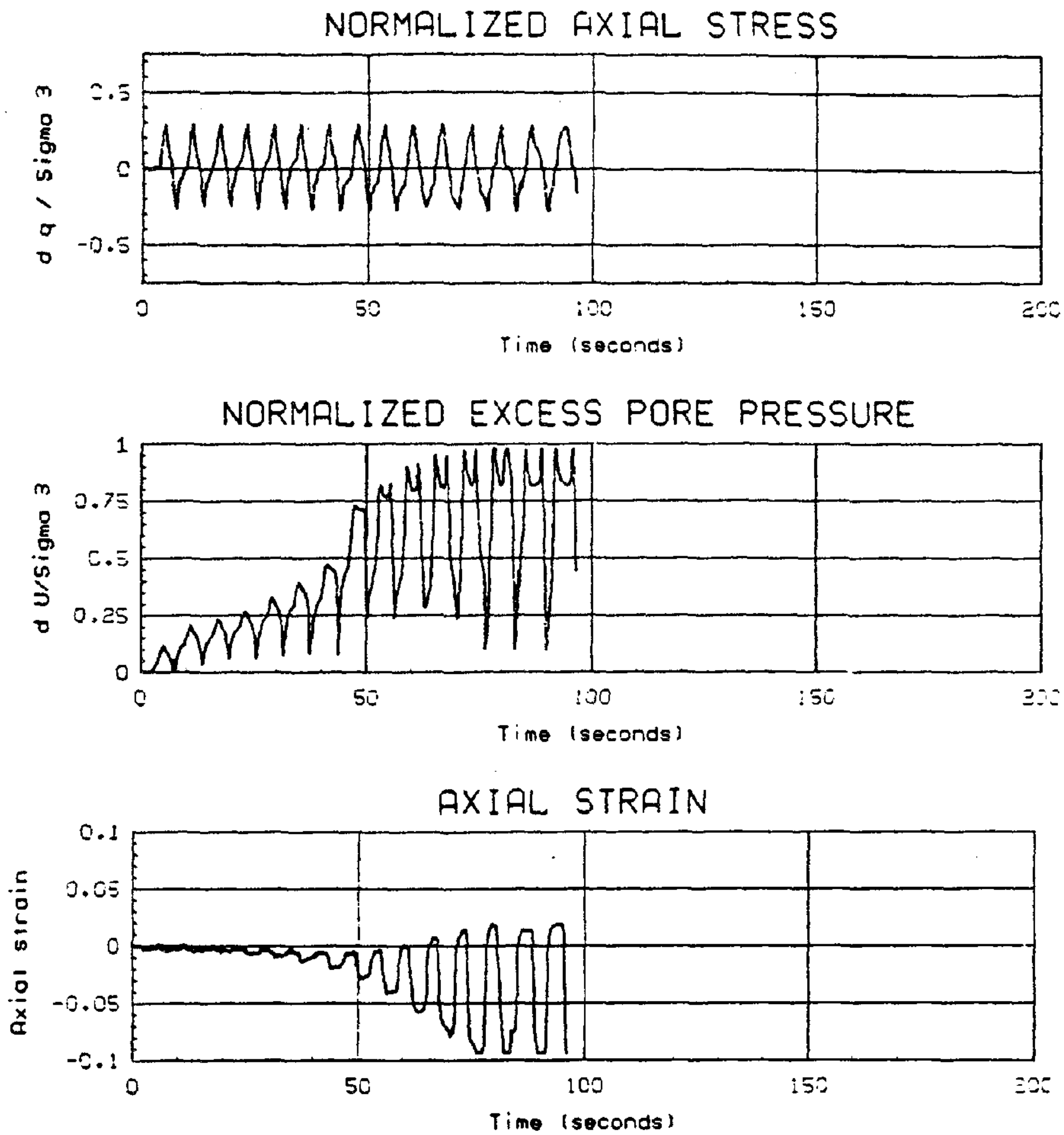


Figure 4.23: Undrained Cyclic Triaxial Test No. 5B (Monterey 16 Sand)

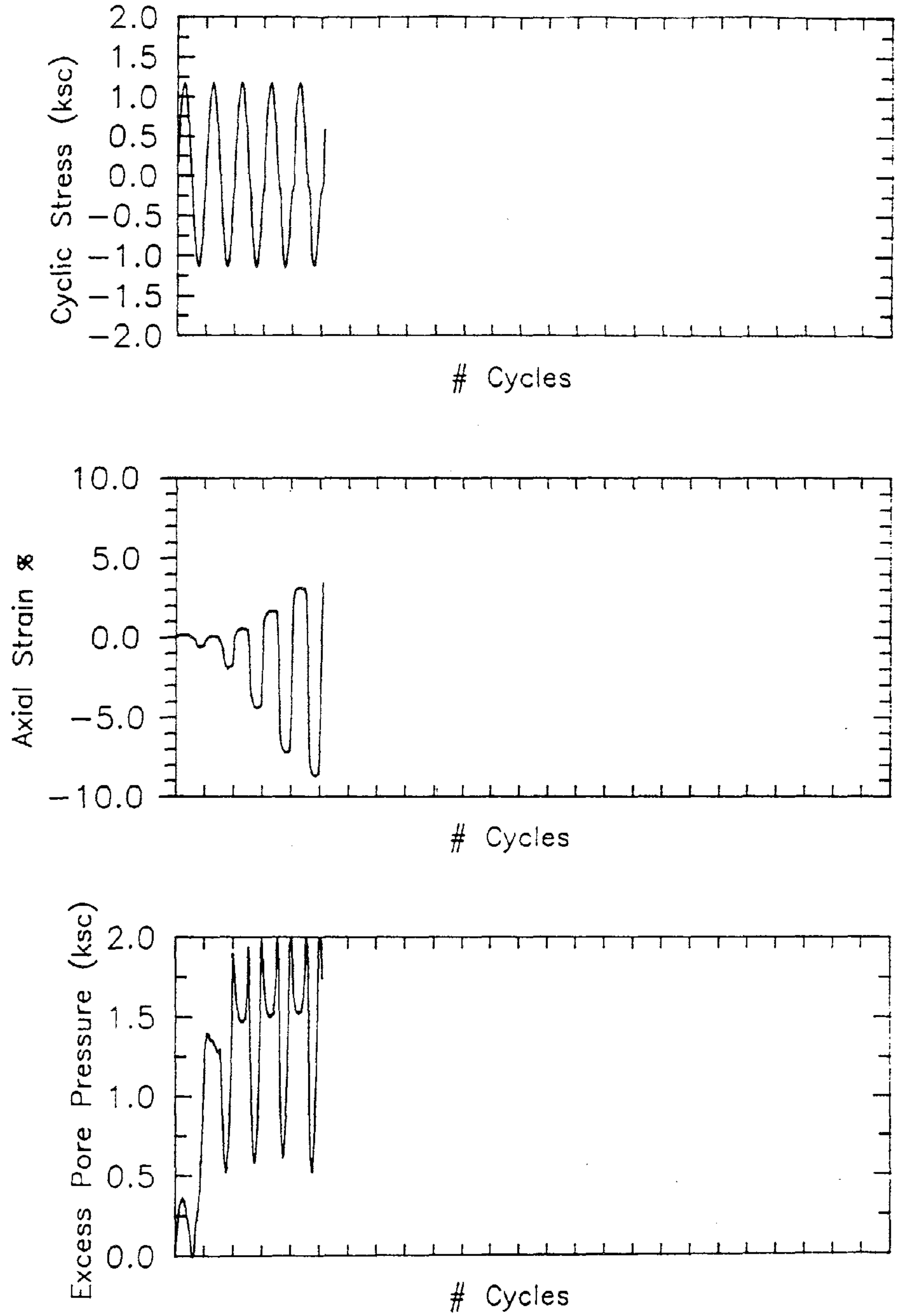


Figure 4.24: Undrained Cyclic Triaxial Test No. PT-14 (Monterey 16 Sand)

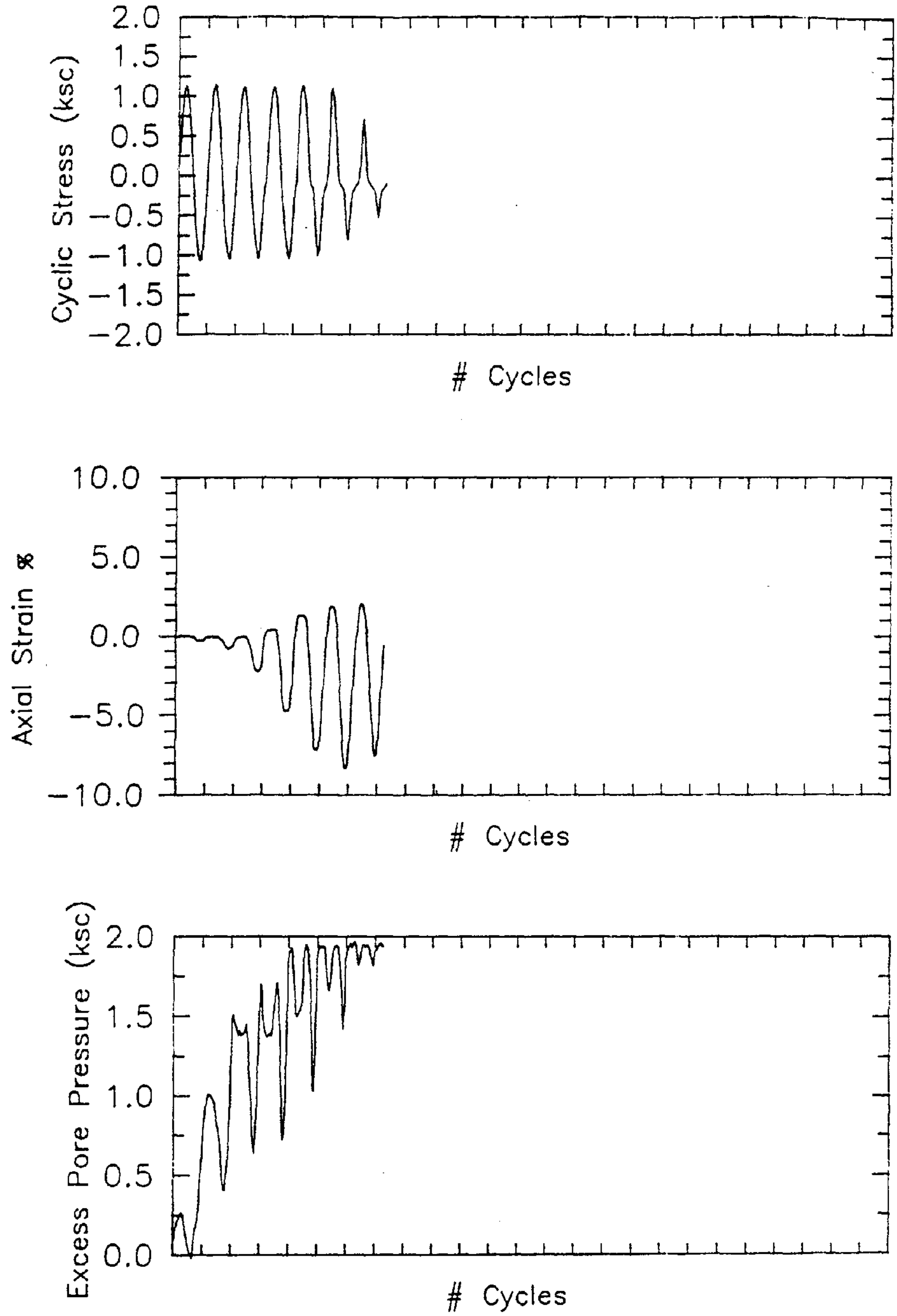


Figure 4.25: Undrained Cyclic Triaxial Test No. PT-11 (Monterey 16 Sand)

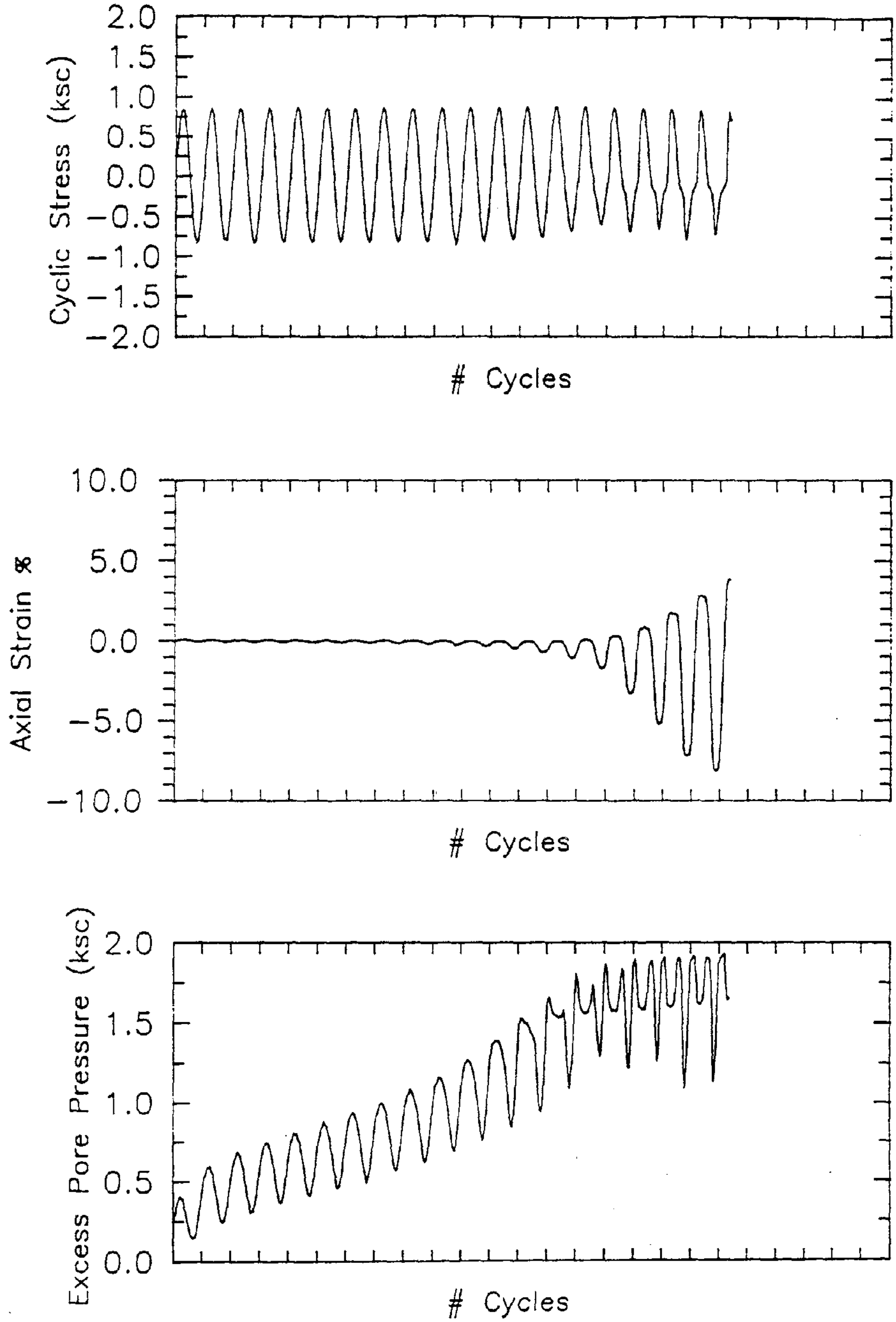


Figure 4.26: Undrained Cyclic Triaxial Test No. PT-12 (Monterey 16 Sand)

to incorporate the larger volume corrections anticipated for large scale (12-inch diameter) samples of significantly coarser materials.

4.3 Testing of 12-Inch Diameter Samples

4.3.1 Testing Equipment/Hardware

The triaxial testing set-up for 12-inch diameter samples was located at the California Department of Water Resources Rockfill Testing Facility in Richmond, California. Figures 4.27 and 4.28 show a photograph and schematic of the testing set-up, respectively. The testing system consisted of a servo-controlled 12-inch hydraulic ram controlled by an IBM PC-AT compatible microcomputer equipped with a Metrobyte A/D board to convert between analog and digital signals. A 100,000 lb. load cell was mounted at the top of the confining chamber with which loads could be read with an accuracy of greater than ± 0.5 psi. Axial displacements of up to 10 inches were recorded by means of an LVDT mounted to the load ram, such that ± 5 inches could be applied to the samples, and axial deformations were continuously monitored with an accuracy of ± 0.01 inches.

Effective sample confining pressures were continually monitored by a calibrated differential pressure transducer with one side connected to the back-pressure/pore-pressure line and the other directly to the top of the testing chamber to monitor chamber pressure. Confining pressures were monitored with an accuracy of ± 0.005 ksc. Both the back-pressure and the chamber pressure were also visually monitored by separate pressure gages with scaled increments of 0.01 ksc.

Volume change devices included a set of three calibrated burette cylinders, each with a different diameter, so as to be able to accommodate a wide range of different possible volume changes which various soils would generate, without

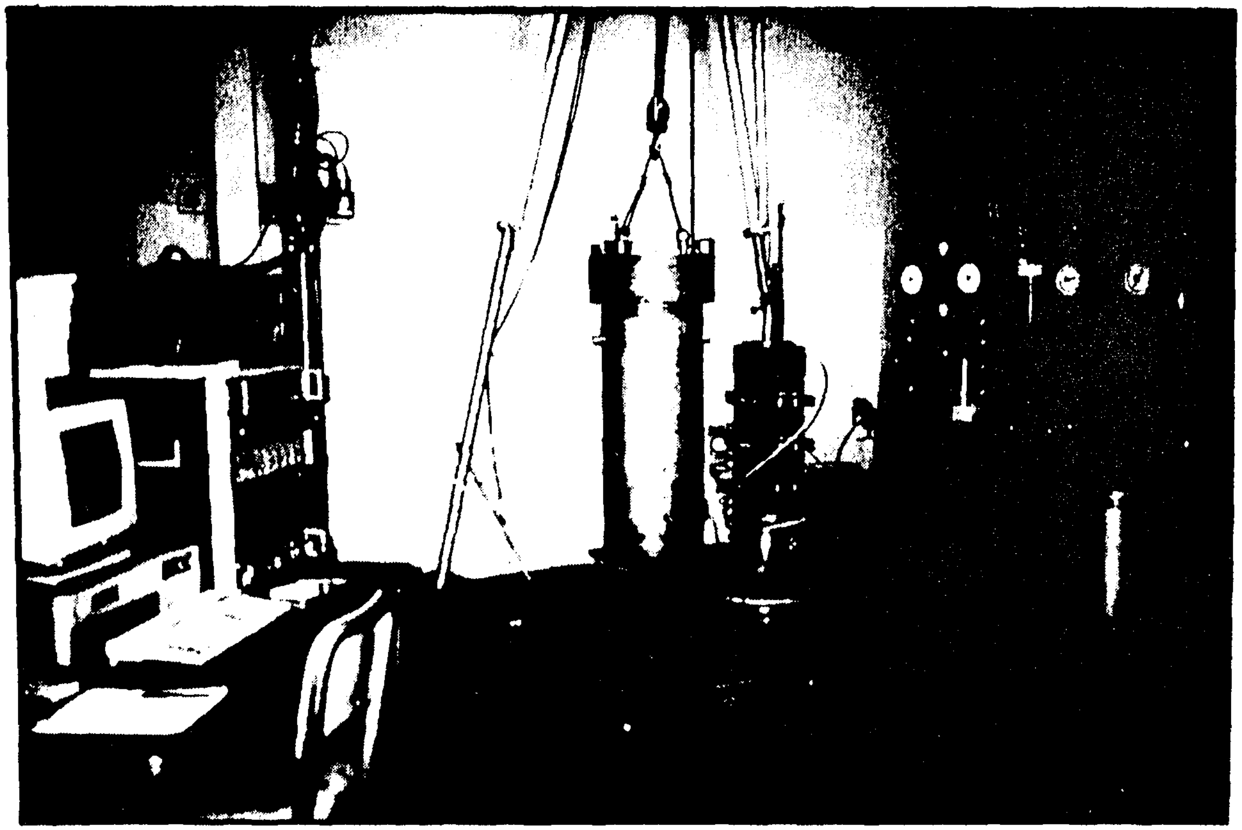


Figure 4.27: Photograph of Large-Scale Testing Equipment for Testing 12-Inch Diameter Specimens

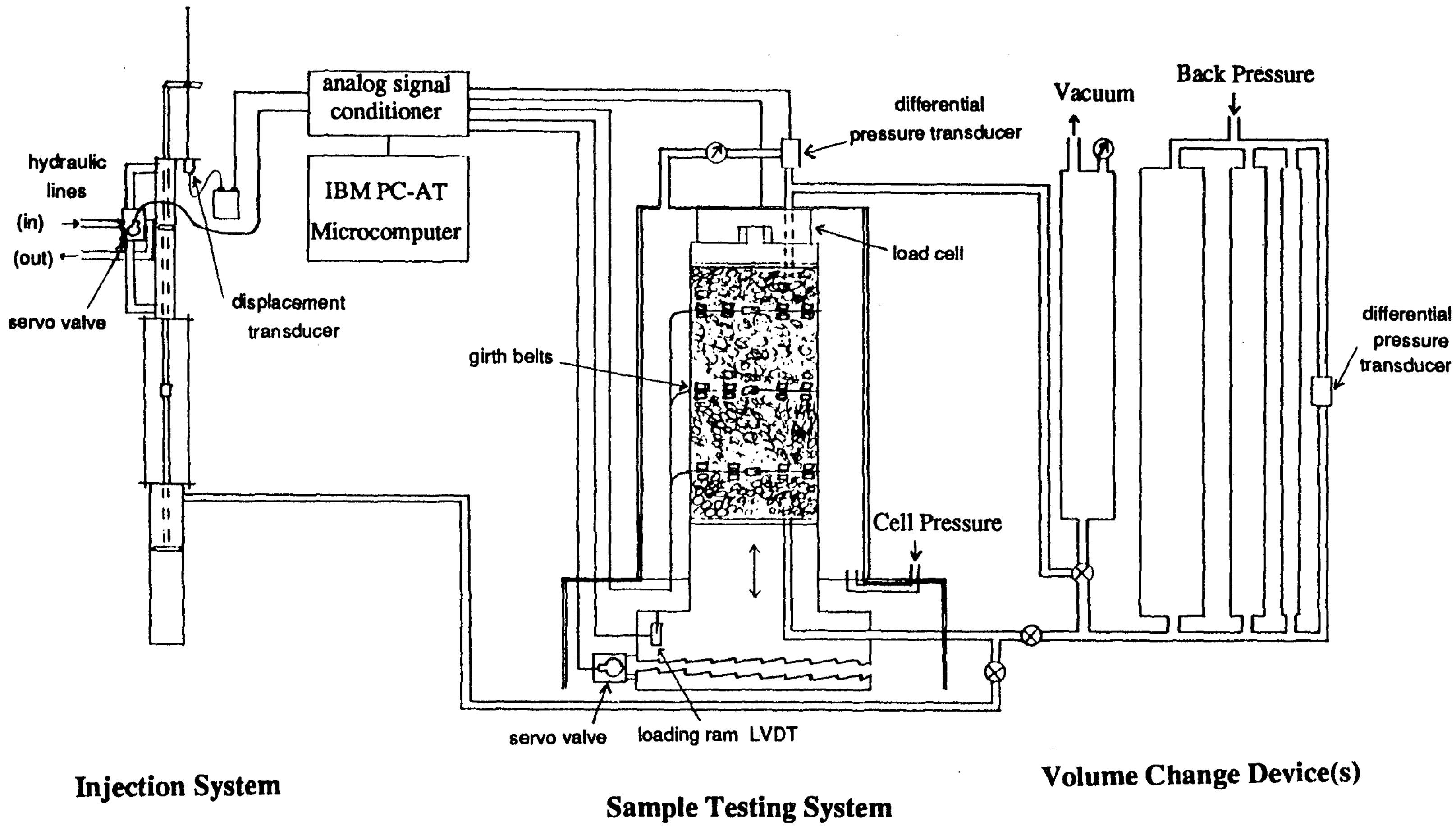


Figure 4.28: Schematic Illustration of Large-Scale Testing Set-Up for Testing 12-Inch Diameter Specimens

sacrificing accuracy of volume change readings. A differential pressure transducer was connected between the top and bottom of the volume change measuring system so that volume change readings could also be recorded directly by the data acquisition recorder. Depending on which of the three burette cylinders was used, the accuracy of volume change measurement ranged from ± 0.04 in^3 to ± 0.2 in^3 or roughly .005% to .025% of the overall sample volume.

All LVDT's, load cells, and pressure transducers were connected to analog signal conditioners (Paul Gross Associates SC-5, SC-5A) where voltage gains and sensitivities could be calibrated to give the best range of signals while maintaining the highest level of accuracy possible. The signal conditioners were then connected to the Metrobyte A/D conversion board installed in the micro-computer.

4.3.2 Controlling Computer Software

The tests were performed using an automated testing control program (ATS) from Digital Control Systems, Berkeley, which is capable of controlling monotonic and dynamic loading test as well as displacement-controlled tests. This software package was found to give excellent control of testing parameters as gains and sensitivities for each control channel could be input at the beginning of each test and could also be edited and updated during testing if necessary for testing accuracy. The ATS software is also capable of recording up to 16 channels of data and writing the data to a specified output file in the desired testing units. Unique test-specific output recording schedules could be easily designed and selected by the software program, enabling the user to choose the frequency of data acquisition for any number of different parameters during the tests. The software can also simply convert any data to other desired units through a built-in subroutine. The ATS testing programs could be quickly and efficiently modified to alter all pertinent testing parameters, and the system was able to run up to four different types of tests simultaneously. This simultaneous test ability made it possible to repeat the same test with or without the compliance mitigating injection program, so that only one variable would be changed between the two tests.

4.3.3 Sample Preparation

All large-scale specimens were nominally 12-inches in diameter and approximately 24 to 26 inches tall. Samples were prepared by "moist tamping" whereby 8 equal volume layers were compacted within a 12-inch diameter steel mold using a hand-held tamper with a round 5-inch diameter plate at the end of a rod. Each layer of material was individually prepared with the proper weighed gradation and completely mixed at a specified water content in order to achieve a consistent uniformity throughout the layer. The length of the tamping rod was adjusted for each layer to ensure that the material was compacted to the correct volume and thus to the desired density. Uniform density throughout the entire sample was achieved by varying the weight of soil in each layer with slightly less material at the bottom and increasing incrementally towards the top of the sample, as the compaction of each successive layer tends to further densify previously compacted layers. Different amounts of weight variations were tried for each desired sample density until the samples displayed uniform densities. Higher density samples required less variation of weight between layers than did loose samples, as the lower layers of the denser samples resisted further densification from the compaction of overlying layers. The flat tamping plate and tamping guide assured a "flat" horizontal surface on top of the last compacted layer on which a porous disk and top cap were placed. It was found that in order to construct gravel samples with very low relative densities (on the order of 10-25%) it was necessary to pluviate the material through standing water and then use the tamper to achieve the desired volume per layer. With some practice, uniform samples of different low densities could be constructed by varying the fall heights through different heads of standing water. The method of low density construction used by Evans and Seed (1987) of hand placing a few grains at a time into the mold was attempted, but it was found that better control of uniformity could be achieved for very low densities by the "wet pluviation" method.

A vacuum of nearly one full atmosphere was applied to confine the sample while the sample mold was removed and the rubber membrane was securely sealed to the top cap. An initial check for leaks was conducted once the internal sample pressure reached that of the full vacuum, by checking to see whether or not the sample could hold the full vacuum while all valves to the sample were closed. If any leakage was detected, it was located and repaired before sample preparation was resumed.

Sample dimensions were then measured in order to evaluate the actual initial sample density. Height measurements were made at 90 degree intervals around the circumference of the sample, and the average of the four readings was recorded. Sample diameters were measured at three equally spaced intervals at the top, middle, and bottom quarter points of the sample height. This was done using a "pi tape" which is a flexible verniered scale, from which diameters can be read by wrapping the scale around the circumference of a sample. These were then corrected to account for membrane thickness.

Since accurate pore water pressure readings were critical to the types of undrained triaxial testing performed as part of this research, care was taken to ensure the highest possible saturation of samples prior to testing. Saturation was achieved by the following systematic "vacuum/back-pressure" saturation method.

Initially, deaired water was allowed to be drawn up through the sample from the bottom to the top under a small differential vacuum, while maintaining a pressure differential of not greater than 5.0 psi to safeguard against preconsolidating the specimen. Once water flowed out of the top of the sample with no further detectable traces of air bubbles, the vacuum line was shut off. The testing chamber was then secured in place and the sample was manually raised just until contact was made with the load cell at the top of the chamber. It was then securely fastened to the chamber and the chamber was sealed. As more deaired water was allowed to be drawn into the sample under the remaining vacuum in the sample, the chamber pressure was simultaneously increased to maintain the initial effective confining stress of approximately 1 atmosphere. While continually maintaining the initial effective pressure, the chamber pressure was slowly increased and a back-pressure was simultaneously applied to the sample pore water until it was estimated that sufficient back pressure had been applied to achieve a B-value of at least 0.97. At this stage a rigorous check of the B-value was made. The B-value parameter, equal to the ratio of the change in pore pressure for a given change in chamber pressure while the sample was in an undrained condition, was obtained by monitoring the change in effective stress while a small increment of additional chamber pressure was applied, and making the necessary computations. If saturation was not deemed sufficient, chamber pressure and back-pressure were increased further, and another B-value check was made. This process was continued until the minimum standard of saturation ($B \geq 0.97$) was achieved or exceeded. In some cases it was necessary to use tanks of compressed air to achieve the high chamber and back pressures necessary for high degrees of saturation.

Samples were then fully consolidated under the desired effective consolidation stress, after which they were ready for testing. A minimum consolidation time of approximately two hours was typically used even for rapidly draining coarse gravels, as it has been demonstrated by previous investigators (Molenkamp and Luger, 1981; Baldi and Nova, 1984; Seed and Anwar, 1986) that creep effects and strain softening of the membranes may affect compliance values.

CHAPTER 5

DEVELOPMENT OF A LARGE SCALE MEMBRANE COMPLIANCE MITIGATION SYSTEM

5.1 Introduction

There has been great progress made over the last 25 years in the area of obtaining more correct undrained test results for materials that are prone to the errors induced by membrane compliance. But prior to this effort, there had not yet been any verification of a reliable and accurate means by which to eliminate or correct for the volumetric error introduced by membrane compliance during undrained testing. As described in the previous chapter, the method of continuous computer-controlled injection or removal of water to or from a sample to offset the membrane compliance-induced error, has been verified as just such a valid method of achieving this goal for "conventional" (2.8-inch diameter) small-scale samples.

It has been clearly shown by numerous investigators that the effects of membrane compliance become more pronounced for coarser materials. As pointed out in Chapter 1, many earth structures are constructed of, or founded on, coarser materials (particularly gravels) which may be susceptible to significant membrane compliance errors. These coarser soils have historically been considered to have a "strong" resistance to liquefaction based on tests in which membrane compliance induced errors may have been quite significant. There is therefore a need to develop large scale testing techniques which will be able to provide more accurate assessments of the actual undrained strengths of these coarser materials. Unfortunately, the "coarseness" of the material that can be tested in the small-scale apparatus is limited by the sample size constraints of conventional triaxial testing equipment. It has been suggested (Seed, 1979) that the maximum ratio of material particle size to sample diameter should not exceed 1:8 for uniformly graded materials and 1:6 for other soil gradations. As a result, the coarsest material that can be accurately evaluated for its undrained strength or resistance to liquefaction under cyclic loadings using conventional testing equipment, even with the implementation of a continuous computer-controlled compliance mitigation system such as that developed by Seed and Anwar (1986), is a coarse sand. For materials coarser than coarse sand, larger testing equipment must be employed.

The first step in developing this type of compliance-mitigation system is to show that the amount of volumetric compliance can be accurately and reliably pre-determined for a soil specimen prior to any test in which a compensating correction would be made based on such a pre-determination. Once this is achieved, a system must be designed and constructed that will continuously and completely offset that volume error with sufficient speed and accuracy throughout undrained testing of a specimen. Therefore, the methodology used in developing the large-scale system to completely mitigate membrane compliance includes two basic parts: (1) the volumetric magnitude of membrane compliance must first be pre-determined for the soil to be tested, as a function of the sample effective confining stress, and (2) the volumetric error introduced by membrane compliance during undrained testing must be continuously offset by injecting or removing an amount of water equal to the volumetric error by means of a computer-controlled system.

5.2 Pre-Determination of Membrane Compliance

For the "large-scale" triaxial samples, which had diameters of approximately 12 inches, membrane compliance was measured directly by recording axial and radial strains of the soil skeleton, and subtracting the "true" sample skeletal volume change from the total volume change measured by the volume change device. Axial strains were recorded by monitoring the LVDT that measured displacement of the 12-inch diameter loading piston. Radial strains were recorded using a set of three "girth belts" (discussed in Sections 2.2.1 and 3.3), each containing an LVDT, which measured changes in sample circumference during the application of varying confining stresses. Use of the radial girth belts allowed direct calculations of volumetric strains without making any of the customary assumptions regarding relationships between axial and radial strains.

For volumetric compliance evaluations made for samples of less than six inches in diameter, however, the measurement of radial strains is difficult to perform with sufficient accuracy as to make the use of radial measurements a viable method of pre-determining compliance magnitudes for such "smaller" samples. The method of assuming isotropic behavior during isotropic unloading tests, where volumetric strain is assumed to be nearly three times the axial strain, has been shown to lack reliable accuracy for a range of different soils. In considering the assumption of isotropic strain behavior proposed by Vaid and Negussey (1984), it should be noted that for the sand tested, the volume change due to membrane compliance was considerably greater than that of sample skeleton volume change, such that errors introduced by any inaccuracy of the isotropic strain behavior of the soil sample would be of much less significance than for a soil sample in which the volume change components were more closely equated. Instead, the "two sample scale model" method proposed by Seed and

Anwar (1986), and discussed in Section 2.2.2, appeared to be the most reliable and accurate method by which to make compliance volume evaluations for these smaller samples, and was used for all compliance determinations made for the "smaller" samples in this study. A complete description of the compliance measurements made and the different soils tested is presented in Section 5.3.

5.3 Membrane Compliance Measurements

The magnitude of membrane compliance is virtually always determined by performing drained isotropic compression and rebound tests on saturated samples. Compliance volume evaluations made as a part of this study were performed using such drained test results, from which the magnitude of membrane compliance was determined by subtracting the true soil volumetric strain from the total sample volumetric strain measured.

The membranes used for the majority of tests performed on 12-inch diameter specimens were made of rolled black rubber tire-tread stock obtained from the American Rubber Manufacturing Co. of Emeryville, CA, and were approximately 0.12 inches in thickness. Compliance measurements were made utilizing the large-scale apparatus for a variety of gradations types whose "representative" grain sizes ranged from coarse gravels to coarse sands whose particle diameters approached the thickness of the rubber membrane, at which point measured compliance volumes were expectedly low.

While most of the soils tested for compliance induced volume changes were uniformly graded materials of various sizes, a number of very different gradations (e.g. well-graded and gap-graded) were also investigated in order to further define and generalize the relationships developed. The results of the individual compliance measurement tests and description of each of the soils tested is given in

Section 5.3.2. A compilation of the test results obtained in this study is presented along with those of previous investigators, whose test data was deemed to be of reasonable accuracy, in order to derive an updated correlation for membrane compliance characteristics as a function of material particle sizes.

Most of the samples tested for membrane compliance determinations as part of this study were prepared by "moist tamping" in layers. Exceptions to this rule were "small-scale" samples prepared by dry pluviation to investigate the significance of sample preparation method on membrane compliance measurements.

The testing procedures for determining membrane compliance characteristics were essentially the same for all samples, with some previously mentioned differences in measurement equipment for different sample sizes. Once samples were prepared in their respective sample molds, a vacuum of nearly one full atmosphere was applied to the samples. The samples were then saturated to a high degree with a vacuum/back pressure system (as described earlier in Section 4.3.3) until a B-value of no less than 0.97 was achieved, while maintaining a constant effective confining stress on the sample. After saturation, the samples were subjected to fully drained isotropic loading and unloading while total sample volume changes and sample deformations were recorded. The volume change due to isotropic loading and unloading was measured to an accuracy of $0.001 cm^3$ for conventional 2.8-inch diameter and 1.4-inch diameter specimens, and $7.3 cm^3$ for large-scale (12-inch diameter) samples. For both of these cases, the accuracy of measurements was greater than 0.05% with respect to each of the respective sample volumes.

It has been demonstrated by numerous investigators (e.g. El-Sohby, 1964; Frydman et al., 1973; Keikbusch and Schuppener, 1977; Raju and Venkataramana, 1980; Baldi and Nova, 1984; Seed and Anwar, 1986; etc.) that volumetric membrane compliance magnitude is a direct and repeatable function of sample effective confining stress. It has also been demonstrated that the relationship between compliance-induced volume change per unit area of membrane and $\log_{10}$ of the sample effective confining stress is essentially linear over the range of interest for most undrained testing. The slope of the semi-log function, referred to as the "normalized unit membrane penetration" (S), can therefore be used to characterize the volumetric compliance for a given soil. All of the studies of factors affecting unit membrane compliance indicate that such a pre-determination of volumetric membrane compliance represents a viable basis for control of injection-mitigation during undrained testing.

5.3.1 Evaluation of Factors Affecting Membrane Compliance

A number of investigators have tried to identify the factors that may affect the volumetric membrane compliance of different soils. During the preliminary phase of this investigation a number of possibly significant soil variables were examined in order to evaluate their effect on compliance value measurements. The factors investigated included: (a) soil grain size and gradation, (b) soil density, (c) soil grain angularity, (d) soil fabric or method of sample preparation, and (e) membrane thickness and the use of multiple membranes. An additional important finding was that samples cyclically loaded to a state of liquefaction ($r_u = 100\%$) and then re-consolidated, exhibited membrane compliance behavior essentially identical to that measured on similarly prepared samples not loaded prior to membrane compliance measurements.

The series of tests performed for purposes of evaluating factors that may affect membrane compliance, some of which were previously reported by Anwar et al. (1989), are briefly discussed here in order to identify the importance of each.

5.3.1.1 Soil Grain Size and Gradation

The most important factor affecting the magnitude of measured volumetric compliance was found to be soil gradation characteristics. This finding was in agreement with earlier investigators. In relation to the gradation characteristics of a soil (material grain size and grain size distribution), all other factors were found to be secondary. An important point recognized in reviewing the literature of previous investigations, was that most previous investigators had consistently correlated membrane compliance magnitudes with mean particle size (D_{50}). It was also noticed that virtually all materials examined by previous investigators had been uniformly graded. As discussed briefly in Chapter 2 (Section 2.2.4) it was found that for non-uniformly graded sandy soils, unit membrane compliance was much better correlated with smaller particle sizes (D_{20}) than with mean grain size. These findings were further confirmed by the tests performed in this study on a wide range of gravelly soils. A detailed investigation which more clearly defines the effects and the relationship of soil gradation characteristics on normalized membrane compliance is presented in Section 5.3.2.

5.3.1.2 Soil Density

The influence of sample density on measured compliance values has been investigated by a number of earlier researchers, and has been shown to be of fairly consistent but relatively minor significance. Figure 5.1 shows a plot of unit membrane compliance versus effective confining stress for five samples of Monterey 16 sand tested at relative densities of 45%, 50%, 55% and 60%. As this Figure shows, there is a slight variation in measured compliance for the different densities tested, and it is suggested that the generalized relationship given later in Figure 5.49 is most appropriate for samples of medium density. For samples at

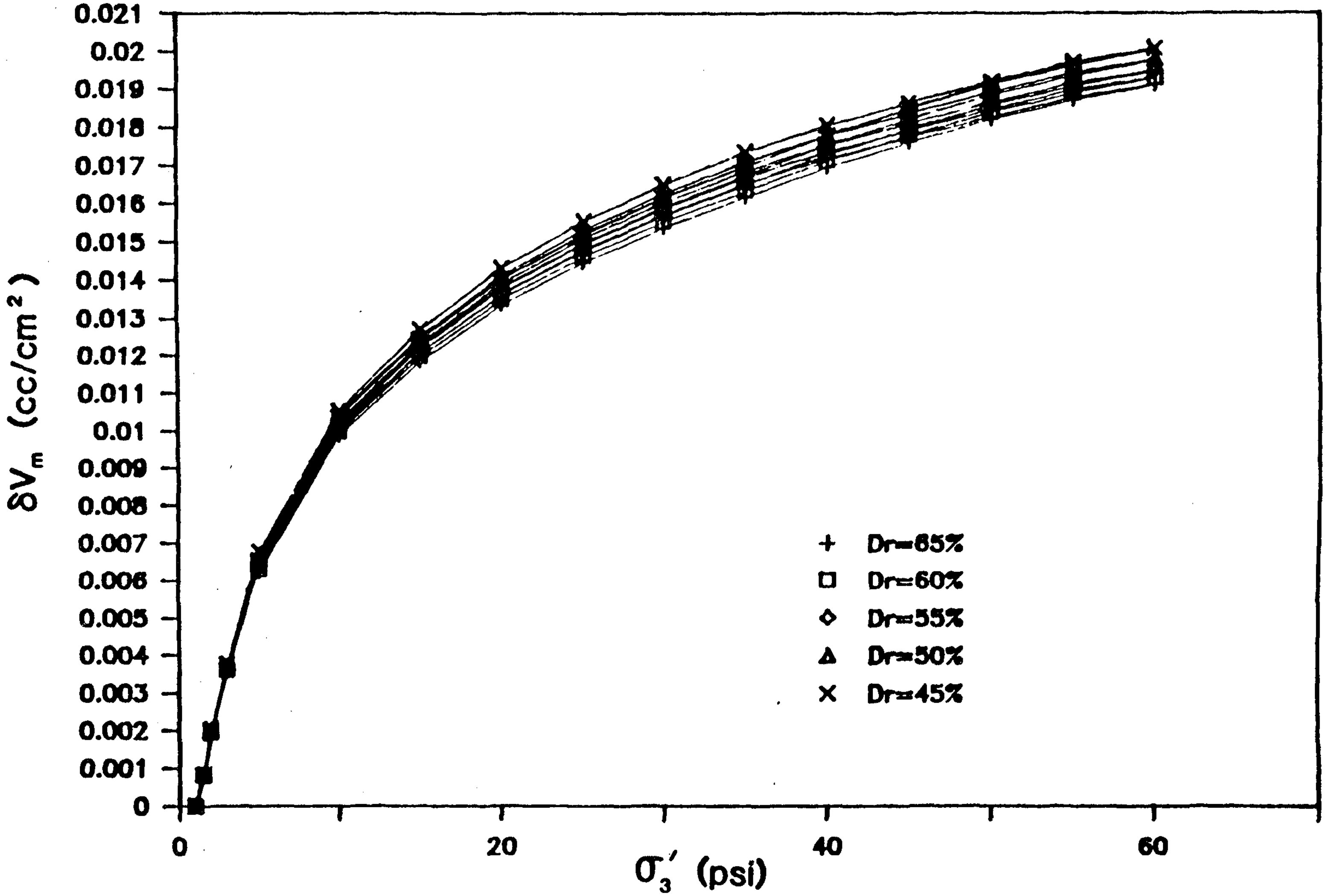


Figure 5.1: Unit Membrane Compliance vs. Effective Confining Stress for Monterey 16 Sand Over a Range of Relative Densities

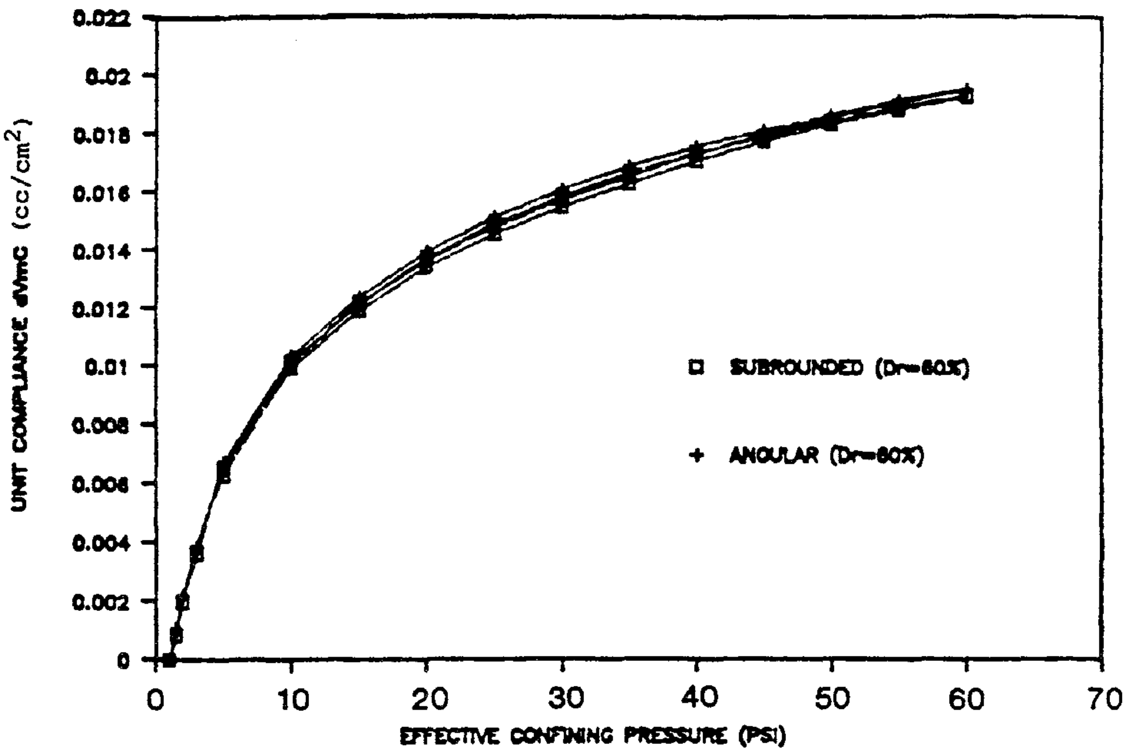
significantly higher or lower densities, adjustments should be made to compliance estimates, or compliance measurements should be made for those samples. It has been suggested that the maximum adjustment to S values that would have to be made for extreme cases of high or low density samples should be no more than 10% (Anwar et al., 1989).

5.3.1.3 Soil Angularity

The potential influence of different particle shapes or angularity was examined as part of the first phase of this research program reported by Anwar et al. (1989). Monterey 16 sand was separated into its angular and sub-rounded components, and was then checked to assure that the gradations of the two components were similar. It was found that the influence of very different particle shapes, or angularity, had no significant effect on the measured compliance values for samples of essentially the same gradation and prepared at the same relative density, as shown in Figure 5.2.

5.3.1.4 Soil Fabric

The "fabric" of a soil sample is initially a function of sample preparation method. Later on in an undrained test, soil fabric may be significantly altered by rearrangement of particle grains. It is therefore important that any significant influence of differnt soil fabrics be identified. Samples of Monterey sand were prepared at the same relative density by various methods including moist tamping, dry tamping, dry pluviation and vibration. Fortunately, no significant effect on measured compliance values was found due to the different sample preparation methods. Results of the unit membrane compliance evaluations made on the samples prepared by these different methods are given in Figure 5.3. Furthermore, it was found that particle rearrangement during cyclic loading appeared to have no



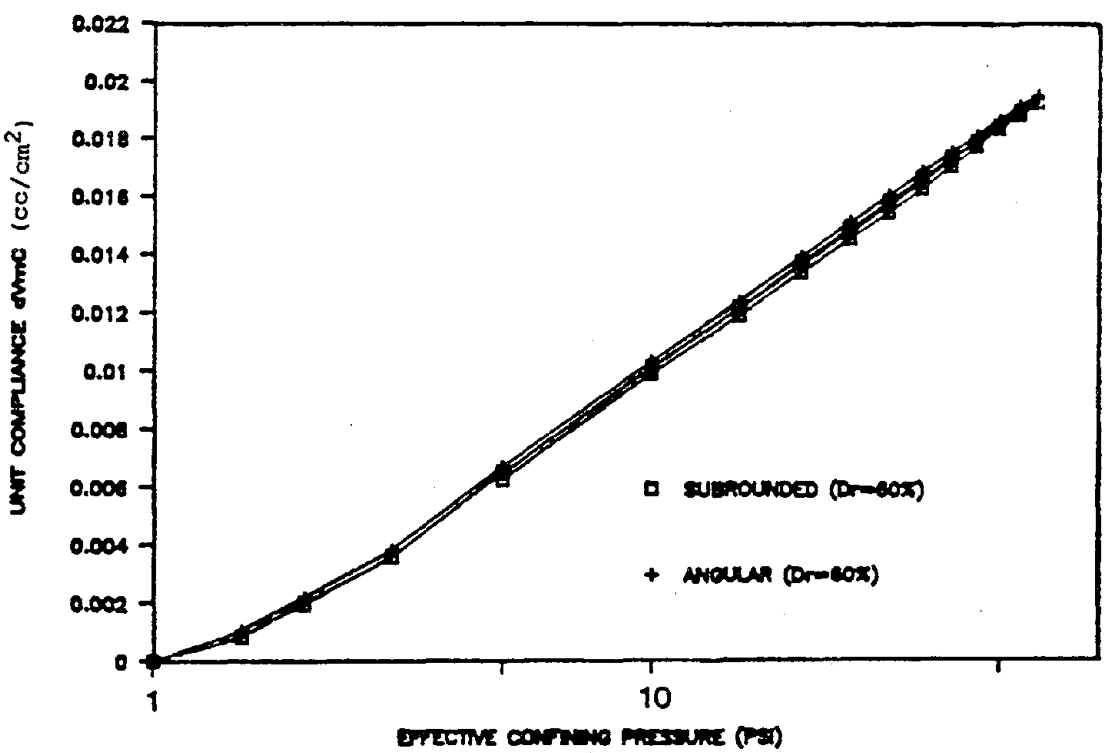


Figure 5.2: The Influence of Particle Angularity on Membrane Compliance

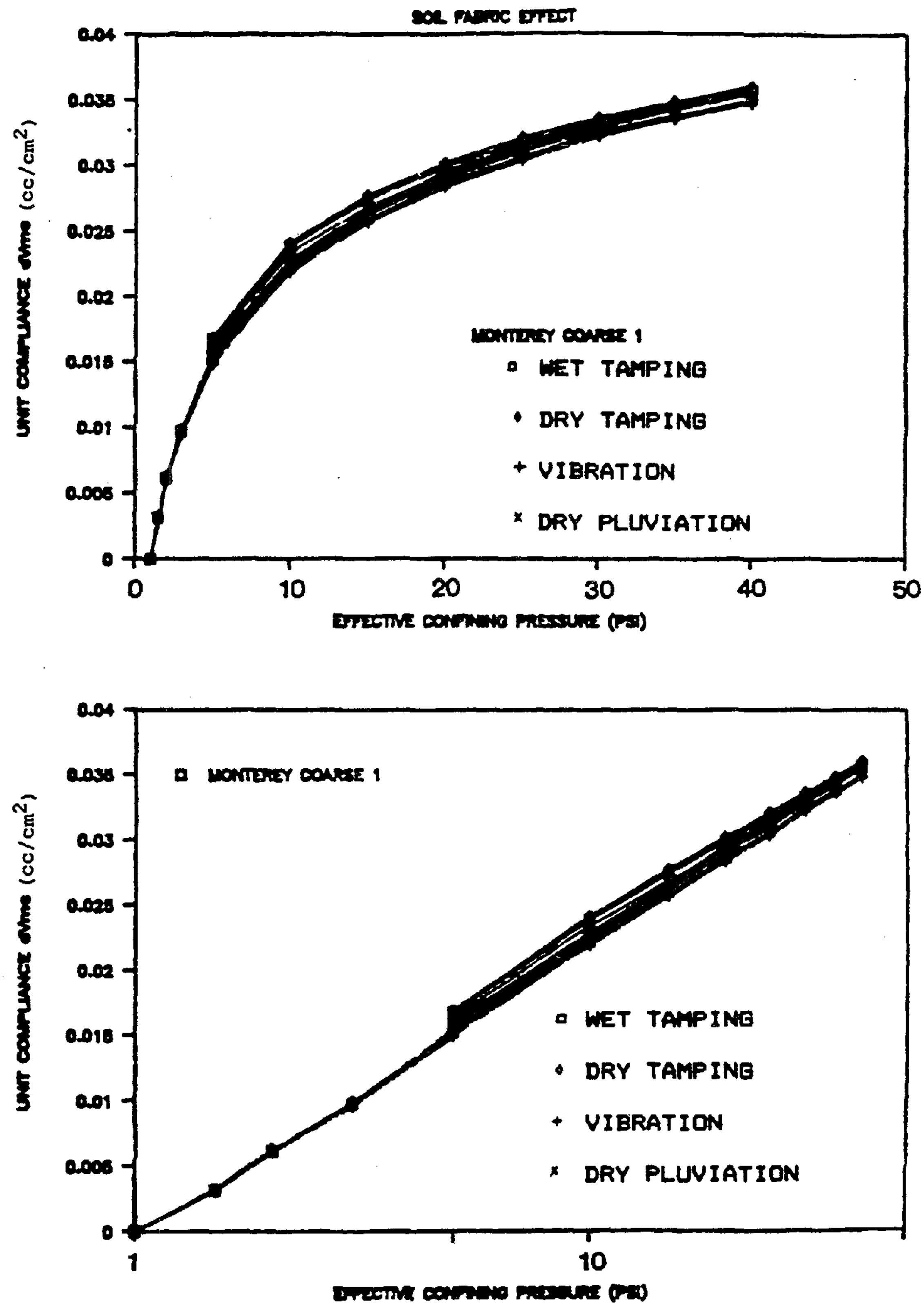


Figure 5.3: <u>The Influence of Initial Soil Fabric, or Method of Sample Preparation, on Membrane Compliance (Monterey Coarse 1 Sand at D_R = 60%)</u>

effect on membrane compliance. This finding was illustrated by the following procedure. A number of samples of Ottawa sand prepared at relative densities of approximately D_R = 50% were initially tested for unit membrane compliance. Identically prepared samples were then cyclically loaded to conditions of full liquefaction (r_u = 100%). These samples were then reconsolidated and tested for compliance evaluations. As can be seen in Figure 5.4, the measured compliance values were found to be nearly identical for the samples evaluated before and after cyclic loading to liquefaction.

5.3.1.5 Membrane Thickness and Multiple Membranes

The significance or influence of using multiple membranes and different membrane thicknesses on membrane compliance measurements has been investigated by several researchers. The results of those investigations have led to conclusions for particular sample sizes and membranes, but no clear picture has yet been developed and conclusively verified regarding how to join or separate these different data sets. An attempt to bridge the gap between the difference in conclusions made about the significance of using multiple membranes and various membrane thicknesses and materials for different sized samples was made as a part of this study. A disscussion of the results of that investigation is presented here. In addition, recent work on this subject by Kramer et al. (1989) appears very promising.

Experimental studies performed on small-scale (less than 6-inch diameter) samples have shown that using different thicknesses and numbers of latex membranes had little or no influence on unit membrane compliance (Ramana and Raju, 1982; Martin et al., 1978). Studies by Seed and Anwar (1986) verified this conclusion, and Kramer et al. (1989) proposed empirically-derived relationships

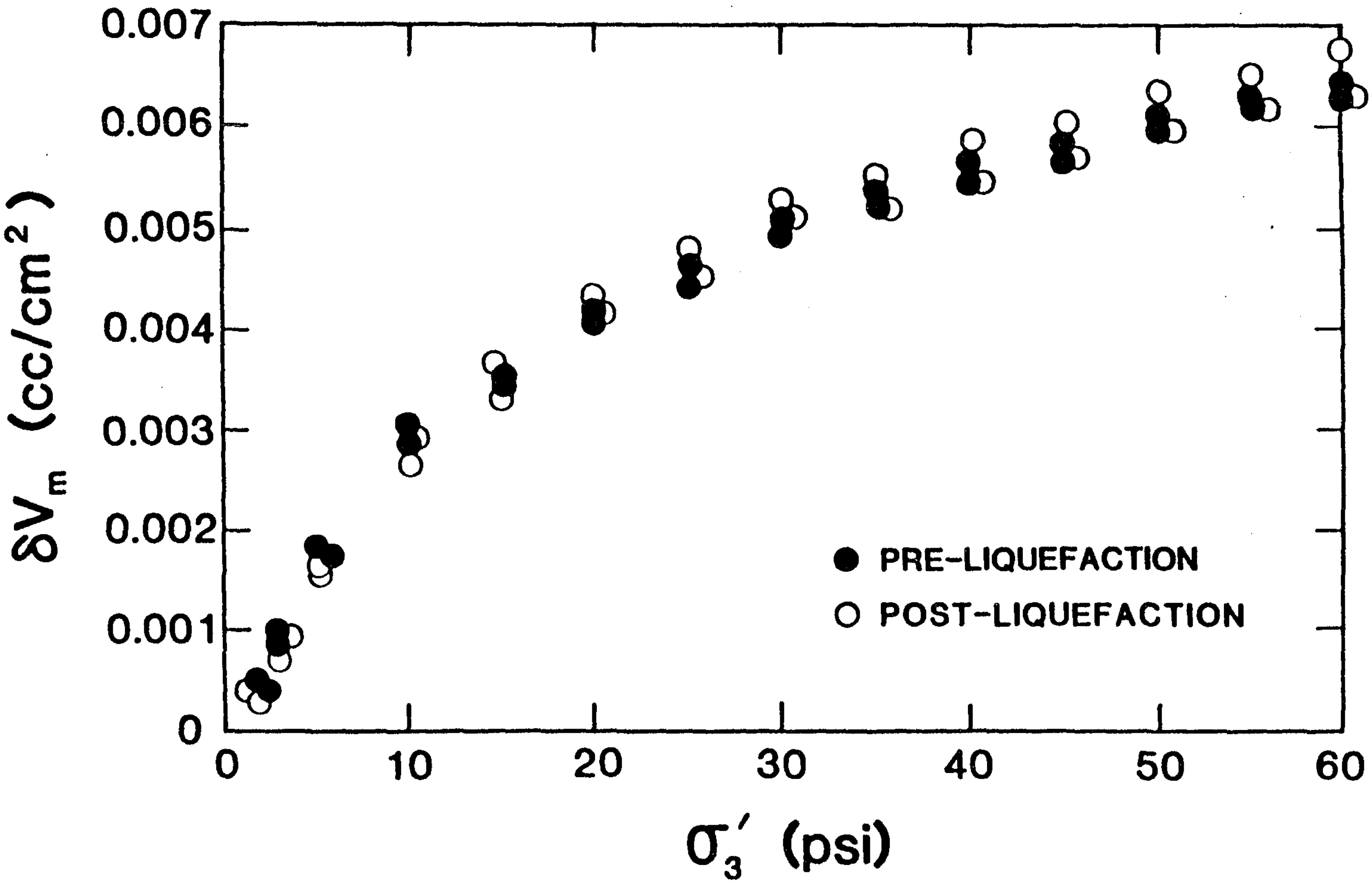


Figure 5.4: Pre- and Post-Liquefaction Membrane Compliance Curves for Fine Ottawa Sand at D_R = 50%

for estimation of the relatively slight influence of membrane thickness and stiffness.

Molenkamp and Luger (1981) and Baldi and Nova (1984) theoretically demonstrated the unimportance of membrane thickness and stiffness. Baldi and Nova proposed that even a five-fold increase in membrane thickness would have no significant effect on membrane compliance.

An overlap of sample sizes and membrane thicknesses were tested in this study for a number of materials whose representative grain size (D_{20}) was between coarse sand and fine gravel. The results suggest that at some limiting minimum grain size (on the order of twice the thickness of the confining membrane) membrane thickness begins to become an important factor in compliance volume measurements.

It is interesting to note that for those soils whose representative particle sizes do not approach the thickness of the larger membrane, unit compliance measurements made using both small and large scale systems compared extremely well, suggesting the insignificance of such factors as sample size and different membrane types. Figure 5.5 shows a comparison of measured unit membrane compliance for a fine gravelly material as measured with the small-scale apparatus with a 2.8-inch diameter sample confined by a single 0.014-inch thick latex membrane, and the large-scale apparatus with a 12-inch diameter sample confined by a 0.12-inch thick rubber membrane. For this material, the typical "representative" (D_{20}) grain size diameter that controls compliane is significantly greater than twice the thickness of the large-scale testing membrane, and membrane thickness therefore does not greatly influence compliance values. For conventional small scale (2.8-inch diameter) samples, the use of different numbers of multiple membranes and varied membranes thicknesses did not appear to

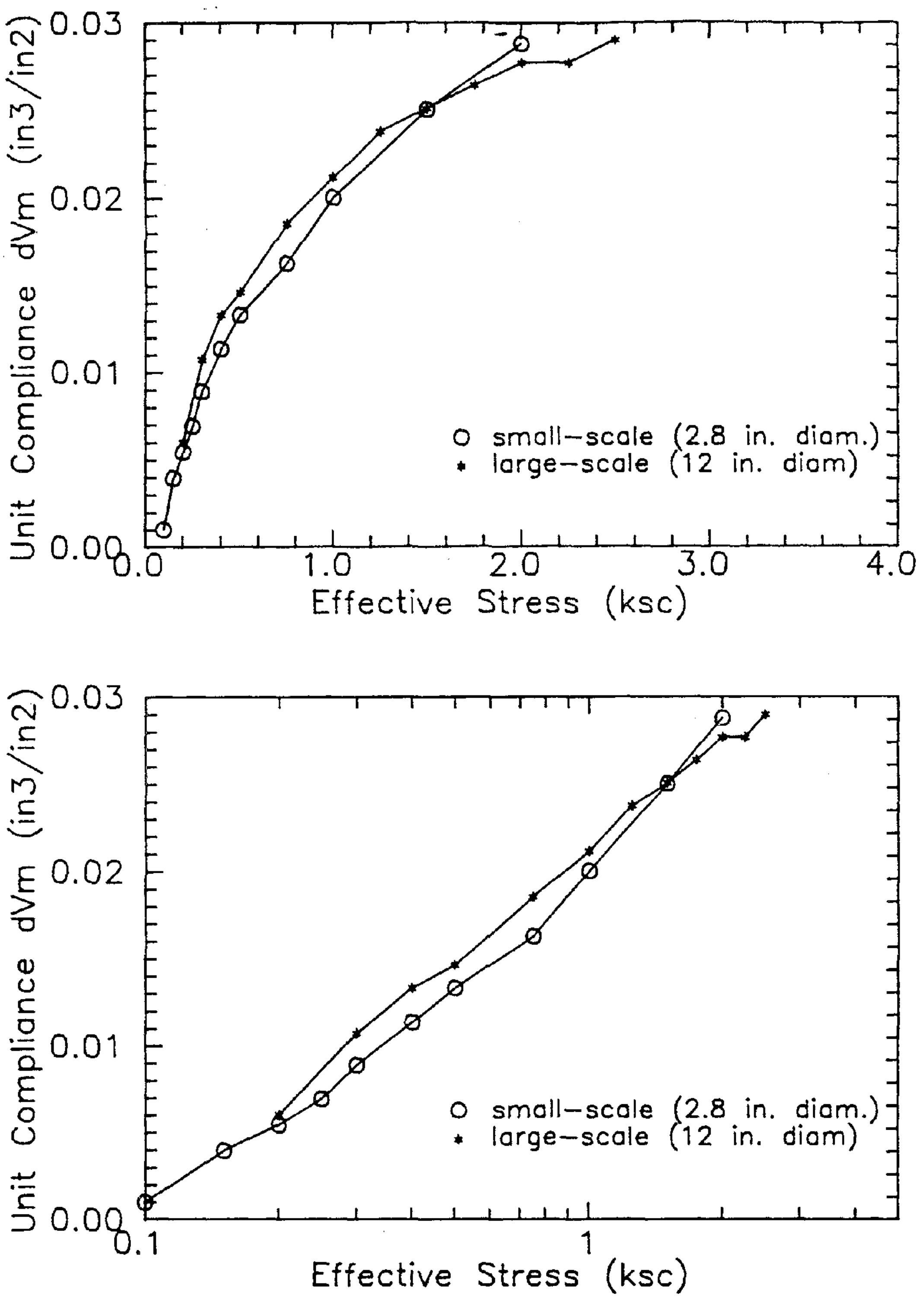


Figure 5.5: The Influence of Different Sized Testing Equipment on Membrane Compliance Measurements (Fine Gravel, Material #9)

significantly affect volumetric compliance measurements (Seed and Anwar, 1986). Figure 5.6 shows the results of tests on 2.8-inch diameter samples using different thicknesses (and numbers) of latex membranes. For the test performed using 0.028 inch thickness of membrane (two membranes, 0.014 inch thick each), the nearly quadrupling of membrane thickness over the 0.008 inch thick membrane decreased the measured compliance values by only approximately 5% for fine sands and by less than 5% for coarse sands. As grain sizes approached the range of membrane thicknesses for these small-scale samples, compliance volumes became so small that the accuracy of the measurements fell in the same range as that of the actual volume changes. Because of this it was concluded that the effects of different membrane configurations on these samples was insignificant with regard to compliance measurements.

Evans and Seed (1987) reported that differences in cyclic loading resistance between 12-inch diameter specimens confined by two thin latex membranes and those confined by a considerably thicker and stiffer rubber membrane were small, indicating that membrane compliance effects were not significantly influenced by such variations in the membrane configurations used (e.g. membrane material properties and thicknesses). An additional investigation made by Evans and Seed on a large-scale (12-inch diameter) sample, was made to evaluate the effect of using multiple membranes on compliance volume measurements. They reported that measured compliance volumes were nearly twice those found when using one membrane instead of two, but that using greater numbers of membranes had little additional effect. It was hypothesized that adhesion between the membranes was responsible for the decrease in measured compliance volumes, and not the added thicknesses. This suggestion was partially supported by results from this study where samples of coarse materials were tested with single membranes of different

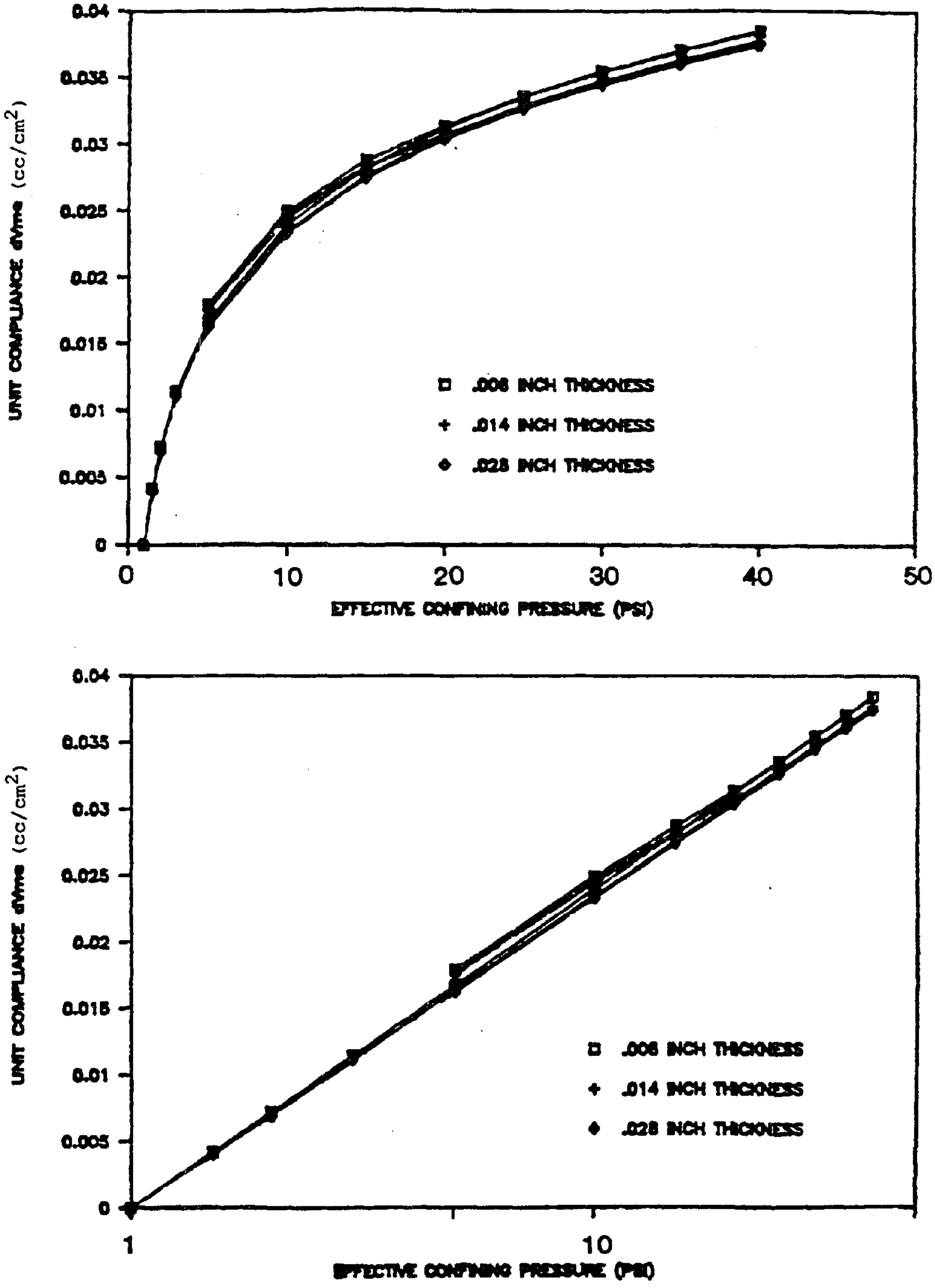


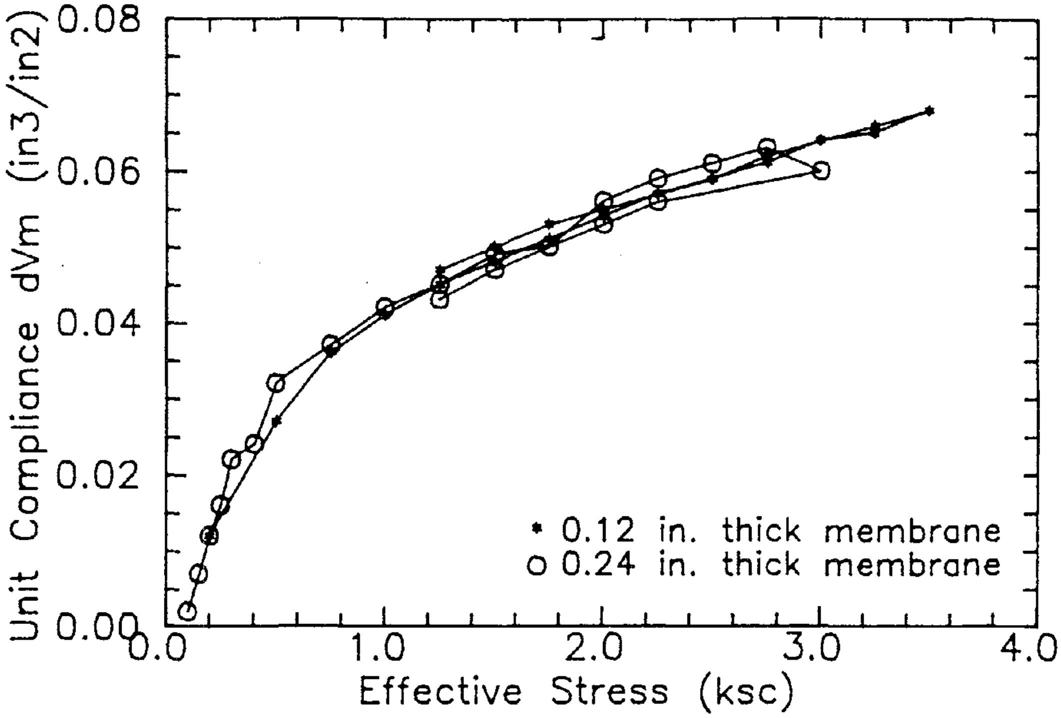
Figure 5.6: <u>The Influence of Membrane Thickness on Membrane Compliance (Monterey Coarse 2 Sand at $D_R \approx 60\%$)</u>

thicknesses and multiple membranes. The use of different thicknesses of individual membranes had very little effect on compliance volume measurements. Using different numbers of membranes appeared to have a noticeable effect, although the effect of using multiple membranes was found to be much less significant when tested in this study. Figure 5.7 shows the difference between compliance values measured as part of this study for similarly prepared samples of a medium gravel with membrane thicknesses of 0.12 and 0.24 inches. Figure 5.8 shows a comparison between compliance values measured for similarly prepared samples of a medium gravel with one and two membranes.

5.3.2 Compliance Measurement Results

A number of samples of sand with a variety of soil characteristics, including different gradation types and soil particle sizes, angularity, etc., were tested for volumetric compliance as part of the preliminary phase of this study previously reported by Anwar et al. (1989). For completeness, these results are also presented here. A listing of the sandy soils tested, with a brief summary description and membrane compliance behavior evaluated for each, is given in Table 5.1. Figures 5.9 through 5.21 show the measured membrane compliance characteristics of these soils, and Figures 5.22 through 5.25 show the gradations of these materials. All compliance measurements are plotted as unit membrane compliance (volume change per unit membrane area: cc/cm^2) vs. effective confining stress and $\log_{10}$ of effective confining stress. All of the soils listed in Table 5.1 were prepared to $D_R \approx 60\%$.

In addition to the sandy soils, nearly a dozen different gravelly soils were tested for volumetric compliance utilizing the large-scale testing equipment. Again, a wide range of soil gradations and particle types were tested, and the range of



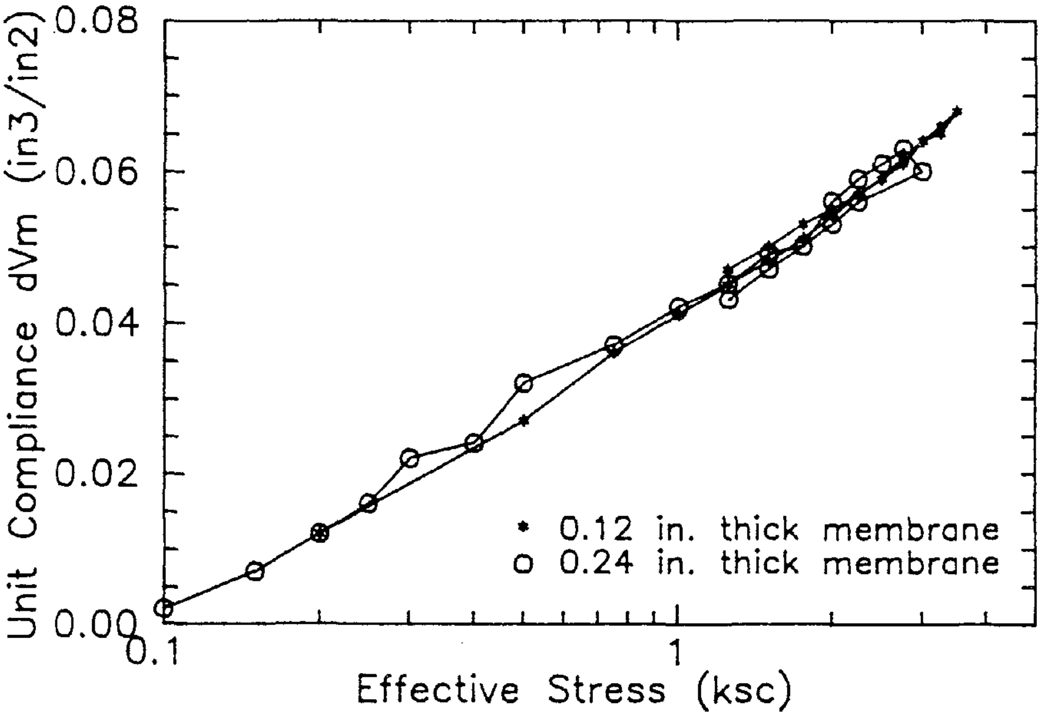


Figure 5.7: <u>The Influence of Different Large-Scale Membrane Thicknesses on Membrane Compliance Measurements (Medium Gravel, Material #1)</u>

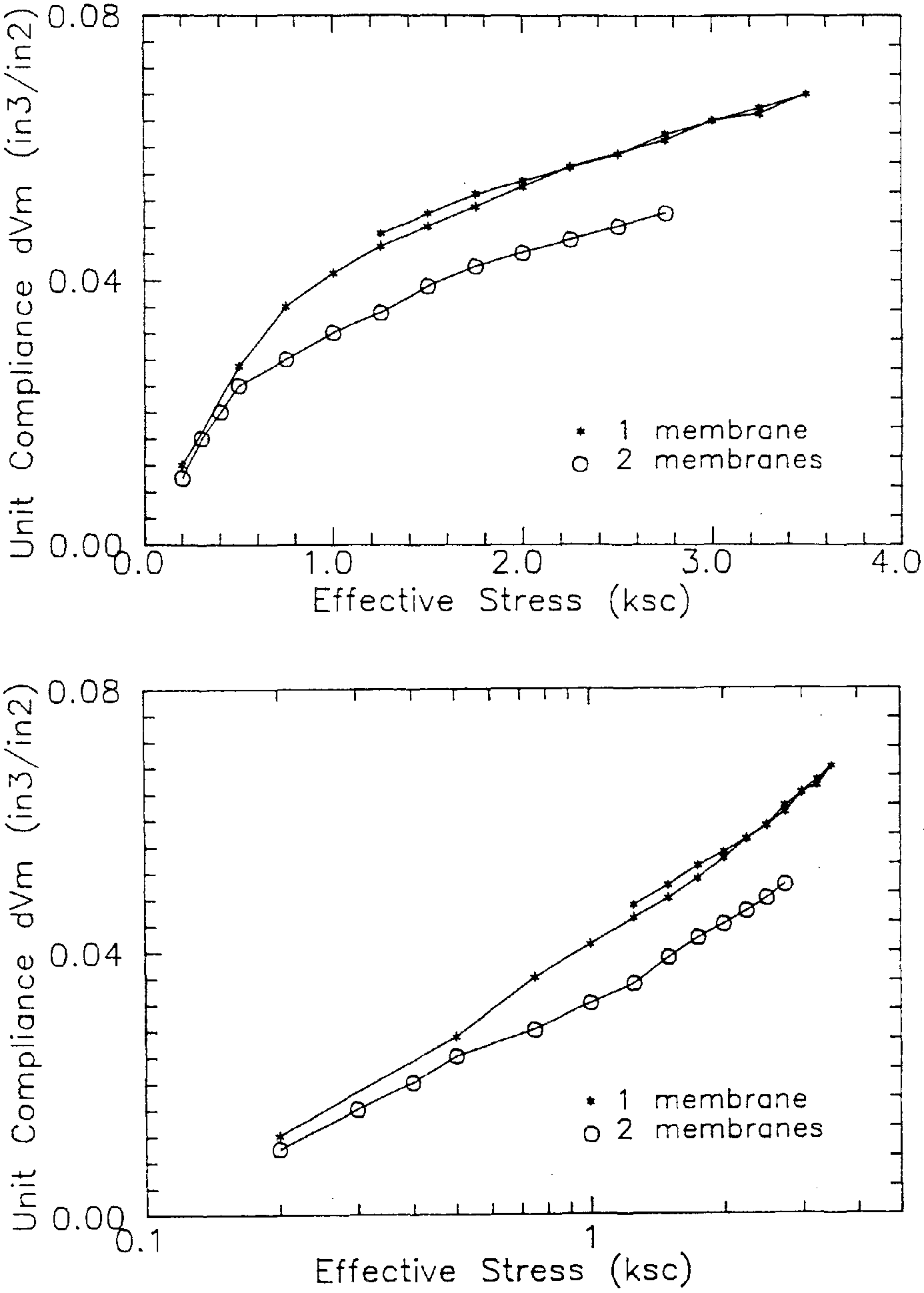


Figure 5.8: The Influence of Different Numbers of Large-Scale Membranes on Membrane Compliance Measurements (Medium Gravel, Material #1)

Table 5.1 Sandy Soils Tested for Membrane Compliance Magnitude

Soil Name	USCS Classification	D_{10} (mm)	D_{20} (mm)	D_{50} (mm)	$S^{\dagger}$ (cm/$\Delta\log\sigma_3'$)	Figure
Monterey Coarse 1	SP	1.900	2.200	2.600	0.02200	3.1
Monterey Coarse 2	SP	2.290	2.490	2.900	0.02350	3.7
Well Graded 1	SW-ML	0.032	0.059	0.200	0.00075	3.8
Well Graded 2	SW	0.130	0.200	0.600	0.00180	3.9
Well Graded 3	SW	0.230	0.320	0.700	0.00222	3.10
Well Graded 4	SW	0.490	0.700	1.400	0.01060	3.11
Mod. Sacramento	SP	0.205	0.230	0.305	0.00201	3.12
Ottawa Fine	SP	0.240	0.300	0.400	0.00480	3.13
Ottawa 20-30	SP	0.605	0.630	0.720	0.00927	3.14
Monterey '0'	SP	0.240	0.300	0.330	0.00500	3.15
Monterey 16	SP	0.720	0.980	1.250	0.01195	3.16
Gap Graded 1	SP	0.240	0.305	1.800	0.00085	3.17
Gap Graded 2	SP	0.630	0.680	0.820	0.00710	3.18

Gap Graded 1: MONTEREY CR. : MONTEREY 16 : MOD SACRAMENTO (4:1:2)
Gap Graded 2: MONTEREY COARSE : OTTAWA 20-30 (1:1)
Well Graded 1: MONTEREY SAND WITH 25% SILT
Well Graded 2: MONTEREY SAND, SOME GRAVEL, 5% SILT

† = Normalized Unit Compliance; change in volume per unit membrane area per log-cycle change in effective conf. stress (cc/cm^2/$\Delta\log\sigma_3'$).

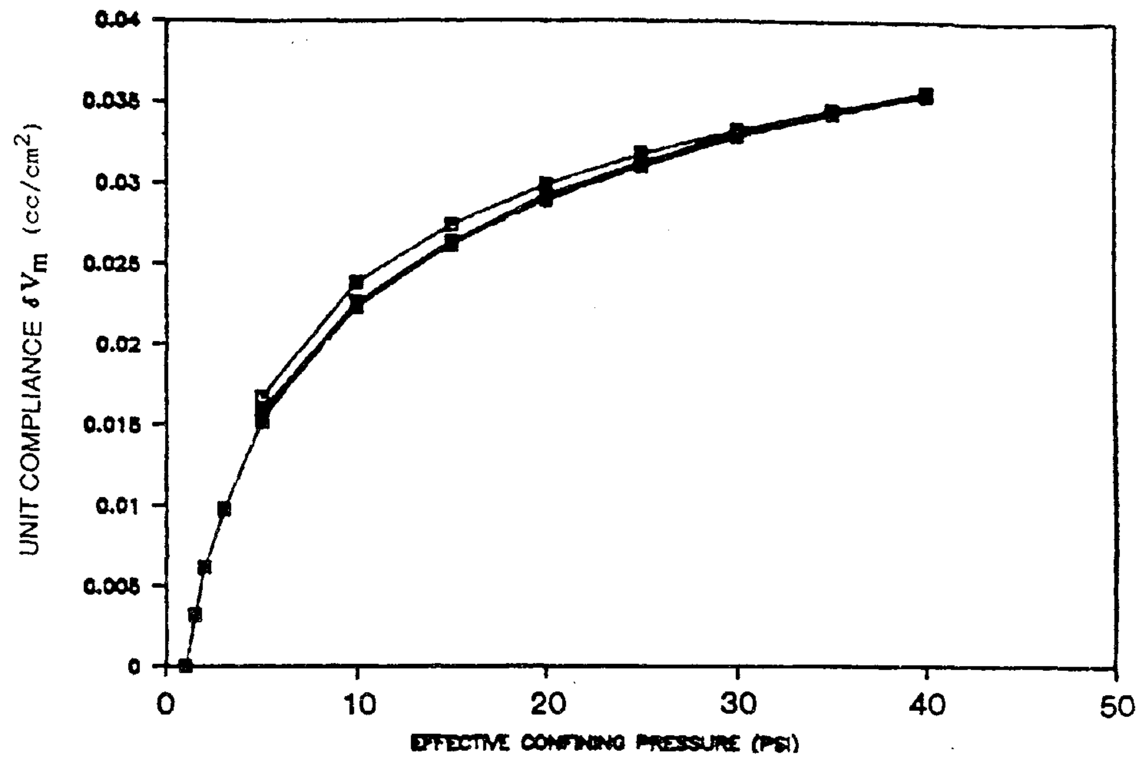


Figure 5.9a: Unit Membrane Compliance vs. Effective Confining Pressure: Modified Monterey Coarse 1 Sand at D_R = 60%

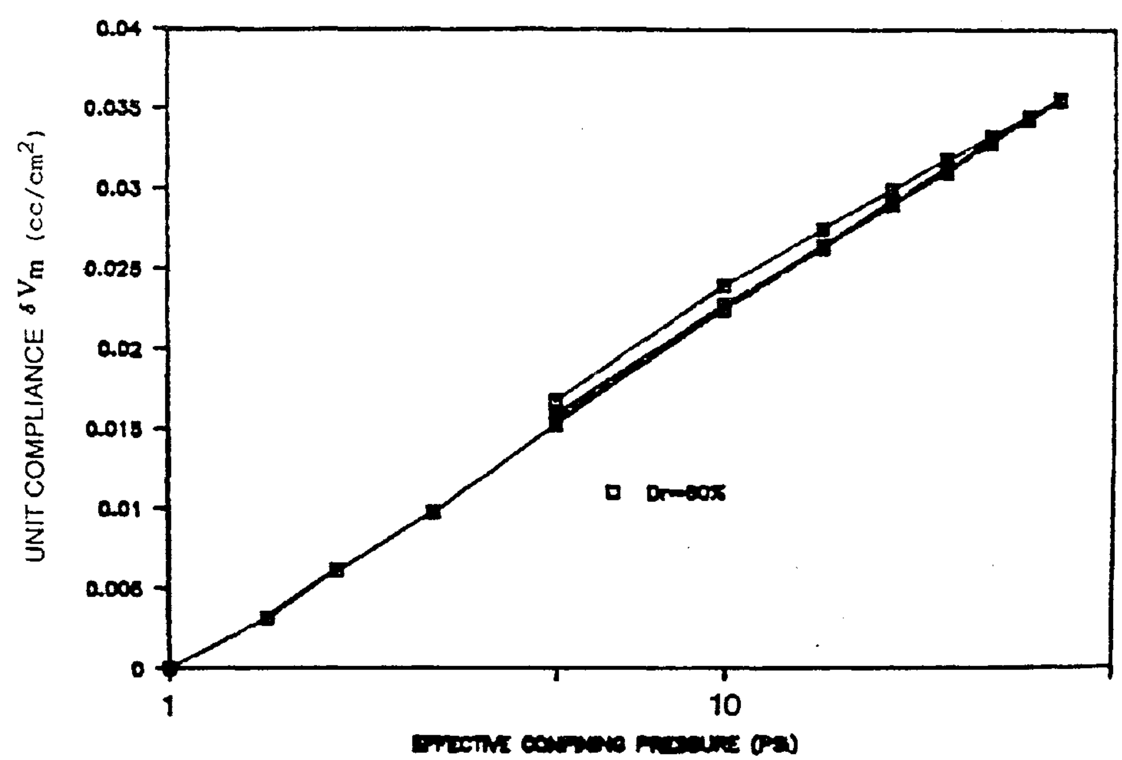


Figure 5.9b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Modified Monterey Coarse 1 Sand at D_R = 60%

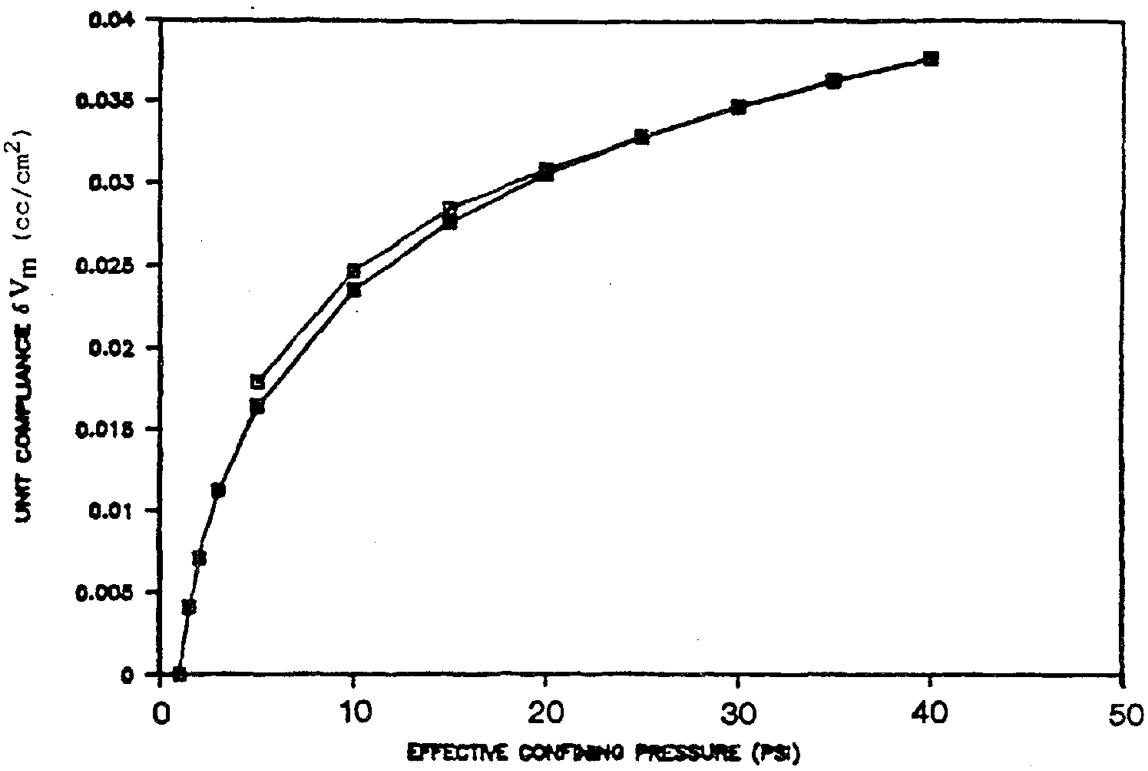


Figure 5.10a: Unit Membrane Compliance vs. Effective Confining Pressure: Modified Monterey Coarse 2

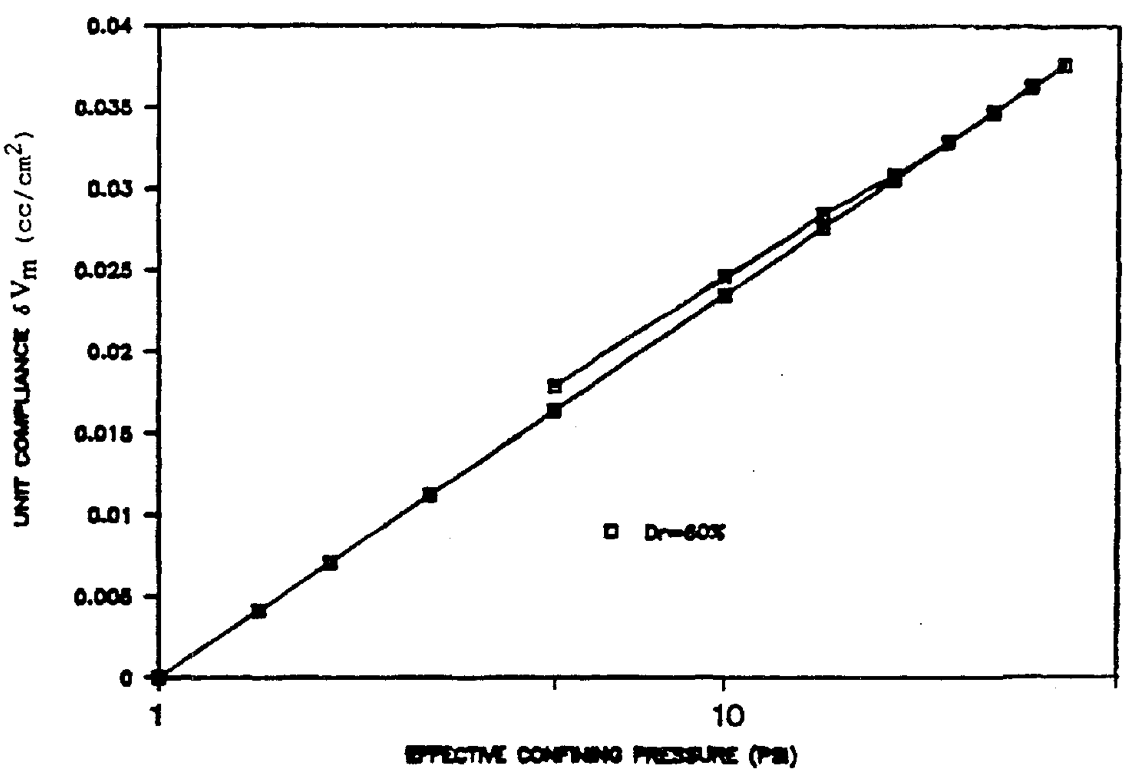


Figure 5.10b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Modified Monterey Coarse 2

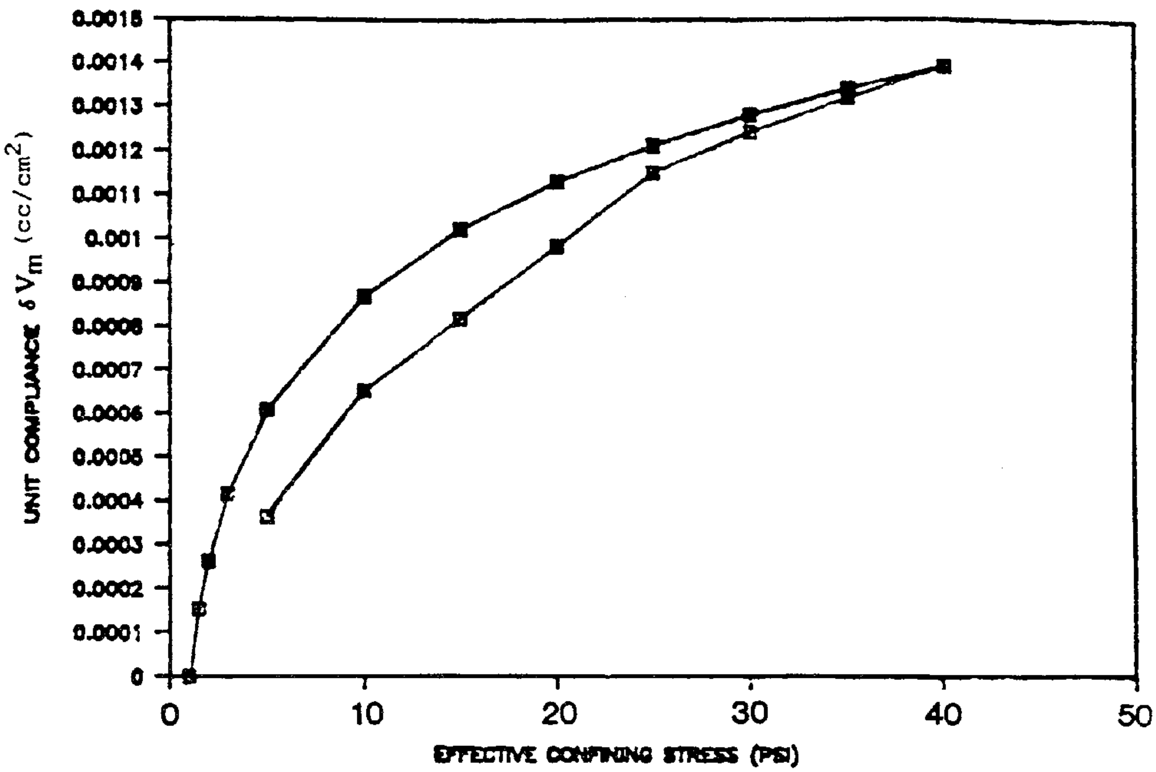


Figure 5.11a: Unit Membrane Compliance vs. Effective Confining Pressure: Well Graded 1

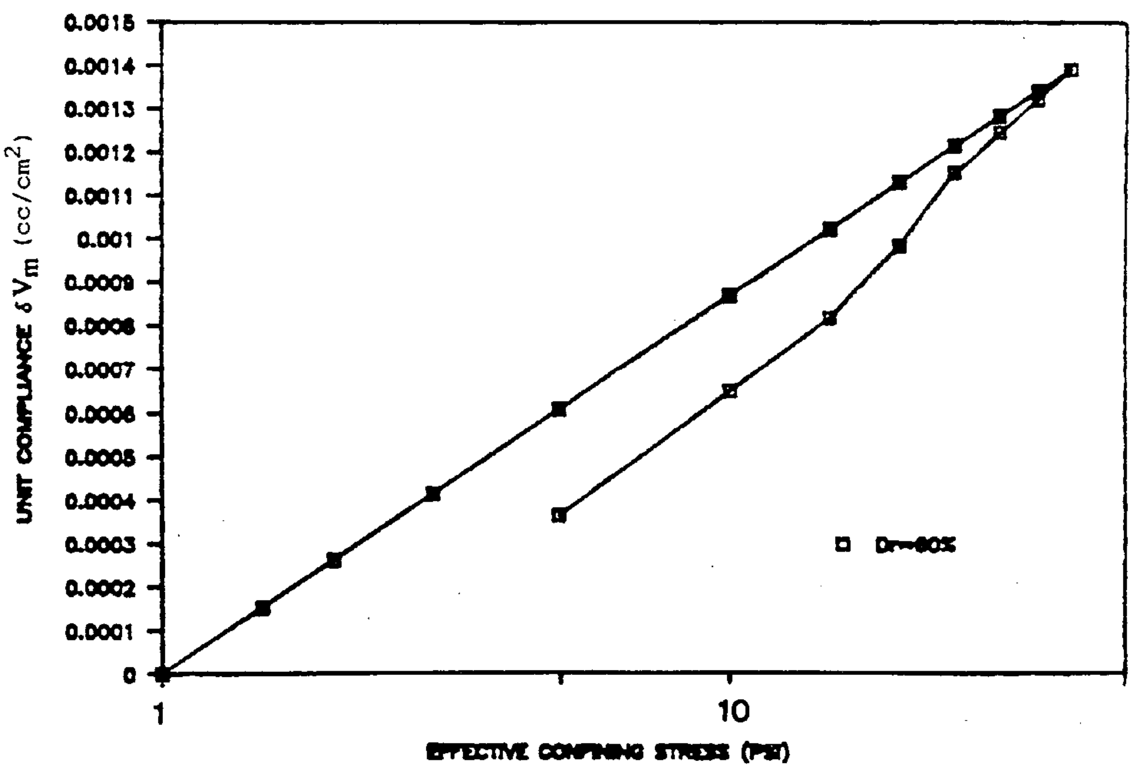


Figure 5.11b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Well Graded 1

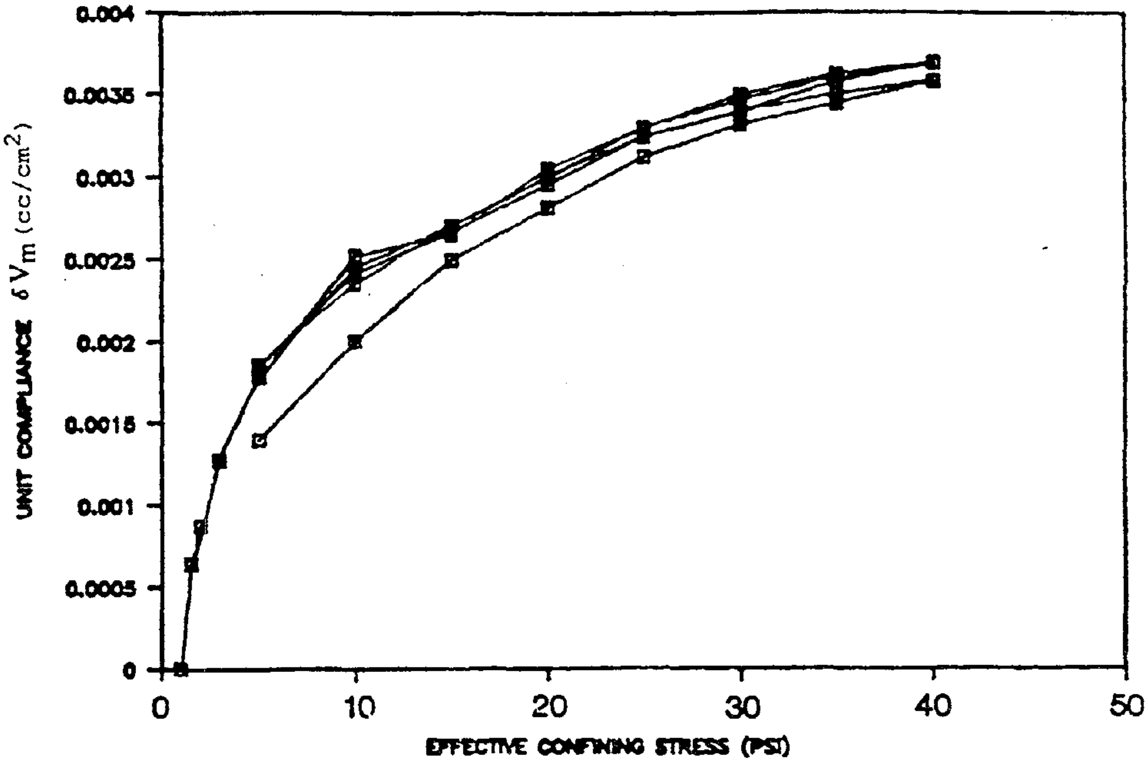


Figure 5.12a: Unit Membrane Compliance vs. Effective Confining Pressure: Well Graded 2

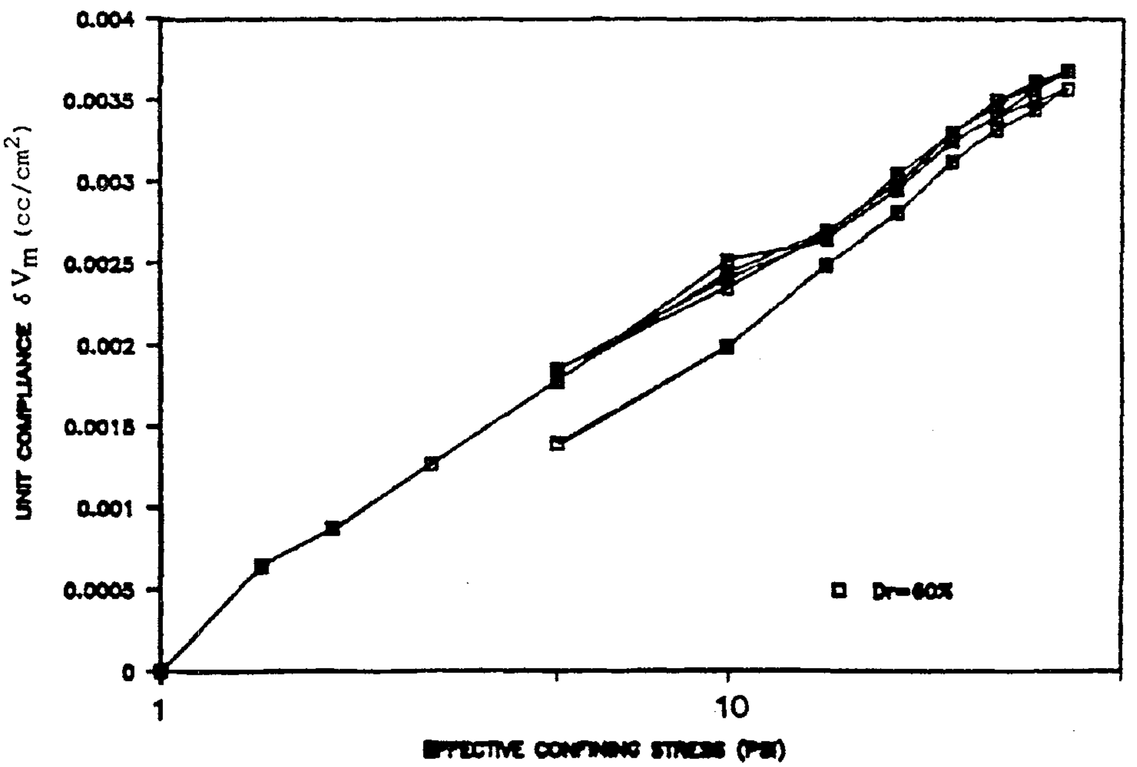


Figure 5.12b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Well Graded 2

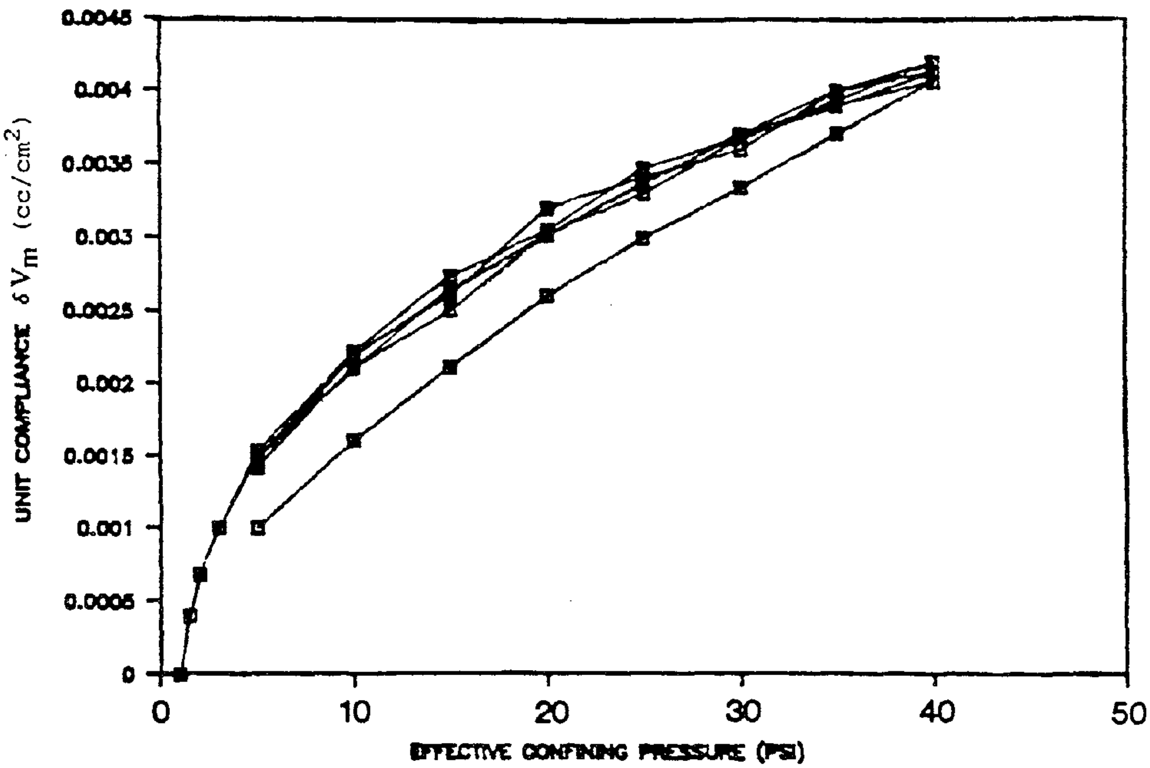


Figure 5.13a: Unit Membrane Compliance vs. Effective Confining Pressure: Well Graded 3

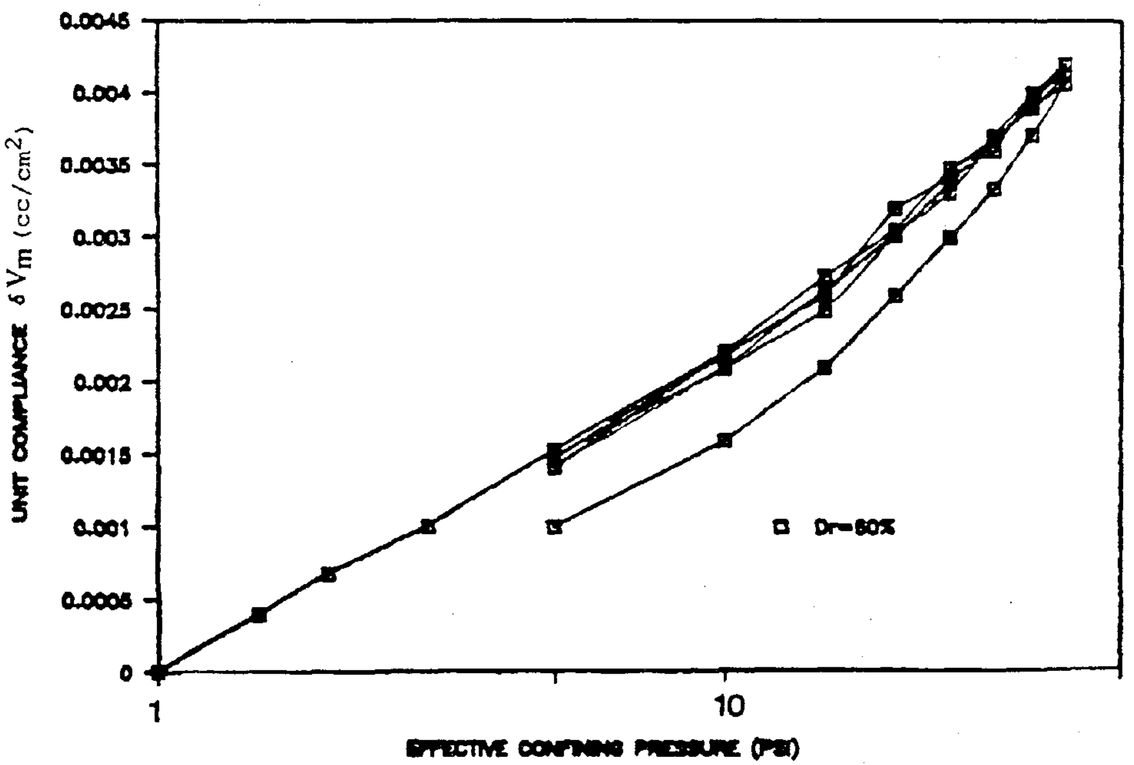


Figure 5.13b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Well Graded 3

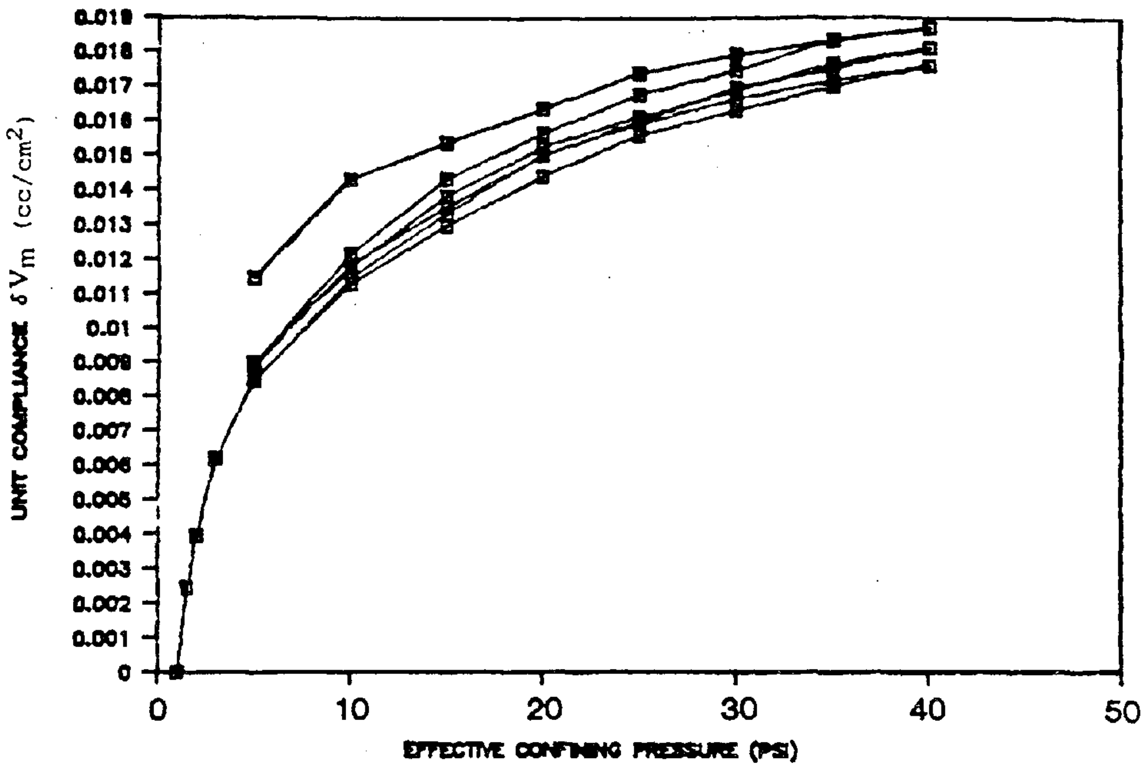


Figure 5.14a: Unit Membrane Compliance vs. Effective Confining Pressure: Well Graded 4

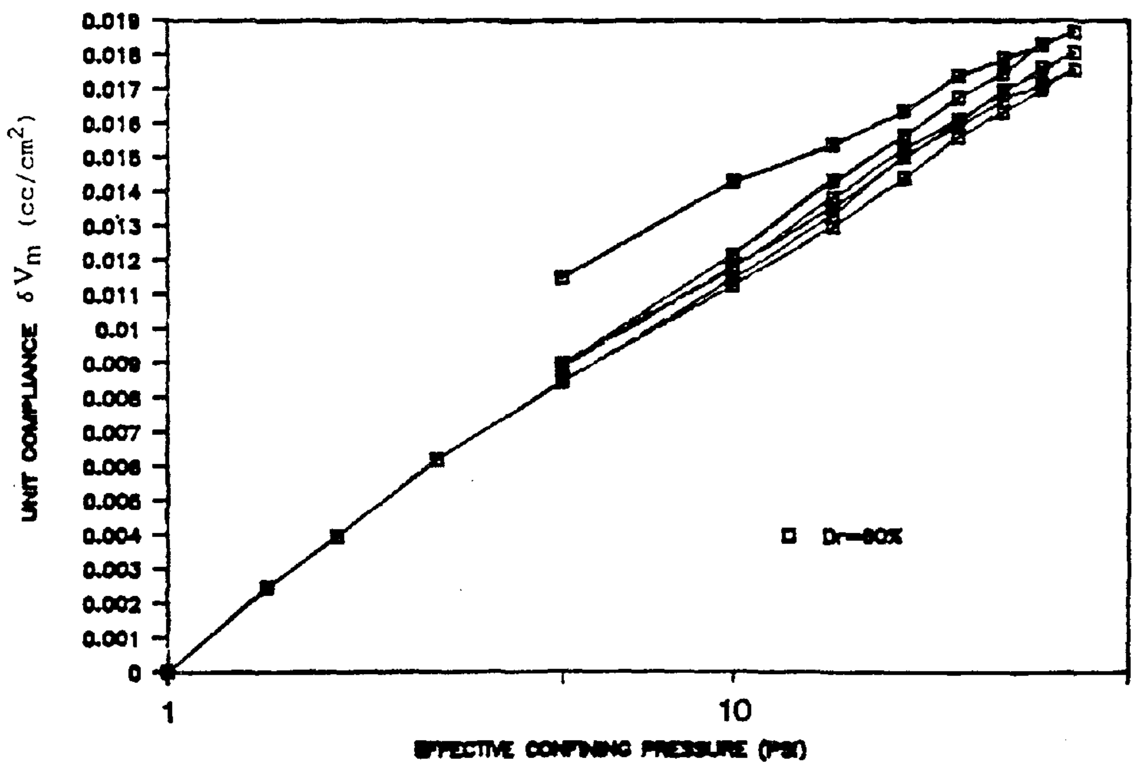


Figure 5.14b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Well Graded 4

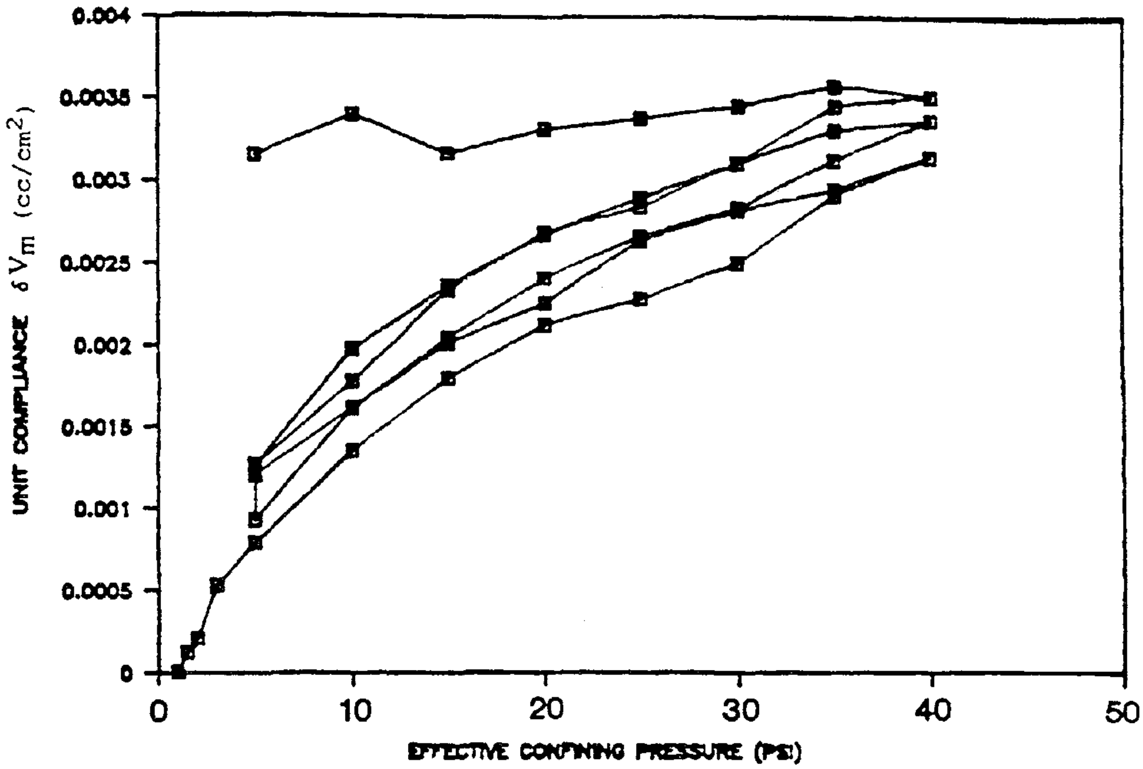


Figure 5.15a: Unit Membrane Compliance vs. Effective Confining Pressure: Modified Sacramento River Sand

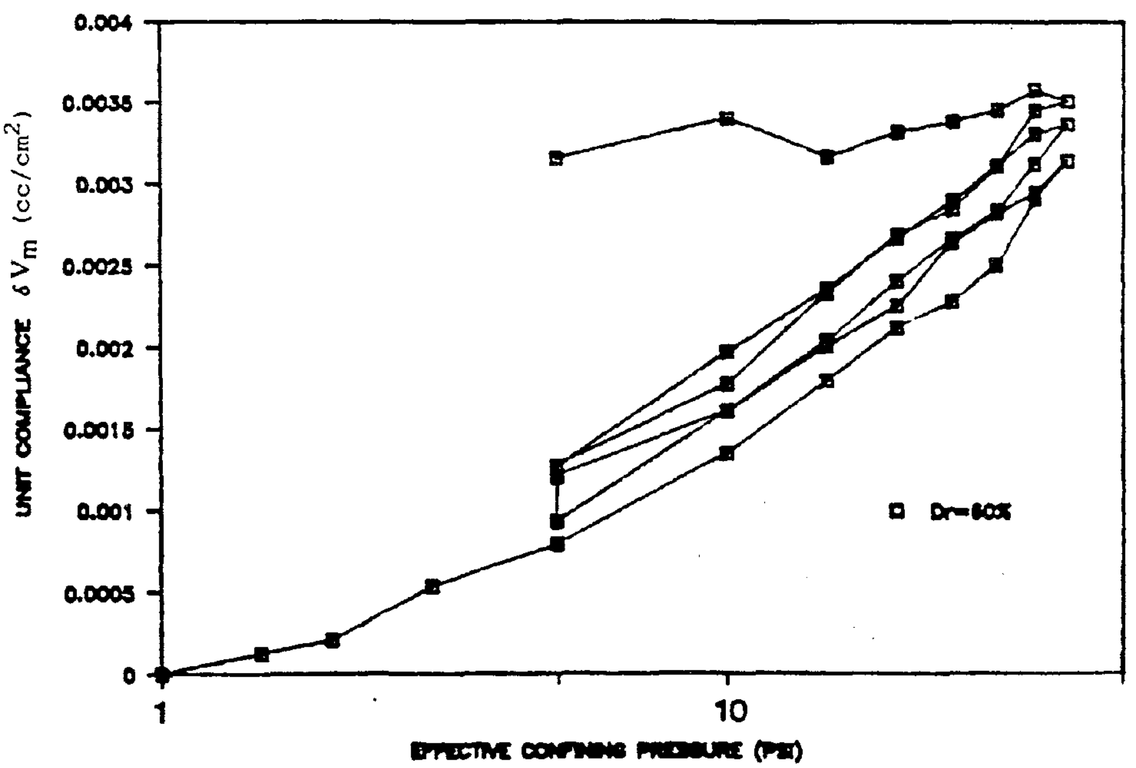


Figure 5.15b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Modified Sacramento River Sand

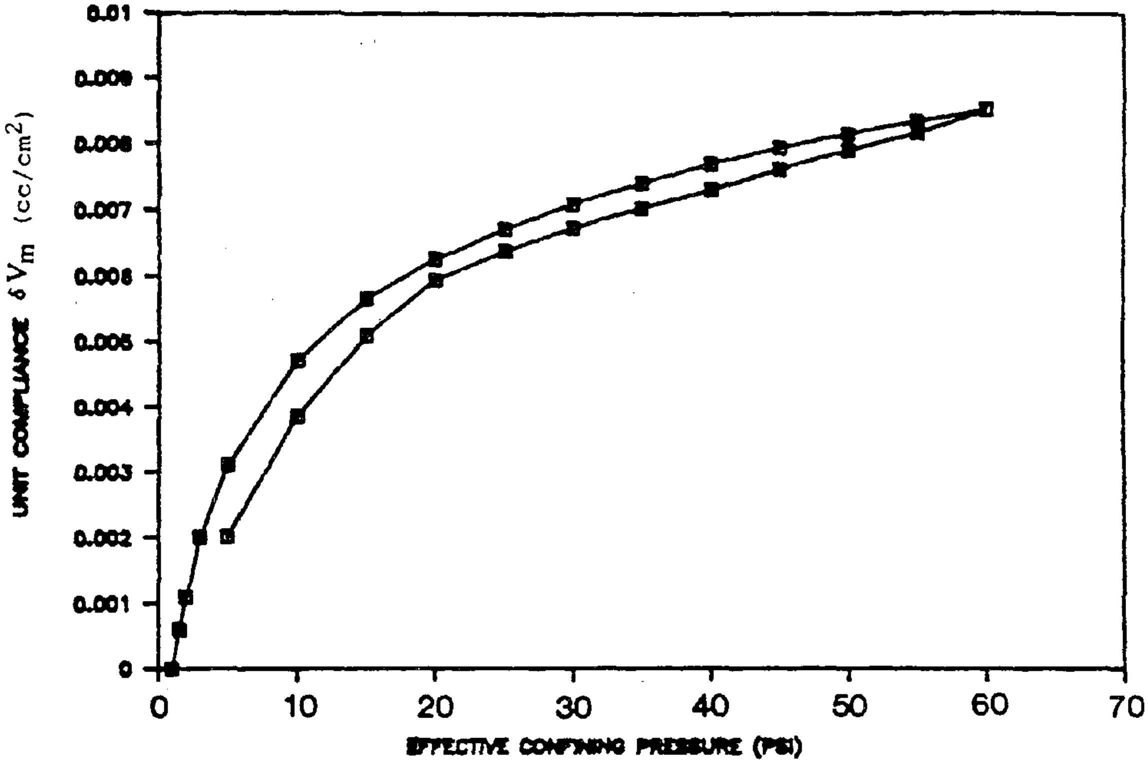


Figure 5.16a: Unit Membrane Compliance vs. Effective Confining Pressure: Ottawa Fine Sand

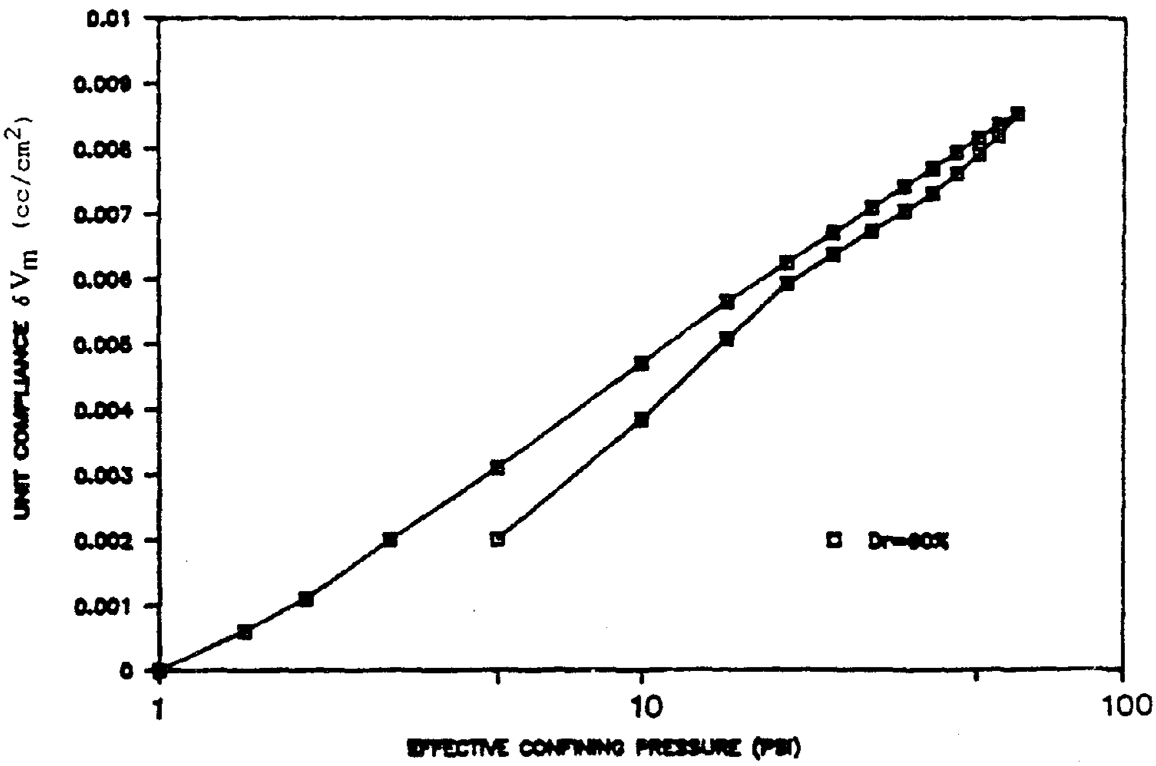


Figure 5.16b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Ottawa Fine Sand

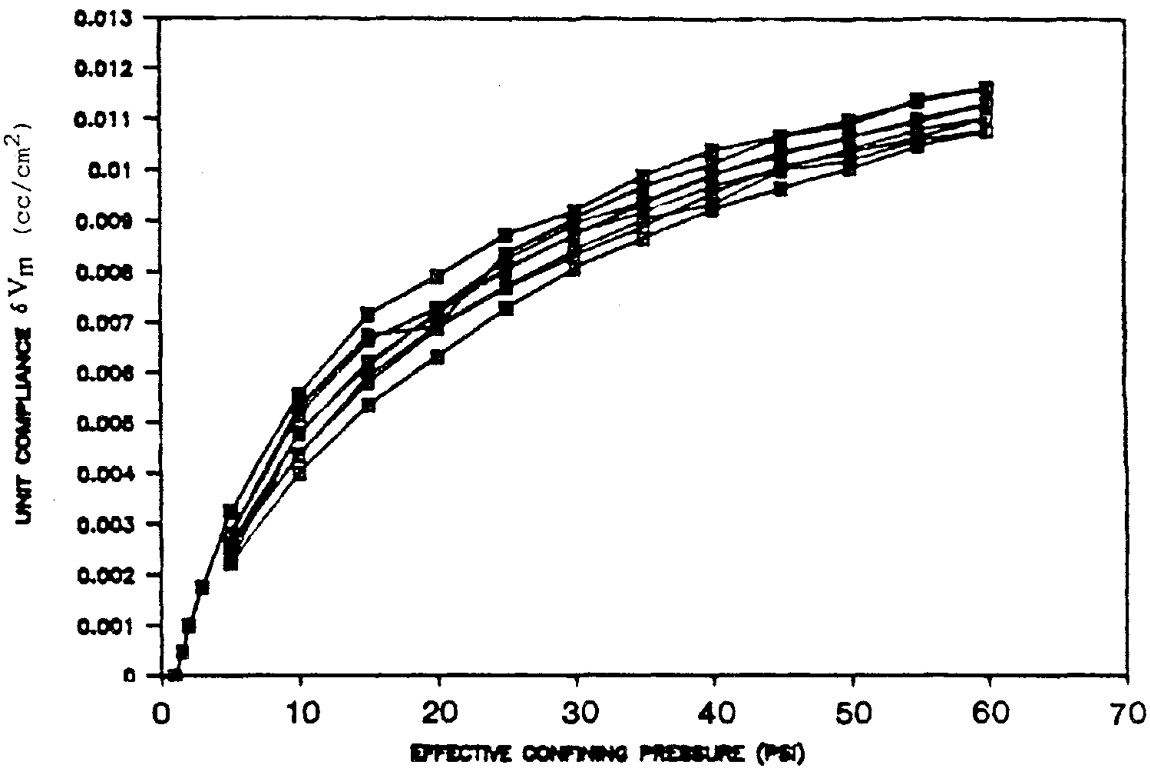


Figure 5.17a: Unit Membrane Compliance vs. Effective Confining Pressure: Ottawa 20-30 Sand

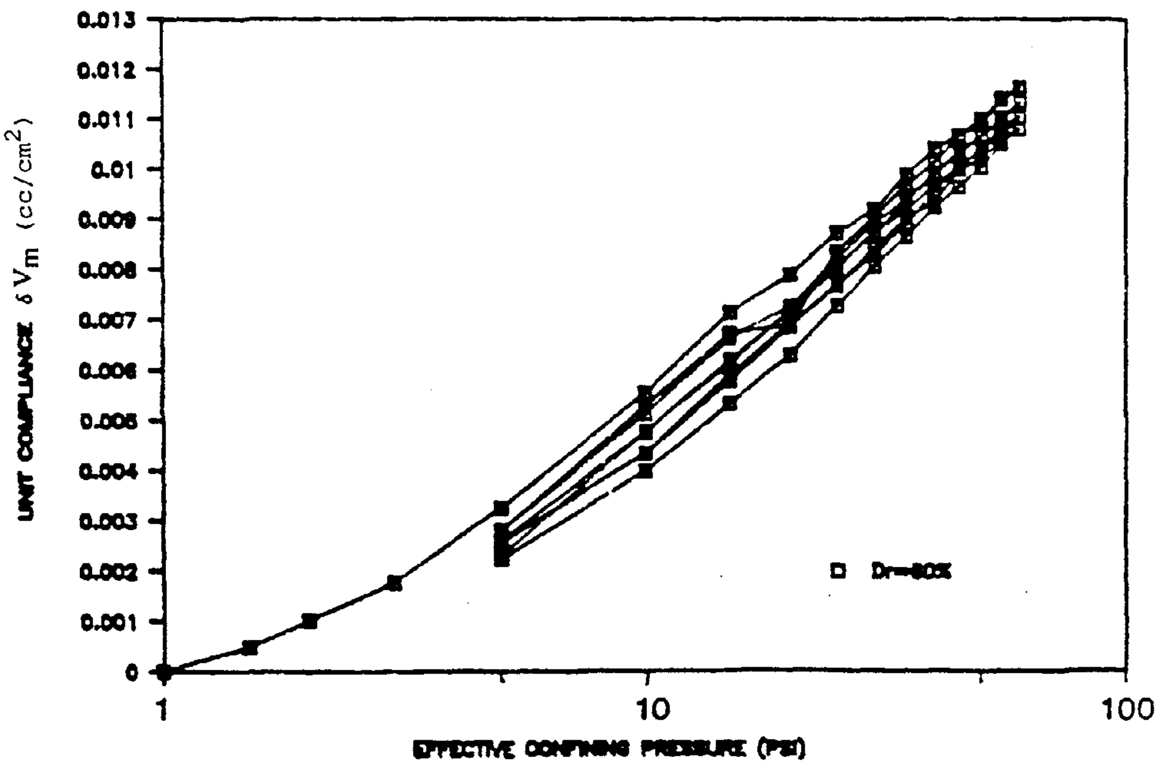


Figure 5.17b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Ottawa 20-30 Sand

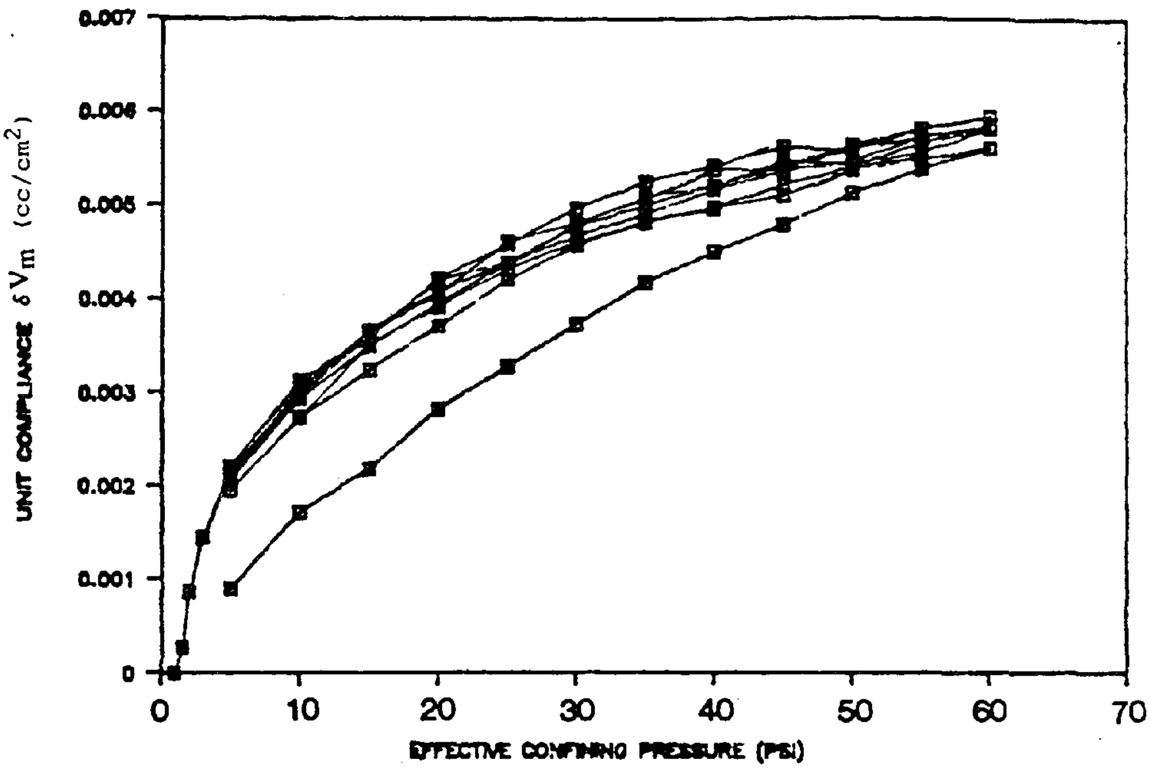


Figure 5.18a: Unit Membrane Compliance vs. Effective Confining Pressure: Monterey "0" Sand

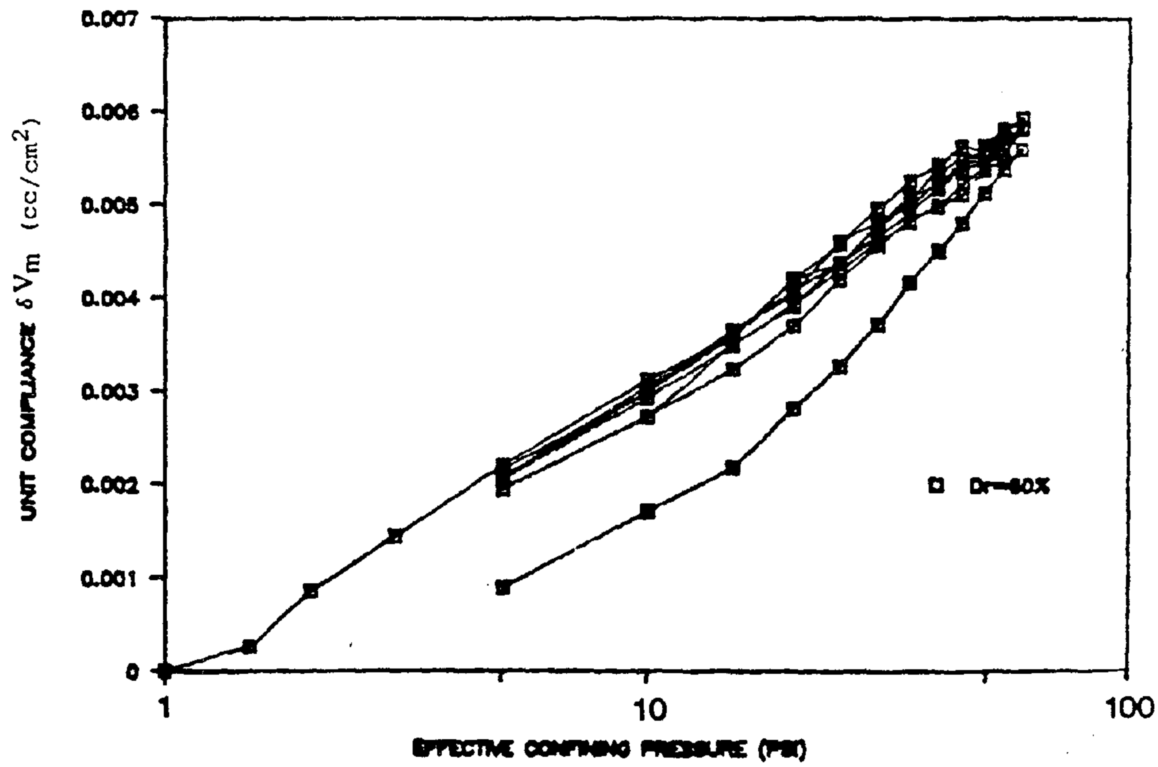


Figure 5.18b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Monterey "0" Sand

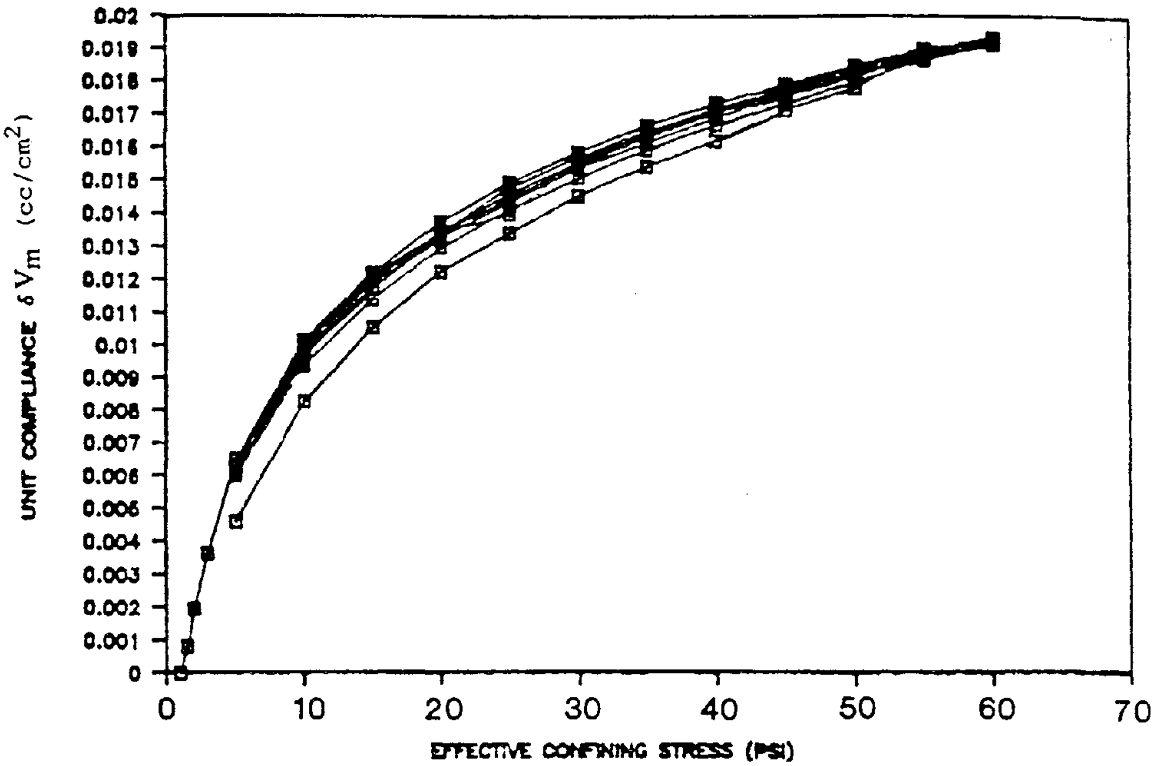


Figure 5.19a: Unit Membrane Compliance vs. Effective Confining Pressure: Monterey 16 Sand

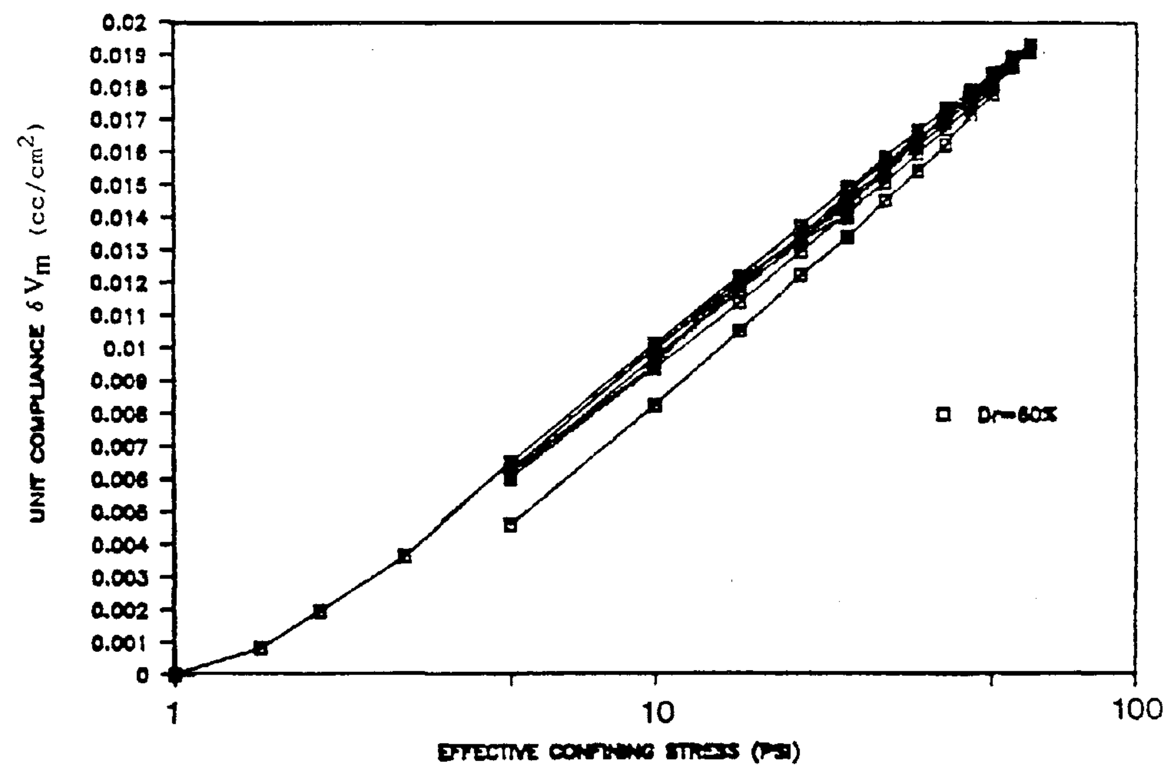


Figure 5.19b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Monterey 16 Sand

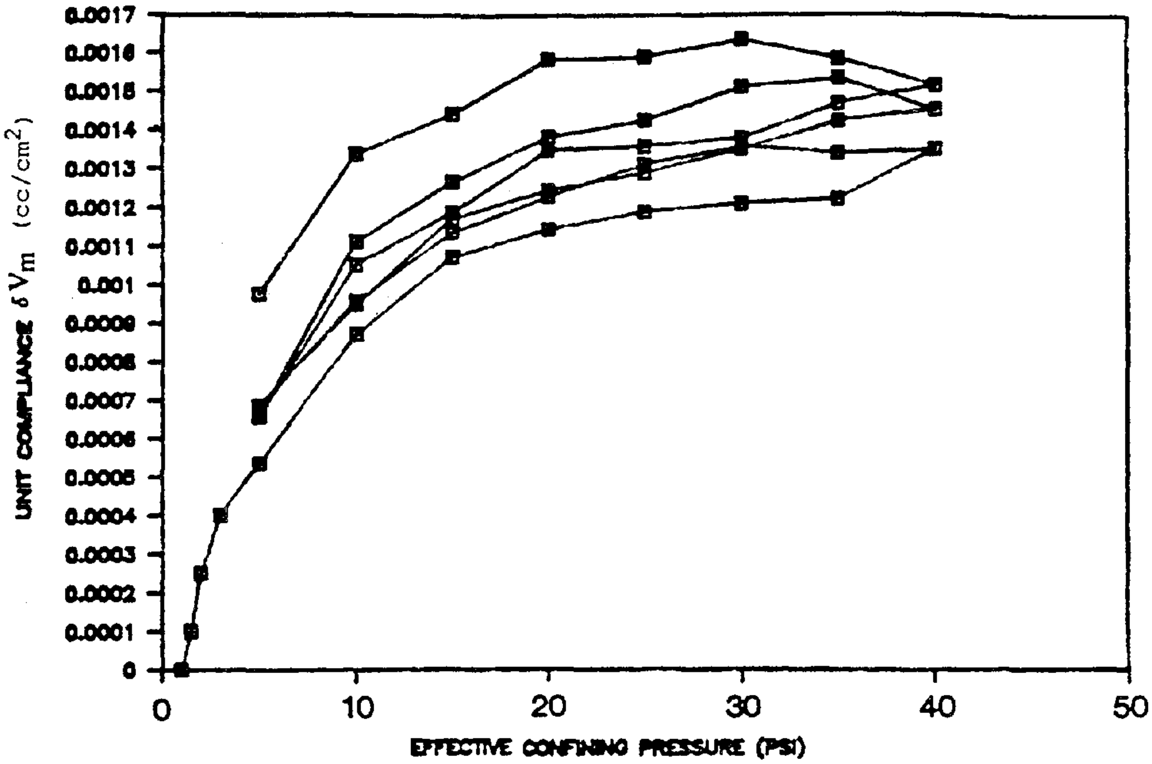


Figure 5.20a: Unit Membrane Compliance vs. Effective Confining Pressure: Gap Graded 1

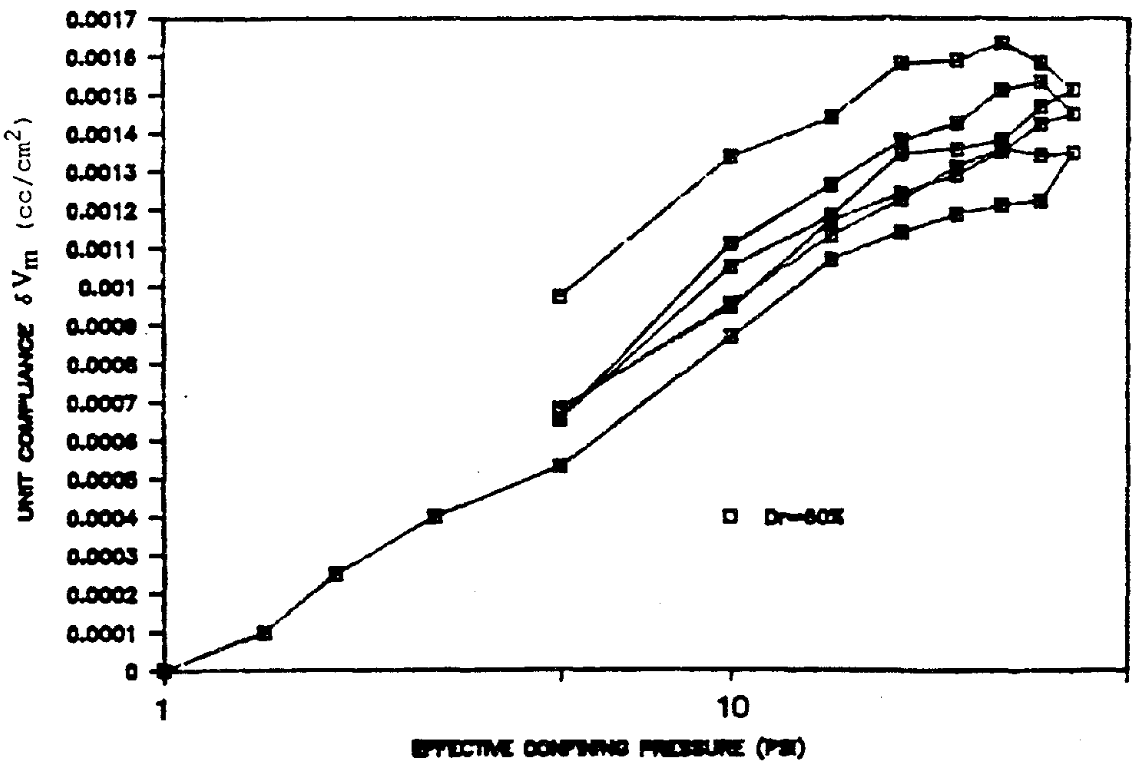


Figure 5.20b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Gap Graded 1

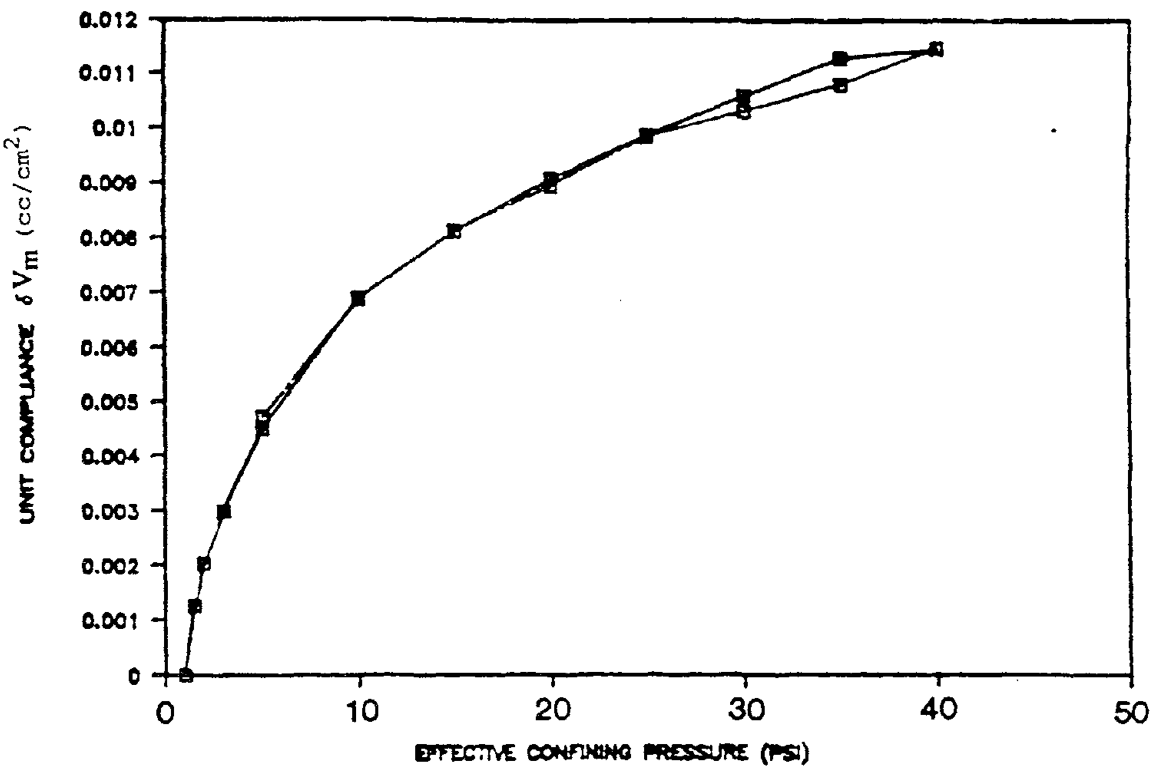


Figure 5.21a: Unit Membrane Compliance vs. Effective Confining Pressure: Gap Graded 2

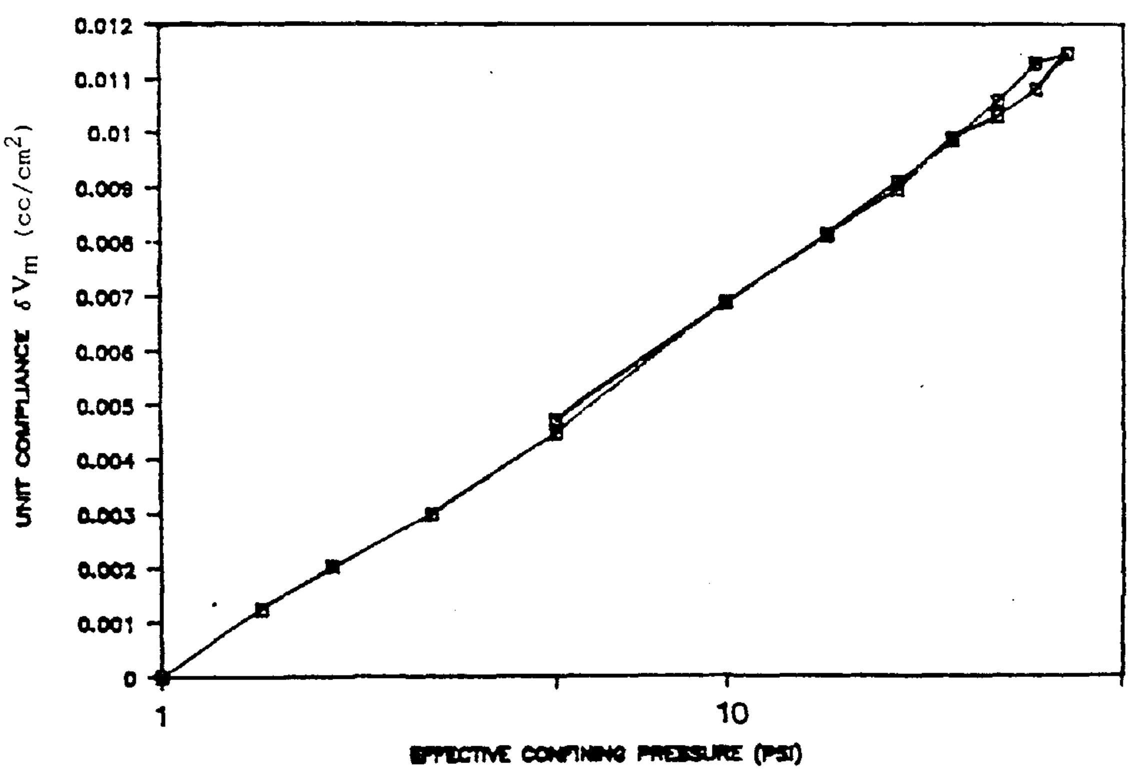


Figure 5.21b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Gap Graded 2

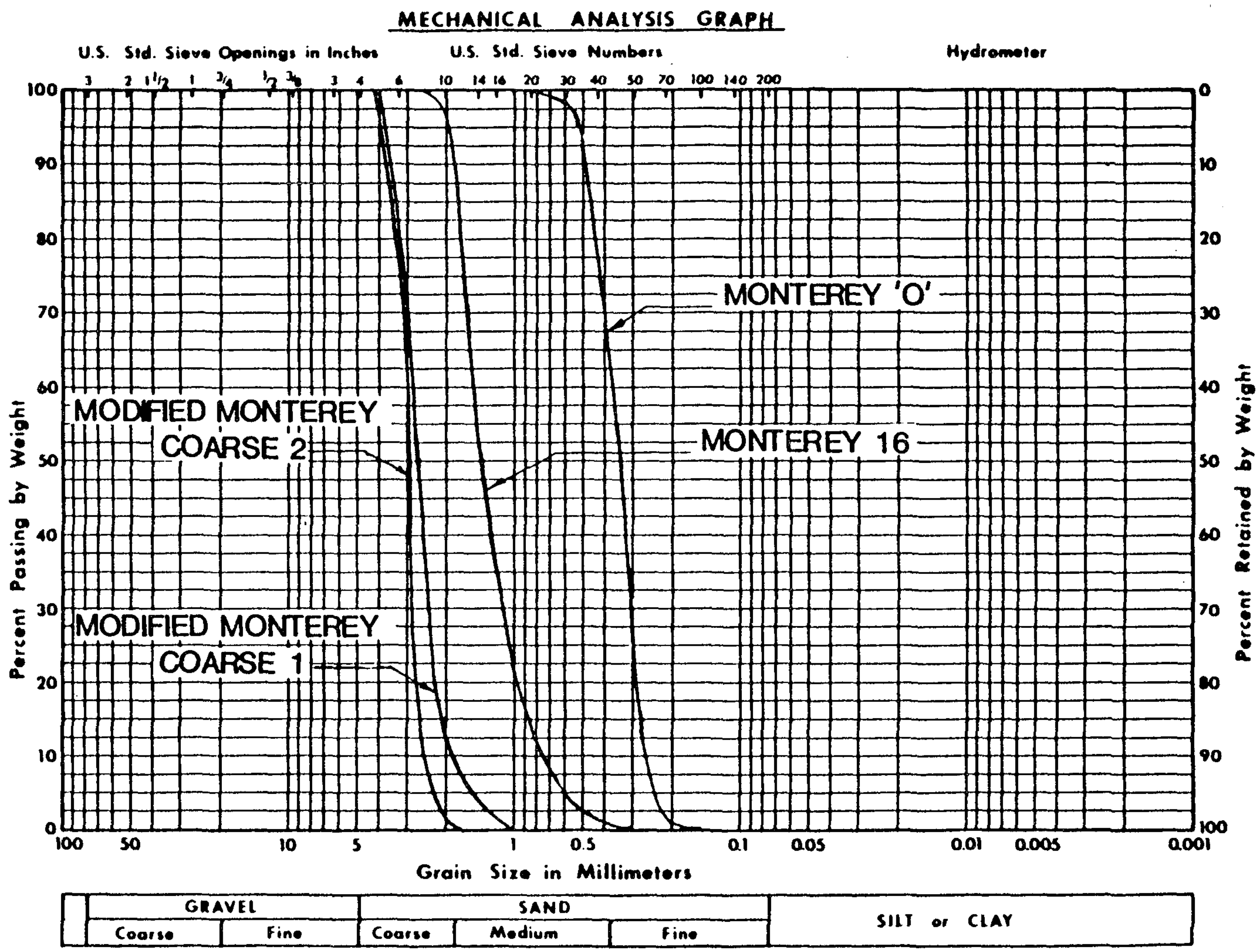


Figure 5.22: Gradations of Soils Tested to Investigate Membrane Compliance as Part of These Studies

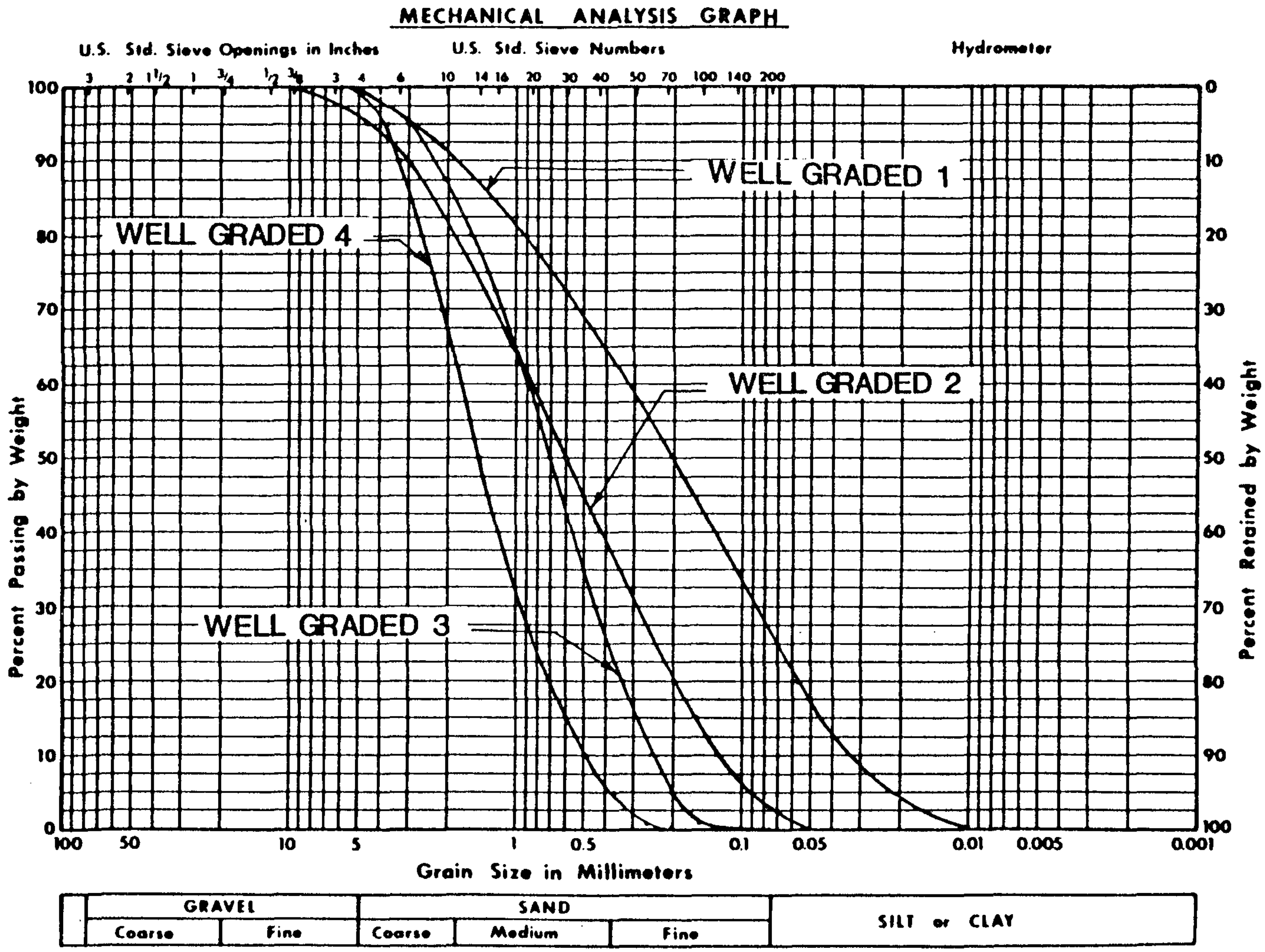


Figure 5.23: Gradations of Soils Tested to Investigate Membrane Compliance as Part of These Studies

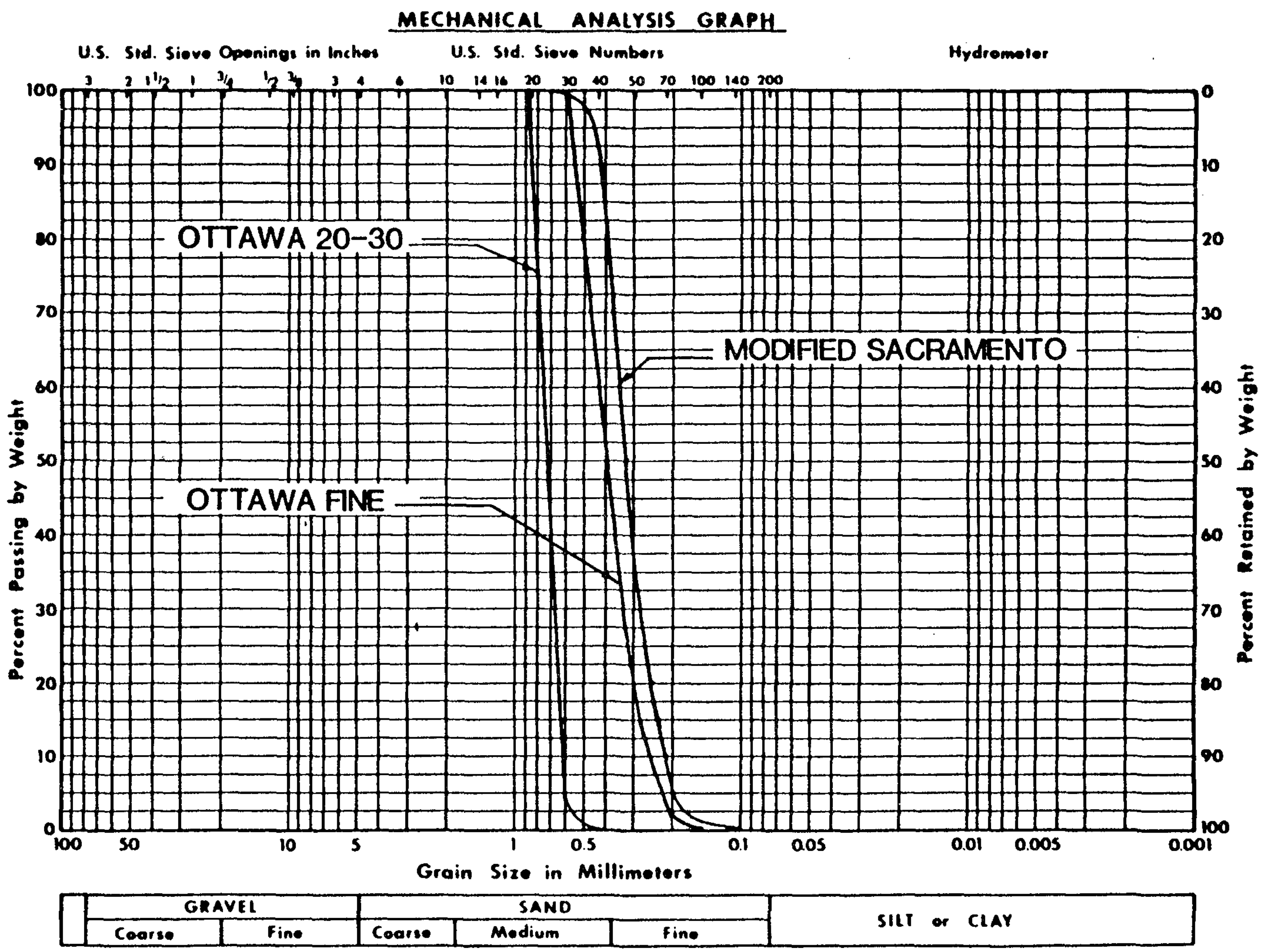


Figure 5.24: Gradations of Soils Tested to Investigate Membrane Compliance as Part of These Studies

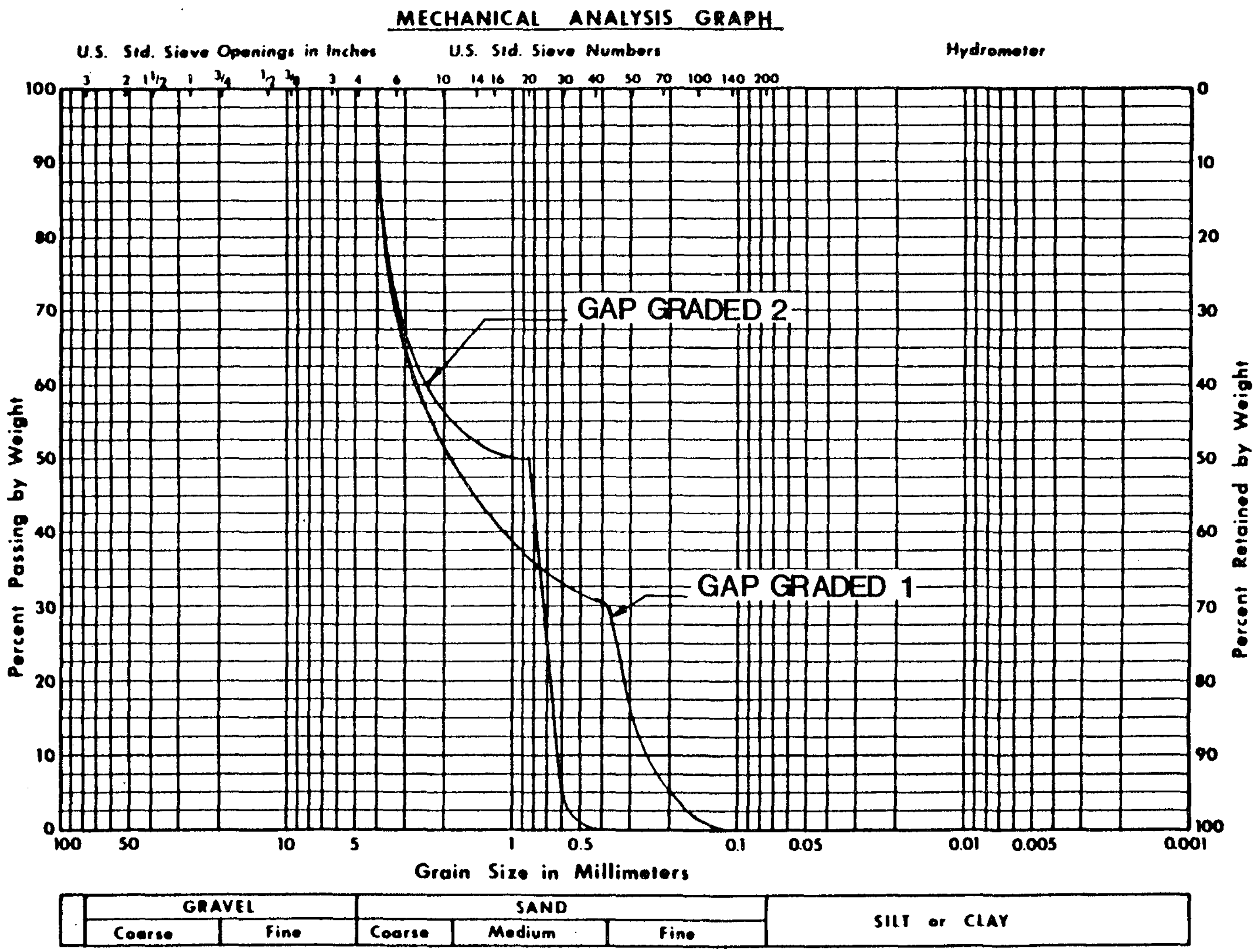


Figure 5.25: Gradations of Soils Tested to Investigate Membrane Compliance as Part of These Studies

representative particle sizes for the gravelly materials spanned three orders of magnitude from medium sands through coarse gravels. Among these were several more broadly graded materials. A listing of these gravelly soils is presented in Table 5.2, with a brief summary description and membrane compliance characteristics evaluated for each. Figures 5.26 through 5.36 show the membrane compliance values measured for these gravelly soils, and Figures 5.37 through 5.41 show the gradations for these materials. Again, all compliance evaluations were plotted as unit membrane compliance (volume change per unit membrane area: in^3/in^2) vs. effective confining stress and $\log_{10}$ of effective confining stress. All of the gravelly soils tested were prepared to $D_R \approx 55\%$, except where otherwise noted.

The results of the compliance measurement tests show very clearly the log-linear relationship between unit compliance and effective stress from which the slope can be taken to give the normalized unit compliance (S) for each material.

A number of additional data points were found in a review of the literature. Those data points for which the compliance measurement techniques were judged likely to provide reasonably accurate results were selected and combined with ours to build up a database with which to generate a relationship between measured compliance values and material particle sizes. Table 5.3 lists those materials tested by previous investigators with a brief summary description and membrane compliance value evaluated for each. Figures 5.42 through 5.47 show the gradations for these additional materials.

A composite plot of the relationship between normalized compliance values (S) as a function of mean particle size (D_{50}) for all of the available data for which the compliance measurement techniques were judged likely to provide reasonably accurate results, is shown in Figure 5.48. The scatter of points in this plot is

Table 5.2: Gradation and Membrane Compliance Characteristics of Gravelly Soils Tested

Soil Name	USCS Classification	D_{10} (mm)	D_{20} (mm)	D_{50} (mm)	$S^{\dagger}$ (cm/$\Delta\log\sigma_3'$)	Figure
Material 1	GP	11	12	14	0.117	5.26
Material 2	GP	15	18	24	0.157	5.27
Material 3	GP	5.9	7	8	0.072	5.28
Material 4	GP	39	47	56	0.406	5.29
Material 5	GP	3.1	4	5	0.0403	5.30
Material 6	GP	4.2	9	25	0.088	5.31
Material 7	GP	3.8	5.4	12.7	0.055	5.32
Material 8	GP	3.1	3.8	4.2	0.04	5.33
Material 9	GP	4.2	5	5.4	0.052	5.34
Material 10	GW	0.23	0.85	8.5	0.0115	5.35
Material 11	GP	3	6.5	19	0.0645	5.36

† = Normalized Unit Compliance; change in volume per unit membrane area per log-cycle change in effective conf. stress (cc/cm²/$\Delta\log\sigma_3'$).

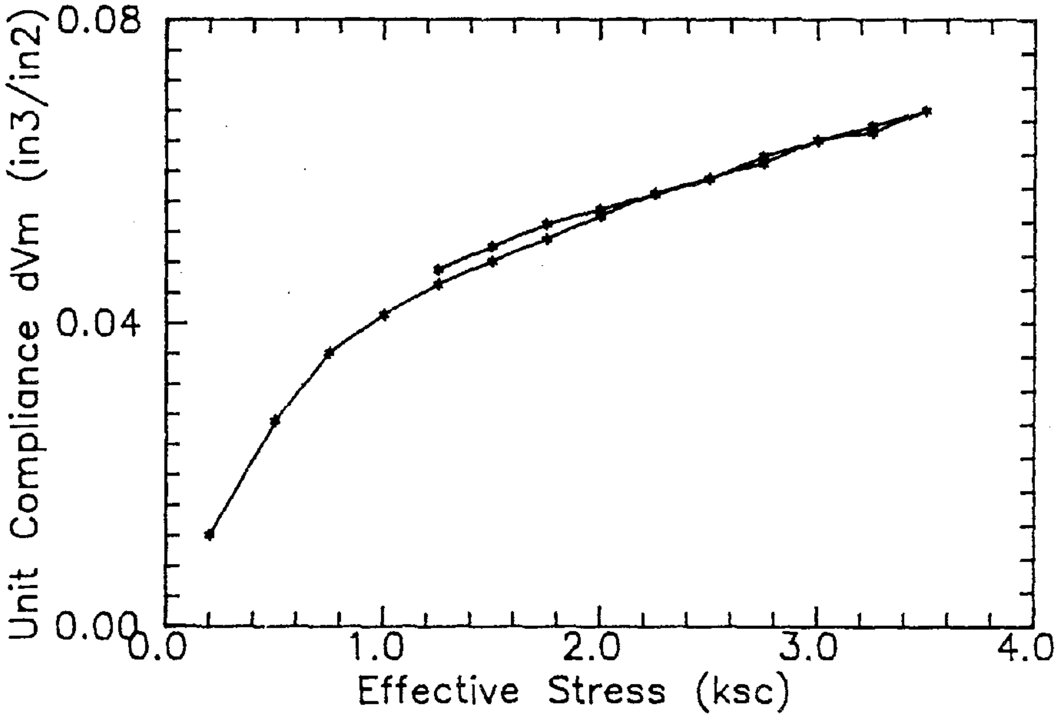


Figure 5.26a: Unit Membrane Compliance vs. Effective Confining Stress: Material 1

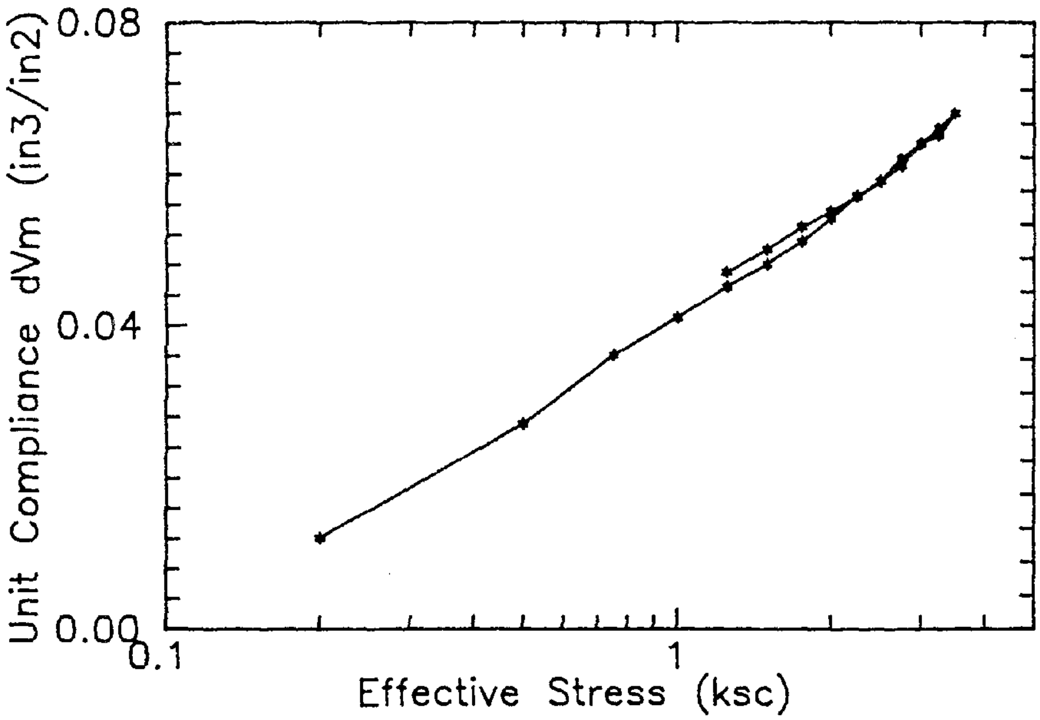


Figure 5.26b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 1

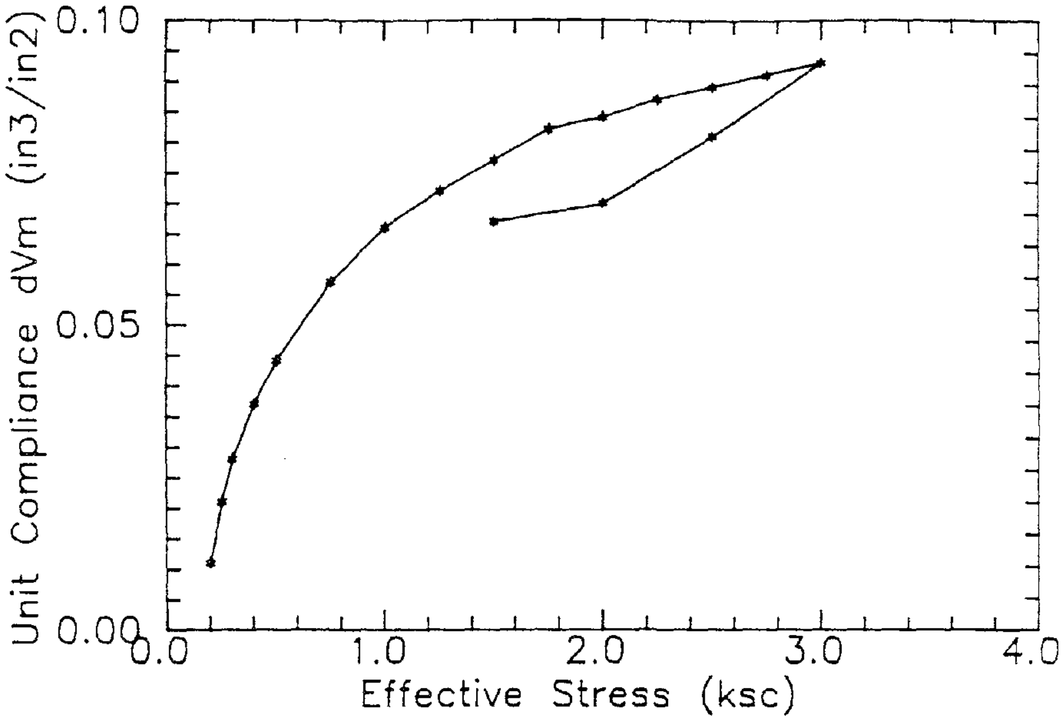


Figure 5.27a: Unit Membrane Compliance vs. Effective Confining Stress: Material 2

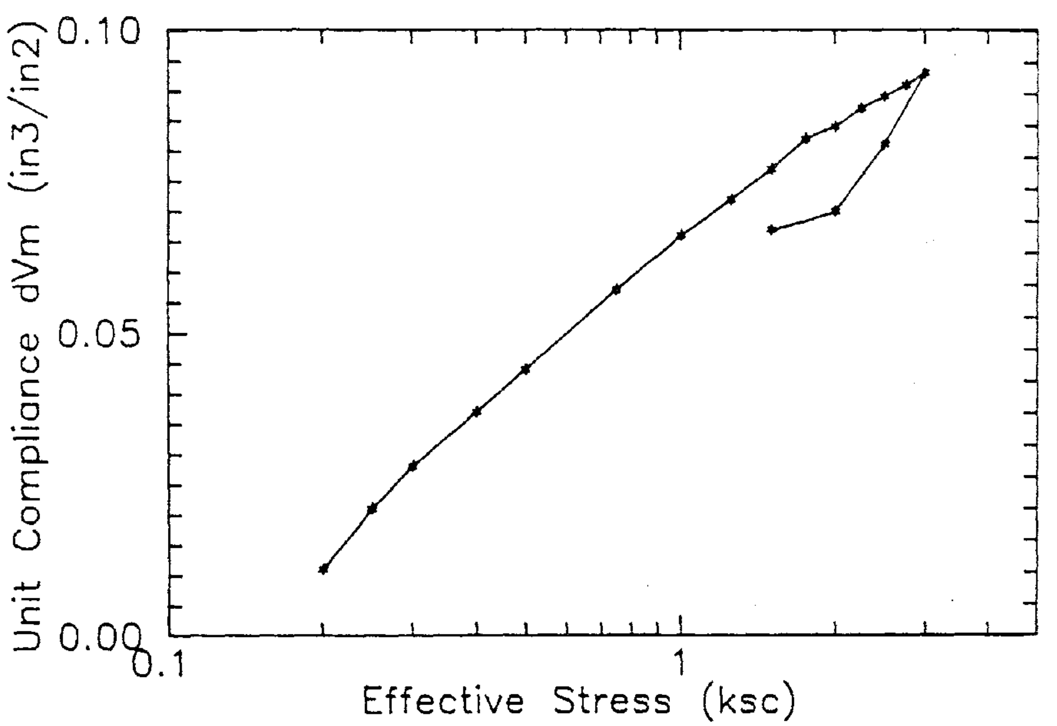


Figure 5.27b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 2

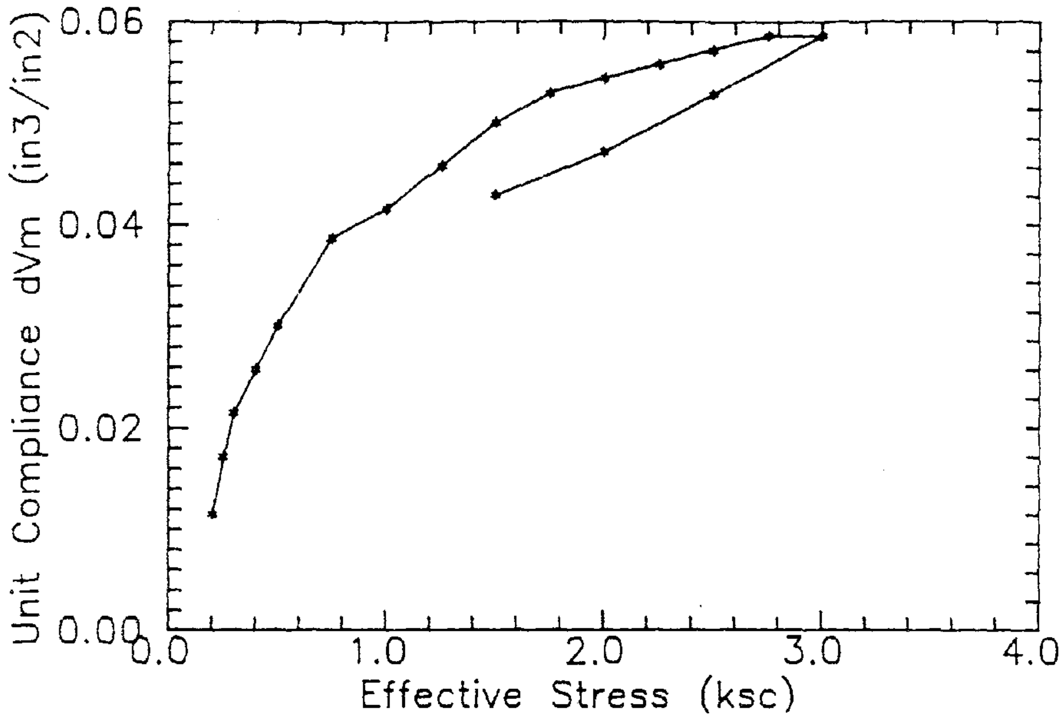


Figure 5.28a: Unit Membrane Compliance vs. Effective Confining Stress: Material 3

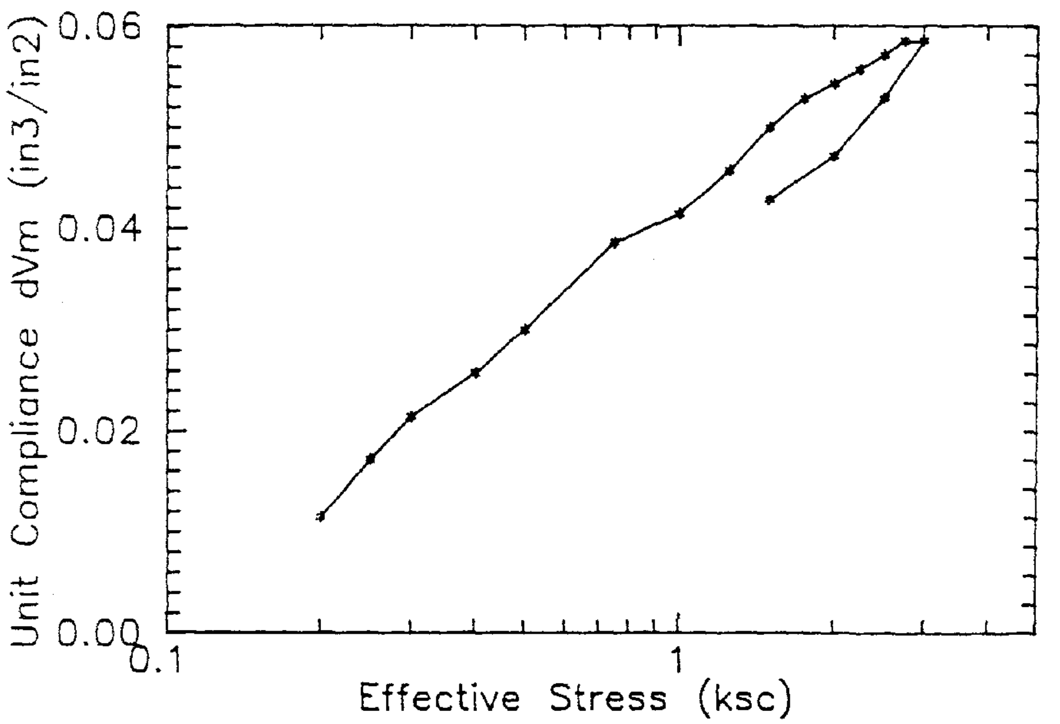


Figure 5.28b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 3

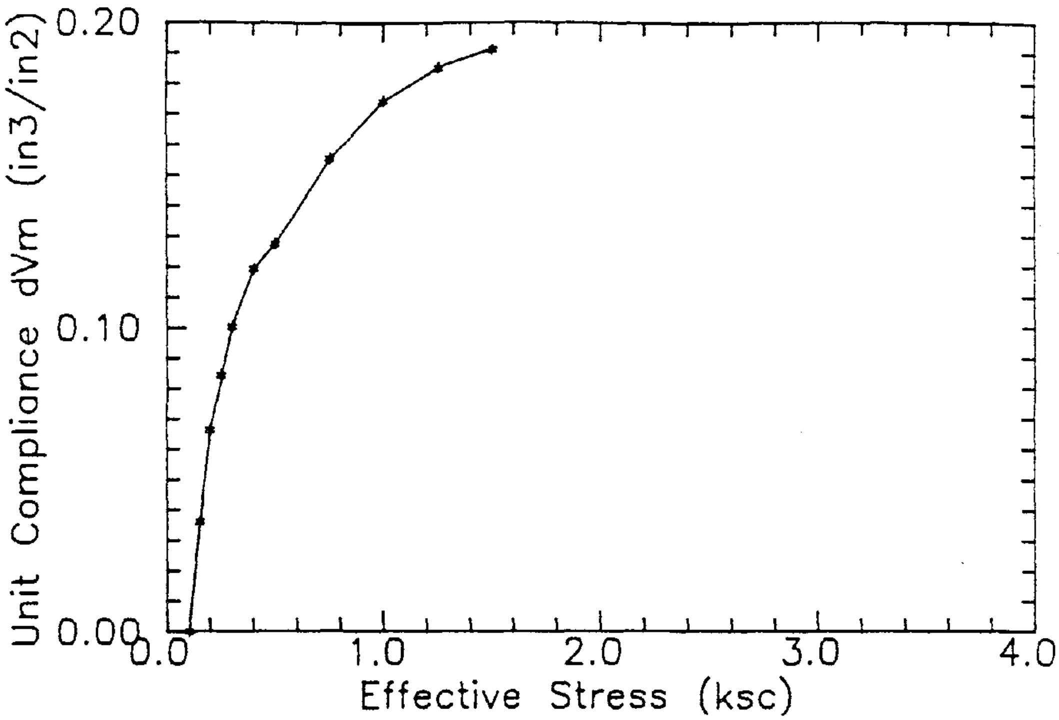


Figure 5.29a: Unit Membrane Compliance vs. Effective Confining Stress: Material 4

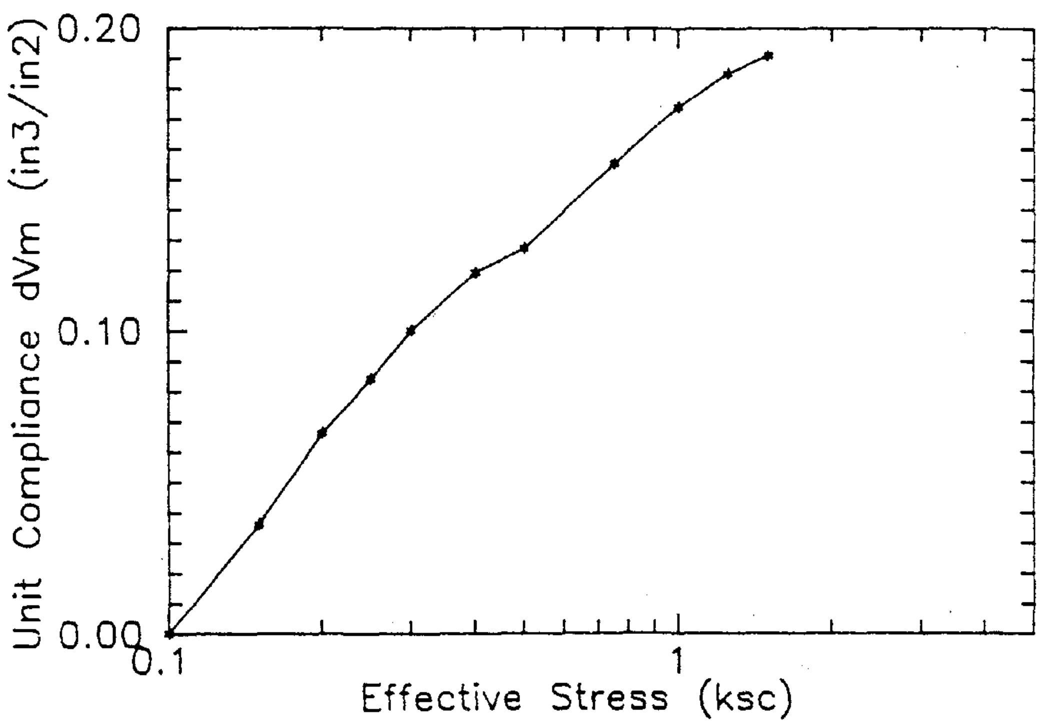


Figure 5.29b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 4

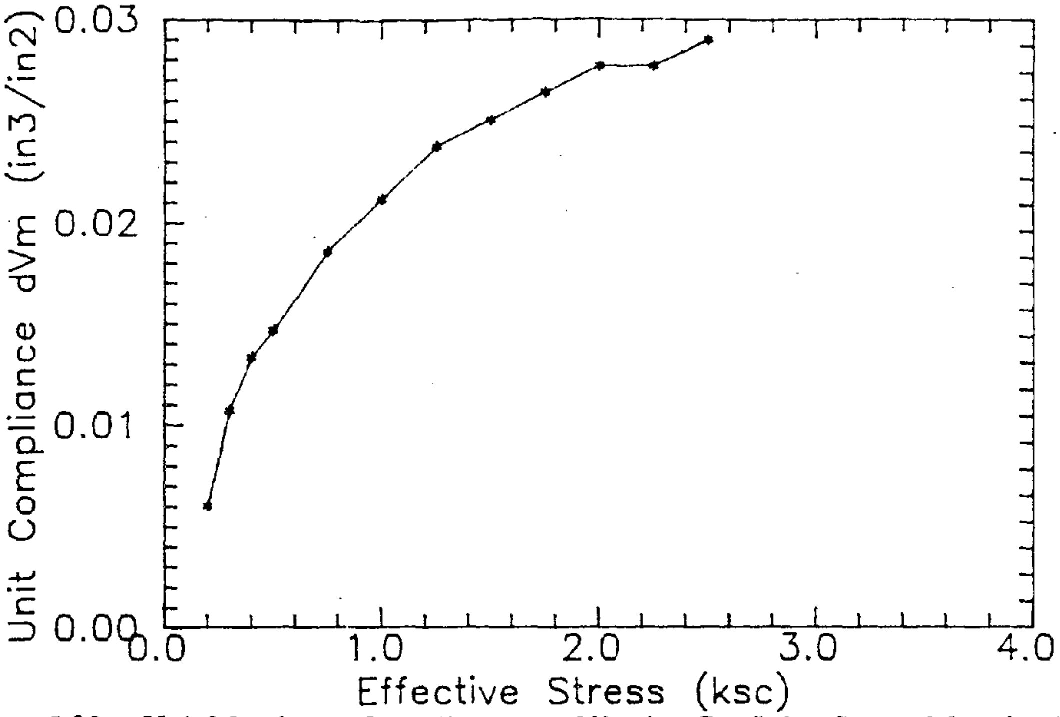


Figure 5.30a: Unit Membrane Compliance vs. Effective Confining Stress: Material 5

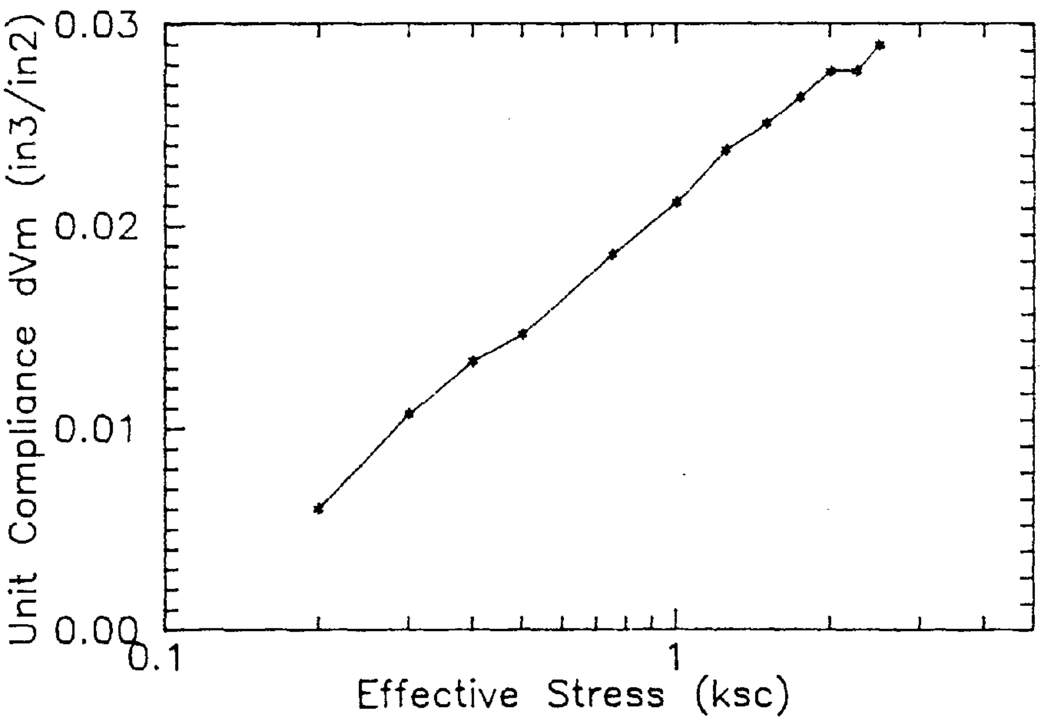


Figure 5.30b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 5

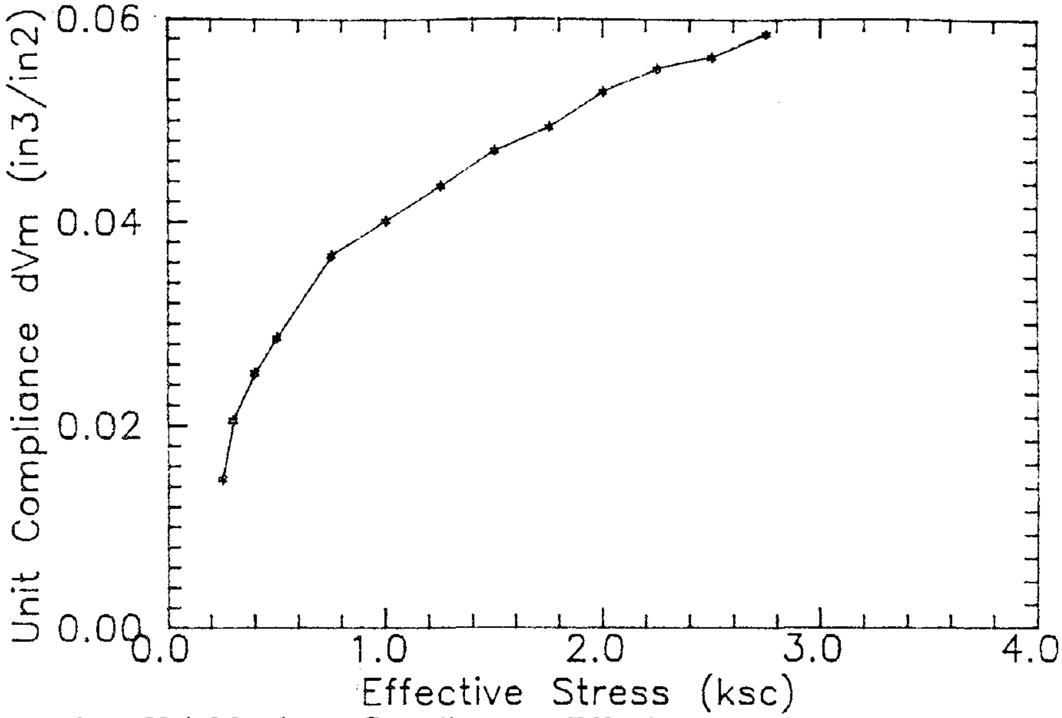


Figure 5.31a: Unit Membrane Compliance vs. Effective Confining Stress: Material 6

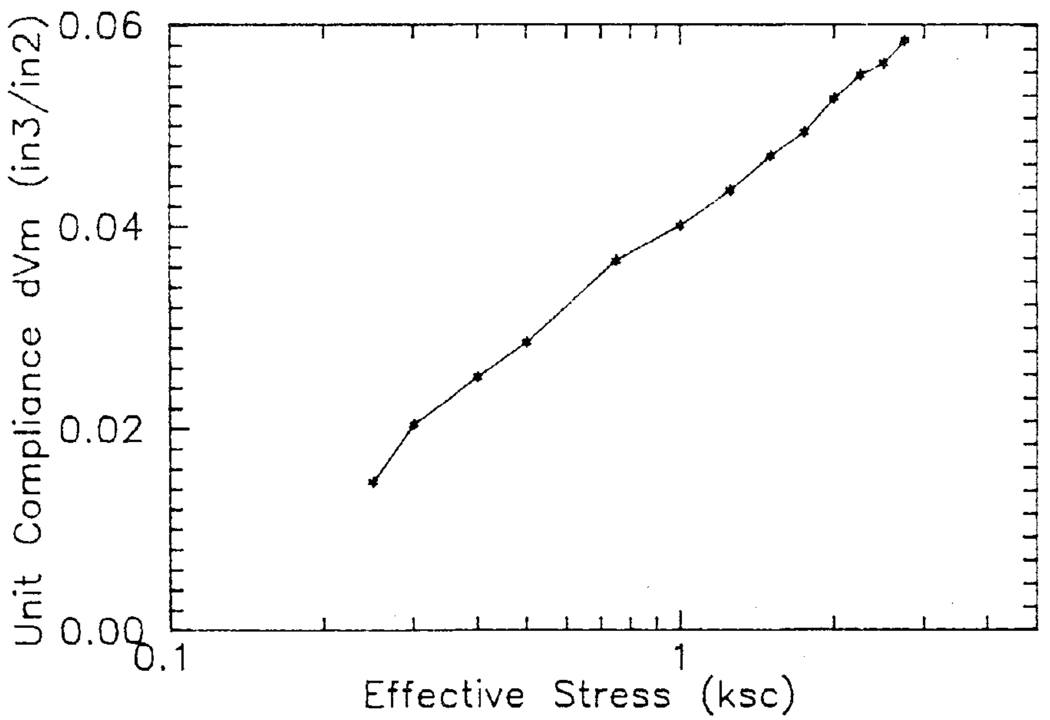


Figure 5.31b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 6

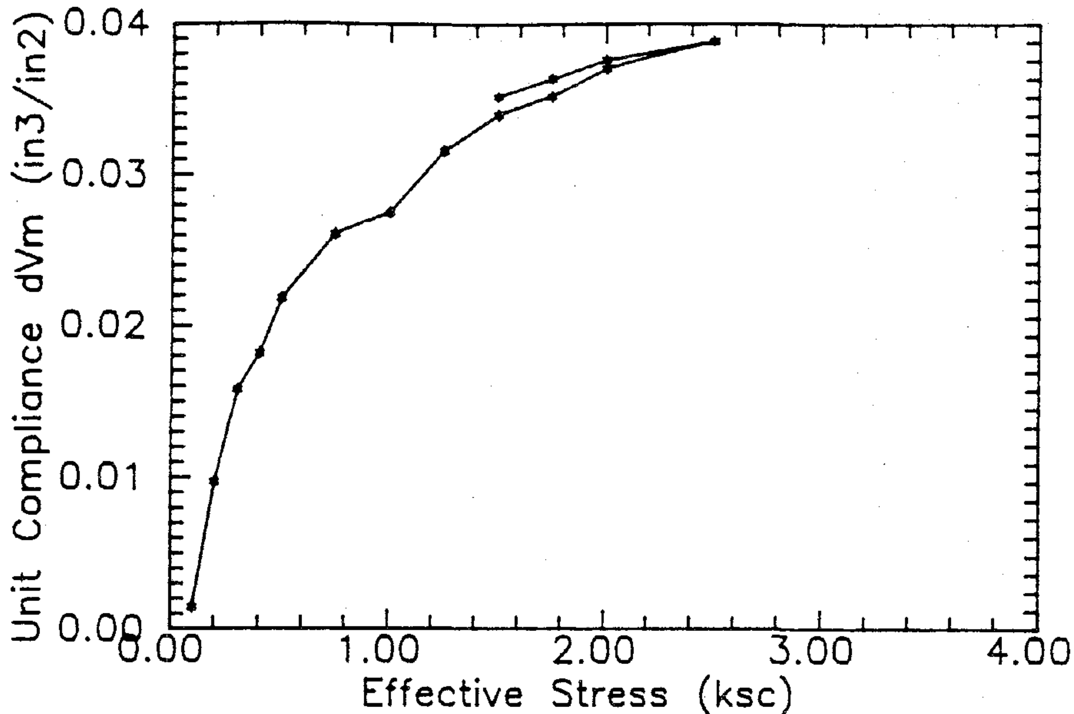


Figure 5.32a: Unit Membrane Compliance vs. Effective Confining Stress: Material 7

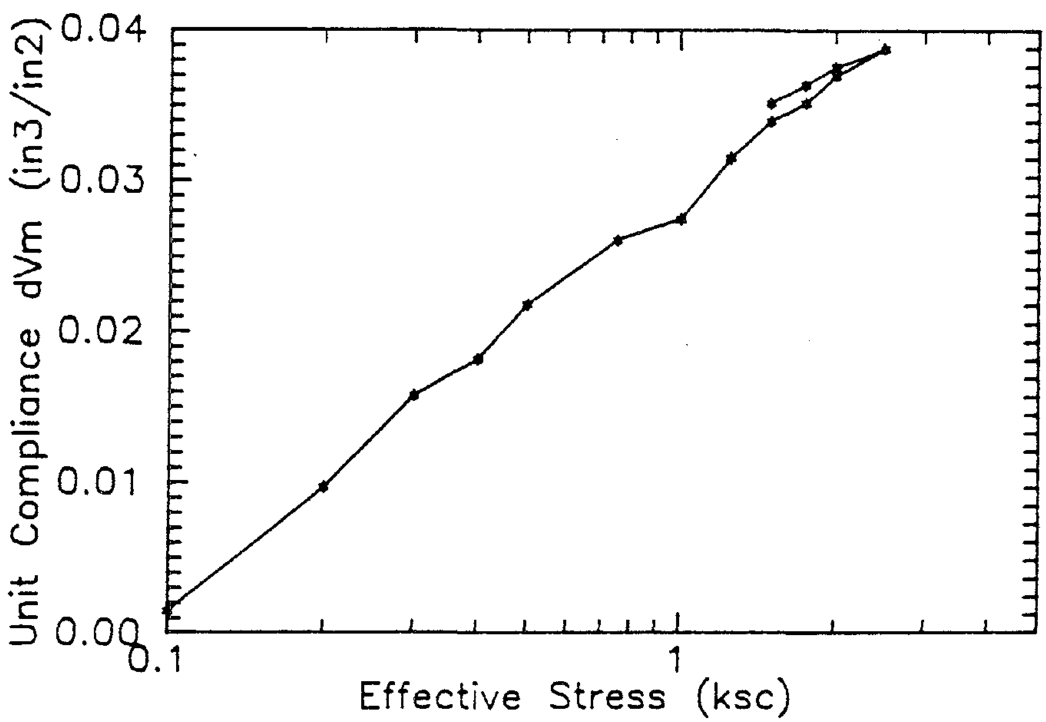


Figure 5.32b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 7

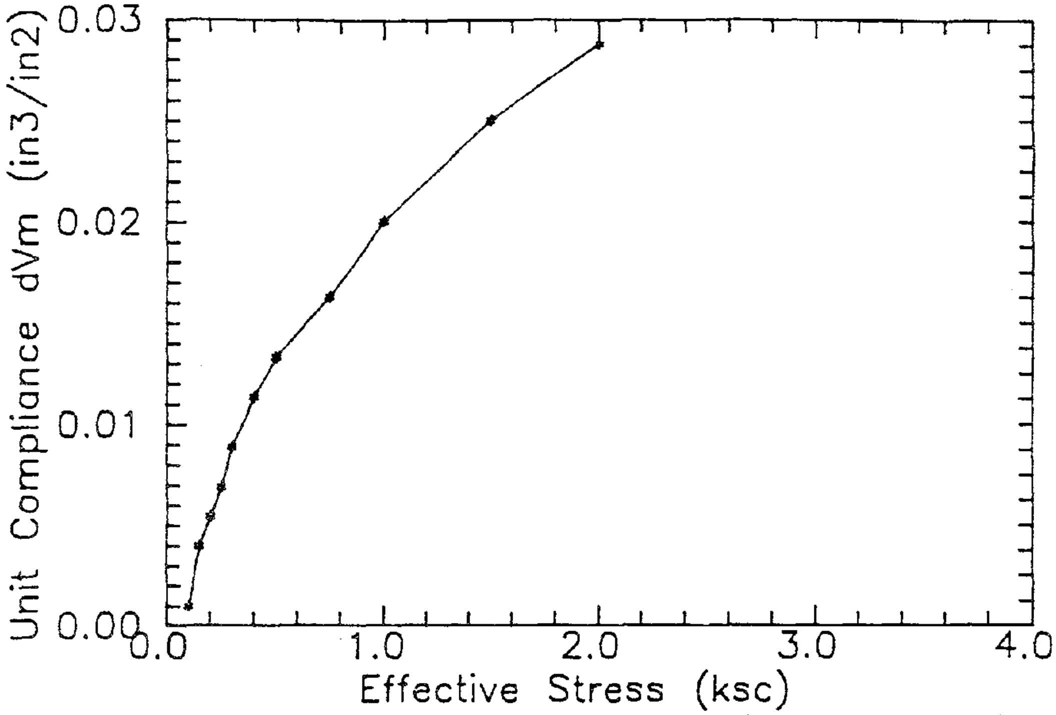


Figure 5.33a: Unit Membrane Compliance vs. Effective Confining Stress: Material 8

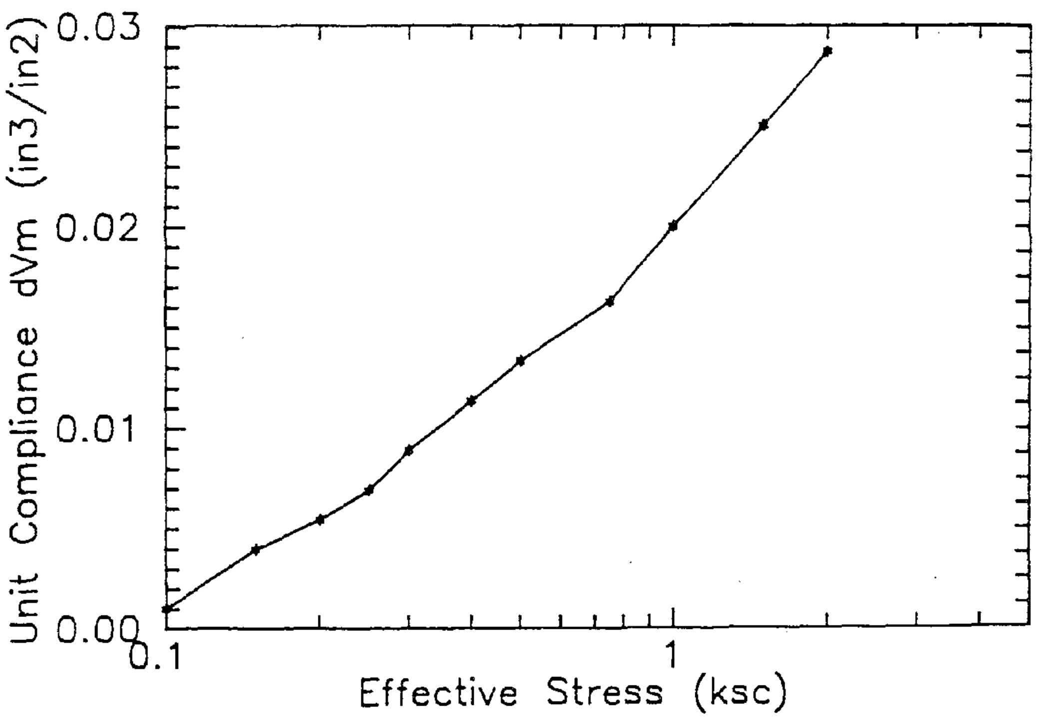


Figure 5.33b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 8

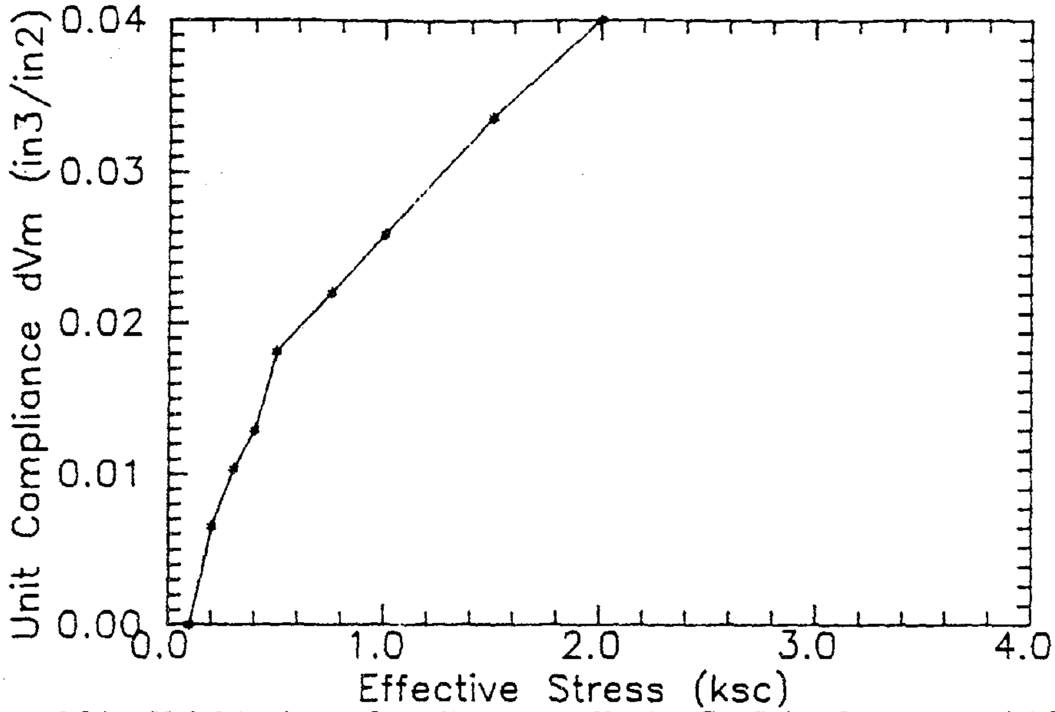


Figure 5.34a: Unit Membrane Compliance vs. Effective Confining Stress: Material 9

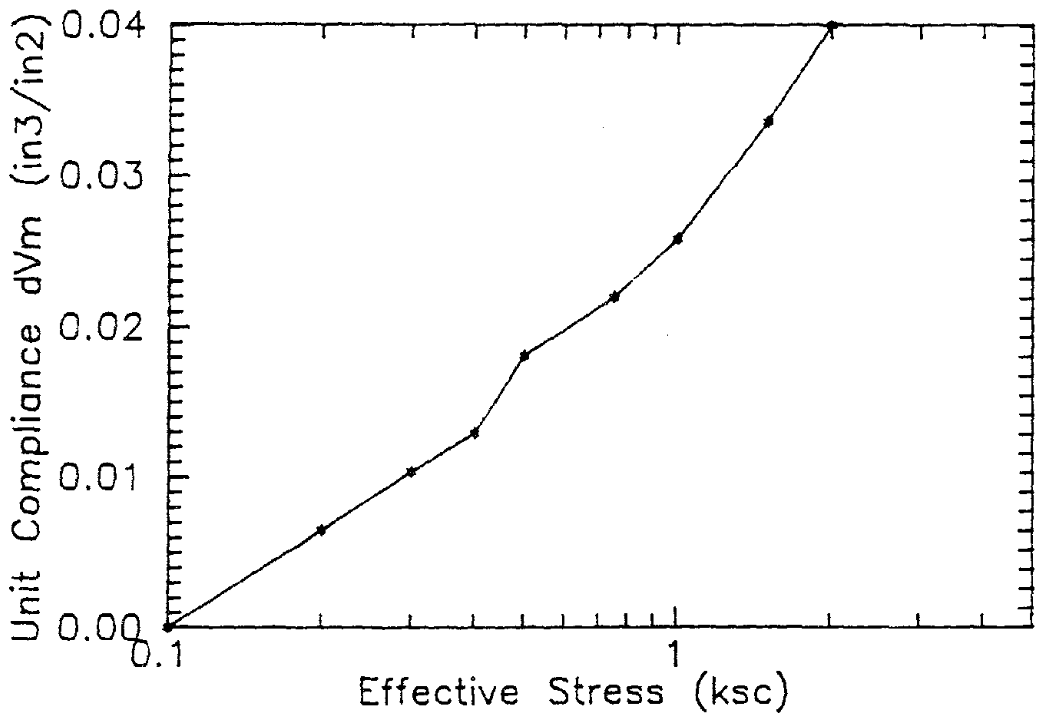


Figure 5.34b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 9

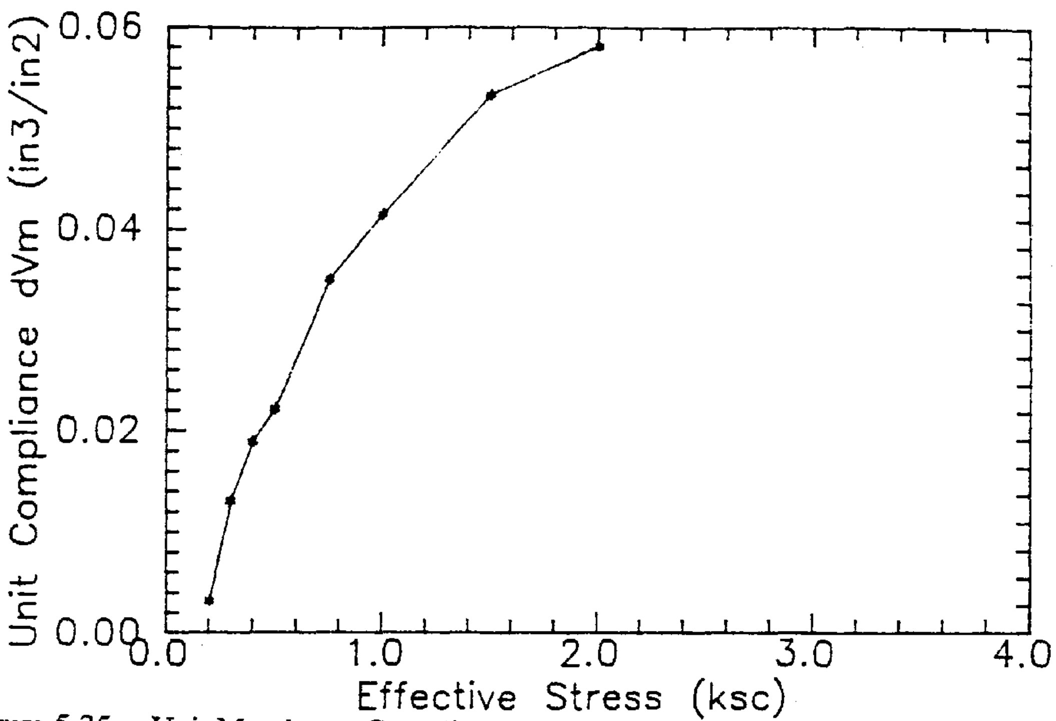


Figure 5.35a: Unit Membrane Compliance vs. Effective Confining Stress: Material 10

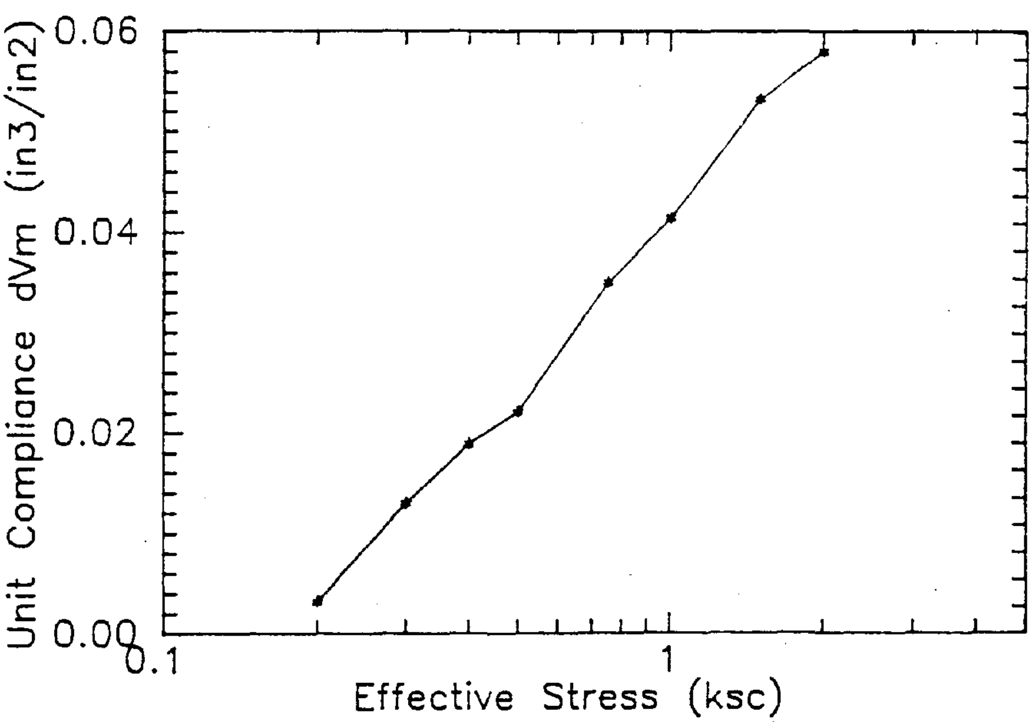


Figure 5.35b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 10

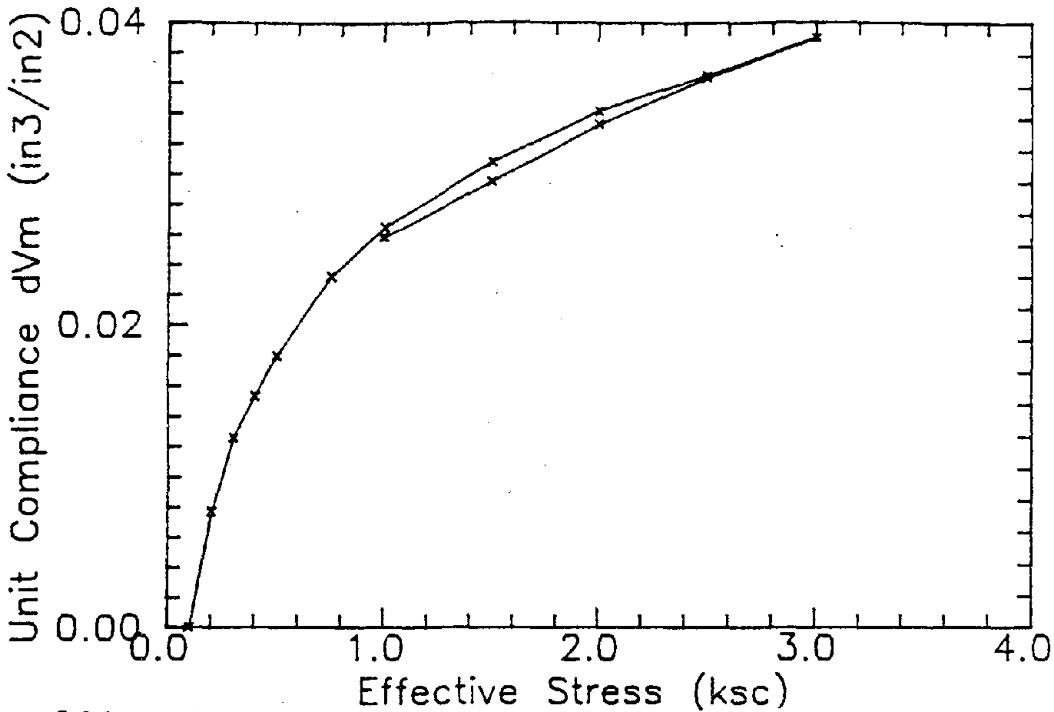


Figure 5.36a: Unit Membrane Compliance vs. Effective Confining Stress: Material 11

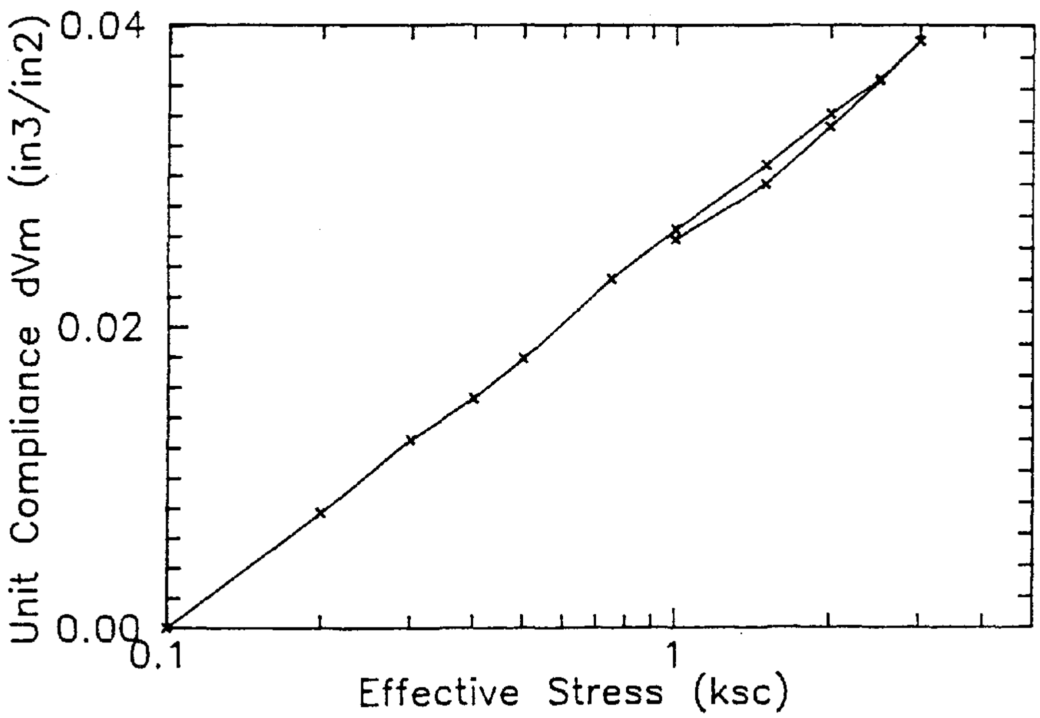


Figure 5.36b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 11

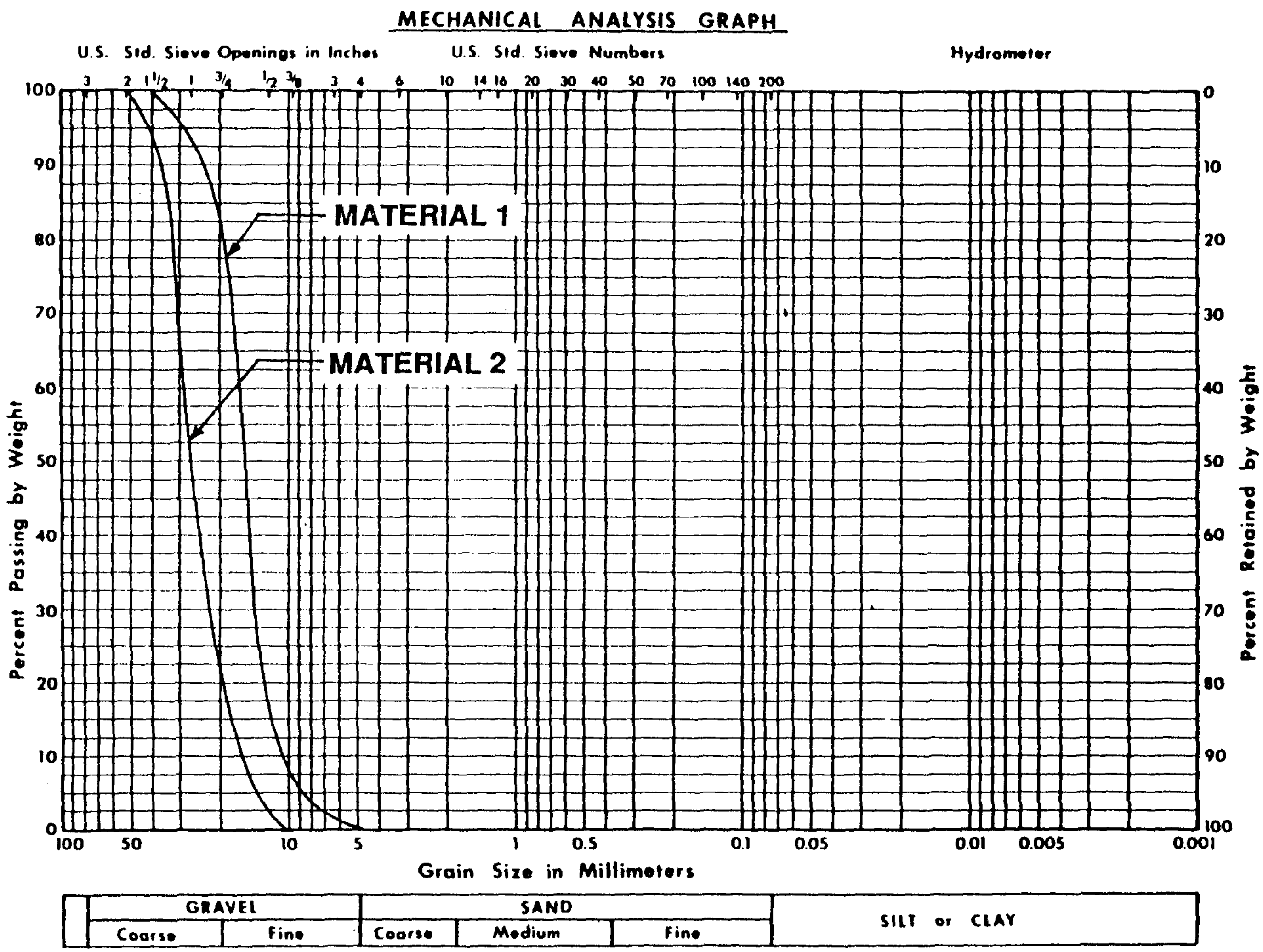


Figure 5.37: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

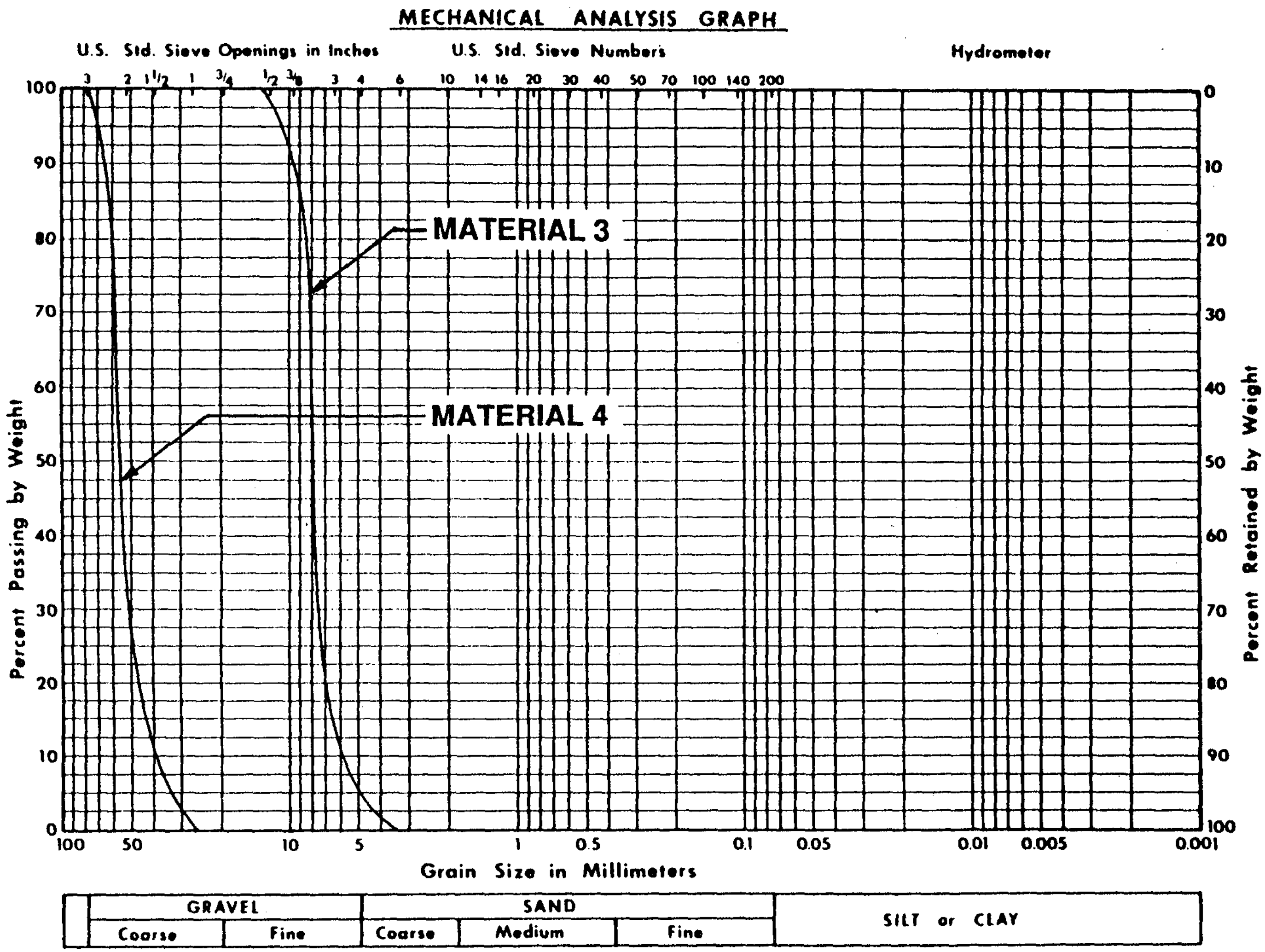


Figure 5.38: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

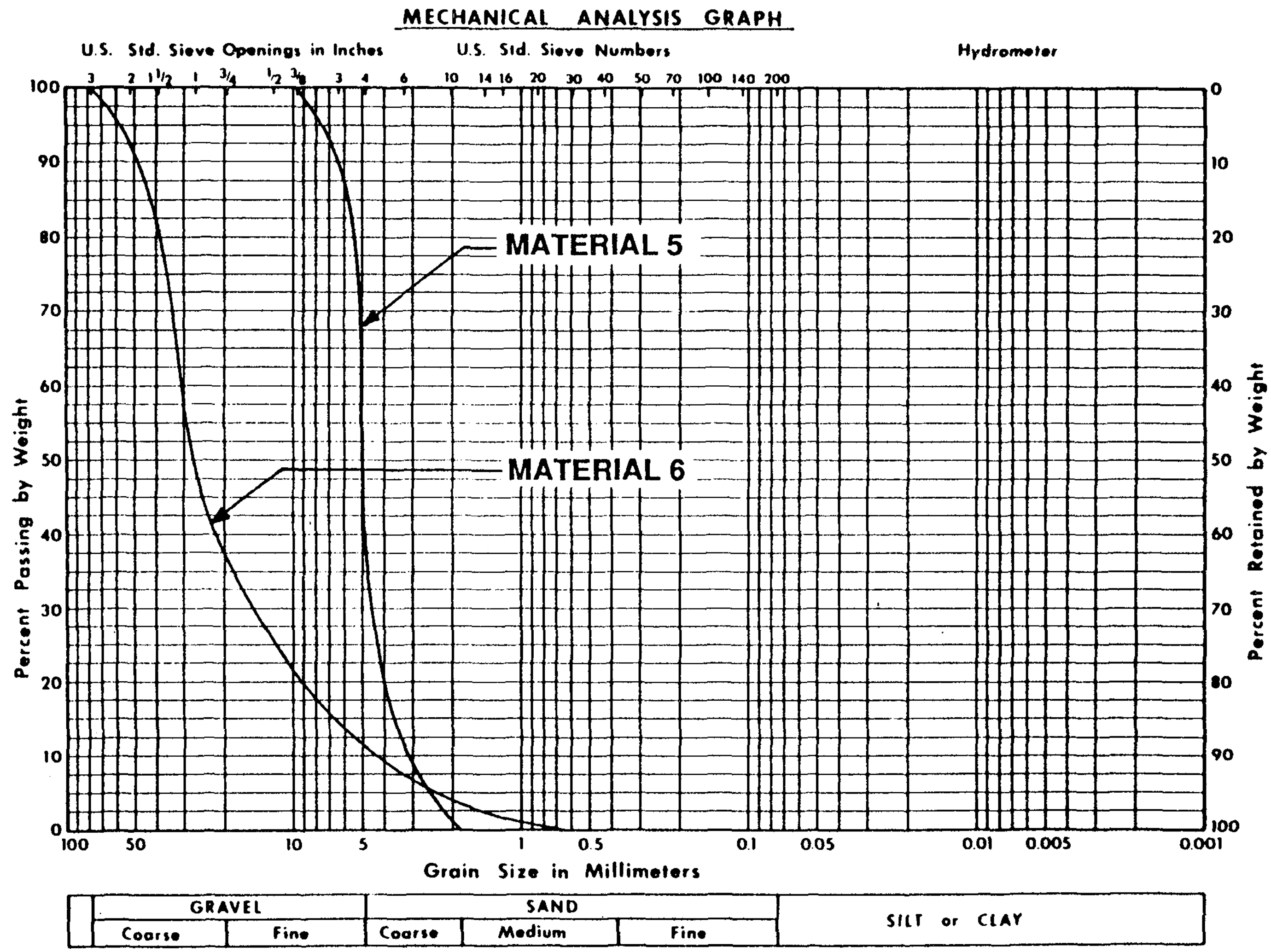


Figure 5.39: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

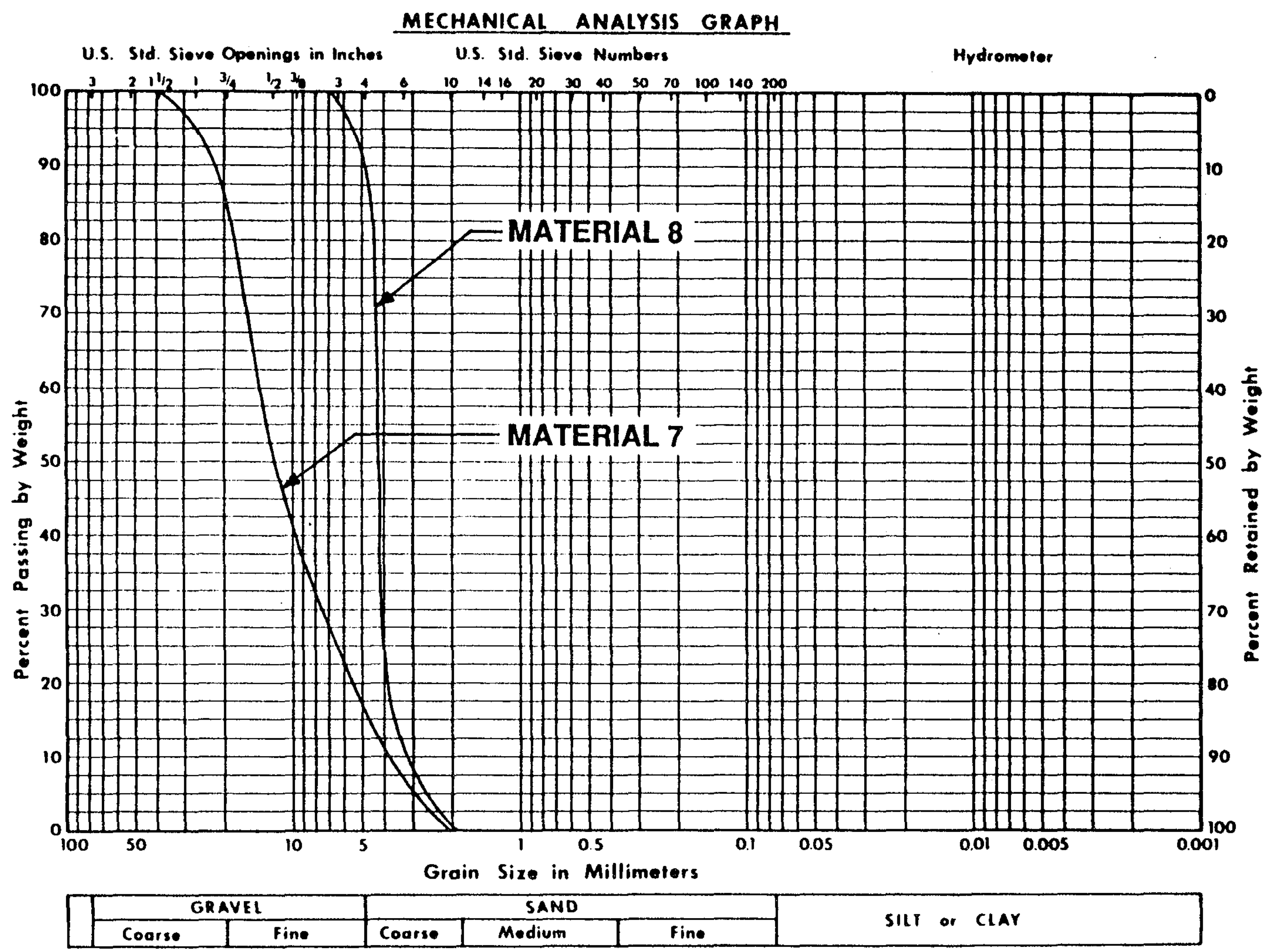


Figure 5.40: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

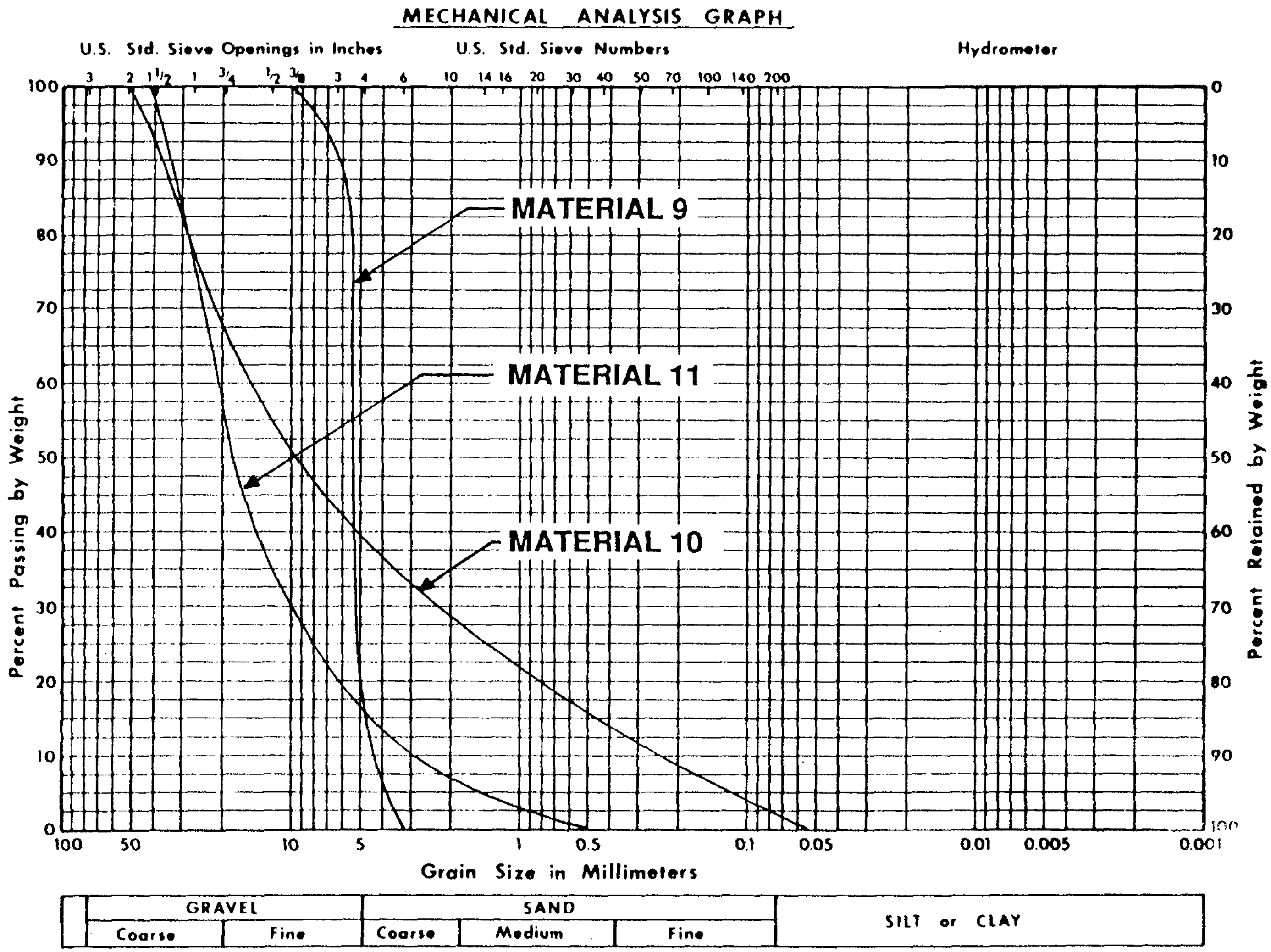


Figure 5.41: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

Table 5.3: Gradation and Membrane Compliance Characteristics of Soils Tested by Selected Alternate Investigators

Soil Name	USCS Classification	D_{10} (mm)	D_{20} (mm)	D_{50} (mm)	S (cm/$\Delta\log\sigma'_3$)
Kiekbusch A	SP	0.3	0.33	0.40	0.0045
Kiekbusch B	SP	0.1	0.14	0.18	0.002
Kiekbusch C	SP	0.07	0.08	0.09	0.0014
Frydman A	SP	0.163	0.17	0.18	0.0027
Frydman B	SP	1.68	1.70	1.85	0.0165
Frydman C	SP	.283	0.29	0.30	0.0045
Steinbach 1	SP	.22	0.23	0.30	0.0035
Steinbach 2	SP	.45	0.51	0.60	0.009
Steinbach 3	SP	.84	1.05	1.50	0.0147
Siddiqi A	GW	.13	0.40	3.8	0.0053
Siddiqi B	SW-SM	.074	0.21	1.4	0.0041
Siddiqi C	SP-SM	.40	0.80	4.0	0.014
Evans	GP	5.5	6.1	6.5	0.056
Hynes	GM	.40	6.0	20.0	0.039

Note: S = Normalized unit membrane penetration; the volumetric change (cc.) due to membrane penetration per square cm. of membrane area per logarithmic cycle of change in effective confining stress (σ'_3)

References:

1. Keikbusch, M. and Schuppener, B. (1977)
2. Frydman, S., Zeitlen, J. G. and Alpan, I. (1973)
3. Steinbach, J. (1967)
4. Siddiqi, F. H., Seed, R. B., Chan, C. K., Seed, H. B. and Pyke, R. M. (1987)
5. Evans, M. D. and Seed, H. B. (1987)
6. Hynes, M. E. (1988)

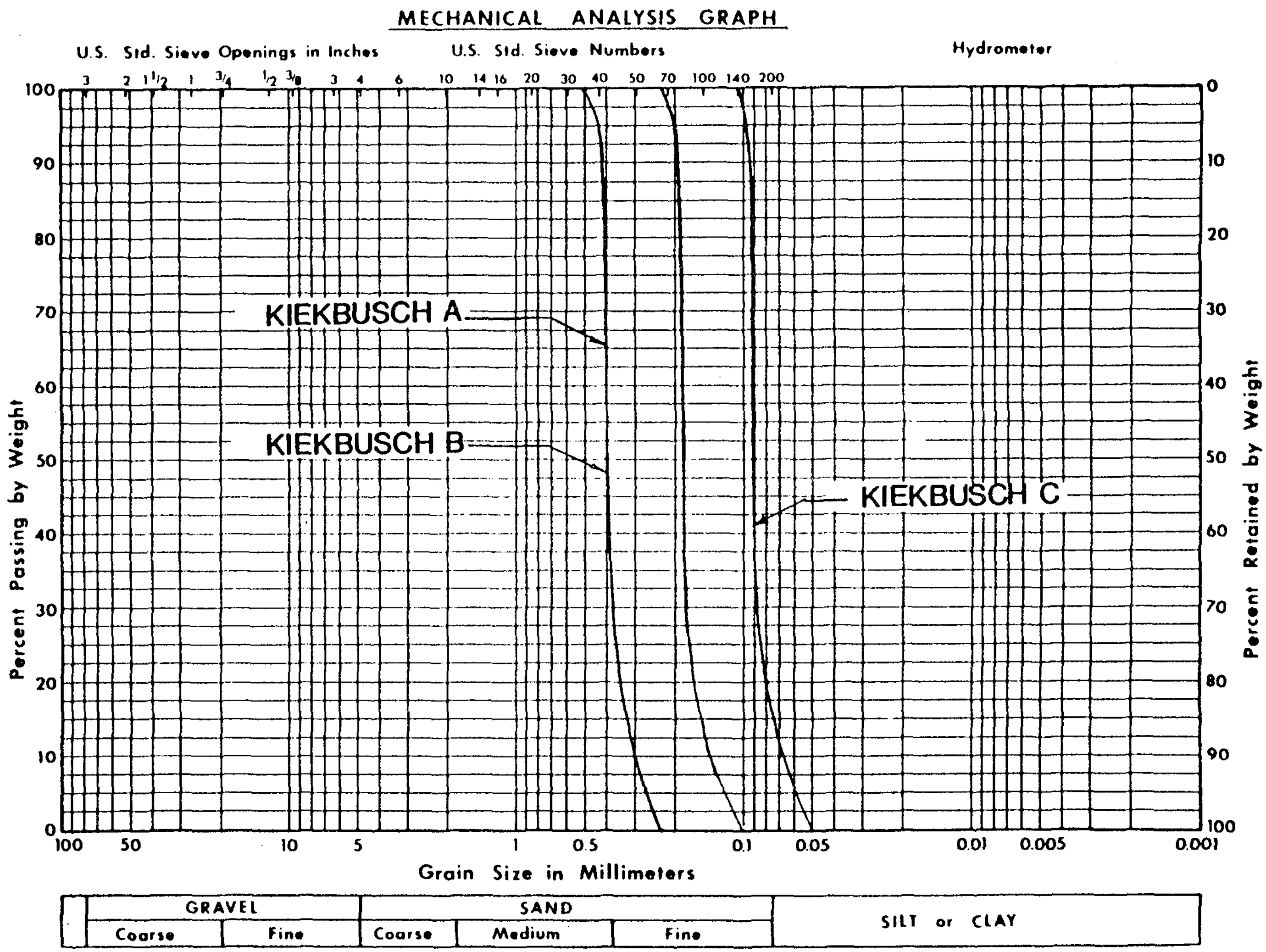


Figure 5.42: Gradations of Soils Tested for Membrane Compliance by Kiekbusch and Shuppener (1977)

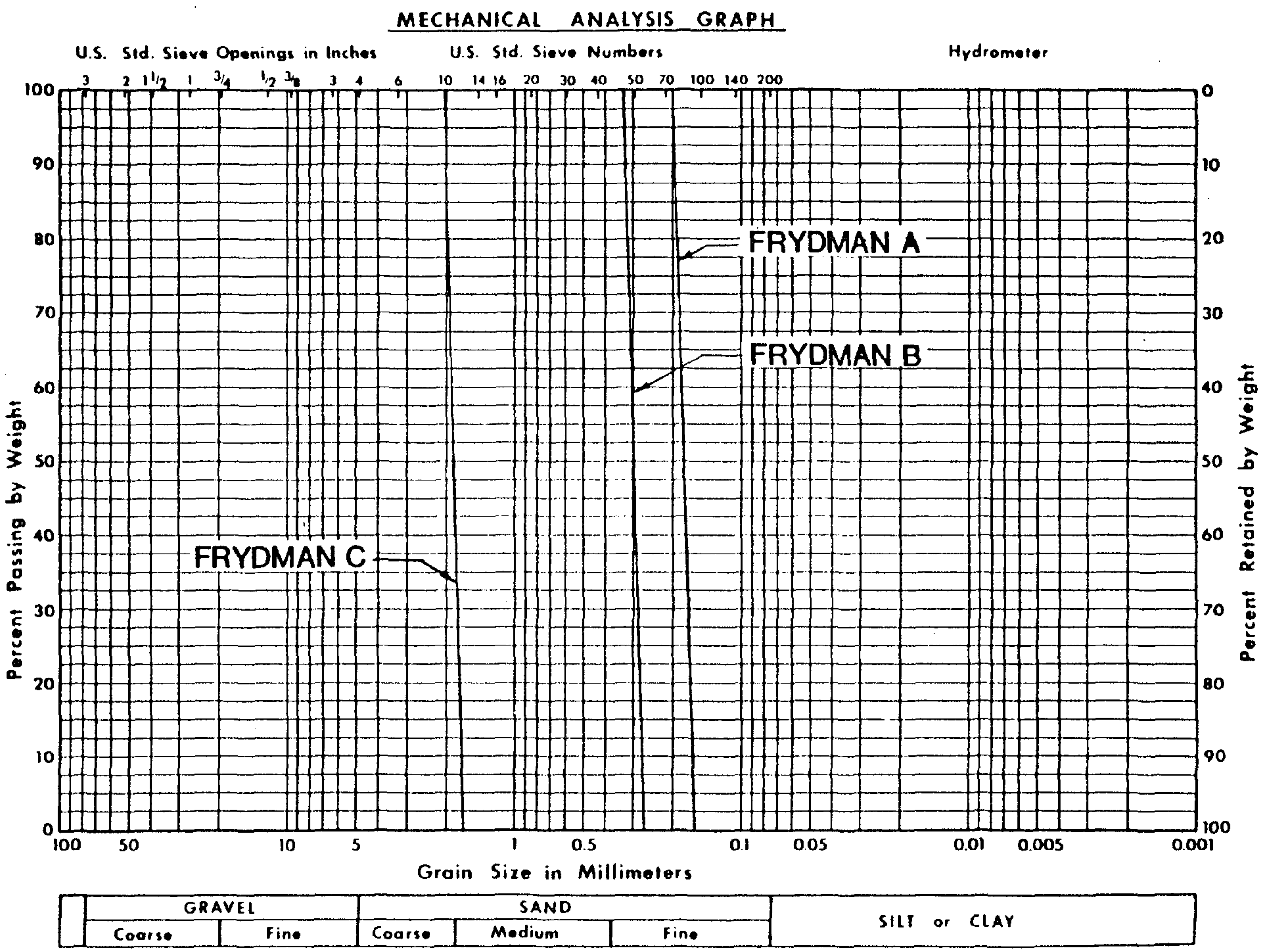


Figure 5.43: Gradations of Soils Tested for Membrane Compliance by Frydman et al. (1973)

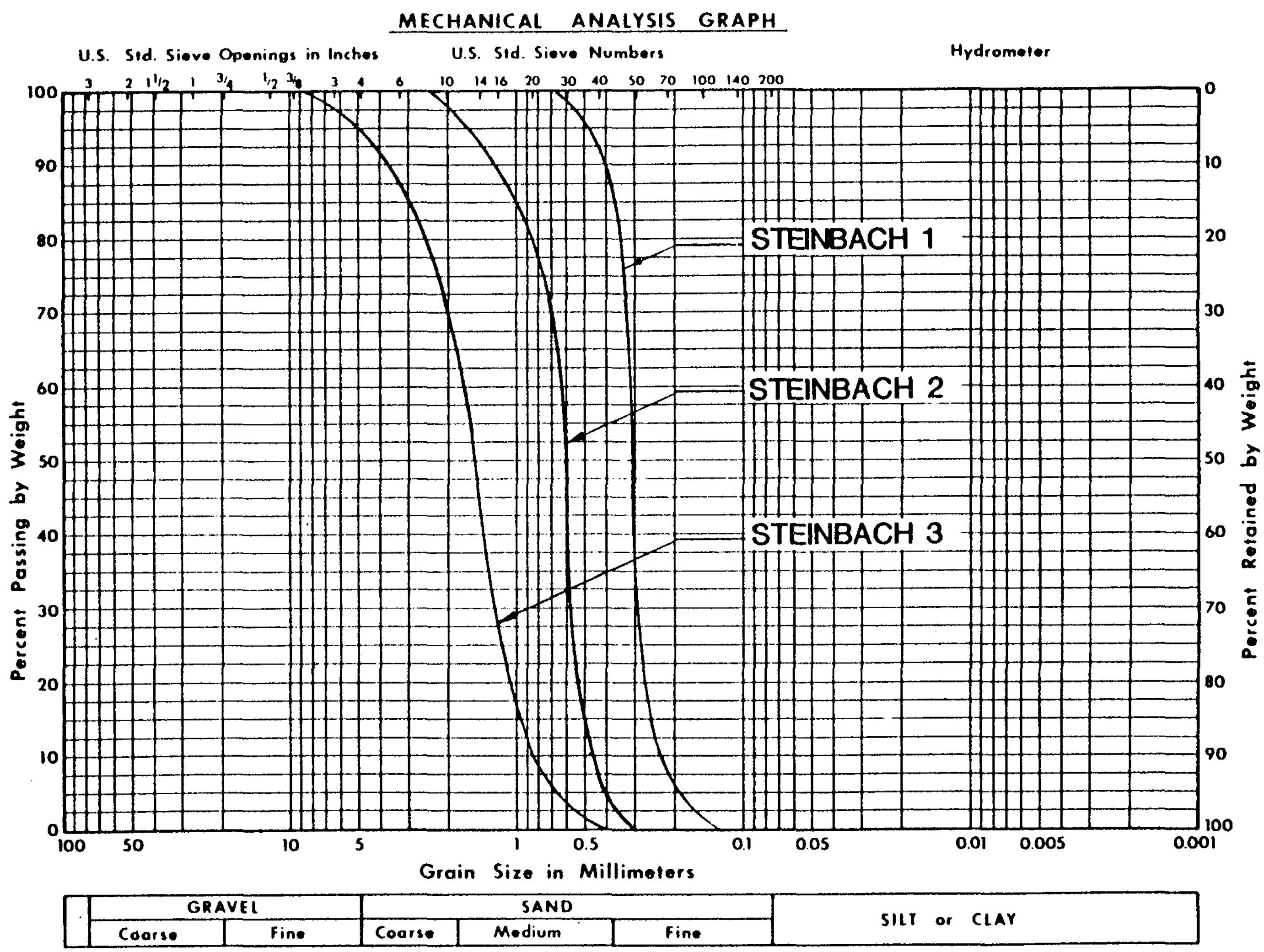


Figure 5.44: Gradations of Soils Tested for Membrane Compliance by Steinbach (1977)

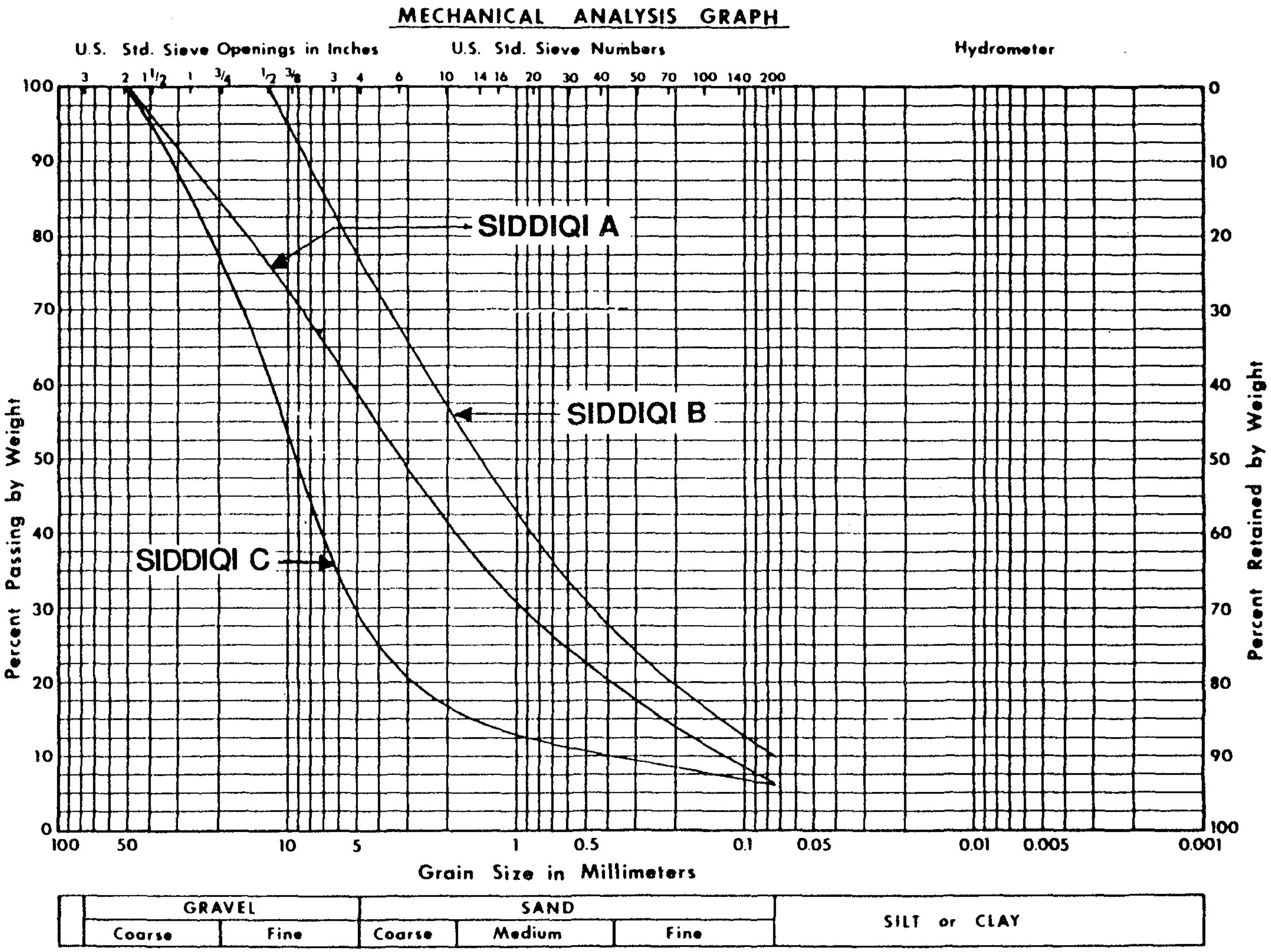


Figure 5.45: Gradations of Soils Tested for Membrane Compliance by Siddiqi (1984)

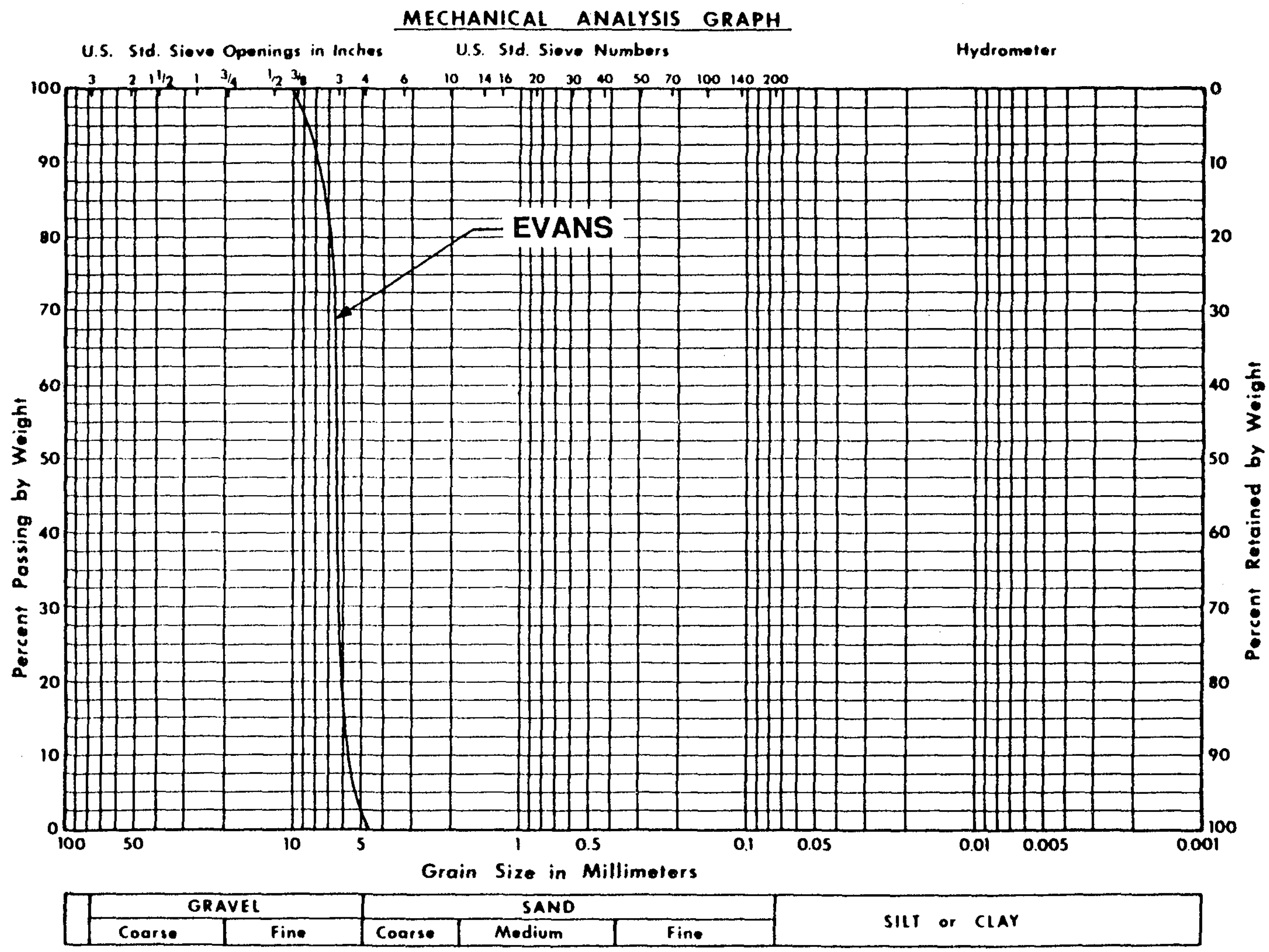


Figure 5.46: Gradations of Soils Tested for Membrane Compliance by Evans (1987)

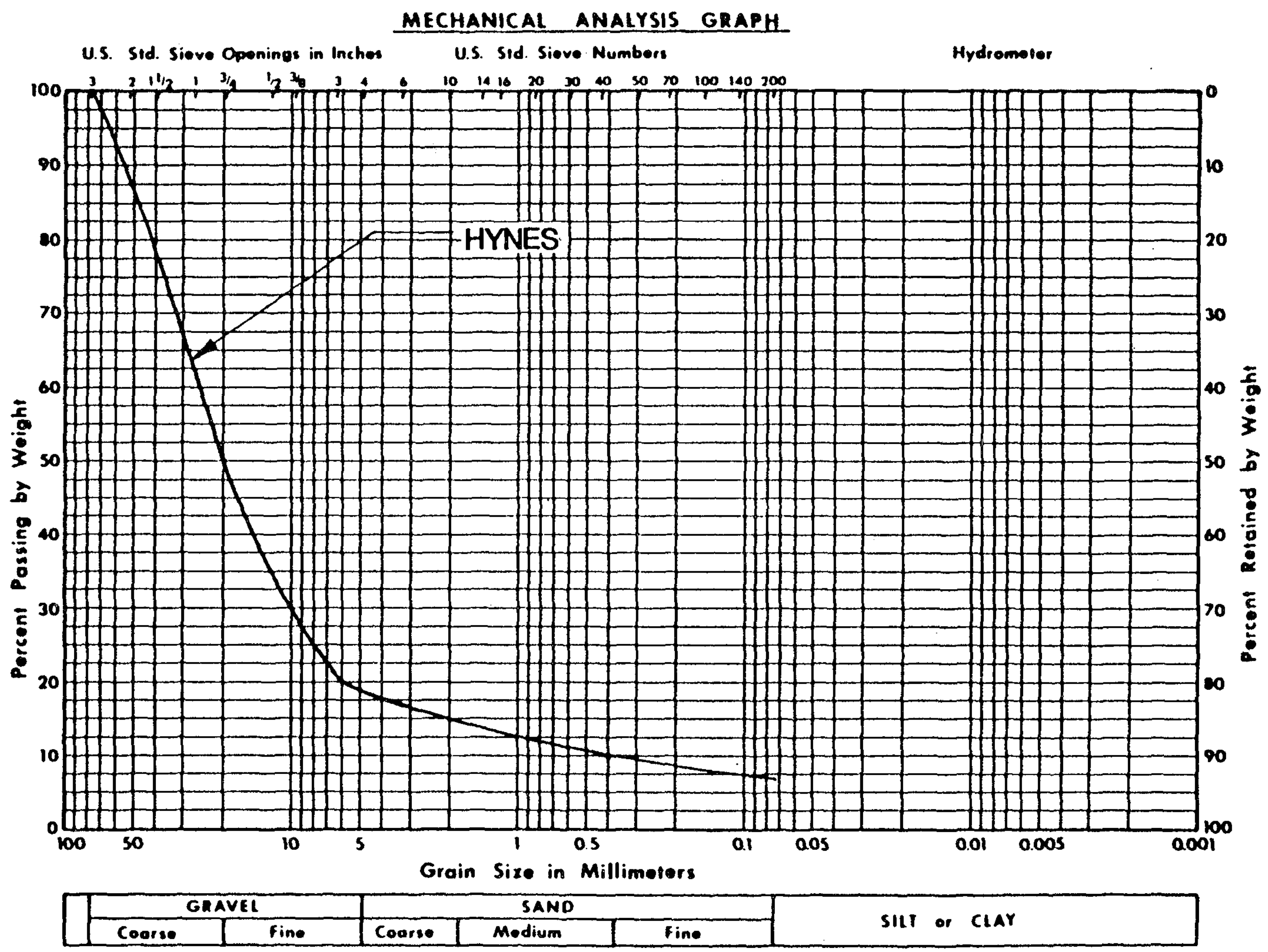


Figure 5.47: Gradation of Soil Tested for Membrane Compliance by Hynes (1988)

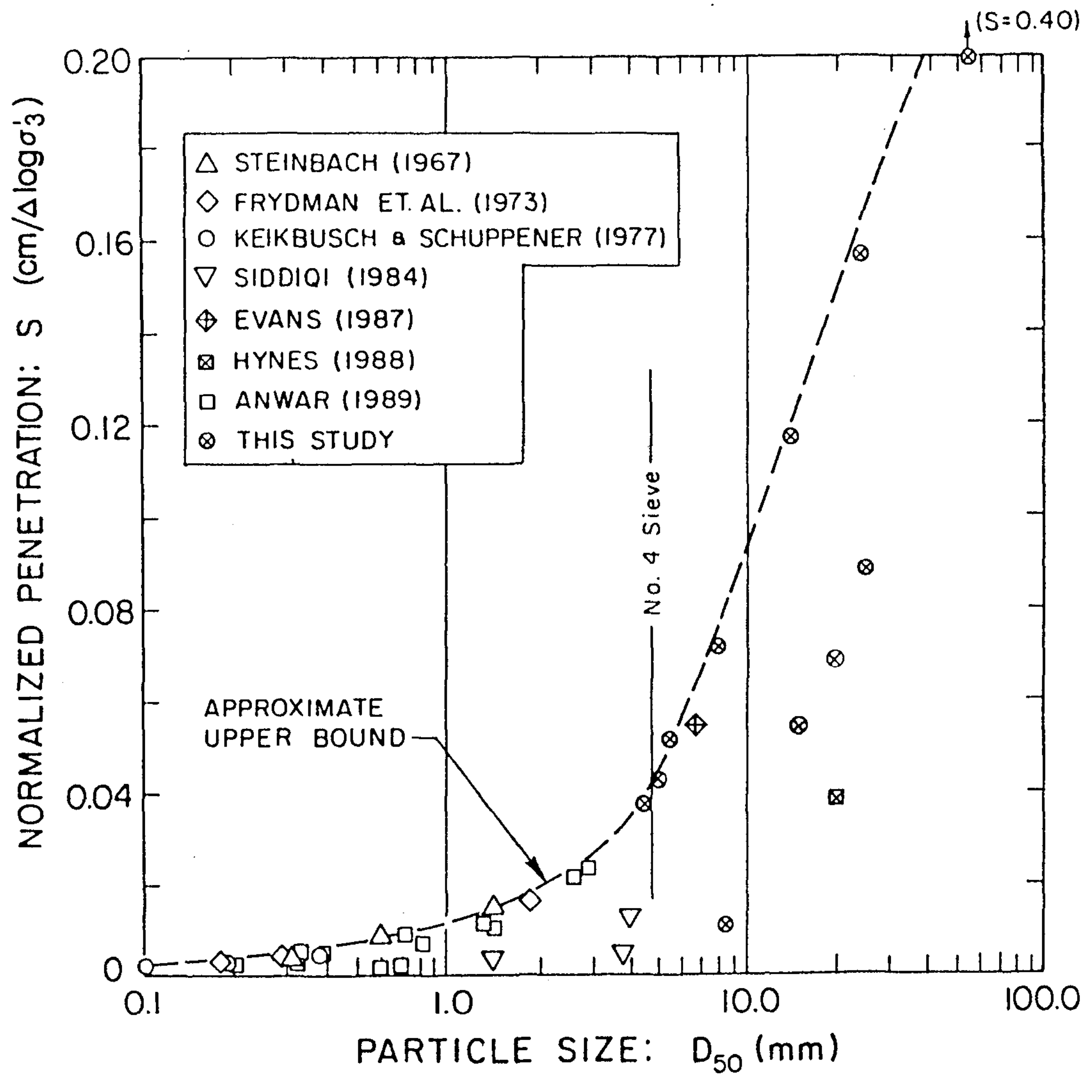


Figure 5.48: Relationship Between Normalized Unit Membrane Penetration (S) and D_{50}

primarily due to the more broadly graded soils which exhibit considerably less penetration than uniformly graded soils due to "finer" particles partially filling the peripheral sample voids between the larger soil grains. A number of particle size indices finer than D_{50} were examined in order to find that which gave the best correlation for all of the available data. It was found from this investigation that D_{20} was the one particle size index with which the membrane penetration characteristics of all different soil types could be best correlated. A composite plot is given in Figure 5.49 showing the relationship between normalized compliance S and D_{20} for all of the same data points that were included in Figure 5.48. This plot shows that by using D_{20} as the "representative" grain size for this wide range of soil types and sizes, a very narrowly banded correlation results demonstrating the significant improvement which can be achieved using the D_{20} grain size for this relationship. The number of data points that have been added to the existing database now gives a clear picture of how the compliance relationship (S vs. D_{20}) should look for soils whose representative grain sizes fall between coarse sand and medium to coarse gravel. This new data fills in the large range of uncertainties which had previously existed.

5.4 Development of a Correlation Between Compliance Characteristics and Material Gradation

An important part of this study was an effort to define a relationship between the normalized unit membrane penetration and material particle sizes for materials whose grain size distributions extended beyond the well documented sand sizes. As previously reported by Seed and Anwar (1986) and Baldi and Nova (1984), the relationship between normalized unit membrane compliance and "representative" grain size does not appear to be strictly log-linear as had been

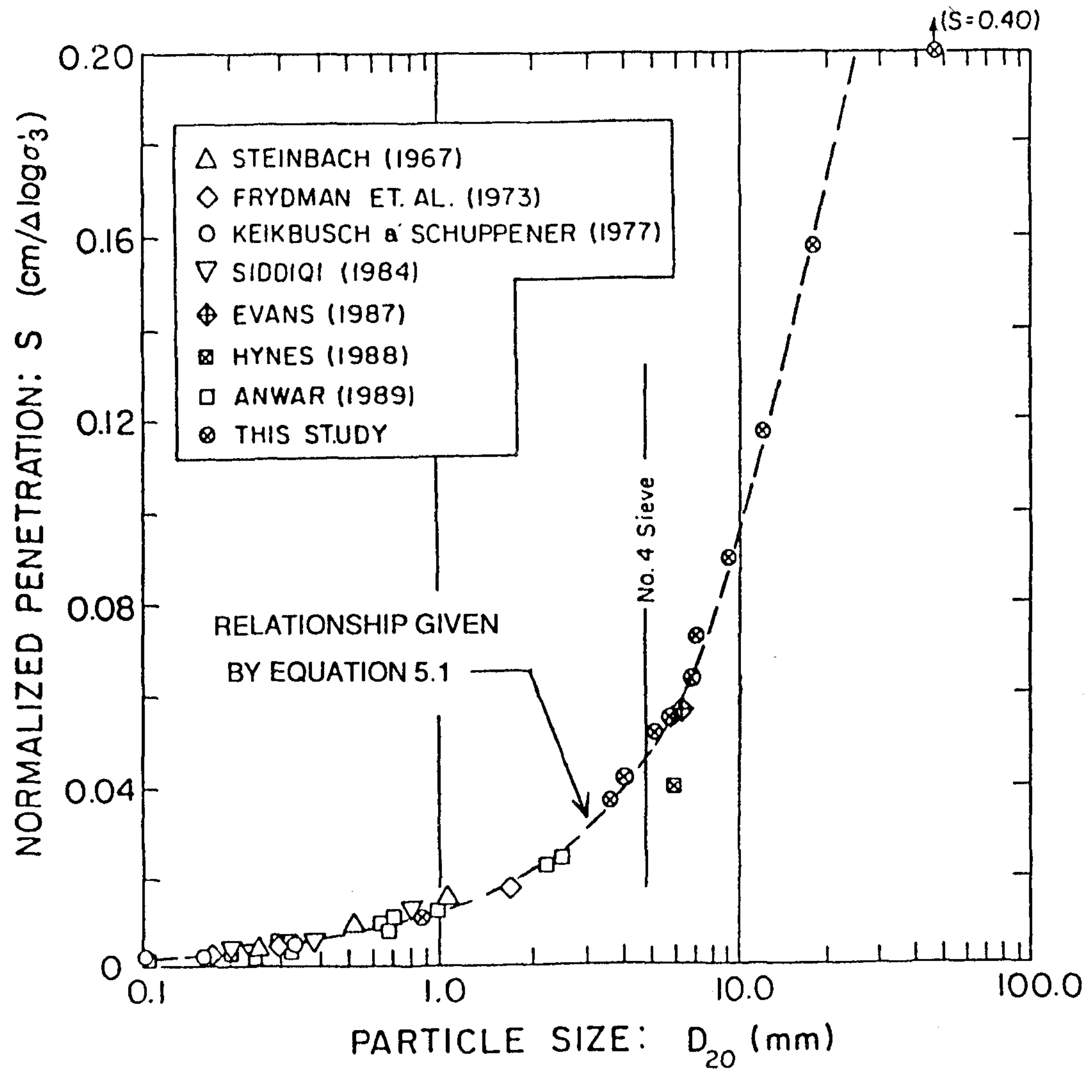


Figure 5.49: Relationship Between Normalized Unit Membrane Penetration (S) and D20

speculated by several earlier investigators, but may be better represented by a curve or multi-log-linear shape.

Seed and Anwar (1986) addressed the question of what part of the gradation curve should be used as a representative grain size for compliance measurement relationships. Historically researchers have always used the median grain size of a material (D_{50}) as the basis for compliance volume relationships. Investigations initialized by Seed and Anwar (1986) and continued as a part of this study have showed that the compliance relationship correlates much better using a finer point of the gradation curves. The particle size indice D_{20} was found to provide the best correlation between material particle size and normalized unit compliance S. Incorporating the available data from previous compliance measurements with those made as a part of this study, an empirical relationship was developed for the database using a polynomial line fitting routine. For all of the data examined, the equation that gave the best "smooth" curve through the data for D_{20} vs. S is:

$$S = 2.029^{3} + 9.248^{3X} - 1.4126\ ^{5X^2} \qquad \text{[Eq. 5.1]}$$

where:

S = Normalized unit compliance; membrane compliance induced volume change per unit area of membrane per log-cycle change in effective confining stress: (cc/cm^2) per log cycle change in σ_3', and

X = D_{20} grain size of a soil in mm.

The curve representing this equation is plotted on Figure 5.49. By using this equation, one would need to know only the gradation of the material in order to make a reasonably accurate estimate of the normalized membrane compliance for any soil. It should be noted that for samples at very high or low relative densities appropriate (though relatively minor) adjustments may have to be made to any estimate made by this relationship. Another possible exception may also be for cases where soils contain an extremely broad gradational "tail" of material finer than D_{20}, as identified by a long tail of the gradation curve. For such soils, normalized unit compliance can be less than that predicted by Eq. 5.1.

CHAPTER 6

IMPLEMENTATION OF A MEMBRANE COMPLIANCE MITIGATION SYSTEM FOR TESTING OF COARSE GRAVELLY SOILS

6.1 Implementation of a Compliance Mitigation System

The technique behind the compliance mitigation method used in this study, originally proposed and tested by Raju and Venkataramana (1980) and Ramana and Raju (1981,1982), is to offset any volume error encountered in undrained tests by injecting to (or removing from) the sample, a volume of water equal to the compliance induced volumetric error. This has been attempted on small scale (2.8-inch diameter) samples in a number of ways. Ramana and Raju attempted this by manual injection using a manually operated system and injecting water at stepwise intervals during tests. Seed and Anwar (1986) used a computer-controlled digital motor driven injector to continuously inject/remove water. Tokimatsu and Nakamura (1986) used a computer-controlled system that incorporated a pneumatic air-pressure system to inject/remove water for these "small scale" tests. The manual system proved to be too slow to be useful, but both of the computer-controlled systems showed promise, and the efficiency and accuracy of the Seed and Anwar system was verified by the tests described in Section 4.2.

A number of attempts were made to control injection with a servo-controlled air pressure system for large-scale samples, but it was found that the air pressure type of system used by Tokimatsu and Nakamura could not efficiently control the relatively large volumes of water that would be necessary to compensate for the membrane compliance errors that would be expected for coarse gravelly soils with sufficient accuracy as to preclude introducing deleterious secondary problems. Of the types of injection methods previously implemented, only the Seed and Anwar type of system appeared to give verifiably continuous and sufficiently accurate (precise) test results utilizing a system suitably adaptable for use with larger scale tests of gravelly materials. The "correct" test results reported by Seed and Anwar using their compliance compensation system were verified by a series of tests performed on the same material using 12-inch diameter samples which exhibited negligible compliance errors for that material, as described previously in Section 4.2.

Accordingly, a large-scale hydraulic piston injector that could be accurately and efficiently controlled by a digital computer program was designed and constructed. The components and implementation of that system is outlined in the following sections.

6.1.1 Components of the Injection-Correction System for Testing of Large-Scale Samples of Coarse Material

6.1.1.1 Compliance Mitigation System Hardware

Figure 6.1 shows a photograph of the computer-controlled injection/removal system that was developed for mitigation of membrane compliance effects during undrained large-scale triaxial testing. A schematic of the injection set-up is shown in Figure 6.2. In addition to the IBM PC-AT microcomputer described previously, the injection/removal system consisted of a series of two hydraulic cylinders mounted on a stiff aluminum channel, so that no relative movement would be permitted between the two. One of the two pistons was driven by a single computer-controlled servo valve. The piston rods of the two cylinders were securely fastened to each other with a threaded compression collar.

The controlling cylinder was a double-acting 1.5-inch I.D. 24-inch stroke heavy duty hydraulic cylinder filled with hydraulic oil under a pressure of

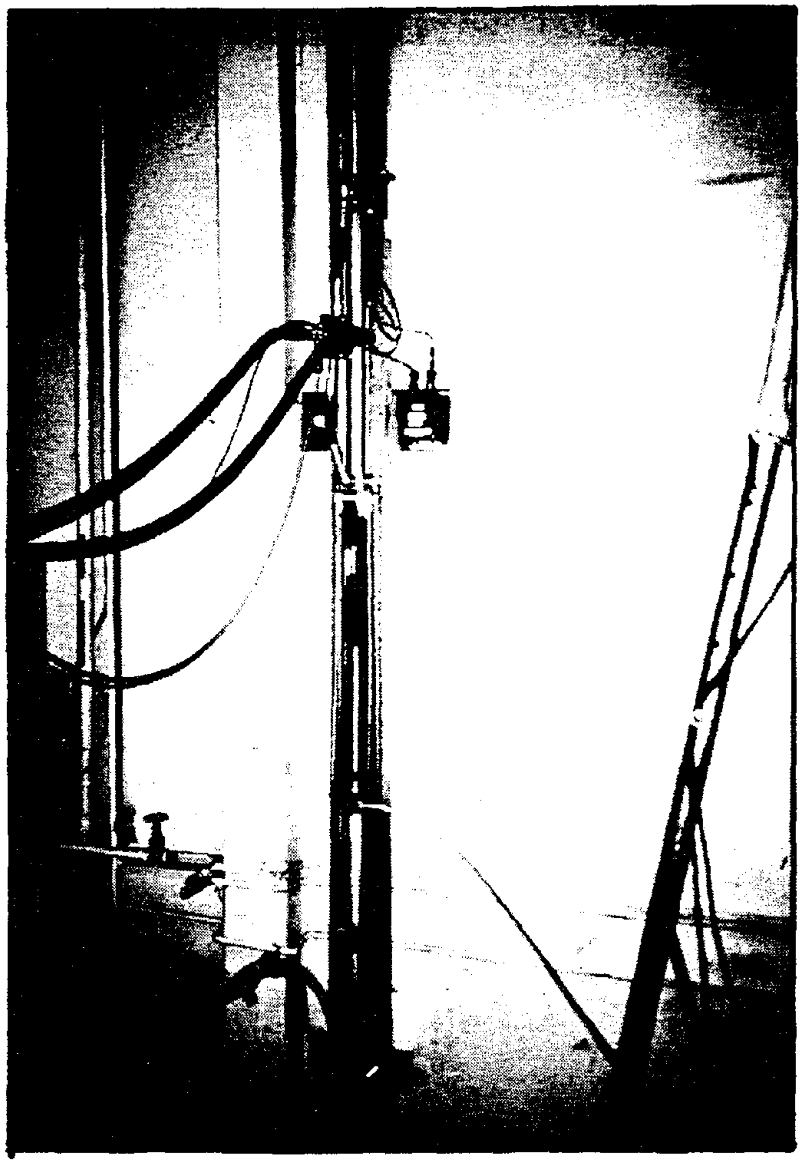


Figure 6.1: Large-Scale Computer-Controlled Injection/Removal System

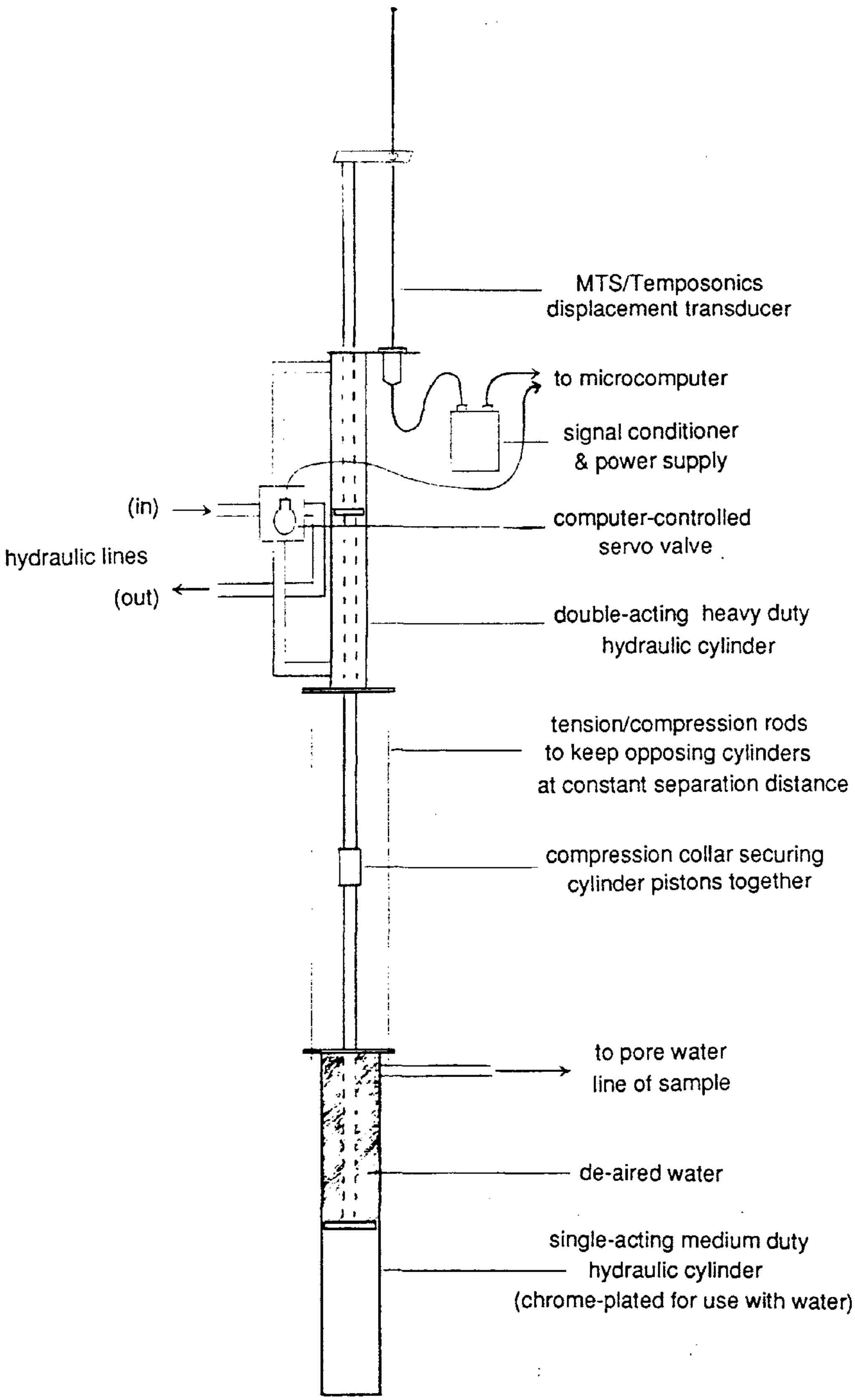


Figure 6.2: Schematic of Large-Scale Injection Set-Up

approximately 2700 psi whose displacement was controlled by the computer-controlled dynamic servo-valve. The motion of the first piston commanded an equal displacement of the piston of the second cylinder, which was a single-acting 2-inch I.D. 24-inch stroke medium duty hydraulic cylinder specially modified with interior chrome plating for use with water. One side of the second cylinder was filled with de-aired water under a pressure equal to that of the back-pressure in the sample at the initiation of the test, and that side of the cylinder holding the water was connected to the pore pressure lines of the sample. Displacements of the cylinder pistons were measured by means of a 24-inch MTS/Temposonics displacement transducer attached to one end of the cylinder configuration with an accuracy of ± 0.004 inches. Injected/removed water volumes were calculated by means of a calibration of the linear displacement of the cylinder pistons with corresponding volumes of water held by the water cylinder. Controlled increments of water volumes could then be injected/removed to/from the sample by allowing specified displacements of the cylinder pistons. The resulting accuracy of the system was found to be ± 0.40 in^3, representing approximately 0.015% of the total soil sample volume tested.

This system requires only a single connection to the sample pore water lines of a triaxial testing system, and could therefore be relatively easily adapted to any existing large scale triaxial system.

6.1.1.2 Compliance Mitigation Program Software

The program implemented to offset the effects of the compliance induced volumetric errors was formulated to run as a part of the ATS system. During the course of this research the "injection program", as it became to be known, was modified a number of times until it satisfied all of the requirements that were demanded of it for a variety of different testing situations. It was important that one single program could be used for all ēncounterable situations and that it be generic enough to be applied to any different configuration of sample sizes and material particle sizes.

A simplified description of the compliance mitigation algorithm of the final version of the program is presented here. At the initiation of the test, readings are recorded for the current effective confining stress and the position of the injection system cylinder pistons. After a short specified time interval, additional readings of effective stress are taken. The new readings of effective stress are then compared to the previously recorded readings. For any change in effective confining stress, the program calculates the volumetric error induced by membrane compliance based on pre-determined values for the material being tested and the sample geometry, and commands the servo-controlled injection cylinder pistons to move accordingly. The loop continues until termination of the test.

There are a number of variable test parameters that are required by the program in order to be able to properly calculate the volumetric errors to be offset. Among these are the S-value or normalized compliance (volumetric error per unit area of membrane per log cycle change in effective confining stress), sample geometry given in appropriately corresponding units, and the minimum effective stress below which no further injection-corrections will be implemented. This last parameter is necessary to adjust for the non-linearity of the semi-log plot of unit compliance as a function of effective confining stress at very low levels of effective stress. Two additional optional testing parameters were also included in the program to overcome possible irregularities and/or potential problems that might be encountered for different materials or testing equipment. These included: (a) an option to change the rate of injection (time interval between runs of the injection control loop), and (b) a specified percentage of the calculated volumetric error to be injected at one time. The objective of these variables was to protect against sudden pressure surges that might be induced to the system from rapid injection of large amounts of water, thereby leading to overcompensation by the injection system. By slowing the rate of the injections, internal pressures were allowed to equilibrate before the next reading of effective stress was taken. Injecting only a portion of the calculated water volume prevented any over-injection at a single point in the test. The full amount or "correct" volume offset was converged to quickly within the next few iterations. Only very subtle uses of these optional variables is advised so as to maintain "complete and continuous" corrections. With some practice, a careful balance between these two optional variables would "smooth" out the injections without sacrificing the accuracy of the corrections.

One further addition to the software that was written into the program was to average a number of very rapid effective stress readings to "smooth" or eliminate any values of spikes or valleys from the pore-pressure transducer readings which would otherwise result from water-hammer pulses caused by rapid injection surges into a closed test system.

6.1.2 Injection System Specifications

The accuracy to which the system could inject or remove volumes of water to/from the sample, was to within 2cc. The maximum rate at which accurately controlled injections could be performed was on the order of 100cc per 0.5sec. The speed of the injection software loop could be run as fast as once every 3 milliseconds. It was this high degree of accuracy, and the rapid correction loop speed possibilities, that enabled the system to be able to completely and continuously mitigate the effects of membrane compliance throughout undrained testing.

6.2 Tests Performed on 12 Inch Diameter Samples

6.2.1 General

The results of the tests performed on 12-inch diameter specimens as a part of this research program are presented in this chapter. Test data is categorized into groups depending on test type (e.g., monotonic or cyclic loading), and whether or not the injection-correction system was used. Four series of tests were performed: strain-controlled undrained monotonic triaxial load tests performed with and without the effects of membrane compliance mitigated; and stress-controlled undrained cyclic triaxial tests preformed with and without the effects of membrane compliance eliminated by the injection-correction system.

6.2.2 Material Tested

The material used for the comparative tests in this study was a uniformly-graded medium gravel whose grain size distribution is shown in Figure 6.3. A photograph of this material is also shown in Figure 6.4. This material (termed PT-Gravel) was chosen for a number of reasons including its availability and because its characteristic properties were such that some anticipated problems might be avoided. The grains were typically sub-angular to sub-rounded, which was helpful in avoiding tearing of the confining membranes during compaction or testing, and allowed ease in identifying whether or not individual grains were broken during sample preparation or testing. The maximum particle size of this material (1.5 inches) fell well within the limits recommended for testing of these sample dimensions (12-inch dia.), where it has been recommended that a ratio of specimen diameter to maximum grain size be no greater than 6:1 for uniformly graded soils to avoid stress concentration problems that might otherwise invalidate test results. The gradation was coarse enough to provided large enough amounts of volumetric compliance so that compliance effects would significantly affect undrained test

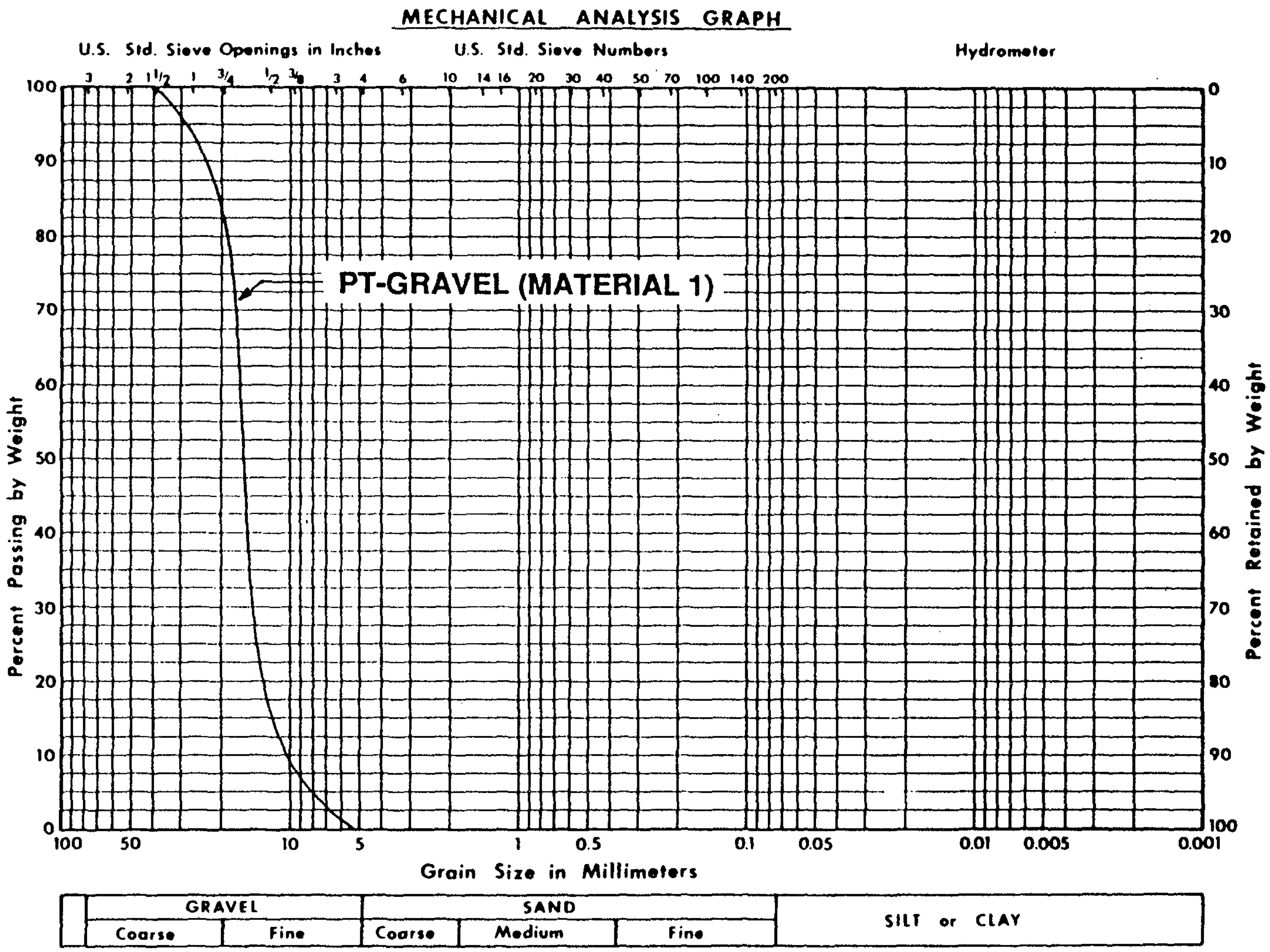


Figure 6.3: Gradation for PT-Gravel Used for Comparative Large-Scale Tests

Figure 6.4: Photograph of PT-Gravel

results, and to assure that membrane thickness would be insignificant in affecting membrane compliance. The normalized compliance, or S-value of this material was approximately 0.117 cc/cm^2 per log cycle change in effective confining stress, which was in the middle of the range of the gravels for which compliance measurements were made, and also towards the middle of the range of gravel sizes best suited for the large scale testing apparatus. This amount of volumetric compliance was also large enough so that a high degree of accuracy could be maintained in compliance measurements and water volumes to be injected. Because of these reasons, the material was chosen for these tests as it seemed to be an average, representative material for testing in this size equipment.

6.2.3 Determination of Maximum & Minimum Density for Coarse Soils

The difficulty in evaluating the actual maximum and minimum density values for cohesionless soils has long been recognized. Some of the problems associated with these density determinations become even more evident when dealing with soils so coarse that "true" density values tend to be underestimated due to a lesser volume of soil at the edges of reasonably sized samples.

The gravel used for most of this study gave a wide range of values for minimum density varying with sample size. The variation of measured densities over the range of area/volume (A/V) ratios showed that as sample size is increased, and the ratio of surficial area to sample volume decreases, the deleterious effect of container edge volume errors on soil density determinations is reduced. Using the density values obtained over a range of A/V ratios, true maximum and minimum density values were extrapolated. To report accurate density values at which individual tests were conducted, measured density values needed to be interpreted by means of comparison of the tested sample geometries to curves representing variation of density with A/V ratios. Such curves were developed for the variation of maximum and minimum densities of the PT-gravel for a range of A/V ratios using the ASTM-D standard "large" compaction mold and the 12-inch diameter sample preparation mold used for large-scale tests. These curves are presented in Figure 6.5. In addition to the density tests performed for developing these density "correction" curves, large-scale vibration table tests were also performed, both wet and dry, in order to get a second evaluation of maximum density for the material. The results of the vibration table density tests agreed very well with those values determined by vibrating individual layers in the 12-inch diameter sample preparation mold.

Samples for minimum density tests were prepared by pluviation through standing water and hand placement of grains (previously suggested by Evans and Seed, 1987), but the "wet pluviation" method appeared to give more consistent and lower density values and were therefore ultimately used to determine minimum density. The values reported for maximum and minimum density of the PT-Gravel were 110.0 pcf and 92.2 pcf respectively.

6.3 Test Results

Four series of tests were performed on the PT-Gravel material. Undrained monotonic load tests ($\overline{\text{IC-U}}$) were performed at controlled strain rates so that a clear record of peak and residual strengths and pore water pressures could be collected. One series of tests were performed with the use of the injection-mitigation system, and one without. A series of undrained cyclic strength tests were performed on samples at approximately 50% relative density, to obtain a cyclic strength curve for the material without corrections for compliance errors. A similar series of cyclic tests were performed on samples identically prepared with the injection-correction system implemented.

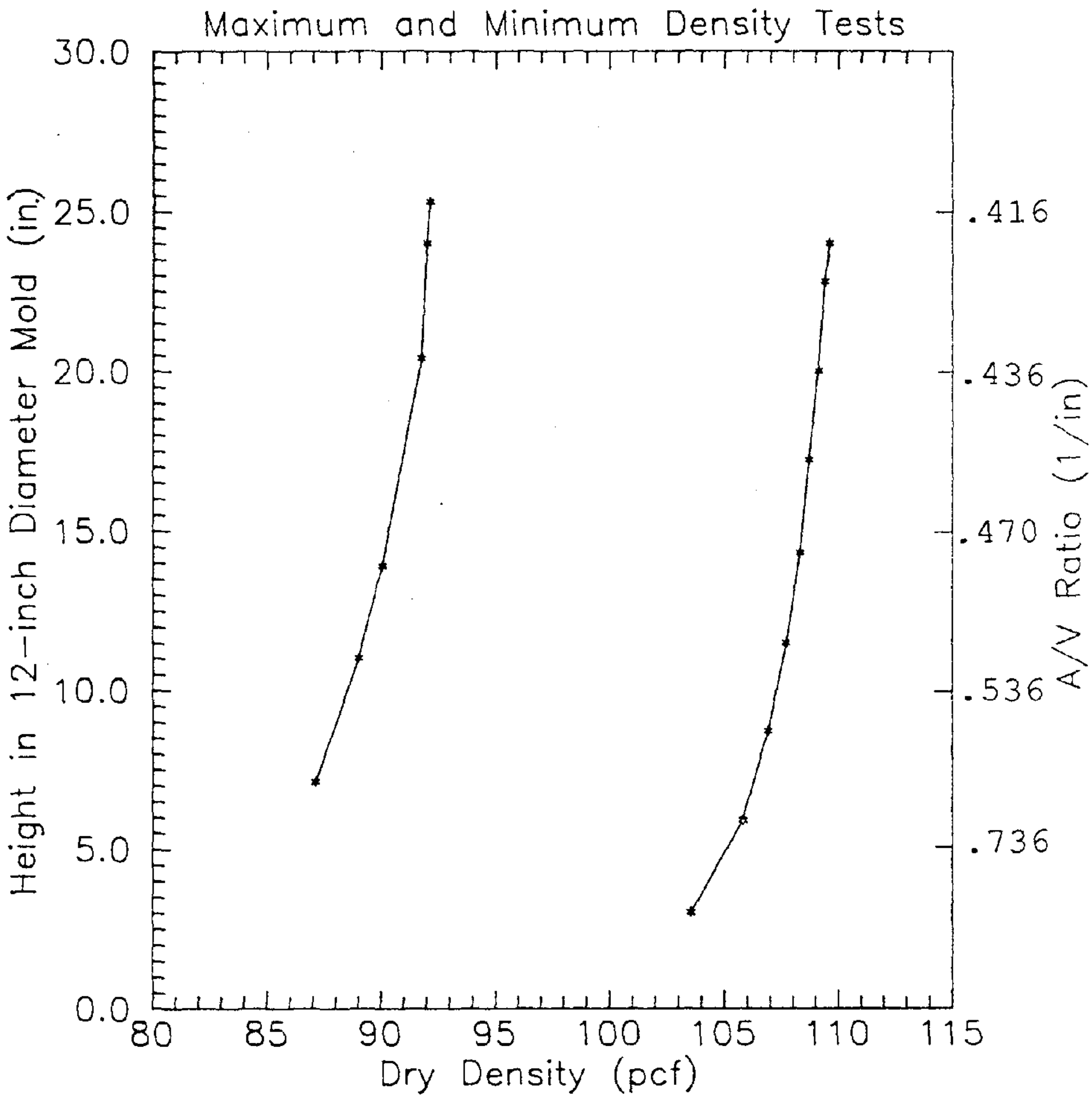


Figure 6.5: Relationship Between Measured Relative Density and Sample Size for PT-Gravel

6.3.1 Results of Undrained Static Load Tests

Table 6.1 presents the results of the undrained static load triaxial tests performed on 12-inch diameter samples of PT-Gravel with and without the use of the injection-mitigation system. The results of the individual tests are given in Figures 6.6 through 6.18. All static load tests were performed as "strain-controlled" tests at a constant axial strain rate of 0.25% per minute. All loads reported for these tests were the deviatoric stresses applied to the samples in excess of the isotropic consolidation/confining stresses. These loads were corrected to represent assumed changes in sample areas during loading based on assumed "right-cylindrical" deformation. Testing was discontinued when a minimum of 12% axial strain was reached. It was found that at significantly greater strain values, an appreciable amount of grain breakage occurred and the membranes had a tendency to tear thus making testing to greater strains of little practical value.

It is interesting to note that for some of the earlier tests in which the injection was unsteady and over-corrected during the initial stages of the tests (e.g. PT-42 and PT-46), the final test results (pore-pressure and deviator stress values at 12% ϵ_a) were essentially equal to those obtained from tests performed for which the injections were "smoothed out". This implies that "correct" steady-state or critical state (residual strength) $\overline{\text{IC-U}}$ test results can be obtained even with a "rough" injection system, as long as the accuracy of the injection correction is adequate once the system has stabilized.

A composite of the results from the two series of static $\overline{\text{IC-U}}$ tests are plotted in Figure 6.19, showing the differences in slope between the uncorrected and "correct" steady state lines for the test material. As expected, for samples whose densities were below critical state, the contractive behavior of those samples generated positive pore-pressures which reduced membrane penetration and

Table 6.1: Testing Conditions: $\overline{\text{IC-U}}$ Triaxial Tests on PT-Gravel With and Without Membrane Compliance Mitigation

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,i}$ (ksc)	B-Value	$\sigma_{d,f}$ (ksc)	$\sigma'_{3,f}$ (ksc)
PT-54	24	No	2	0.981	5.3	1.51
PT-57	42	No	2	0.979	5.9	1.79
PT-56	54	No	2	0.976	7.25	1.94
PT-66	80	No	2	0.980	8.7	2.2
PT-55	94	No	2	0.976	9.8	2.35
PT-44	17.5	Yes	2	0.980	4.0	1.15
PT-45	38	Yes	2	0.971	5.4	1.58
PT-42	48.5	Yes	2	0.976	6.3	1.78
PT-69	49	Yes	2	0.980	6.4	1.8
PT-47	67.5	Yes	2	0.980	8.25	2.1
PT-64	80	Yes	2	0.979	9.75	2.41
PT-68	95	Yes	2	0.978	11.25	2.75
PT-46	97	Yes	2	0.972	11.0	2.80

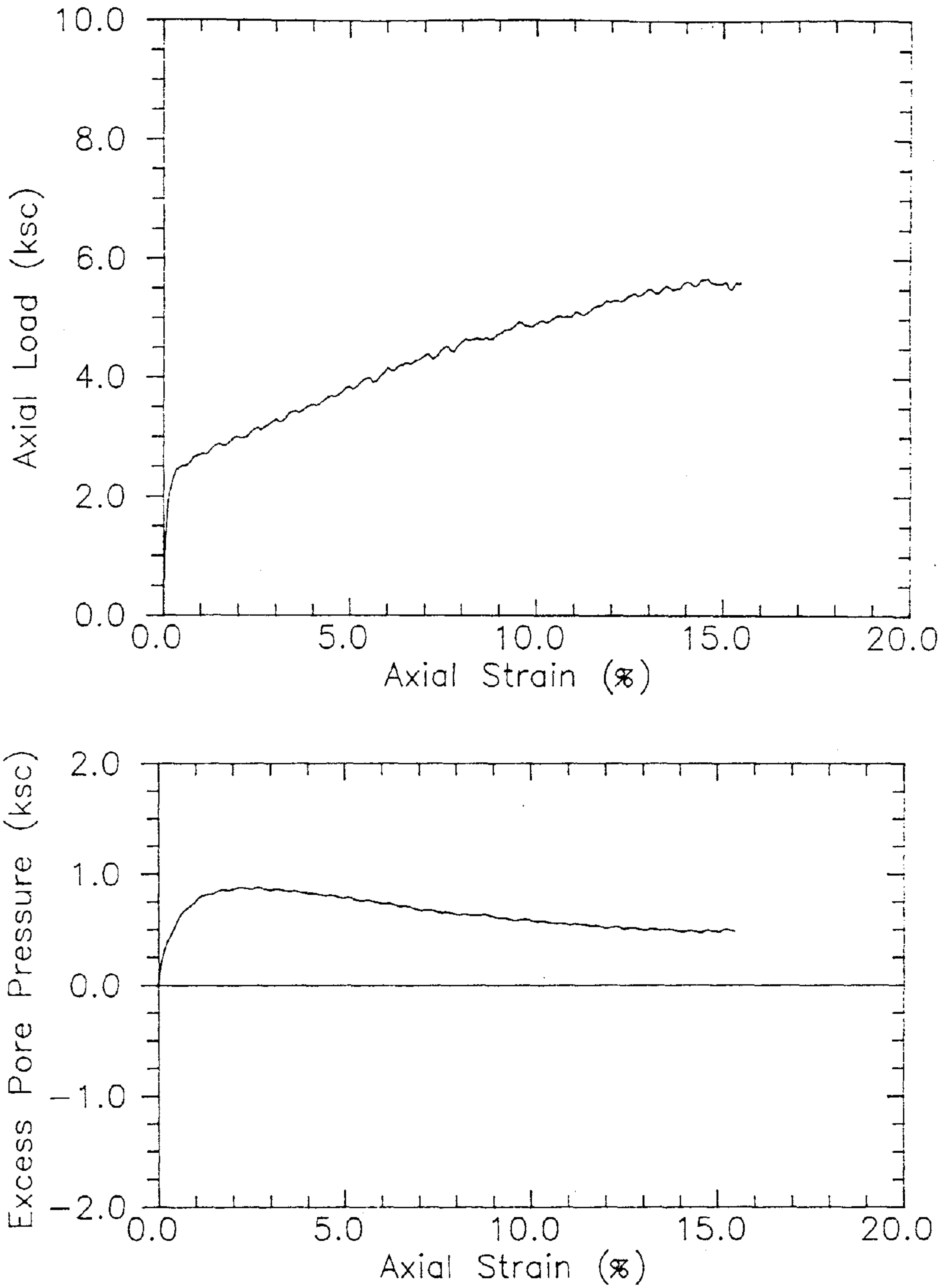


Figure 6.6: IC-U Triaxial Test No. PT-54 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 24\%$)

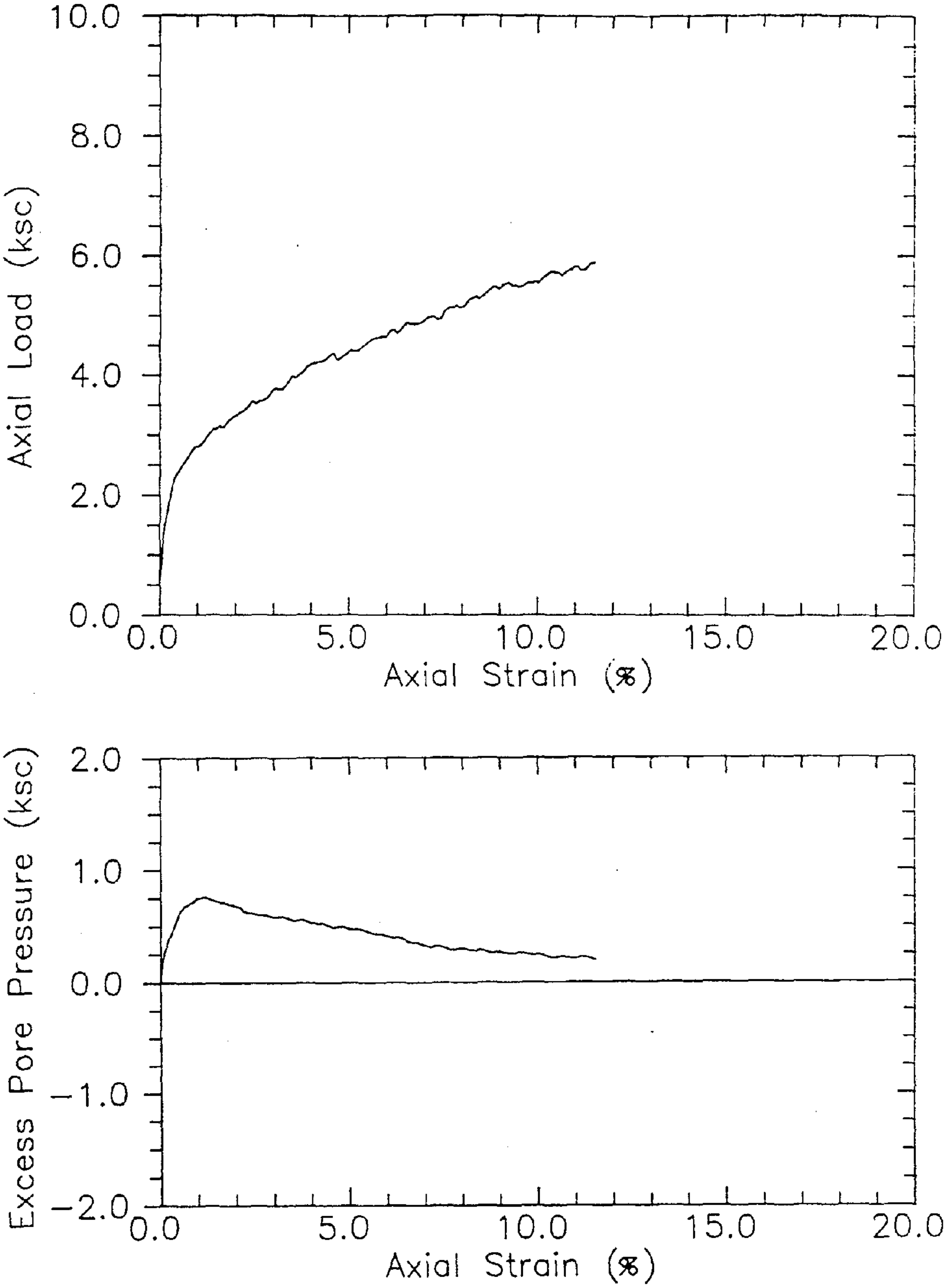


Figure 6.7: $\overline{\text{IC}}$-U Triaxial Test No. PT-57 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 42\%$)

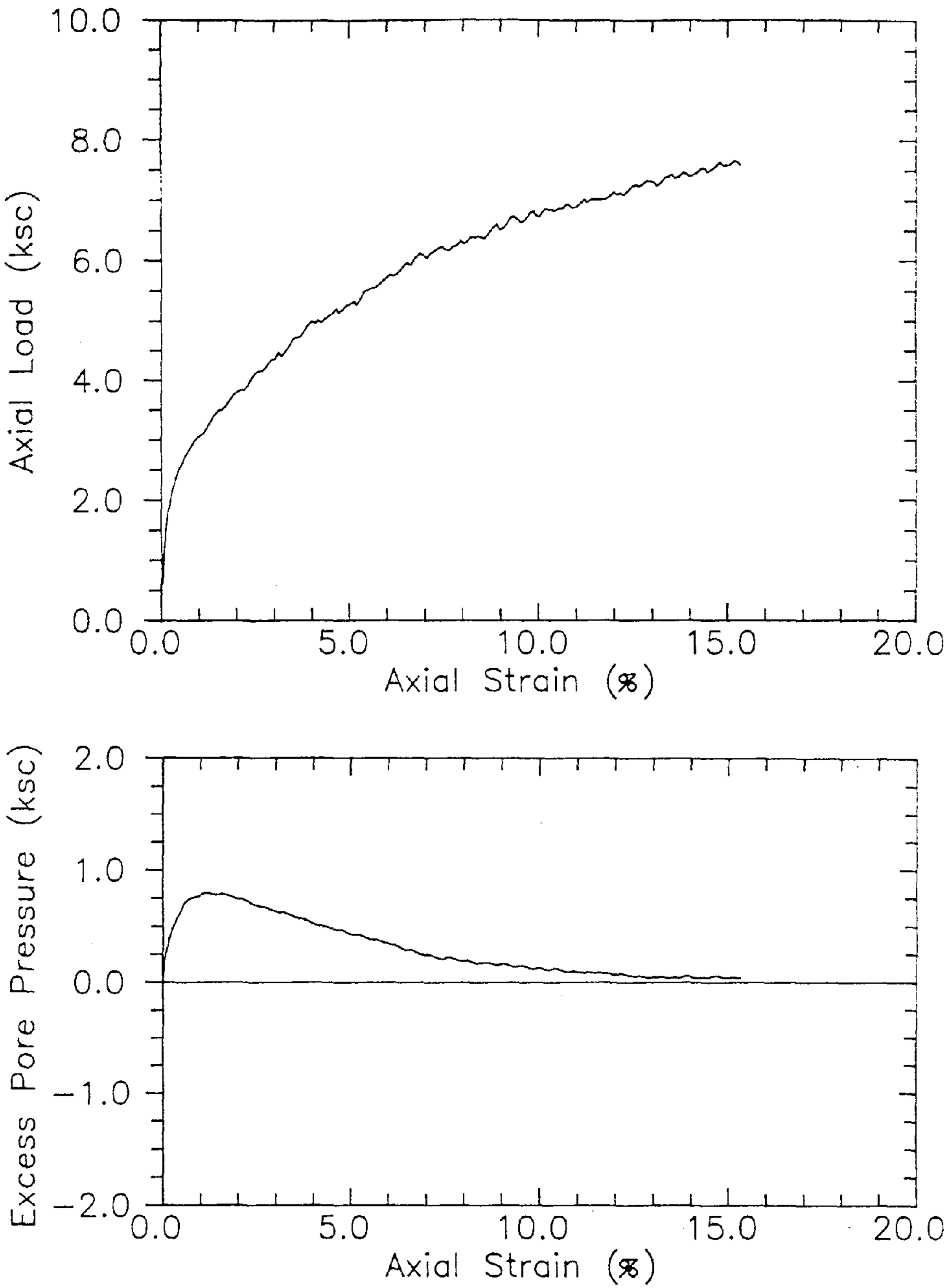
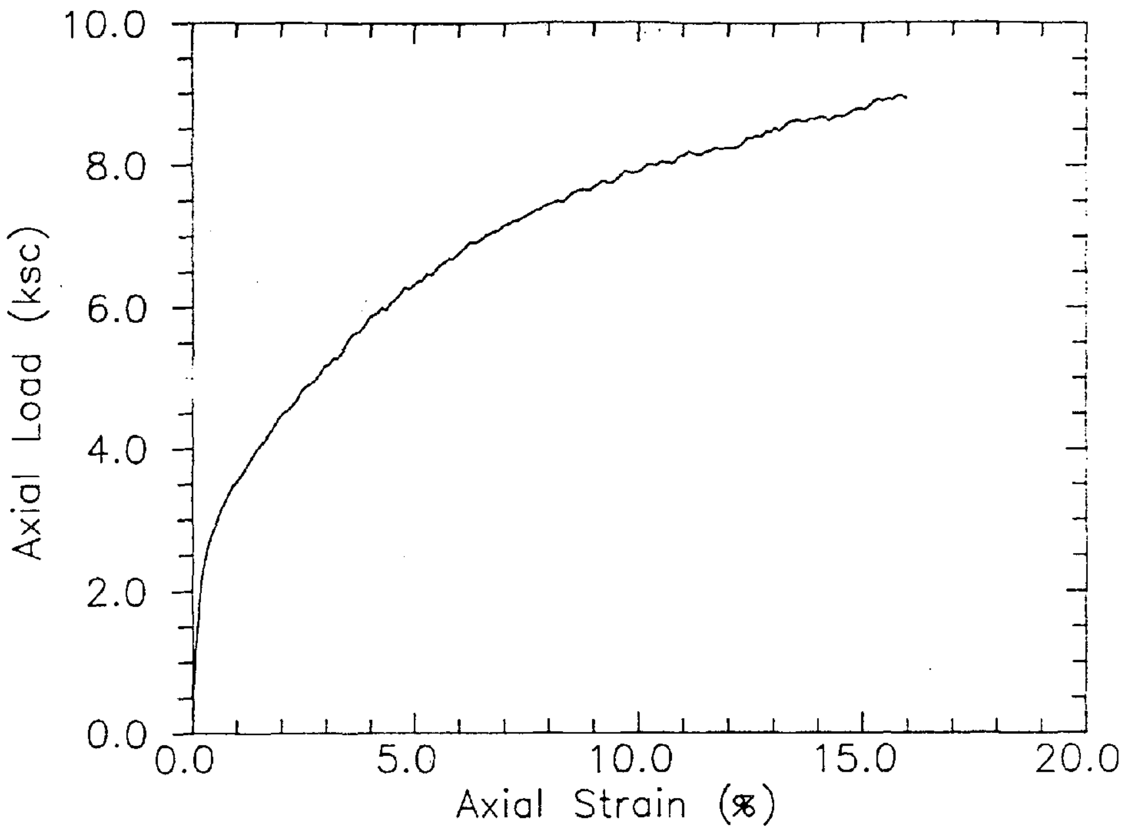


Figure 6.8: $\overline{\text{IC-U}}$ Triaxial Test No. PT-56 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 54\%$)



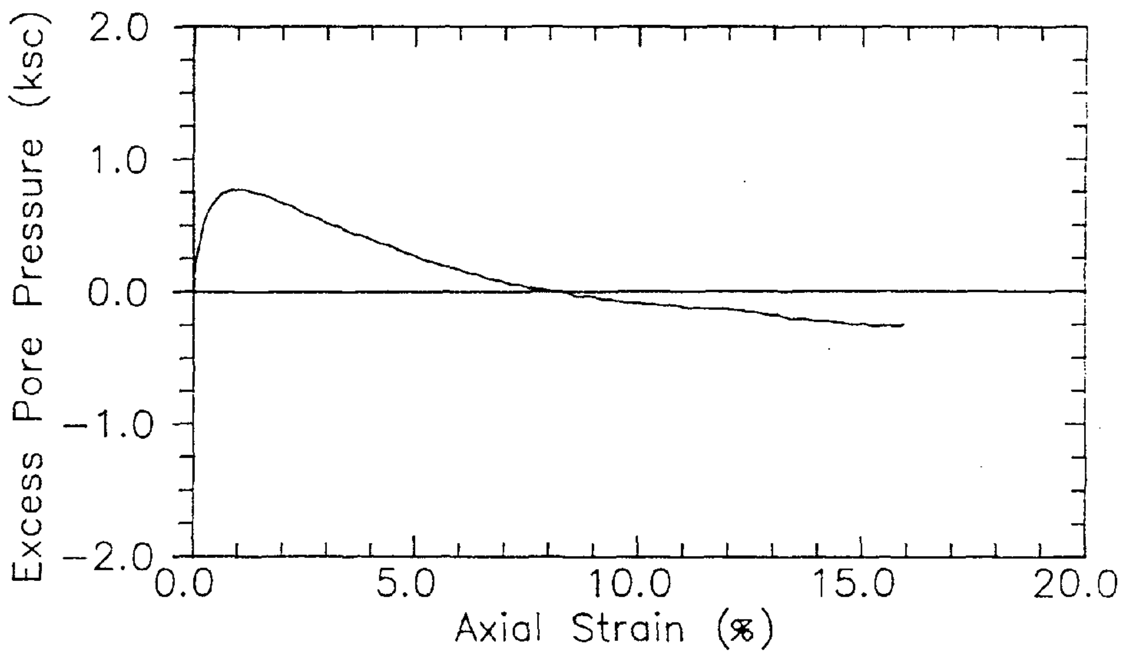


Figure 6.9: IC-U Triaxial Test No. PT-66 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 80\%$)

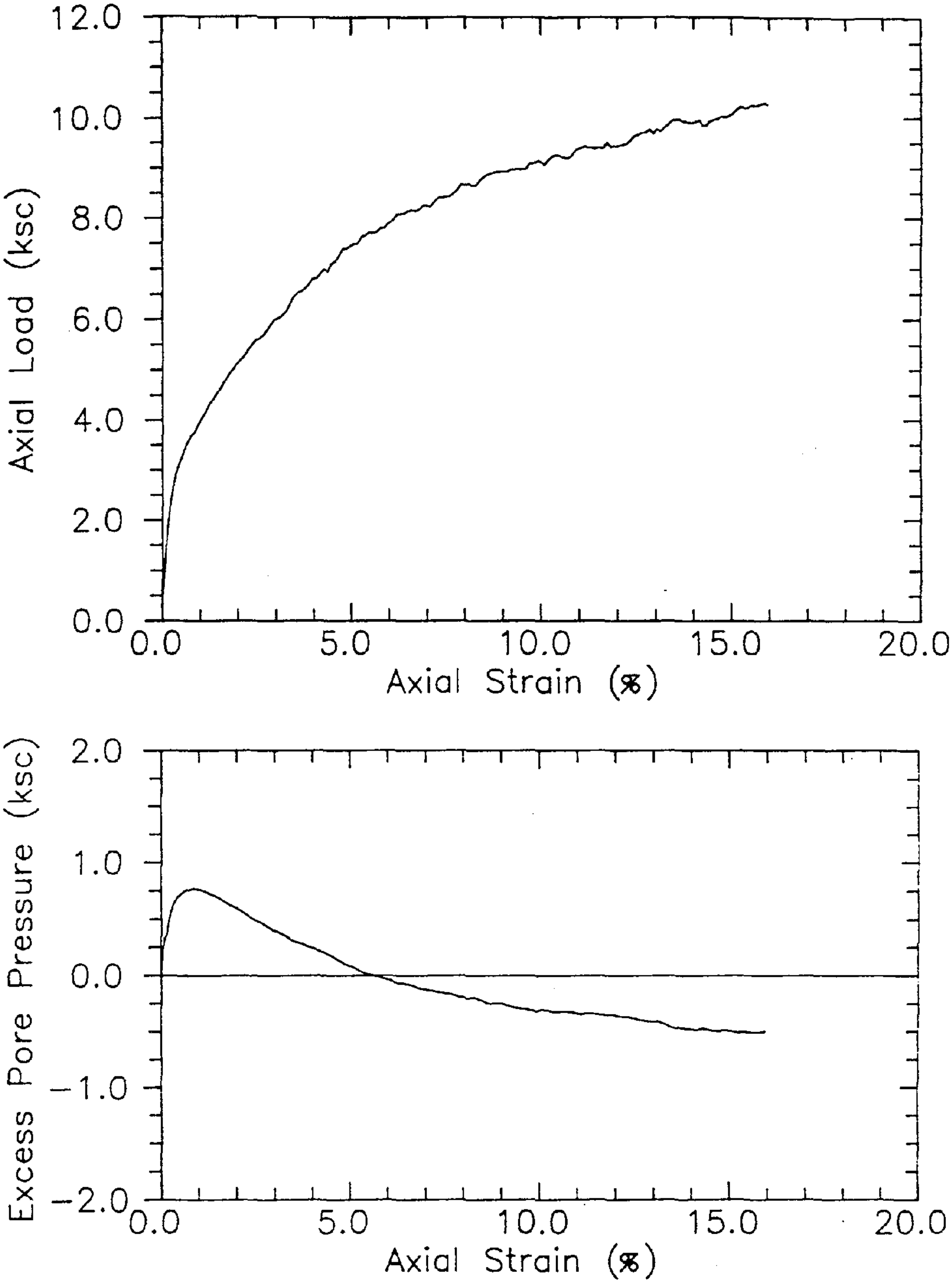


Figure 6.10: IC-U Triaxial Test No. PT-55 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 94\%$)

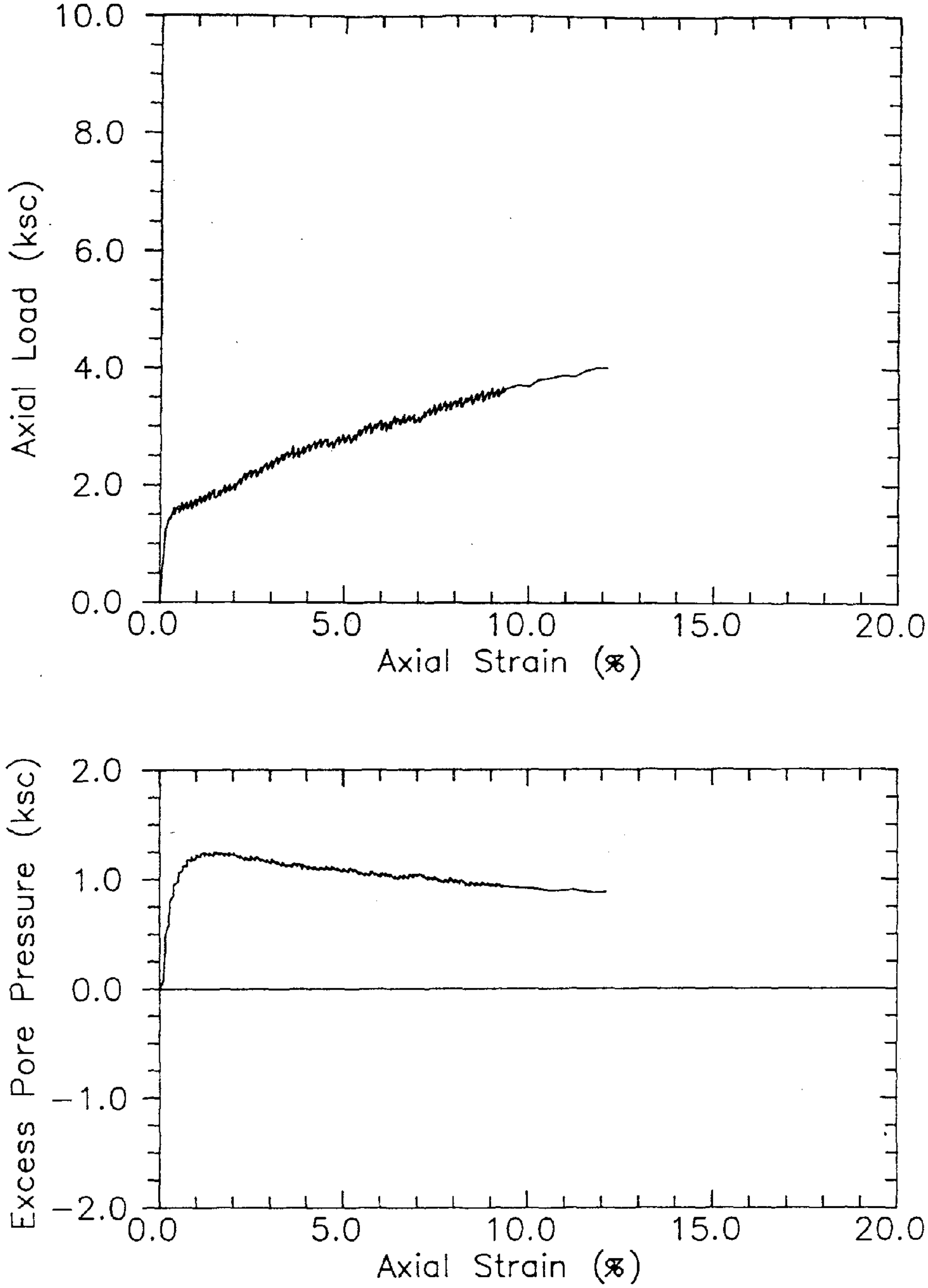


Figure 6.11: $\overline{\text{IC-U}}$ Triaxial Test No. PT-44 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 17.5\%$)

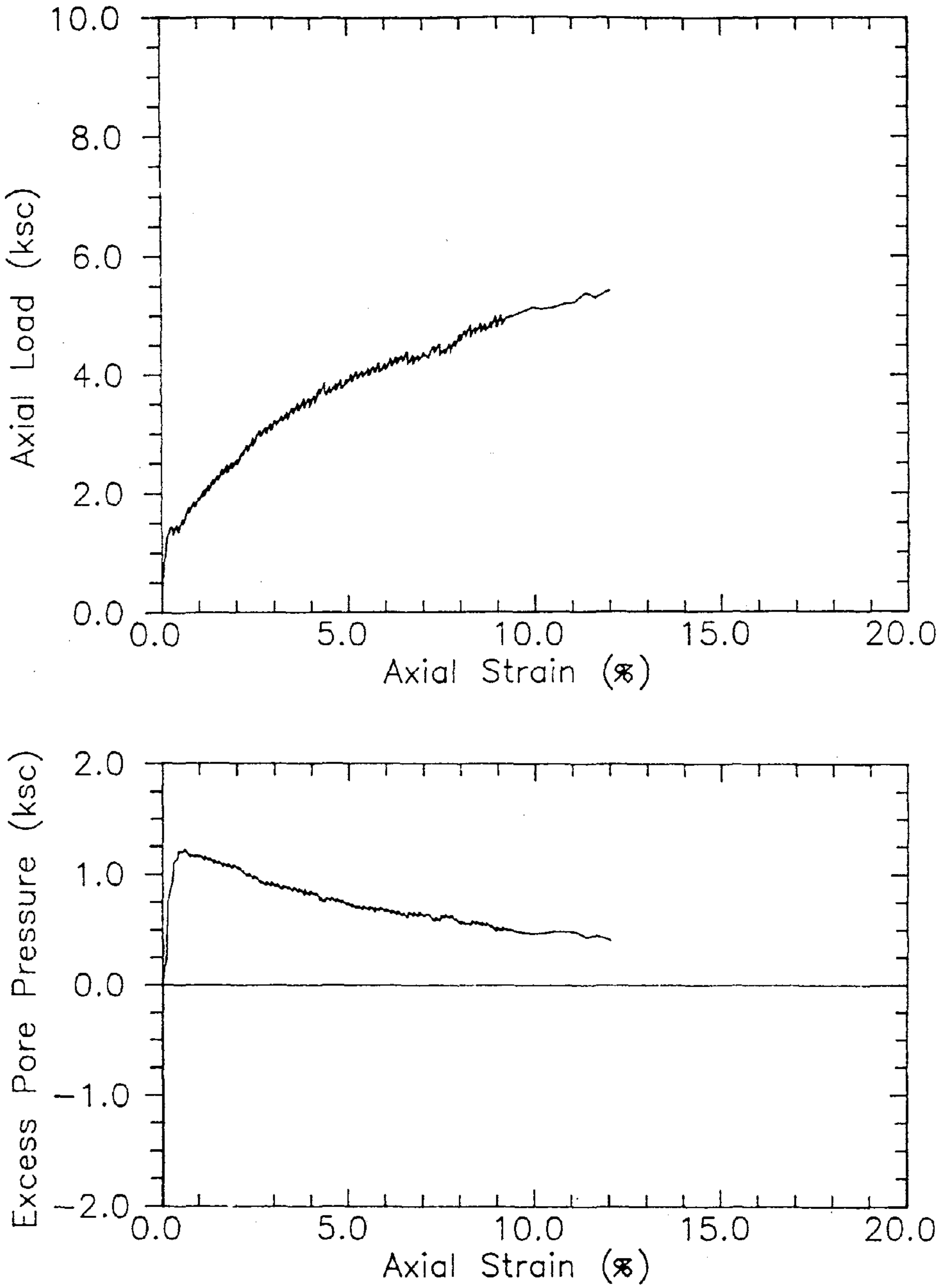


Figure 6.12: $\overline{\text{IC-U}}$ Triaxial Test No. PT-45 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 38\%$)

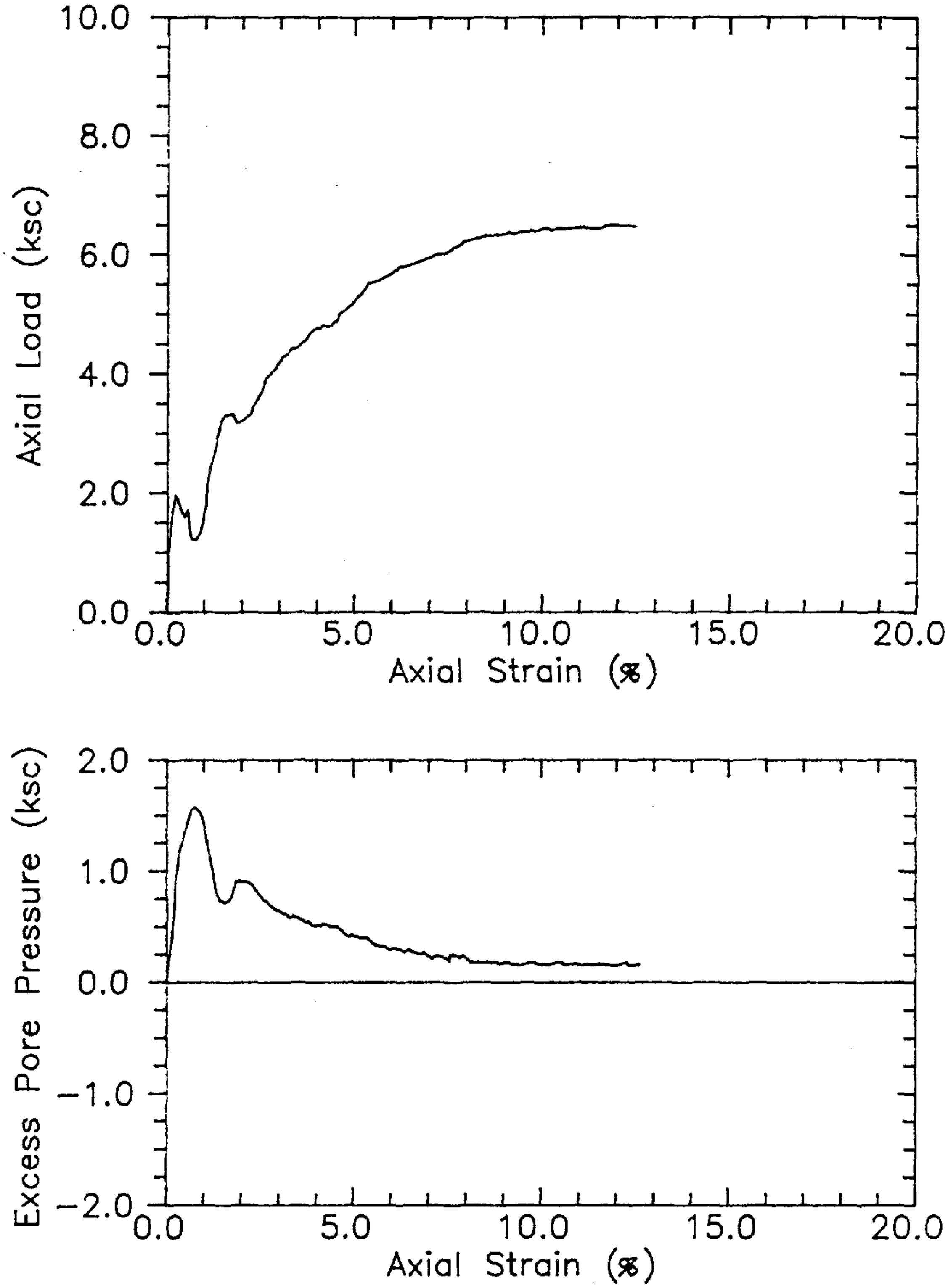


Figure 6.13: $\overline{\text{IC}}$-U Triaxial Test No. PT-42 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 48.5\%$)

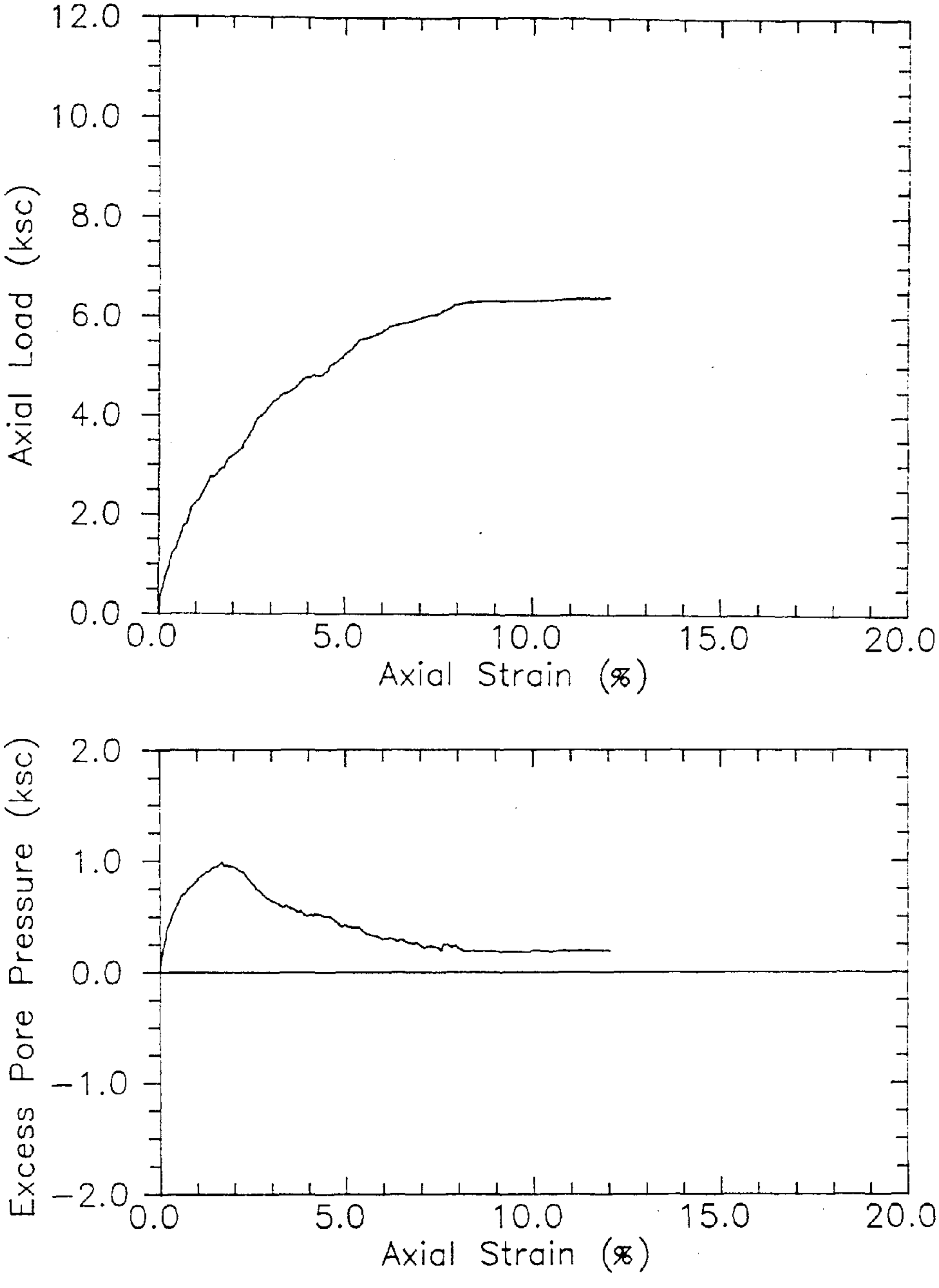


Figure 6.14: $\overline{\text{IC}}$-U Triaxial Test No. PT-69 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 49\%$)

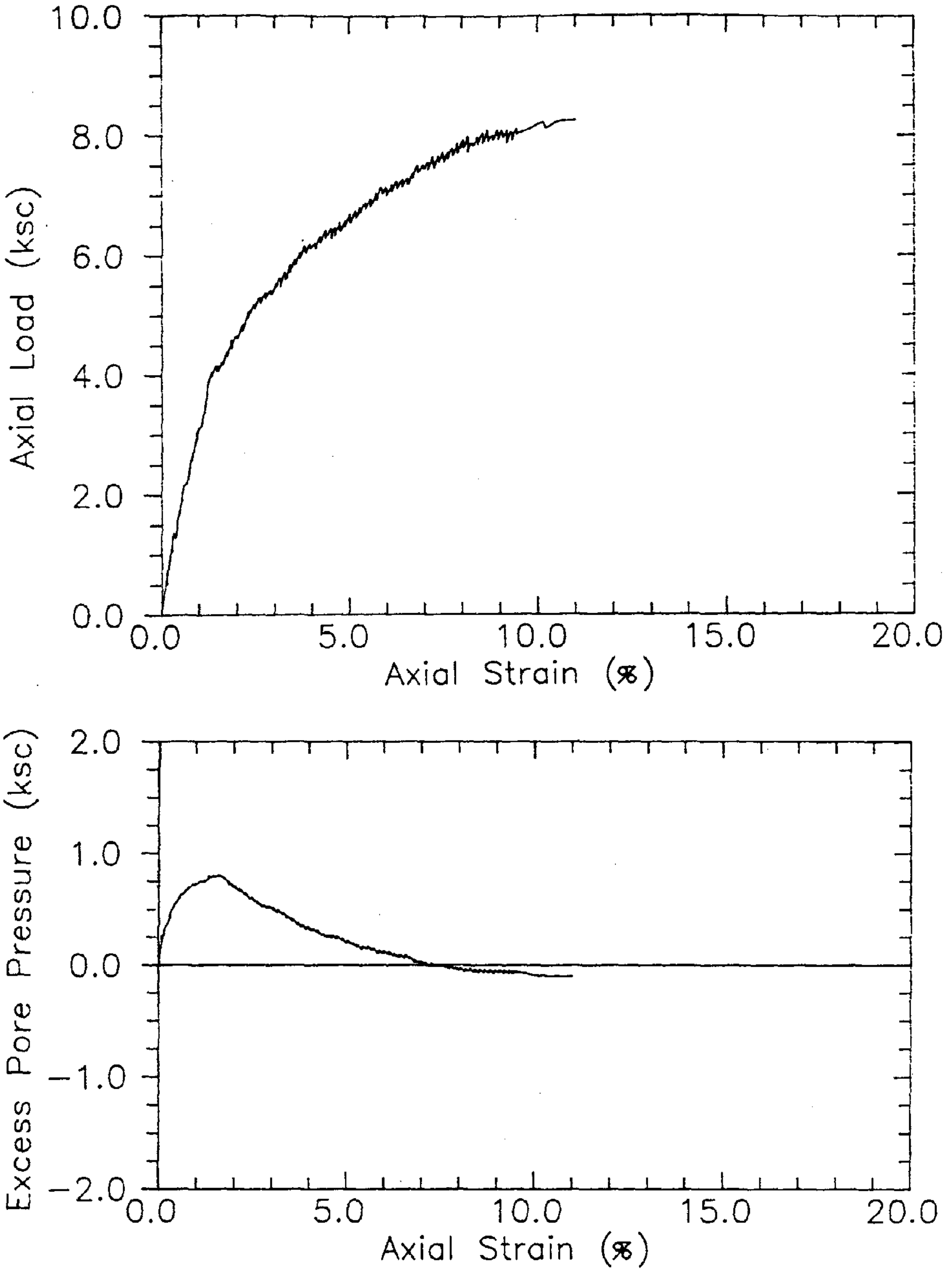


Figure 6.15: IC-U Triaxial Test No. PT-47 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 67.5\%$)

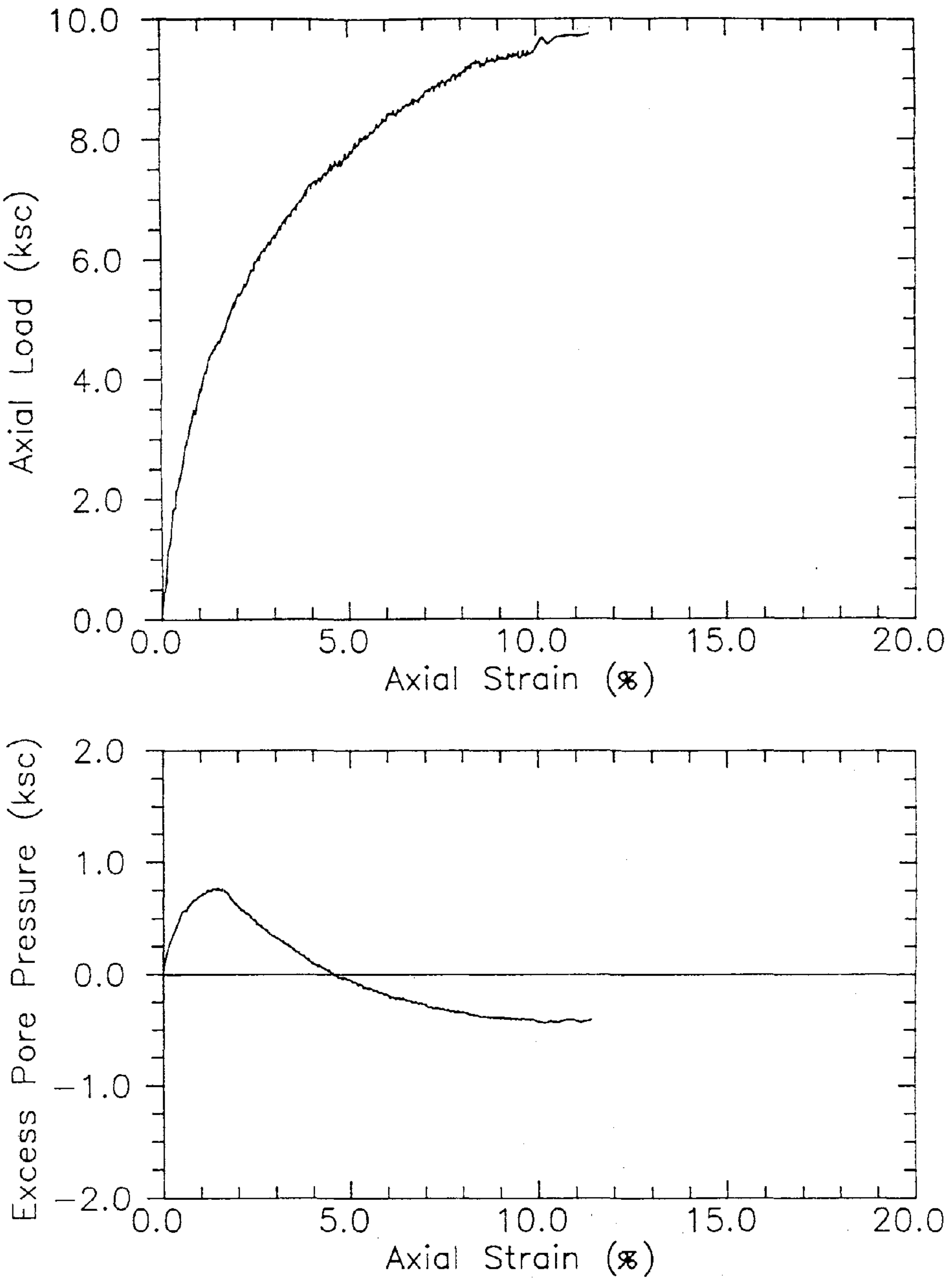


Figure 6.16: $\overline{IC}$-U Triaxial Test No. PT-64 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 80\%$)

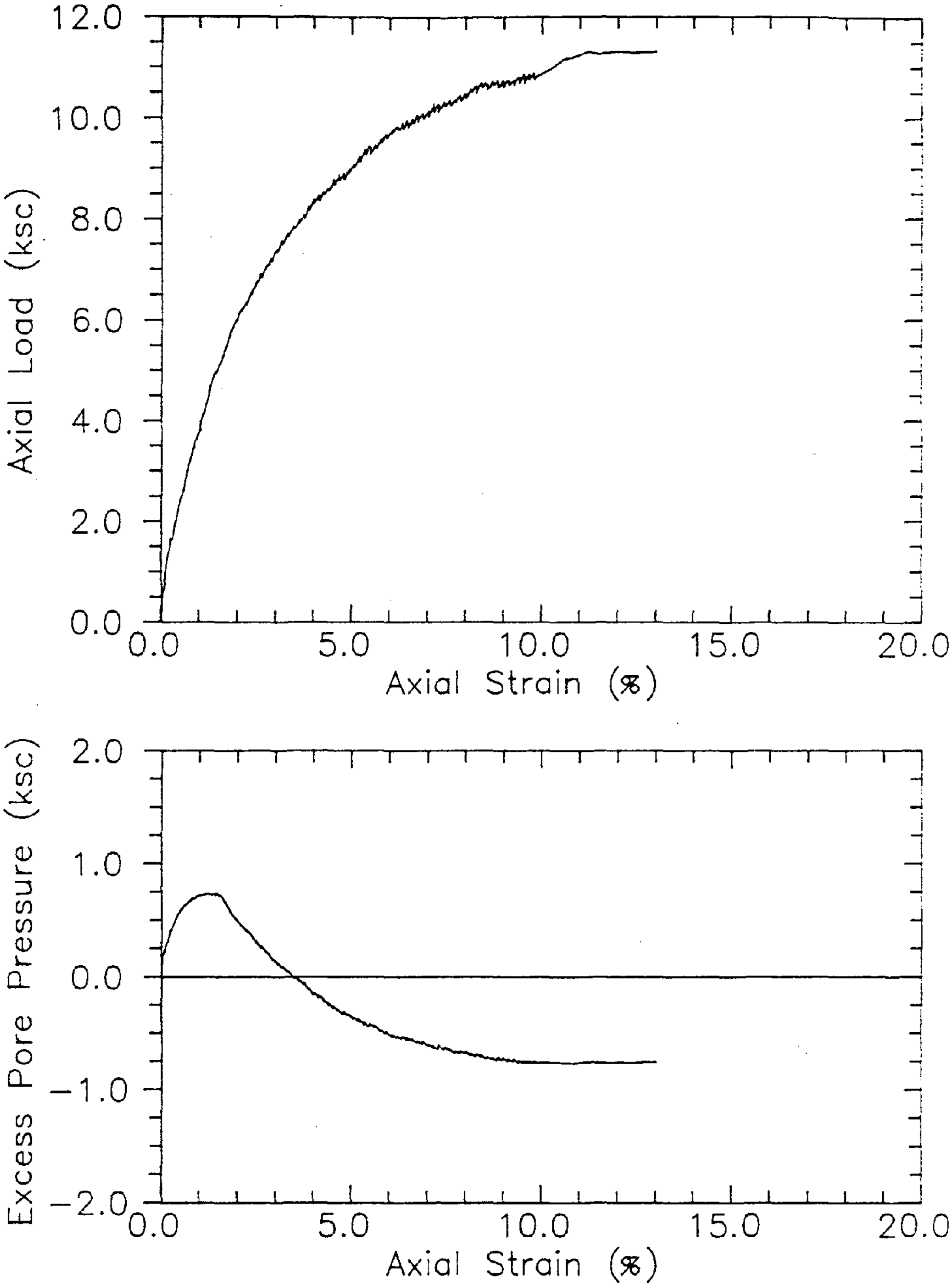
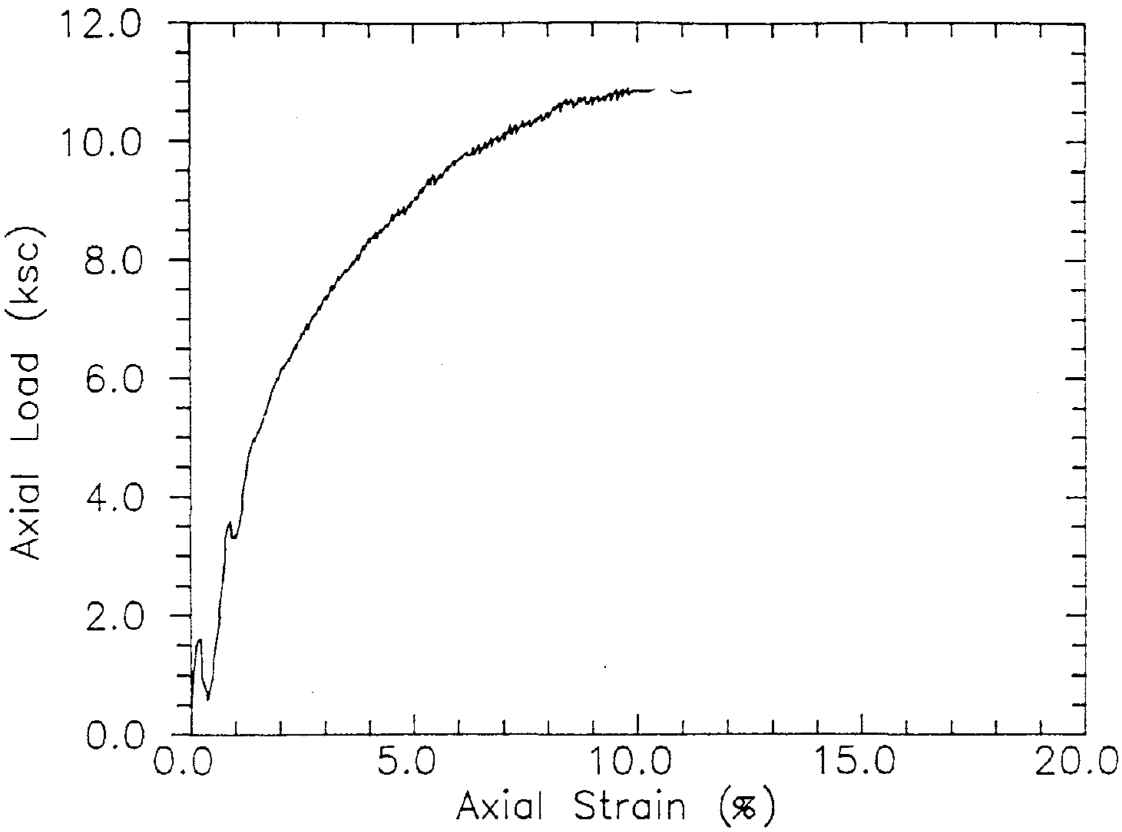


Figure 6.17: IC-U Triaxial Test No. PT-68 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 95\%$)



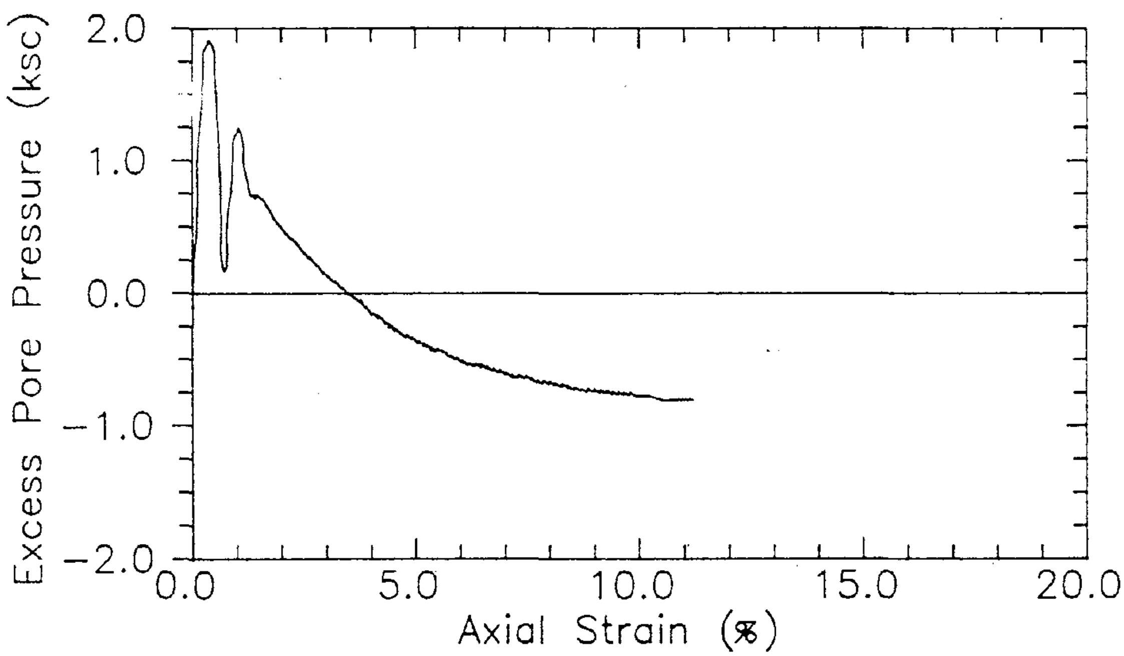


Figure 6.18: $\overline{\text{IC}}$-U Triaxial Test No. PT-46 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 97\%$)

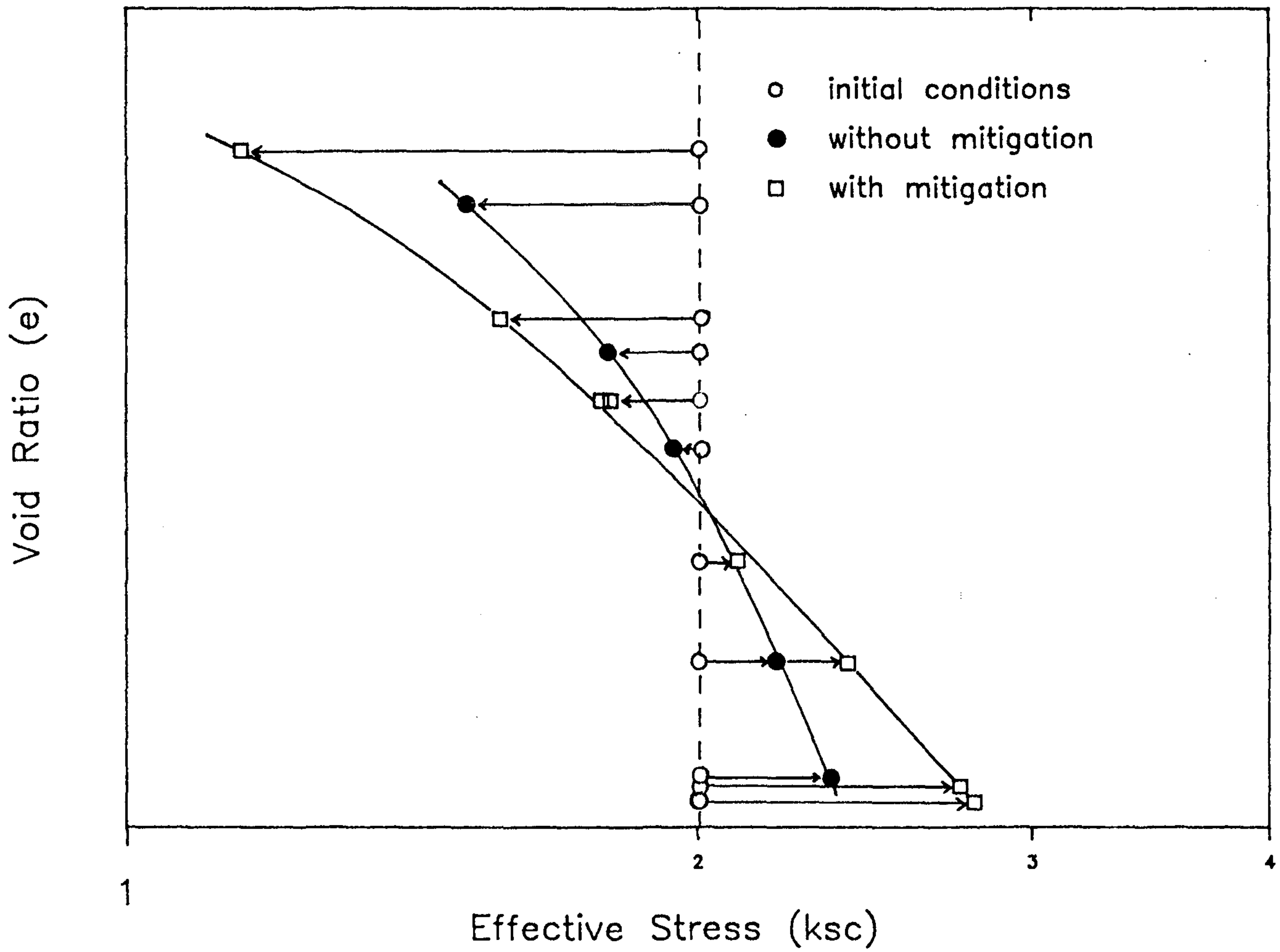


Figure 6.19: <u>Critical-State Plot for $\overline{\text{IC}}$-U Triaxial Tests on PT-Gravel With and Without Membrane Compliance Mitigation</u>

therefore showed residual effective stresses higher than they should, thus representing unconservatively higher static strengths. $\overline{\text{IC}}$-U tests performed with the injection compliance-mitigation system reduced the residual strengths for the "loose" samples to more representative values. For samples that were constructed at densities higher than critical state the opposite was true. As the "dense" samples without compliance mitigation dilated, negative pore-pressures were generated for which the mitigation system removed water from the samples. The results of the uncorrected tests was to generate overconservative strength evaluations, as higher residual strengths were recorded in the corrected tests.

The importance of these findings becomes apparent when we consider that the use of static or residual strengths is becoming much more widely used as a design parameter for a number of critical earth structures. This becomes most critical for those soils in a relatively loose state as these are the ones most susceptible to very unconservative strength estimates when undrained triaxial tests are used as a basis for strength evaluations in which membrane compliance effects are not accounted for.

An attempt was made to see whether or not the test results obtained while using the injection-compensation system was employed could be replicated by simple mathematical corrections to volume changes based on compliance volume measurements, as was done for the small-scale tests performed by Seed and Anwar (1986). Figure 6.20 shows the results of the $\overline{\text{IC}}$-U tests with and without injection, and without injection with the simple mathematical adjustment for volume error based on the observed total change in effective confining stress and the predetermined compliance curve (volumetric error as a function of $\Delta\sigma'_3$). It is apparent from this plot that a simple adjustment of the sample volumes to the unmitigated test results does not fully account for the amount of error induced by

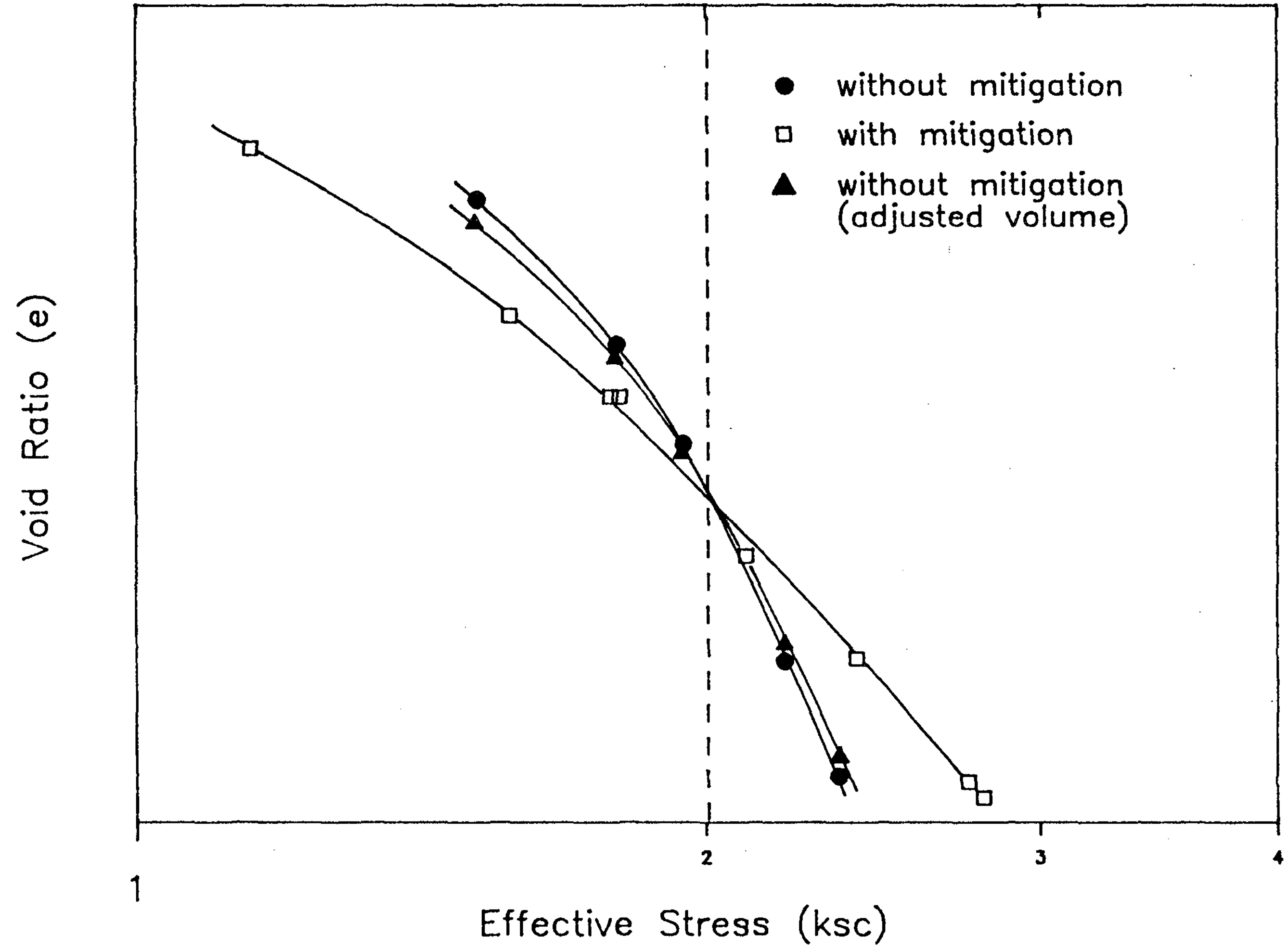


Figure 6.20: Critical-State Plot for $\overline{\text{IC}}$-U Triaxial Tests on PT-Gravel With and Without Membrane Compliance Mitigation and Without Mitigation With Mathematical Adjustment

membrane compliance for these samples. This leads to the conclusion that the testing errors incurred are more complicated and are likely to be the result of the interrelationships between volume change tendencies and pore-pressure generation. Until such a time that a theoretical model can be developed which can accurately model and predict the "correct" results of undrained tests performed on coarser materials, it is suggested that for large-scale testing of coarse gravelly soils a membrane compliance compensation technique is necessary to fully ensure that more accurate and conservative strength estimates are obtained.

6.3.2 Results of Cyclic Triaxial Tests

Table 6.2 presents a listing of the cyclic triaxial tests performed on 12-inch diameter samples of PT-Gravel. Individual results for these tests are shown in Figures 6.21 through 6.33. The cyclic load reported for these tests is the peak deviatoric load above and below the applied isotropic confining stress. The definition of "failure" for cyclic load tests in this study was taken to be $\pm 2.5\% \ \epsilon_a$ (5% double amplitude axial strain). It has been demonstrated that for gravelly materials this corresponds closely to the point at which full pore pressures ($r_u = 100\%$) are developed (Evans and Seed, 1987; Hynes, 1988), which is the definition used here for "initial liquefaction". This correlation has also been observed by this author for tests performed on a gravelly material tested as part of another study, as well as for the PT-Gravel used for these tests. This relationship can be seen in Figures 6.21 through 6.33.

The cyclic tests were load-controlled with dynamic loadings based on the initial sample area calculated at the end of consolidation. Corrections to dynamic loads applied to the specimens for membrane strength were calculated to be considerably less than 1% using an expression suggested by Bishop and Henkel

Table 6.2: Isotropically Consolidated Undrained Cyclic Triaxial Tests on PT-Gravel With and Without Membrane Compliance Mitigation

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,i}$ (ksc)	B-Value	CSR ($\sigma_{d,c}/2\sigma'_a$)	No. of Cycles to ± 2.5% ϵ_A
PT-29	51.0	No	2	.980	.350	2.5
PT-30	50.5	No	2	.982	.3075	6
PT-27	49.5	No	2	.976	.2825	10
PT-28	49.9	No	2	.972	.267	22
PT-19	50.8	No	2	.981	.235	*See Note
PT-51	51.8	Yes	2	.980	.346	1.2
PT-50	52.0	Yes	2	.982	.301	2
PT-39	49.4	Yes	2	.980	.275	3
PT-38	51.0	Yes	2	.976	.259	4
PT-53	49.9	Yes	2	.980	.240	4.5
PT-40	51.8	Yes	2	.976	.233	6
PT-67	50.4	Yes	2	.981	.223	16
PT-41	51.5	Yes	2	.979	.215	82

Note: Test No. PT-19 was stopped at 400 cycles with pore pressure ratio of $r_u \approx 0.55$.

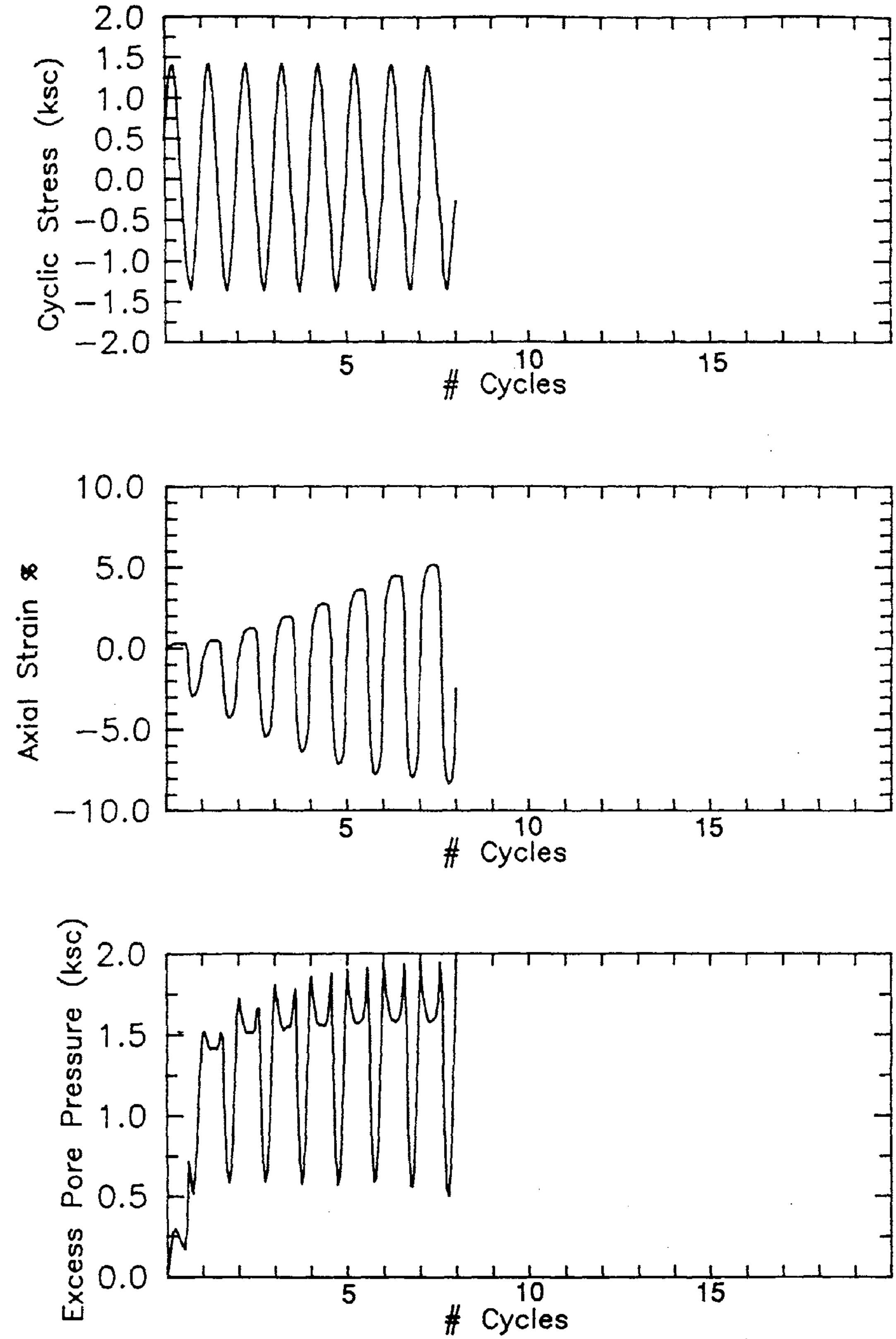


Figure 6.21: Cyclic Triaxial Test No. PT-29 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \simeq 51\%$)

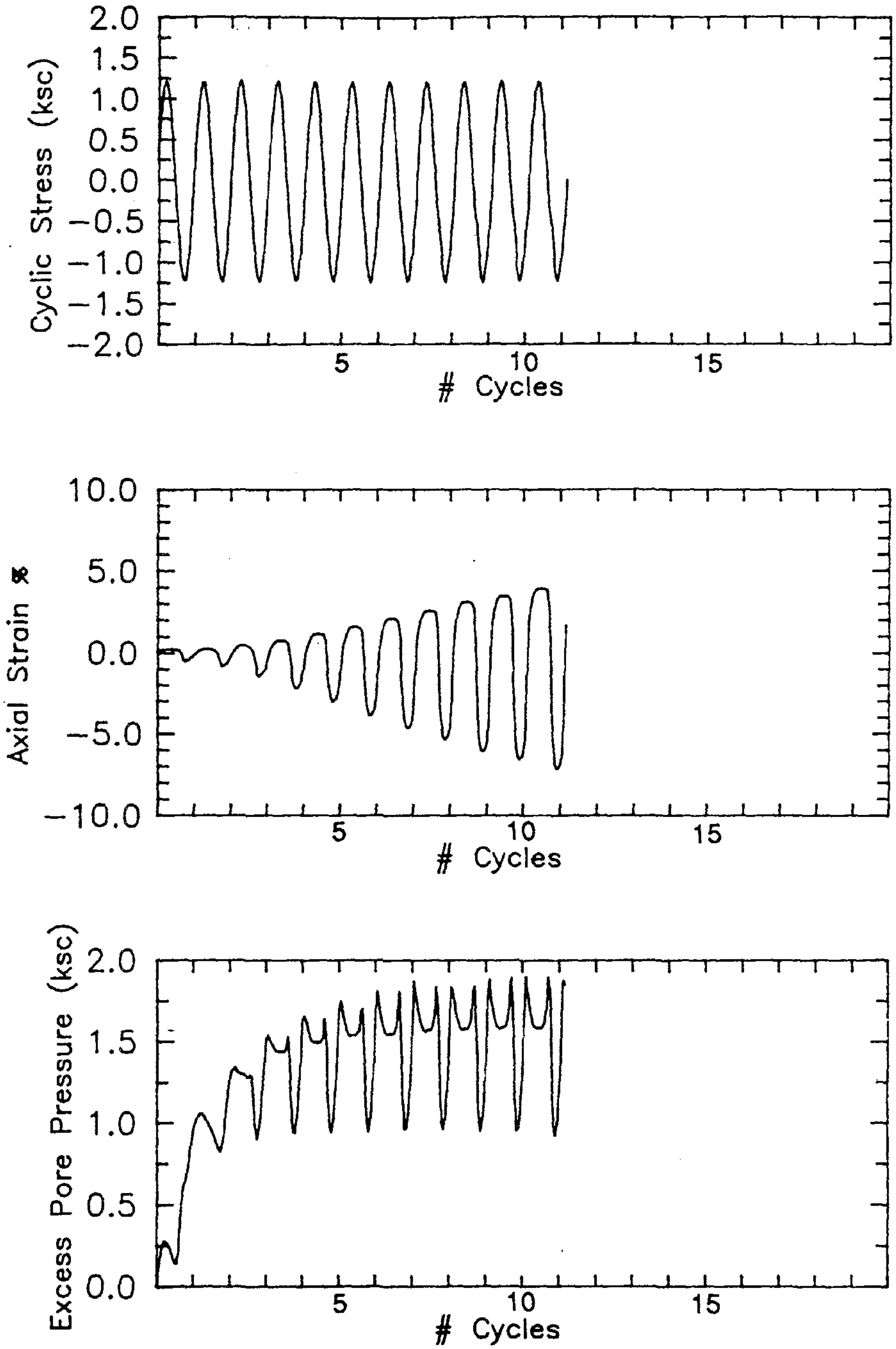


Figure 6.22: Cyclic Triaxial Test No. PT-30 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 51\%$)

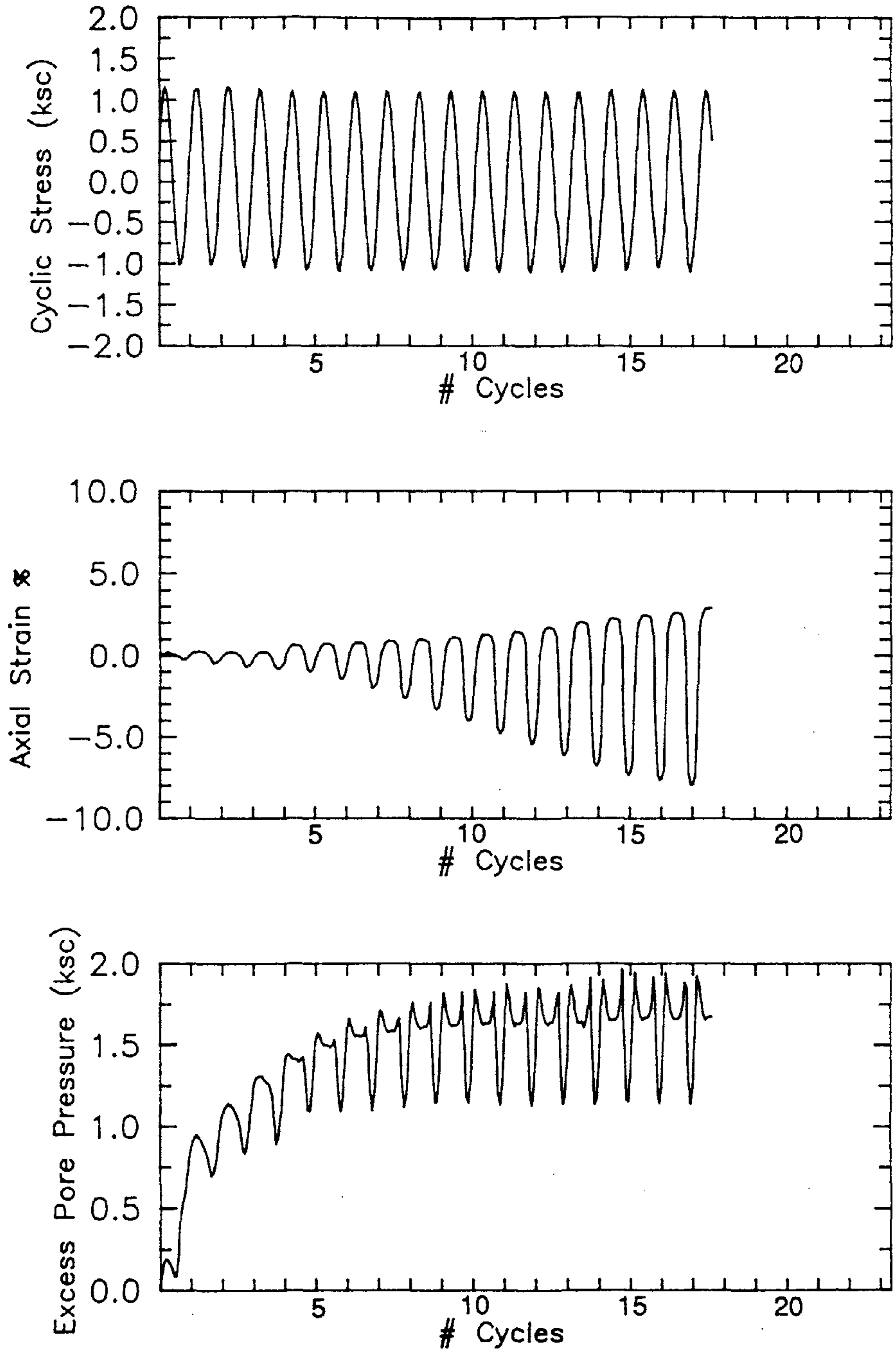


Figure 6.23: Cyclic Triaxial Test No. PT-27 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 50\%$)

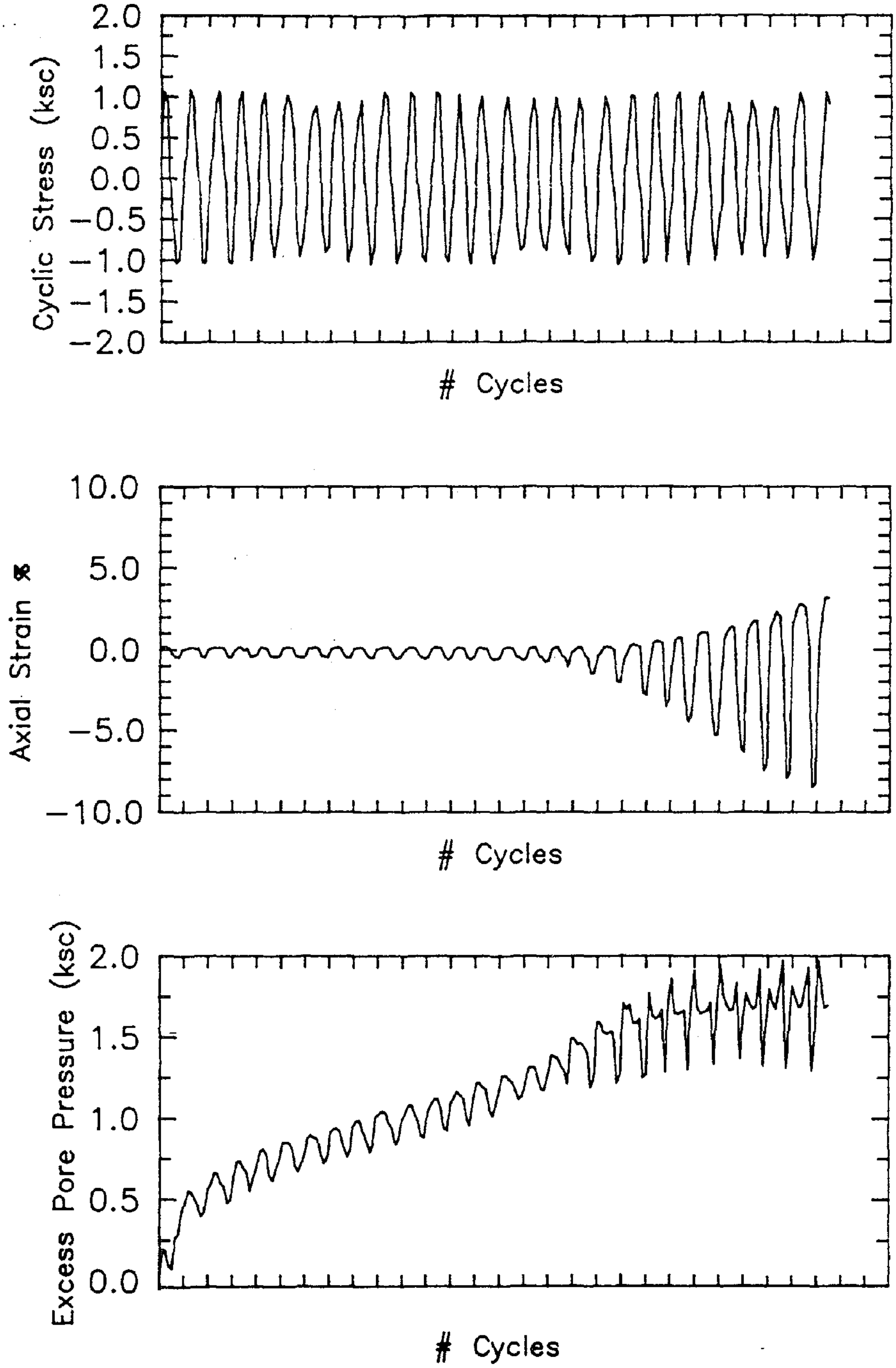


Figure 6.24: Cyclic Triaxial Test No. PT-28 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 51\%$)

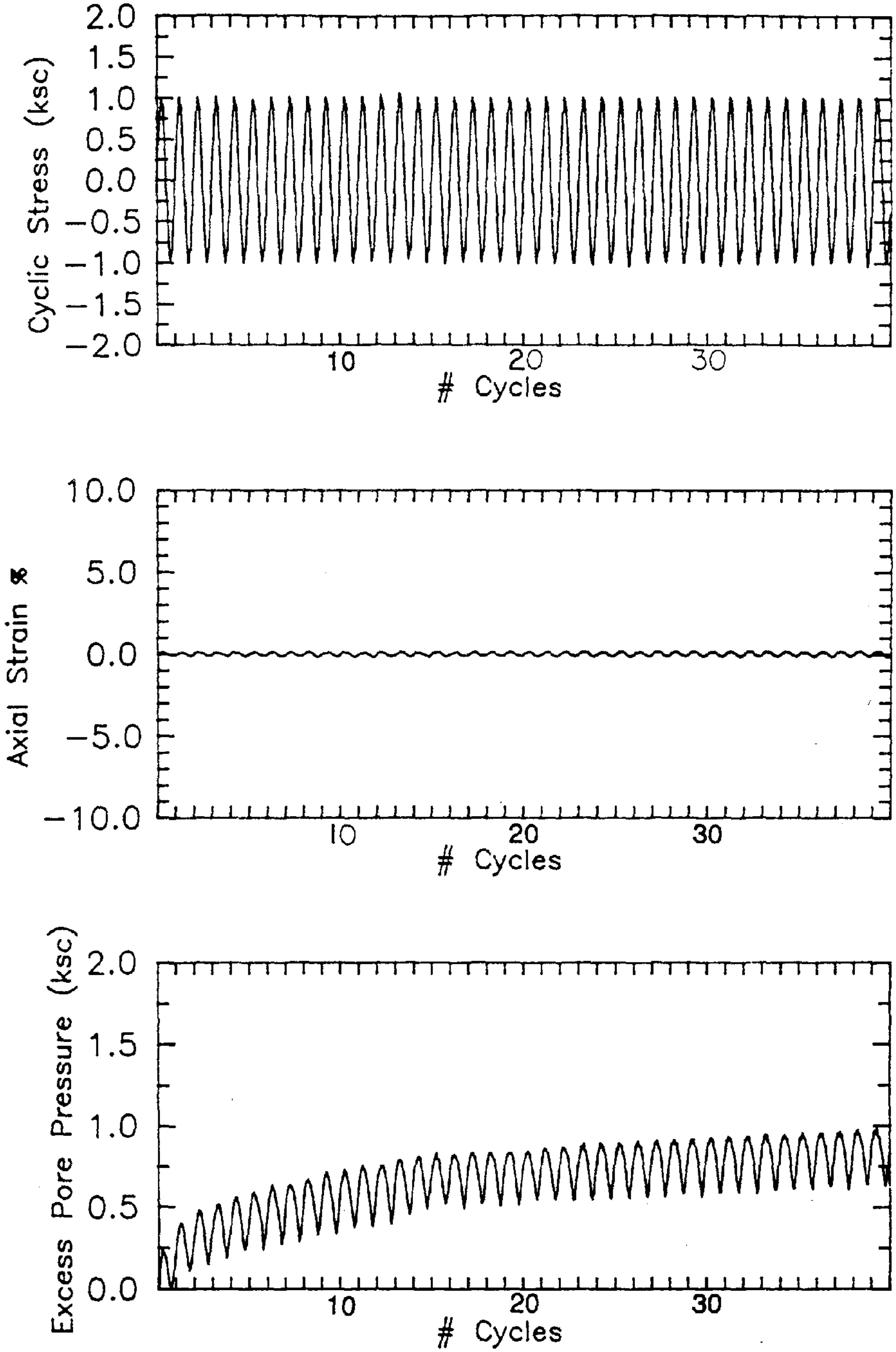


Figure 6.25: Cyclic Triaxial Test No. PT-19 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 51\%$)

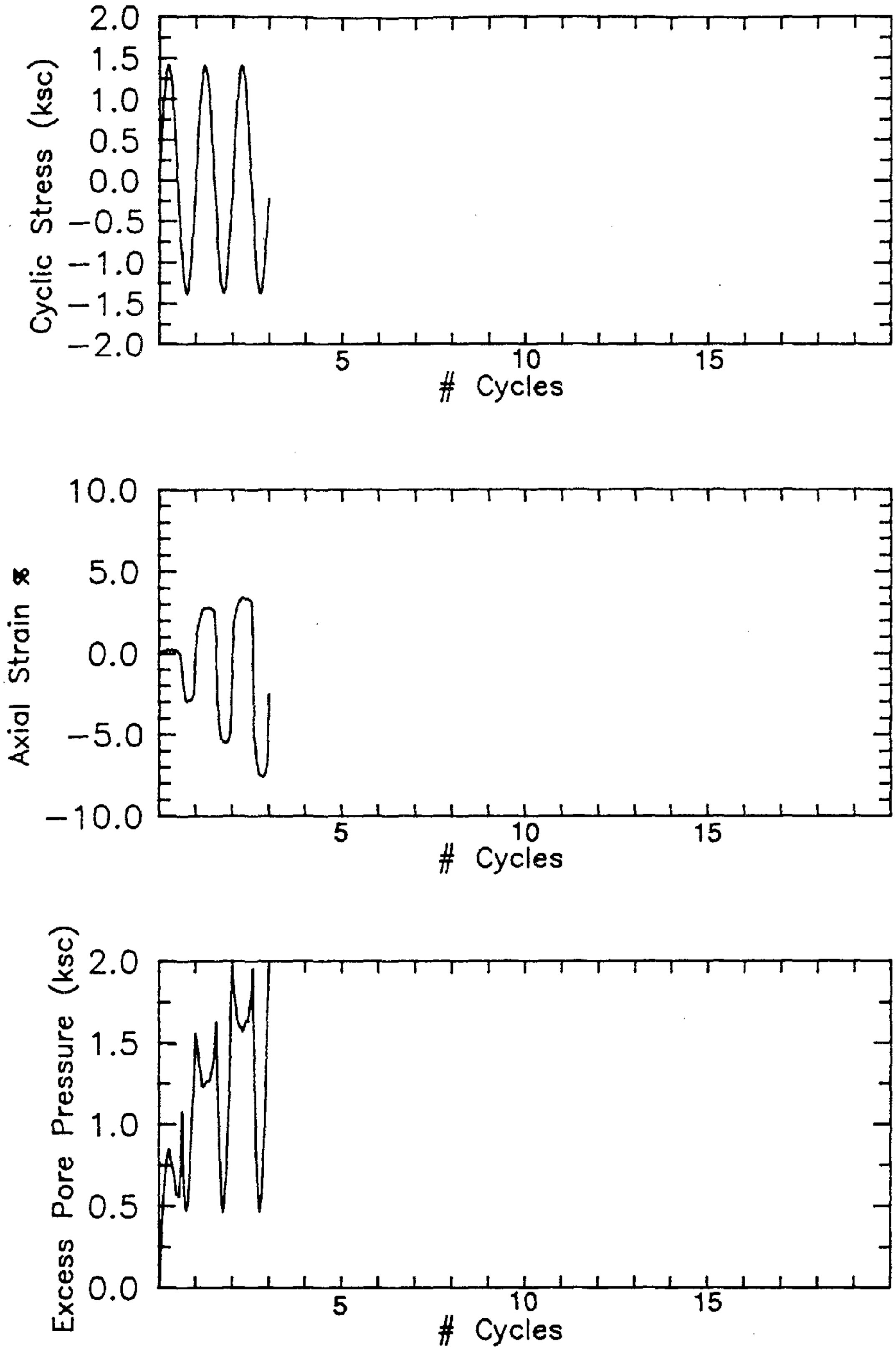


Figure 6.26: Cyclic Triaxial Test No. PT-51 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 52\%$)

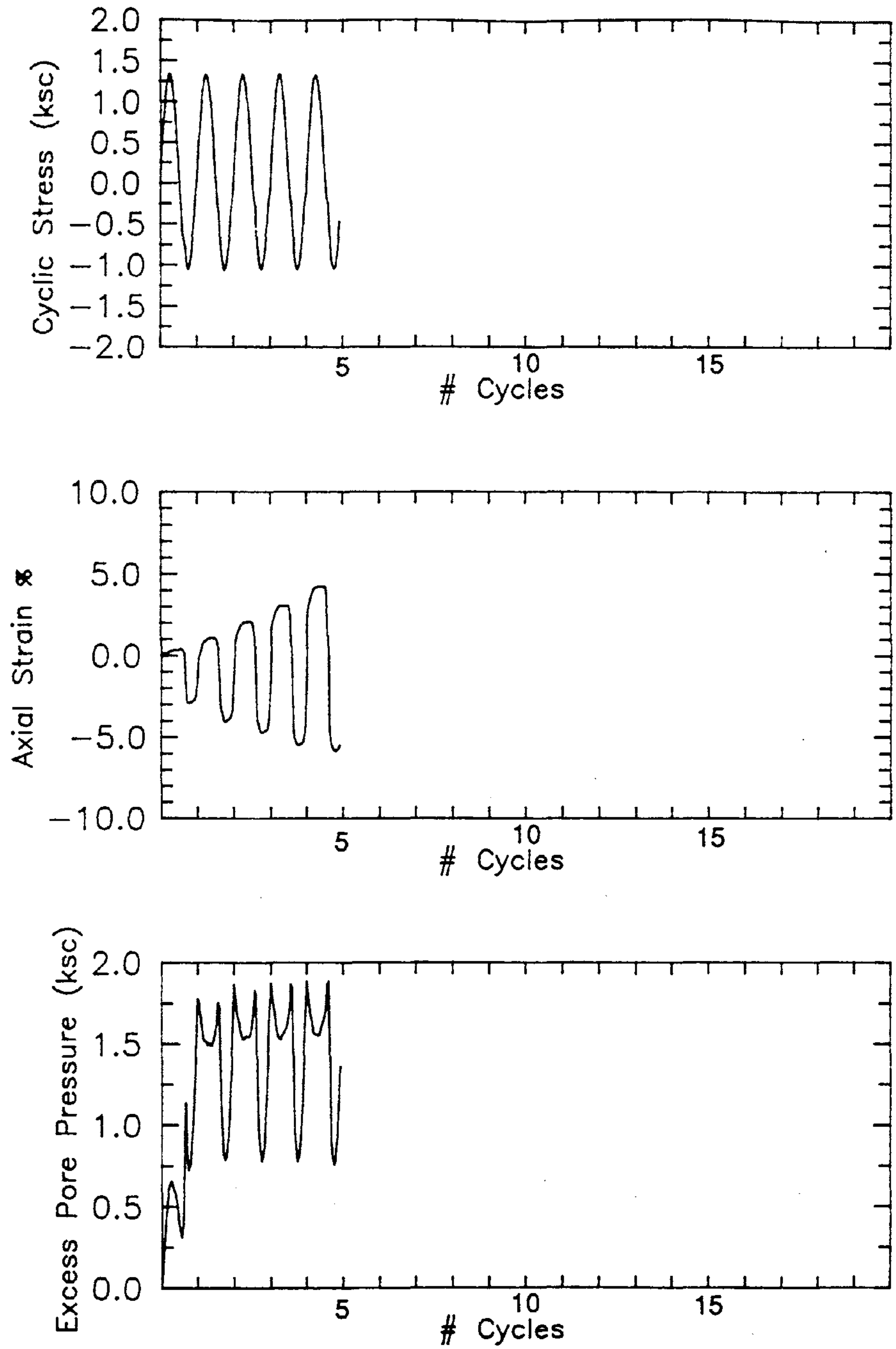


Figure 6.27: <u>Cyclic Triaxial Test No. PT-50 (PT-Gravel With Membrane Compliance Mitigation, D_R=52%)</u>

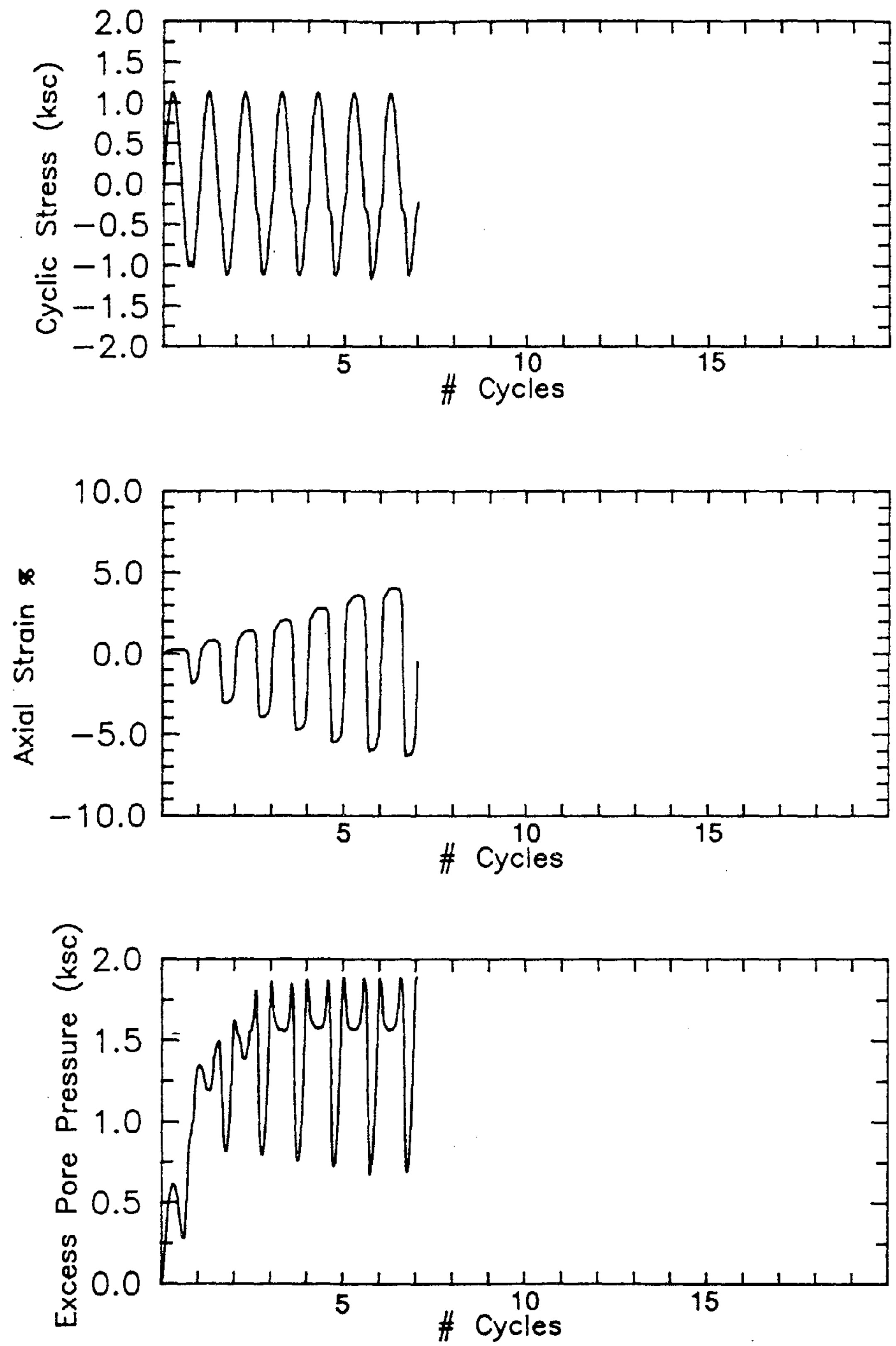


Figure 6.28: Cyclic Triaxial Test No. PT-39 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 49\%$)

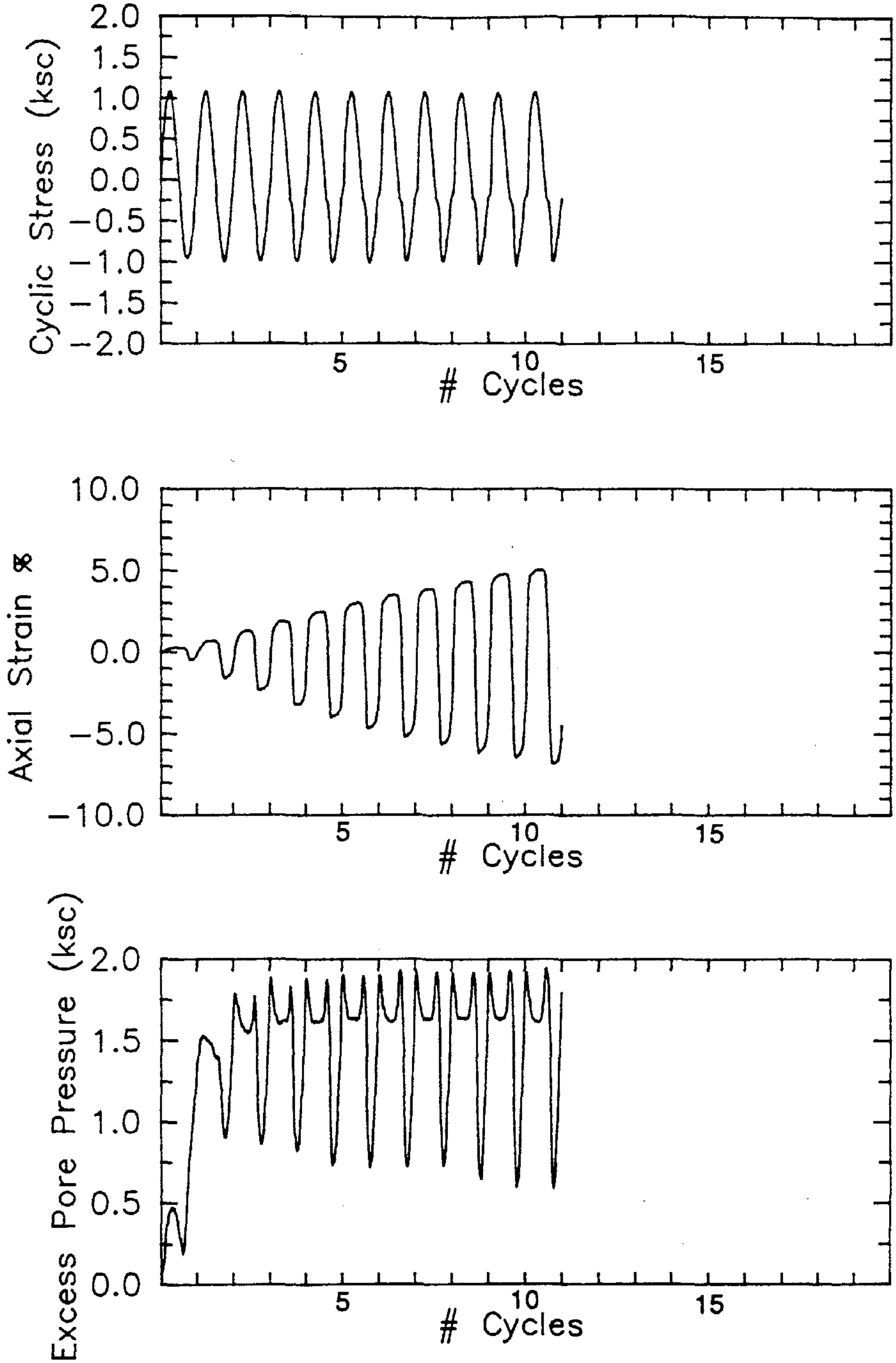


Figure 6.29: Cyclic Triaxial Test No. PT-38 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 51\%$)

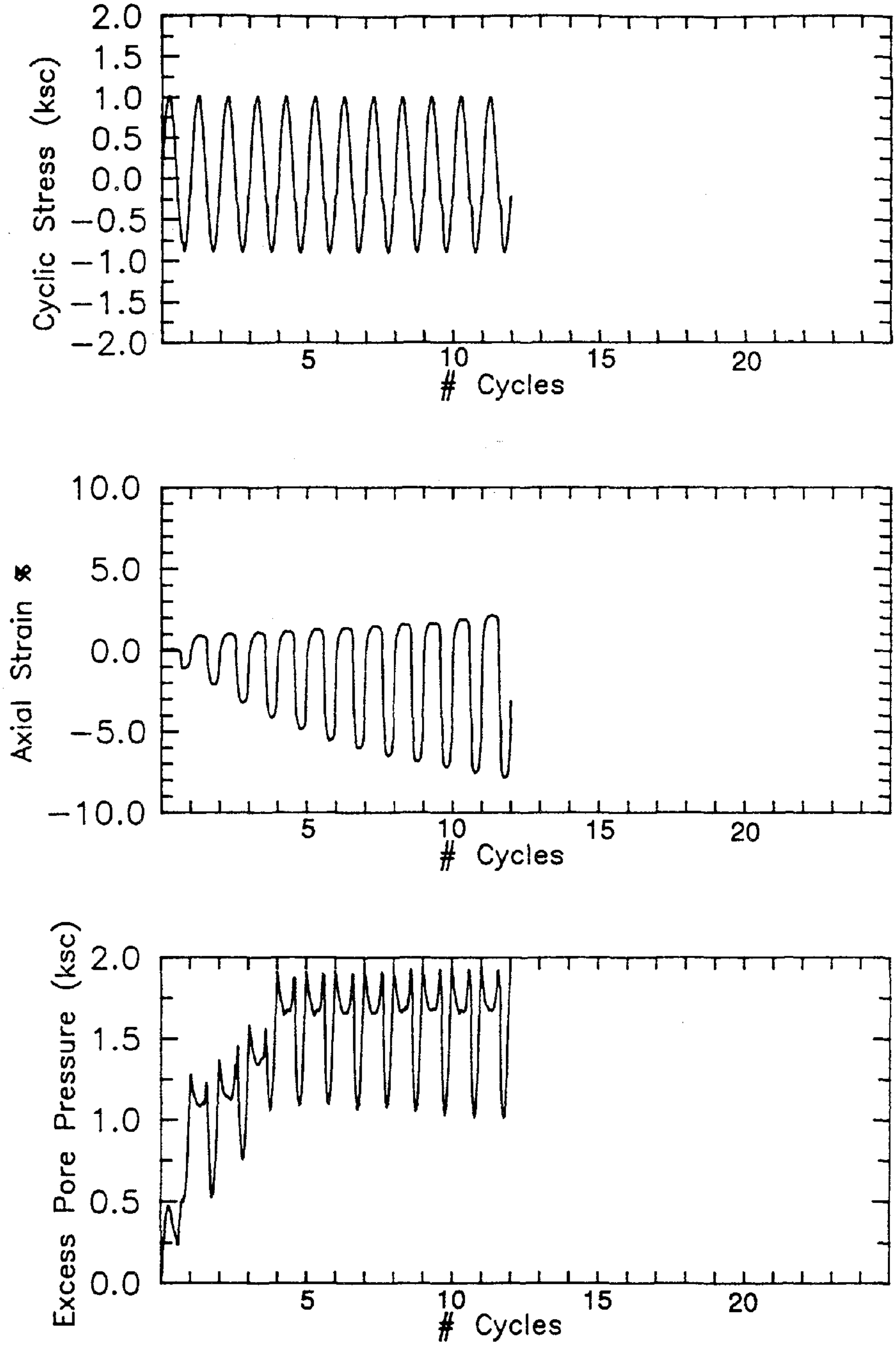


Figure 6.30: Cyclic Triaxial Test No. PT-53 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 51\%$)

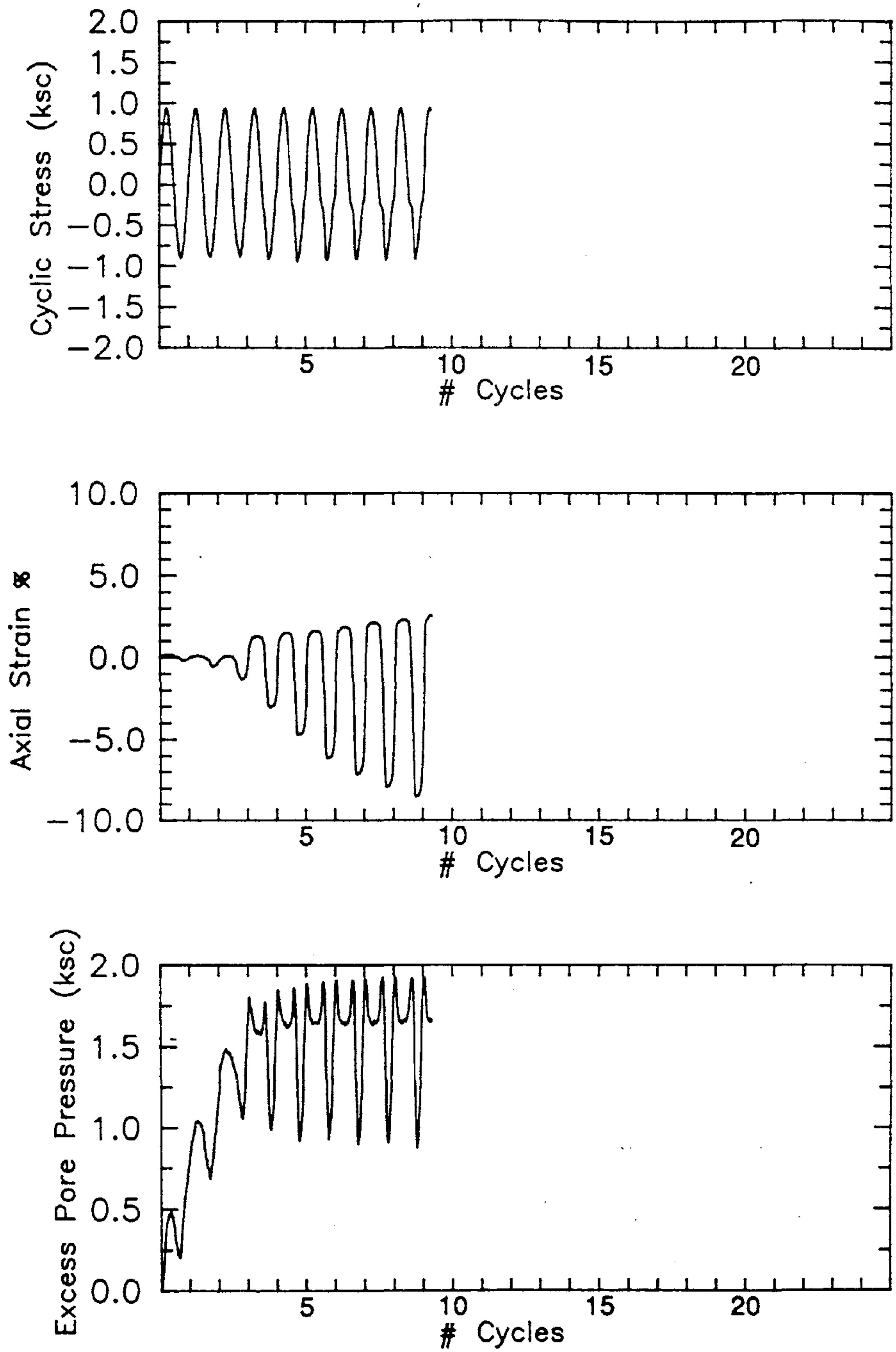


Figure 6.31: Cyclic Triaxial Test No. PT-40 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 52\%$)

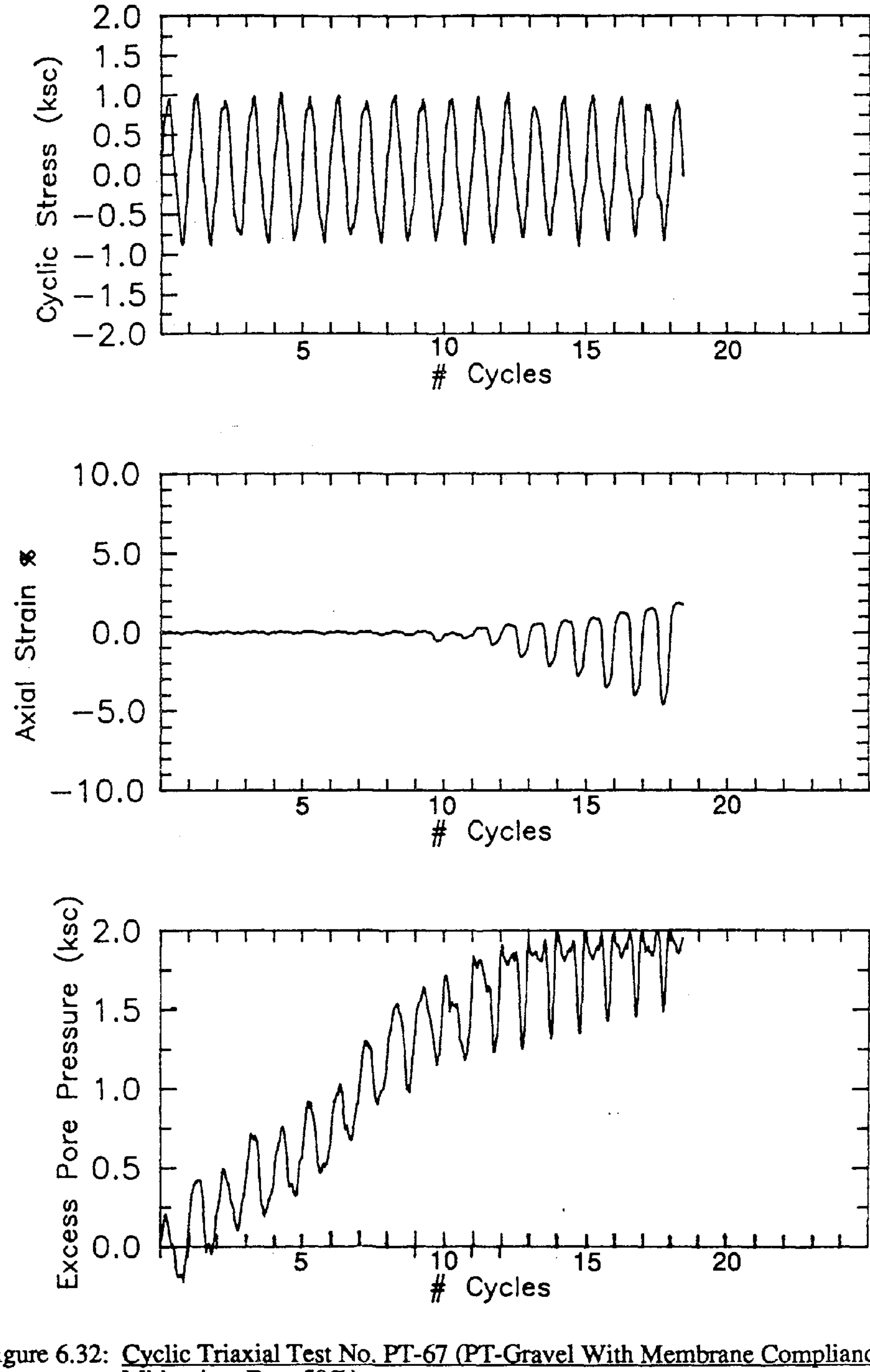


Figure 6.32: Cyclic Triaxial Test No. PT-67 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 50\%$)

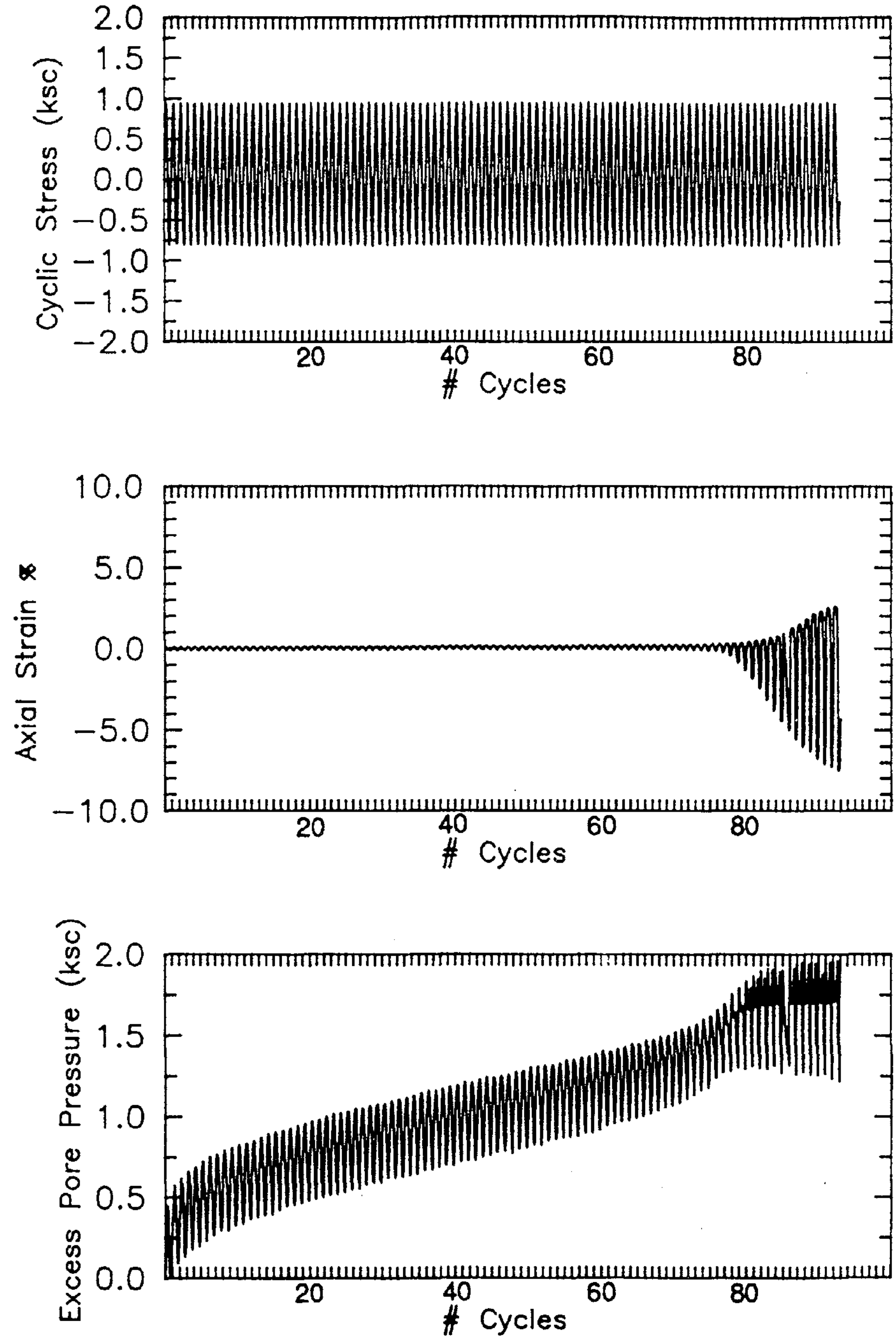


Figure 6.33: Cyclic Triaxial Test No. PT-41 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 52\%$)

(1962) using the elasticity modulus determined in laboratory tests by Banerjee et al. (1979).

The rate of cyclic loading for 12-inch diameter samples was typically 1 cycle per minute. This slower rate of loading allowed for more accurate and even loads to be applied to the specimens. Equally important, this also allowed for more uniform distributions of pore pressures generated during undrained testing, and facilitated even injection/removal for compliance mitigation. Previous studies have shown that the cyclic strength of granular materials is not significantly influenced by the frequency of cyclic loading (Lee and Fitton, 1969; Wong et al., 1974; Seed and Anwar, 1986).

A composite plot of the cyclic test results is shown in Figure 6.34 as a pair of cyclic strength curves for the two sets of test data (tests performed with and without injection-mitigation). The cyclic stresses for each test are represented in this plot as cyclic stress ratio or CSR, which is equal to the average single amplitude peak cyclic deviator stress divided by twice the initial effective confining stress. Comparing the results of the cyclic tests performed with and without compliance mitigation, it can be easily seen that the uncorrected tests greatly overestimate the cyclic loading resistance of the gravel at this density. The error, in terms of cyclic stress ratio necessary to induce liquefaction in a given number of cycles, is on the order of nearly 25%. The relative importance of this error in dynamic strength evaluation is realized when anticipated dynamic loadings from possible earthquakes are compared to the cyclic strength curves. Given the geology and potential seismicity for any given location, the maximum expected degree of ground shaking can be derived. This amount of ground shaking may be defined as a dynamic shear stress applied for a certain duration which may be interpreted as a number of equivalent cycles applied in a dynamic test. The cyclic strength curves

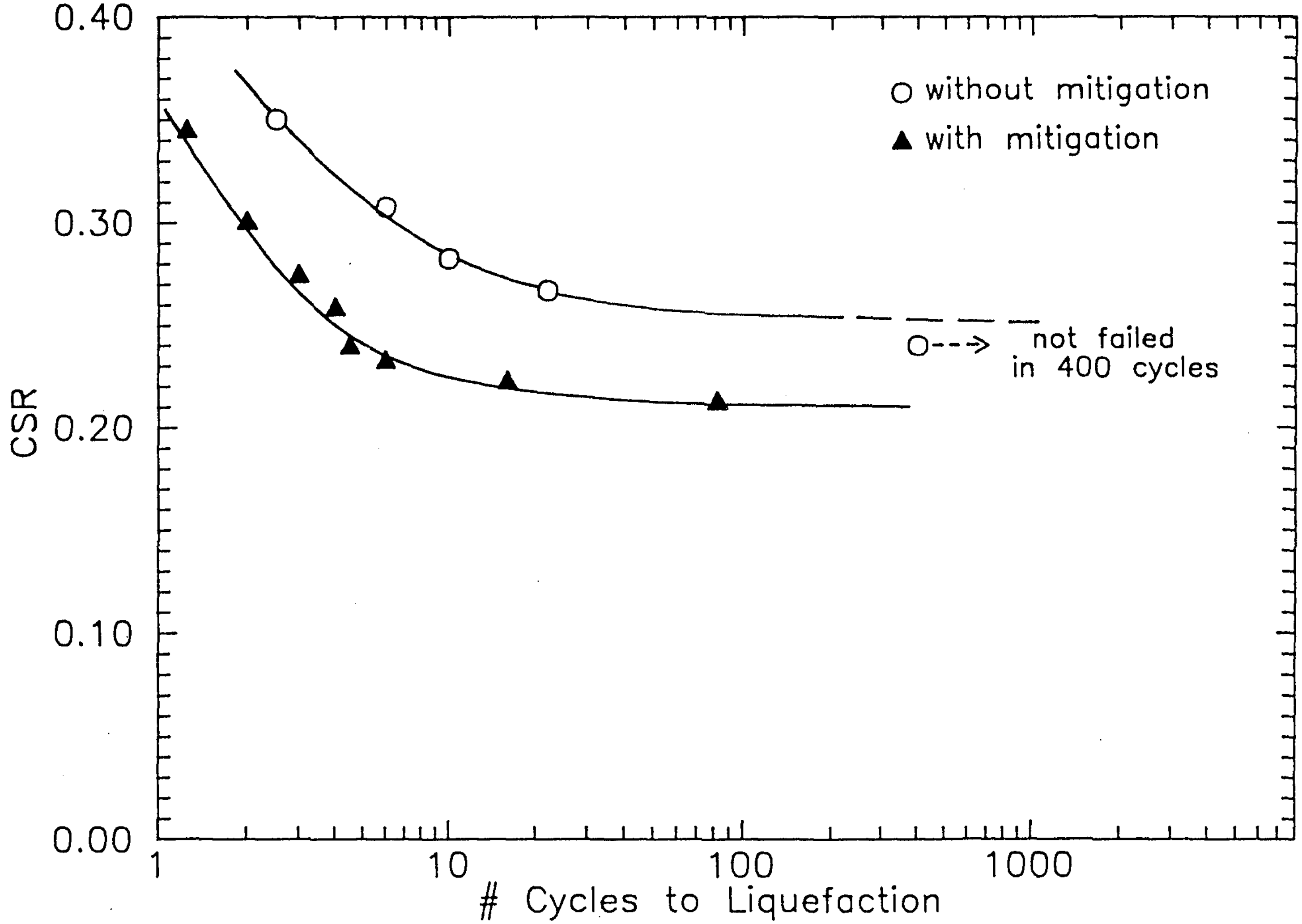


Figure 6.34: <u>Results of Isotropically Consolidated-Undrained Cyclic Triaxial Tests on Samples of PT-Gravel at $D_R \approx 50\%$ With and Without Mitigation of Membrane Compliance Effects</u>

divide the field of applied stresses and duration of loads into two categories. Points that plot above the curve are considered "unsafe" while those below are considered "safe". Furthermore, the minimum acceptable factor of safety for analysis and design of earth structures constructed of or upon this type of material, again in terms of cyclic stress ratio (CSR), is typically on the order of 1.3 to 1.4. Given the error in cyclic strength observed in this study, it can be seen that a great majority of that factor of safety will be consumed by unconservative compliance-induced testing error, making those designs and analyses much more critical. Noting that the material used for the testing performed in this study had a normalized compliance value in the mid-range of gravel sizes, it may be assumed that for coarser materials the unconservative error in cyclic strength may be considerably greater.

Looking at the pore-pressure generation curves for both uncorrected and "corrected" cyclic tests, it appears that the tests performed with the use of the compliance mitigation system generated curves similar in shape to those performed without the injection correction, suggesting that the injection system is giving "corrected" test data representative of the soil behavior.

The accuracy and reliability of the computer-controlled injection correction system devised for use with conventional sized (2.8-inch diameter) samples was verified by the use of large-scale (12-inch diameter) specimens as described in Chapter 4. Unfortunately, this type of experimental verification is unrealistic for the "corrected" large-scale tests due to the necessary size of the samples that would be needed to achieve the same relative increase in sample size as was possible between 2.8-inch and 12-inch diameter samples. In lieu of this type of physically verified accuracy, an analytical check was made by comparing the results of the cyclic triaxial tests performed on 12-inch diameter samples of the PT-gravel with and without the use of the injection-correction system, to the analytical prediction made by Martin et al. (1975) regarding the likely error (in terms of CSR) due to membrane compliance as a function of sample geometry (size and diameter) and soil gradation (D_{50}). The model proposed by Martin et al. predicts the necessary correction of cyclic stress ratio for the stresses necessary to cause liquefaction of the samples in 30 stress cycles as a function of mean grain sizes (D_{50}) of uniformly graded soils, for different sized samples.

The cyclic stress ratio correction calculated for the medium uniformly-graded gravel used in this study is plotted in Figure 6.35. It can be seen from this plot that the experimentally observed results from the tests performed with implementation of the computer-controlled injection correction system are in very good agreement with the analytically predicted correction proposed by Martin et al. This finding provides two mutually complementary benefits. The agreement between these two very different approaches to obtaining "correct" cyclic test results for a material which is prone to significant membrane compliance induced testing errors gives further backing to the accuracy and correctness of the physical testing correction. In addition, the "correct" test results obtained by using the computer-controlled injection correction system, whose fundamental accuracy was verified by virtue of the comparison between 2.8-inch and 12-inch diameter samples of Monterey 16 sand, provide a viable check of the analytical correction prediction.

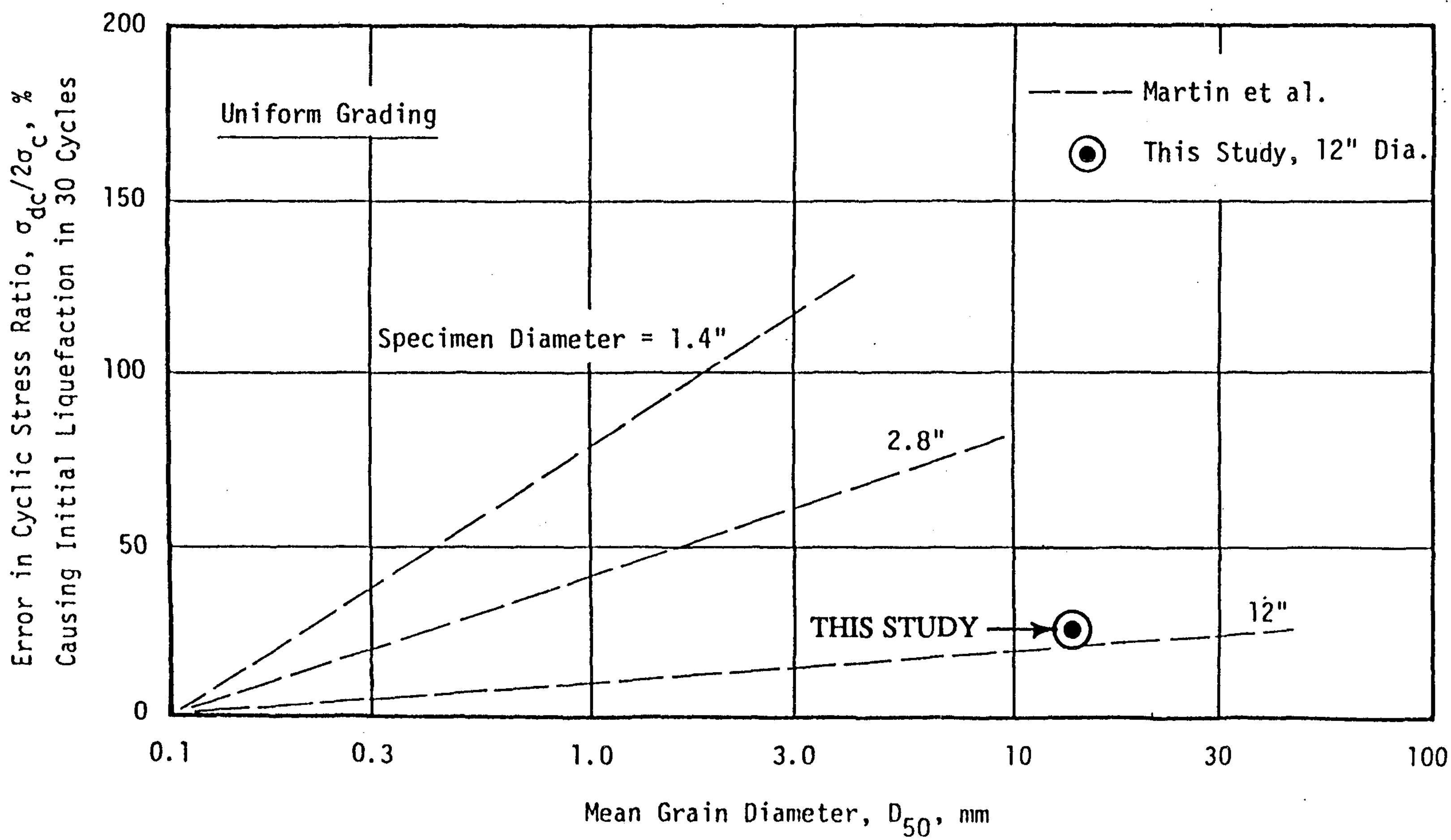


Figure 6.35: Comparison Between Laboratory-Determined Errors in Cyclic Stress Ratio Due to Membrane Compliance as a Function of Mean Grain Size for Various Specimen Diameters vs. Theoretical Error proposed by Martin et al., 1978

CHAPTER 7

RESEARCH SUMMARY AND CONCLUSIONS

The research program has represented the second phase of a two-stage program intended to develop techniques for fully and continuously eliminating the adverse effects of membrane compliance in undrained monotonic and cyclic testing of coarse, gravelly soils. Numerous investigators have worked on various aspects of this problem over the past 32 years, and many of their findings contributed significantly to the results obtained by these present studies.

The first phase of this research effort, reported in detail by Anwar et al. (1989), involved development and implementation of a technique for elimination of membrane compliance effects in testing of conventional ("small scale") 2.8-inch diameter triaxial samples. Based on review of previous work, the methodology selected for development and implementation consisted of: (1) predetermining membrane compliance magnitude as a function of sample geometry and changes in effective confining stress, and (2) use of a computer-controlled system to continuously inject or remove water from the "undrained" sample to precisely offset the volumetric error induced by membrane compliance.

The second phase of these studies, reported herein, consisted of two basic tasks: (1) verification of the accuracy and reliability (or "correctness") of the injection-mitigation techniques developed in the first phase of this research effort, and (2) development and implementation of a testing system using these same techniques to fully and continuously mitigate membrane compliance effects in "large-scale" undrained testing of coarse, gravelly soils. Both of these objectives were successfully achieved, and the resulting testing system appears to be the first to successfully perform accurate, unbiased undrained monotonic and cyclic loading tests on coarse, gravelly soils.

Verification of the small-scale injection/correction methodology developed and reported by Anwar et al. (1989) was achieved by repeating the tests on 2.8-inch diameter samples of sand performed by Anwar et al. with and without mitigation of compliance effects. The repeated tests, however, were performed using 12-inch diameter samples, at which size scale effects rendered potential membrane compliance insignificant for the sandy soil tested. The results of 12-inch diameter monotonic and cyclic tests were found to be in excellent agreement with the corresponding "injection-corrected" tests on 2.8-inch diameter samples, providing strong support for the accuracy of these injection techniques.

As the second stage of these current studies, a large-scale computer-controlled system was then developed and used to mitigate membrane compliance effects in performing undrained tests on 12-inch diameter samples. Both monotonic and cyclic tests were then performed on 12-inch diameter samples of a medium gravel. Essentially identical tests of both types were performed both with and without use of the computer-controlled injection system. The significant differences between the "corrected" and uncorrected test results clearly demonstrated the potential importance of compliance in this type of testing. Moreover, the apparent compliance-induced error in the uncorrected (unmitigated) cyclic tests was found to be in good agreement with that predicted theoretically by Martin et al. (1975), and this was taken as providing additional support for the accuracy and validity of the large-scale injection/mitigation system developed.

A necessary corollary part of both phases of this research effort was the demonstration that: (a) volumetric membrane compliance could be accurately and reliably pre-determined, and (b) that it was a repeatable function of effective confining stress. This too, was successfully accomplished, and it was further demonstrated that compliance magnitude could be strongly correlated to the soil grain size index D_{20}. A considerable number of soils were tested, and a relationship between normalized unit membrane compliance (S) and D_{20} for both sandy and gravelly soils was developed.

The techniques and procedures developed in these studies provide tools for performing large-scale undrained triaxial tests on coarse soils without any adverse impact from membrane compliance. Such tests may be expected to be useful directly in evaluating the undrained loading behavior of such soils, and also for developing data for correlation with large-scale in situ penetration tests (e.g. Becker Hammer) for development of empirical techniques for evaluation of the in situ liquefaction resistance of coarse, gravelly soils.

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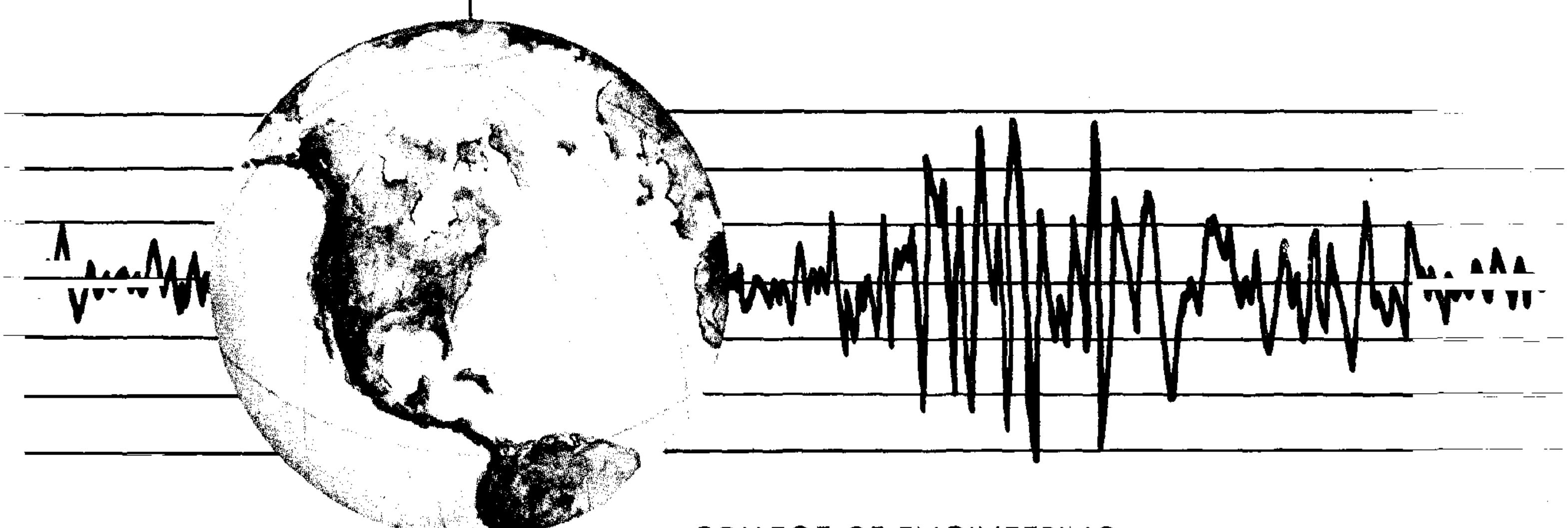
MEASUREMENT AND ELIMINATION OF MEMBRANE COMPLIANCE EFFECTS IN UNDRAINED TRIAXIAL TESTING

by

PETER G. NICHOLSON
RAYMOND B. SEED
HUSAYN ANWAR

Report to the National Science Foundation



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16. Abstract (Limit: 200 words) Undrained loading tests are widely used to investigate the susceptibility of soils to liquefaction. Changes in effective confining stress cause variations in the degrees of penetration of the confining membrane into peripheral sample voids. This phenomenon, known as membrane compliance, invalidates the fundamental assumption of constant volume during undrained testing. Membrane compliance can have a serious detrimental effect on the accuracy and validity of such tests, particularly for medium to coarse sands and gravels.

An improved testing procedure for the elimination of compliance effects during testing, recently developed for conventional scale (2.8-inch diameter) samples of sand, was verified by comparison with results of large-scale (12-inch diameter) tests of similar materials, for which compliance effects were negligible. The technique was then further developed and implemented for large-scale testing of gravelly soils.

The compliance mitigation procedure used in these studies involved predetermining the magnitude of volumetric compliance as a function of effective confining stress and soil parameters, and then using computer-controlled injection or removal of water to continuously eliminate membrane compliance effects. Monotonic and cyclic load tests were performed on uniformly-graded gravel with and without implementation of the computer-controlled compliance mitigation system. These tests are the first performed on gravels for which membrane compliance effects have been completely mitigated, and the results support the hypothesis that such soils are much more susceptible to liquefaction than had previously been thought.

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MEASUREMENT AND ELIMINATION OF MEMBRANE COMPLIANCE EFFECTS IN UNDRAINED TRIAXIAL TESTING

by

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and Husayn Anwar

REPORT NO. UCB/EERC-89/10

December, 1989

Earthquake Engineering Research Center
University of California
Berkeley, California

ABSTRACT

Undrained loading tests are widely used to investigate the susceptibility of soils to liquefaction. Changes in effective confining stress cause variations in the degrees of penetration of the confining membrane into peripheral sample voids. This phenomenon, known as membrane compliance, invalidates the fundamental assumption of constant volume during undrained testing. Membrane compliance can have a serious detrimental effect on the accuracy and validity of such tests, particularly for medium to coarse sands and gravels.

Historically, gravelly soils have typically been assumed to be "safe" from liquefaction failures. However, recent improvements in understanding of the effects of membrane compliance on triaxial test results, along with documentation of a number of field cases in which gravels have liquefied, has led to the realization that such soils can liquefy. As a result, the importance of developing a method to correctly assess the dynamic strength of these coarser soils has become apparent. Previous attempts at mitigating membrane compliance effects during testing, or making post-test corrections to conventional undrained test results, have not been fully successful in providing verifiably correct test results. An improved testing procedure for the elimination of compliance effects during testing, recently developed for conventional scale (2.8-inch diameter) samples of sand, was verified by comparison to results of large-scale (12-inch diameter) tests of similar materials, for which compliance effects were negligible. The technique was then further developed and implemented for large-scale testing of gravelly soils.

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or removal of water to continuously eliminate membrane compliance effects. Monotonic and cyclic load tests were performed on uniformly-graded gravel with and without implementation of the computer-controlled compliance mitigation system. These tests are the first performed on gravels for which membrane compliance effects have been completely mitigated, and the results support the hypothesis that such soils are much more susceptible to liquefaction than had previously been thought. In addition, these tests, performed with and without mitigation, also allow a check of the validity and accuracy of theoretical post-test corrections suggested for gravelly soils.

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NOTATIONS

A_m	=	Membrane surface area.
$\overline{\text{AC-U}}$	=	Anisotropically consolidated-undrained test with pore pressure measurements.
D	=	Sample diameter.
D_R	=	Relative density (%).
D_{20}	=	The soil particle size such that 20% (by dry weight) of the soil is finer.
D_{50}	=	The mean soil particle size.
d_g	=	Grain size (mm).
E	=	Young's modulus.
E_m	=	Young's modulus of confining membrane.
H	=	Sample height.
$\overline{\text{IC-U}}$	=	Isotropically consolidated-undrained test with pore pressure measurements.
K_c	=	$\frac{\sigma'_{1,i}}{\sigma'_{3,i}}$
K_o	=	Coefficient of lateral earth pressure at-rest.
P	=	Applied vertical load.
S	=	δV_m per logarithmic cycle change in σ'_3 (δV_m per order of magnitude change in σ'_3).
t_m	=	Membrane thickness.
u	=	Pore pressure.
V_o	=	Initial volume; or initial sample volume.
V_T	=	Volume of a soil sample.

Δu	=	Change in pore pressure.
ΔV	=	Change in volume.
ΔV_c	=	Volume change due to compression of the soil (sample skeletal compression).
ΔV_m	=	Volume change due to membrane compliance.
ΔV_t	=	Total volume change.
δV_m	=	Membrane compliance induced volume change unit area of membrane surface.
ϵ_a	=	Axial strain.
ϵ_r	=	Radial strain.
ϵ_v	=	Volumetric strain.
$\epsilon_{v,m}$	=	Volumetric strain due to membrane compliance.
$\epsilon_{v,s}$	=	Volumetric strain of a soil sample.
σ'_1	=	Major principal effective stress.
$\sigma'_{1,i}$	=	Initial major principal effective stress.
σ'_3	=	Effective confining stress; effective minor principal stress.
$\sigma'_{3,i}$	=	Initial effective confining stress.

CHAPTER ONE

INTRODUCTION - IDENTIFICATION OF THE PROBLEM

1.1 Evaluation of the Liquefaction Potential of Soils

Liquefaction can be one of the most dramatic and devastating types of soil failures that can occur due to earthquake shaking. Since the mechanics of this type of soil failure were first understood in the early 1960's, there has been a greatly increased interest in evaluating the resistance of soils comprising particular sites and earth structures to liquefaction as a result of dynamic loadings. Over the last 30 years there has been much research and investigation into developing accurate methods to estimate or evaluate the in situ dynamic strength and liquefaction resistance of soils.

Two general types of testing methods are used to evaluate the liquefaction potential of soils. One of these is in-situ testing, including most prominently field penetration testing wherein measurements are taken of the resistance of the soil to penetration by standardized testing equipment. The standard penetration test (SPT) has become an industry standard to evaluate the in situ liquefaction resistance of sandy and silty soils. Samples can be obtained at intervals during the "drilling" of each test hole in order to give a cross check of the penetration results and can be used to perform further investigations of the soils found at different depths. It has been demonstrated through numerous investigations and correlations that SPT test results can give reasonably accurate evaluations of liquefaction potential for these "finer" grained soils. More recently, the static cone penetration test (CPT) has become an increasingly attractive alternative to the SPT as it has a number of advantages including: (1) a rapid and continuous log of the soil resistance to penetration by the cone, (2) measurement of both point resistance (normal load) and shear resistance along the side shaft of the instrument, and (3) improved standardization and repeatability relative to the SPT. These advantages are offset, however, by the smaller field case study data base upon which to base empirical correlations between CPT penetration resistance and in situ liquefaction resistance.

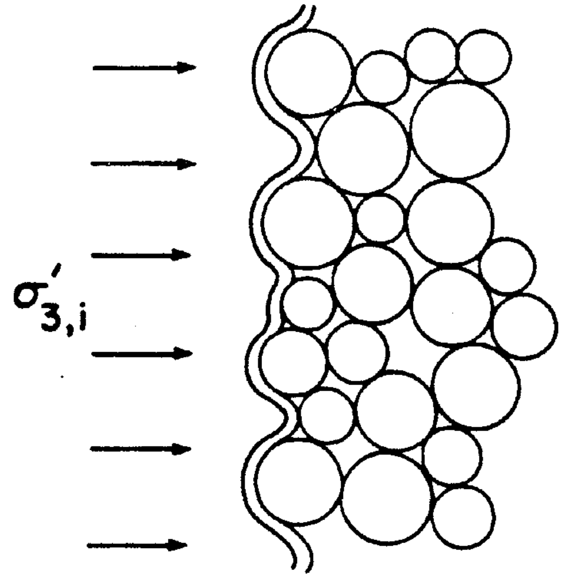
Unfortunately, while these field penetration tests provide reasonably accurate and reliable evaluations of liquefaction potential for most silty and sandy soils, they cannot be used to reliably perform the same kind of strength evaluations for coarser gravelly materials. In coarser soils, the particle sizes are either too large for the standard types of equipment to penetrate, or else unreasonably high and very unrepresentative results are obtained from tests of these soils. A recent investigation (Harder and Seed, 1986) suggests that a large-scale penetration test may be possible using the Becker Hammer for evaluating the liquefaction potential of coarse gravelly soils. This type of application has great promise for the future if a significant database can be generated with which to correlate large-scale penetration test results.

The second type of method used to evaluate soil liquefaction potential is laboratory testing of "representative" samples. This approach is not generally viable for evaluation of the in situ undrained loading characteristics of sandy soils as a result of "sampling disturbance" effects, but evidence available to date suggests that reasonably representative results might be obtained for coarser gravelly soils if the deleterious effects of membrane compliance can be eliminated. Several types of undrained loading tests (e.g. triaxial, simple shear, and torsional shear) are used to assess the susceptibility of soils to liquefaction. The implicit assumption of any of these undrained tests is that no sample volume change occurs during undrained loading except for a nominal compression of pore water resulting from increased pore pressure. A phenomenon known as membrane compliance, described in the next section, invalidates this important assumption for testing of coarse, granular soils.

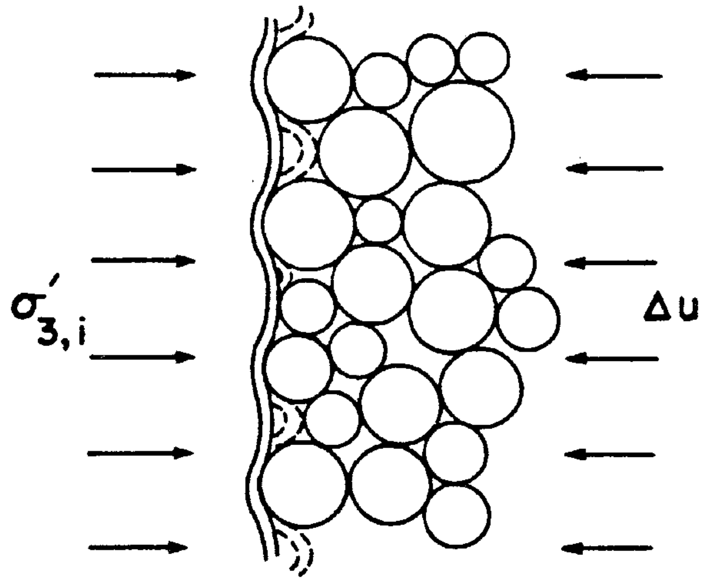
1.2 Effects of Membrane Compliance

When loading stresses are applied to relatively loose samples under undrained conditions, there is a resulting progressive increase in pore pressure within the sample, which reduces the effective confining stress and weakens the sample until strain amplitudes become large signaling the onset of liquefaction or cyclic mobility. In the laboratory, saturated soil samples are typically confined by a thin rubber membrane and subjected to cyclic or monotonic loading under undrained conditions. As a confining stress is applied to a sample, the confining membrane conforms to the shape of the sample surface and penetrates any perpheral surface voids in the sample. This effect is termed "membrane penetration". During undrained loading of relatively loose samples, the increase of pore pressure within the sample tends to push the membrane out of the interstitial voids at the sample edges. The changes in degree of penetration that develop due to changes in confining stress during undrained testing is called "membrane compliance".

Figure 1.1(a) illustrates the phenomenon of membrane penetration acting on a sample under an initial confining stress. Figure 1.1(b) schematically illustrates the effects of an increase in sample pore pressure resulting in a decrease in sample effective confining stress, which in turn results in a decrease in the degree of membrane penetration into peripheral sample voids. Due to compliance of the membrane with changes in effective confining stresses, the assumption of constant volume in an undrained test is invalidated. This change in shape of the confining



a) Initial Conditions



b) Conditions After Pore Pressure Increase

Figure 1.1: Schematic Representation of Membrane Penetration and Membrane Compliance

membrane that can introduce significant testing error in undrained tests. Membrane compliance allows pore water to migrate from within the sample center towards the vacated edge voids. The result is that internal pore pressures are relaxed, which then gives the sample an incorrect, and in most critical cases a higher (fictitious) strength.

The degree to which membrane compliance may affect the results of an undrained test is a function of the soil grain size and overall geometry of the test specimen. With fine sands and silts tested in conventional 2.8-inch diameter samples, membrane compliance effects may be negligible since even very thin membranes cannot penetrate significantly into the small surficial voids (Martin et al., 1978; Ramana and Raju, 1982). For medium and coarse sands, however, membrane compliance effects may have a significant influence on test results. Figure 1.2 illustrates a comparison between undrained monotonic loading tests for a fine Sacramento river sand and a coarse uniformly graded sand, both at essentially the same initial relative density and subjected to the same loading conditions. In spite of the initial low relative density, the coarse sand shows no reduction in strength or other evidence of liquefaction with increasing strain, although it would be expected to do so under truly undrained conditions in the field. This is due primarily to the effects of membrane compliance on the laboratory test results. Likewise, for undrained cyclic tests, membrane compliance has the potential to result in considerable overestimation of the resistance of samples to liquefaction under dynamic loadings.

1.3 Liquefaction Potential of Gravelly Soils

Laboratory techniques used for testing sands to obtain estimates of their static and dynamic strength characteristics have been well established by a

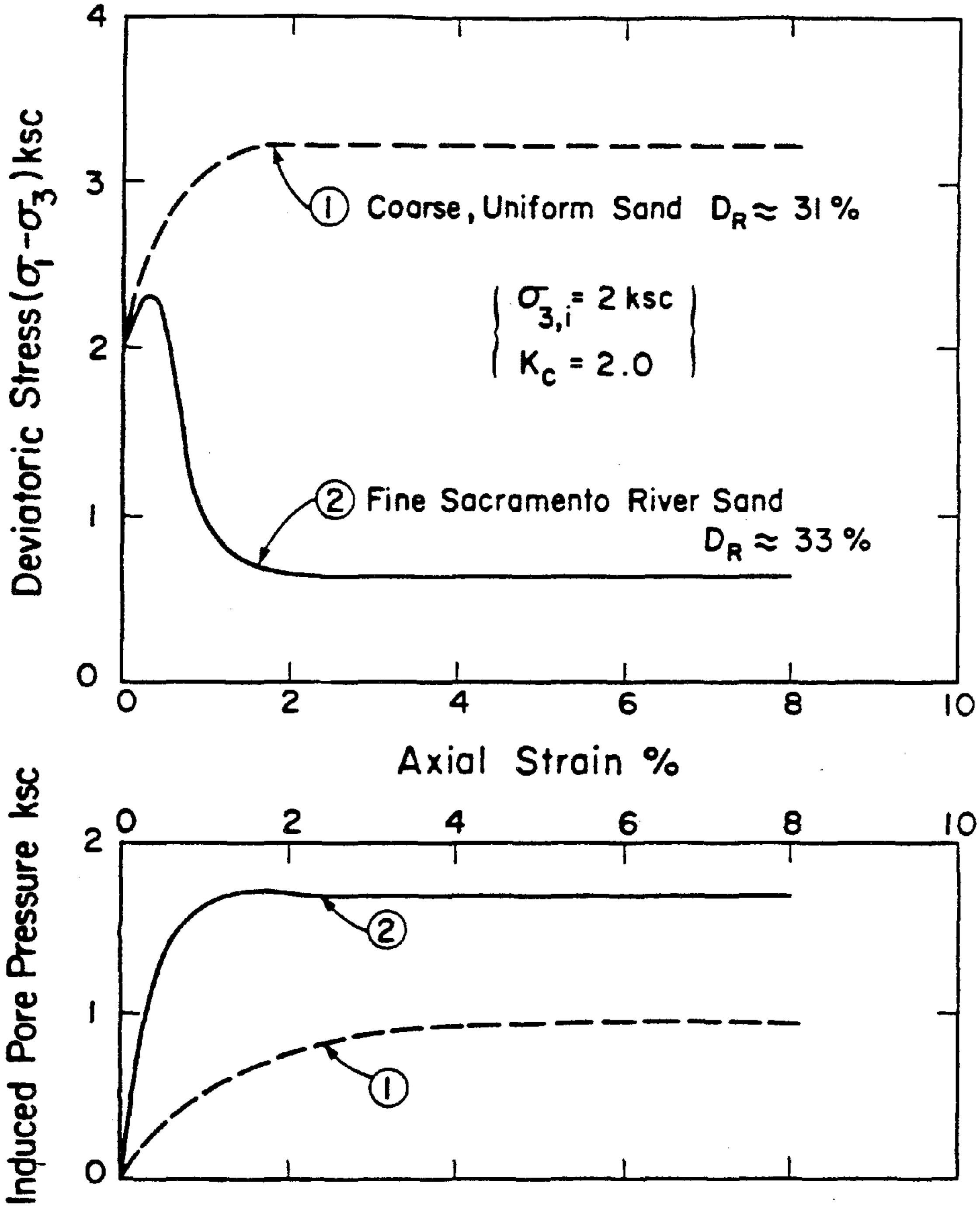


Figure 1.2: Typical $\overline{AC-U}$ Triaxial Test Results for Coarse and Fine Sands at Low Initial Relative Densities

multitude of investigators over the years. Extensive studies have been made in attempts to account for the problems that have been encountered in the testing of sand under undrained conditions in the laboratory. Much less research has been conducted to explain and mitigate the problems associated with obtaining "truly undrained" representative test results for coarser, gravelly soils. The greatest problems encountered in obtaining correct laboratory test results for these coarser soils are those associated with membrane compliance. Membrane compliance effects, and methods to reduce or mitigate those effects, have been studied primarily for soils no coarser than coarse sand. The effects of membrane compliance are greatly increased for gravelly soils due to the larger interstitial spaces between the larger grains. This allows for a greater amount of volumetric compliance error in undrained tests for these coarser grained materials. For loose gravels subjected to undrained testing, the development of large excess pore-pressures that would cause liquefaction failures is inhibited due to compliance effects, and in some cases sufficiently high pore pressures may never be attained. This is an unsafe error that can potentially lead to failure of earth structures or foundations that had been considered to be "safe" according to conventional test results.

One of the main reasons that the problems of conducting undrained tests on gravels have not been studied to a greater extent is that until recently gravels had not generally been considered to be liquefiable for several reasons including:

(a) The capacity of gravelly soils to dissipate induced pore pressure changes is, in many field cases, sufficiently large enough that initial liquefaction cannot be attained.

(b) Few cyclic laboratory tests have been conducted that provide evidence that gravels are liquefiable.

(c) There are very few case histories in which gravels have been conclusively shown to have liquefied in the field.

(d) Some studies have speculated that gravels may be inherently more resistant to liquefaction than sand.

In response to each of the aforementioned reasons why gravels have not been a major liquefaction concern are the following arguments:

(a) If the drainage from gravelly soils is sufficiently impeded, either by finer surrounding soils or by finer grains infilling the pores between the gravelly soil particles, then an undrained or partially undrained condition may exist in the field, whereby pore pressures may be able to build up to the point of causing initial liquefaction.

(b) The reason that most laboratory tests have shown gravelly soils to be much less susceptible to liquefaction than sands is most likely largely due to the effects of membrane compliance. A recent study by Evans and Seed (1987) used a modified version of the analytical correction procedure proposed by Martin, Finn, and Seed (1978) to correct the data of Lee and Fitton (1969) and Wong, Seed, and Chan (1975). It was concluded that the difference in liquefaction resistance between gravels and sands previously reported could be attributed to membrane compliance effects, and further concluded that sand and gravel specimens at the same relative density under the same test conditions had essentially the same resistance to liquefaction failure.

(c) Although there have, until recently, been very few conclusive field case histories where gravels have been shown to have liquefied in comparison to the abundance of cases involving the liquefaction of sands, a number of noteworthy cases have recently been reported wherein the liquefaction of gravels has occurred. These include: (1) The liquefaction of a gravelly-sand alluvial fan deposit in the in the 1948 Fukui Earthquake (Ishihara, 1985), (2) The liquefaction induced flow slide at Valdez occurring in gravelly sand and sandy gravel in the 1964 Alaska Earthquake (Coulter and Migliaccio, 1966), (3) The slide of the upstream sandy gravel slope of the Baihe Dam in the 1974 Tangshan Earthquake (Chang, 1978; Wang, 1984), and (4) The liquefaction of gravelly soils during the 1983 Mount Borah Earthquake (Youd et al., 1985; Harder and Seed, 1986). A much older report from the 1906 San Francisco earthquake (Lawson, 1908) gives an undeniably clear description of the liquefaction of coarse river gravels that were brought up to the surface from a "blow-hole", although at the time of the report the investigators had little idea of the mechanics behind what they had observed. In the wake of the Mt. Borah Earthquake in Idaho, a drilling investigation discovered that the soil horizon that had liquefied beneath the observed sand boils was in actuality a coarse sandy-gravel. It was suggested that the resulting sand boils revealed only the finer and lighter sand inclusions of the liquefied layer, or that they consisted of material from the dense sandy layer overlying the liquefied zone that had been washed to the surface as water was ejected from the liquefied zone. This finding raises the question as to how many other occurrences of reported sand boils may have obscured the finding of more field cases in which gravels had liquefied. In retrospect, it seems quite possible that in fact there may have been many more cases of gravel liquefaction that were not recognized by surficial investigations and therefore remained unnoticed.

As a result of these recent findings, new concerns are developing regarding the liquefaction potential of gravelly soils. At the recent 50th Anniversary Conference of the International Soil Mechanics and Foundation Engineering held in San Francisco, Dr. Ishihara of Tokyo University, in his state of the art address on the subject of liquefaction, declared that development of methods for evaluation of the liquefaction potential of gravelly soils was one of the most important and urgent problems in this important field of soil mechanics. In a recent state of the art summary report by the National Research Council Committee on Earthquake Engineering (1985), it was concluded that "...our understanding of the dynamic strength of gravels and gravelly soils is not complete, and that these soils can be susceptible to liquefaction."

1.4 Scope of Research Performed

The scope of this research investigation was concentrated in four basic steps: (1) to examine and evaluate the available methods to measure and characterize membrane compliance, and to review the methods previously proposed to mitigate the problems associated with membrane compliance effects, (2) to develop an improved understanding of the factors affecting membrane compliance for a range of soil types, including a wide range of gradation types and grain sizes from silts through gravels, and provide an updated correlation for estimating membrane compliance characteristics for these soils, (3) to verify the use of a computer-controlled injection-correction technique for mitigation of membrane compliance effects in small-scale undrained testing, and (4) to develop and implement a compliance mitigation technique for undrained testing of coarse gravelly soils.

These studies represent the second and final phase of a two-stage effort. The first stage, as reported in detail by Anwar et al. (1989) consisted of developing and implementing a computer-controlled injection/mitigation procedure for use with conventional (2.8-inch diameter) triaxial testing systems. The two basic objectives of this second stage were: (1) to verify the accuracy and reliability of this "small-scale" injection/mitigation procedure, and (2) to develop and undrained testing of "large-scale" (12-inch diameter) samples of coarse, gravelly soils.

The membrane compliance mitigation methodology used in this study involved first pre-determining the volumetric magnitude of membrane compliance for a given soil under a given set of testing conditions as a function of effective confining stress, and then using a computer-controlled process to continuously compensate for calculated volumetric errors by injecting or removing water from the sample, based on monitored changes in effective confining stresses. In order to implement the use of this method, it was necessary to demonstrate that volumetric compliance could be accurately and reliably pre-determined prior to testing, and that it could be reliably characterized in such a manner that the computer-controlled injection/removal process could be based on monitored changes in sample effective confining stresses. In addition, the compensation process had to be continuous so as not to introduce new errors to the test results.

All of the basic goals of the research investigation were achieved. Compliance measurement methods were demonstrated to be accurate and reliable, and volumetric errors induced by membrane compliance were shown to be direct and repeatable functions of effective confining stress, indicating that monitoring the changes in effective stress was a suitable measure on which to base computer-controlled injection/correction during undrained testing. The volumetric compliance characteristics were evaluated for a significant number of sandy and gravelly materials, which when added to the existing database enabled the development of an updated correlation relating measured compliance magnitudes to "representative" material grain sizes. A computer-controlled injection/correction technique implemented for testing of conventional "small-scale" (2.8-inch diameter) samples, as reported by Anwar et al. (1989), was verified by laboratory diameter) samples, as reported by Anwar et al. (1989), was verified by laboratory techniques employing "large-scale" (12-inch diameter samples) triaxial testing equipment.

A large-scale injection/mitigation system was then developed and used to perform undrained monotonic and cyclic triaxial loading tests on samples of a uniformly-graded medium gravel, with and without the use of the computer-controlled membrane compliance mitigation system, in order to provide a basis for evaluating the effectiveness of the compliance mitigation procedures. All undrained loading tests performed as a part of this study used a large-scale testing apparatus with samples of approximately 12 inches in diameter. The test results provide support for the effectiveness of the membrane compliance mitigation procedure, and show the first test results for gravelly soils tested under undrained conditions where the effects of membrane compliance have been completely eliminated.

Chapter 2 presents a review of the extensive previous research performed regarding: (a) measurement and characterization of membrane compliance, and (b) methods to mitigate of compensate for membrane compliance effects in undrained testing. Chapter 3 presents a brief review of previous research performed involving testing of gravelly soils for both static and dynamic strength evaluations. Chapter 4 describes the process by which a continuous computer-controlled injection/correction system developed for testing of conventional sized (2.8-inch diameter) samples was verified by comparison of the test results obtained in that earlier study to test results utilizing large-scale equipment. This step provided the necessary proof that such a mitigation system provided reliable and accurate results which could be suitably adapted for use with large-scale undrained testing of coarse, gravelly soils in specimens of 12-inch diameter or greater.

Chapter 5 describes the development of the computer-controlled process for mitigation of membrane compliance effects in large-scale testing of coarse, gravelly soils. Also included in this chapter is an overview of the studies performed to develop and verify the accuracy and reliability of techniques to pre-determine membrane compliance characteristics. Chapter 6 describes and presents the results of tests performed on a uniformly-graded medium gravel with and without the implementation of the computer-controlled membrane compliance mitigation system developed for large-scale testing, and examines the effectiveness of the mitigation method based on examination of the test results. Chapter 7 provides a summary of the investigations performed in this study and presents conclusions as to the accuracy, reliability and usefulness of the testing correction procedures developed and evaluated in these studies.

CHAPTER 2

SUMMARY OF PREVIOUS INVESTIGATIONS

2.1 Introduction

The problems arising as a result of membrane penetration were first presented and reported by Newland and Allely (1957, 1959). Since that time, a number of investigations have been conducted in order to develop techniques for evaluation and/or mitigation of the effects of membrane compliance in undrained tests in order to develop reliable procedures by which truly representative test results could be obtained.

A number of methods have been proposed in attempts to test cohesionless soils without the effects of membrane compliance by physically mitigating compliance during testing. Learning from these previous investigations, it was concluded that the technology has advanced to the point where it now appears possible to develop and implement a technique to obtain truly representative undrained test results for granular materials. While many of the previously proposed methods have reduced the effects of compliance, it was not until recently that research had advanced to the point of being potentially able to completely mitigate membrane compliance during testing without introducing additional problems of load corrections or the use of unverified assumptions. A review of previous investigations leading up to the current research to physically mitigate membrane compliance effects in undrained tests is presented in Section 2.3.1.

While many were attempting to discover a method to directly eliminate the effects of membrane compliance during undrained testing by physical mitigation, other investigators conducted studies to devise theoretical post-test corrections. Ideally, these theoretical corrections could be used to correct not only future tests, but also any conventional tests that had been previously performed, where membrane compliance may have given erroneous results. Until recently, it had not been possible to verify the validity of these analytical corrections, as no method had been devised to give correct, truly undrained results with which to compare the theoretical analyses. A summary of some of these theoretical post-testing correction methods is presented in Section 2.3.4.

In spite of the progress and advances made in testing and measurement techniques, there still existed several areas in which the problems associated with membrane compliance had not been resolved. These include: (a) the lack of any reliable procedures for correction of conventional test results in order to compensate for compliance effects; (b) the inability to obtain representative undrained testing of saturated soils coarser than coarse sands to fine gravels as a result of compliance effects, even when employing available large-scale testing apparatus; (c) the lack of verification of the efficacy of recently developed (and promising) techniques for mitigation of membrane compliance effects, and (d) the inability of large-scale triaxial testing systems to provide representative undrained strength and liquefaction evaluations for most gravelly soils and rockfill.

This chapter presents a brief summary of previous research investigations that have been performed leading to development of methods to evaluate and mitigate membrane compliance effects in undrained testing of saturated granular soils.

2.2 Evaluation of Membrane Compliance Effects

2.2.1 General:

Over the past 30 years there has been considerable progress made in developing methods to accurately evaluate the magnitude of membrane compliance effects. Accurate evaluation of these effects is vital, and represents a necessary first step in developing procedures to mitigate the effects of membrane compliance in undrained testing. All of the methods that have been proposed for evaluation of membrane compliance effects incorporate the use of fully drained tests in order to determine the volumetric magnitude of membrane penetration changes as a result of changes in the effective confining stress on a sample.

As the effective confining stress applied to a sample under drained conditions is changed, the total sample volume contained within the sample membrane is changed. This change in sample volume (ΔV_T) is conventionally measured by means of a calibrated device which measures the volume of water either expelled or drawn into the sample. This volume of water is actually the sum of two volume change components. One component is the "true" or skeletal volume change of the soil sample, which is equal to the "true" sample volumetric strain ($\epsilon_{v,s}$) multiplied by the overall sample volume (V_T). The second component of the total measured volume change is due to the variation in the amount of membrane penetration (membrane compliance). This second component of volume change can be expressed as the compliance-induced volume change per unit area of the membrane (δV_m) multiplied by the total area of the membrane (A_m). The total measured volume change can then be taken as the sum of these components as:

$$\Delta V_T = [\epsilon_{v,s} * V_T] + [\delta V_m * A_m] \qquad \text{[Eq. 2.1]}$$

The different methods developed to evaluate the magnitude of volumetric membrane compliance have been based primarily on developing different techniques to differentiate between the two volume change components ($\epsilon_{v,s}$ and δV_m). For large-scale triaxial samples (diameters ≥ 12 inches) this is not such a difficult problem as both axial and radial strains can be measured directly in order to evaluate the "true" sample skeletal volume change. Radial strains for such large-scale samples can be measured with the use of so called "girth-belts" which can accurately measure changes in sample circumference during the application of changes in effective sample confining stress. A illustration of the use of girth belts is shown in Figure 2.1. When dealing with smaller-scale samples (diameters < 6 inches), however, it is exceedingly difficult to measure radial strains with sufficient accuracy by this method, and other techniques must be employed to differentiate between the volume change components due to sample skeletal volume change and membrane compliance.

2.2.2 Measurement Methods to Evaluate Membrane Compliance

Newland and Allely (1957, 1959) were the first investigators to propose a method for evaluation of volumetric membrane compliance due to changes in applied effective confining pressures in triaxial testing. Assuming isotropic compression and rebound of triaxial specimens under varying hydrostatic loadings, they calculated volumetric membrane compliance as the difference between total volumetric strain (ϵ_v) and three times the measured axial strain ($3\epsilon_a$) induced by application of an isotropic stress increase in drained tests on saturated specimens. This procedure implicitly assumes that sample radial strains will be equal to the axial strain; an assumption subsequently found to be untrue. Nonetheless, this early work by Newland and Allely established important precedents and was the forerunner for numerous subsequent investigations of membrane compliance.

Since this early work, considerable progress has been made by various investigators in developing more accurate and reliable techniques for determining the magnitude of membrane compliance. A common feature of all the methods developed thus far is the use of fully drained tests to determine the volumetric magnitude of membrane penetration as a result of variations in effective confining

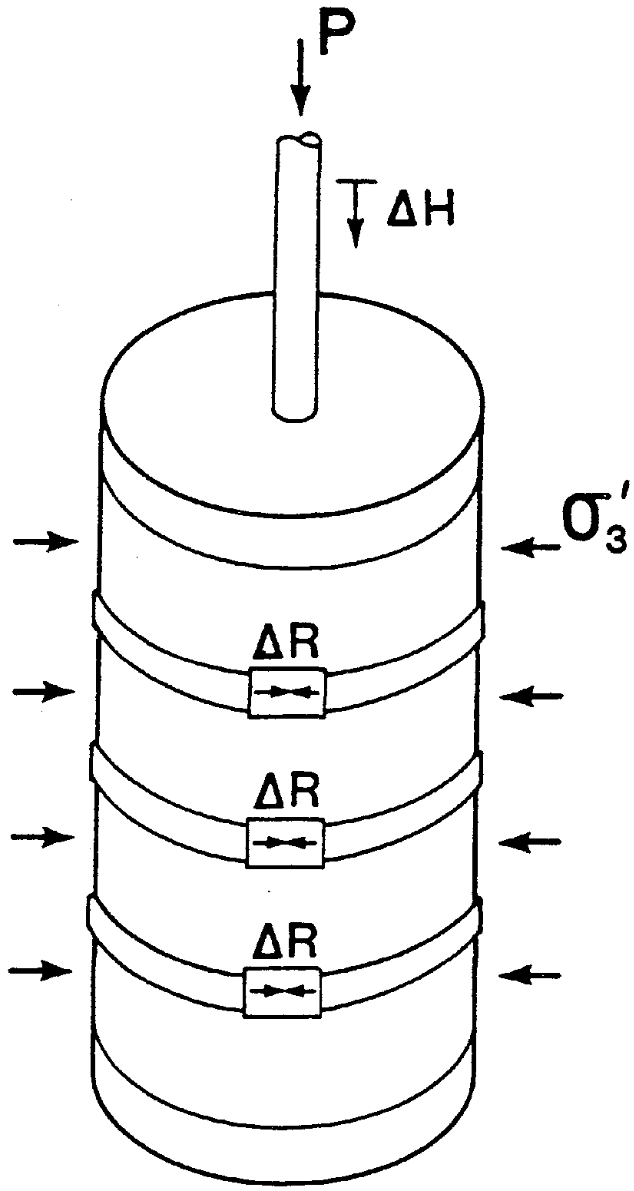
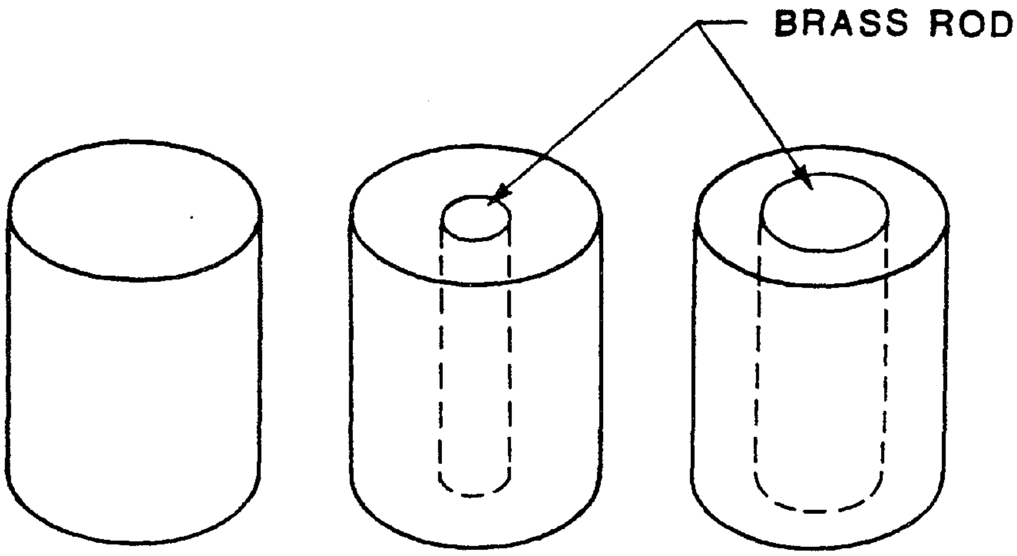


Figure 2.1: Schematic Illustration of the Use of Girth Belts to Measure Radial Strains

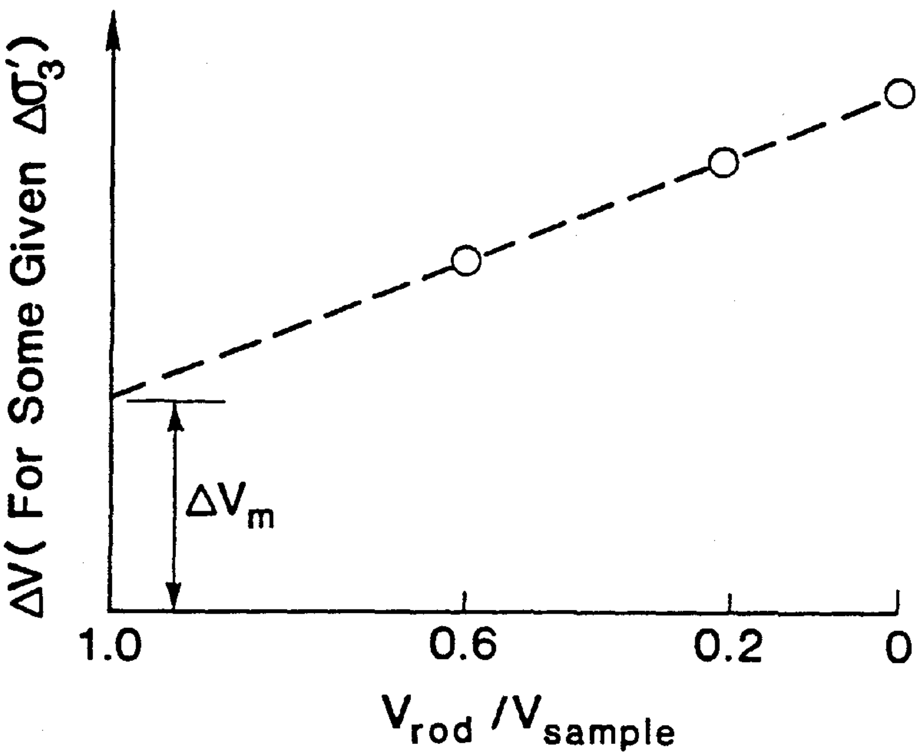
stress. Roscoe et al. (1963) proposed two methods for evaluating membrane penetration effects. The first method was similar to that of Newland and Allely and thus over-estimated membrane penetration. The error in the above method stems from the fact that hydrostatic loading of granular soils leads to anisotropic deformations, with greater radial than axial strains (Vaid & Negussey, 1984). Thus membrane compliance is overestimated due to the assumption that skeletal volumetric strain is equal to three times the skeletal axial strain.

Noting correctly that the assumption of isotropic behavior is unrealistic, Roscoe proposed a second method for evaluating compliance by placing brass rods coaxially in 1.5 inch diameter triaxial specimens as illustrated schematically in Figure 2.2(a). The rod heights were equal to the heights of the specimens, but two different rod diameters (0.25 and 1.37 inches) were used. As the diameter of the rods increased, the remaining sample soil volume decreased while membrane surface area remained constant. A sample with no rod inclusion was also tested. Volumetric membrane compliance was estimated by plotting measured volume changes for a given change in applied effective confining stress versus rod diameter. Using linear extrapolation, membrane penetration was assumed to be the total remaining measured volume change at a projected rod diameter equal to the diameter of the sample (at which point skeletal volume change would be equal to zero, so that all remaining volume change would be due the membrane compliance.)

El-Sohby (1964) and Lee (1966) also used the central rod method with minor variations in rod sizes to evaluate compliance effects. Thurairaja and Roscoe (1965) performed a subsequent study and concluded that the central rod method suffered from a number of drawbacks and in fact was not markedly better than Newland and Allely's original method based on assumed sample isotropy.



(a) Samples with Central Rods of Various Diameters



(b) Plot of ΔV vs. Sample Soil Volume (or the Ratio of Rod Volume: Soil Volume) for Some $\Delta\sigma_3'$

Figure 2.2: <u>Schematic Illustration of the Use of Central Rods in Triaxial Samples to Evaluate Membrane Compliance</u>

Steinbach (1967) concurred and used the original Newland and Allely method in a parametric study of compliance effects and concluded that the major factor influencing membrane penetration is grain size, with gradation, particle shape and density exerting minor effects. Raju and Sadasivian (1974) noted that the main drawbacks of the central rod method were twofold: (a) the rod itself caused stress concentrations within the sample due to axial rigidity; and (b) the assumption of a linear relationship between rod diameter and total volume change was incorrect. They developed a modified ("flexible") top platen which theoretically provided an improved stress field within the sample, and they demonstrated that a linear relationship existed between total volume change under some incremental load increase and actual sample volume, not rod diameter, as illustrated schematically in Figure 2.2(b).

Roscoe's central rod method was conceptually correct except for the assumption that the change in sample volume was linearly related to rod diameter. Raju and Sadasivian, having recognized this point, corrected it but failed to significantly reduce the other error introduced in the experimental method by employing a "flexible" top platen, as this did not prevent stress concentrations and non-uniformity of stress fields within the samples. This conclusion is drawn from comparing Roscoe's results with those of Raju and Sadasivian. After correcting Roscoe's original results using the correct relationship between skeletal volume change and soil volume (instead of rod diameter), no significant difference is detected between the two sets of results. The problem with both approaches is that the internal stress and strain fields within the samples vary considerably as a function of rod diameter (as a result of stress concentrations), so that exact linearity of the relationship between measured volume change and soil volume is not likely. Also, sample quality in terms of reproducible homogeneity and density control is poor due to difficulties in sample preparation with the obstruction presented by the central rod. Nonetheless, this method can provide good results when soil particles are "large" relative to sample diameter, so that volume changes due to membrane compliance are large relative to volume changes due to sample skeletal compression or rebound.

Frydman, et al. (1973), assumed that isotropic loading on various samples having similar densities yields identical volumetric strains, and developed a method for evaluating volumetric membrane compliance using hollow cylindrical samples as shown in Figure 2.3. The samples, formed from monosized glass beads, had central cylindrical voids of varying diameters that eliminated the rod stiffness and rod shearing effects (stress concentrations) inherent in the use of the "central rod method", while the central cylindrical voids caused sample volume and surface area (and the ratio between them) to vary. The pressure within the central void was varied in conjunction with the confining pressure applied on the outer face of each sample. For a given variation in applied effective confining stress, the total volumetric strain (ϵ_v) was plotted versus the ratio between membrane surface area and initial sample volume (A_m/V_o) for samples prepared with various internal void diameters, as shown in Figure 2.4. The points were found to plot linearly and intercepted the ϵ_v axis at the true sample volumetric strain. The slope was equal to the unit volumetric membrane penetration (volumetric membrane compliance per unit membrane area; δV_m). By plotting unit membrane compliance against the log of effective confining pressure, Frydman obtained linear relationships with slope S (normalized membrane penetration) which, when plotted against the log of mean particle size (D_{50}), resulted in a general relationship (see Figure 2.5) that was subsequently used to back-calculate membrane compliance corrections for his tests.

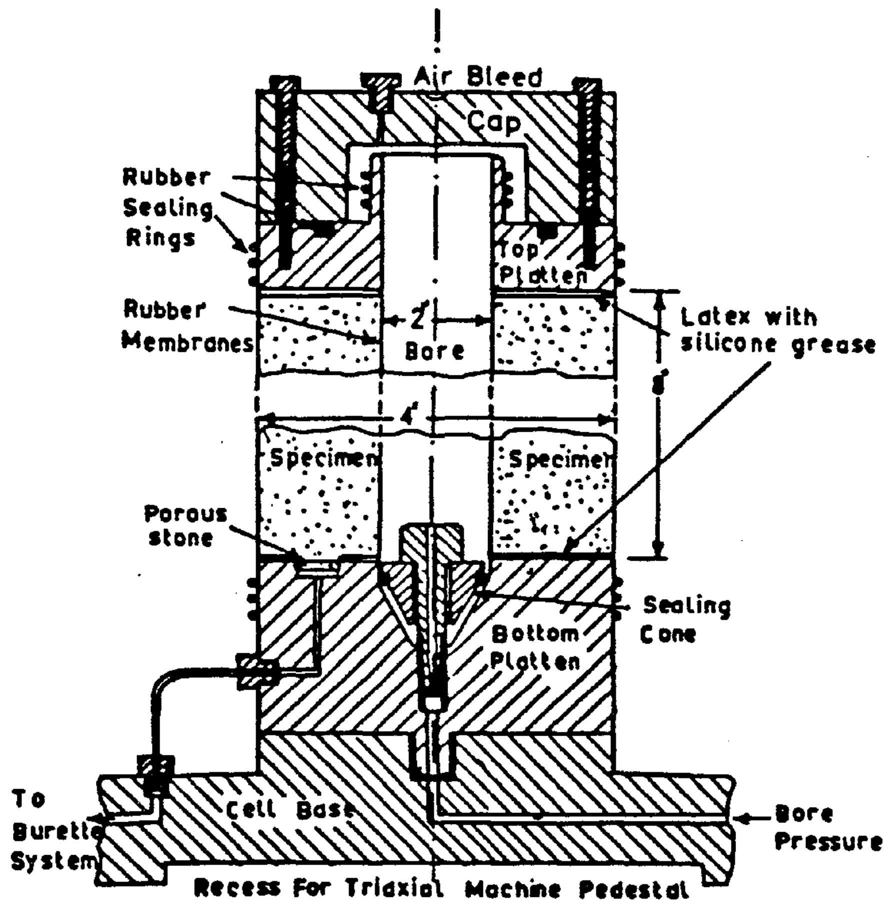


Figure 2.3: Schematic Illustration of Hollow Cylinder Samples Used to Evaluate Membrane Compliance: (Frydman et al., 1973)

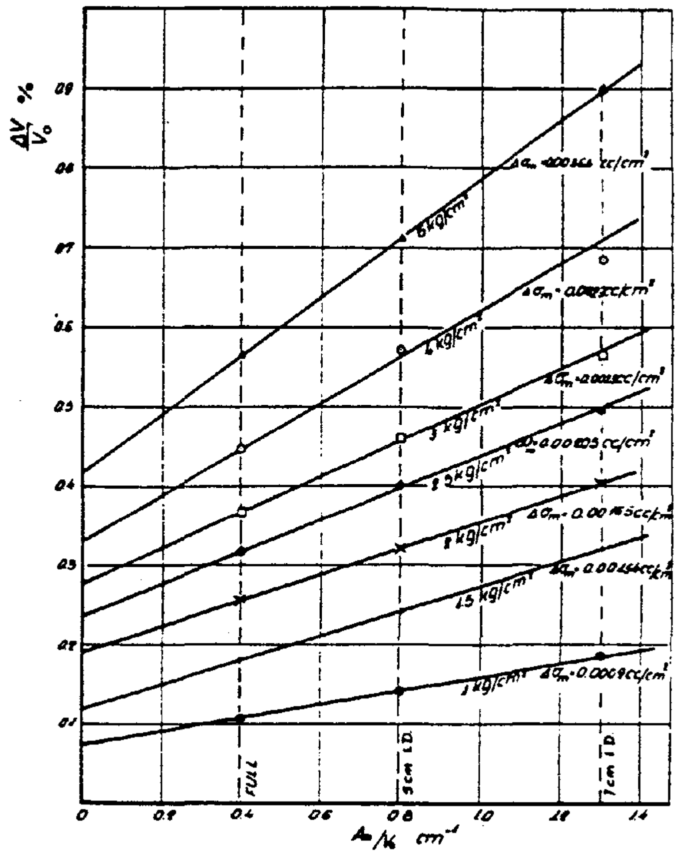


Figure 2.4: Plot of Volumetric Strain vs. A_m/V_o for a Given Change in Applied Effective Confining Stress:(Frydman et al., 1973)

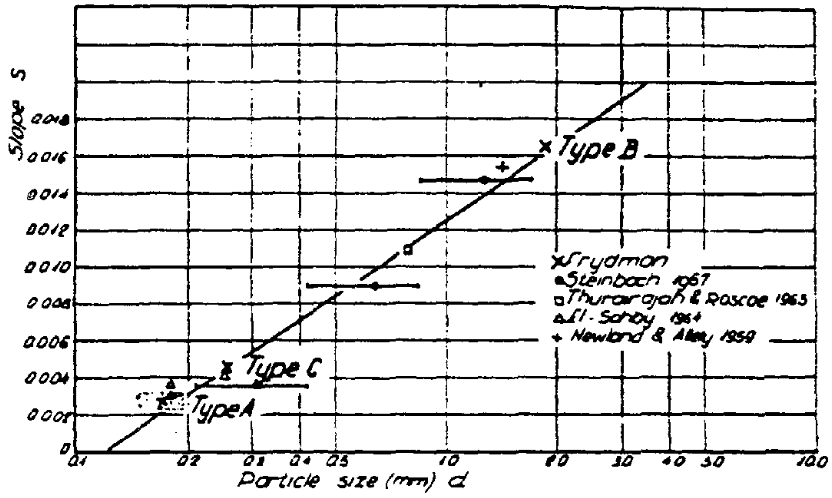
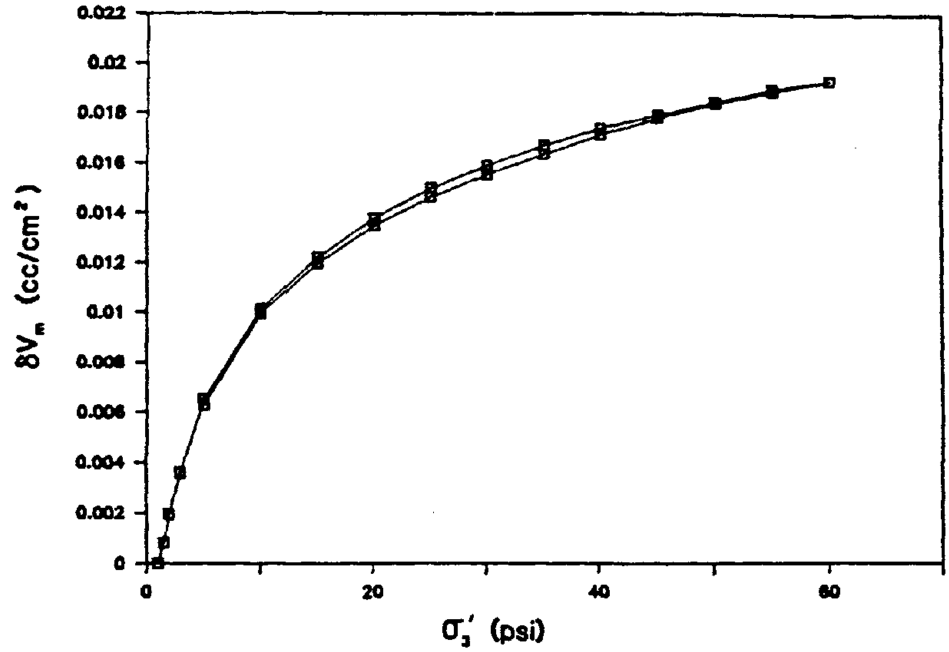


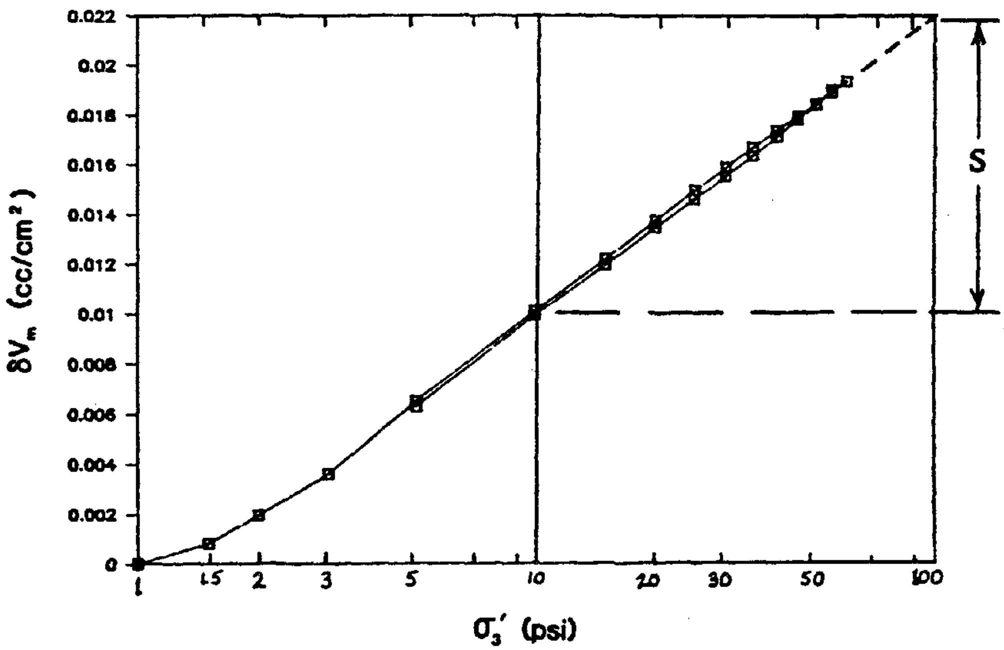
Figure 2.5: **Proposed Relationship Between Mean Grain Size (D_{50}) and Normalized Penetration (S): (Frydman et al., 1973)**

Frydman's method of membrane correction satisfies the basic assumptions of uniform stresses and strains within and between specimens, and yields more reliable estimates of volumetric membrane compliance than previous techniques. However, hollow samples are extremely difficult to prepare and duplicate with the precise duplication of sample density required; thus a simpler technique for evaluation of membrane compliance, based on simpler and more readily reproducible sample preparation techniques is desirable. The concept of unit membrane compliance introduced in his work represents a significant contribution as it is useful as a normalization tool for comparing compliance for different situations and materials. It is now well-established that the nonlinear relationship between membrane-induced volume change and applied effective stress, as illustrated in Figure 2.6(a), becomes essentially linear when plotted on a semi-logarithmic scale as shown in Figure 2.6(b). The slope of the relationship between unit membrane compliance (δV_m: change in volume per unit membrane area) and log (σ_3') will hereafter be referred to as the normalized penetration (S). S will thus be formally defined as δV_m per log-cycle change in σ_3'.

Keikbusch and Schuppener (1977) developed a procedure in which a saturated soil sample was placed in a shallow well in the base of a triaxial cell. The top surface of the sample was flush with the base of the cell, and was covered with a sheet membrane. Pressure was applied to the top of the sample, and a sensitive deflection guage measured the resulting vertical deflection of a tripod mounted on the membrane covering the sample, allowing differentiation between volume changes due to membrane penetration and those due to sample compression. Their investigation examined specimens ranging from poorly graded to very well graded sandy and silty soils and yielded an interesting nonlinear relationship for normalized membrane penetration (S) versus mean grain size (D_{50}). This



(a) Unit Membrane Compliance (δV_m) vs. Effective Stress (σ_3')



(b) δV_m vs. log (σ_3'); Defined Normalized Penetration (S)

Figure 2.6: Typical Plots of Unit Membrane Compliance vs. Effective Confining Stress (Monterey #16 Sand at $D_R \approx 60\%$)

relationship, shown in Figure 2.7, will be discussed later. Unfortunately, most of the soils tested had very high fines contents so that membrane compliance was either negligible or was dominated by consolidation effects in the sample which, due to the choice of measurement method, could not be determined with the level of accuracy desirable. A further problem with this method was the potential for edge effects during sample loading which could also introduce errors in the results. This method, which is essentially a one-dimensional version of Roscoe's approach, may potentially yield good results if soil volume were minimized so that changes in volume would be principally due to membrane compliance, and provides good results when compliance-induced volume changes are large (when soils are relatively coarse).

Vaid and Negussey (1984) examined the fundamental assumptions involved in the assessment of sample volume changes due to membrane compliance in triaxial tests on granular soils. They concluded that the necessary assumptions render invalid methods that (a) use dummy rod inclusions or (b) assume isotropic sample behavior. They noted that if, for a given change in confining pressure, the total resulting volume change is measured, then this volume change is the sum of volume changes due to (a) membrane penetration effects and (b) soil deformation effects, and can be expressed as:

$$\Delta V_t = \Delta V_m + \Delta V_c \qquad \text{[Eq. 2.2]}$$

where ΔV_t = total volume change,

ΔV_m = volume change caused by membrane penetration, and

ΔV_c = volume change due to soil skeletal deformation.

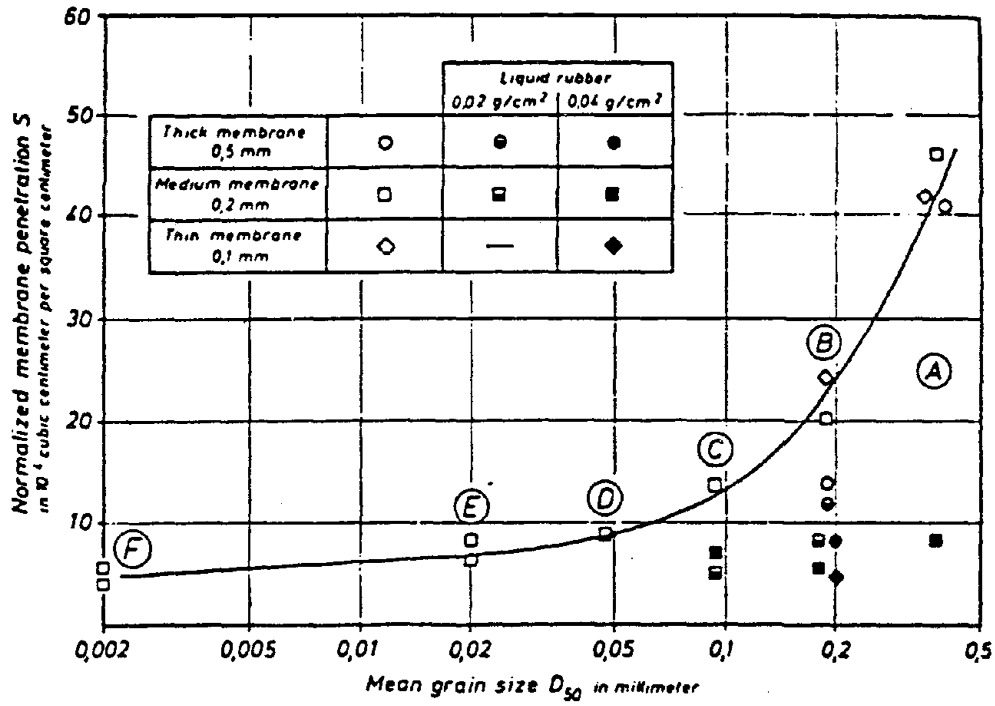


Figure 2.7: Proposed Relationship Between Mean Grain Size (D_{50}) and Normalized Penetration (S): (Keikbusch and Schuppener, 1977)

If δV_m is membrane penetration per unit membrane surface area, and ϵ_{vs} is soil volumetric strain, Eq. 2.2 can be rewritten as shown earlier in Eq. 2.1.

Vaid and Negussey argued that Newland and Allely's method overestimates the membrane effects since experimental results suggest hydrostatic loading of sand is associated with anisotropic strains, with larger radial than axial strains, and not with isotropic behavior as originally assumed. For the central rod method the assumption that sample volume is linearly related to rod diameter was shown to be erroneous by Raju and Sadasivan (1974), but the modification they suggested (which was aimed at ensuring that under an incremental change in confining pressure the sample would be subjected to a state of hydrostatic compression) was not fully successful. However, the assumption of equal soil volumetric strain for a given change in confining pressure among specimens with different rod diameters is still central to the interpretation of test results.

Vaid and Negussey suggested that reliable membrane compliance determinations can be made by either (a) Performing multiple isotropic tests on specimens having different diameters, then solving for the component of volume change due to membrane compliance using Equations 2.1 and 2.2 (a method also employed by Wong, 1983), or (b) Performing single tests on triaxial specimens subjected to triaxial unloading ("isotropic rebound"), in which case the assumption that volumetric sample strain is equal to three times the axial strain is nearly valid. They performed tests of both types on samples of Ottawa sand, a uniformly graded medium sand at a relative density of approximately $D_R \approx 50\%$, and found that both methods yielded similar results.

In considering Vaid and Negussey's alternate proposed technique, based on the assumption of isotropic strain behavior during unloading, however, it should be noted that for the Ottawa sand tested, the magnitude of the volume changes due to membrane compliance was significantly greater than that due to changes in volume of the soil itself. Errors introduced due to the assumption of isotropic rebound were therefore not as significant as would be expected for finer-grained soils. This "isotropic rebound" assumption thus appears to provide good results when membrane compliance effects are large relative to sample volume, but may be suspect when compliance-related volume changes are small. A more refined examination of this isotropic rebound assumption (Anwar et al., 1989) showed that sample strains in rebound are indeed more nearly isotropic than in initial compression, but that they are not fully isotropic. Moreover, the relationship between axial strain and radial strain (ϵ_a/ϵ_r) in rebound was found to be a function of sample density. Assumption of isotropic rebound behavior can introduce significant error in the measurement of membrane compliance for soils (and sample sizes) in which the volume changes due to membrane penetration variation are not large relative to sample skeletal volume changes resulting from variation of effective sample confining stress.

The use of a pair of samples of different diameter represents an excellent technique so long as both samples are prepared to exactly the same density (so that skeletal volumetric strains will be the same under the same applied effective stress changes.) This approach can be further improved by preparing the two samples with different diameters, but with identical height:diameter ratios, so that internal stress and strain fields will be similar, ensuring appropriate scaling of sample skeletal volume changes.

Kramer and Sivaneswaran (1988) carried this type of measurement technique one step further by devising a membrane penetration test frame with a thin sheet membrane stretched over a 7 by 9 inch opening between 3/4 inch thick aluminum plates. Theoretically, a single layer of particles could be placed beneath the membrane in specified packing arrangements so that there could be no collapse of the soil structure adding to volumetric strains. An advantage of this newer testing apparatus was that the material below the membrane could be vacuum saturated while a regulated vacuum pressure was applied below and above the membrane. This way no excess pressure would be applied to the membrane at the start of the test. Both uniform steel spheres and actual soil samples were tested in this apparatus. Assuming that the layer of spheres or soil grains and the testing frame were virtually incompressible, the measured volume change resulted solely from membrane penetration. Another variation of the "shallow well" or "thin" sample method to determine membrane penetration errors was performed by Lo et al. (1989), in which a special cell called the "error cell" was developed and used for the tests. A schematic of the error cell is shown in Figure 2.8. This measurement technique was used to more accurately determine the effectiveness of using membranes coated with liquid rubber, as suggested by Keikbusch and Schuppener (1977), to reduce the effects of membrane compliance from undrained triaxial tests. This method is discussed in more detail in Section 2.3.3.

Kramer and Sivaneswaran (1988) also conducted a second set of membrane penetration measurements in order to develop analytical expressions for membrane penetration behavior. The second set of tests were performed on a number of coarse-grained soils prepared in conventional triaxial samples employing a two membrane, "non-destructive" method. The argument for using this method was that every sample constructed has a slightly different amount of compliance, so that the actual amount of volumetric compliance should be measured for each individual sample to be tested. Most methods for measuring volumetric compliance result in an appreciable amount of sample disturbance due to loading and unloading of the sample. In that study, a conventional triaxial

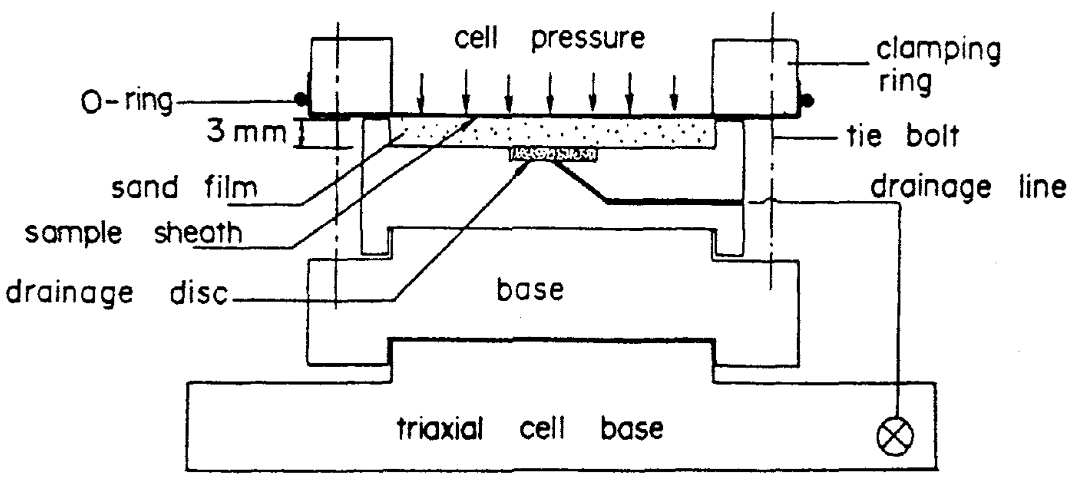


Figure 2.8: Schematic Illustration of the "Error Cell" Used to Measure Membrane Penetration:(Lo et al., 1989)

sample was constructed with a single membrane. A second membrane with a separate drain line connected to the annular space between the two membranes was placed outside of the first and the full consolidation stress was applied. Then without causing any changes of the stresses on the sample or the inner membrane, the pressures inside and outside of the outer membrane were adjusted so that the effective stress on the outer membrane was allowed to be incrementally lowered from the full confining stress to a nominal effective stress. As the outer membrane was relaxed, recordings were made of the volume of water drawn into the annular void thus representing the volumetric error induced by membrane compliance over the range of effective stresses of concern for the test.

2.2.3 Summary of Compliance Measurement Research

A comprehensive study of early investigations of membrane compliance was summarized by Ramana and Raju (1982). It was concluded that the most dominant factors affecting volumetric membrane compliance were: (a) mean grain size, (b) effective confining stress, and to a lesser extent, (c) sample density. Empirical equations have been proposed to estimate the volume changes that may be expected due to membrane penetration as a function of these parameters. It should be noted that most empirical equations are applicable only to uniformly graded materials, and would not necessarily hold for other types of soil gradations.

Seed and Anwar (1986) conducted a more complete investigation into the factors affecting volumetric membrane compliance. They proposed that the use of grain size indice D_{20} gave a significantly better correlation to volumetric compliance measurements than D_{50}, which has been most commonly used in empirical relationships. The effect of different types of soil gradations on compliance values was also investigated by Seed and Anwar, but at this point it has not been proven conclusively that the effects of specific gradations can be quantitatively accounted for. It does appear, however, from the limited amount of data available, that gap-graded soils and well graded soils that contain a high percentage of fines, tend to exhibit a somewhat lower amount of compliance than would be predicted from the D_{20} relationship. Factors that appeared to have little or no significant contribution to membrane compliance included: (a) sample particle angularity, (b) sample fabric or method of sample preparation (also reported from the investigations made by Banerjee et al., 1979), (c) sample density, and (d) membrane thickness or stiffness (within reasonable limits). While sample density has been shown to have minor effects, studies have demonstrated that only some minor adjustments would have to be made for very loose or very dense soils. An example of the difference in unit membrane compliance for a soil over a range of relative densities can be seen in Figure 2.9.

The effect of membrane thickness or stiffness has been studied by several investigators including Keikbusch and Schuppener (1977), Ramana and Raju (1982), Seed and Anwar (1986), Evans and Seed (1987), and Kramer and Sivaneswaran (1988). While most researchers have suggested that membranes of different thicknesses had only a nominal effect on values of unit membrane compliance, some others have described appreciable variations in compliance values with varying thicknesses or numbers of membranes, especially when the ratio of mean particle size to total membrane thickness becomes large (Kramer and Sivaneswaran, 1988; Evans and Seed, 1987). Theoretical studies that demonstrated insignificant effects of different membrane thicknesses on unit membrane compliance were performed by Molenkamp and Luger (1981) and Baldi and Nova (1982).

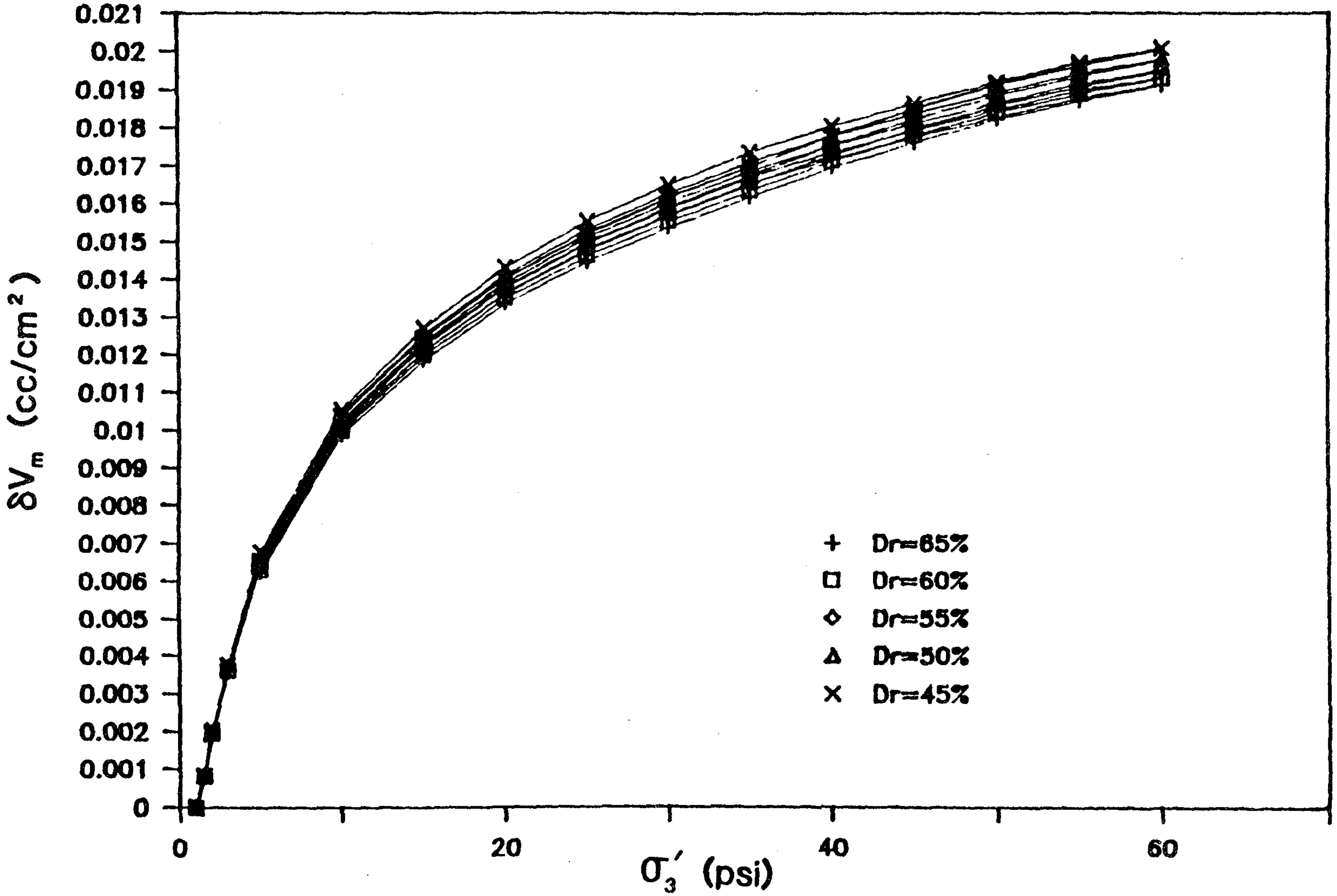


Figure 2.9: Unit Membrane Compliance vs. Effective Confining Stress for Monterey 16 Sand Over a Range of Relative Densities

It was noted by Molenkamp and Luger (1981), Seed and Anwar (1986), and Evans and Seed (1987), that some creep effects and strain softening of the latex membranes is likely to affect membrane compliance, and so it is suggested that samples should be left standing under the initial effective confining stress for approximately one hour before taking volumetric compliance measurements.

Evans and Seed (1987) performed an investigation to account for the possible effects of using varied numbers of membranes to confine 12-inch samples. Their investigation showed that the difference between using a single membrane as opposed to using two membranes to confine a sample, made a difference in measured volumetric membrane compliance of nearly two times. However, very little difference was noted between using two and four membranes, suggesting that this compliance effect had to do with the use of multiple membranes, and not necessarily the total thickness of the membrane(s). The effect of using multiple membranes was attributed to an adhesion between membranes which results in a non-recoverable residual penetration of the membranes into the external sample voids. Results of the investigation by Evans and Seed of using different numbers of confining membranes is shown in Figure 2.10. Investigations made as a part of this study did not fully support the findings of Evans and Seed, but instead concluded that the difference between confining the 12-inch samples with one or two membranes had a differential effect on the amount of membrane compliance of less than 15%.

With all of these possible affecting influences duly accounted for, it is apparent that volumetric membrane compliance is a direct and repeatable function that can be accurately and reliably pre-determined for a soil at a given density prior to testing.

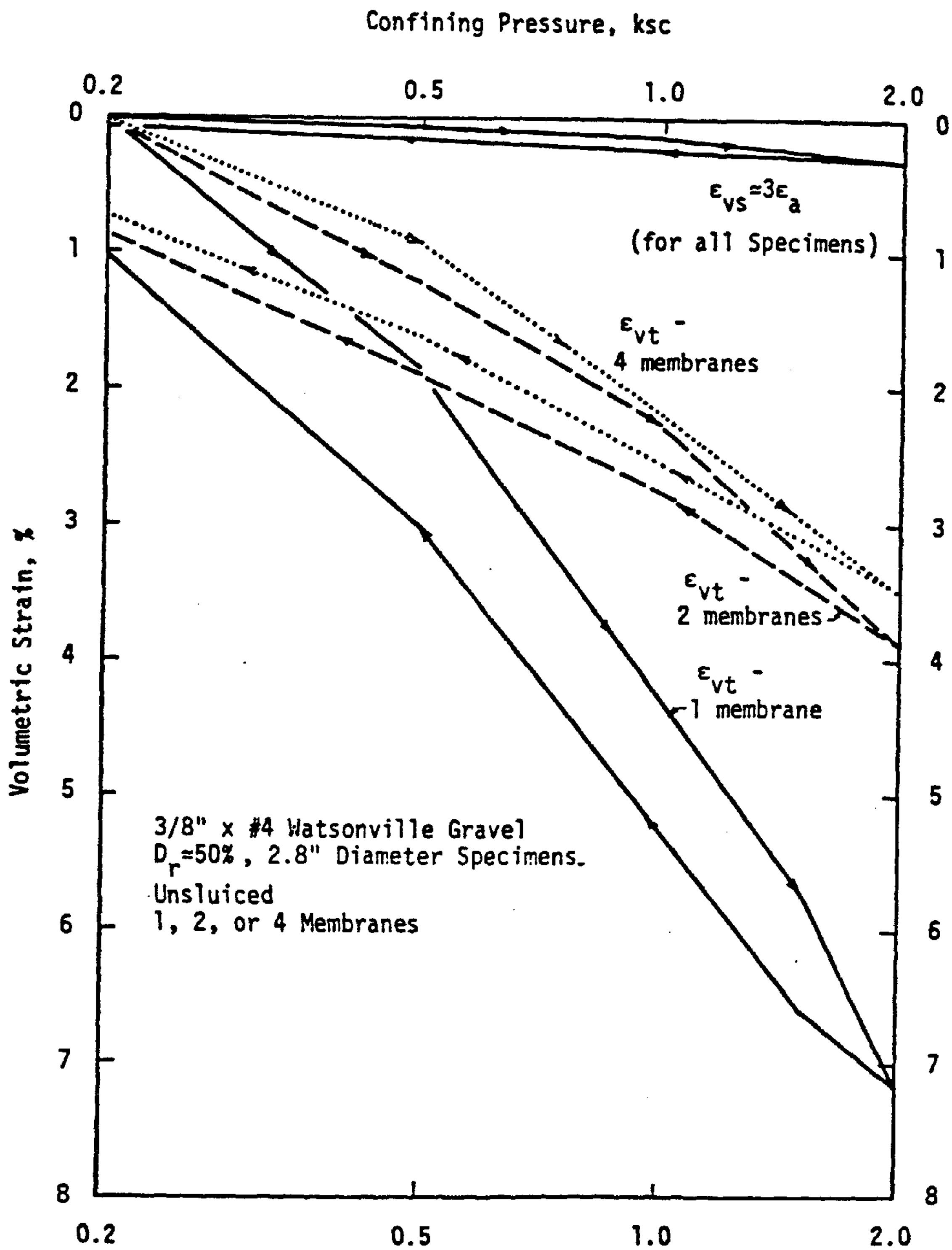


Figure 2.10: Total and Skeletal Volumetric Strains in 2.8-Inch Diameter Specimens Confined With 1, 2, or 4 Membranes: (Evans and Seed, 1987)

2.2.4 Mathematical Modelling of Membrane Compliance

A number of studies have been made including those by Flavigny and Darve (1977), Molenkamp and Luger (1981), and Kramer and Sivaneswaran (1989) to devise analytical or "theoretical" models in attempts to simulate membrane compliance behavior. Although these models may be of interest as they suggest possible mechanisms that may control aspects of compliance behavior, they do not yet appear to be able to provide accurate and reliable representations of compliance magnitudes for different soils and situations as they have been measured by the most sophisticated laboratory tests. Furthermore, it is interesting to note that for these "theoretical" models, expressions that may at first appear to give erroneous results could be reasonably "fit" to compliance measurement test data by manipulating some of the modelling parameters, such as membrane thickness and Young's modulus (stiffness), which have been shown to have minimal effect on membrane compliance values.

A model has been developed by Kramer and Sivaneswaran (1989), that assumes a more correct deformed membrane shape than the more simplified shapes assumed by previous studies. Figure 2.11 shows a comparison between the membrane deformation models used by Molenkamp and Luger (1981), Baldi and Nova (1984), and that used by Kramer and Sivaneswaran. The model used by Molenkamp and Luger assumed a simple three-dimensional indentation mechanism for the membrane, shown in Figure 2.11(a). The model described by Baldi and Nova (1984) proposed that a better representation of the deformed membrane would be as shown in Figure 2.11(b). The results of the modelling performed using the assumed deformation configuration suggested by Kramer and Sivaneswaran (1989), shown in Figure 2.11(c), compared favorably with the experimental results performed on a sample of coarse sand, and appears to give a

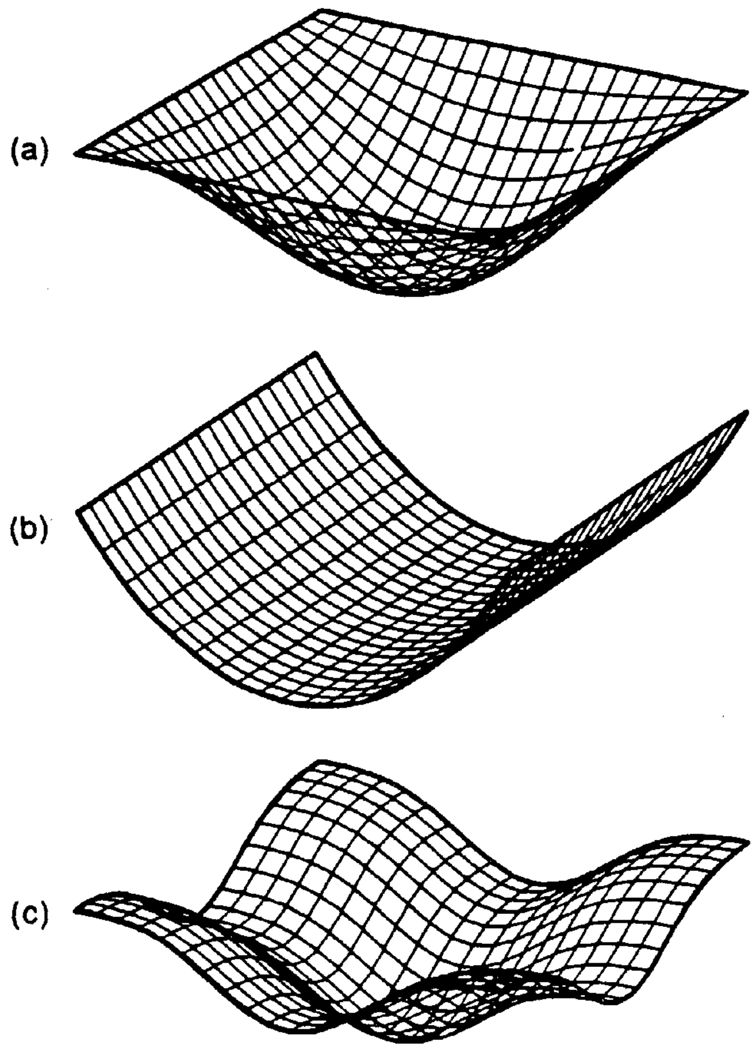


Figure 2.11: Assumed Deformed Shapes of Membranes in Unit Cell by (a) Molemkamp and Luger (1981); (b) Baldi and Nova (1984); and (c) Kramer et al. (1989)

good numerical solution to predicting membrane penetration volume change behavior. Figure 2.12 shows a comparison of the observed and analytically predicted membrane penetration behavior for coarse sand using the different membrane deformation models.

Empirical techniques and correlations have been developed by a number of investigators for evaluation of membrane compliance. Frydman et al. (1973) proposed one of the earliest empirical correlations between normalized membrane penetration (S) and mean particle size (D_{50}), previously shown in Figure 2.5.

Keikbusch and Schuppener (1977) presented a summary of their test results with a correlation between S and D_{50} in the form of a plot shown earlier as Figure 2.7. It is interesting to note that the relationship given by Keikbusch and Schuppener depicted a nonlinear slope between the normalized compliance (S) and $\log_{10}$ of soil particle size, with the values of normalized compliance increasing at an increasing rate with larger particle sizes. This trend of nonlinear correlation has since been concurred with by a number of subsequent investigations.

Ramana and Raju (1982) proposed an empirical equation for the estimation of unit membrane compliance (δV_m) as a function of mean particle size (D_{50}) based on the summarized data of previous investigators. This equation, applicable only to uniform soils whose D_{50} is between 0.08mm and 2.0mm, is given as:

$$\delta V_m = (D_{50}/80) \log(\sigma_3'/\sigma_{3,i}') \qquad \text{[Eq. 2.6]}$$

where:

δV_m = unit membrane penetration ($\Delta V_m / A_m$) in mm,

D_{50} = mean particle size (mm),

$\sigma_{3,i}'$ = initial effective confining stress (kN/m^2),

σ_3' = current effective confining stress (kN/m^2).

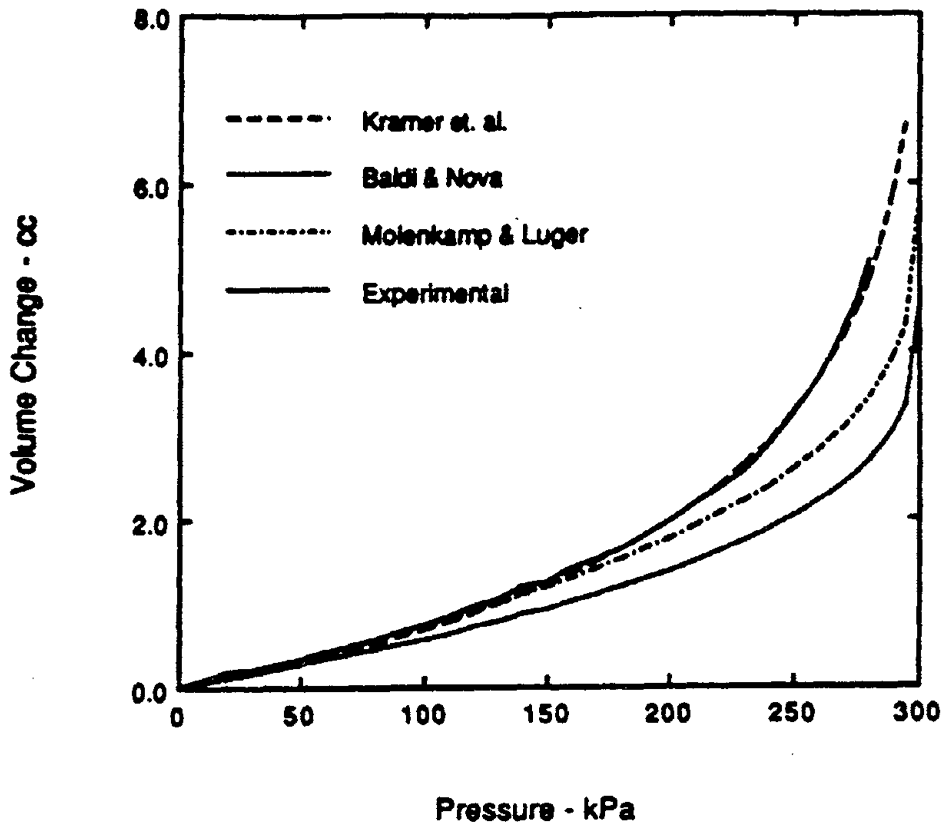


Figure 2.12: Comparison of Observed and Analytically Predicted Membrane Penetration Behavior for Coarse Sand:(Kramer and Sivaneswaran, 1989)

Baldi and Nova (1984) combined an analytical model with empiricle findings based on the data provided by previous investigations, including in their equation parameters pertaining to membrane thickness and geometric shape of material particles. The major drawback of that equation is that it is applicable only to uniform soils whose mean grain size is less than 0.5 mm and for effective confining stresses less than 400 KPa. The equation is given as:

$$\Delta V_m = 0.5(d_g / D) V_o [(\sigma_3{}' * d_g)/(E_m * t_m)]^{1/3} \qquad \text{[Eq. 2.7]}$$

where:

ΔV_m = membrane penetration volume change (cc),

d_g = mean grain size (mm),

D = sample diameter (cm),

V_o = initial sample volume (cc),

σ_3' = effective confining pressure (kPa),

Em = Young's modulus of the membrane (kPa), and

t_m = thickness of the membrane (mm).

These investigators deduced from their studies and this equation that membrane compliance and its associated effects on undrained tests could be reduced by increasing sample diameter, a suggestion that has been confirmed by a number of investigators. They also proposed a non-linear relationship between normalized penetration (S) as a function of the $\log_{10}$ of mean particle size of a material by applying the above equation (Equation 2.7) to a conventional sample size and typical testing membrane. That relationship is shown in Figure 2.13, and is similar to that given by Keikbusch and Schuppener (1977). Baldi and Nova argued that their mathematical correlation fit well with the available data of previous investigators, suggesting that the scatter of some data points may be the result of

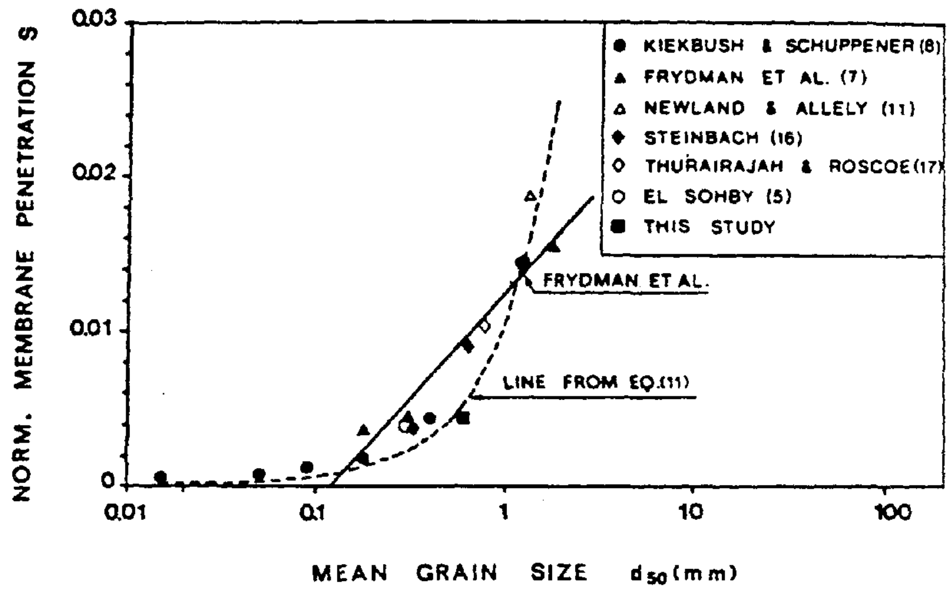


Figure 2.13: Proposed Relationship Between Mean Grain Size (D_{50}) and Normalized Penetration (S): (Baldi & Nova, 1984)

varied experimental testing techniques. It is important to note that for almost all of the investigations made to evaluate membrane compliance values up to that time, only uniformly graded materials had been tested. The few more broadly graded materials for which compliance measurements had been made did not fit their correlation nearly as well. Since that time, many more soil gradation types have been tested for volumetric compliance, showing that indeed, the relationships based on the data of uniformly graded soils did not hold for some of these other soil gradations.

In studies conducted as preliminary phases of this research investigation, reported by Anwar, Nicholson, and Seed (1989), a number of sandy soils with a wide range of grain sizes, from silty sands to coarse sands, and gradation types, including well graded and gap graded soils, were tested in order to evaluate their volumetric membrane compliance. Brief summary descriptions and gradation curves, as well as compliance measurement data for those soils tested are presented in Chapter 5. The compliance measurement tests performed on those soils was based on the "two sample scale model" method described in Section 2.2.2. The results of these compliance evaluations were combined with the results from other investigators using compliance measurement techniques judged likely to provide reasonably accurate results in order to derive a more widely applicable relationship between normalized compliance (S) and particle grain size. It was found that unit membrane compliance magnitude was much better correlated with smaller particle sizes (D_{20}) than with the mean grain size (D_{50}). This is not surprising, as membrane penetration is a phenomenon associated primarily with the interstitial voids between soil grains, and studies of both flow and "soil filter" characteristics have long recognized that soil particles finer than the mean grain size control the characteristics of these inter-particle voids.

It was also found that when soils were characterized by D_{20} grain size, sample gradation exerted a relatively minor influence on the value of normalized membrane compliance. Figures 2.14 and 2.15 show plots of the collective compliance measurement data using "representative" grain sizes of D_{50} and D_{20} respectively for silty and sandy soils. As can be seen in the relationship between S and D_{50}, there is a significant scatter of data points prevalent in the plot, which include the data points for most of the non-uniformly graded soils. Figure 2.15 shows the relationship between S and D_{20} for all of the same soils as in Figure 2.14. Clearly, the relationship is much more robust for a wide range of soil grain sizes and gradation types, and the use of D_{20} as a representative grain size for the characterization of compliance appears to remove much of the scatter from the relationship. Other representative or "characteristic" particle size indices were also tried (eg. D_{10}, D_{15}, D_{25}, etc.), but the correlation of S with D_{20} was found to be the strongest.

An equation was derived to "fit" the relationship between S and D_{20}, as illustrated in Figure 2.15, and is expressed as:

$$S = 0.0009\,[\log_{10}(D_{20}) + 2.1]^{3.57} \qquad \text{[Eq. 2.8]}$$

where:

S = membrane-compliance-induced volume change (cm^3) per square centimeter of membrane surface area per order of magnitude (per log-cycle) change in effective confining stress (σ_3'), and

D_{20} = the soil particle size such that 20% of the soil is "finer" (mm).

This empirically derived relationship provided a significantly improved basis for estimating normalized compliance values than had previously been proposed by

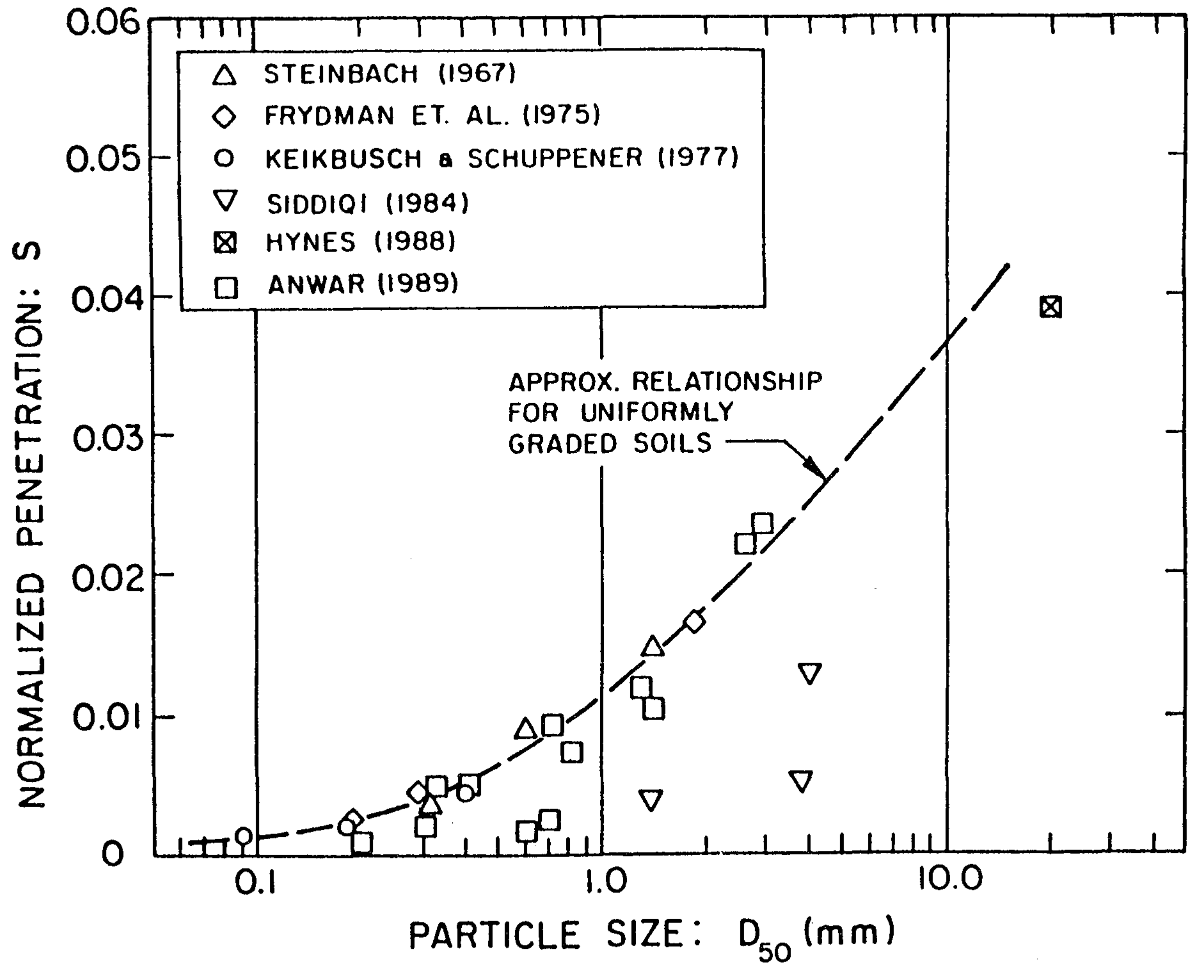


Figure 2.14: Relationship Between Normalized Unit Membrane Penetration (S) and D_{50}

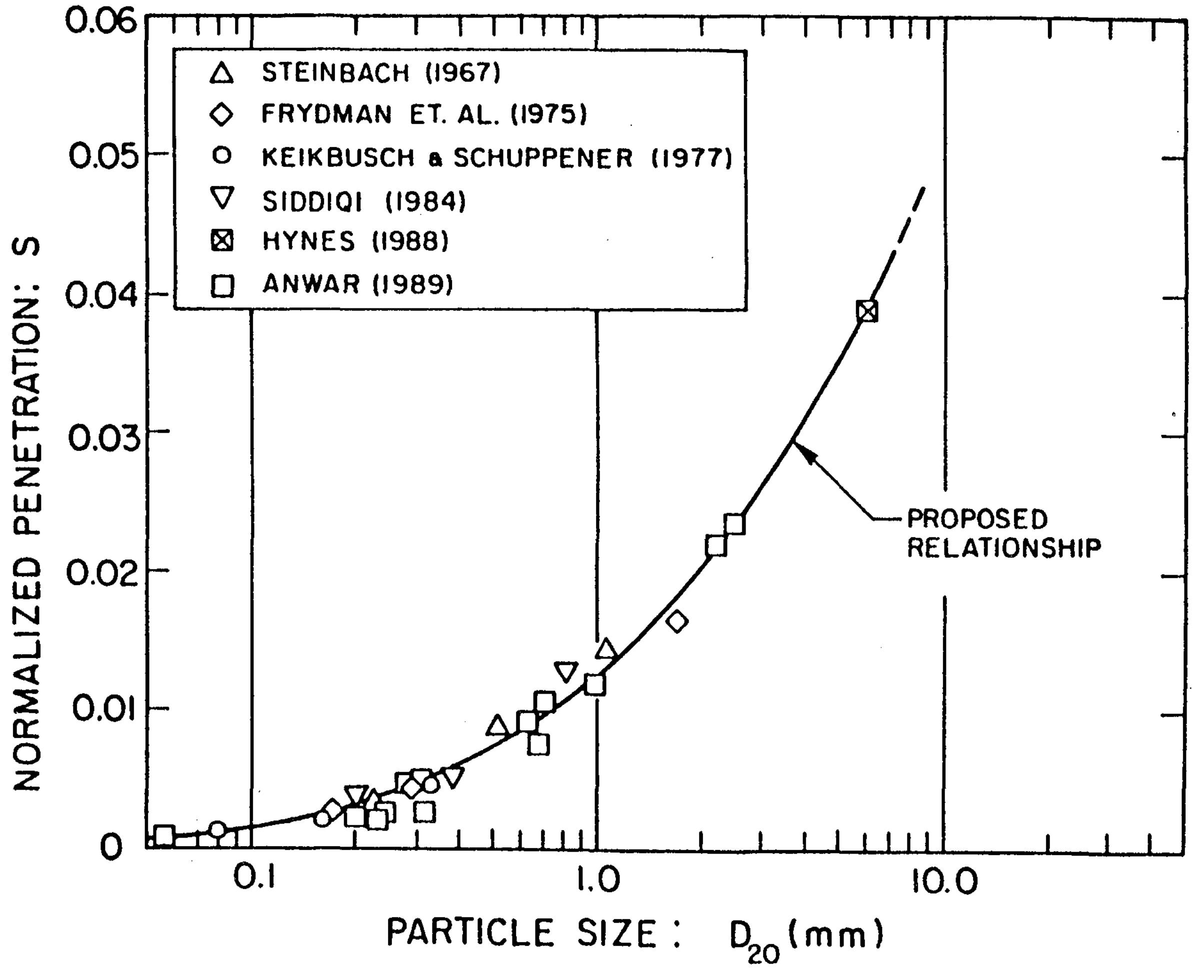


Figure 2.15: <u>Relationship Between Normalized Unit Membrane Penetration (S) and D_{20}</u>

correlations which were based on D_{50} grain size, which neglected the potential influence of non-uniform sample gradations.

Figure 2.16 schematically illustrates how the smaller particles in a soil gradation have a controlling influence on membrane compliance characteristics of more broadly graded soils. In Figure 2.16(a), a uniformly graded soil exibits considerably more penetration than the more broadly graded soil of Figure 2.16(b), where "finer" particles partially fill the peripheral sample voids between the larger soil grains.

2.3 Methods of Mitigating Compliance Effects

2.3.1 General

In order to obtain meaningful representative results for undrained tests of saturated granular materials, it would be most desirable to mitigate the adverse effects of membrane compliance from the tests. Numerous attempts at achieving this goal have been investigated over the last 25 years. The different types of methods that have been developed for mitigation of, or compensation for, membrane compliance effects can be grouped into three categories:

1. Use of larger sample sizes in order to reduce the the proportional impact of compliance effects.
2. Physical mitigation of compliance effects during actual undrained testing.
3. Post-testing correction of test results in order to compensate for compliance effects.

Each of these types of approaches that have been proposed, and the variations of each, will be reviewed in the following sections.

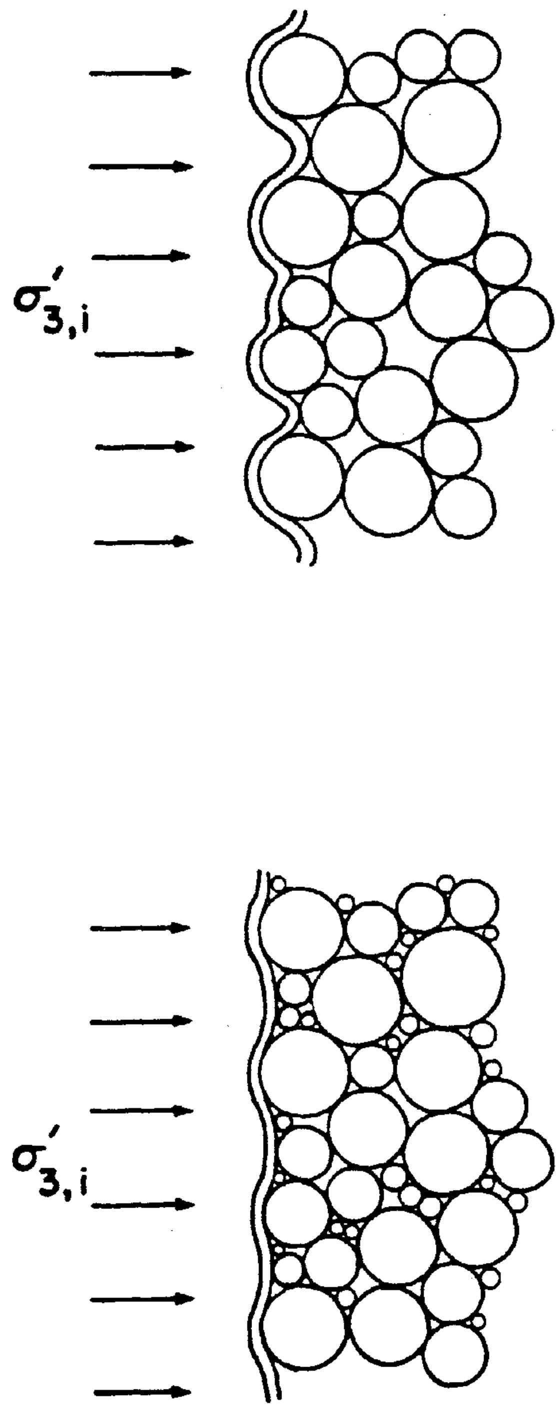


Figure 2.16: Schematic Illustration of the Controlling Influence of "Finer" Particles on the Membrane Compliance Characteristics of Broadly-Graded Soils

2.3.2 Use of Larger Sample Sizes

Conventional triaxial tests are typically performed on samples two to three inches in diameter, and provide good results for tests of silts and fine sands which experience minimal effects due to membrane compliance. For soils coarser than fine sands, the effects of membrane compliance are significant for these typical sample sizes. This results in unreliable and often unconservative test results for these coarser soils.

Newland and Allely (1959) first suggested that the effect of membrane compliance could be significantly reduced by testing larger diameter samples, and/or by using thicker (stiffer) membranes. These insightful suggestions were later tested by several investigators implementing a variety of methods.

The idea behind testing samples with larger diameters is to reduce the ratio of membrane surface area to sample volume, which in turn reduces the effects of compliance, as compliance is a function of the amount of membrane area with respect to the overall volume of the sample. As sample size is increased, the sample volume increases with the third power of sample diameter, while the sample membrane surface increases only with the square of the sample diameter. This geometric scale effect is demonstrated schematically in Figure 2.17. The practical applicability of this procedure was verified by a number of investigators including Wong, Seed and Chan (1975), Martin, Finn and Seed (1978), Chan (1978), and Seed, Anwar, and Nicholson (1989). The use of large scale testing apparatus (with sample diameters $\approx$ 12 inches), although too expensive for most applications, is one method to achieve representative tests for soils whose grain sizes do not exceed medium to coarse sand. However, for soils coarser than fine gravels, even these large scale testing facilities do not reduce the effects of membrane compliance enough to avoid significant errors in strength evaluations.

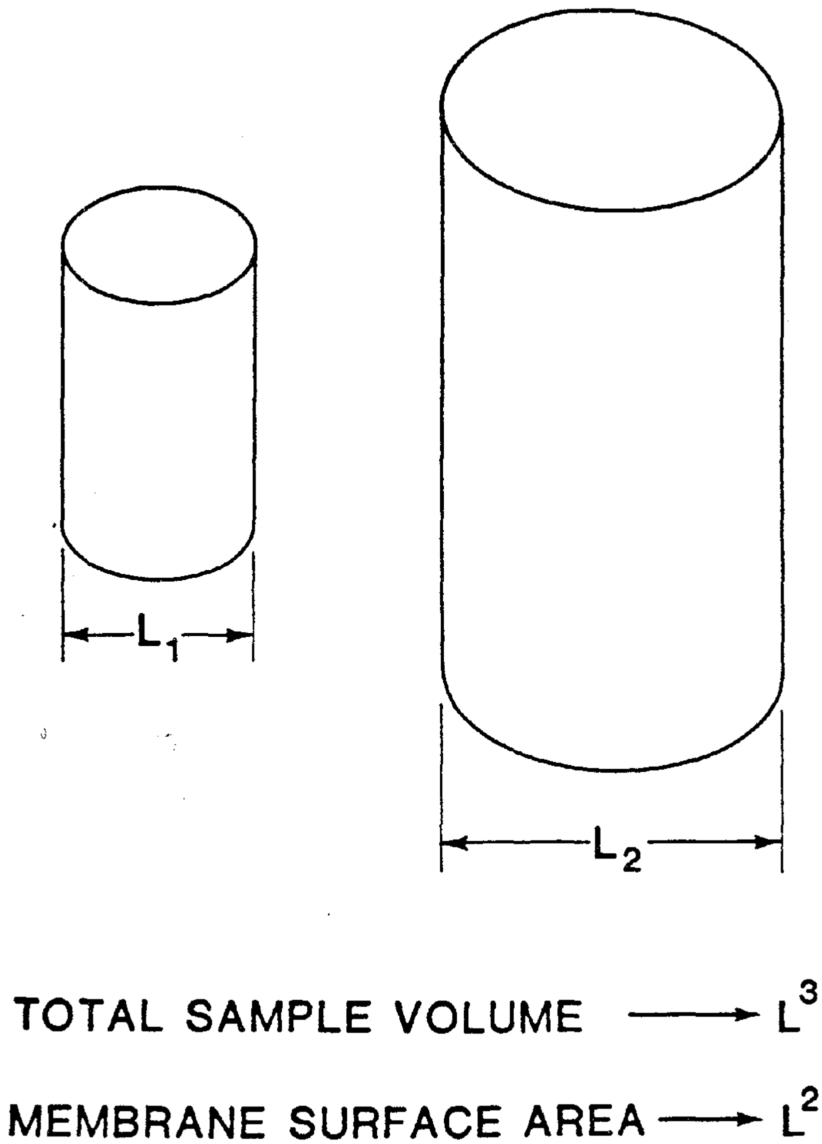


Figure 2.17: Scale Effects: Influence of Sample Size on the Ratio of Sample Volume to Membrane Surface Area

2.3.3 Physical Mitigation of Membrane Compliance During Testing

A number of approaches have been taken to achieve physical mitigation of membrane compliance effects during actual undrained testing. The effectiveness or usefulness of each method varies. The types of methods that have been attempted may be grouped into six basic categories as outlined in Table 2.1. This section presents a brief discussion of each of these methods.

(1) Use of protective plates adjacent to the rubber membrane used to confine the soil sample

A number of studies have been conducted in an attempt to mitigate membrane compliance effects by the use of a variety of protective plates placed between the membrane and soil sample. Chan (1972) experimented with a special membrane originally conceived for the purpose of preventing membrane rupture when testing rockfill at high confining stresses. The procedure involved covering the confined sample with high density polyethylene plates which would bridge over the external sample voids. It was noted that this method would also significantly reduce the effects of membrane compliance. Unfortunately, the polyethylene plates deformed plastically when significant confining stress was applied, and formed a stiff shell that caused serious errors in the measurement of axial load applied to the sample. Figure 2.18 illustrates this method.

Lade and Hernandez (1977) employed a system of overlapping brass plates placed between two membranes confining the soil sample, as illustrated in Figure 2.19(a). These brass plates were slightly curved in order to conform to the cylindrical shape of the sample. This procedure successfully mitigated much of the membrane compliance, but the amount of friction between the overlapping plates again caused the "armored membrane" to carry a great deal of the axial load

Table 2-1: <u>Methods for Mitigation of Membrane Compliance Effects During Undrained Testing</u>

1. Use of protective plates between the rubber membrane used to confine a soil sample and the sample soil grains.
2. Infilling of external peripheral voids in the membrane face at the condition of maximum membrane penetration prior to testing.
3. Filling of internal sample voids directly adjacent to the membrane prior to testing.
4. "Constant-volume" fully drained simple shear testing.
5. Maintaining a controlled confining cell volume outside of, and surrounding, a triaxial sample in order to preclude variation in membrane penetration.
6. Injection of water into otherwise "undrained" samples to offset pre-determined volume changes due to membrane compliance.

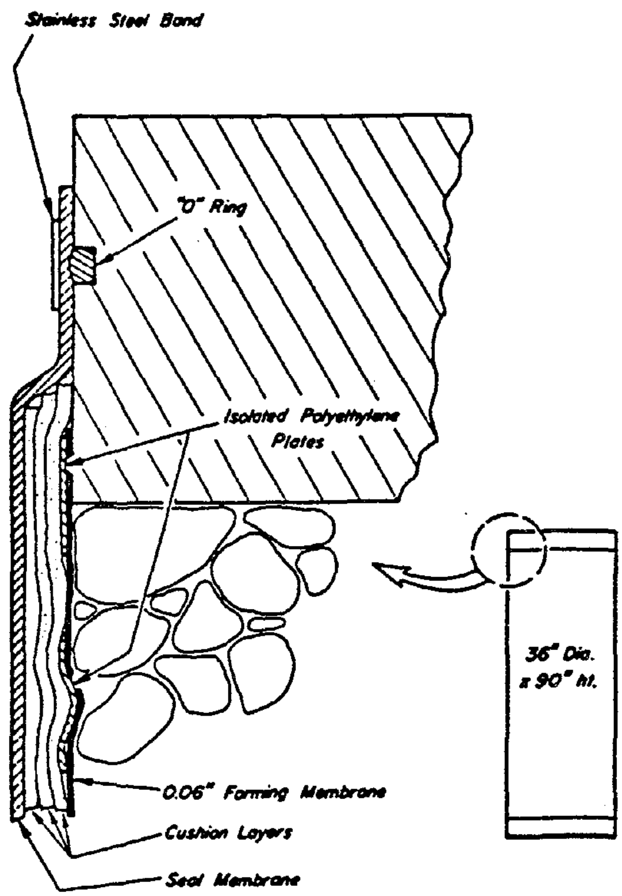
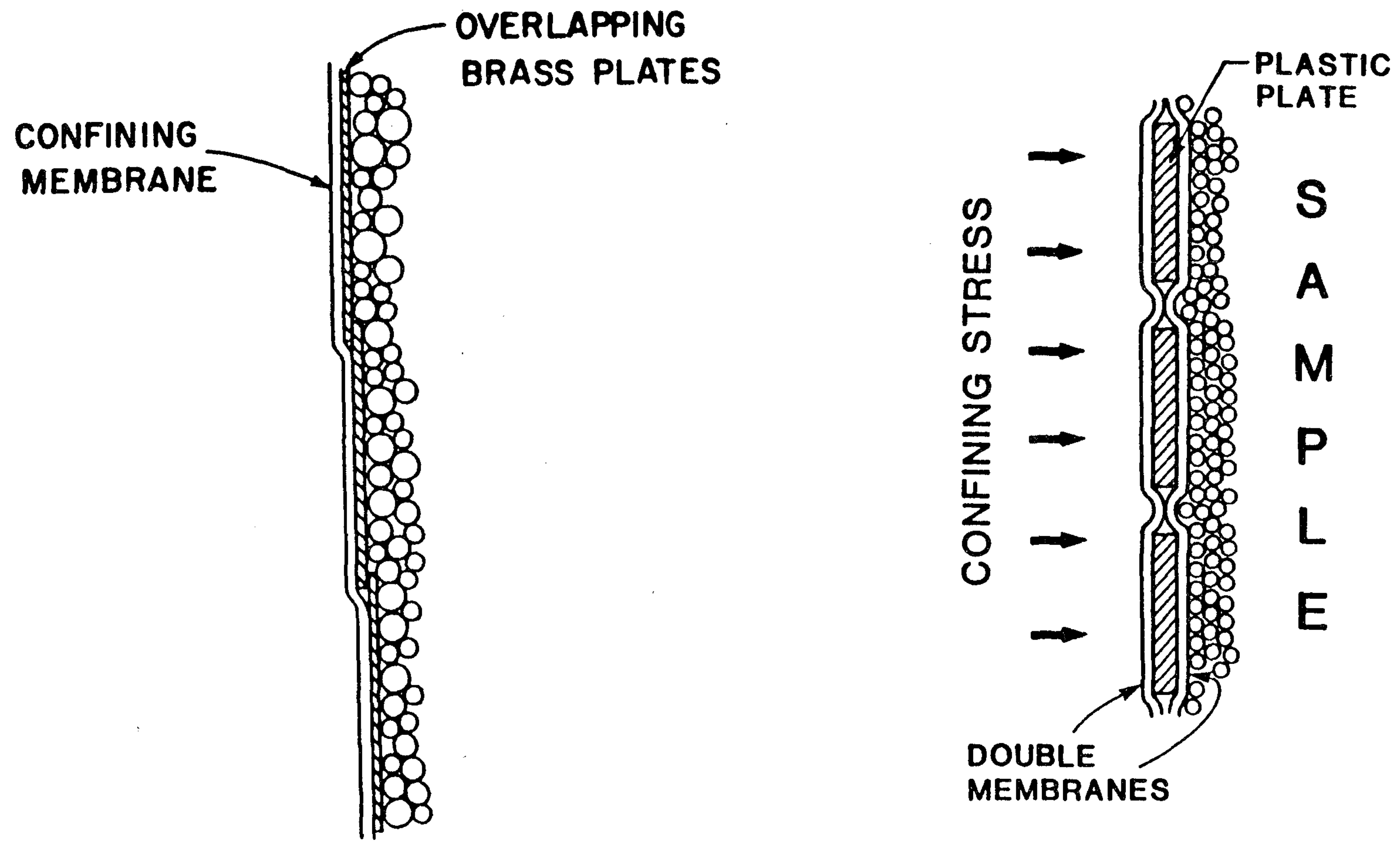


Figure 2.18: Special Membrane Developed for Testing Rockfills: (Chan, 1972)



(a) Overlapping Protective Plates

(b) Segmentally-Armored Membrane

Figure 2.19: Schematic Representation of the Use of Overlapping or Segmented Plates to Mitigate Membrane Compliance Effects

applied to the sample, which precluded the usefulness of this procedure. Raju and Venkataramana (1980) used polythene strips with silicone grease between the membrane and the sample in an attempt to overcome the frictional resistance encountered in previous investigations, but the results of their tests showed that membrane stiffness continued to contribute unacceptable bias to the test results.

Other investigators devised a variety of other plate configurations such as using segmentally armored-plates, as illustrated in Figure 2.19(b), to overcome the problems encountered with frictional forces. Unfortunately, while these configurations were found to greatly reduce the amount of membrane penetration where the plates were positioned, it was also found that they added an indeterminate ficticious contribution to the apparent strength and stiffness of the samples.

(2) Infilling of External Membrane Voids

Another method used in attempting to negate membrane compliance errors involved infilling the external peripheral voids in the membrane face at the condition of maximum membrane penetration. This method is schematically illustrated in Figure 2.20(a). Chan (1982) attempted to minimize the effects of membrane compliance by coating the outside of the membrane with liquid latex rubber. It was noted that during cyclic loading of 2.8-inch diameter samples, pore pressures were built up much faster and to higher pore pressure ratios than for specimens tested without the coated membranes. However, the effects that the coating had on membrane compliance could not be accurately quantified due to the wide scatter of test results. The scatter of data was attributed to an inconsistent thickness of the rubber coating which was not closely monitored.

Similar testing was performed by Raju and Venkataramana (1980), Wong (1983), and Torres (1983), in which some quantified results were obtained. These

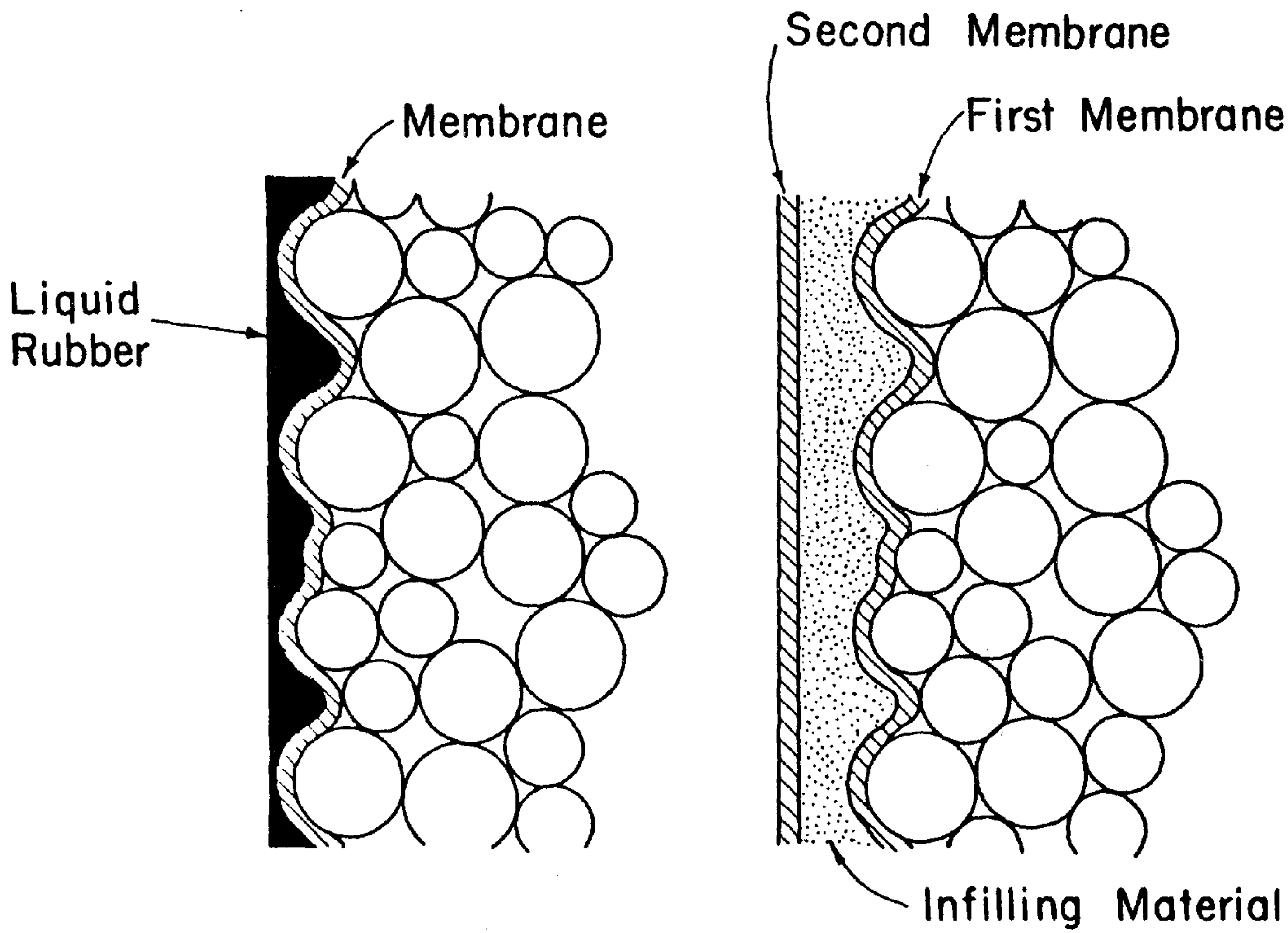


Figure 2.20: <u>Infilling of External Voids in the Membrane Prior to Undrained Testing</u>

tests showed that although the use of external rubber coating reduced membrane compliance effects significantly, corresponding calculated correction factors were much lower than those that had been predicted by Martin et al. (1978). This was an indication that the rubber coating did not completely eliminate the effects of membrane compliance and/or that the formation of a thicker, stiffer confining membrane contributes an indeterminate fictitious strength and stiffness to the sample.

Attempts to mitigate membrane compliance by using a layer of clay or sand between two confining membranes were made by Evans and Seed (1987) and Hynes (1988), as well as a number of unpublished efforts, but the results of these tests showed that this method would not prove to be useful in reducing the problem, as it was again accompanied by an added undesireable "ficticious" strength. This method is illustrated in Figure 2.20(b). Additional unpublished efforts have also been made that involved filling external sample membrane voids with other infilling materials between two membranes, and applying a separate back-pressure to the infilling material in order to minimize the effective confining stress applied to the material. But this did not appear to eliminate the problem, and in addition, sample preparation and testing procedures were excessively complicated and difficult.

(3) Filling of Internal Peripheral Sample Voids

Keikbusch and Schuppener (1977) experimented with coating the inside surface of confining membranes with liquid rubber so that it would penetrate the peripheral sample voids when confining pressure was applied. The liquid rubber was allowed to set under the initial confining stress before testing. The results of this method were to reduce the volumetric compliance to only 15% of the corresponding value for those samples tested without rubber coating. The effects of the additional thickness and strength added to the membrane on axial load measurements were not determined for these tests, which may invalidate this type of test for use in measuring actual sample strength. The problems arising from thickening the membrane with irregular amounts of rubber would be even more of a problem for gravelly soil specimens since the amount of rubber coating may have to be over an inch thick depending on the grain size distribution of the soil. Figure 2.21(a) illustrates the filling of internal peripheral sample voids with liquid rubber.

Raju and Venkataramana (1980) also tried a membrane system that involved filling of internal sample peripheral voids. In this study, a thin film of polyurethane was spread on the inside of the membrane prior to constructing the sample. They reported that by using coated membranes, compliance values were reduced to between 15% and 25% of the values measured when using uncoated membranes.

Some additional unpublished investigations have been performed at the University of California at Berkeley and at the U.S. Army Corps of Engineers Waterways Experiment Station in which soils have been used as an infilling material between the sample soil particles and the confining membrane, as illustrated in Figure 2.21(b). Once again it was found that a "ficticious" strength and stiffness was added to the sample, and furthermore, the infilling materials reportedly tended to "pump" into the interior of the samples during undrained loading, so that compliance mitigation became less effective as testing progressed.

In a recent study, Evans and Seed (1987) investigated the use of sluicing gravels with sand. This study had multiple purposes, including an attempt to mitigate membrane compliance effects. Sluicing of gravels with sand is not a new concept in itself, as it was conducted as part of the construction procedures for the Malpaso Canyon Dam in Peru and the Aswan High Dam in Egypt. However,

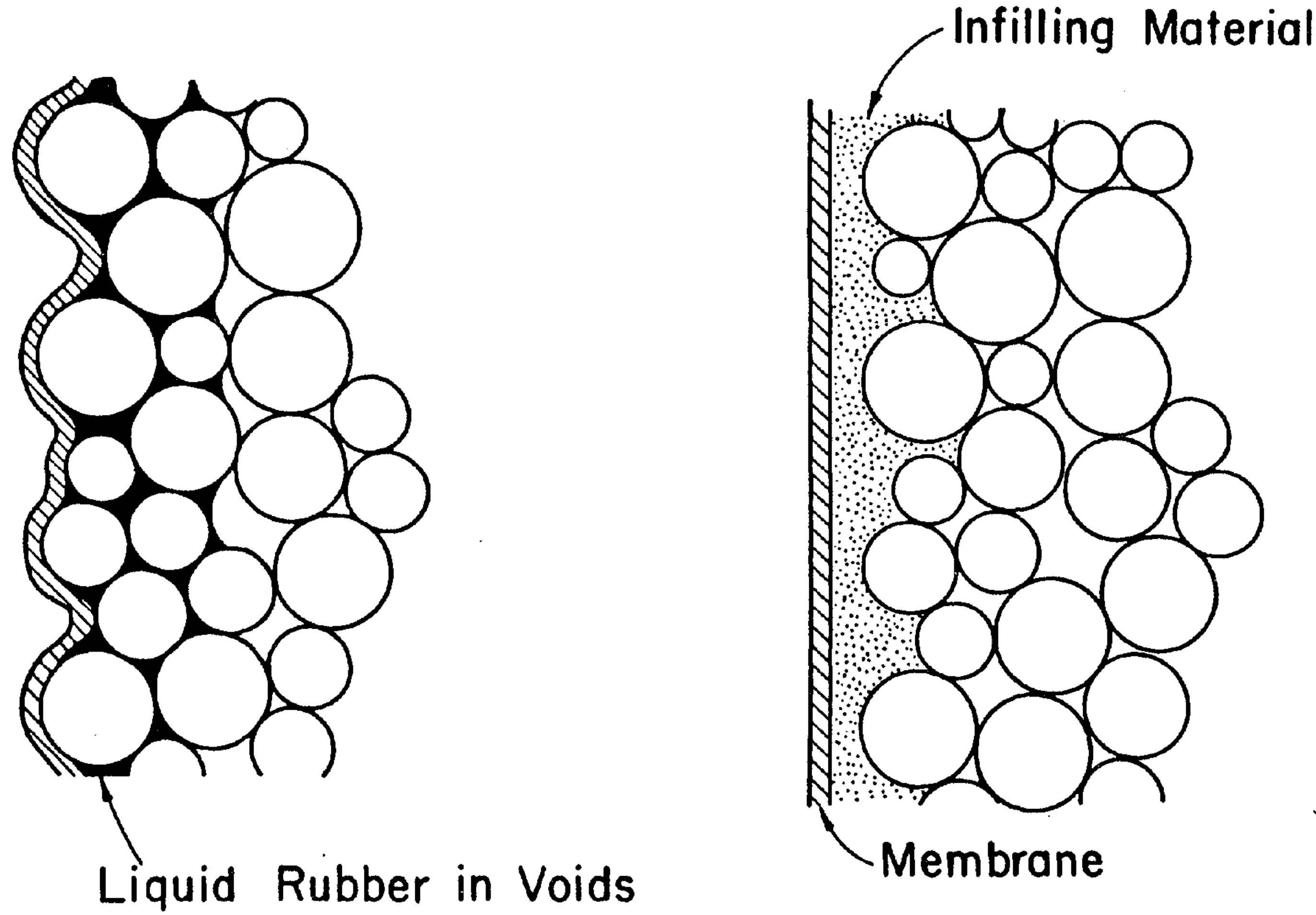


Figure 2.21: <u>Filling of Internal Peripheral Sample Voids</u>

sluicing had not previously been explored as a tool to mitigate compliance effects in undrained testing of soils. The idea behind this investigation was that by sluicing sand into the voids within gravel specimens, the sample/membrane contact would be much smoother, and therefore would exhibit greatly reduced membrane compliance during undrained cyclic loading. Clearly, this method is not applicable to testing of finer grained soils, but many of the methods described previously could not be realistically adapted to the testing of gravels. It was believed that the sluicing material was "loose" enough so as not to add significant strength to the samples and since the sluicing material was nearly continuous throughout the sample, there would be no tendency for the material to "pump" towards the interior of the sample as had been noted in earlier experiments.

Sluicing was accomplished by first constructing the complete gravel sample to a desired density and structure, and then replacing the water in the voids with the sluicing material. It is suggested that by constructing the samples in this manner, the individual gravel particles form a continuous, load carrying structure, while the sluiced material merely fills the void spaces in a loose state without adding to the strength of the sample.

Undrained cyclic triaxial tests were performed on 2.8-inch and 12-inch diameter samples of sluiced and unsluiced uniformly-graded gravel samples at various densities. From the results of these tests, it was concluded that sluicing of gravel specimens was a viable method of significantly reducing the effects of membrane compliance, although it did not completely mitigate compliance. It was noted that the sluicing sand may have had an effect on increasing cyclic strength by prohibiting grain rearrangement. But, despite the claim of the authors that those effects had only a secondary effect, this hypothesis has not been proven conclusively. Figure 2.22 shows some of the results obtained by using sluicing sand to reduce the effects of membrane compliance for gravel specimens.

(4) Constant Volume Fully-Drained Simple Shear Testing

Pickering (1973) and Moussa (1973, 1975) suggested the use of drained constant volume cyclic simple shear tests to obtain correct cyclic strength evaluations by eliminating the pore pressure generation which encompasses the errors induced by membrane compliance. This procedure, which is similar to the one first used by Bjerrum and Landua (1966) to investigate the behavior of quick clays, involves performing drained cyclic simple shear tests on samples that are kept at constant volume by locking the load ram once the initial vertical load has been applied. As the sample densifies during cyclic shearing, the sample tends to "drop away" from the locked top cap, thus reducing effective stresses. These changes in effective stresses are assumed to represent reductions in effective stresses that would occur as a result of pore pressure increases during undrained tests. A schematic representation of this testing setup is illustrated in Figure 2.23.

This technique of obtaining "representative" test results was examined by Finn and Vaid (1977) by comparing the results of conventional undrained cyclic simple shear tests to drained constant volume tests on similar sand. The result of the investigation was that liquefaction resistance was lower for the drained constant volume tests, indicating that compliance effects may have been successfully mitigated. Although the evidence to support this conclusion is very limited, this method may potentially have a future in mitigating membrane compliance effects once the procedure is verified. Unfortunately, like several of the other possible mitigation methods, this procedure is not likely to be adapted to performing representative tests on coarse grained soils due to the size limitations of the testing equipment.

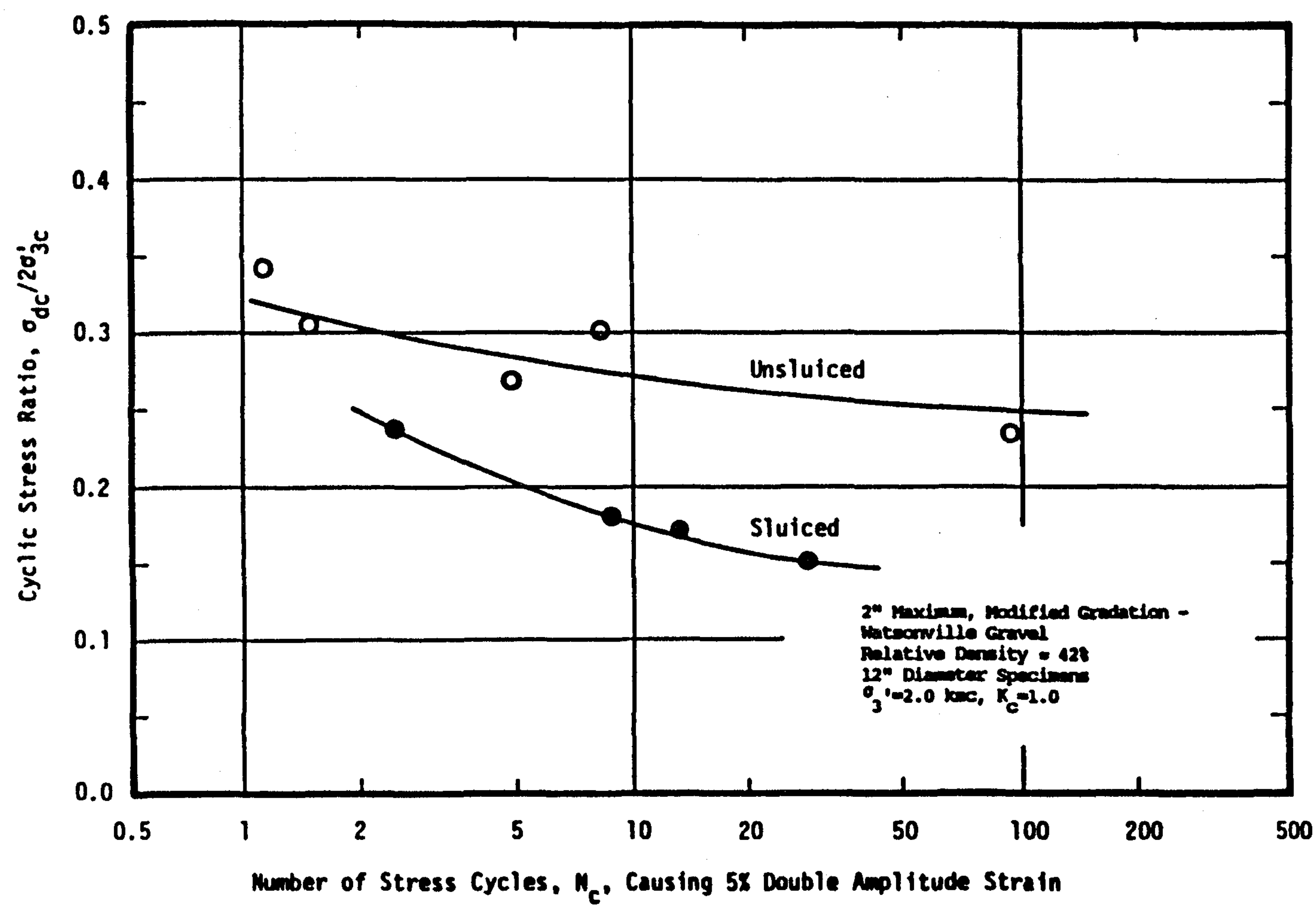


Figure 2.22: Comparison of Relationship Between Cyclic Stress Ratio and Number of Cycles Causing 5% Double Amplitude Strain For Sluiced and Unsluiced Samples:(Evans and Seed, 1987)

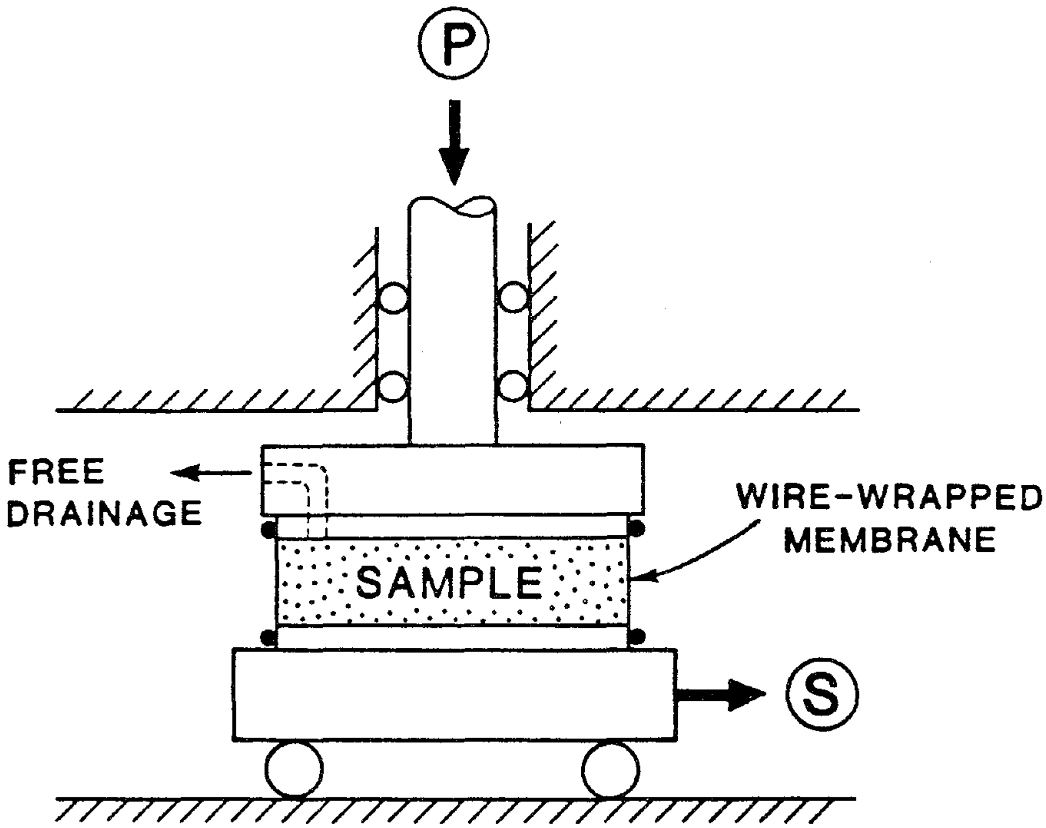


Figure 2.23: Constant-Volume Fully Drained Simple Shear Testing (Schematic)

(5) Maintaining a Controlled "Constant" Cell Volume

Some recently completed unpublished investigations, including one at the University of California at Berkeley, have investigated the possibility of preventing membrane compliance effects by controlling the confining cell volume that surrounds the soil sample being tested. This procedure is schematically illustrated in Figure 2.24. The idea behind this method is to offset the volume change in the cell due to the ingress or egress of the loading ram, by adding or removing an equal volume of water from the cell. If this is accomplished, then the volume of the sample itself undergoes no change, resulting in a true constant-volume test.

The studies employing this method have shown that, due to several technical difficulties, the necessary accuracy of controlling the required cell fluid volume cannot be accomplished, thus prohibiting the use of this method as a viable alternative for performing representative non-compliant tests.

(6) Compliance Mitigation/Compensation by Injection

A relatively new approach to mitigating membrane compliance, involves compensating for the compliance-induced sample volume changes by injecting and/or removing a volume of water equal to that amount previously pre-determined as a volumetric error for the current effective confining stress.

Compensated undrained tests were first performed by Raju and Venkataramana (1980) to assess the magnitude of pore pressures that would develop if the volumetric compliance was eliminated. Compensation was accomplished by manual injection of a volume of water equal to the amount of pre-determined volumetric compliance. The amount of volumetric compliance was determined using the modified central rod method as described by Raju and Sadasivian (1974). The additional fluid added to the system changes the effective confining pressure which causes additional volumetric compliance which , in turn,

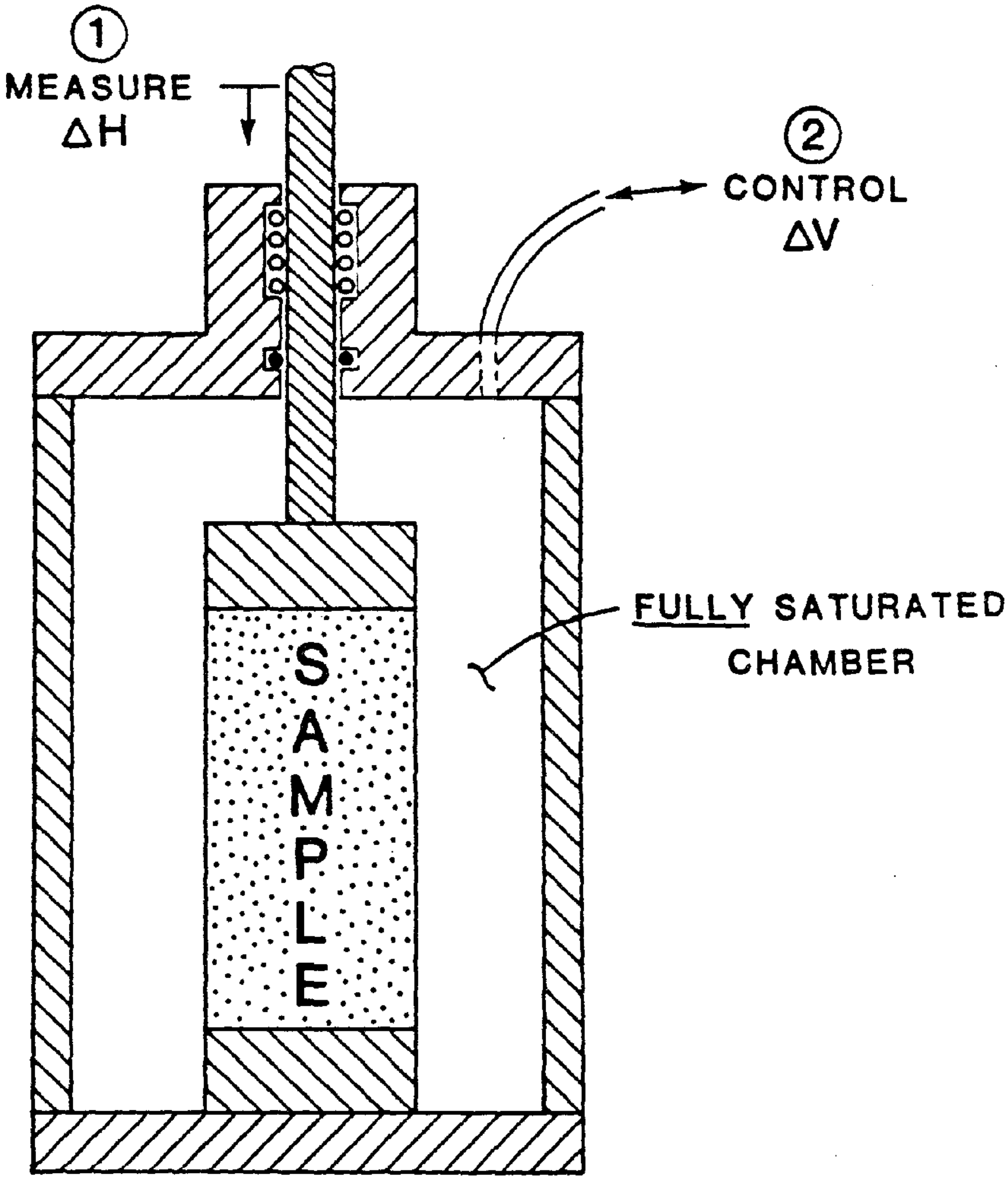
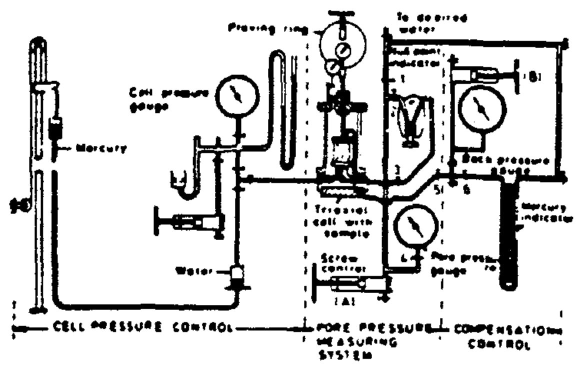


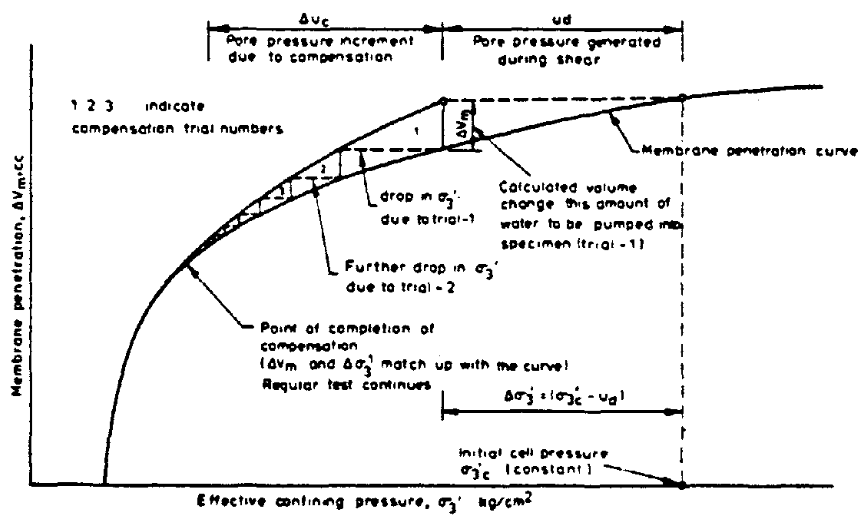
Figure 2.24: Schematic Illustration of Membrane Compliance Prevention by Controlling Confining Cell Volume

must be corrected for. Figure 2.25(b) illustrates the iterative correction process. This process continues until the pore pressure reaches a constant value which requires no further correction. This step-by-step procedure proved to be effective for obtaining more accurate test results, in which membrane compliance effects were significantly reduced. Unfortunately, the time involved to make the corrections for each increase in stress increment, as well as the lack of continuity between increments, was a serious drawback to the procedure.

The manual injection-correction method was further investigated by Ramana and Raju (1981) who applied it to monotonic loading and unloadings, and to cyclic tests, performing the cyclic tests at a rate of one cycle per minute in order to allow enough time for the manual corrections. Figure 2.25(a) shows the test apparatus used and Figure 2.25(b) illustrates the procedure used to compensate for compliance effects at each step of the test. Figures 2.26 and 2.27 show test results for undrained monotonic and cyclic tests performed on uniformly graded medium sand with and without implementation of the manual injection-correction procedure. Even though the tests were run slowly, the time required for the tedious stepwise manual injection process limited the number of iterations that could be performed during each step of the test. The iterative correction process was completed only two or three times during each monotonic loading test, and two to three times during each cycle of cyclic tests, so that compliance corrections were probably not complete. Further drawbacks to the manual injection-correction process were that: (a) The injection-correction only occurs at irregular and widely spaced intervals during testing so that sample volume is "correct" only at certain intervals during each test, (b) The periodic injection process necessitates injection of relatively large water volumes causing relatively large and sudden increases in pore pressure which may represent a significant loading mechanism whose effect



a) Test Set-Up



b) Stepwise Injection Process

Figure 2.25: Ramana and Raju's Manual Injection-Correction Procedure

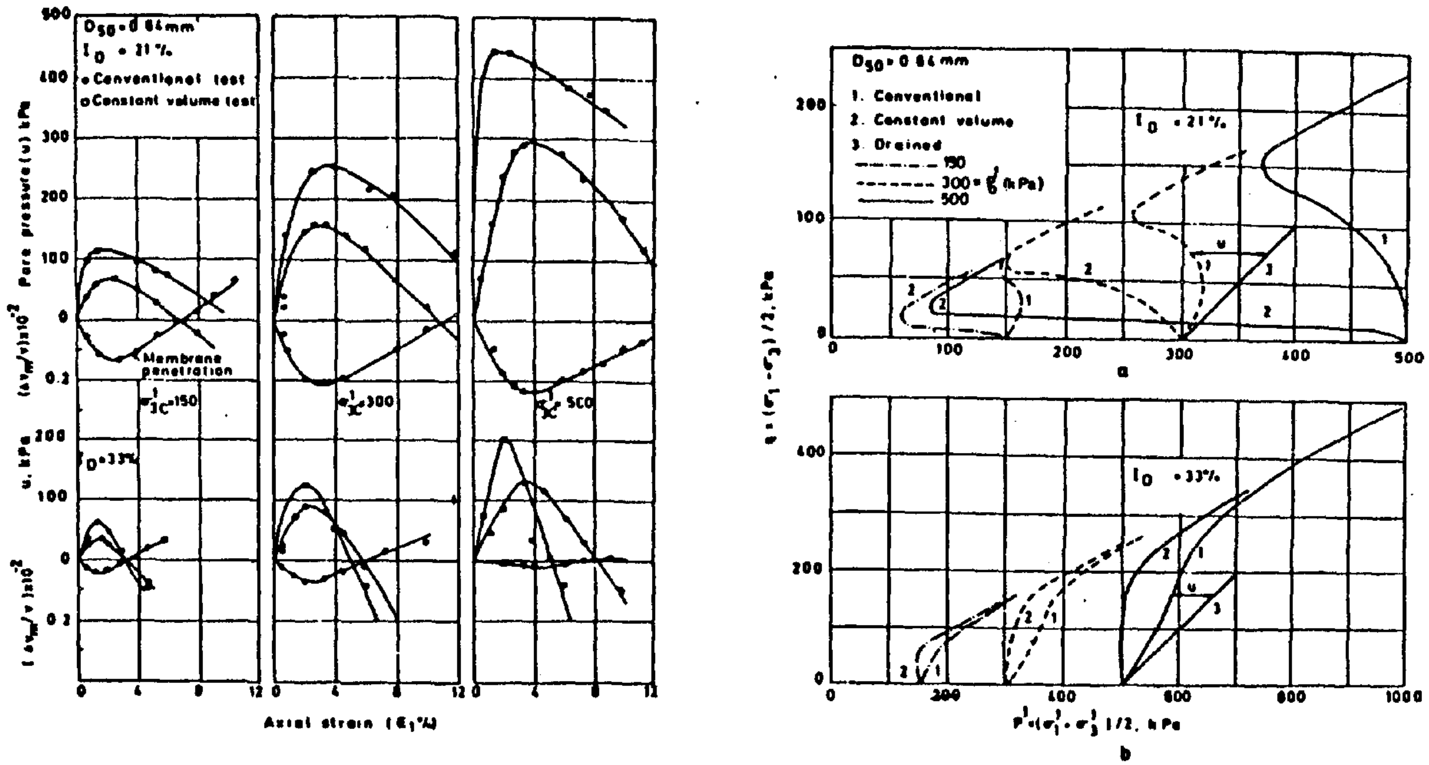


Figure 2.26: Undrained Monotonic Triaxial Test Results With and Without Manual Injection-Correction:(Ramana & Raju, 1981)

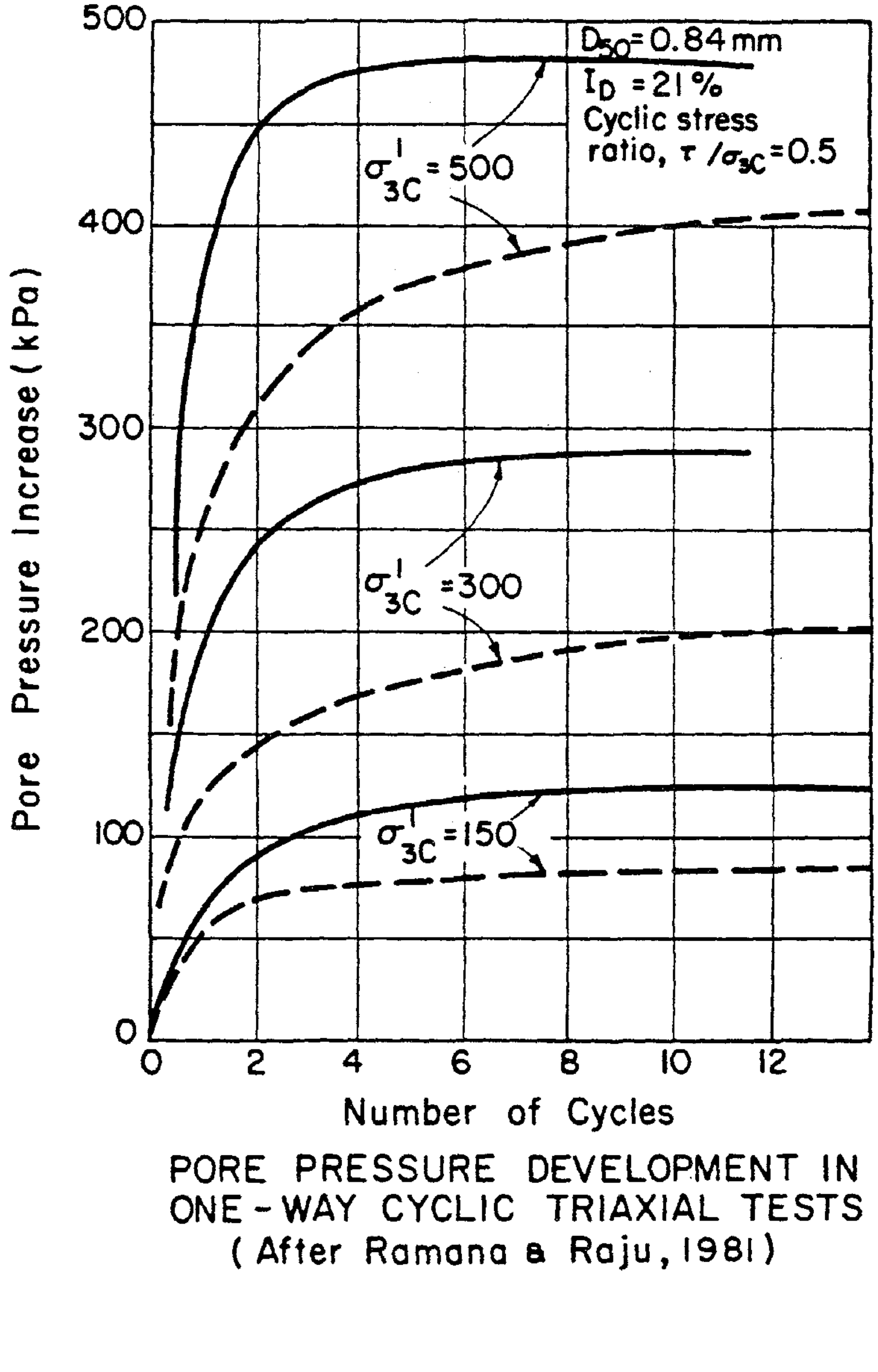


Figure 2.27: Undrained Cyclic Triaxial Test Results With and Without Manual Injection-Correction (after Ramana & Raju, 1981)

on sample behavior is unknown, and (c) due in part to the aforementioned problems, Ramana and Raju were not able to quantitatively evaluate the effectiveness of the overall mitigation process. With these considerations in mind, the test results still showed that the procedure greatly increased the rate of pore pressure generation and apparently reduced the resistance to cyclic loading, which indicated that this procedure was a promising development in the research of physical mitigation of membrane compliance during undrained testing.

Laboratory techniques for performing continuous computer-controlled injection/removal corrections for conventional triaxial tests were developed by Seed and Anwar (1986), and Tokimatsu and Nakamura (1986). The pre-determination of volumetric membrane compliance was demonstrated to be reliably repeatable so that it could be characterized in such a manner that computer-controlled injection/removal could be performed based on monitored changes in the effective confining stress on the sample.

The computer-controlled membrane compensation system devised by Tokimatsu and Nakamura (1986) is illustrated schematically in Figure 2.28. Compensation was accomplished by adjusting the measured specimen volume by pneumatic pressure control of the monitored volumes in burettes, based on membrane compliance error measurements performed prior to undrained testing. Compliance measurements for use with these compensation tests were performed by the single sample unloading method described by Vaid and Negussey (1984).

The computer-controlled system used by Seed and Anwar (1986) consisted of an IBM PC-AT microcomputer and a GDS digital pressure/volume controller (injection piston) which was modified to bypass its internal circuitry in favor of direct control of the injection system by the microcomputer. A schematic

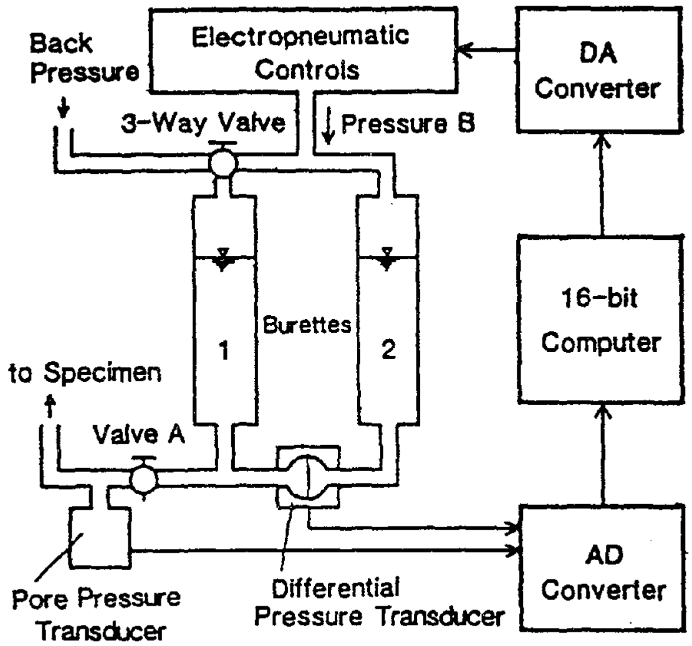


Figure 2.28: Schematic Diagram of the Pneumatic Membrane Compensation System: (Tokimatsu and Nakamura, 1986)

illustration of the computer-controlled injection system developed by Seed and Anwar is presented in Figure 2.29.

Implementation of the injection-mitigation system used by Seed and Anwar for undrained triaxial tests first involved pre-determining the unit membrane compliance for a given soil at a given density, for which the two sample scale model method (described in Section 2.2.2) was employed. The normalized unit membrane compliance S (cm^3 per cm^2 of membrane area per log cycle change in σ_3') was derived from the compliance measurements, and, along with sample dimensions, was entered into a computer program which would calculate the appropriate amount of water to inject to or remove from the sample for monitored changes in effective confining stress during undrained loading. Both monotonic and cylic tests were performed with and without employment of the computer-controlled injection-correction system on samples of two uniformly graded sands. Plots of the test results for those tests are shown in Figures 2.30 through 2.34. Individual test results for the tests performed on Monterey 16 sand are presented in Chapter 4.

Mathematical corrections of volumetric error based on the unit membrane compliance curve were used to theoretically "correct" the unmitigated monotonic test results (critical state conditions), giving support to the correctness of the test results obtained for those tests performed with compliance-mitigation. A plot of the monotonic test results performed with injection-mitigation as compared to the mathematical corrections of the unmitigated tests for Monterey 16 sand is shown in Figure 2.34.

2.3.4 Theoretical Post-Test Corrections

Post-testing corrections began as empirical corrections based on relationships developed to compare various sample characteristics and the amount

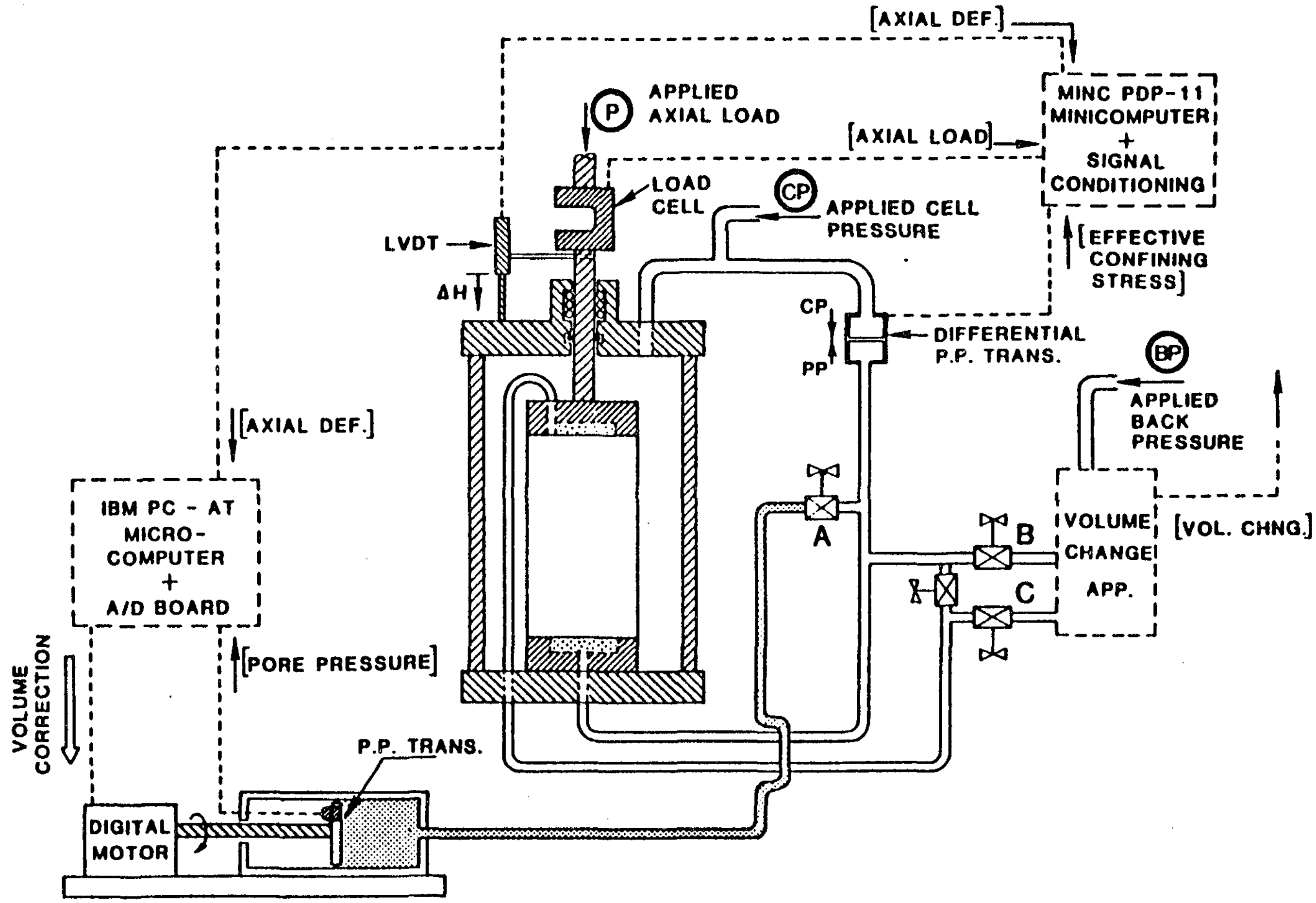


Figure 2.29: Schematic Illustration of Computer-Controlled Injection/Removal System for Membrane Compliance Mitigation During Undrained Triaxial Testing

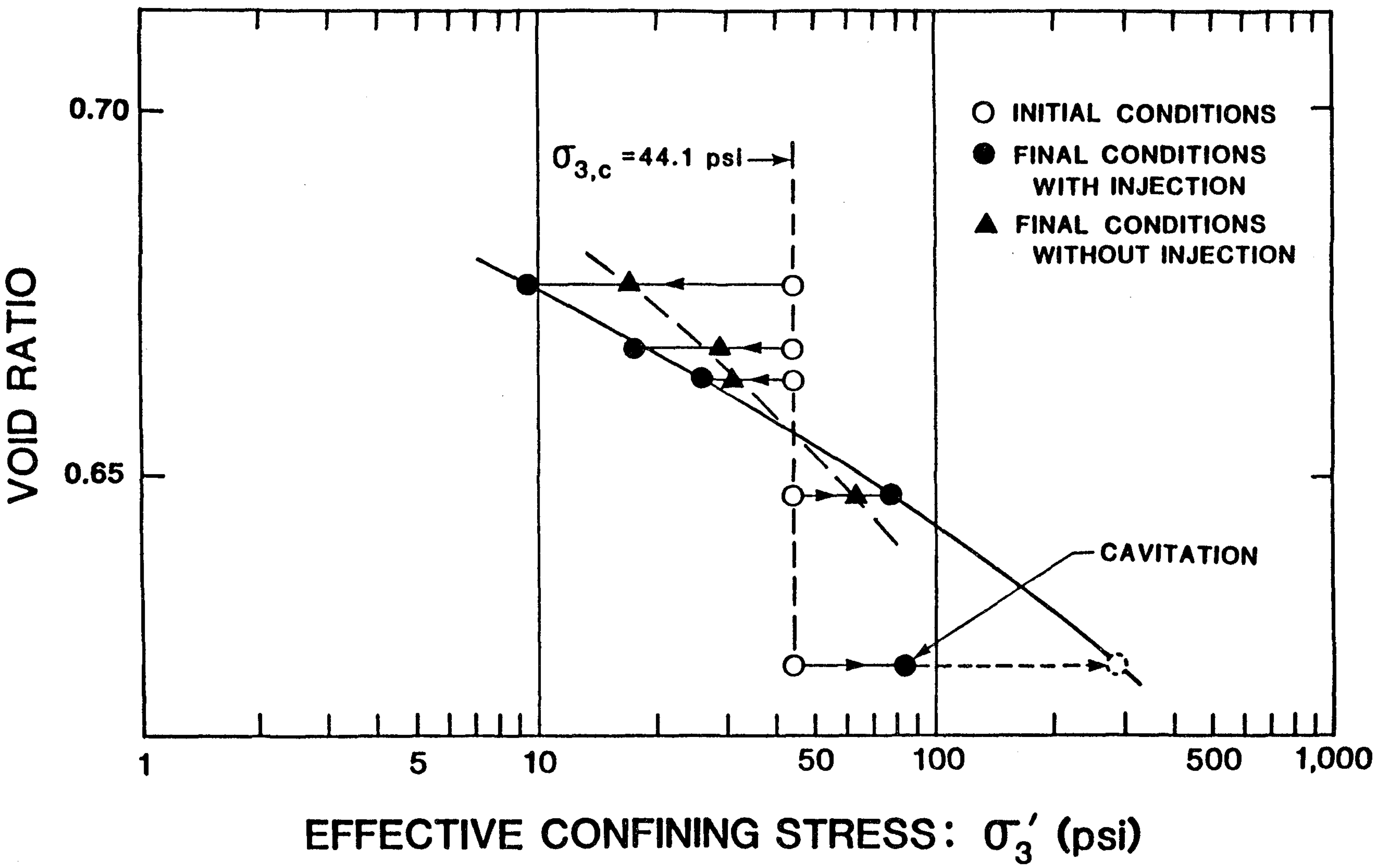


Figure 2.30: Critical-State Plot for $\overline{\text{IC-U}}$ Triaxial Tests on Monterey 16 Sand With and Without Membrane Compliance Mitigation

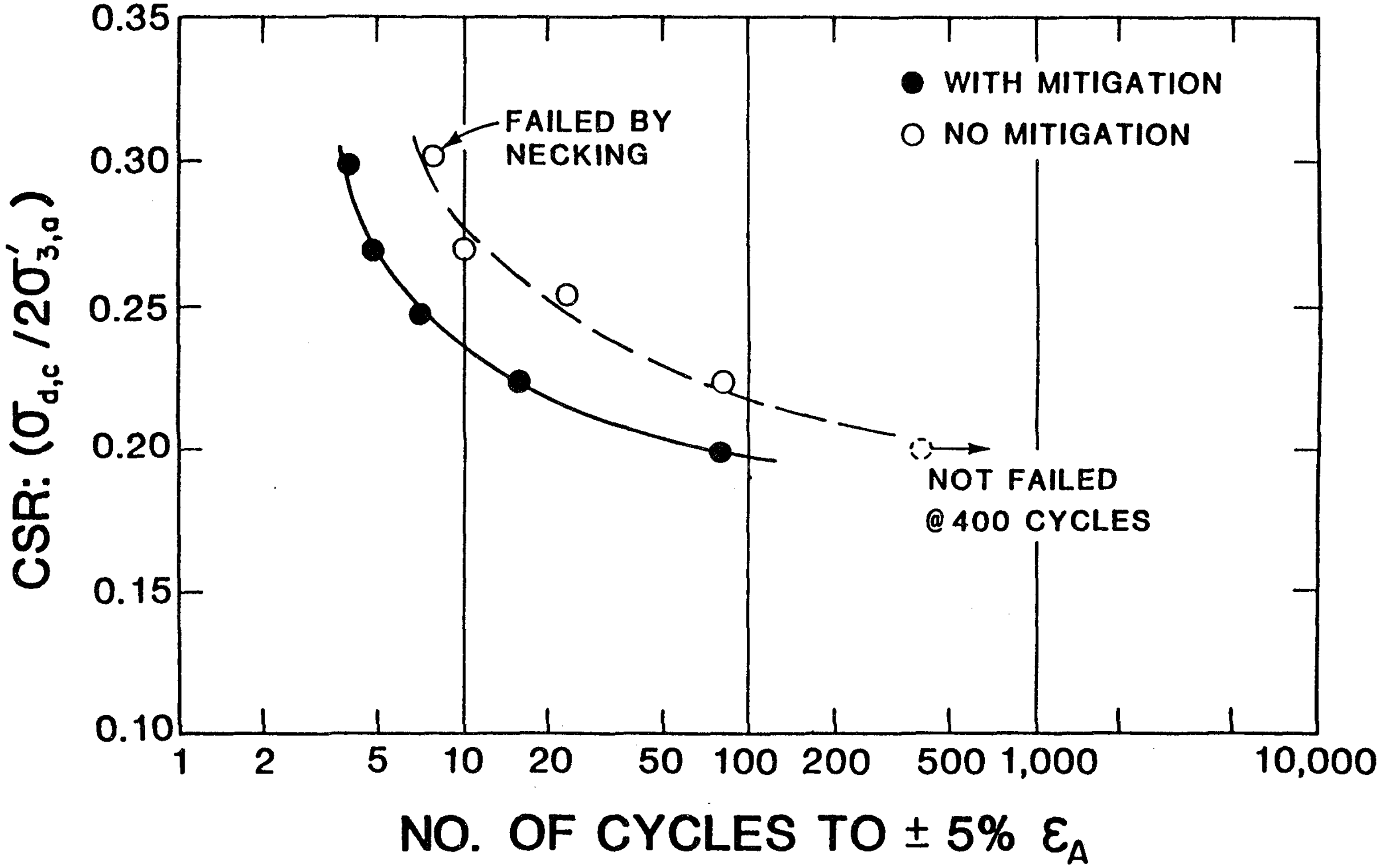


Figure 2.31: Results of Isotropically Consolidated-Undrained Cyclic Triaxial Tests on Samples of Monterey 16 Sand at $D_R \approx 55\%$ (With and Without Mitigation of Membrane Compliance Effects)

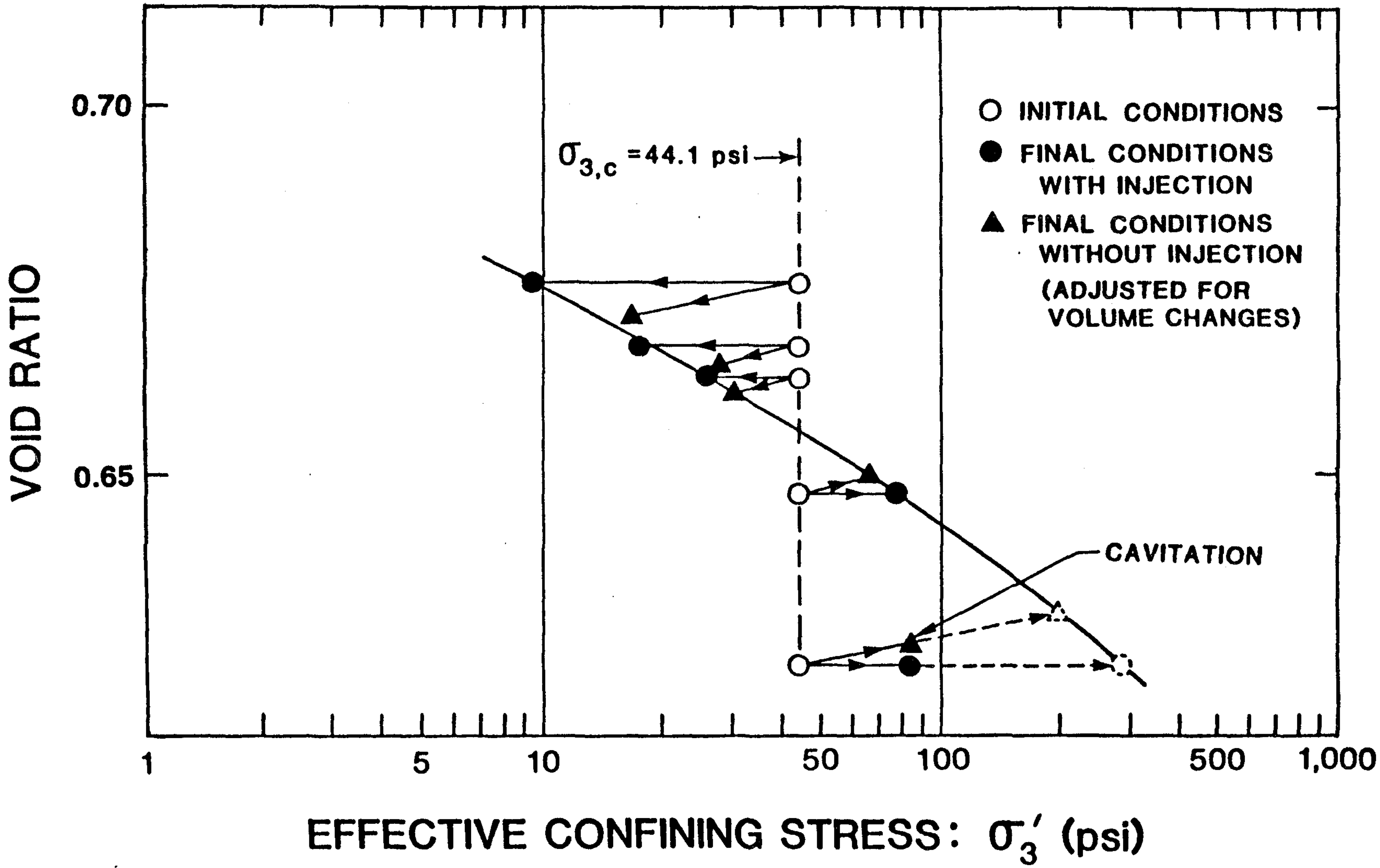


Figure 2.34: Critical-State Plot for $\overline{\text{IC}}$-U Triaxial Tests on Monterey 16 Sand With and Without Membrane Compliance Mitigation (With Correction for Membrane Compliance-Induced Volume Changes)

of membrane compliance that could be expected for those characteristics at different effective confining stresses. Empirical equations for expected compliance magnitudes as functions of soil characteristics have been made by several investigators including Steinbach (1967), Ramana and Raju (1982), and Seed et al. (1989), and are continually being modified and refined as more updated testing data becomes available.

An early theoretical membrane compliance correction developed by DeAlba, Chan, and Seed (1975), was based on the analytical model of soil behavior proposed by Martin, Finn, and Seed (1975). This correction method involves the relationships between shear modulus, sample volumetric strain and shear strain from equivalent drained tests.

Assessment of the correction factor was determined by performing hydrostatic rebound tests on large-scale shake table samples of different heights. Volumetric strain due to membrane compliance was determined by extrapolating sample height to zero, at which point all of the change was due to membrane compliance.

A theoretical procedure for correcting conventional test results subsequent to the completion of the tests was developed by Martin, Finn, and Seed (1978). The procedure, which proposes a theoretical stress ratio correction for cyclic simple shear tests that can be applied to conventional triaxial tests, is based on the fundamental model for pore pressure generation devised by Martin et al. (1975). A membrane compliance ratio, equal to the ratio of the average slope of the rebound curve of the sample skeleton to that of the membrane penetration volume change curve, was computed for 1.4-inch diameter samples, and the results were then used to construct curves for 2.8-inch and 12-inch diameter samples by reducing the error in proportion to the inverse of the sample diameter. This correction procedure has since been modified to reflect the findings of more recent test data, as will be discussed in Chapter 3.

Baldi and Nova (1984) proposed a theoretical post-testing correction procedure similar to that of Martin et al. (1978) except that it was applicable only to single cycle tests, and was based on estimation of sample compressibility as a function of sample stress state.

Raju and Venkataramana (1980) proposed a post-testing correction procedure based on a simplified version of the theoretical model for compliance effects on pore pressure during undrained testing developed by Lade and Hernandez (1977).

A study by Raines et al. (1988), modelled the results of undrained monotonic and cyclic triaxial tests with and without the effects of membrane compliance, by numerical analyses. Test results for the "true undrained" tests was achieved by the computer-controlled method described by Seed and Anwar (1986). Initially, a constitutive model capable of modeling stress-strain and pore pressure development for actual uncorrected tests was developed, and appropriate parameters based on the uncorrected data were derived. After the compliance induced volumetric error was evaluated over the range of effective confining stresses for the tests, "corrected" or "true undrained" behavior could be represented by incrementally adjusting the pore pressures of the model, derived from the relationship between changes in pore pressure and void ratio.

The Modified Cam Clay constitutive model (Borja and Kavazanjian, 1985) coupled with a pore pressure analysis algorithm (Borja, 1986) was used to model the pore pressure generation in monotonic tests. Figure 2.35 shows a comparison of the stress vs. axial strain and pore pressure vs. axial strain for one set of modelled and actual test data, on a transformed plane appropriate to the

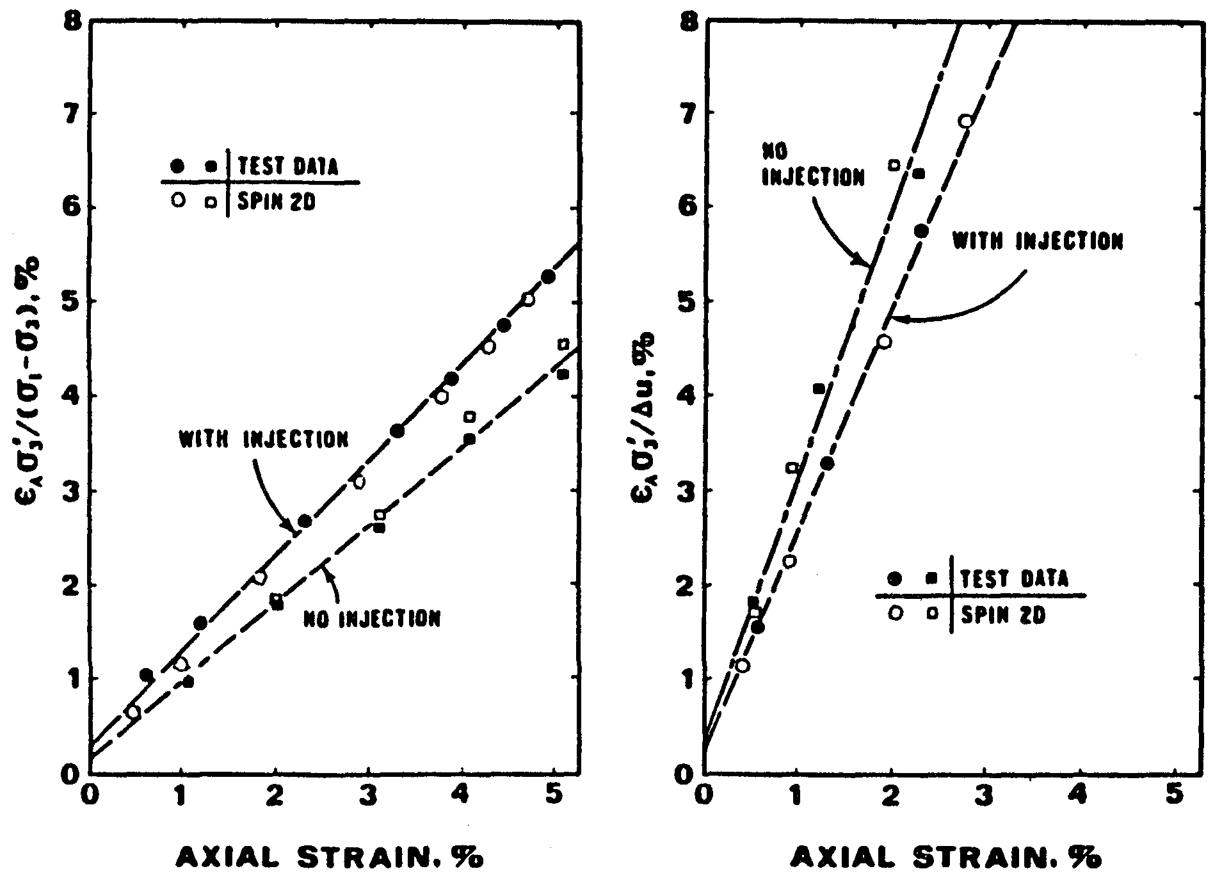


Figure 2.35: Numerically Modelled Stress-Strain and Pore Pressure-Strain vs. Test Results With and Without Membrane Compliance Mitigation: (Raines et al., 1987)

hyperbolic relationships implicitly assumed by the constitutive model. Similar agreement between modelled and actual data was also noted for the other sets of data studied.

The correction procedure developed by Martin, Finn, and Seed (1978) was modified by Evans and Seed (1987) to incorporate their test results of 12-inch diameter tests. This correction, given as a correction of cyclic stress ratios for given tests, was further modified by Hynes (1988) by correlating errors in stress ratios to D_{20} rather than D_{50} as suggested by Seed and Anwar (1986) to give better correlation with currently available penetration data. This updated model of the original procedure appeared to work well for the test results obtained by Hynes (1988).

A correction method has recently been proposed by Kramer and Sivaneswaran (1989) which incorporates a more accurate uniform assumption of the deformed shape of a unit cell of membrane into a constitutive bounding-surface plasticity model. The results of corrections to conventional "uncorrected" tests compared favorably to static tests conducted as a part of that same investigation in which manual injection was used to offset pre-determined volumetric errors. A drawback to that correction method is that the parameters that are necessary for the constitutive model must be obtained from a number of laboratory tests. This precludes the use of this type of correction method for application to previously performed tests from which the necessary constitutive model parameters may not be obtainable. It may also be recognized that all of the evidence supporting the results of the correction method come from tests on one type of uniformly graded sand and one confining membrane type. Whether or not this method will also work for other configurations of materials, membrane types, and sample sizes has not been shown conclusively.

Before tests are performed in which membrane compliance effects are completely and conclusively mitigated, the accuracy of theoretical corrections cannot be proved. Evidence produced as a part of this study and reported by Seed, Anwar, and Nicholson (1989), is described in Chapter 4 to show that the method of continuous computer-controlled injection/removal of water to offset the volumetric error induced by membrane compliance, appears to completely mitigate the deleterious effects from tests conducted on compliant materials. From the test results reported by Seed, Anwar, and Nicholson, and the additional data from tests carried out as part of this study, the accuracy of previous and future theoretical and analytical corrections may be able to be more accurately evaluated.

CHAPTER 3

PREVIOUS TESTING OF GRAVELS

3.1 Introduction

Some of the reasons why there have been so few tests performed on gravelly soils have been explained in Chapter 1. These include the fact that until recently gravels had not generally been considered to be liquefiable, and that many of the undrained static strength evaluations implied that gravels were inherently very strong. It is now understood that these strength evaluations may have been unconservatively erroneous due to apparent errors encountered as a result of membrane compliance during undrained tests. Another reason that testing of gravelly soils has not been extensively researched is the necessary sample sizes that are required to avoid stress concentration problems as pointed out in studies by Holtz and Gibbs (1956), Leslie (1963), Wong, Seed, and Chan (1975), and others. These previous investigations have suggested that the ratio of sample diameter to maximum particle size should be on the order of 6 to 8, depending on the gradation of the soil. For many gravels commonly used for construction or used as naturally occurring foundations, the maximum grain size dictates the use of 12-inch diameter or larger samples in order to avoid stress concentration problems in triaxial tests. The number of facilities which are capable of conducting tests on samples of these sizes are few, and the cost of performing the tests is usually prohibitive except for research.

3.2 Static Strength Evaluations of Gravelly Soils

The amount of test data available to date for strength evaluations of gravelly soils is very limited. This is primarily due to the limiting size of available testing equipment, and further complicated by testing problems involved when attempting to determine reasonable strength parameters for these materials. One possible solution to the problems encountered in testing gravelly soils was to make correlations with tests performed on finer grained material. Among the first to propose a method for estimating the static strength of oversized rockfill from laboratory tests of finer grained material were Zeller and Wullimann (1957). They proposed that by scalping the large particles from the material, a series of tests could be made on remaining graded fragments with different maximum particle sizes, and that the static strength of the total material could then be extrapolated to the maximum particle size of the rockfill gradation.

Lowe (1964) conducted tests on 6-inch diameter specimens with a maximum particle size of 1.5 inches employing the assumption that a parallel gradation to the prototype material (with 12-inch maximum particle size) should give similar shear strength results.

Marachi et al. (1969) further investigated the use of parallel gradations suggested by Lowe to predict friction angles of rockfill material from tests performed on small-scale specimens. They conducted tests on a wide range of parallel gradations by using sample diameters of 36 inches, 12 inches, and 2.8 inches, with maximum particle sizes of 6 inches, 2 inches, and 0.5 inches respectively. Their findings were that the different sample sizes gave similar strengths based on friction angles, with variations between different sample sizes amounting to less than 10%.

Siddiqi (1984) noted some inherent problems with estimating the strengths of gravelly soils by testing parallel gradations. Among them was the fact that parallel gradations tend to introduce undesirably high fractions of fines to the tested material which can have significant effects on soil strengths, especially in evaluating the resistance of soils to cyclic loading.

Torrey and Donaghe (1985) proposed that a better method of preparing a modelled gradation for laboratory testing was by scalping and replacement of the oversized particles. From the undrained triaxial compression tests performed as part of those studies, it was concluded that the scalp and replacement procedure provided conservative strength parameters for their earth-rock prototype material based on total stresses. The undrained strengths of the total earth-rock material were considerably larger than the undrained strengths from tests made of the minus No. 4 sieve fraction of the soil gradation, suggesting that scalping may be over-conservative.

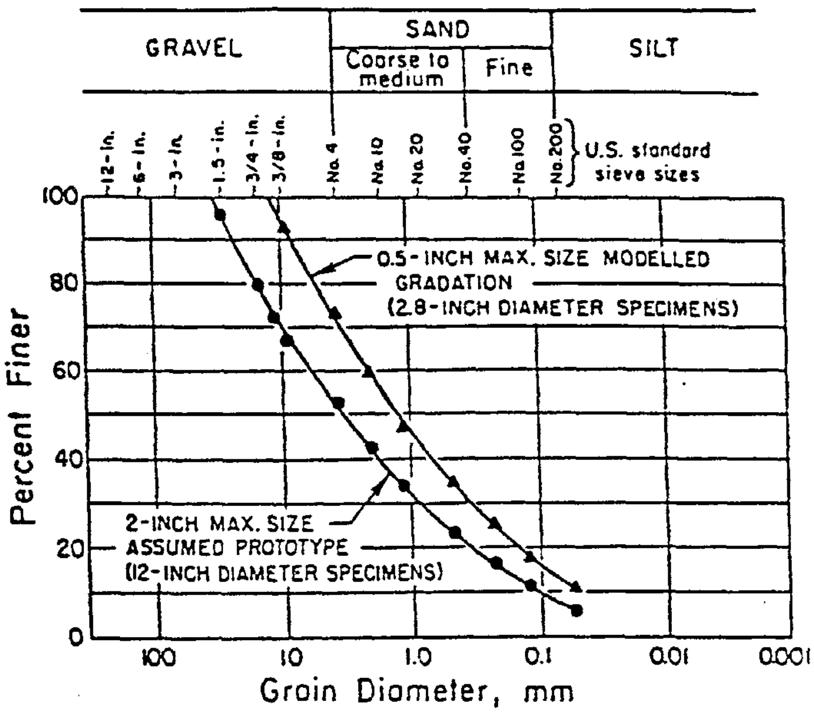
3.3 Cyclic Testing of Gravelly Soils

The amount of cyclic testing that has been performed on gravelly soils prior to this study has been even more limited than undrained static testing since the resistance of gravelly soils had until recently not been a major concern. Among the researchers that have contributed to the small database available are Lee and Fitton (1969), Wong, Seed, and Chan (1974), Banerjee, Seed, and Chan (1979), Siddiqi et al. (1987), Evans and Seed (1987), and Hynes (1988).

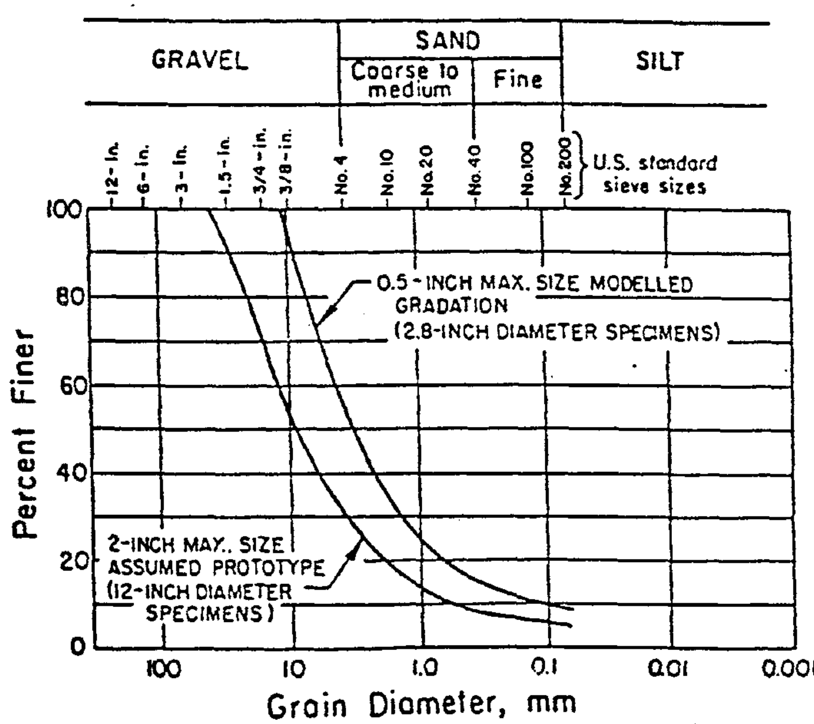
Lee and Fitton (1969) performed tests on 2.8-inch diameter samples including uniformly-graded gravels with a maximum particle size of 3/4 inch. It was reported that with all other test conditions held constant, the cyclic shear stress required to cause initial liquefaction in the gravel specimens was nearly twice that needed for similar specimens of sand. These erroneous results have since been attributed to the adverse effects of membrane compliance and stress concentration problems associated with the large ratio of particle size to sample diameters encountered in those specimens.

Wong, Seed, and Chan (1974) performed tests on sands and gravels using both 2.8-inch and 12-inch diameter samples. They reported that the gravel specimens required somewhat higher cyclic stresses to cause given strains than for similar samples of sand at the same relative densities. This result led the investigators to speculate that gravels may be inherently more resistant to liquefaction failure than sands. It was also reported in that study that lower stress ratios were required to cause given strains for well-graded specimens than for uniformly graded specimens. The investigators recognized that the noted phenomenon was an indication of the effects of membrane compliance but did not attempt to make compliance corrections. An attempt was made by Seed and Anwar (1986) to make numerical corrections to volumetric compliance magnitudes for non-uniform soil gradations, but presently there is not enough of a database to make a realistic evaluation of the accuracy of such quantitative corrections.

Banerjee et al. (1979) reported the results of large scale (12-inch diameter) and small scale (2.8-inch diameter) cyclic tests on dense, well-graded gravels with maximum particle sizes of 2 inches and 0.5 inches respectively. Banerjee used the method of testing modelled parallel gradations to evaluate the cyclic strengths of coarse gravelly materials from Oroville and Lake Valley Dams. The modelled gradations used for the testing of 2.8-inch and 12-inch diameter samples are shown in Figure 3.1. The cyclic test results presented in Figure 3.2 appear to support the use of parallel gradations for these types of tests. Among the parameters investigated by Banerjee were the effects of sample preparation, and sustained high confining pressure. The method of sample preparation appeared to be insignificant while the sustained confining load reportedly increased the cyclic loading resistance of the gravel specimens. The increase in cyclic strength or cyclic resistance due to a sustained confining load found by Banerjee is shown in Figure 3.3. These results are combined with results from previous investigations (Figure 3.4) to suggest an increase in strength for a wide range of times of sustained pressures on granular soils.

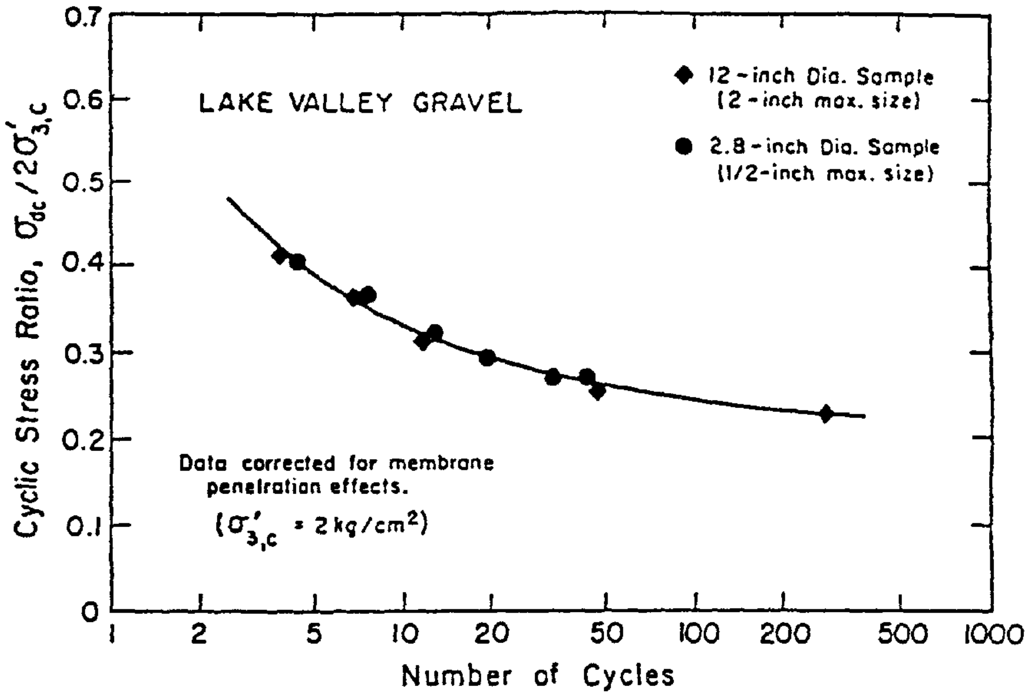


a) Lake Valley Dam Material

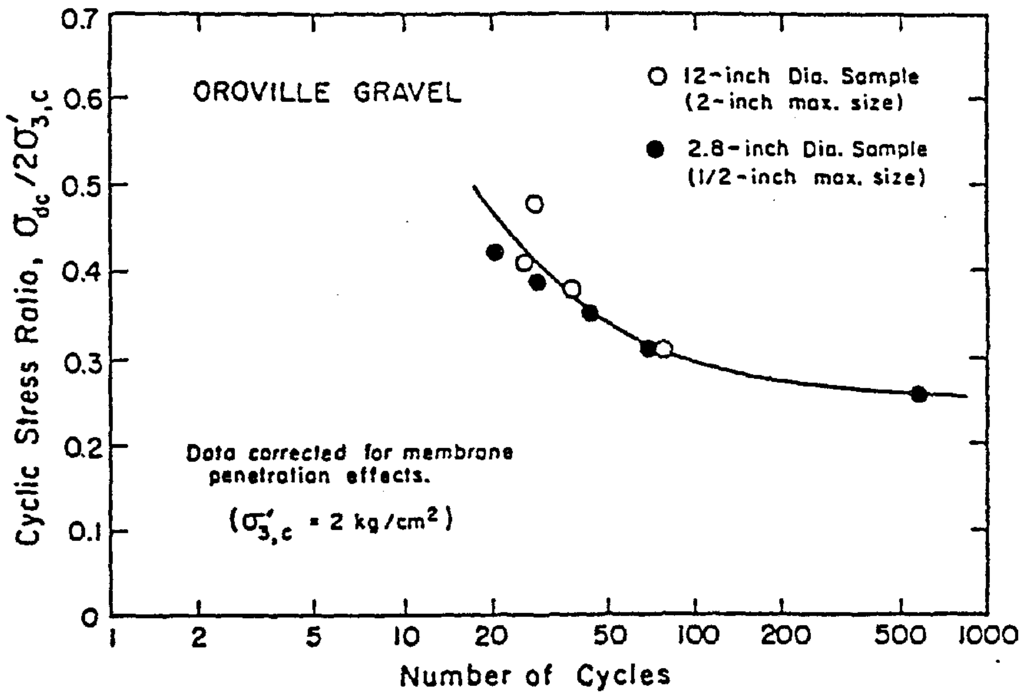


b) Orovolle Dam Material

Figure 3.1: Grain Size Distribution Curves of the Prototype and Modelled Specimens: (Banerjee et al., 1979)



a) Lake Valley Gravel at $D_R = 60\%$



a) Oroville Gravel at $D_R = 84\%$

Figure 3.2: Cyclic Strength Curves for 12-Inch and 2.8-Inch Diameter Samples of Prototype and Modelled Gradations: (Banerjee et al., 1979)

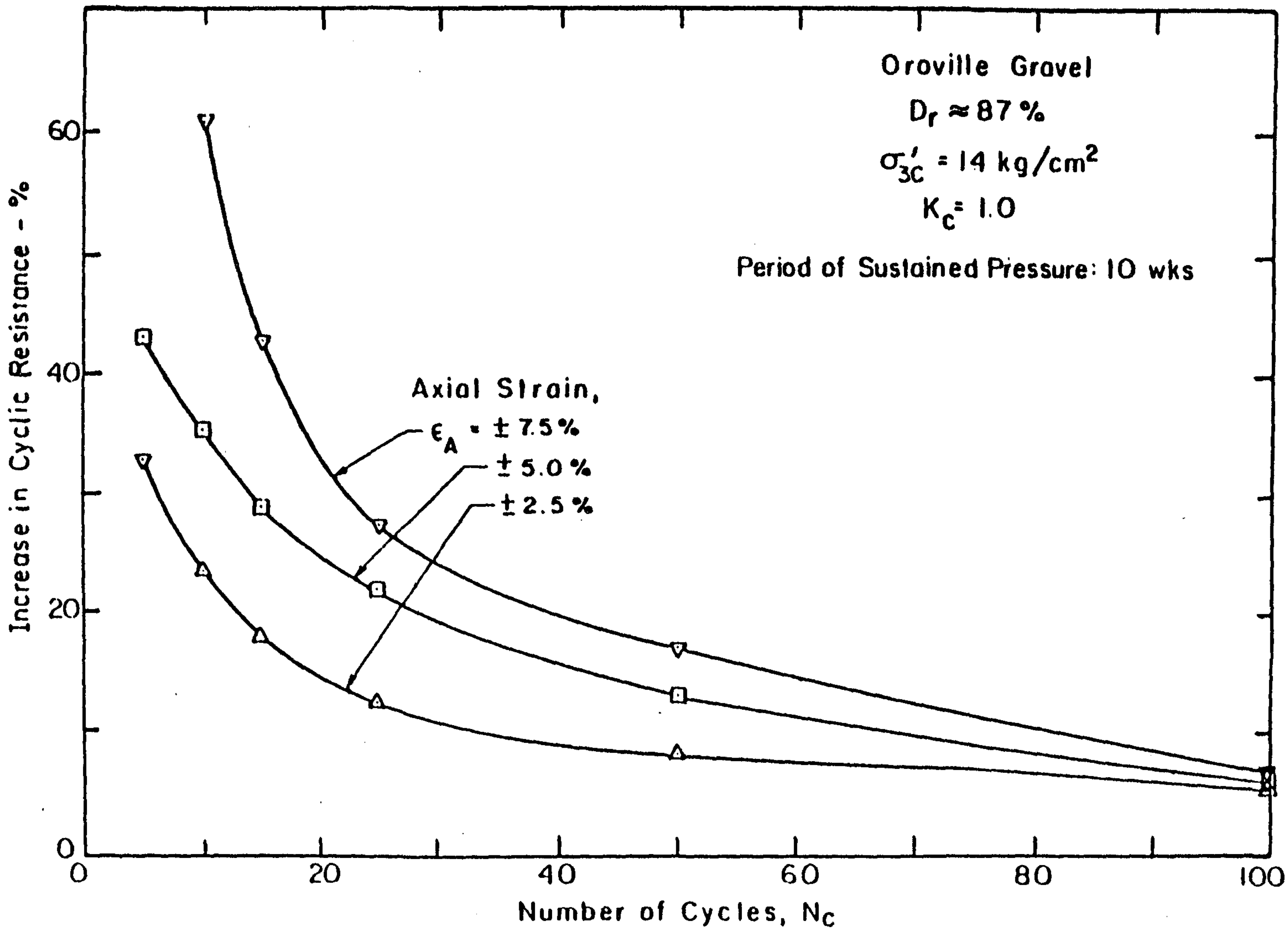


Figure 3.3: Increase in Cyclic Resistance Due to Sustained Pressure Effects: (Banerjee et al., 1979)

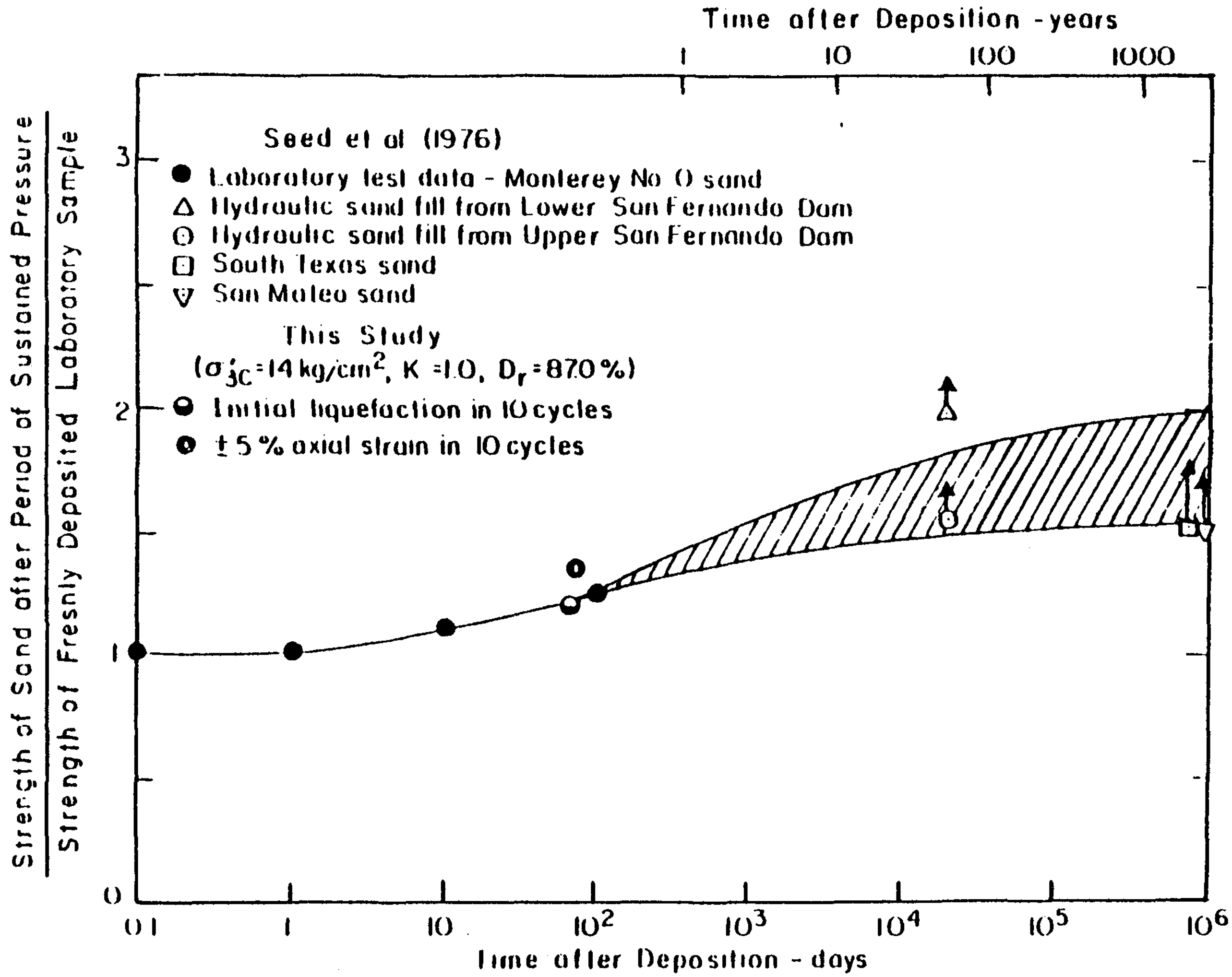


Figure 3.4: <u>Influence of Period of Sustained Pressure on Stress Ratio Required to Cause Pore-Pressure Ratio of r_u = 100% or ± 2.5% to 5.0% Strain in 10 Cycles: (Banerjee et al., 1979)</u>

Banerjee also developed a laboratory procedure for measuring the membrane compliance variations over a range of confining stresses. This procedure which incorporates the use of so-called "girth belts" discussed in section 2.2, allows the measurement of both axial and radial deformations of sample skeletons, from which accurate values of membrane compliance can be calculated. A detailed discussion of the equipment and calculations involved in making the radial strain measurements can be found in the appendix of the report by Banerjee et al. (1979).

Siddiqi et al. (1987) investigated methods to determine representative laboratory gradations for actual field gradations which contained particles too large to be tested with conventional laboratory equipment. Siddiqi made several conclusions pertaining to the scalping of oversized "floating" particles and the densities at which such scalped materials should be tested, in order to obtain representative results from the total prototype materials. An illustration of the added void space due to the inclusion of "oversized" large particles in a "matrix" of smaller particles is given in Figure 3.5. The components of the total volume occupied by such a material is demonstrated schematically in Figure 3.6, demonstrating the need to adjust the densities of materials to be tested so as to properly represent in-situ field densities. Figure 3.7 shows a proposed relationship between theoretical and measured dry densities as functions of percent of gravel contained in the material.

In order to account for, or to offset membrane penetration effects for cyclic triaxial tests performed on gravelly soils, Banerjee applied a simplified correction of adding 10 percent to the cyclic stress ratios actually applied during the tests.

Evans and Seed (1987) attempted to evaluate "correct" strengths for gravels and mitigate compliance effects by sluicing the gravel specimens with loose sand. The results of Evans' tests led the investigators to conclude that the correction

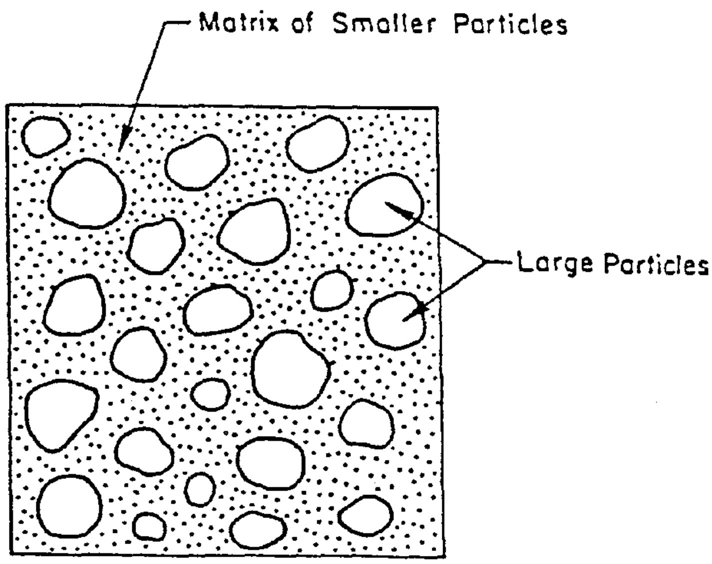


Figure 3.5(a) Illustration of Larger Particles Floating in a Matrix of Finer-Grained Soil

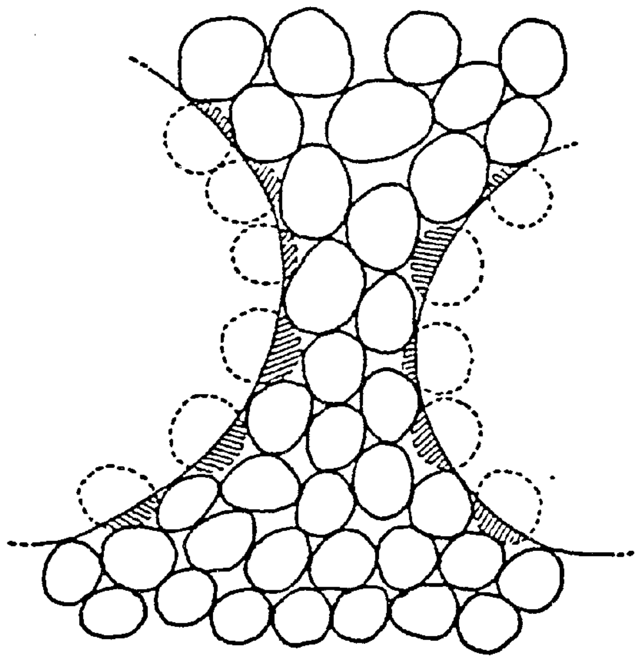


Figure 3.5(b) Illustration of Larger Void Spaces at the Contact Interfaces Between Large Particles and Finer-Grained Particles: (Siddiqi et al., 1987)

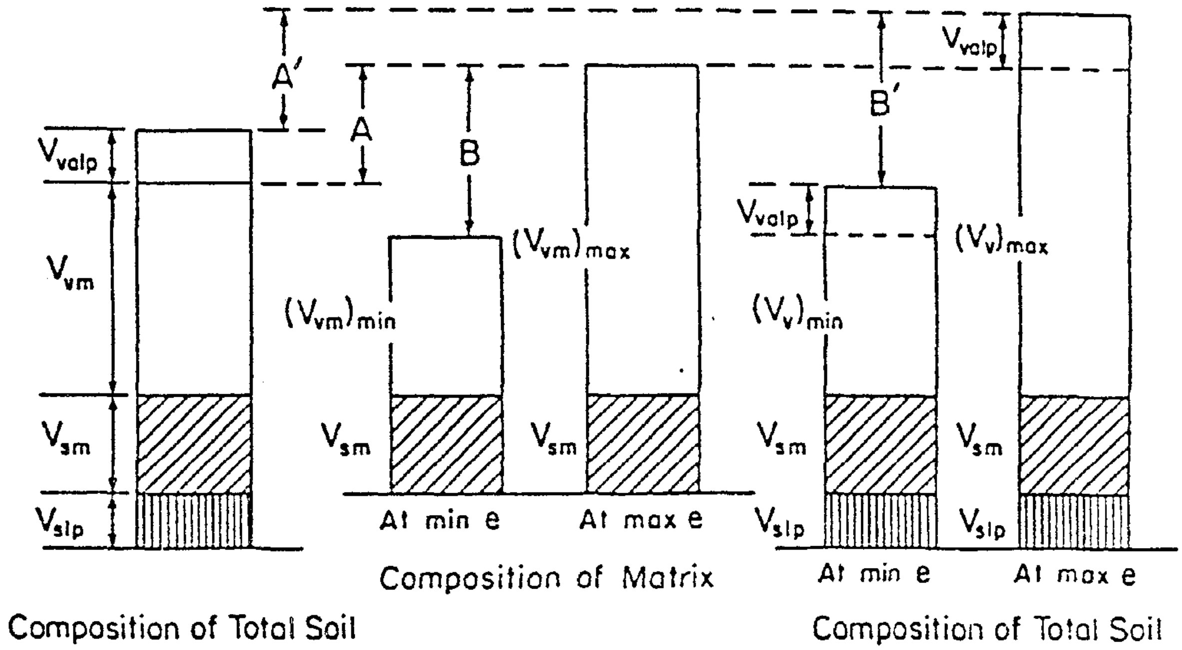
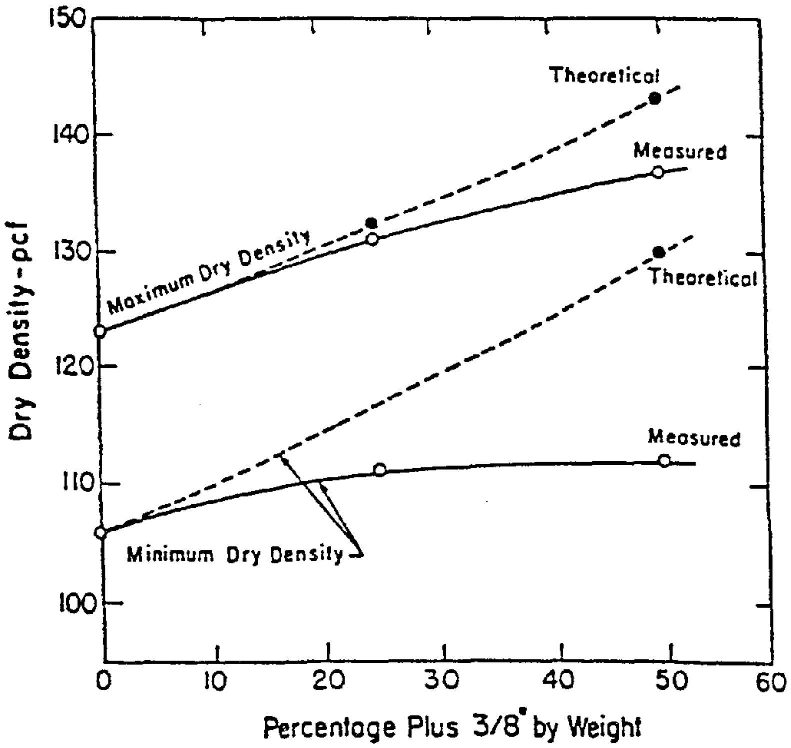
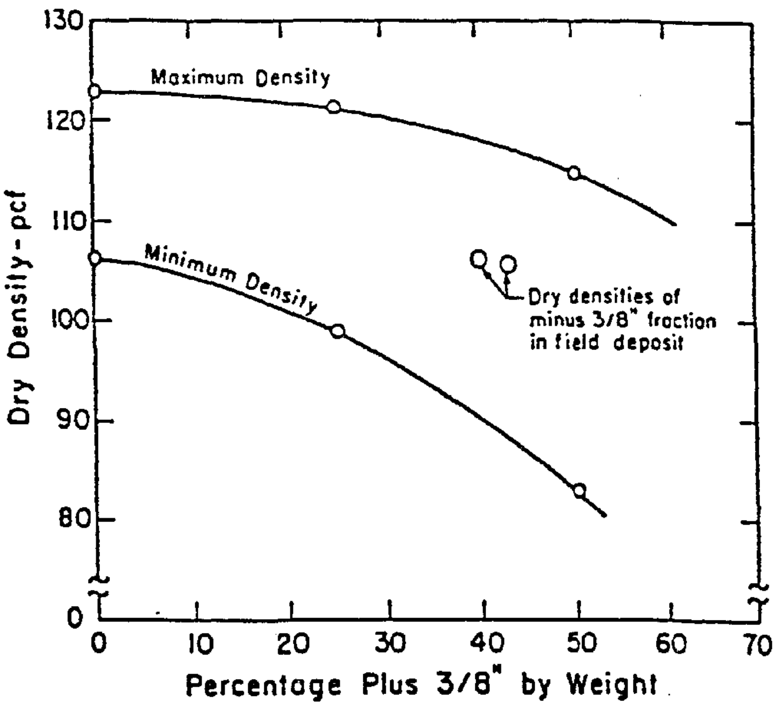


Figure 3.6: Schematic Representation of Components of Coarse-Grained Soil: (Siddiqi et al., 1987)



a) Total Material



b) Minus 3/8-Inch Fraction

Figure 3.7: Effects of Gravel Content on Maximum and Minimum Densities of a Coarse Alluvium: (Siddiqi et al., 1987)

procedure proposed by Martin et al. (1978) should be modified for 12-inch diameter specimens to incorporate a slightly larger compliance induced error. A typical set of results from cyclic tests performed on samples of sluiced and unsluiced gravel are presented in Figure 3.8, showing the reduction in cyclic strength by the sluicing method. Figure 3.9 depicts how the results generated by Evans on sluiced samples of uniformly graded gravel compare with the hypothesized corrections made by Martin et al. (1978). While compliance was significantly reduced by the sluicing method, it was not completely mitigated by employing this procedure. Because of this, the Martin, Finn, and Seed correction procedure may still need to be further modified to give more accurate predictions of what the true corrections should be, once the database becomes more complete.

Hynes (1988) investigated using the threshold strain approach to evaluate the liquefaction potential of gravels. The concept of the threshold strain theory, is that for each soil type there is a certain shear strain at which pore pressures begin to develop. It was suggested by Seed (1979) that threshold strain levels appear to be independent of most of the major factors that affect cyclic strength. Therefore only the threshold strain of the material, and the expected possible strains at a given site, need to be known in order to provide a primary screening of liquefaction potential. The idea being that if the necessary strains needed to cause pore pressure generation are not expected, then liquefaction of the material can be ruled out, and further cyclic testing of the material for liquefaction potential would be unnecessary. Additional objectives of that study included providing pore pressure generation data and characteristics for well-graded gravels at loose and moderately dense relative densities. Membrane compliance measurements were made on at least one of the gravel types tested in that study, and those determinations were used to correct cyclic strength evaluations using the procedure developed by Martin et al. (1978), and modified by Evans and Seed (1987).

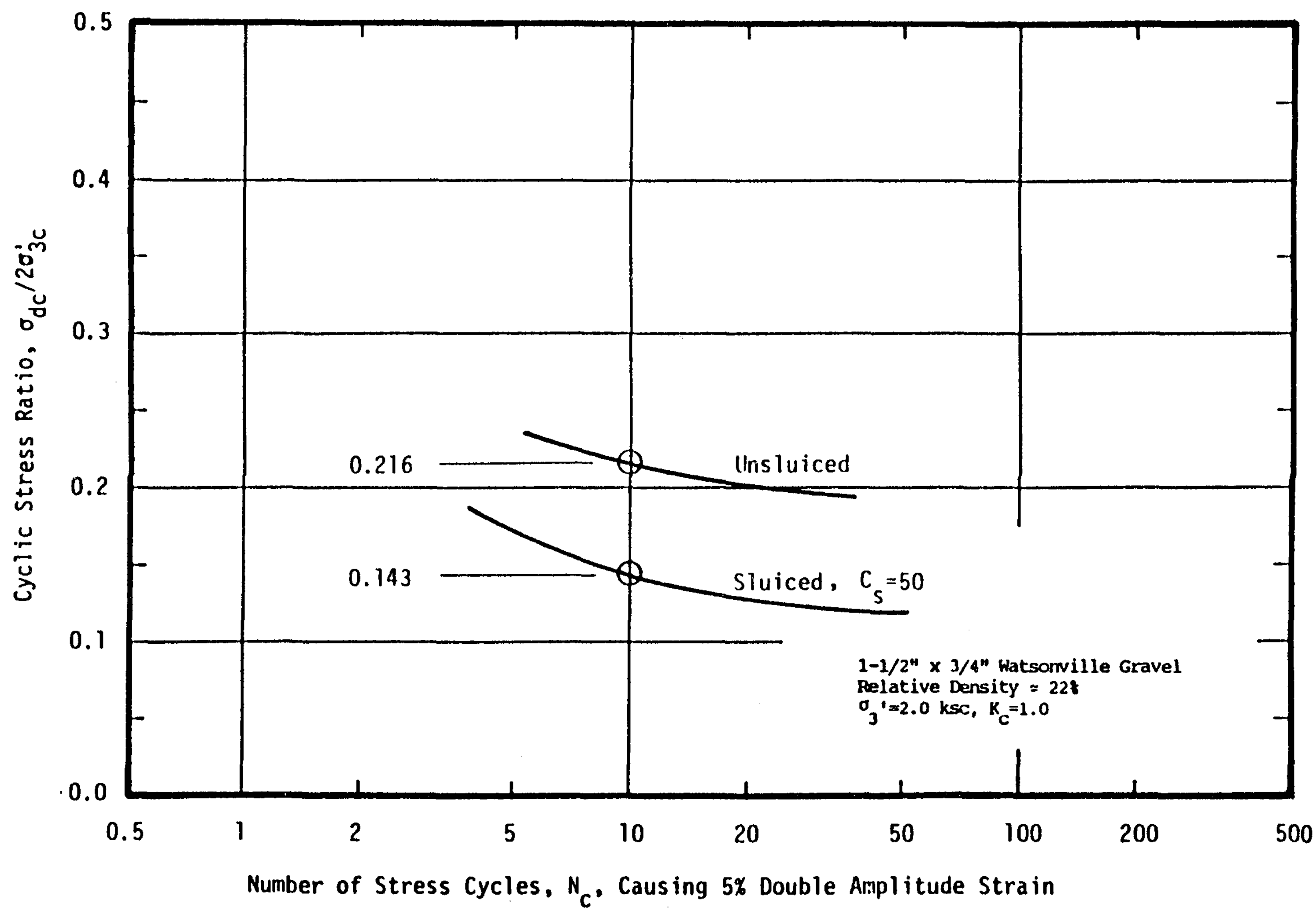


Figure 3.8: <u>Cyclic Strength Curve Showing Effect of Sluicing on 12-Inch Diameter Samples: (Evans and Seed, 1987)</u>

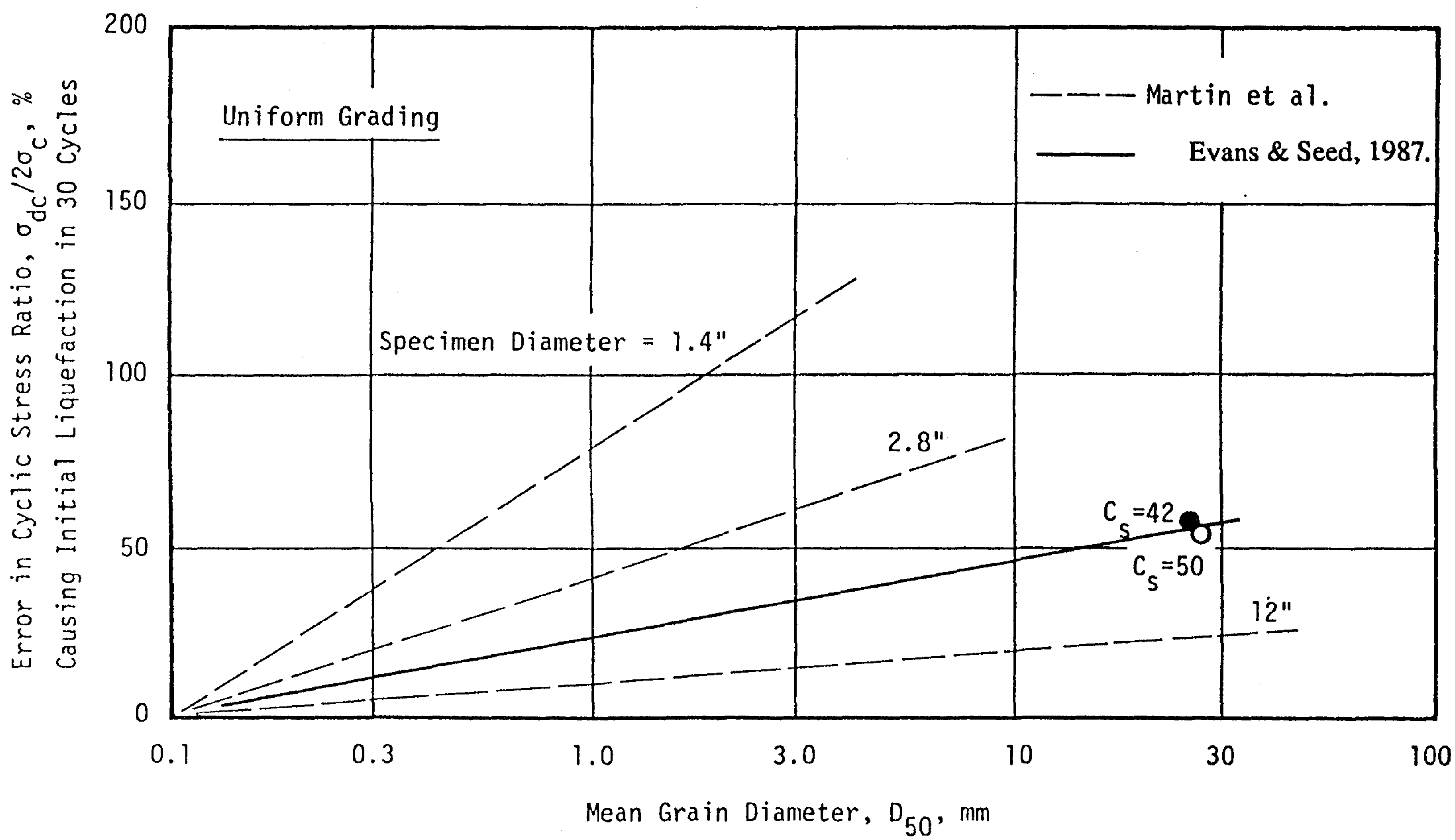


Figure 3.9: Error in Cyclic Stress Ratio Due to Membrane Compliance vs. Mean Grain Size for Various Specimen Diameters: (after Martin et al., 1978; Evans and Seed, 1987)

CHAPTER 4

VERIFICATION OF A COMPUTER-CONTROLLED INJECTION/REMOVAL SYSTEM FOR MITIGATION OF COMPLIANCE EFFECTS

4.1 Introduction

Until tests are performed in which membrane compliance effects are completely and conclusively mitigated, the accuracy of theoretical corrections or compliance mitigation techniques can not be proved. In order to investigate whether or not the proposed method of continuous computer-controlled injection/removal of water to offset the volumetric error induced by membrane compliance completely mitigates the deleterious effects of compliance on undrained tests, a number of monotonic and cyclic triaxial tests performed with the use of the injection-mitigation system devised by Seed and Anwar (1986) on small scale (2.8-inch diameter) samples of Monterey 16 sand were duplicated using large scale (12-inch diameter) samples of the same material for which no corrections were necessary. The results of these large-scale tests were then compared to the results of the earlier small-scale tests. Based on the test results reported in this chapter, the "correctness" and accuracy of such a compliance mitigation system was verified.

4.2 Verification of a Computer-Controlled Injection/Removal System for Mitigation of Compliance Effects

The first phase of implementing a computer-controlled injection-correction system was to verify the accuracy and validity of such a system to eliminate the adverse affects of membrane compliance. The compliance mitigation system implemented by Seed and Anwar (1986) for use with "conventional" sized (2.8-inch diameter) samples appeared to give excellent results for eliminating membrane compliance effects as compared with the results of tests "corrected" for volumetric compliance errors by analytical methods. But the validity of these analytical models themselves could not be verified. Therefore in order to validate the use of the computer-controlled injection-correction system used in the investigation by Seed and Anwar, a physical testing approach was used, repeating the earlier tests with 12-inch diameter samples. As described in Section 2.3.2, the effects of membrane compliance can be eliminated by using a sufficiently large sample size such that volumetric membrane compliance is negligible for the material being tested. The material used by Seed and Anwar for a majority of their tests on 2.8-inch diameter samples performed with and without the implementation of their computer-controlled injection-correction system, was a uniformly graded Monterey 16 sand. A gradation curve for this material is given in Figure 4.1. The grain sizes of this material were sufficiently small as to preclude any "significant volume" of penetration of the large-scale membrane into peripheral sample voids, so that the expected amount of membrane compliance was negligible when 12-inch diameter samples were prepared. As explained in Section 2.3.2, this is due to the very small volume of edge voids with respect to overall sample volume of the "large-scale" samples.

A quantity of the same material that was used for the comparative tests by Seed and Anwar (1986) was obtained for testing in "large-scale" 12-inch diameter samples. Gradation, as well as maximum and minimum densities were checked to ensure that the material was indeed similar to that previously tested. 12-inch diameter samples were prepared by the same method (moist tamping in controlled-volume layers) and at the same relative density for cyclic tests as the 2.8-inch diameter samples had been in the earlier study. For monotonic (static) tests, samples were prepared at a variety of densities as described in Section 4.3.3.

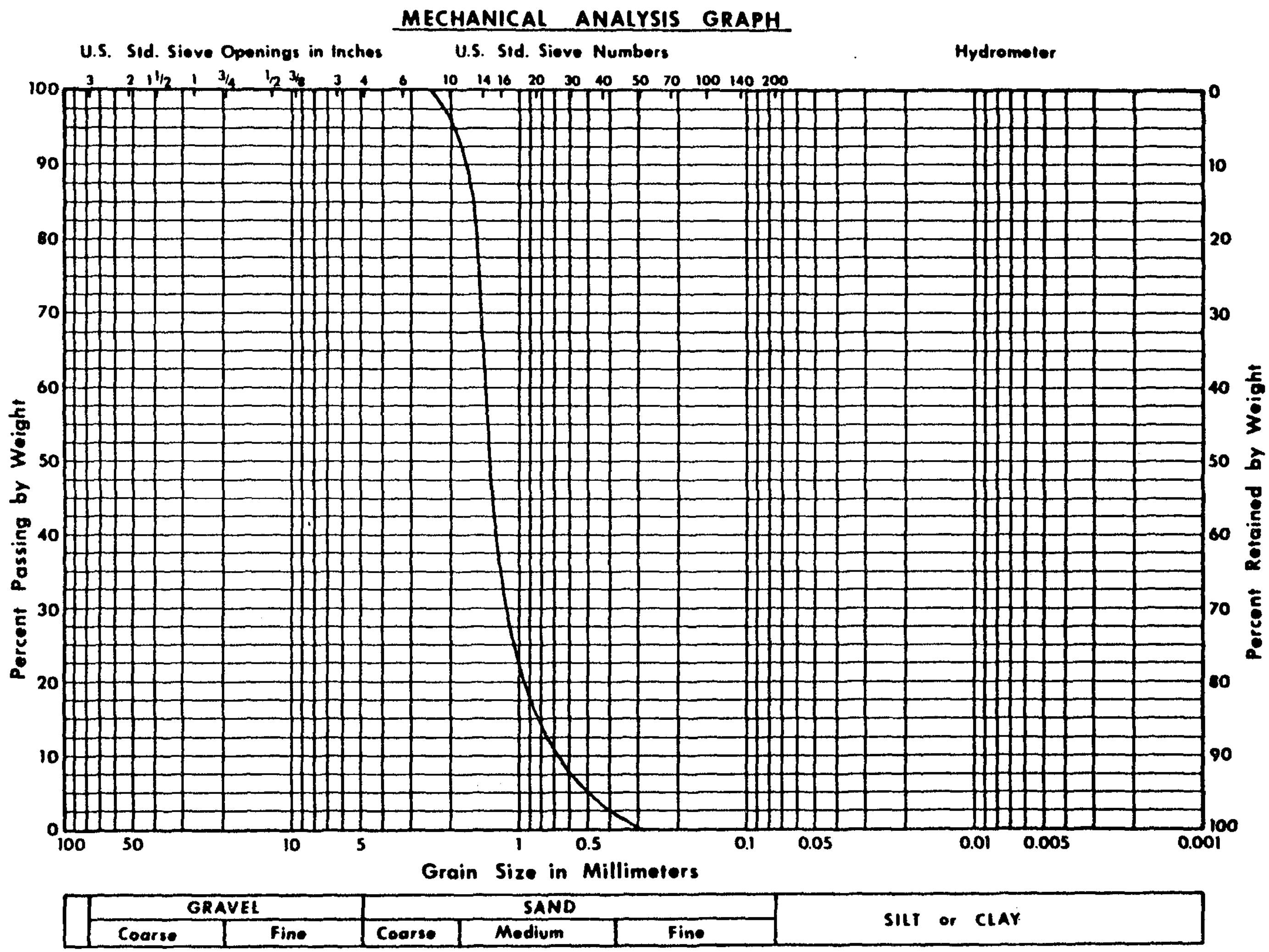


Figure 4.1: Gradation for Monterey 16 Sand

Both static and dynamic (cyclic) undrained load tests were performed on the large-scale samples, employing conventional testing methods. This provided test results which could be taken as "correct" (without the deleterious effects of membrane compliance). These "correct" large-scale test results were then compared to those results presented by Seed and Anwar for the earlier small-scale tests of the same material under similar conditions with the exception that a computer-controlled injection-correction system was employed for the smaller samples (which exhibited considerable compliance in the small-scale tests) to offset the pre-determined volumetric errors expected for those samples. The results of the comparative tests performed by Seed and Anwar on the material with and without the use of the injection-correction system are presented in conjunction with the results of the new tests performed using 12-inch diameter samples in Figures 4.2 and 4.3. It can be seen from these results that the tests performed on the 2.8-inch diameter samples with the use of the computer-controlled injection system appear to be in excellent agreement with "correct" large-scale test results, and that the small-scale samples tested without computer-controlled injection/correction agreed very poorly with the large-scale test results. Accordingly, it can be concluded that the computer-controlled injection system accurately and reliably eliminated the effects of membrane compliance for both the monotonic and cyclic load tests. The initial test conditions and principal results for all of the tests plotted in Figures 4.2 and 4.3 are tabulated in Tables 4.1 and 4.2. Individual test results for each of the tests are presented in Figures 4.4 through 4.26.

Having thus physically verified the accuracy and reliability of the system used by Seed and Anwar, the mitigation system devised for this study was modelled using the same general techniques, but with some obvious modifications necessary

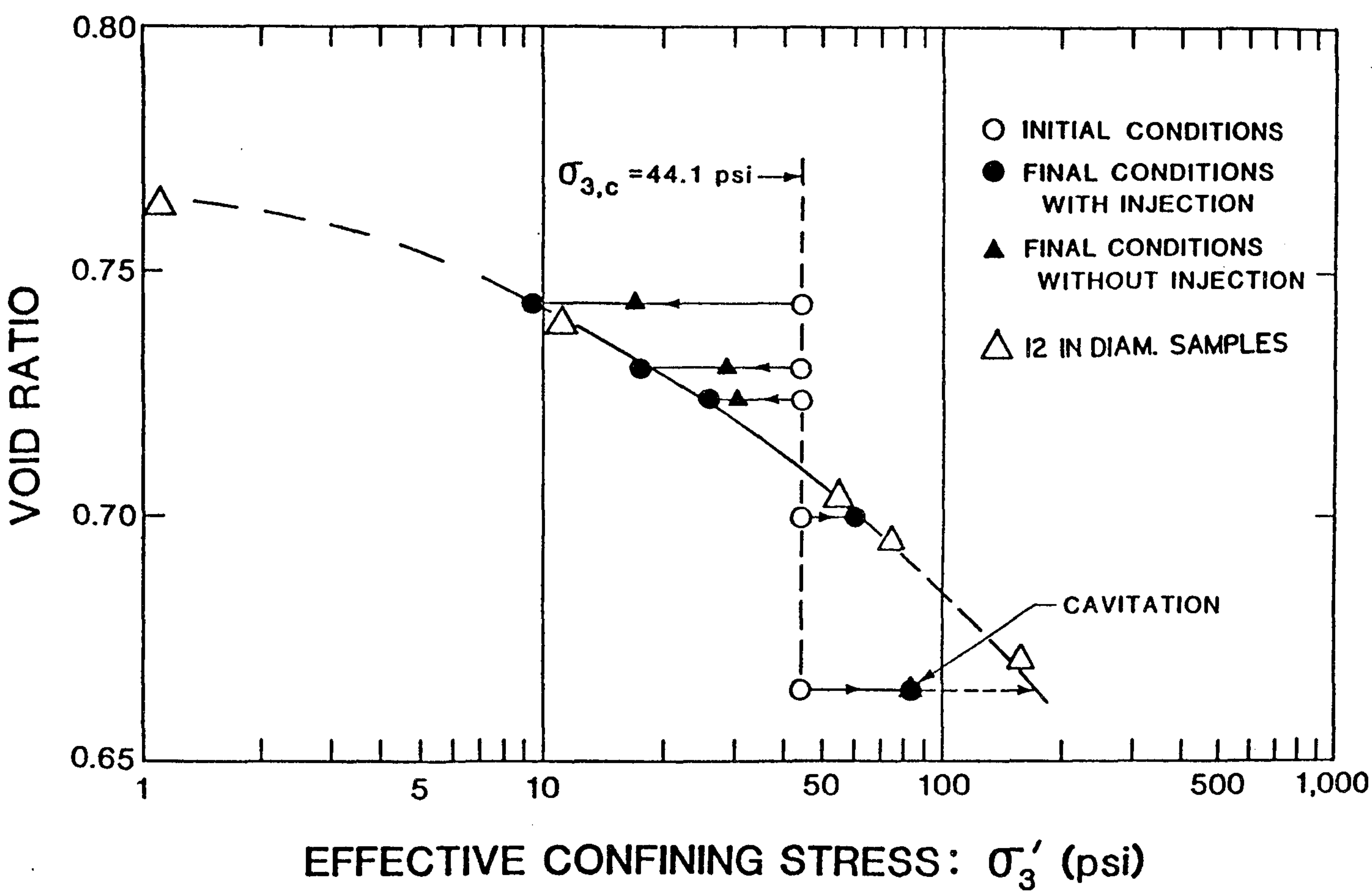


Figure 4.2: Critical-State Plot for $\overline{\text{IC-U}}$ Triaxial Tests on Monterey 16 Sand for 2.8-Inch Diameter Samples With and Without Membrane Compliance Mitigation and for 12-Inch Diameter Samples

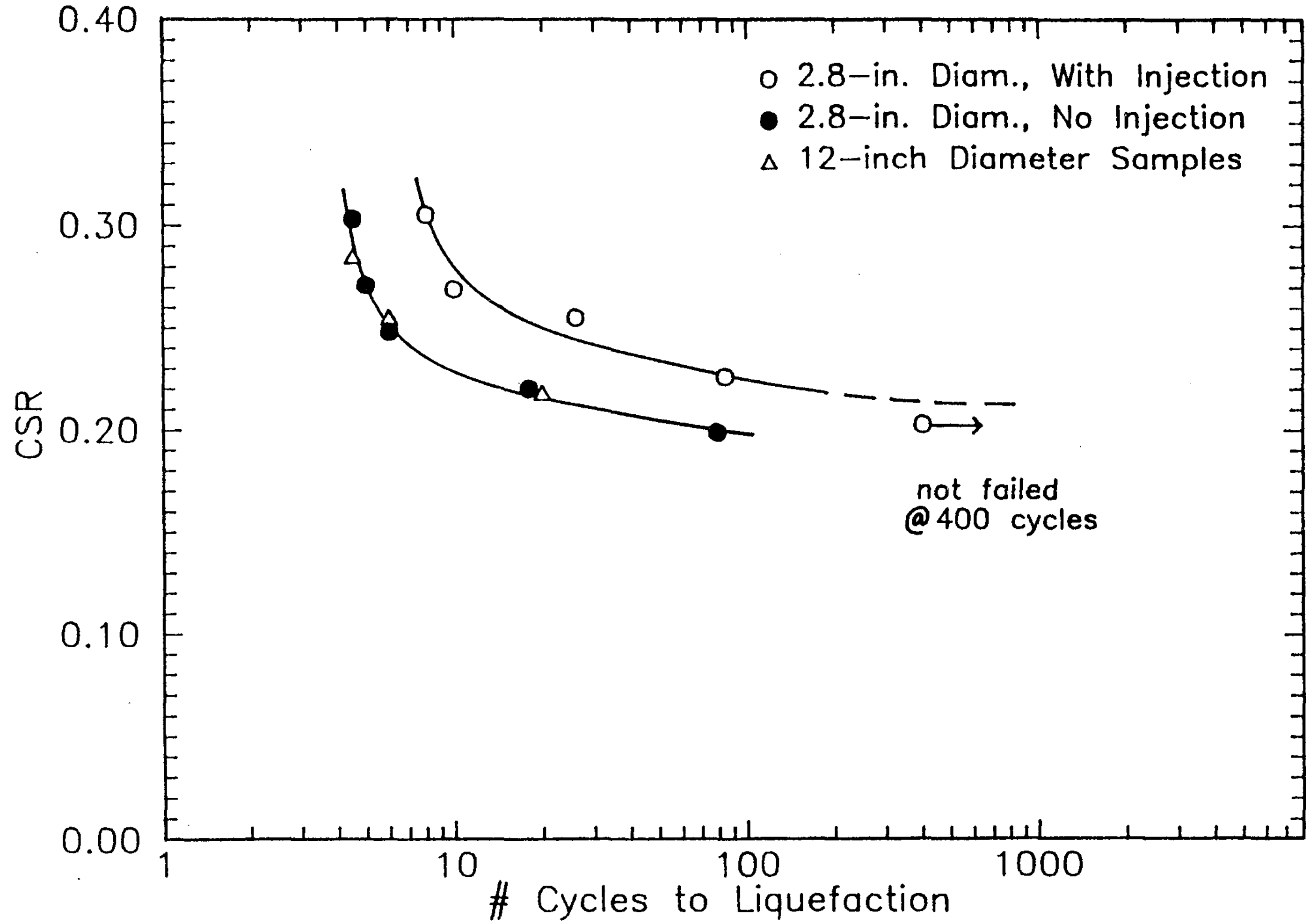


Figure 4.3: <u>Results of Isotropically Consolidated-Undrained Cyclic Triaxial Tests on Monterey 16 Sand at $D_R \approx 55\%$ for 2.8-Inch Diameter Samples With and Without Membrane Compliance Mitigation and for 12-Inch Diameter Samples</u>

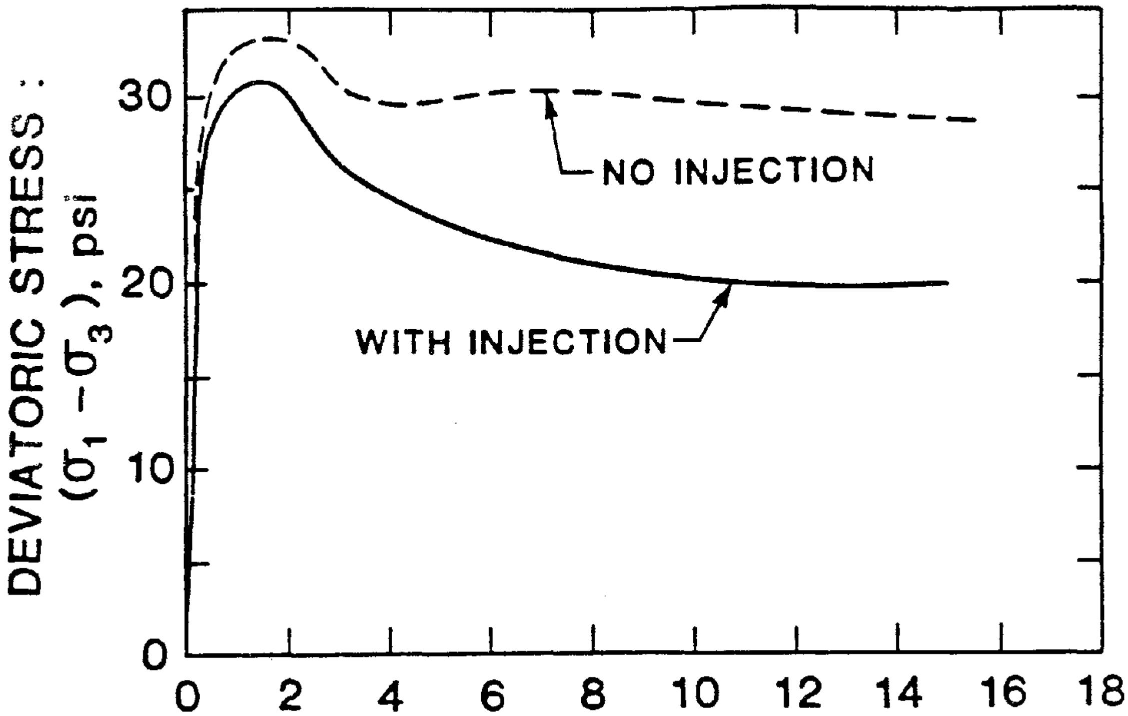
Table 4.1: <u>Testing Conditions: $\overline{\text{IC-U}}$ Triaxial Tests on Monterey 16 Sand With and Without Membrane Compliance Mitigation</u>

(a) 2.8-inch diameter samples:

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,c}$ (psi)	B-Value
1A	15	Yes	44.1	0.991
1B	15	No	44.1	0.993
2A	19	Yes	44.1	0.987
2B	19	No	44.1	0.990
3A	22	Yes	44.1	0.988
3B	22	No	44.1	0.989
4A	29	Yes	44.1	0.983
4B	29	No	44.1	0.990
5A	40	Yes	44.1	0.987
5B	40	No	44.1	0.981

(b) 12-inch diameter samples:

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,c}$ (psi)	B-Value
PT-4	31	No	44.1	0.975
PT-5	27	No	44.1	0.970
PT-6	10	No	44.1	0.970
PT-7	16	No	44.1	0.980
PT-8	37	No	44.1	0.980



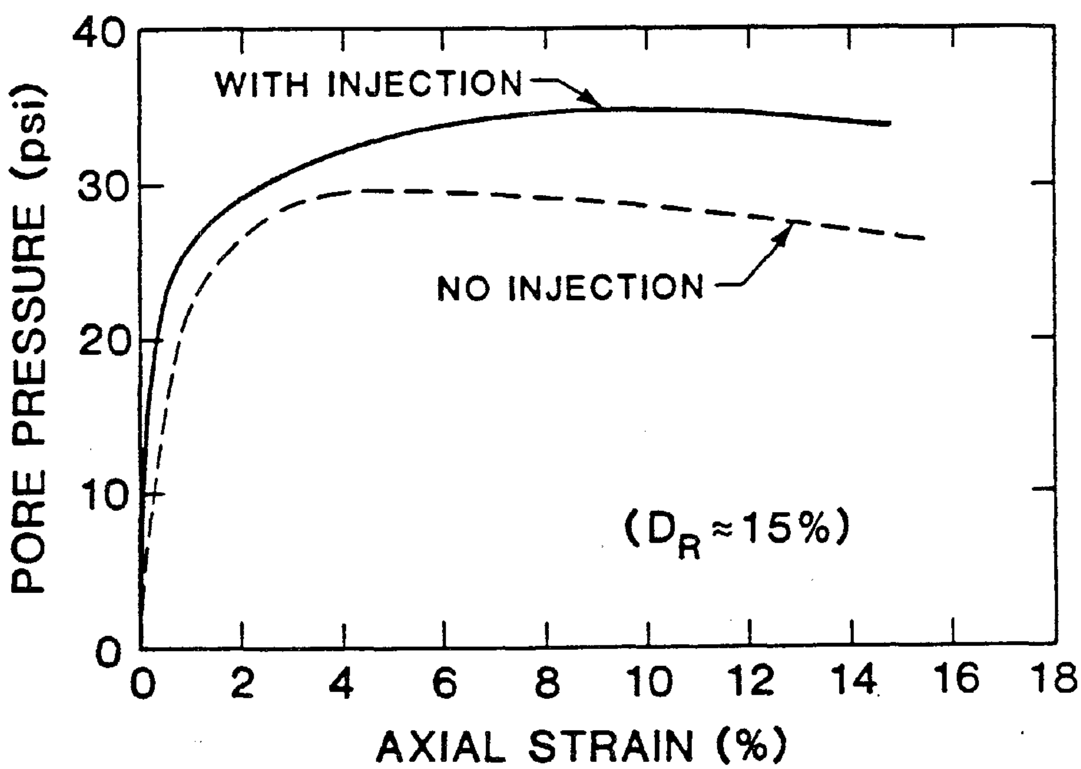


Figure 4.4: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 15\%$)

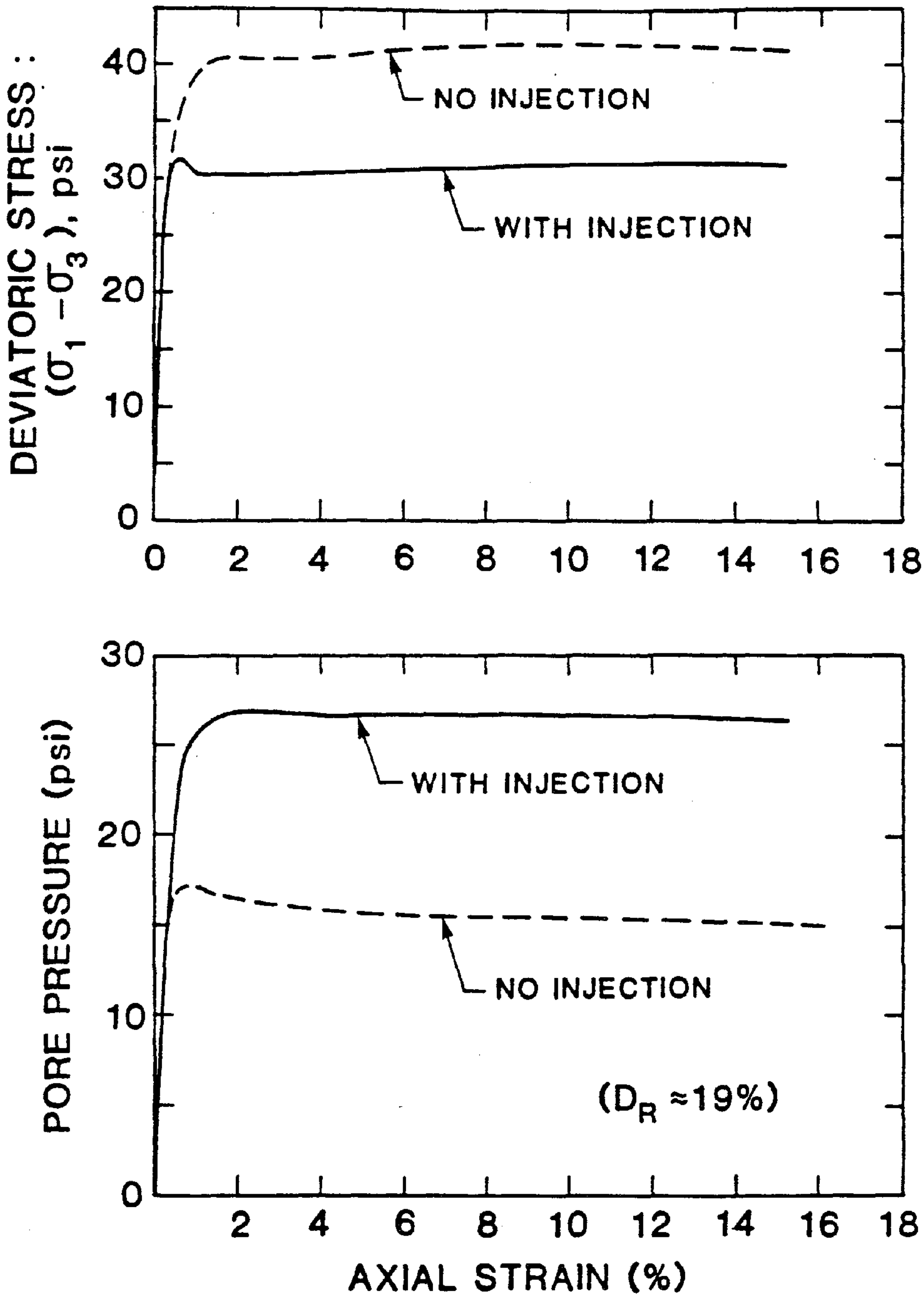


Figure 4.5: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 19\%$)

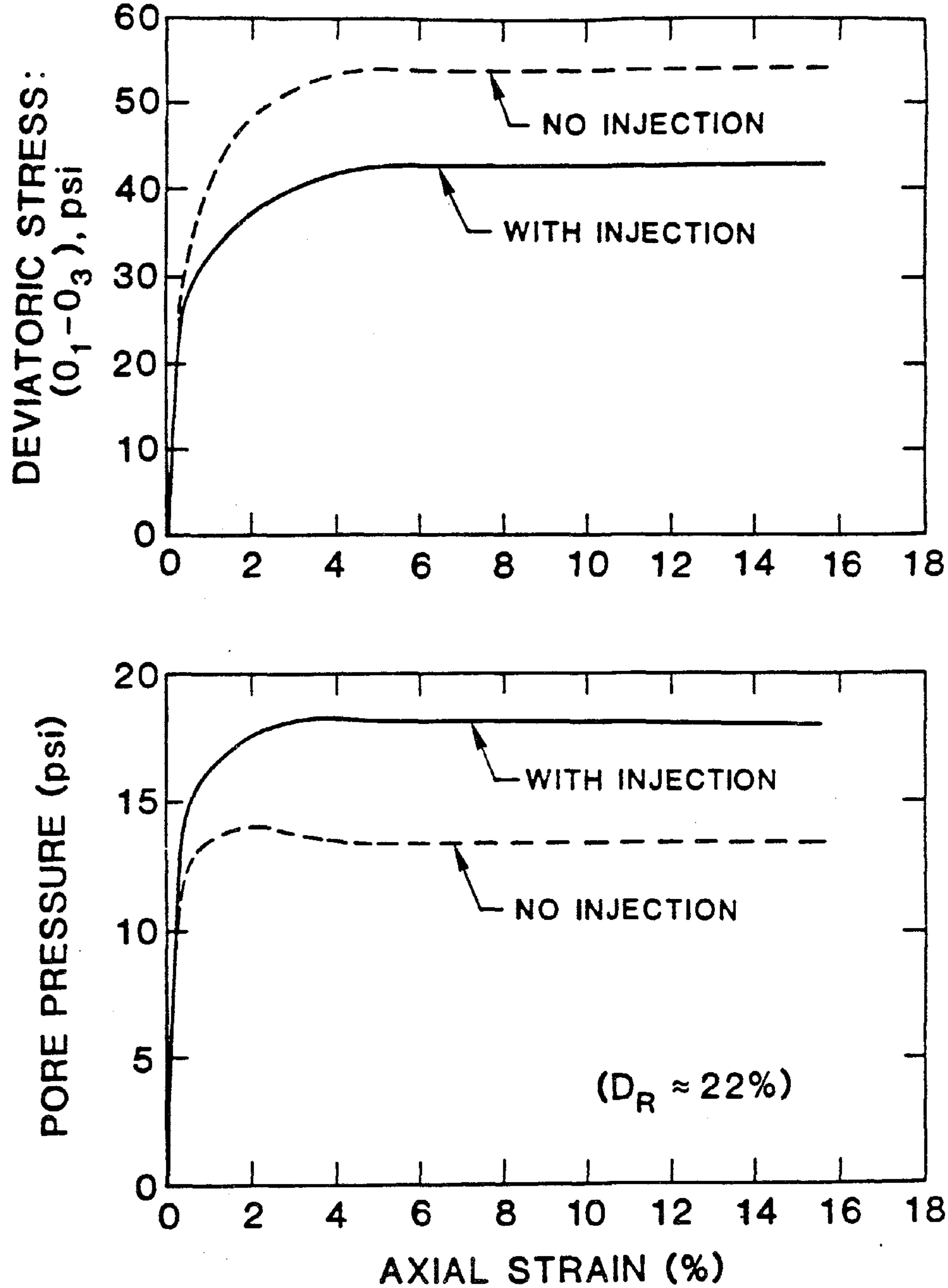


Figure 4.6: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 22\%$)

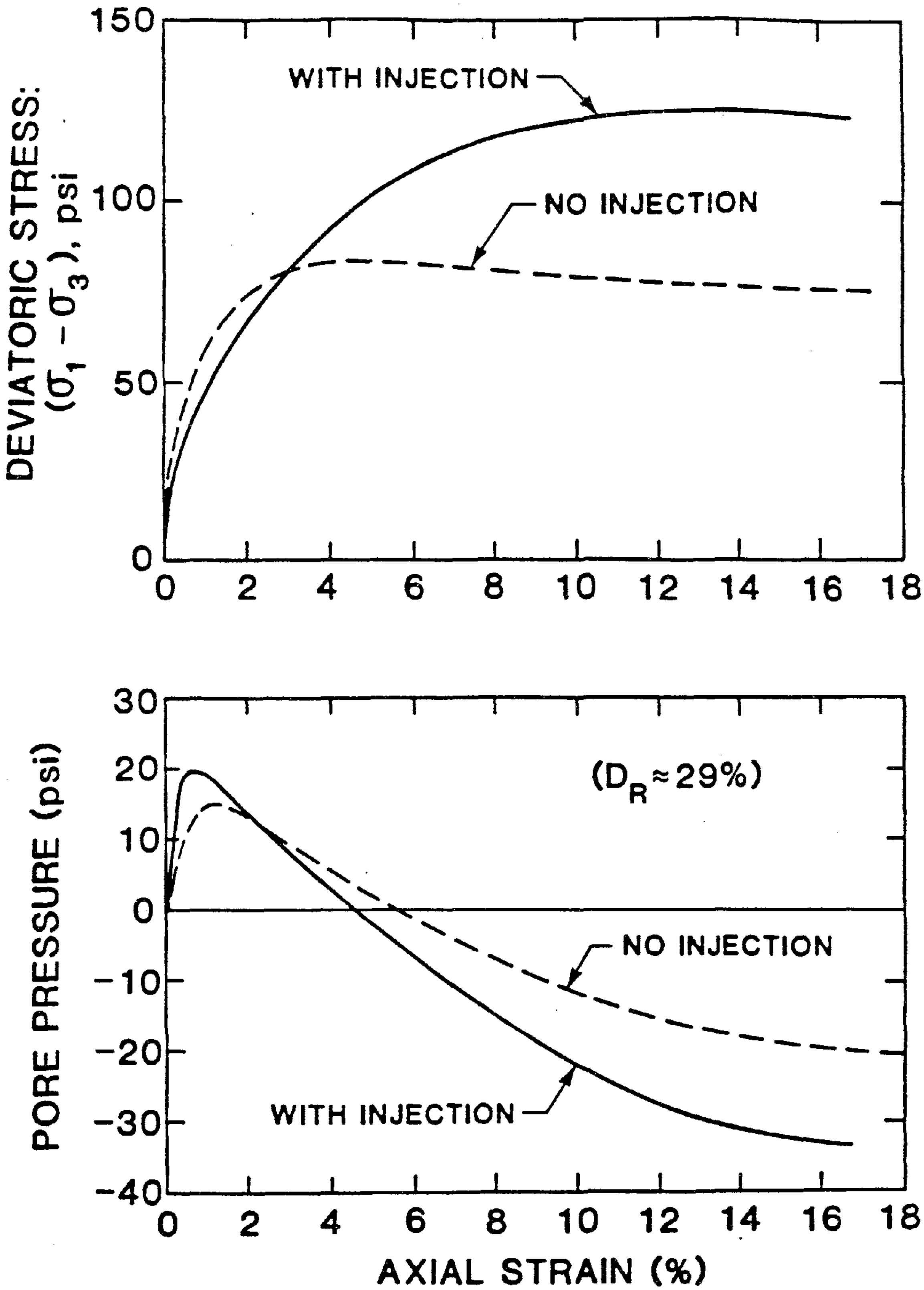


Figure 4.7: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 29\%$)

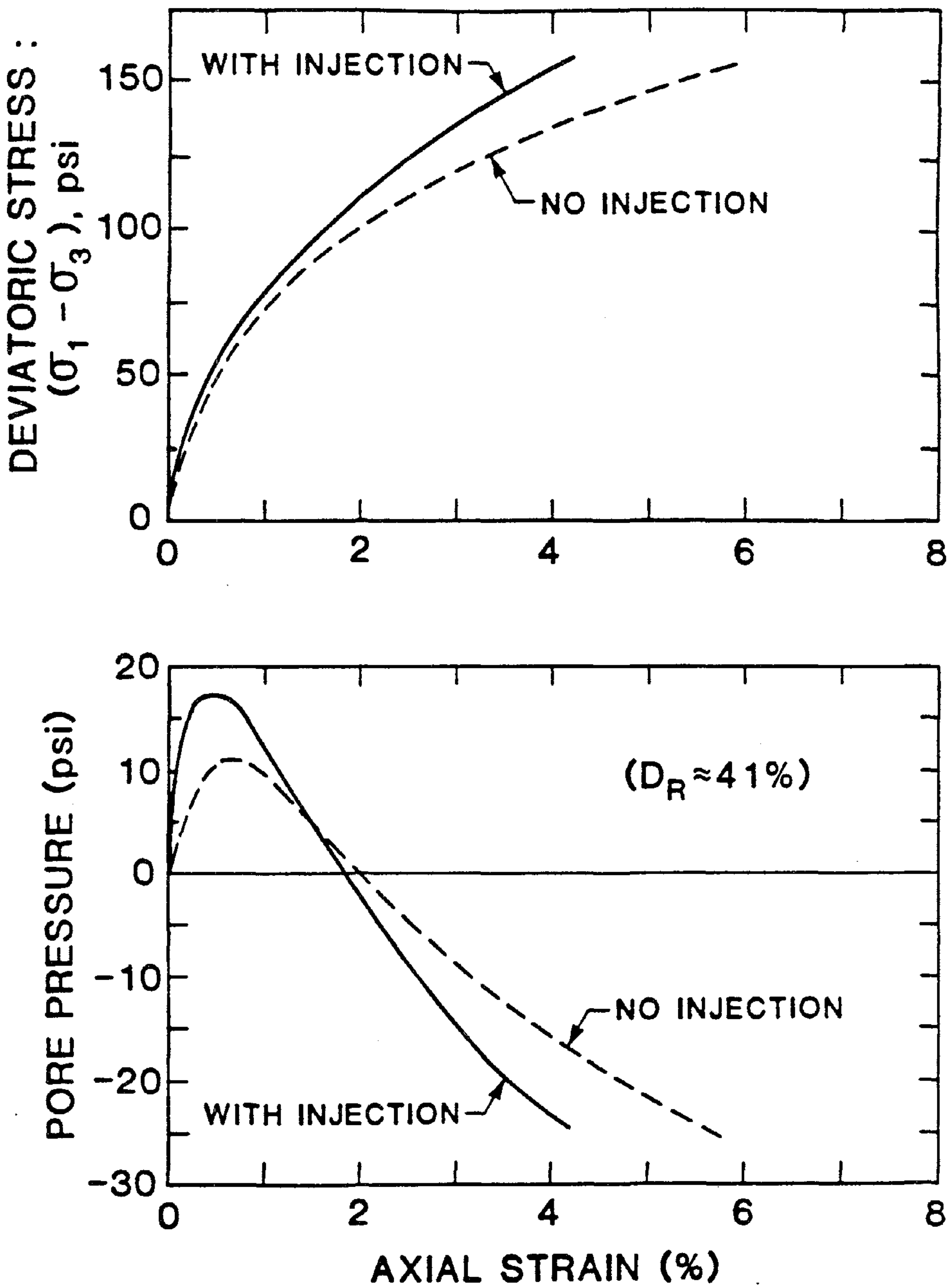


Figure 4.8: IC-U Triaxial Tests on 2.8-Inch Diameter Samples of Monterey 16 Sand With and Without Membrane Compliance Mitigation ($D_R \approx 41\%$)

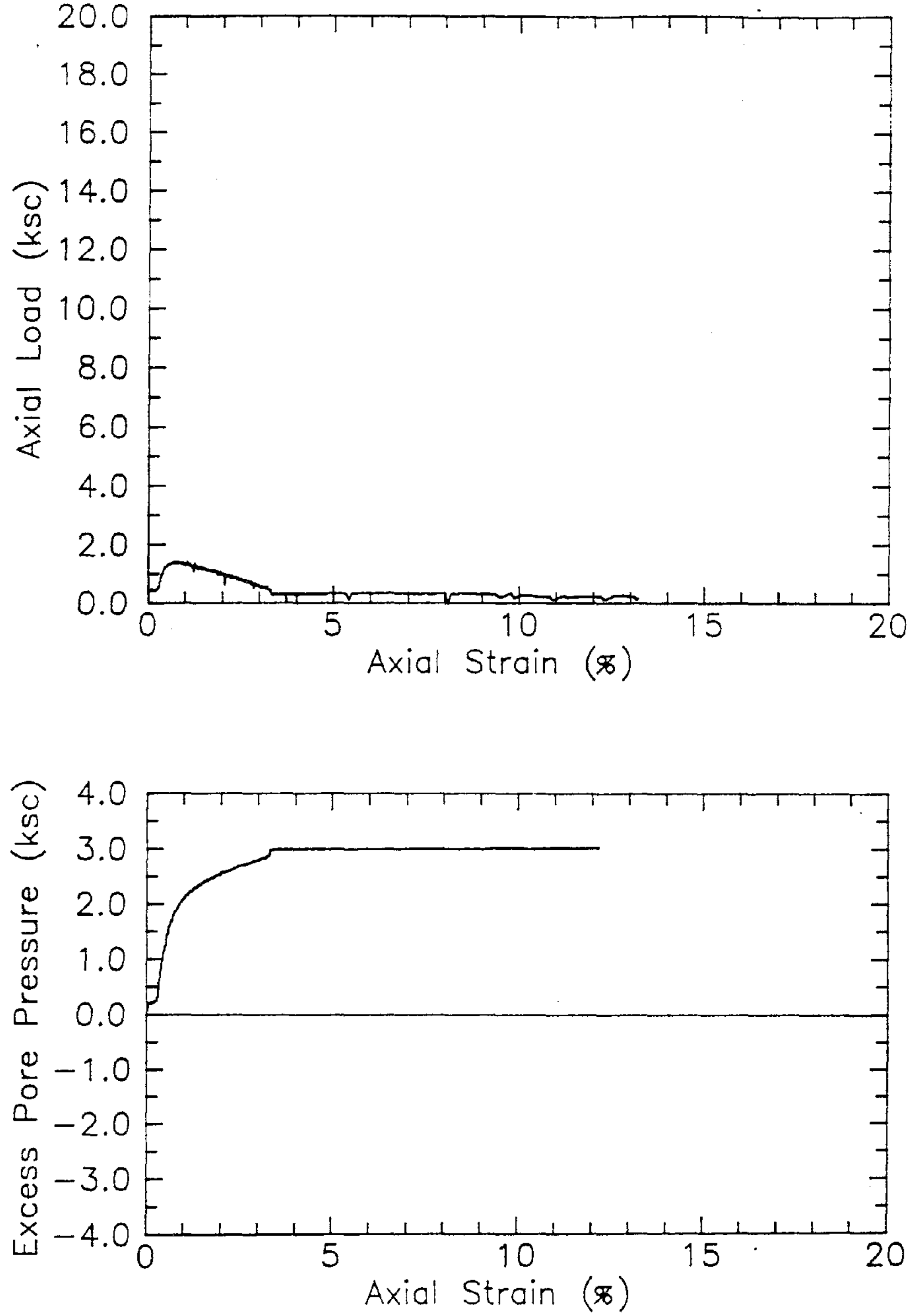


Figure 4.9: IC-U Triaxial Test No. PT-6 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 10\%$)

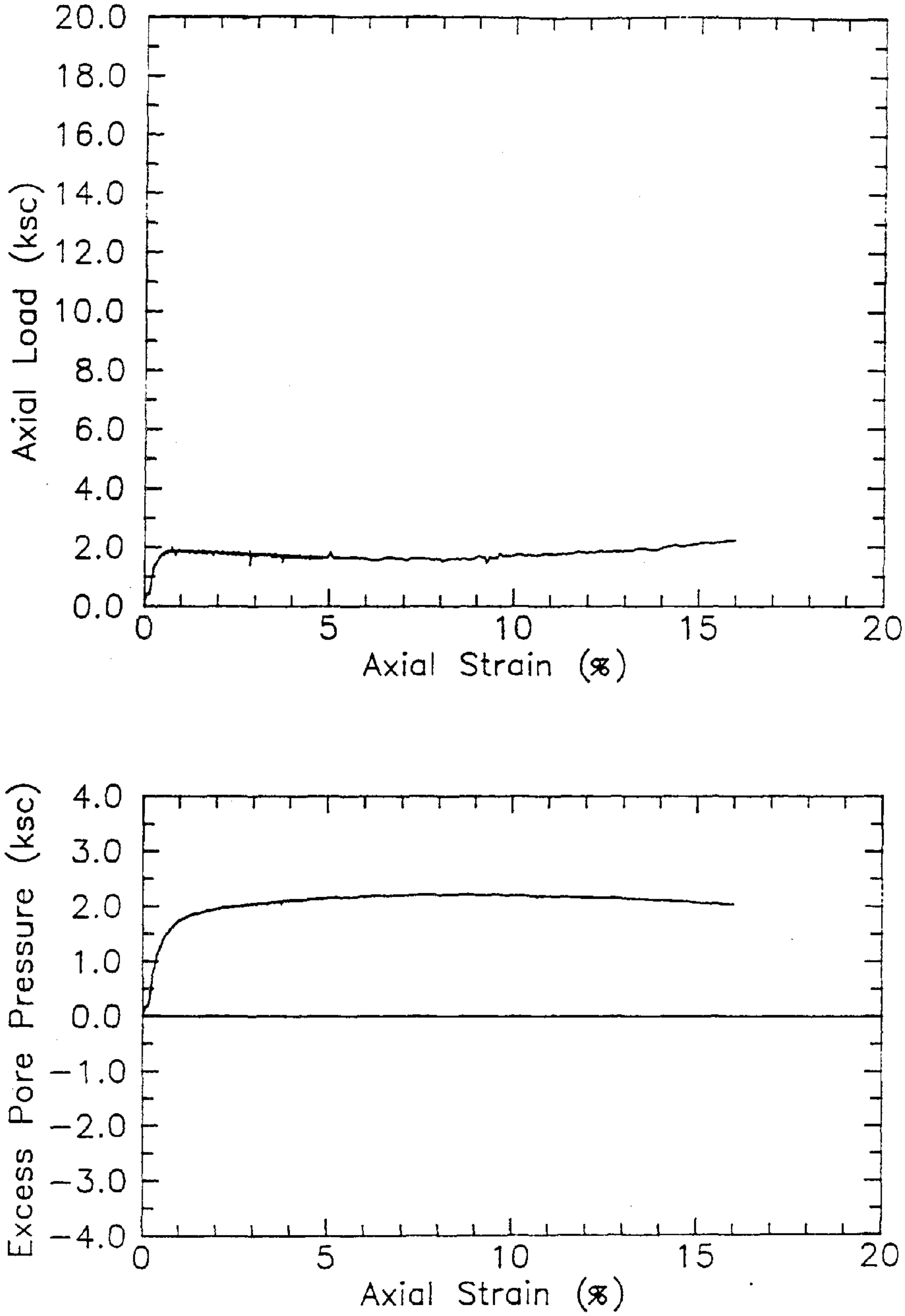


Figure 4.10: $\overline{\text{IC}}$-U Triaxial Test No. PT-7 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 16\%$)

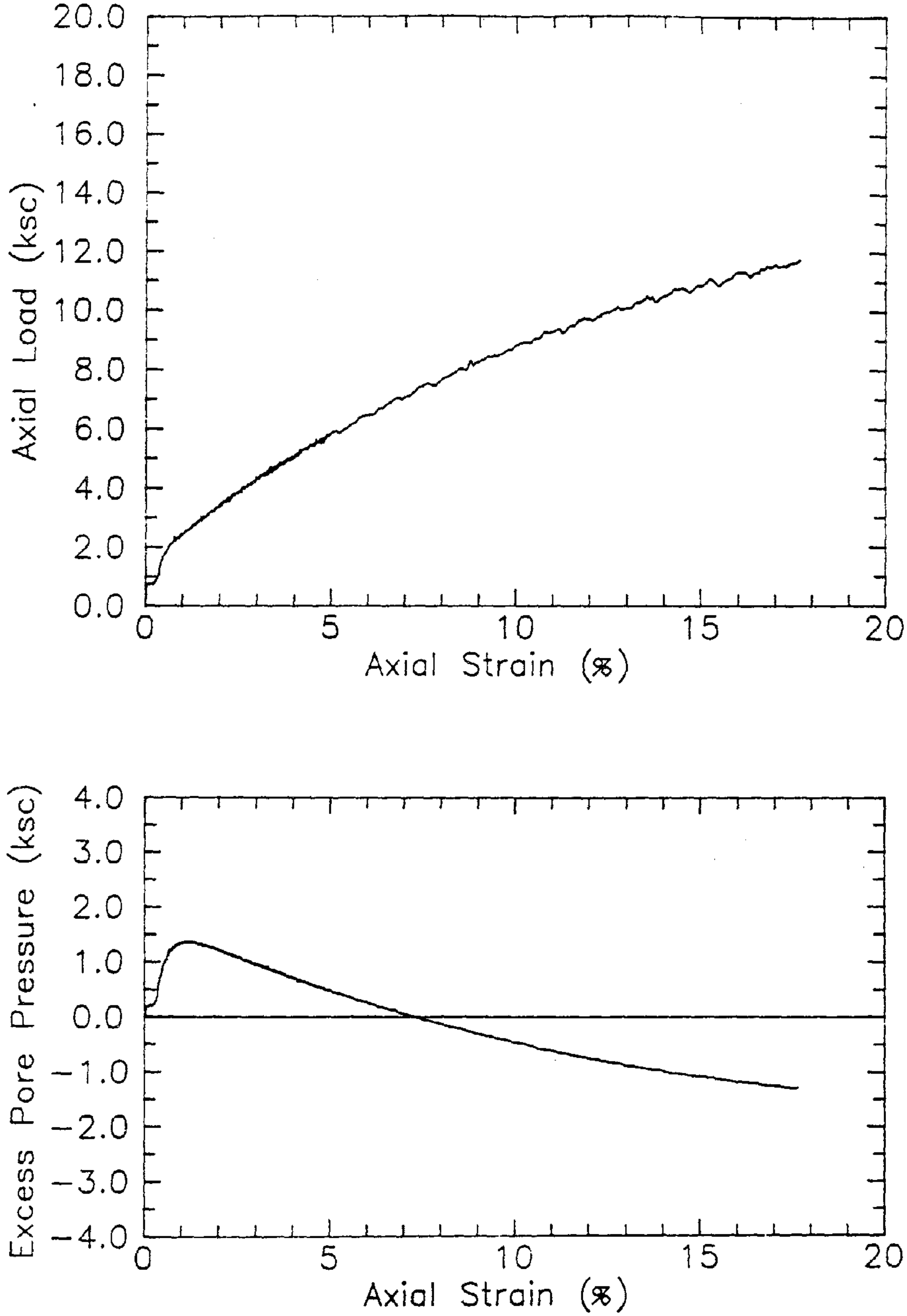


Figure 4.11: $\overline{\text{IC-U}}$ Triaxial Test No. PT-5 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 27\%$)

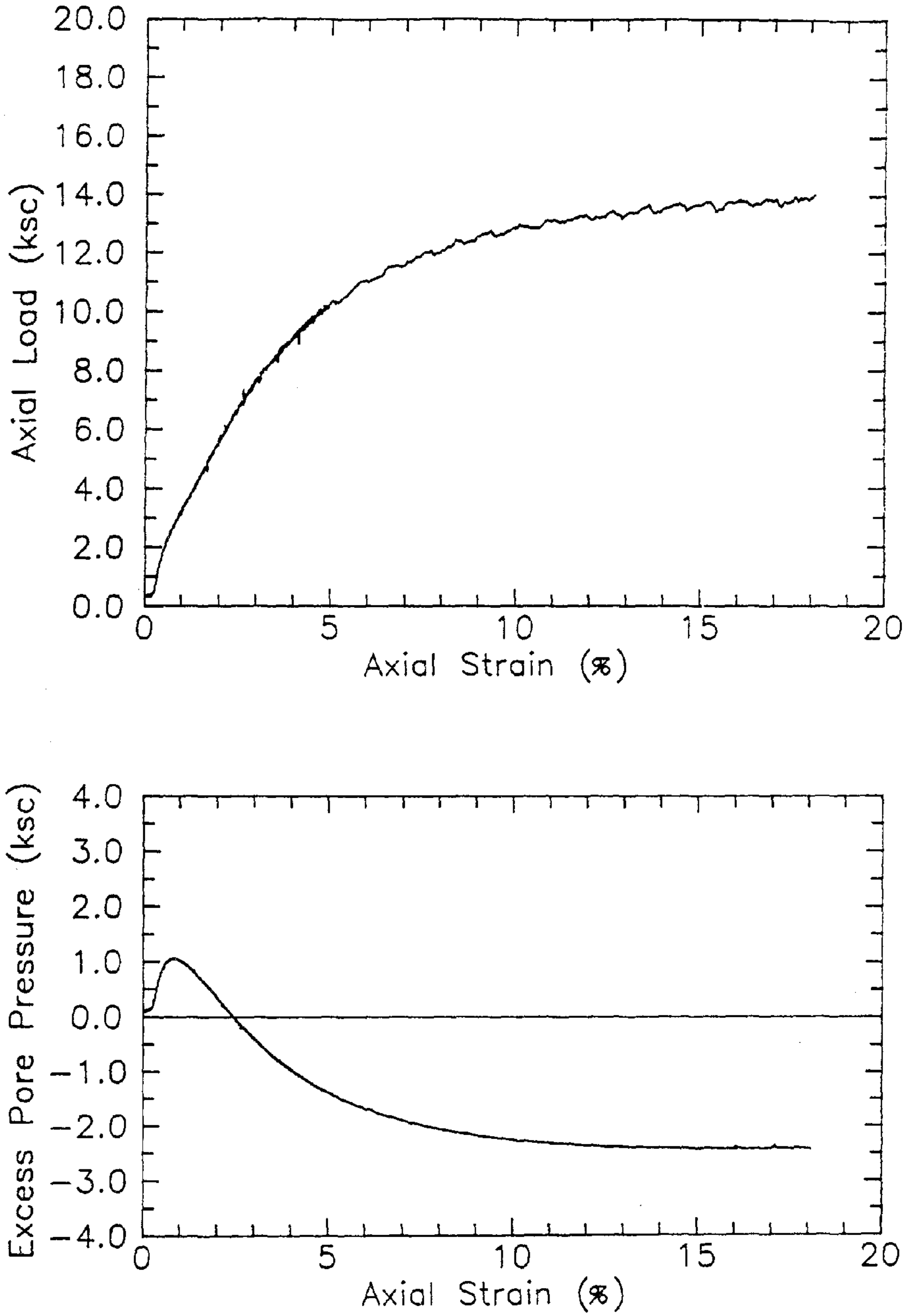
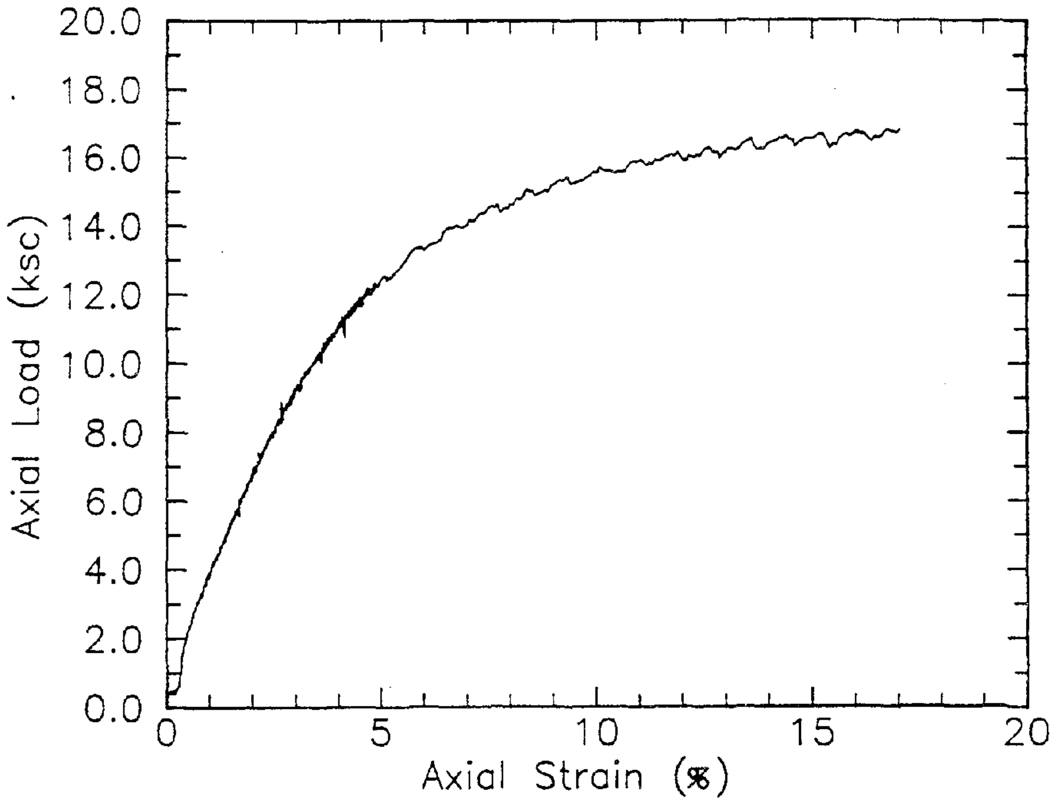


Figure 4.12: IC-U Triaxial Test No. PT-4 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 31\%$)



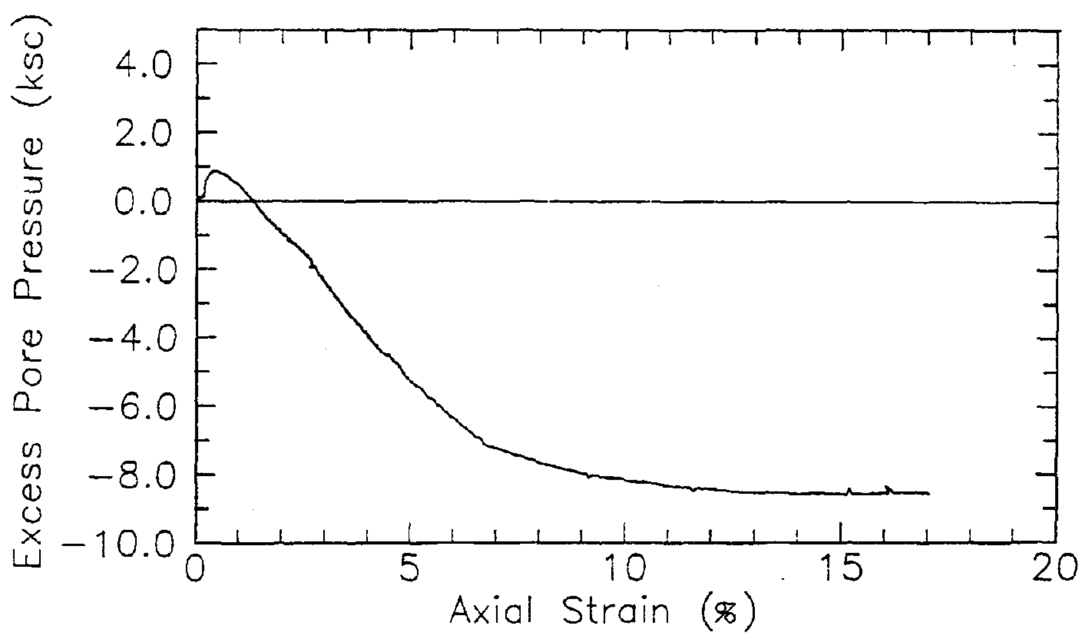


Figure 4.13: IC-U Triaxial Test No. PT-8 (12-Inch Diameter Sample of Monterey 16 Sand, $D_R \approx 37\%$)

Table 4.2: Isotropically Consolidated Undrained Cyclic Triaxial Tests on Monterey 16 Sand With and Without Membrane Compliance Mitigation

(a) 2.8-inch diameter samples:

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,i}$ (psi)	B-Value	CSR ($\sigma_{d,c}/2\sigma'_a$)	No. of Cycles to ± 5% ϵ_A
1A	55.5	Yes	29.4	0.993	0.303	5
2A	55.7	Yes	29.4	0.987	0.248	6
3A	54.8	Yes	29.4	0.993	0.199	79
4A	55.2	Yes	29.4	0.989	0.271	5
5A	55.9	Yes	29.4	0.992	0.220	18
1B	55.0	No	29.4	0.992	0.305	8
2B	56.0	No	29.4	0.996	0.255	26
3B	55.8	No	29.4	0.988	0.203	See Note
4B	54.1	No	29.4	0.992	0.226	84
5B	55.7	No	29.4	0.984	0.269	10

Note: Test No. 3B was stopped at 400 cycles with pore pressure ratio $r_u \approx 0.6$.

(b) 12-inch diameter samples:

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,i}$ (psi)	B-Value	CSR ($\sigma_{d,c}/2\sigma'_a$)	No. of Cycles to ± 5% ϵ_A
PT-11	55.1	No	29.4	0.979	0.255	6
PT-12	56.3	No	29.4	0.980	0.218	18
PT-14	55.3	No	29.4	0.975	0.283	4

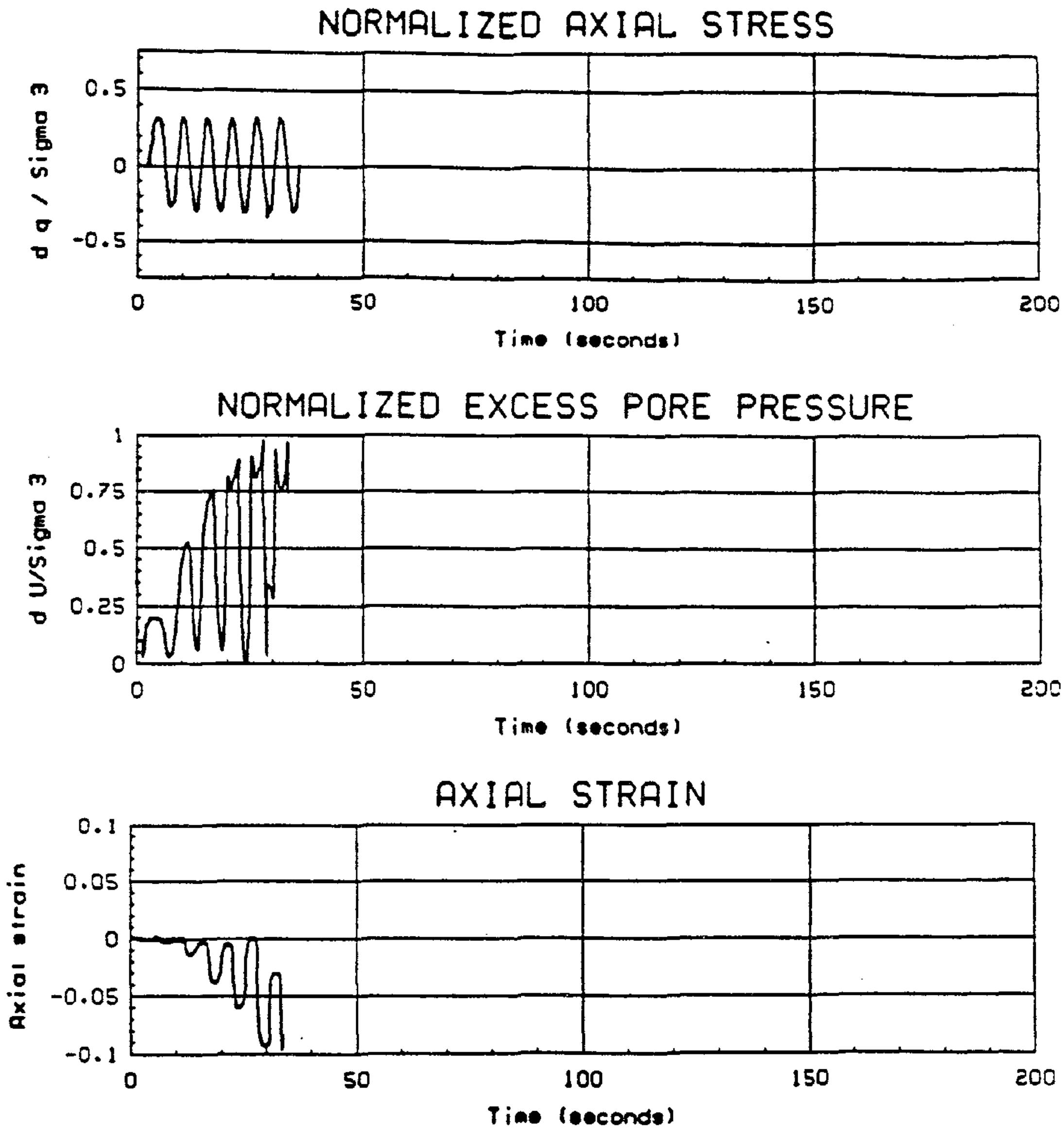


Figure 4.14: Undrained Cyclic Triaxial Test No. 1A (Monterey 16 Sand)

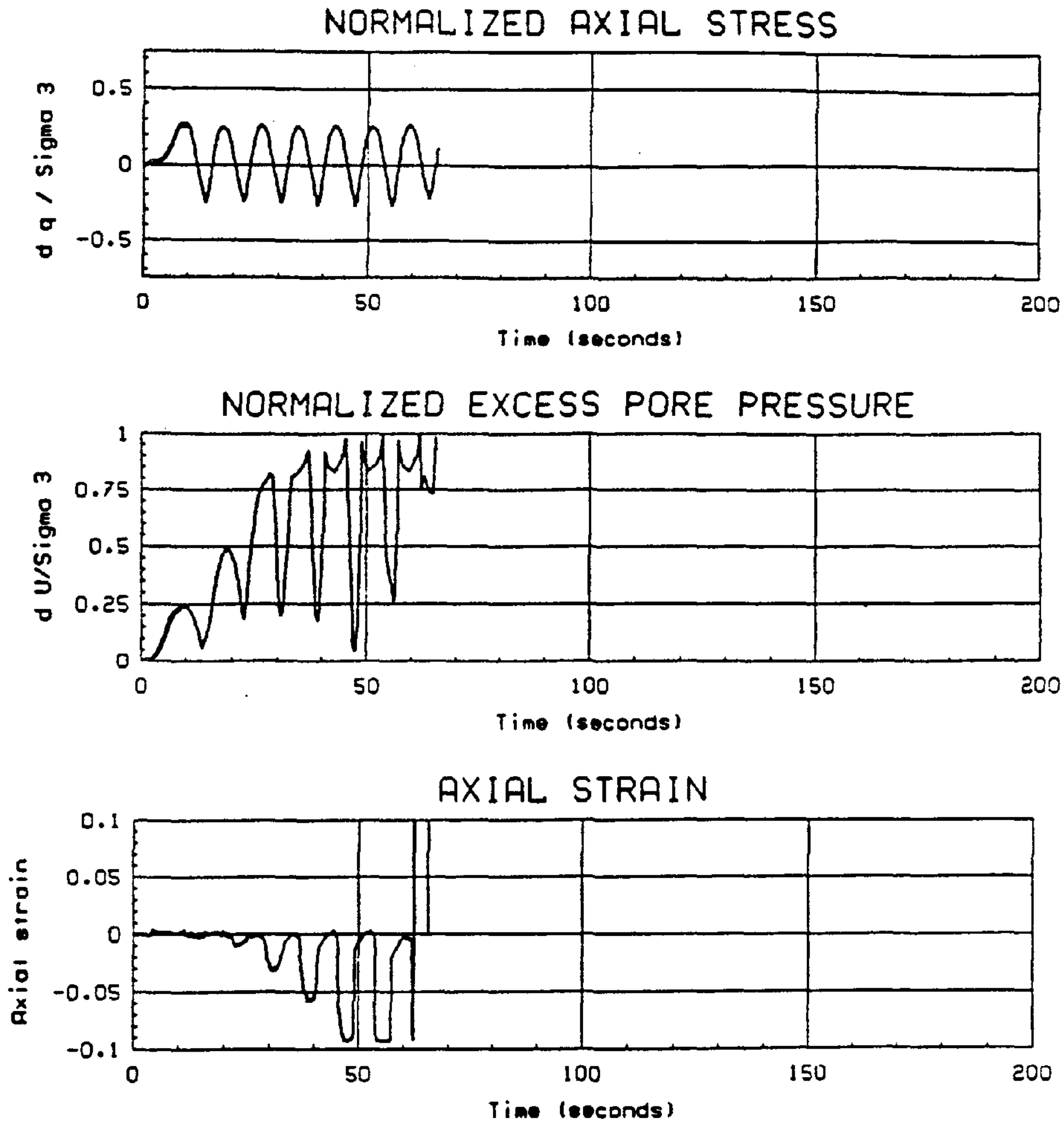


Figure 4.15: Undrained Cyclic Triaxial Test No. 2A (Monterey 16 Sand)

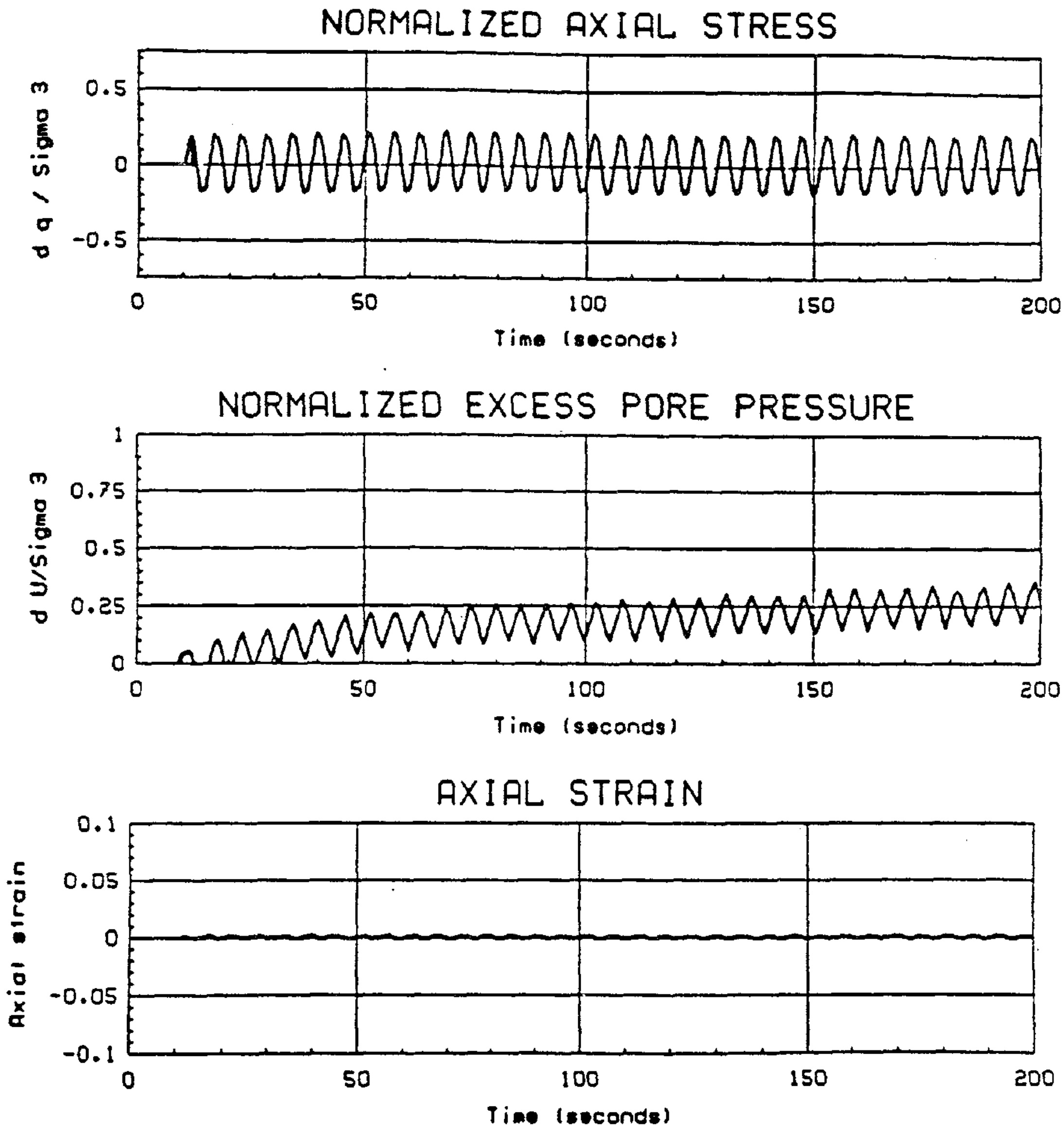


Figure 4.16: Undrained Cyclic Triaxial Test No. 3A (Monterey 16 Sand)

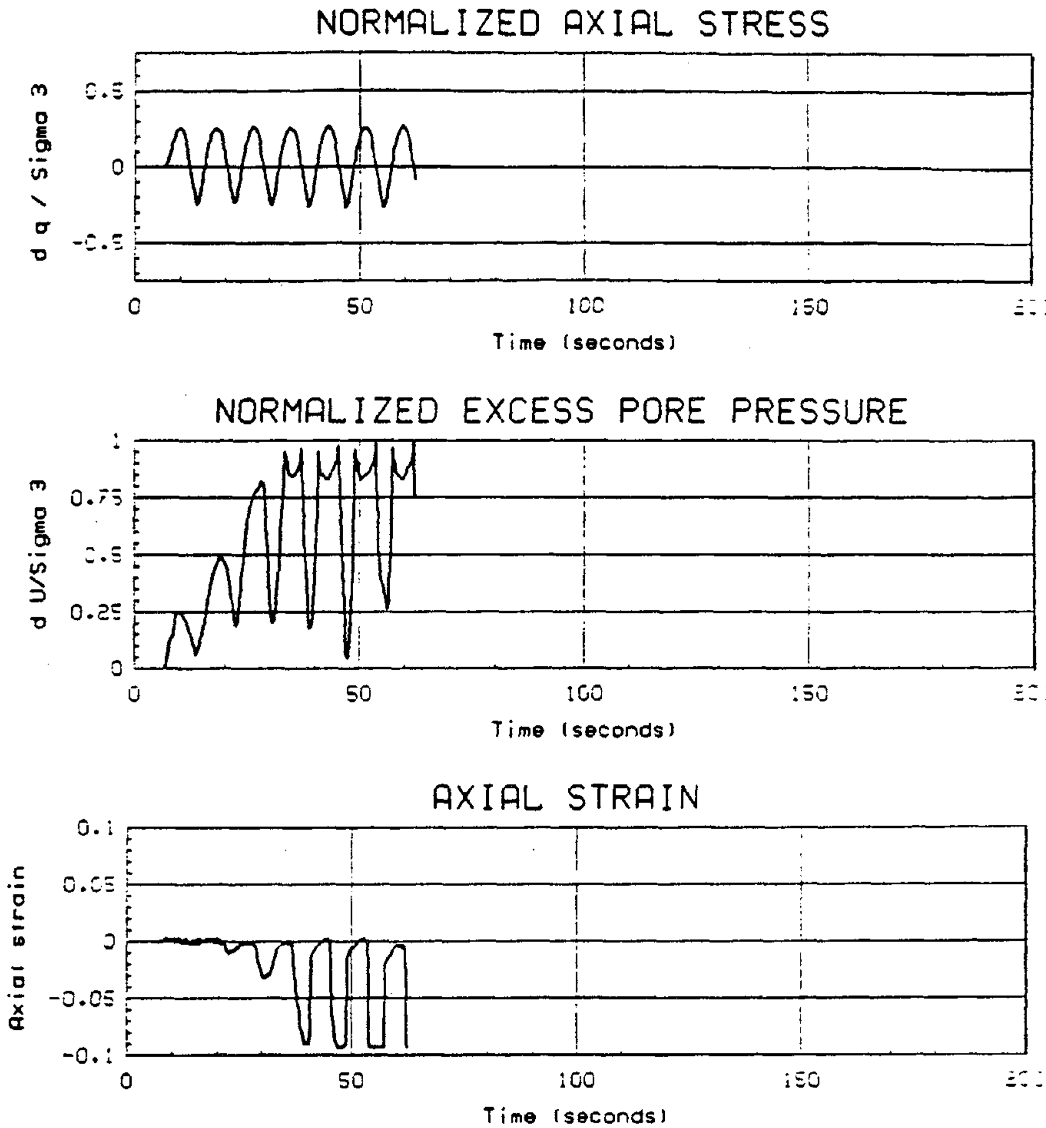


Figure 4.17: Undrained Cyclic Triaxial Test No. 4A (Monterey 16 Sand)

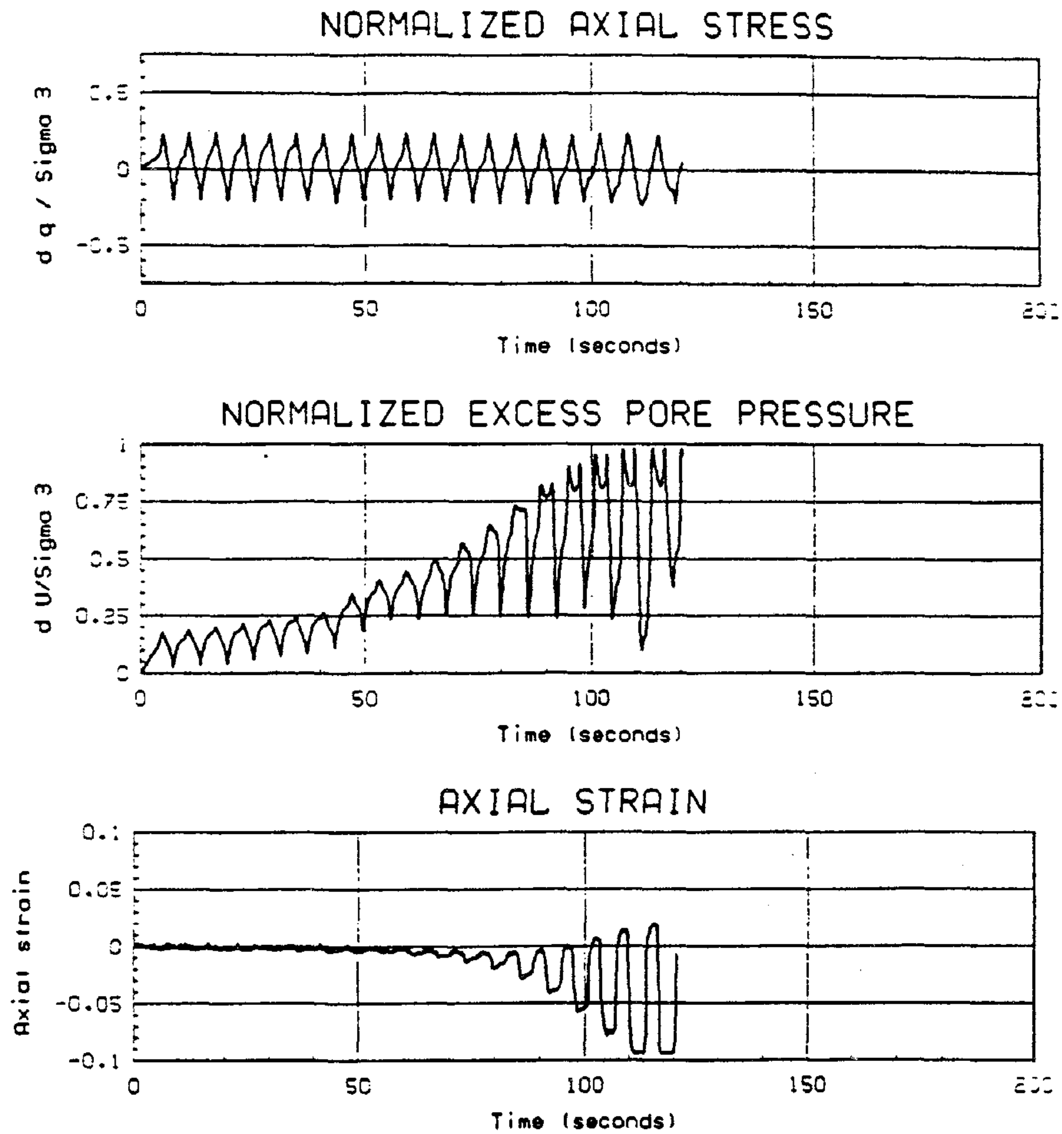


Figure 4.18: Undrained Cyclic Triaxial Test No. 5A (Monterey 16 Sand)

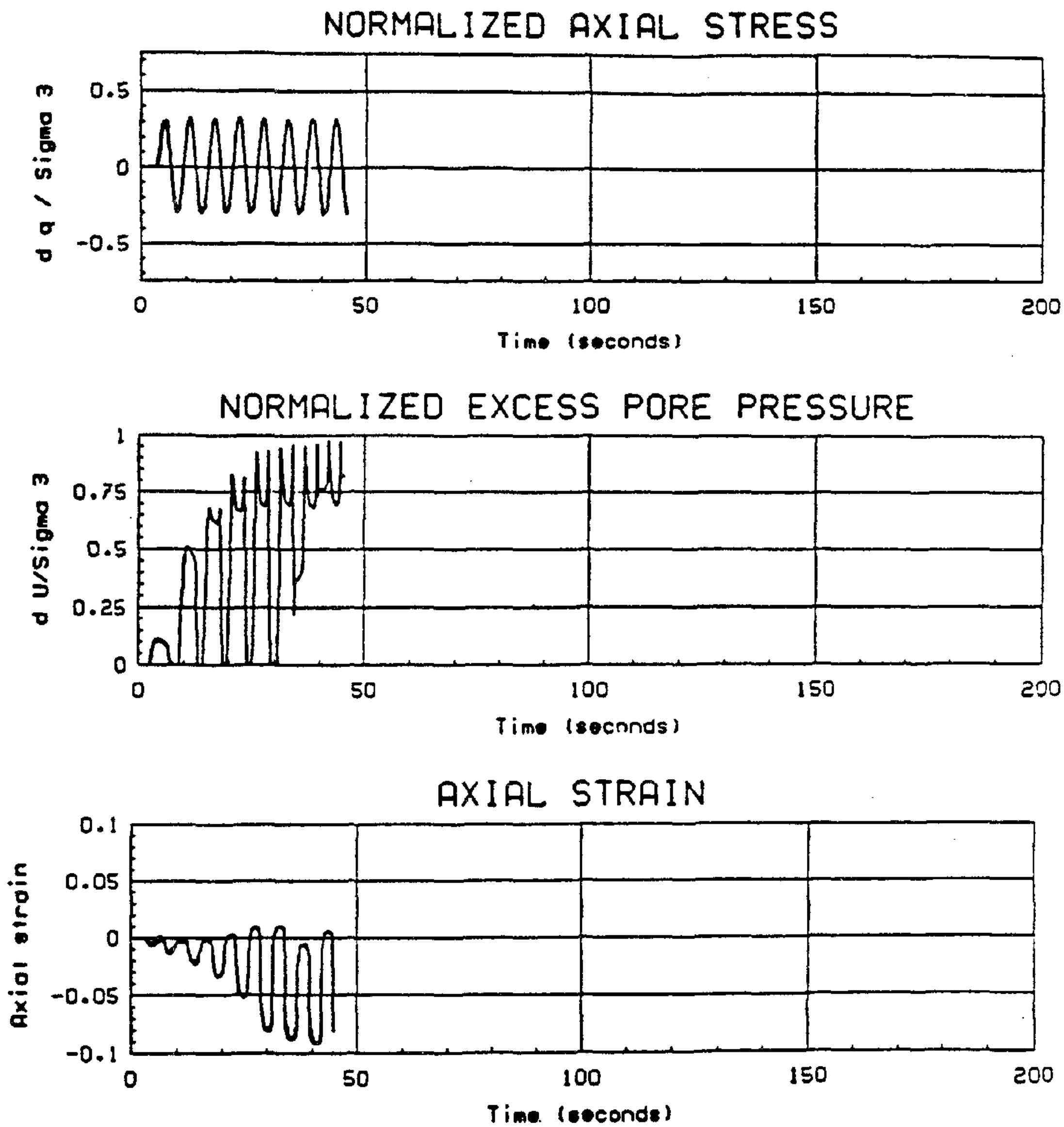


Figure 4.19: Undrained Cyclic Triaxial Test No. 1B (Monterey 16 Sand)

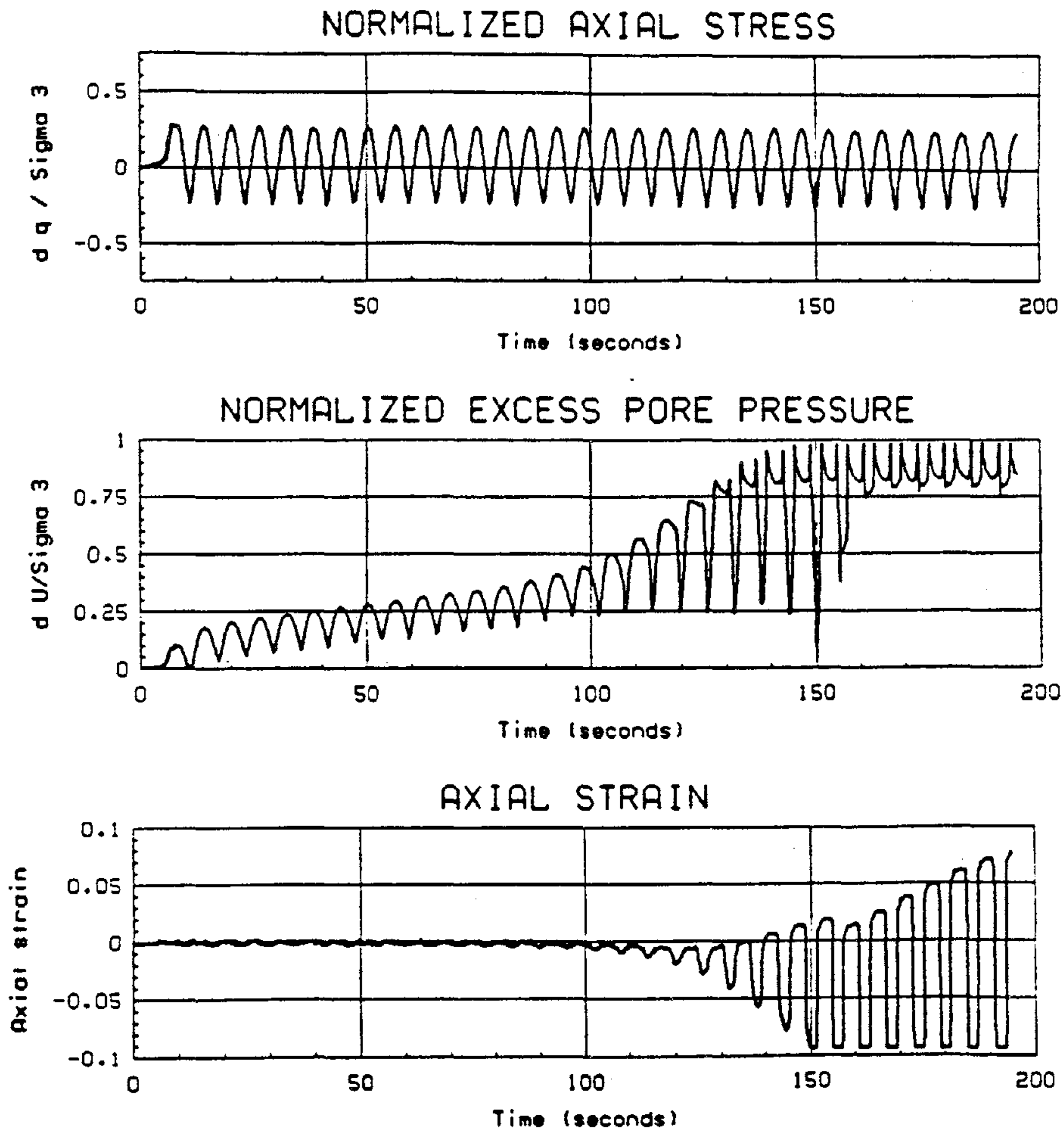


Figure 4.20: Undrained Cyclic Triaxial Test No. 2B (Monterey 16 Sand)

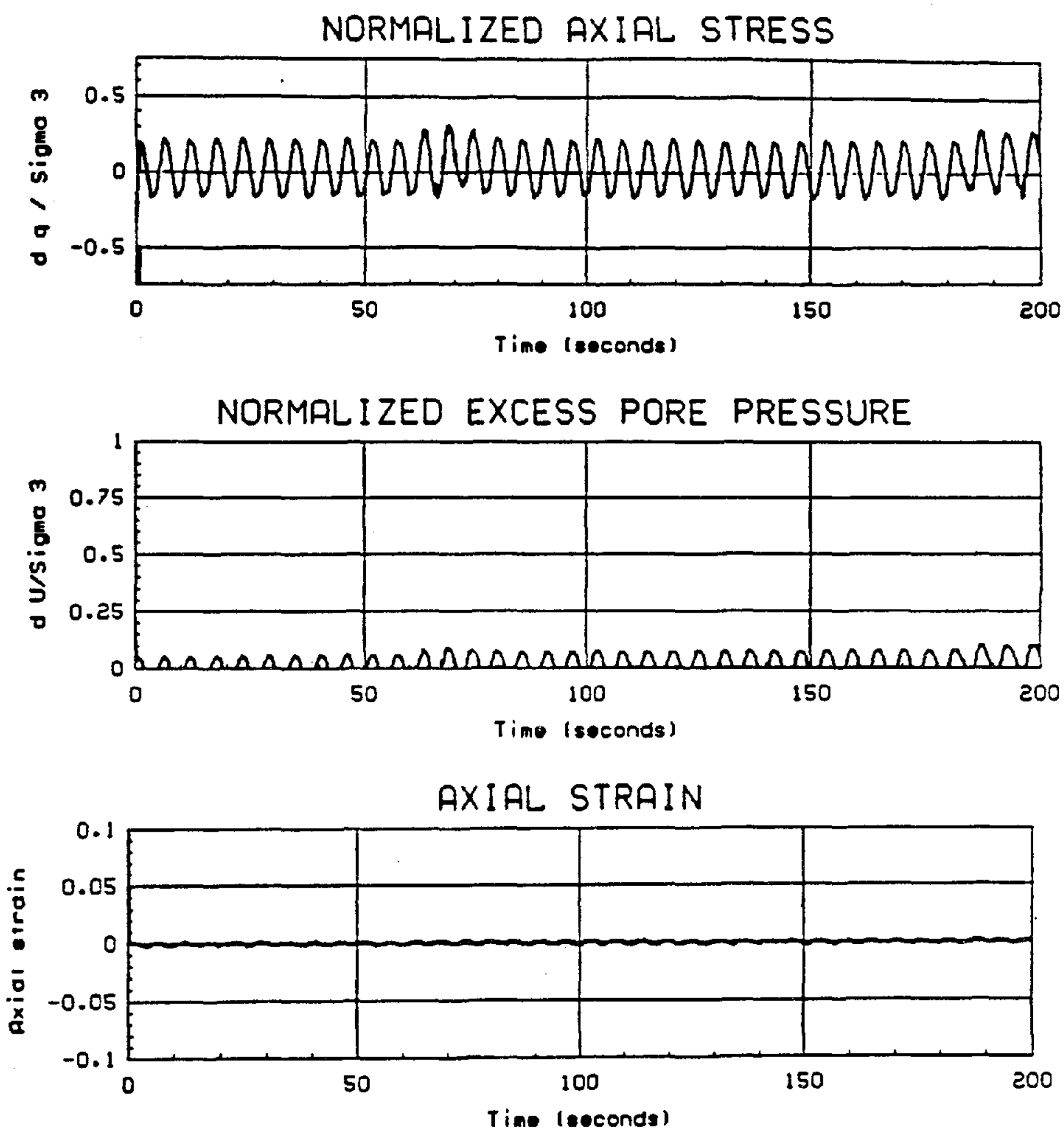


Figure 4.21: Undrained Cyclic Triaxial Test No. 3B (Monterey 16 Sand)

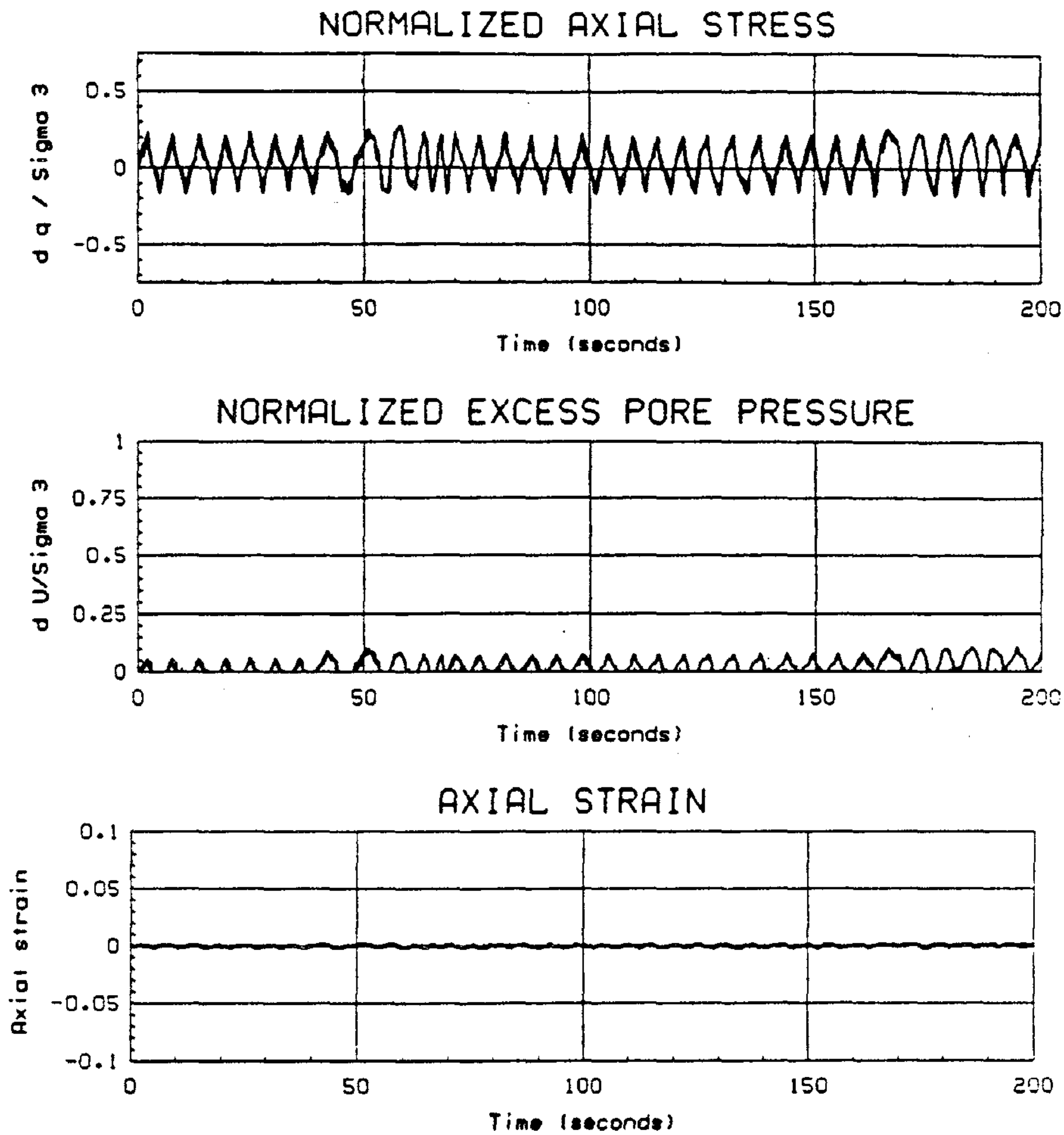


Figure 4.22: Undrained Cyclic Triaxial Test No. 4B (Monterey 16 Sand)

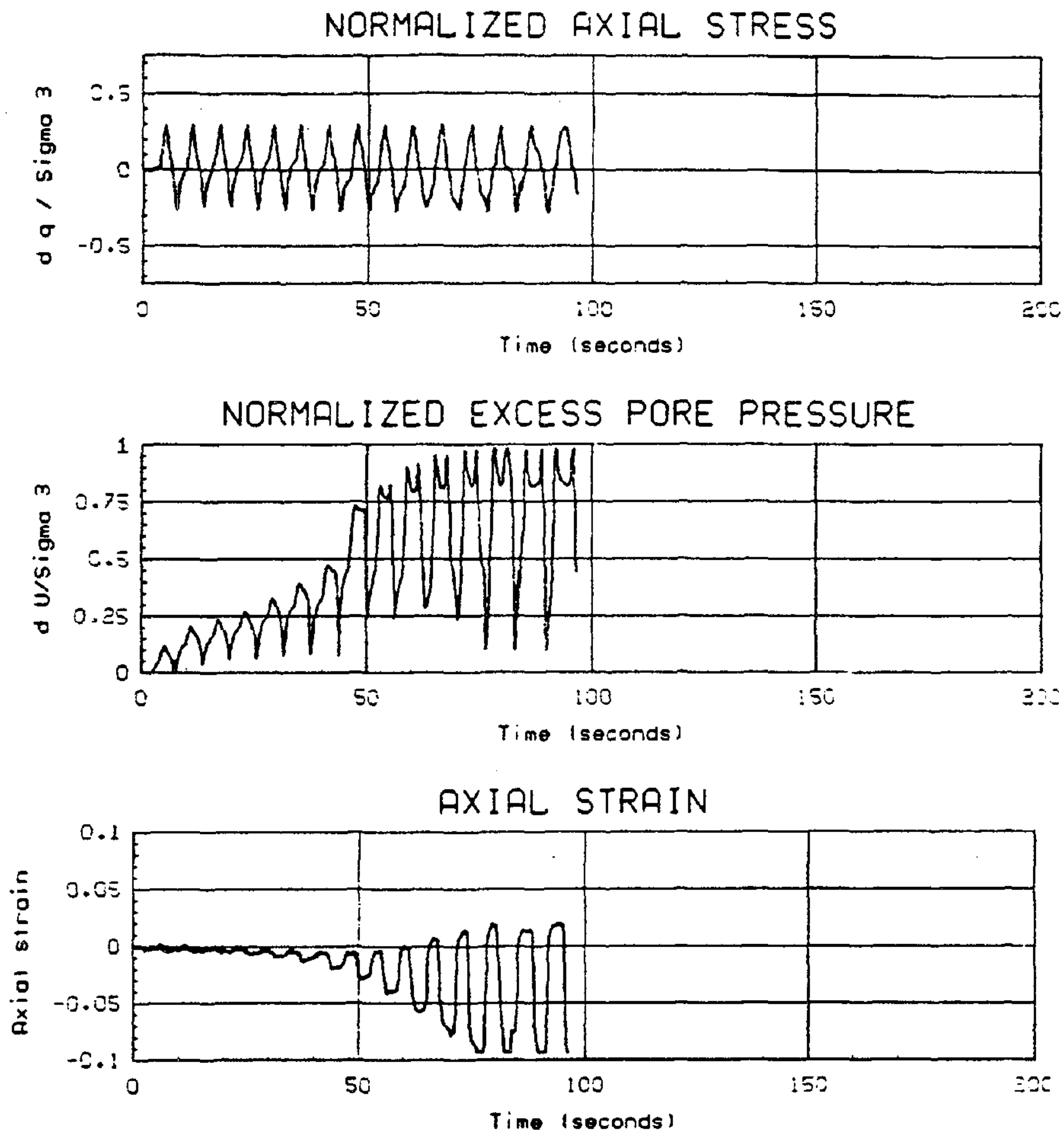


Figure 4.23: Undrained Cyclic Triaxial Test No. 5B (Monterey 16 Sand)

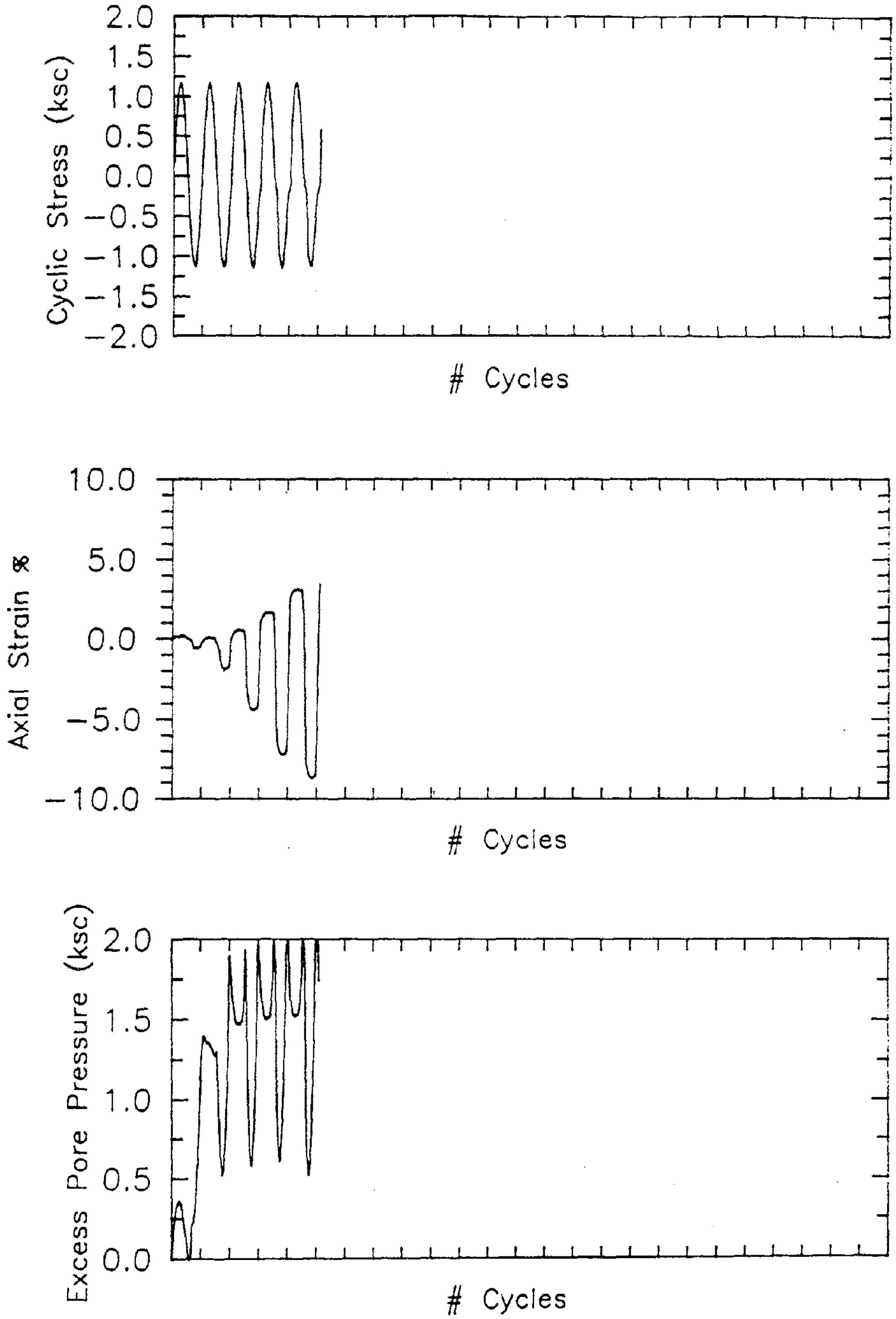


Figure 4.24: Undrained Cyclic Triaxial Test No. PT-14 (Monterey 16 Sand)

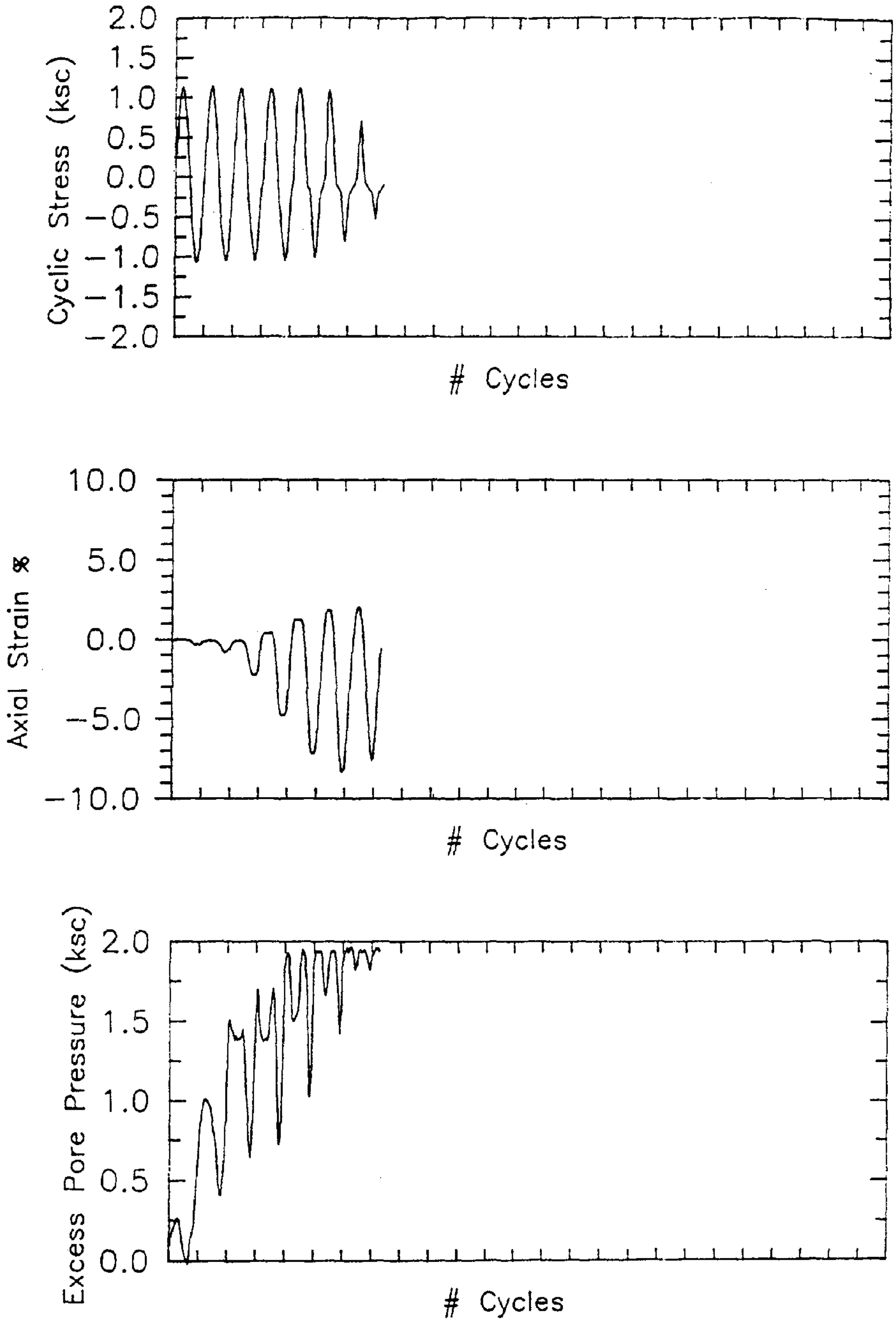


Figure 4.25: Undrained Cyclic Triaxial Test No. PT-11 (Monterey 16 Sand)

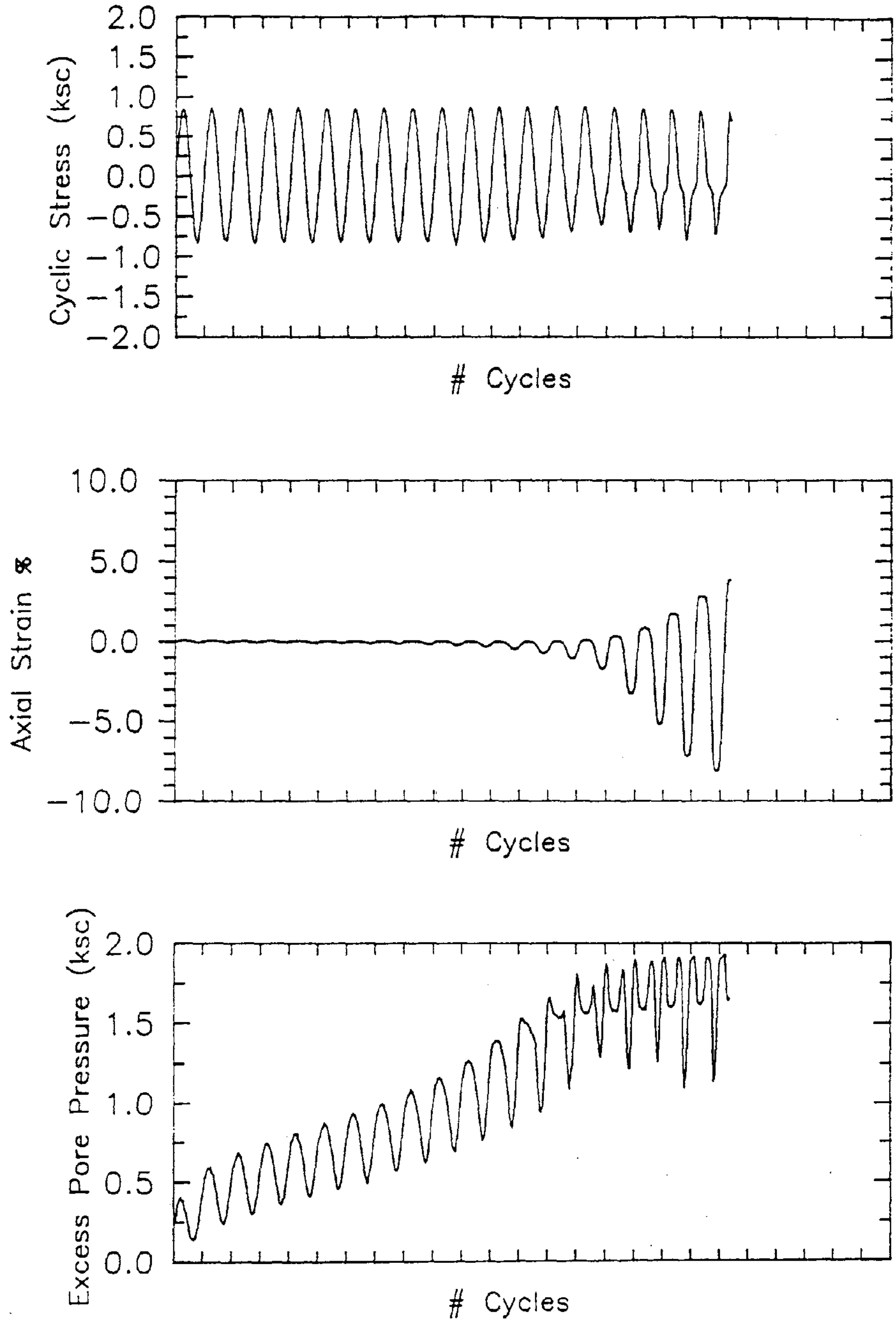


Figure 4.26: <u>Undrained Cyclic Triaxial Test No. PT-12 (Monterey 16 Sand)</u>

to incorporate the larger volume corrections anticipated for large scale (12-inch diameter) samples of significantly coarser materials.

4.3 Testing of 12-Inch Diameter Samples

4.3.1 Testing Equipment/Hardware

The triaxial testing set-up for 12-inch diameter samples was located at the California Department of Water Resources Rockfill Testing Facility in Richmond, California. Figures 4.27 and 4.28 show a photograph and schematic of the testing set-up, respectively. The testing system consisted of a servo-controlled 12-inch hydraulic ram controlled by an IBM PC-AT compatible microcomputer equipped with a Metrobyte A/D board to convert between analog and digital signals. A 100,000 lb. load cell was mounted at the top of the confining chamber with which loads could be read with an accuracy of greater than ± 0.5 psi. Axial displacements of up to 10 inches were recorded by means of an LVDT mounted to the load ram, such that ± 5 inches could be applied to the samples, and axial deformations were continuously monitored with an accuracy of ± 0.01 inches.

Effective sample confining pressures were continually monitored by a calibrated differential pressure transducer with one side connected to the back-pressure/pore-pressure line and the other directly to the top of the testing chamber to monitor chamber pressure. Confining pressures were monitored with an accuracy of ± 0.005 ksc. Both the back-pressure and the chamber pressure were also visually monitored by separate pressure gages with scaled increments of 0.01 ksc.

Volume change devices included a set of three calibrated burette cylinders, each with a different diameter, so as to be able to accommodate a wide range of different possible volume changes which various soils would generate, without

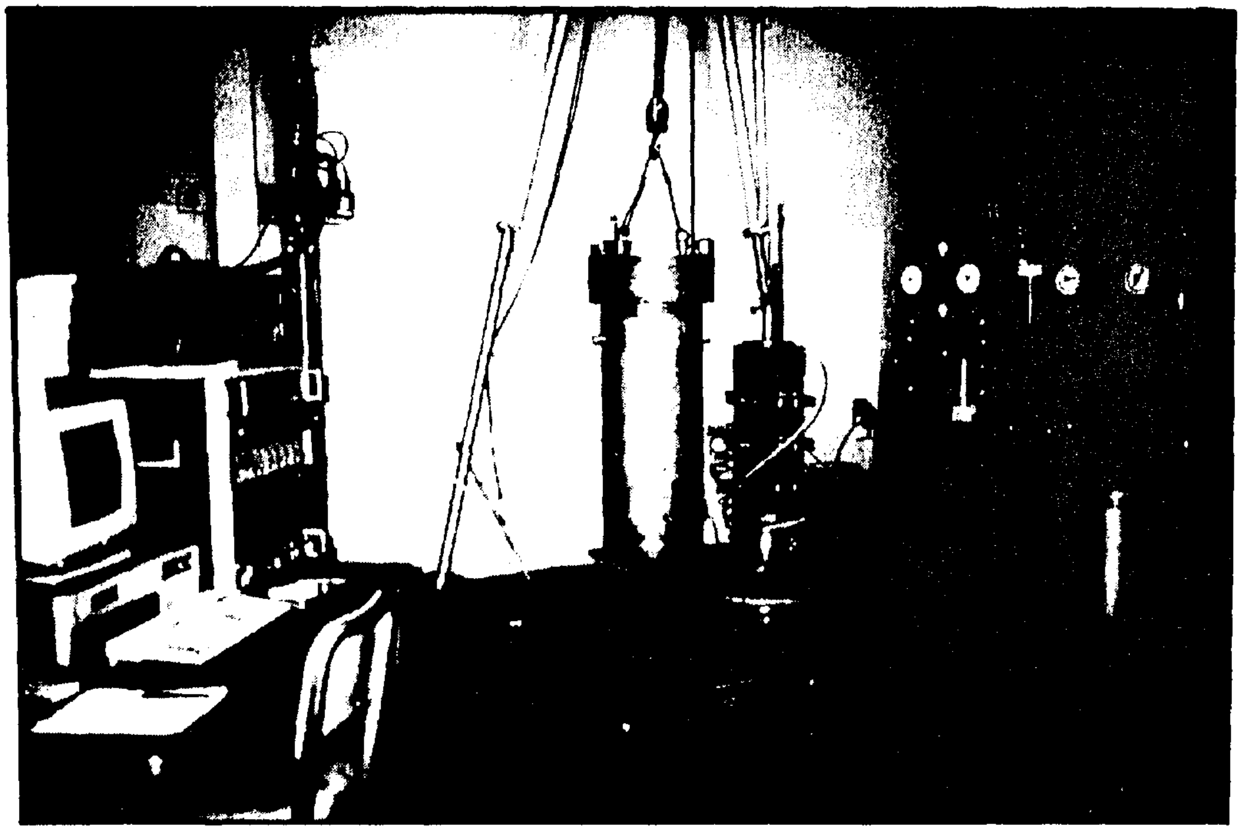


Figure 4.27: Photograph of Large-Scale Testing Equipment for Testing 12-Inch Diameter Specimens

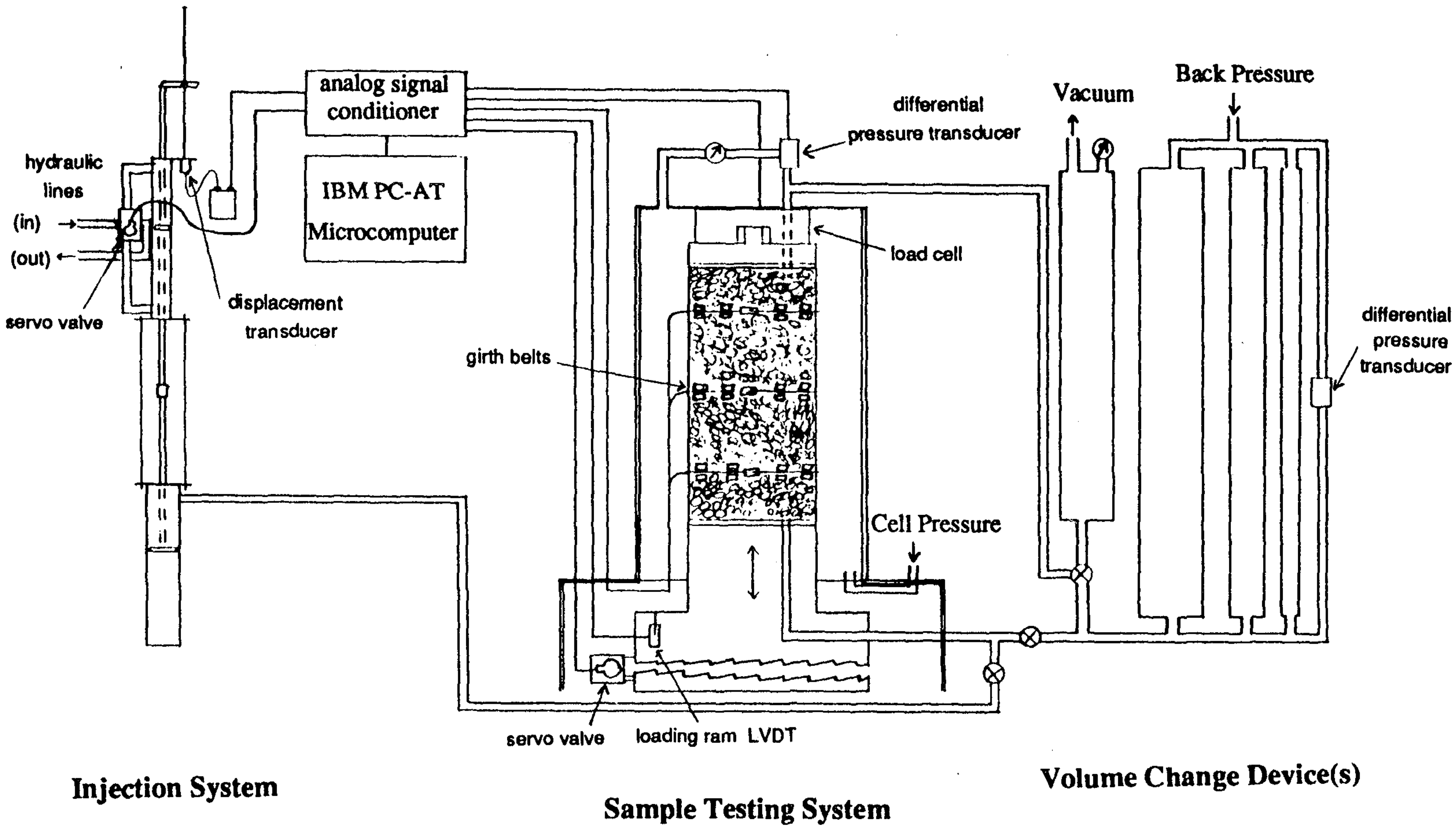


Figure 4.28: Schematic Illustration of Large-Scale Testing Set-Up for Testing 12-Inch Diameter Specimens

sacrificing accuracy of volume change readings. A differential pressure transducer was connected between the top and bottom of the volume change measuring system so that volume change readings could also be recorded directly by the data acquisition recorder. Depending on which of the three burette cylinders was used, the accuracy of volume change measurement ranged from $\pm$ 0.04 in^3 to $\pm$ 0.2 in^3 or roughly .005% to .025% of the overall sample volume.

All LVDT's, load cells, and pressure transducers were connected to analog signal conditioners (Paul Gross Associates SC-5, SC-5A) where voltage gains and sensitivities could be calibrated to give the best range of signals while maintaining the highest level of accuracy possible. The signal conditioners were then connected to the Metrobyte A/D conversion board installed in the micro-computer.

4.3.2 Controlling Computer Software

The tests were performed using an automated testing control program (ATS) from Digital Control Systems, Berkeley, which is capable of controlling monotonic and dynamic loading test as well as displacement-controlled tests. This software package was found to give excellent control of testing parameters as gains and sensitivities for each control channel could be input at the beginning of each test and could also be edited and updated during testing if necessary for testing accuracy. The ATS software is also capable of recording up to 16 channels of data and writing the data to a specified output file in the desired testing units. Unique test-specific output recording schedules could be easily designed and selected by the software program, enabling the user to choose the frequency of data acquisition for any number of different parameters during the tests. The software can also simply convert any data to other desired units through a built-in subroutine. The ATS testing programs could be quickly and efficiently modified to alter all pertinent testing parameters, and the system was able to run up to four different types of tests simultaneously. This simultaneous test ability made it possible to repeat the same test with or without the compliance mitigating injection program, so that only one variable would be changed between the two tests.

4.3.3 Sample Preparation

All large-scale specimens were nominally 12-inches in diameter and approximately 24 to 26 inches tall. Samples were prepared by "moist tamping" whereby 8 equal volume layers were compacted within a 12-inch diameter steel mold using a hand-held tamper with a round 5-inch diameter plate at the end of a rod. Each layer of material was individually prepared with the proper weighed gradation and completely mixed at a specified water content in order to achieve a consistent uniformity throughout the layer. The length of the tamping rod was adjusted for each layer to ensure that the material was compacted to the correct volume and thus to the desired density. Uniform density throughout the entire sample was achieved by varying the weight of soil in each layer with slightly less material at the bottom and increasing incrementally towards the top of the sample, as the compaction of each successive layer tends to further densify previously compacted layers. Different amounts of weight variations were tried for each desired sample density until the samples displayed uniform densities. Higher density samples required less variation of weight between layers than did loose samples, as the lower layers of the denser samples resisted further densification from the compaction of overlying layers. The flat tamping plate and tamping guide assured a "flat" horizontal surface on top of the last compacted layer on which a porous disk and top cap were placed. It was found that in order to construct gravel samples with very low relative densities (on the order of 10-25%) it was necessary to pluviate the material through standing water and then use the tamper to achieve the desired volume per layer. With some practice, uniform samples of different low densities could be constructed by varying the fall heights through different heads of standing water. The method of low density construction used by Evans and Seed (1987) of hand placing a few grains at a time into the mold was attempted, but it was found that better control of uniformity could be achieved for very low densities by the "wet pluviation" method.

A vacuum of nearly one full atmosphere was applied to confine the sample while the sample mold was removed and the rubber membrane was securely sealed to the top cap. An initial check for leaks was conducted once the internal sample pressure reached that of the full vacuum, by checking to see whether or not the sample could hold the full vacuum while all valves to the sample were closed. If any leakage was detected, it was located and repaired before sample preparation was resumed.

Sample dimensions were then measured in order to evaluate the actual initial sample density. Height measurements were made at 90 degree intervals around the circumference of the sample, and the average of the four readings was recorded. Sample diameters were measured at three equally spaced intervals at the top, middle, and bottom quarter points of the sample height. This was done using a "pi tape" which is a flexible verniered scale, from which diameters can be read by wrapping the scale around the circumference of a sample. These were then corrected to account for membrane thickness.

Since accurate pore water pressure readings were critical to the types of undrained triaxial testing performed as part of this research, care was taken to ensure the highest possible saturation of samples prior to testing. Saturation was achieved by the following systematic "vacuum/back-pressure" saturation method.

Initially, deaired water was allowed to be drawn up through the sample from the bottom to the top under a small differential vacuum, while maintaining a pressure differential of not greater than 5.0 psi to safeguard against preconsolidating the specimen. Once water flowed out of the top of the sample with no further detectable traces of air bubbles, the vacuum line was shut off. The testing chamber was then secured in place and the sample was manually raised just until contact was made with the load cell at the top of the chamber. It was then securely fastened to the chamber and the chamber was sealed. As more deaired water was allowed to be drawn into the sample under the remaining vacuum in the sample, the chamber pressure was simultaneously increased to maintain the initial effective confining stress of approximately 1 atmosphere. While continually maintaining the initial effective pressure, the chamber pressure was slowly increased and a back-pressure was simultaneously applied to the sample pore water until it was estimated that sufficient back pressure had been applied to achieve a B-value of at least 0.97. At this stage a rigorous check of the B-value was made. The B-value parameter, equal to the ratio of the change in pore pressure for a given change in chamber pressure while the sample was in an undrained condition, was obtained by monitoring the change in effective stress while a small increment of additional chamber pressure was applied, and making the necessary computations. If saturation was not deemed sufficient, chamber pressure and back-pressure were increased further, and another B-value check was made. This process was continued until the minimum standard of saturation ($B \geq 0.97$) was achieved or exceeded. In some cases it was necessary to use tanks of compressed air to achieve the high chamber and back pressures necessary for high degrees of saturation.

Samples were then fully consolidated under the desired effective consolidation stress, after which they were ready for testing. A minimum consolidation time of approximately two hours was typically used even for rapidly draining coarse gravels, as it has been demonstrated by previous investigators (Molenkamp and Luger, 1981; Baldi and Nova, 1984; Seed and Anwar, 1986) that creep effects and strain softening of the membranes may affect compliance values.

CHAPTER 5

DEVELOPMENT OF A LARGE SCALE MEMBRANE COMPLIANCE MITIGATION SYSTEM

5.1 Introduction

There has been great progress made over the last 25 years in the area of obtaining more correct undrained test results for materials that are prone to the errors induced by membrane compliance. But prior to this effort, there had not yet been any verification of a reliable and accurate means by which to eliminate or correct for the volumetric error introduced by membrane compliance during undrained testing. As described in the previous chapter, the method of continuous computer-controlled injection or removal of water to or from a sample to offset the membrane compliance-induced error, has been verified as just such a valid method of achieving this goal for "conventional" (2.8-inch diameter) small-scale samples.

It has been clearly shown by numerous investigators that the effects of membrane compliance become more pronounced for coarser materials. As pointed out in Chapter 1, many earth structures are constructed of, or founded on, coarser materials (particularly gravels) which may be susceptible to significant membrane compliance errors. These coarser soils have historically been considered to have a "strong" resistance to liquefaction based on tests in which membrane compliance induced errors may have been quite significant. There is therefore a need to develop large scale testing techniques which will be able to provide more accurate assessments of the actual undrained strengths of these coarser materials. Unfortunately, the "coarseness" of the material that can be tested in the small-scale apparatus is limited by the sample size constraints of conventional triaxial testing equipment. It has been suggested (Seed, 1979) that the maximum ratio of material particle size to sample diameter should not exceed 1:8 for uniformly graded materials and 1:6 for other soil gradations. As a result, the coarsest material that can be accurately evaluated for its undrained strength or resistance to liquefaction under cyclic loadings using conventional testing equipment, even with the implementation of a continuous computer-controlled compliance mitigation system such as that developed by Seed and Anwar (1986), is a coarse sand. For materials coarser than coarse sand, larger testing equipment must be employed.

The first step in developing this type of compliance-mitigation system is to show that the amount of volumetric compliance can be accurately and reliably pre-determined for a soil specimen prior to any test in which a compensating correction would be made based on such a pre-determination. Once this is achieved, a system must be designed and constructed that will continuously and completely offset that volume error with sufficient speed and accuracy throughout undrained testing of a specimen. Therefore, the methodology used in developing the large-scale system to completely mitigate membrane compliance includes two basic parts: (1) the volumetric magnitude of membrane compliance must first be pre-determined for the soil to be tested, as a function of the sample effective confining stress, and (2) the volumetric error introduced by membrane compliance during undrained testing must be continuously offset by injecting or removing an amount of water equal to the volumetric error by means of a computer-controlled system.

5.2 Pre-Determination of Membrane Compliance

For the "large-scale" triaxial samples, which had diameters of approximately 12 inches, membrane compliance was measured directly by recording axial and radial strains of the soil skeleton, and subtracting the "true" sample skeletal volume change from the total volume change measured by the volume change device. Axial strains were recorded by monitoring the LVDT that measured displacement of the 12-inch diameter loading piston. Radial strains were recorded using a set of three "girth belts" (discussed in Sections 2.2.1 and 3.3), each containing an LVDT, which measured changes in sample circumference during the application of varying confining stresses. Use of the radial girth belts allowed direct calculations of volumetric strains without making any of the customary assumptions regarding relationships between axial and radial strains.

For volumetric compliance evaluations made for samples of less than six inches in diameter, however, the measurement of radial strains is difficult to perform with sufficient accuracy as to make the use of radial measurements a viable method of pre-determining compliance magnitudes for such "smaller" samples. The method of assuming isotropic behavior during isotropic unloading tests, where volumetric strain is assumed to be nearly three times the axial strain, has been shown to lack reliable accuracy for a range of different soils. In considering the assumption of isotropic strain behavior proposed by Vaid and Negussey (1984), it should be noted that for the sand tested, the volume change due to membrane compliance was considerably greater than that of sample skeleton volume change, such that errors introduced by any inaccuracy of the isotropic strain behavior of the soil sample would be of much less significance than for a soil sample in which the volume change components were more closely equated. Instead, the "two sample scale model" method proposed by Seed and

Anwar (1986), and discussed in Section 2.2.2, appeared to be the most reliable and accurate method by which to make compliance volume evaluations for these smaller samples, and was used for all compliance determinations made for the "smaller" samples in this study. A complete description of the compliance measurements made and the different soils tested is presented in Section 5.3.

5.3 Membrane Compliance Measurements

The magnitude of membrane compliance is virtually always determined by performing drained isotropic compression and rebound tests on saturated samples. Compliance volume evaluations made as a part of this study were performed using such drained test results, from which the magnitude of membrane compliance was determined by subtracting the true soil volumetric strain from the total sample volumetric strain measured.

The membranes used for the majority of tests performed on 12-inch diameter specimens were made of rolled black rubber tire-tread stock obtained from the American Rubber Manufacturing Co. of Emeryville, CA, and were approximately 0.12 inches in thickness. Compliance measurements were made utilizing the large-scale apparatus for a variety of gradations types whose "representative" grain sizes ranged from coarse gravels to coarse sands whose particle diameters approached the thickness of the rubber membrane, at which point measured compliance volumes were expectedly low.

While most of the soils tested for compliance induced volume changes were uniformly graded materials of various sizes, a number of very different gradations (e.g. well-graded and gap-graded) were also investigated in order to further define and generalize the relationships developed. The results of the individual compliance measurement tests and description of each of the soils tested is given in

Section 5.3.2. A compilation of the test results obtained in this study is presented along with those of previous investigators, whose test data was deemed to be of reasonable accuracy, in order to derive an updated correlation for membrane compliance characteristics as a function of material particle sizes.

Most of the samples tested for membrane compliance determinations as part of this study were prepared by "moist tamping" in layers. Exceptions to this rule were "small-scale" samples prepared by dry pluviation to investigate the significance of sample preparation method on membrane compliance measurements.

The testing procedures for determining membrane compliance characteristics were essentially the same for all samples, with some previously mentioned differences in measurement equipment for different sample sizes. Once samples were prepared in their respective sample molds, a vacuum of nearly one full atmosphere was applied to the samples. The samples were then saturated to a high degree with a vacuum/back pressure system (as described earlier in Section 4.3.3) until a B-value of no less than 0.97 was achieved, while maintaining a constant effective confining stress on the sample. After saturation, the samples were subjected to fully drained isotropic loading and unloading while total sample volume changes and sample deformations were recorded. The volume change due to isotropic loading and unloading was measured to an accuracy of $0.001cm^3$ for conventional 2.8-inch diameter and 1.4-inch diameter specimens, and $7.3cm^3$ for large-scale (12-inch diameter) samples. For both of these cases, the accuracy of measurements was greater than 0.05% with respect to each of the respective sample volumes.

It has been demonstrated by numerous investigators (e.g. El-Sohby, 1964; Frydman et al., 1973; Keikbusch and Schuppener, 1977; Raju and Venkataramana, 1980; Baldi and Nova, 1984; Seed and Anwar, 1986; etc.) that volumetric membrane compliance magnitude is a direct and repeatable function of sample effective confining stress. It has also been demonstrated that the relationship between compliance-induced volume change per unit area of membrane and $\log_{10}$ of the sample effective confining stress is essentially linear over the range of interest for most undrained testing. The slope of the semi-log function, referred to as the "normalized unit membrane penetration" (S), can therefore be used to characterize the volumetric compliance for a given soil. All of the studies of factors affecting unit membrane compliance indicate that such a pre-determination of volumetric membrane compliance represents a viable basis for control of injection-mitigation during undrained testing.

5.3.1 Evaluation of Factors Affecting Membrane Compliance

A number of investigators have tried to identify the factors that may affect the volumetric membrane compliance of different soils. During the preliminary phase of this investigation a number of possibly significant soil variables were examined in order to evaluate their effect on compliance value measurements. The factors investigated included: (a) soil grain size and gradation, (b) soil density, (c) soil grain angularity, (d) soil fabric or method of sample preparation, and (e) membrane thickness and the use of multiple membranes. An additional important finding was that samples cyclically loaded to a state of liquefaction ($r_u = 100\%$) and then re-consolidated, exhibited membrane compliance behavior essentially identical to that measured on similarly prepared samples not loaded prior to membrane compliance measurements.

The series of tests performed for purposes of evaluating factors that may affect membrane compliance, some of which were previously reported by Anwar et al. (1989), are briefly discussed here in order to identify the importance of each.

5.3.1.1 Soil Grain Size and Gradation

The most important factor affecting the magnitude of measured volumetric compliance was found to be soil gradation characteristics. This finding was in agreement with earlier investigators. In relation to the gradation characteristics of a soil (material grain size and grain size distribution), all other factors were found to be secondary. An important point recognized in reviewing the literature of previous investigations, was that most previous investigators had consistently correlated membrane compliance magnitudes with mean particle size (D_{50}). It was also noticed that virtually all materials examined by previous investigators had been uniformly graded. As discussed briefly in Chapter 2 (Section 2.2.4) it was found that for non-uniformly graded sandy soils, unit membrane compliance was much better correlated with smaller particle sizes (D_{20}) than with mean grain size. These findings were further confirmed by the tests performed in this study on a wide range of gravelly soils. A detailed investigation which more clearly defines the effects and the relationship of soil gradation characteristics on normalized membrane compliance is presented in Section 5.3.2.

5.3.1.2 Soil Density

The influence of sample density on measured compliance values has been investigated by a number of earlier researchers, and has been shown to be of fairly consistent but relatively minor significance. Figure 5.1 shows a plot of unit membrane compliance versus effective confining stress for five samples of Monterey 16 sand tested at relative densities of 45%, 50%, 55% and 60%. As this Figure shows, there is a slight variation in measured compliance for the different densities tested, and it is suggested that the generalized relationship given later in Figure 5.49 is most appropriate for samples of medium density. For samples at

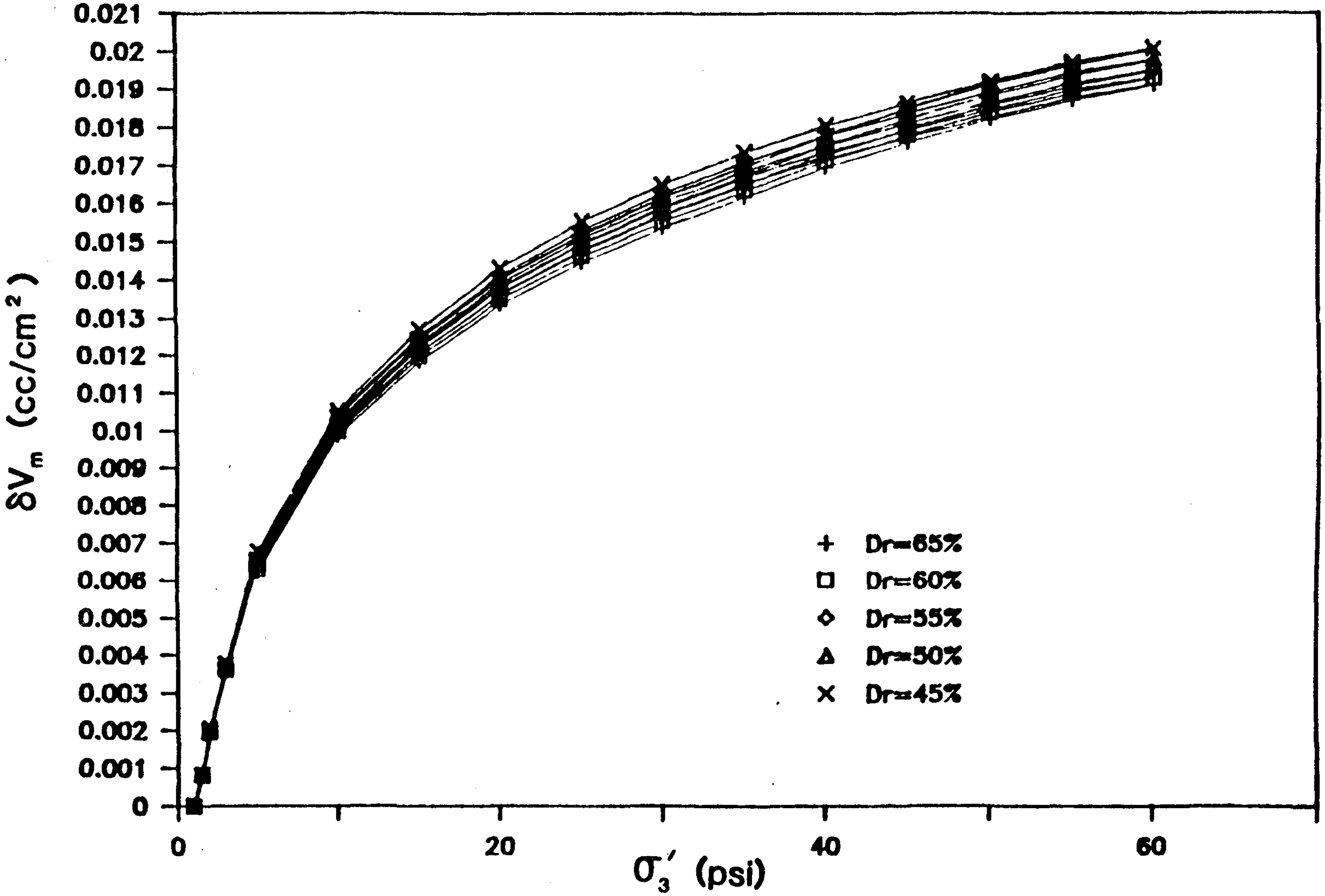


Figure 5.1: Unit Membrane Compliance vs. Effective Confining Stress for Monterey 16 Sand Over a Range of Relative Densities

significantly higher or lower densities, adjustments should be made to compliance estimates, or compliance measurements should be made for those samples. It has been suggested that the maximum adjustment to S values that would have to be made for extreme cases of high or low density samples should be no more than 10% (Anwar et al., 1989).

5.3.1.3 Soil Angularity

The potential influence of different particle shapes or angularity was examined as part of the first phase of this research program reported by Anwar et al. (1989). Monterey 16 sand was separated into its angular and sub-rounded components, and was then checked to assure that the gradations of the two components were similar. It was found that the influence of very different particle shapes, or angularity, had no significant effect on the measured compliance values for samples of essentially the same gradation and prepared at the same relative density, as shown in Figure 5.2.

5.3.1.4 Soil Fabric

The "fabric" of a soil sample is initially a function of sample preparation method. Later on in an undrained test, soil fabric may be significantly altered by rearrangement of particle grains. It is therefore important that any significant influence of differnt soil fabrics be identified. Samples of Monterey sand were prepared at the same relative density by various methods including moist tamping, dry tamping, dry pluviation and vibration. Fortunately, no significant effect on measured compliance values was found due to the different sample preparation methods. Results of the unit membrane compliance evaluations made on the samples prepared by these different methods are given in Figure 5.3. Furthermore, it was found that particle rearrangement during cyclic loading appeared to have no

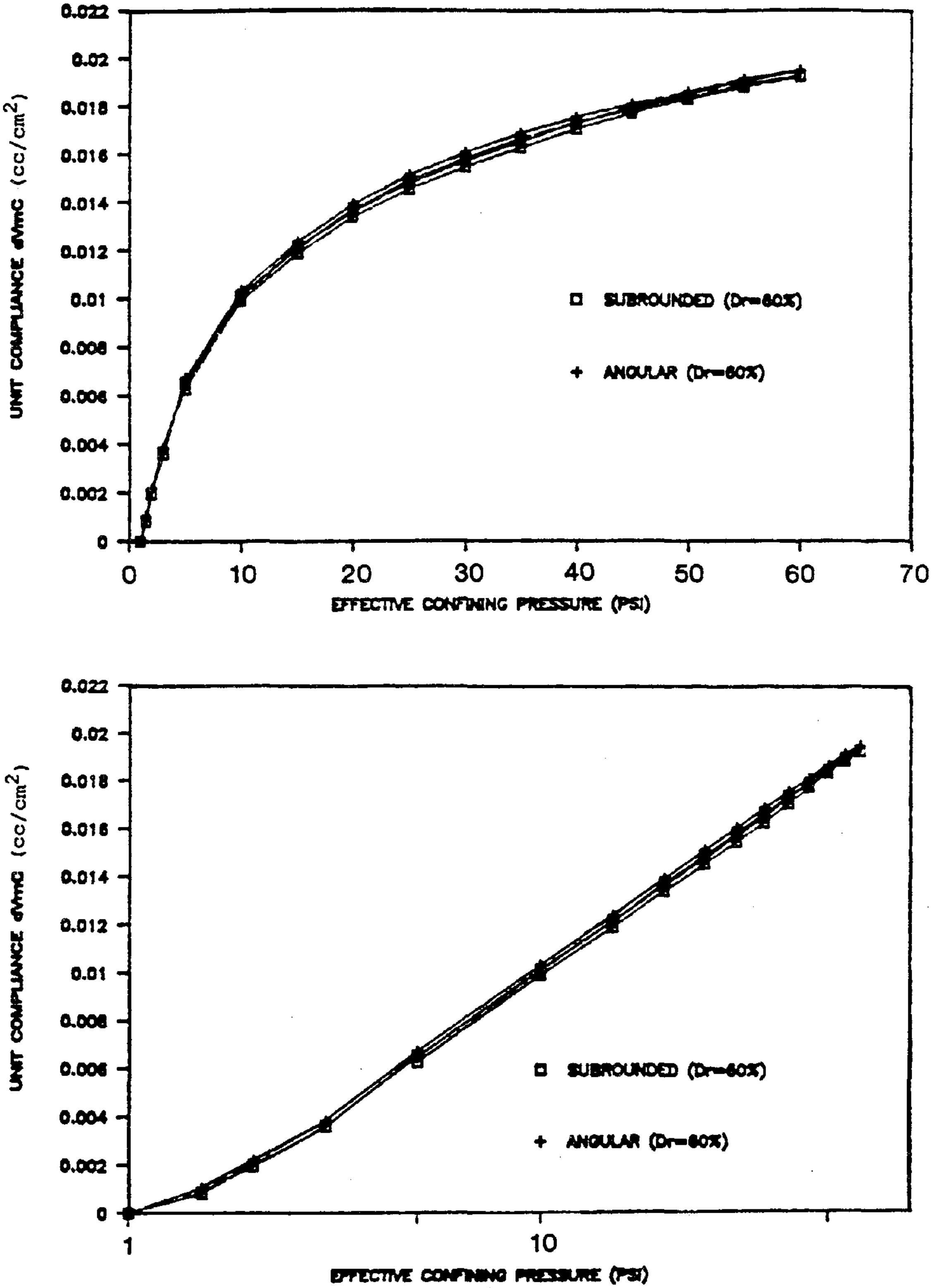


Figure 5.2: The Influence of Particle Angularity on Membrane Compliance

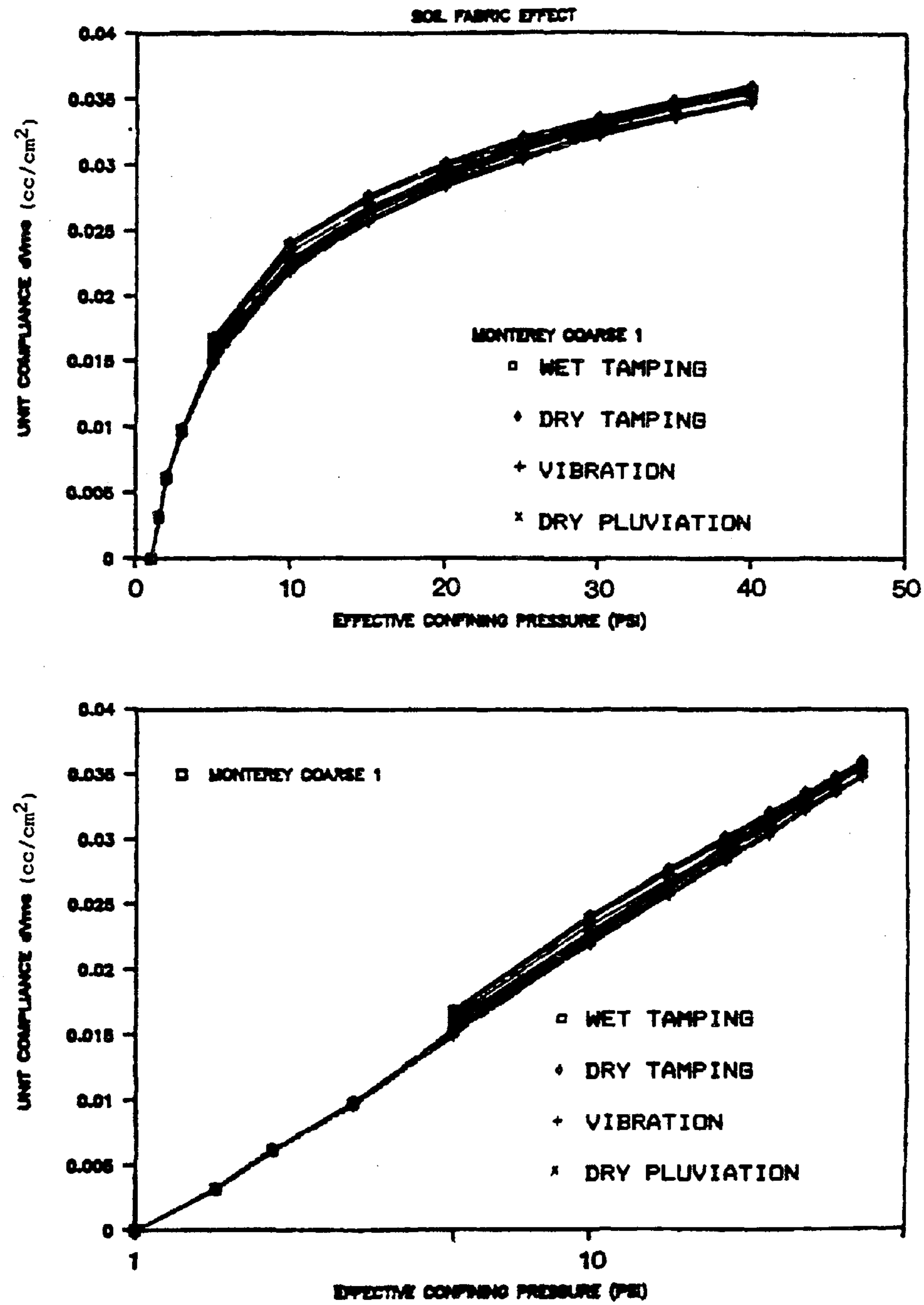


Figure 5.3: The Influence of Initial Soil Fabric, or Method of Sample Preparation, on Membrane Compliance (Monterey Coarse 1 Sand at D_R = 60%)

effect on membrane compliance. This finding was illustrated by the following procedure. A number of samples of Ottawa sand prepared at relative densities of approximately D_R = 50% were initially tested for unit membrane compliance. Identically prepared samples were then cyclically loaded to conditions of full liquefaction (r_u = 100%). These samples were then reconsolidated and tested for compliance evaluations. As can be seen in Figure 5.4, the measured compliance values were found to be nearly identical for the samples evaluated before and after cyclic loading to liquefaction.

5.3.1.5 Membrane Thickness and Multiple Membranes

The significance or influence of using multiple membranes and different membrane thicknesses on membrane compliance measurements has been investigated by several researchers. The results of those investigations have led to conclusions for particular sample sizes and membranes, but no clear picture has yet been developed and conclusively verified regarding how to join or separate these different data sets. An attempt to bridge the gap between the difference in conclusions made about the significance of using multiple membranes and various membrane thicknesses and materials for different sized samples was made as a part of this study. A disscussion of the results of that investigation is presented here. In addition, recent work on this subject by Kramer et al. (1989) appears very promising.

Experimental studies performed on small-scale (less than 6-inch diameter) samples have shown that using different thicknesses and numbers of latex membranes had little or no influence on unit membrane compliance (Ramana and Raju, 1982; Martin et al., 1978). Studies by Seed and Anwar (1986) verified this conclusion, and Kramer et al. (1989) proposed empirically-derived relationships

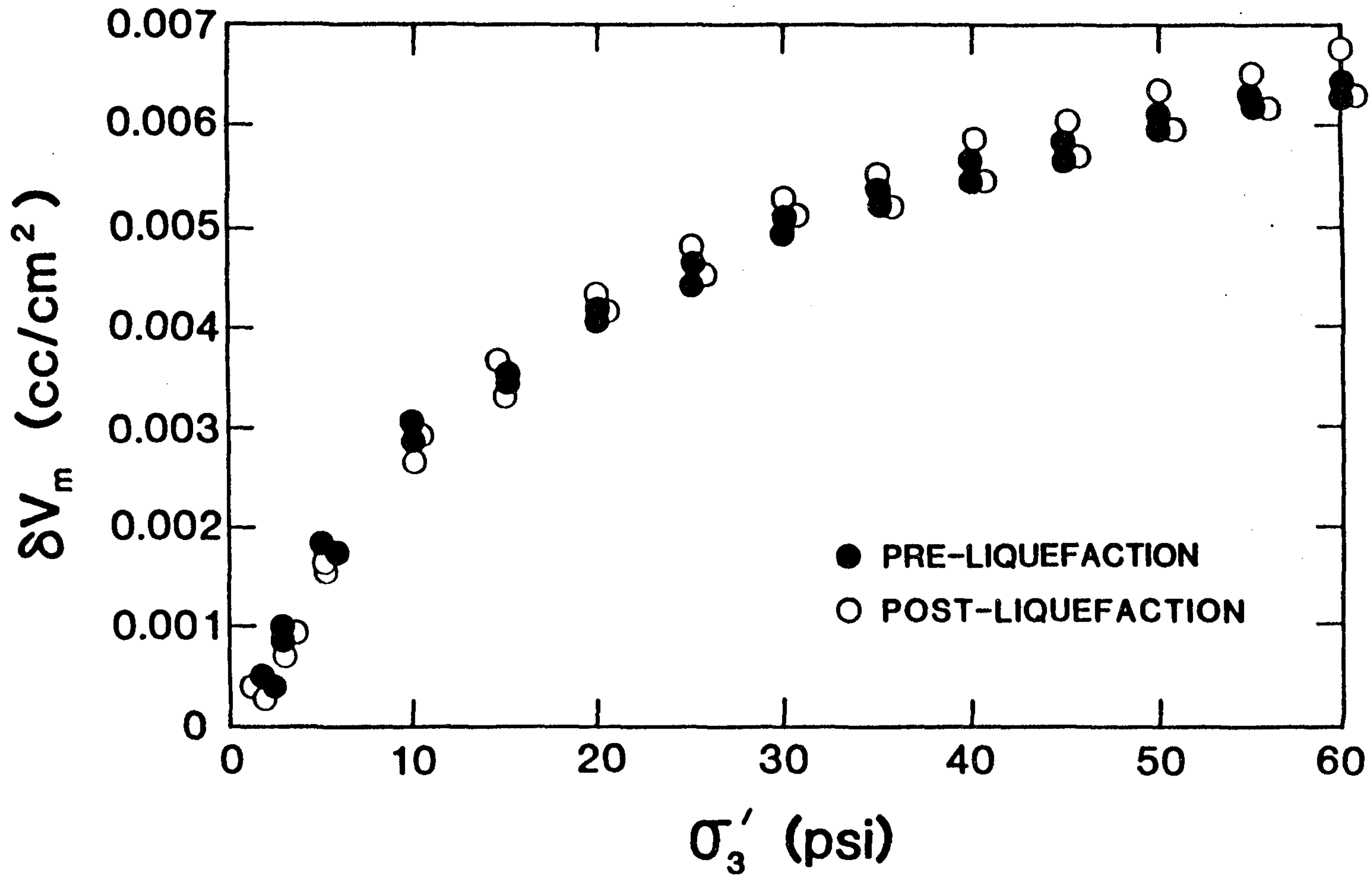


Figure 5.4: Pre- and Post-Liquefaction Membrane Compliance Curves for Fine Ottawa Sand at D_R = 50%

for estimation of the relatively slight influence of membrane thickness and stiffness.

Molenkamp and Luger (1981) and Baldi and Nova (1984) theoretically demonstrated the unimportance of membrane thickness and stiffness. Baldi and Nova proposed that even a five-fold increase in membrane thickness would have no significant effect on membrane compliance.

An overlap of sample sizes and membrane thicknesses were tested in this study for a number of materials whose representative grain size (D_{20}) was between coarse sand and fine gravel. The results suggest that at some limiting minimum grain size (on the order of twice the thickness of the confining membrane) membrane thickness begins to become an important factor in compliance volume measurements.

It is interesting to note that for those soils whose representative particle sizes do not approach the thickness of the larger membrane, unit compliance measurements made using both small and large scale systems compared extremely well, suggesting the insignificance of such factors as sample size and different membrane types. Figure 5.5 shows a comparison of measured unit membrane compliance for a fine gravelly material as measured with the small-scale apparatus with a 2.8-inch diameter sample confined by a single 0.014-inch thick latex membrane, and the large-scale apparatus with a 12-inch diameter sample confined by a 0.12-inch thick rubber membrane. For this material, the typical "representative" (D_{20}) grain size diameter that controls compliane is significantly greater than twice the thickness of the large-scale testing membrane, and membrane thickness therefore does not greatly influence compliance values. For conventional small scale (2.8-inch diameter) samples, the use of different numbers of multiple membranes and varied membranes thicknesses did not appear to

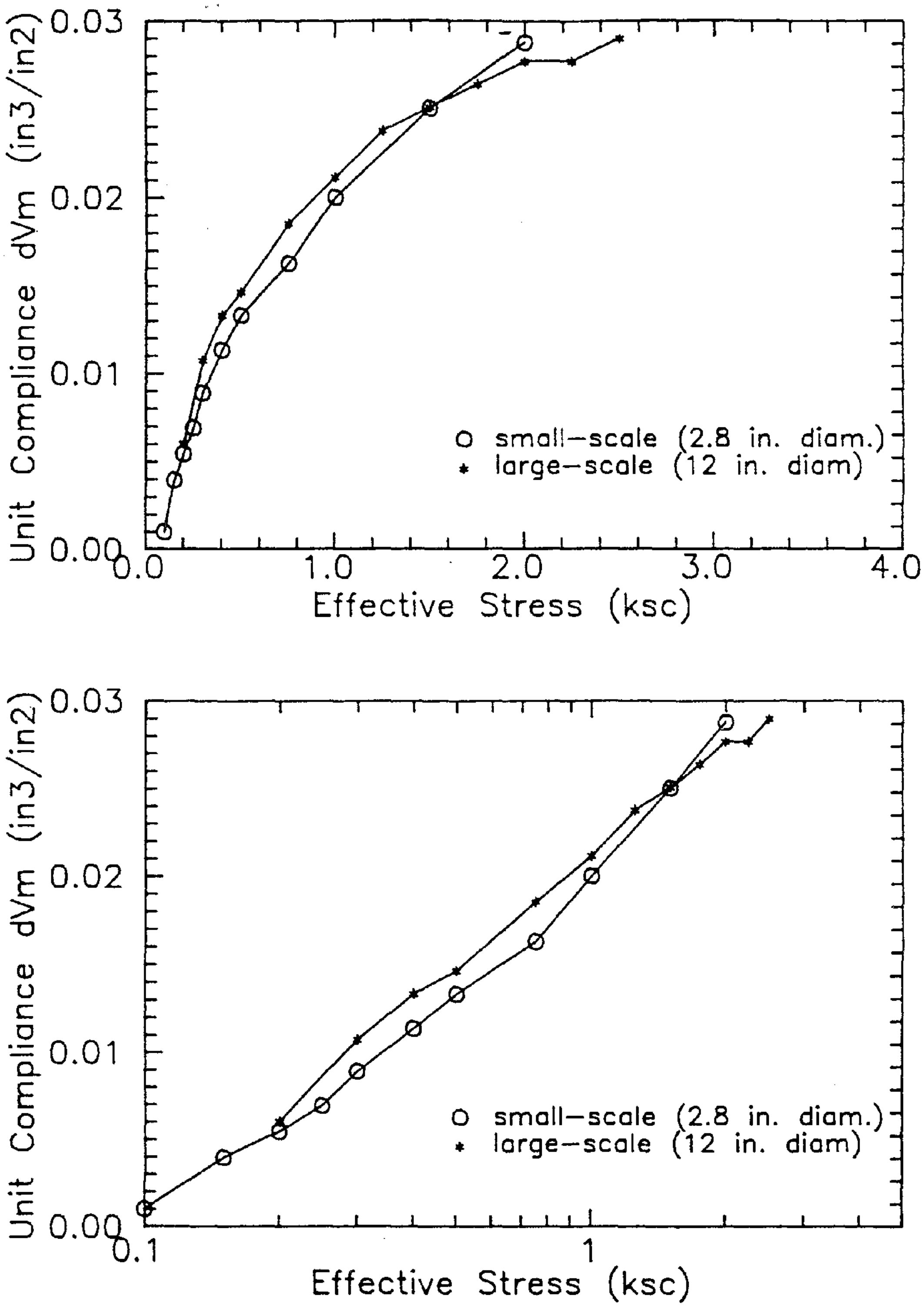
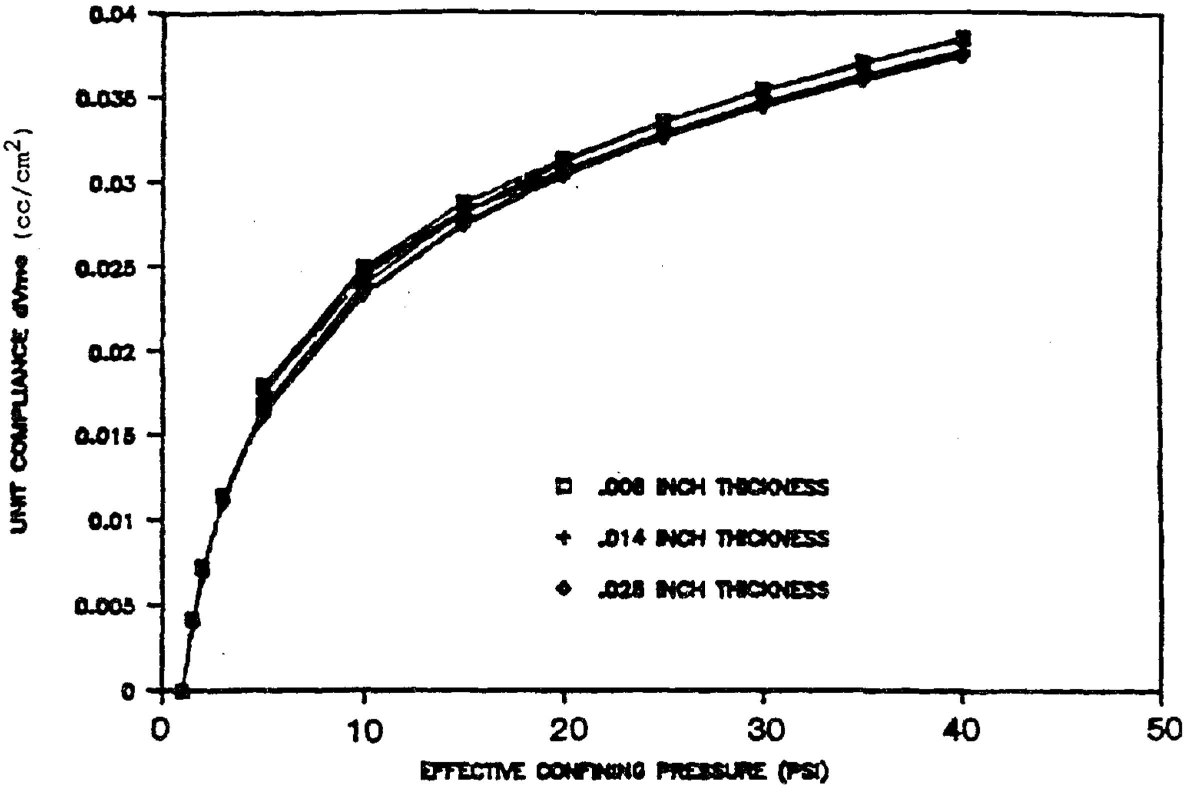


Figure 5.5: The Influence of Different Sized Testing Equipment on Membrane Compliance Measurements (Fine Gravel, Material #9)

significantly affect volumetric compliance measurements (Seed and Anwar, 1986). Figure 5.6 shows the results of tests on 2.8-inch diameter samples using different thicknesses (and numbers) of latex membranes. For the test performed using 0.028 inch thickness of membrane (two membranes, 0.014 inch thick each), the nearly quadrupling of membrane thickness over the 0.008 inch thick membrane decreased the measured compliance values by only approximately 5% for fine sands and by less than 5% for coarse sands. As grain sizes approached the range of membrane thicknesses for these small-scale samples, compliance volumes became so small that the accuracy of the measurements fell in the same range as that of the actual volume changes. Because of this it was concluded that the effects of different membrane configurations on these samples was insignificant with regard to compliance measurements.

Evans and Seed (1987) reported that differences in cyclic loading resistance between 12-inch diameter specimens confined by two thin latex membranes and those confined by a considerably thicker and stiffer rubber membrane were small, indicating that membrane compliance effects were not significantly influenced by such variations in the membrane configurations used (e.g. membrane material properties and thicknesses). An additional investigation made by Evans and Seed on a large-scale (12-inch diameter) sample, was made to evaluate the effect of using multiple membranes on compliance volume measurements. They reported that measured compliance volumes were nearly twice those found when using one membrane instead of two, but that using greater numbers of membranes had little additional effect. It was hypothesized that adhesion between the membranes was responsible for the decrease in measured compliance volumes, and not the added thicknesses. This suggestion was partially supported by results from this study where samples of coarse materials were tested with single membranes of different



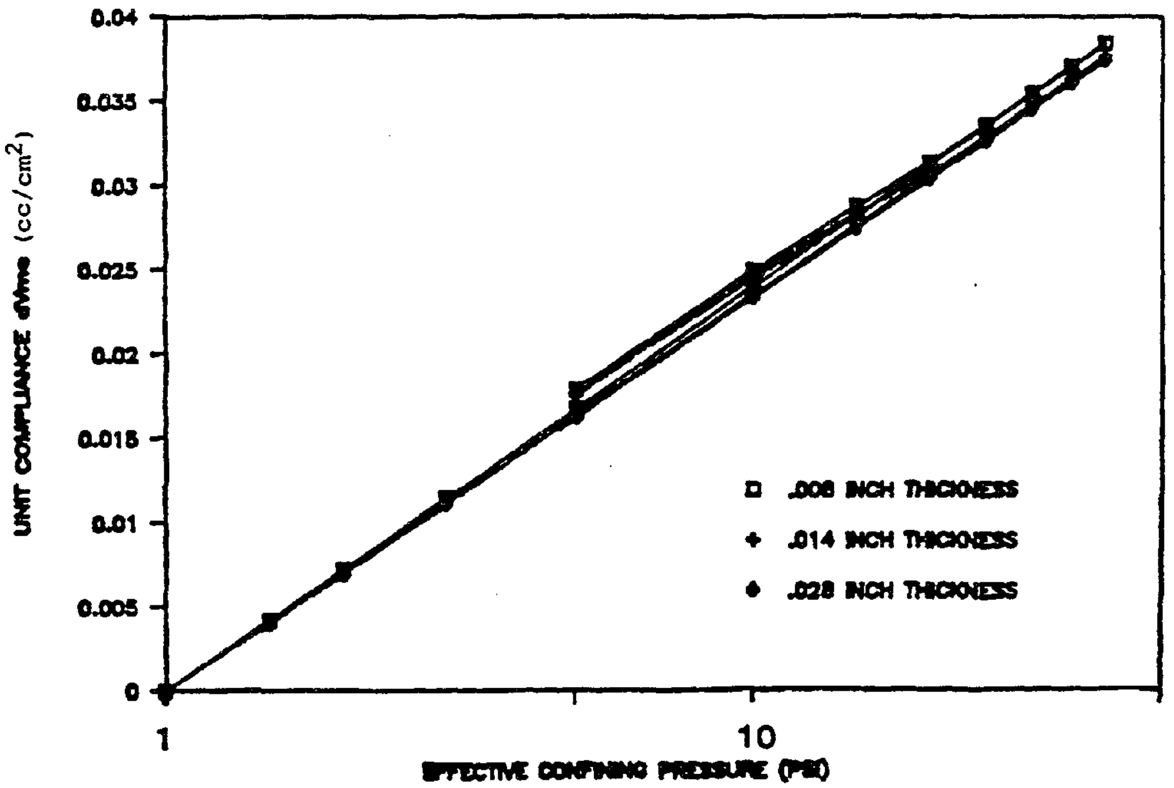


Figure 5.6: The Influence of Membrane Thickness on Membrane Compliance (Monterey Coarse 2 Sand at $D_R \approx 60\%$)

thicknesses and multiple membranes. The use of different thicknesses of individual membranes had very little effect on compliance volume measurements. Using different numbers of membranes appeared to have a noticeable effect, although the effect of using multiple membranes was found to be much less significant when tested in this study. Figure 5.7 shows the difference between compliance values measured as part of this study for similarly prepared samples of a medium gravel with membrane thicknesses of 0.12 and 0.24 inches. Figure 5.8 shows a comparison between compliance values measured for similarly prepared samples of a medium gravel with one and two membranes.

5.3.2 Compliance Measurement Results

A number of samples of sand with a variety of soil characteristics, including different gradation types and soil particle sizes, angularity, etc., were tested for volumetric compliance as part of the preliminary phase of this study previously reported by Anwar et al. (1989). For completeness, these results are also presented here. A listing of the sandy soils tested, with a brief summary description and membrane compliance behavior evaluated for each, is given in Table 5.1. Figures 5.9 through 5.21 show the measured membrane compliance characteristics of these soils, and Figures 5.22 through 5.25 show the gradations of these materials. All compliance measurements are plotted as unit membrane compliance (volume change per unit membrane area: cc/cm^2) vs. effective confining stress and $\log_{10}$ of effective confining stress. All of the soils listed in Table 5.1 were prepared to $D_R \approx 60\%$.

In addition to the sandy soils, nearly a dozen different gravelly soils were tested for volumetric compliance utilizing the large-scale testing equipment. Again, a wide range of soil gradations and particle types were tested, and the range of

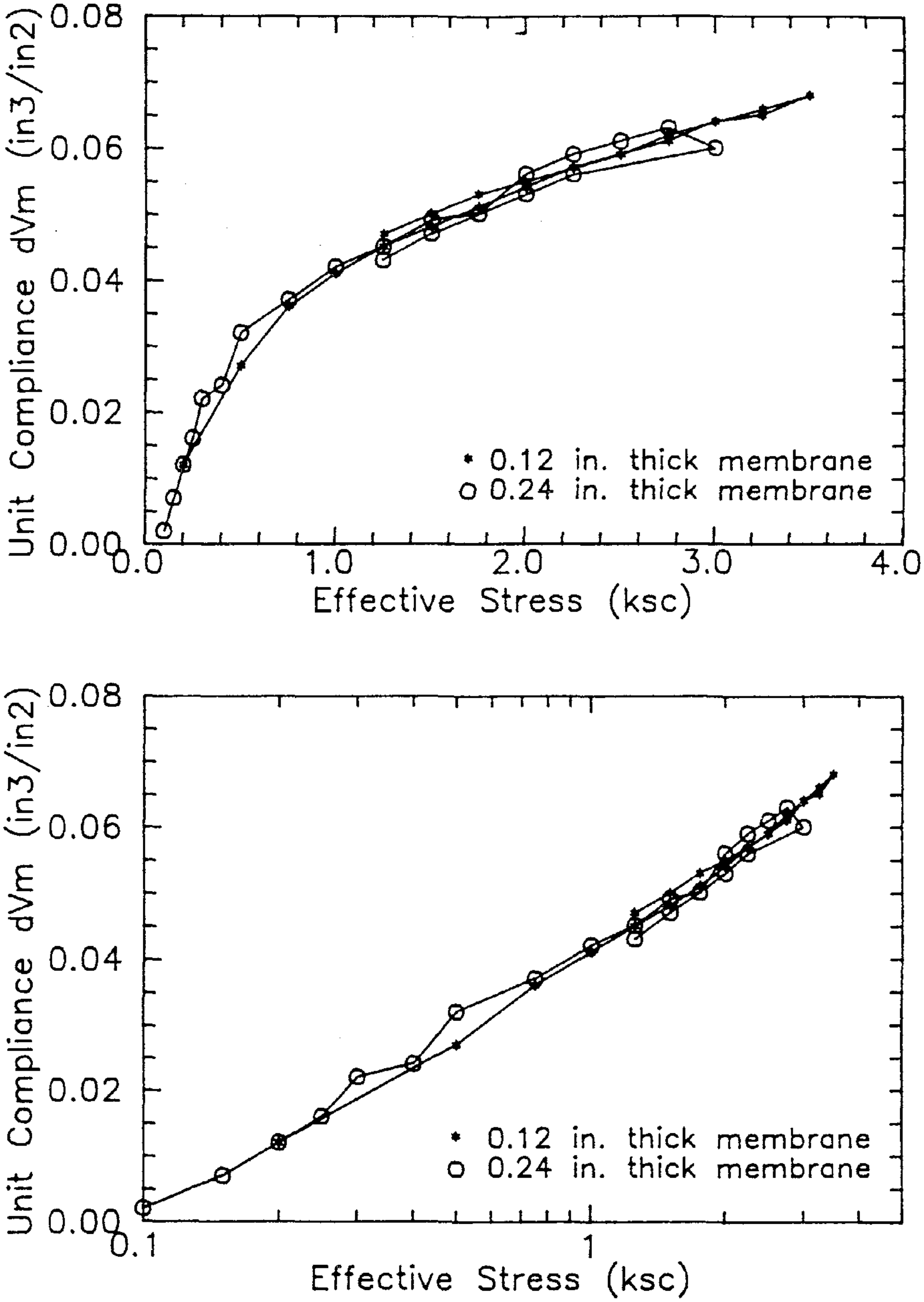


Figure 5.7: The Influence of Different Large-Scale Membrane Thicknesses on Membrane Compliance Measurements (Medium Gravel, Material #1)

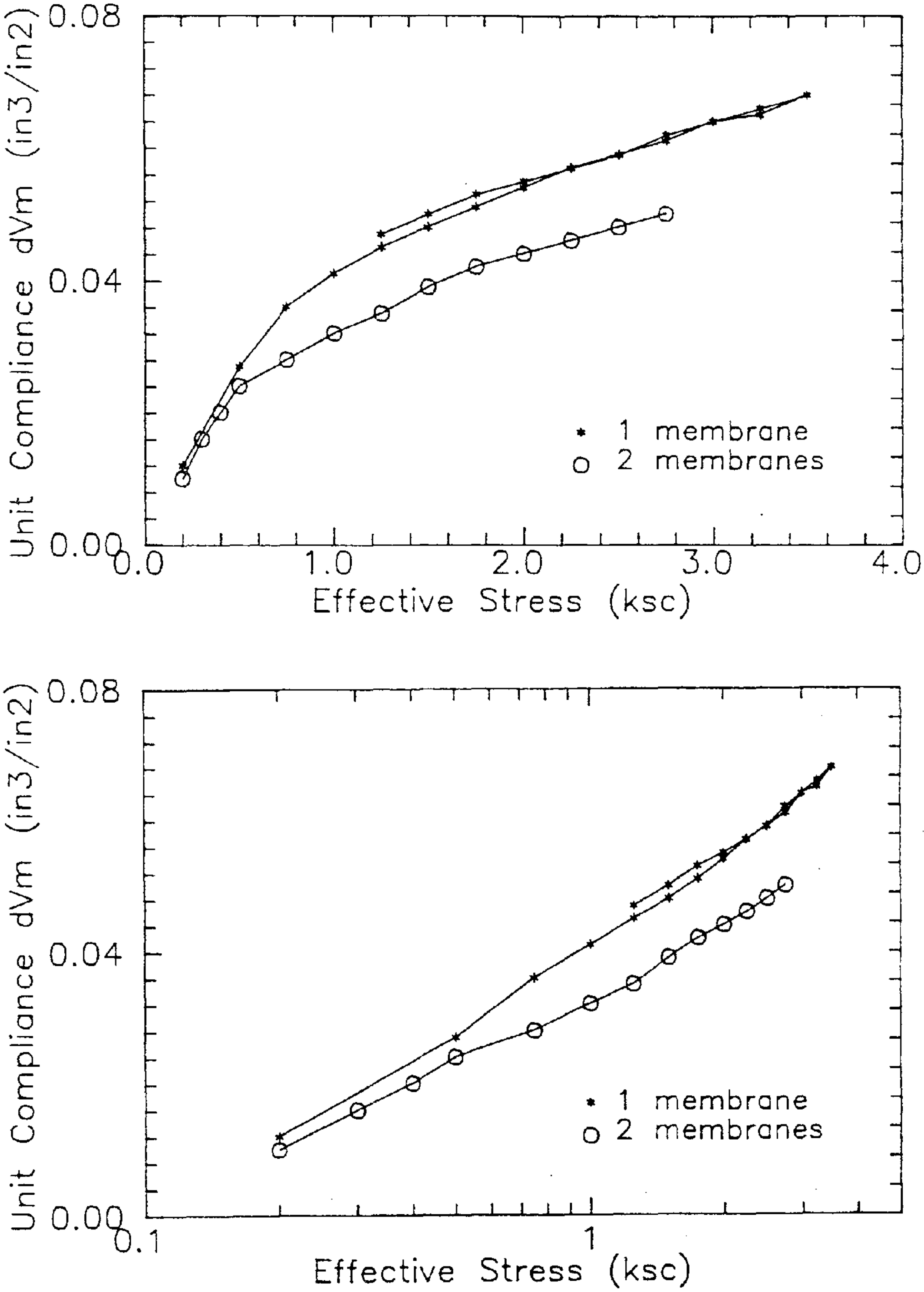


Figure 5.8: The Influence of Different Numbers of Large-Scale Membranes on Membrane Compliance Measurements (Medium Gravel, Material #1)

Table 5.1 Sandy Soils Tested for Membrane Compliance Magnitude

Soil Name	USCS Classification	D_{10} (mm)	D_{20} (mm)	D_{50} (mm)	$S^{\dagger}$ (cm/$\Delta\log\sigma_3'$)	Figure
Monterey Coarse 1	SP	1.900	2.200	2.600	0.02200	3.1
Monterey Coarse 2	SP	2.290	2.490	2.900	0.02350	3.7
Well Graded 1	SW-ML	0.032	0.059	0.200	0.00075	3.8
Well Graded 2	SW	0.130	0.200	0.600	0.00180	3.9
Well Graded 3	SW	0.230	0.320	0.700	0.00222	3.10
Well Graded 4	SW	0.490	0.700	1.400	0.01060	3.11
Mod. Sacramento	SP	0.205	0.230	0.305	0.00201	3.12
Ottawa Fine	SP	0.240	0.300	0.400	0.00480	3.13
Ottawa 20-30	SP	0.605	0.630	0.720	0.00927	3.14
Monterey '0'	SP	0.240	0.300	0.330	0.00500	3.15
Monterey 16	SP	0.720	0.980	1.250	0.01195	3.16
Gap Graded 1	SP	0.240	0.305	1.800	0.00085	3.17
Gap Graded 2	SP	0.630	0.680	0.820	0.00710	3.18

Gap Graded 1: MONTEREY CR. : MONTEREY 16 : MOD SACRAMENTO (4:1:2)
Gap Graded 2: MONTEREY COARSE : OTTAWA 20-30 (1:1)
Well Graded 1: MONTEREY SAND WITH 25% SILT
Well Graded 2: MONTEREY SAND, SOME GRAVEL, 5% SILT

† = Normalized Unit Compliance; change in volume per unit membrane area per log-cycle change in effective conf. stress (cc/cm^2/$\Delta\log\sigma_3'$).

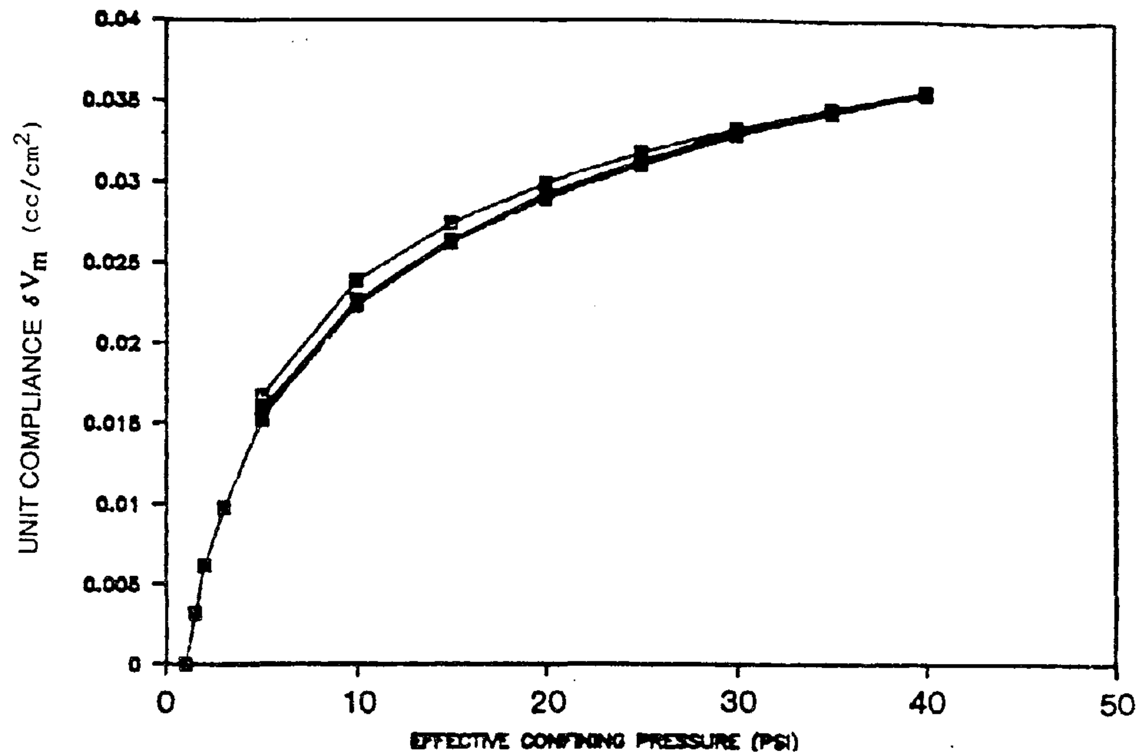


Figure 5.9a: Unit Membrane Compliance vs. Effective Confining Pressure: Modified Monterey Coarse 1 Sand at D_R = 60%

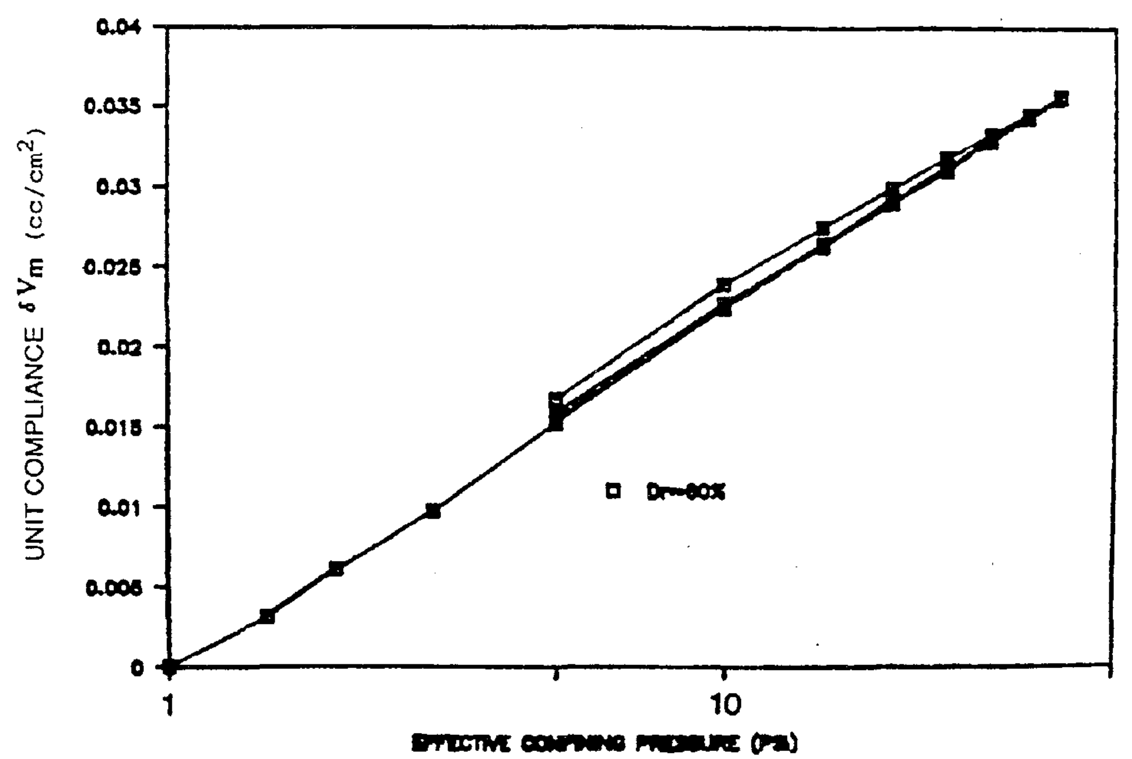


Figure 5.9b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Modified Monterey Coarse 1 Sand at D_R = 60%

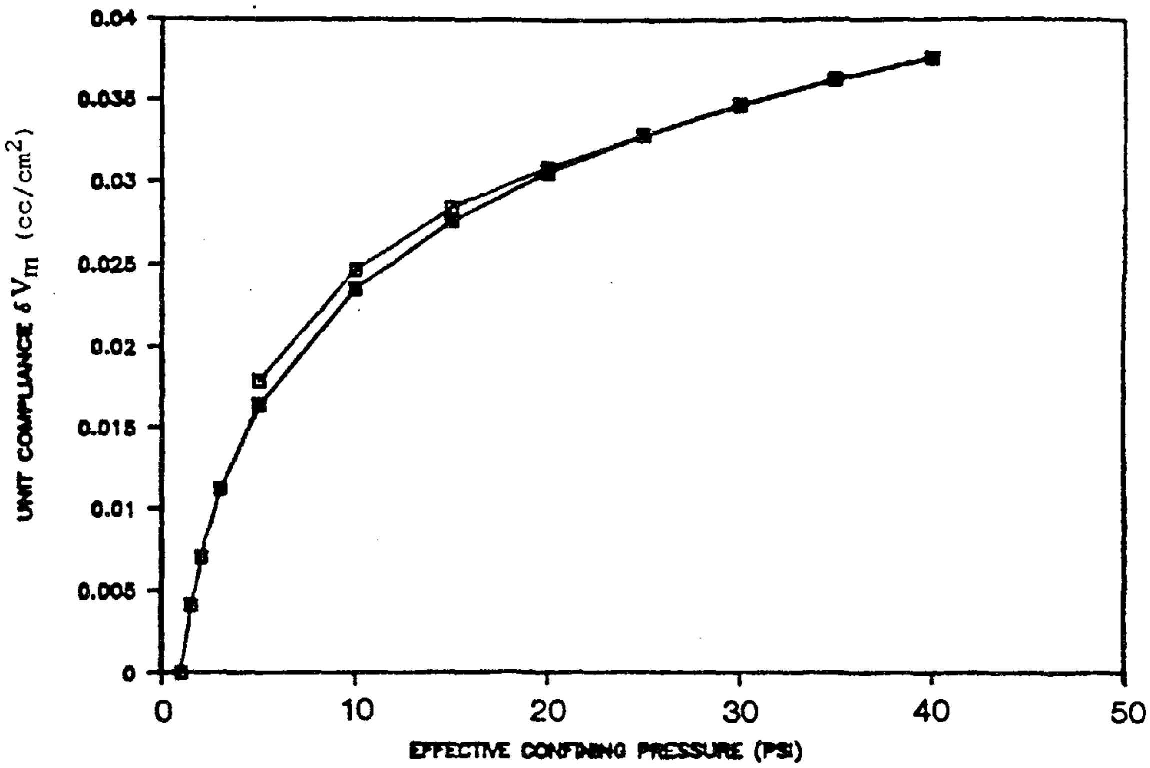


Figure 5.10a: Unit Membrane Compliance vs. Effective Confining Pressure: Modified Monterey Coarse 2

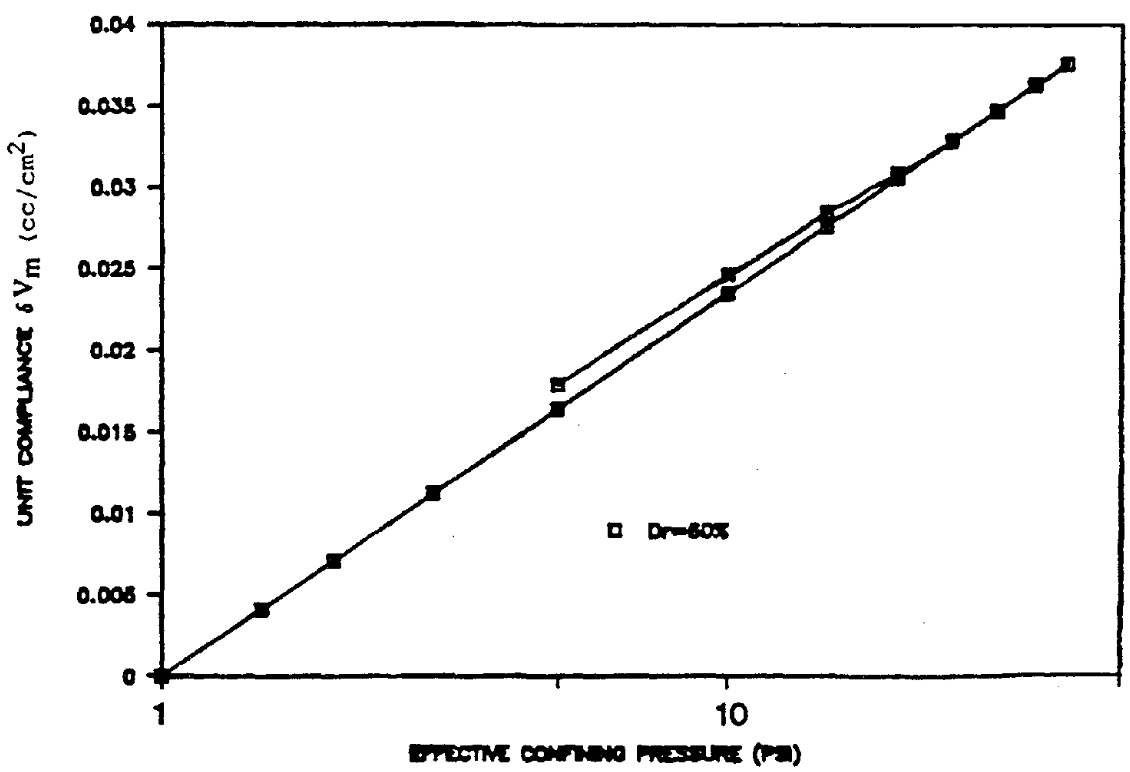


Figure 5.10b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Modified Monterey Coarse 2

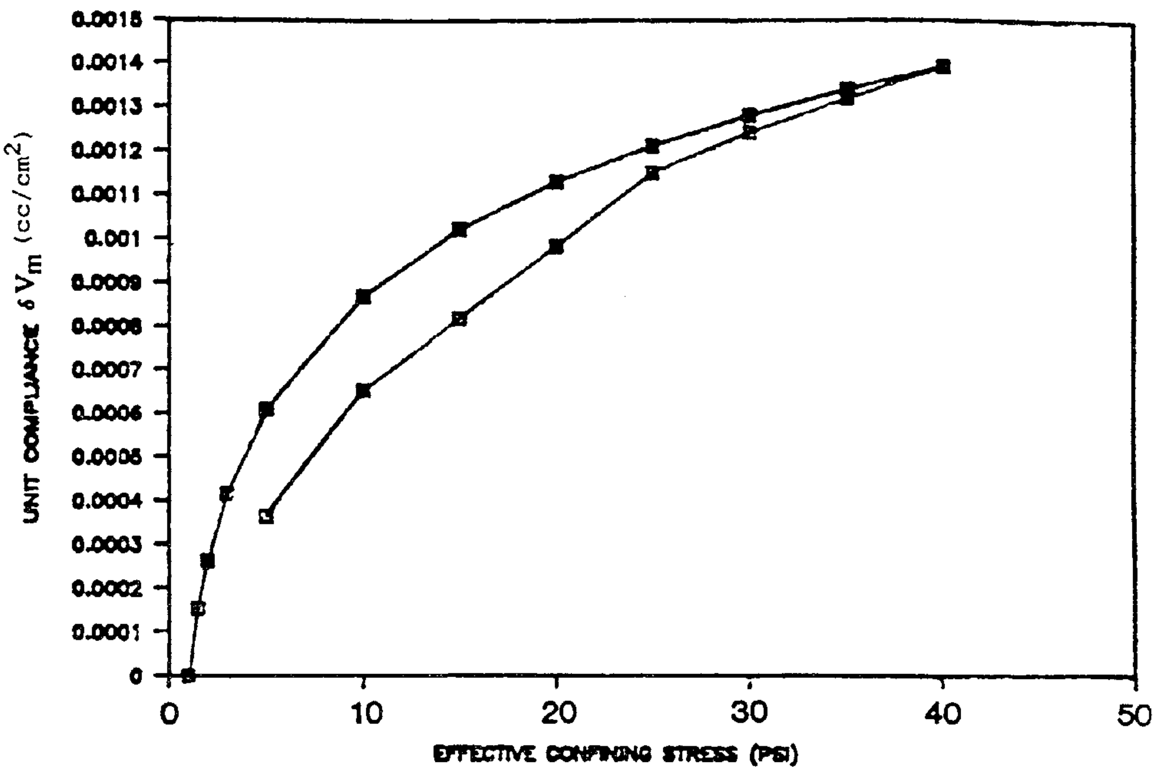


Figure 5.11a: Unit Membrane Compliance vs. Effective Confining Pressure: Well Graded 1

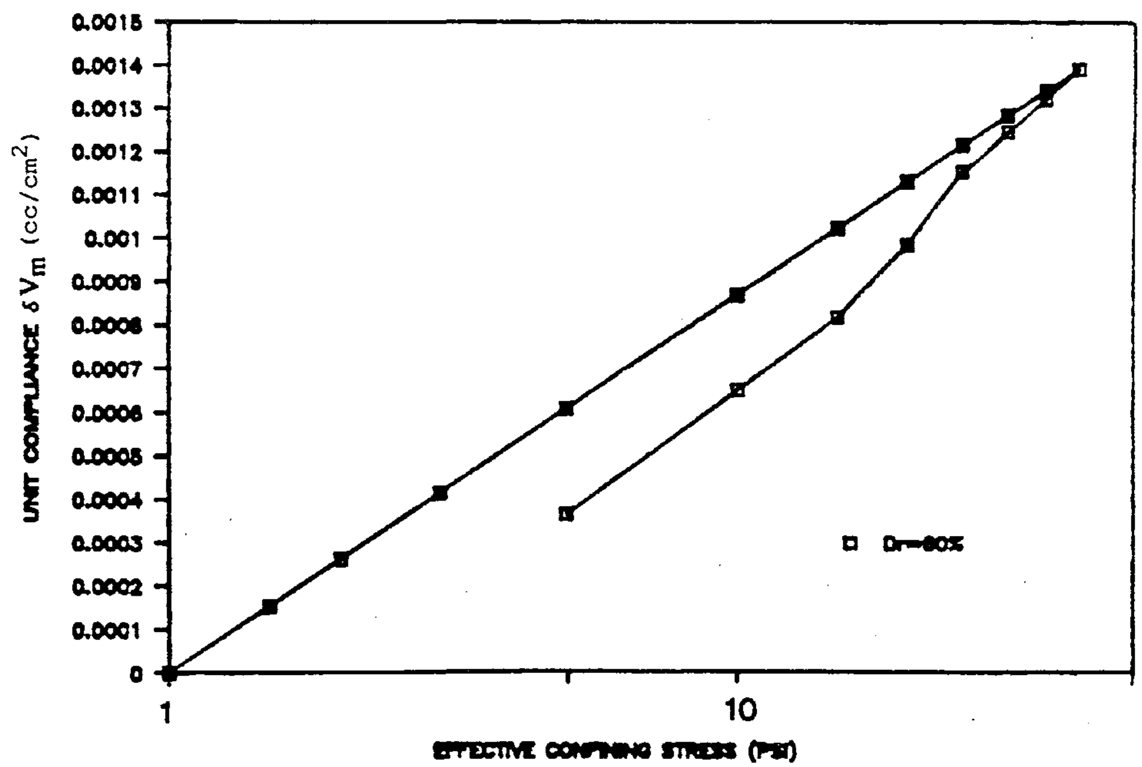


Figure 5.11b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Well Graded 1

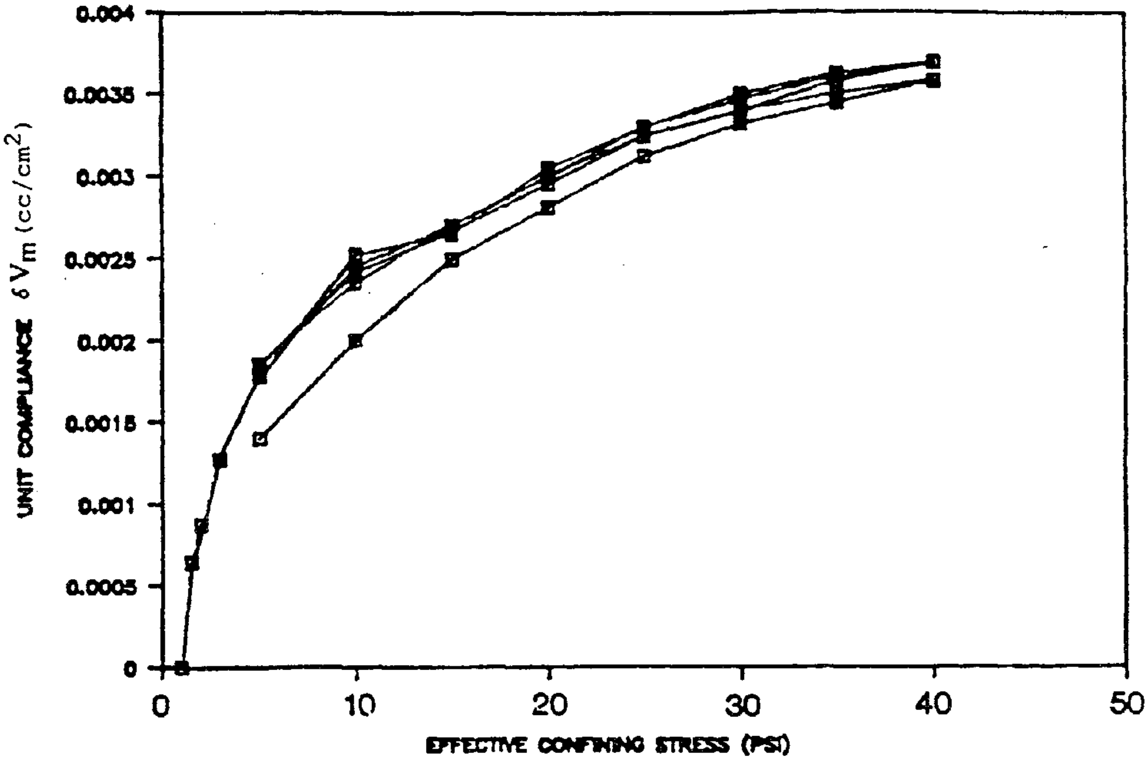


Figure 5.12a: Unit Membrane Compliance vs. Effective Confining Pressure: Well Graded 2

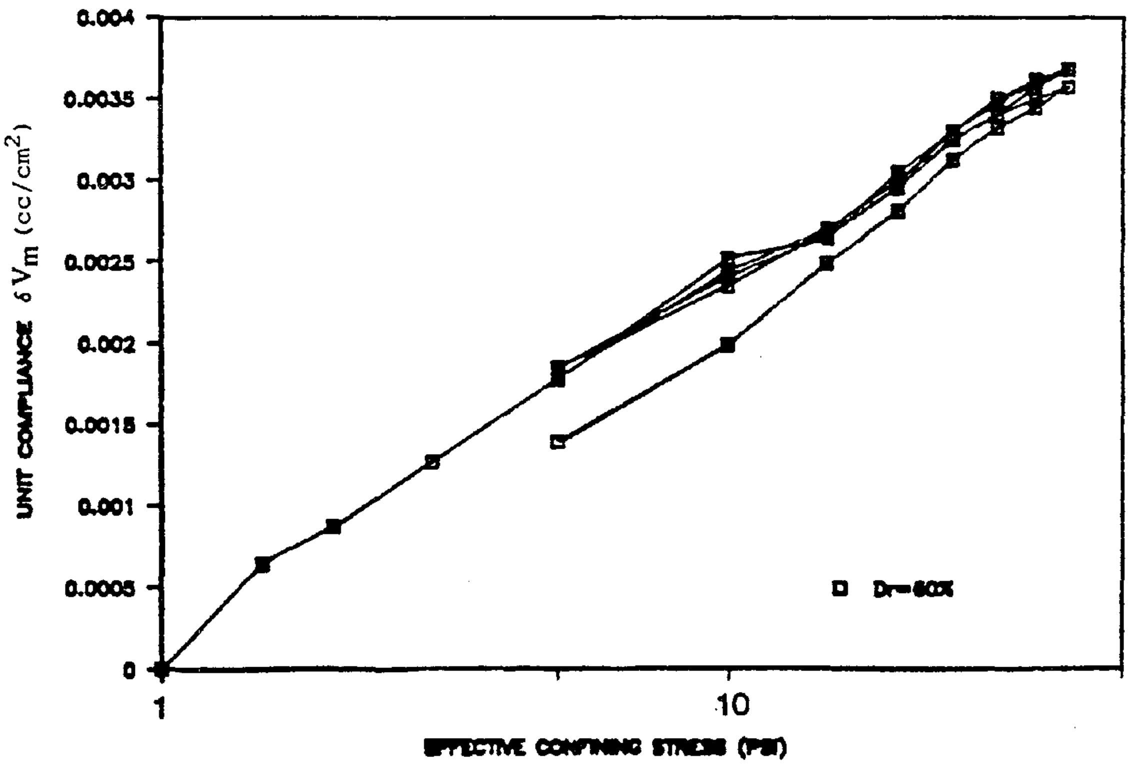


Figure 5.12b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Well Graded 2

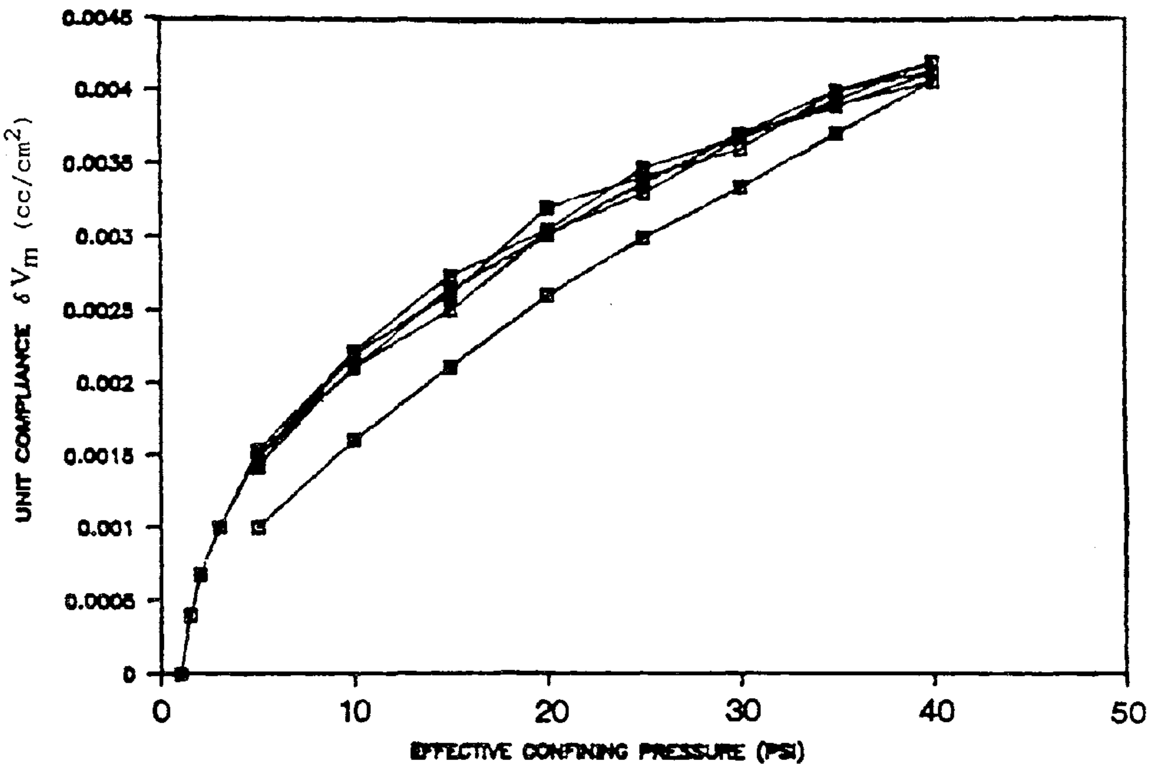


Figure 5.13a: Unit Membrane Compliance vs. Effective Confining Pressure: Well Graded 3

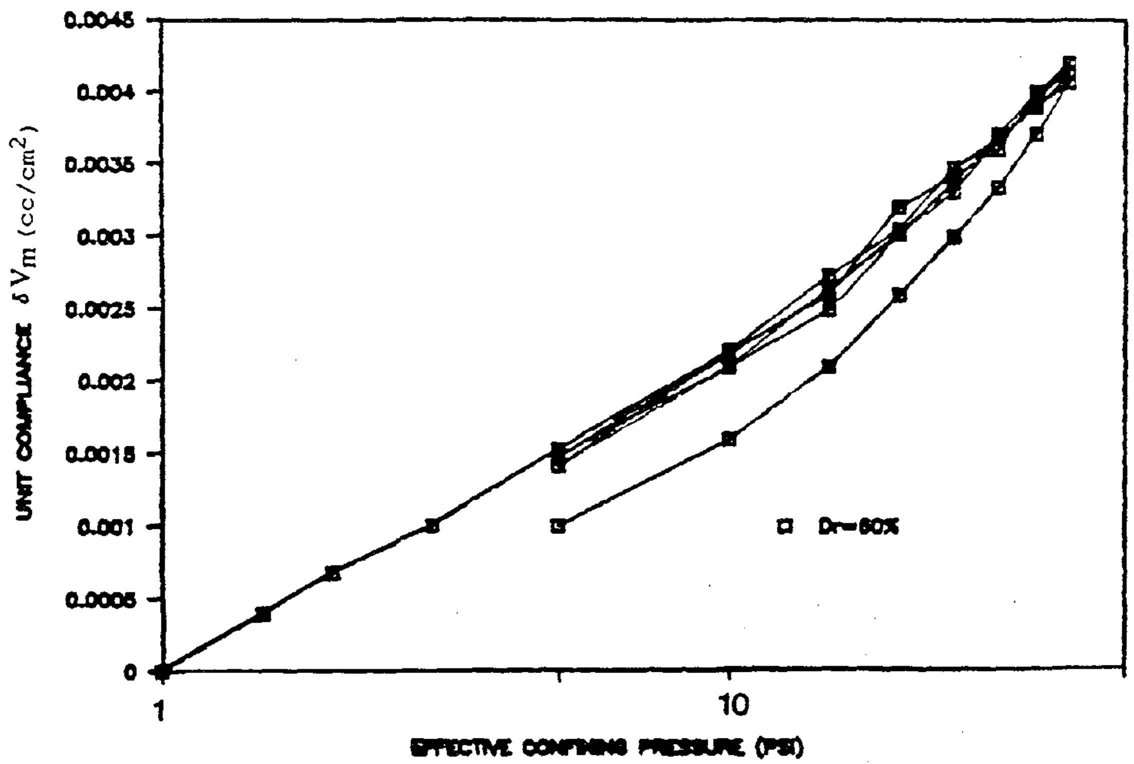


Figure 5.13b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Well Graded 3

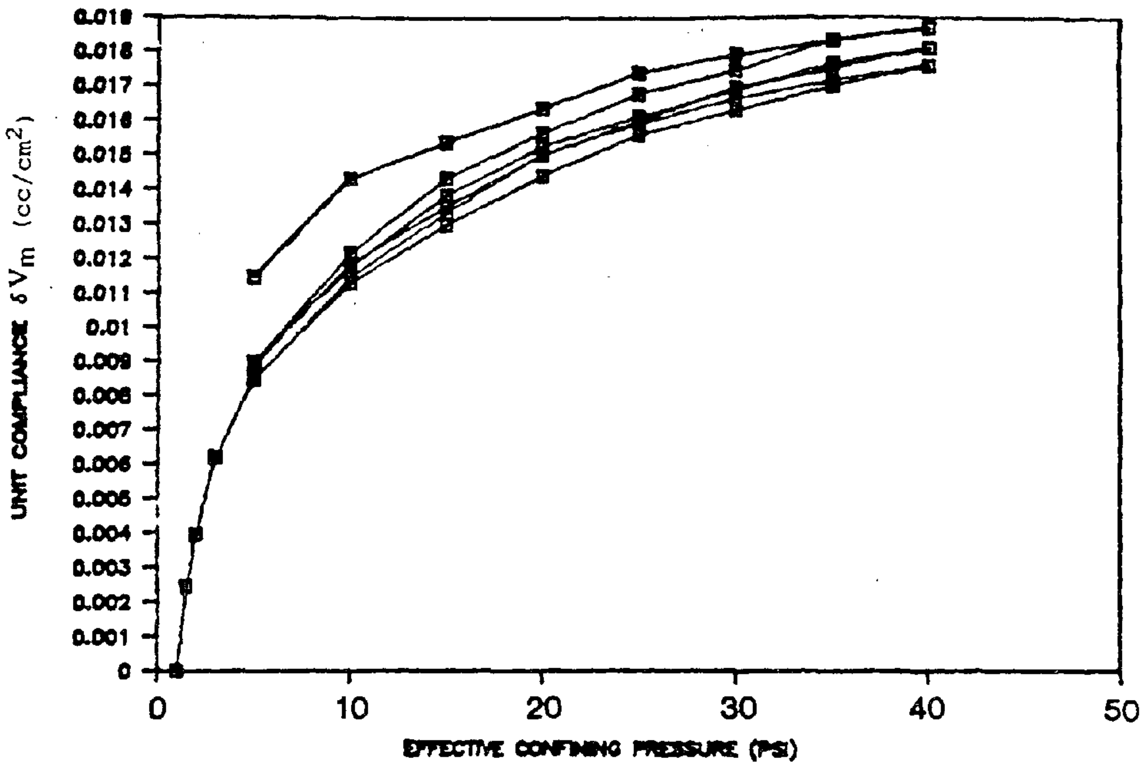


Figure 5.14a: Unit Membrane Compliance vs. Effective Confining Pressure: Well Graded 4

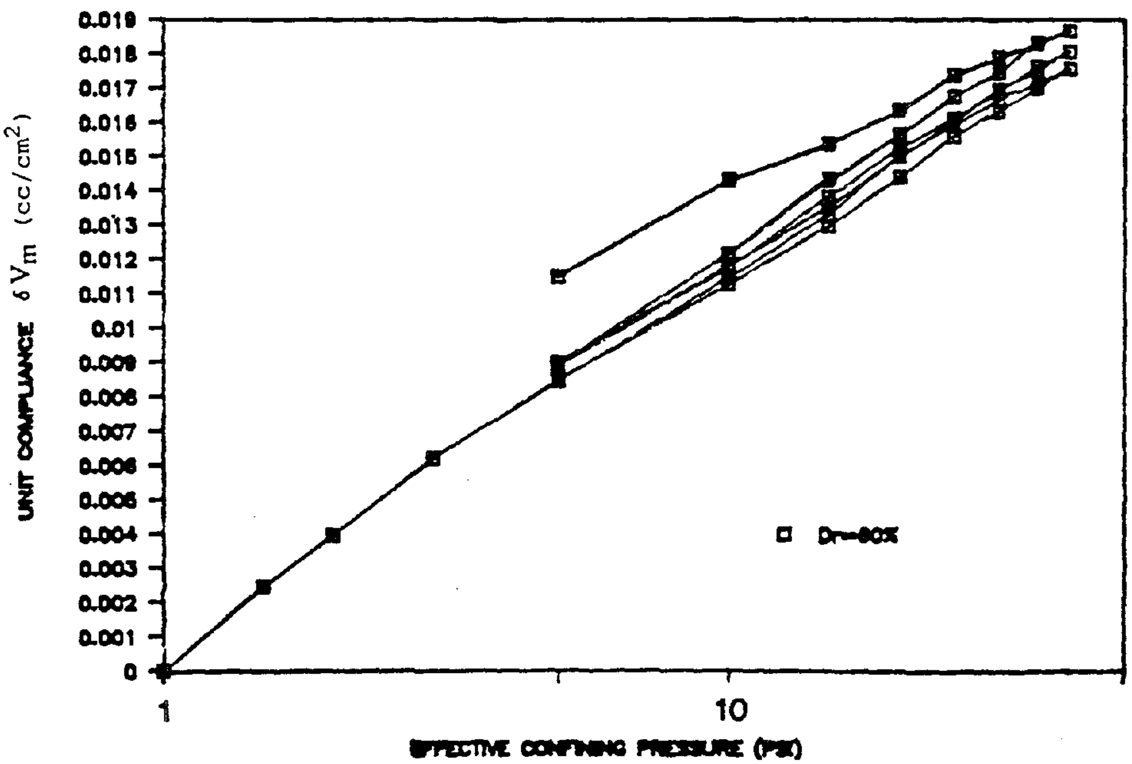


Figure 5.14b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Well Graded 4

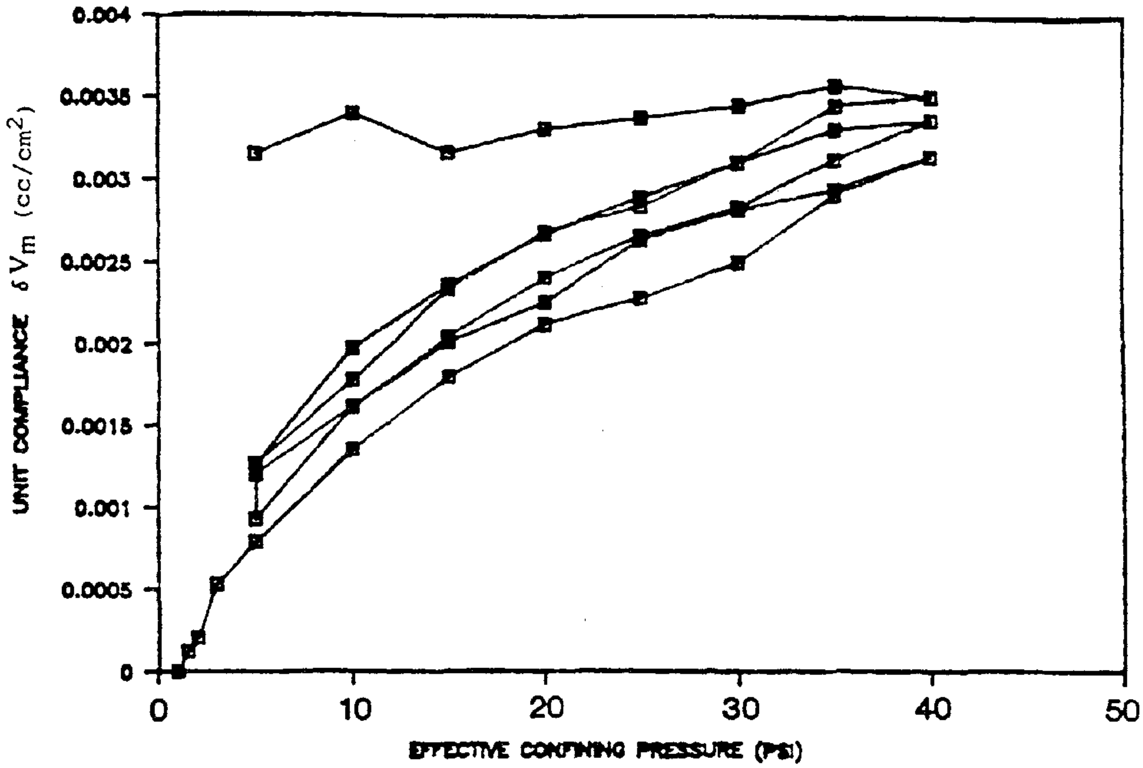


Figure 5.15a: Unit Membrane Compliance vs. Effective Confining Pressure: Modified Sacramento River Sand

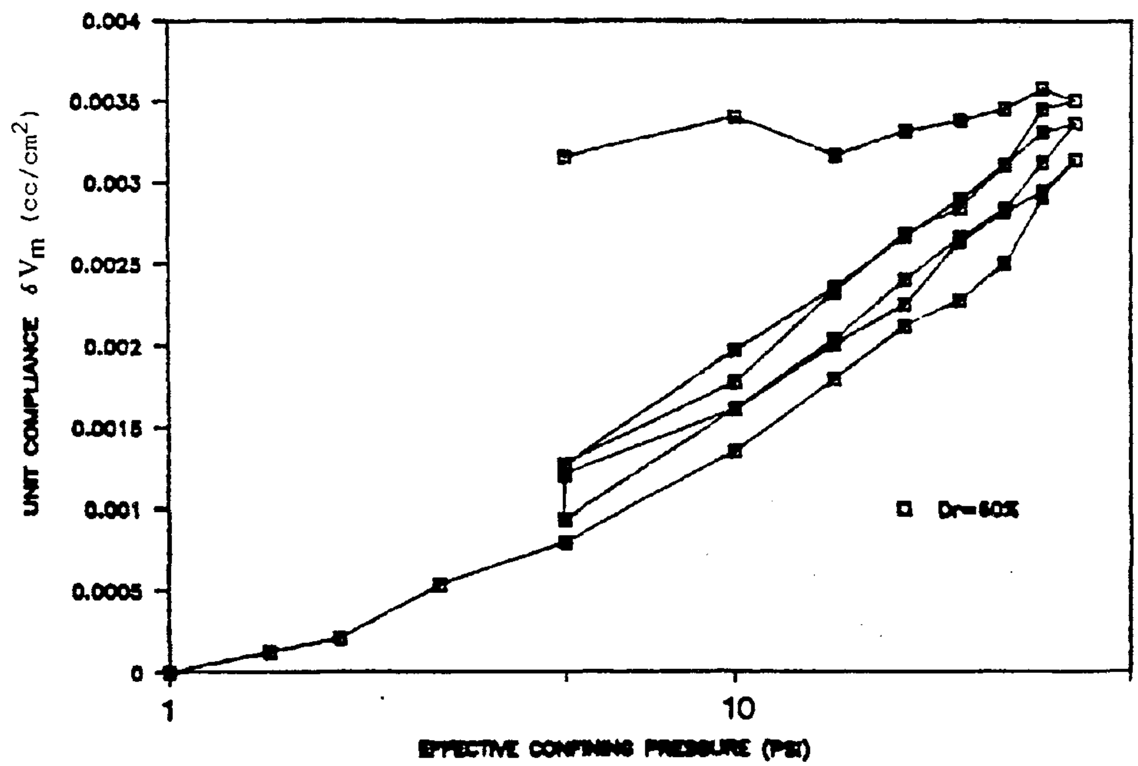


Figure 5.15b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Modified Sacramento River Sand

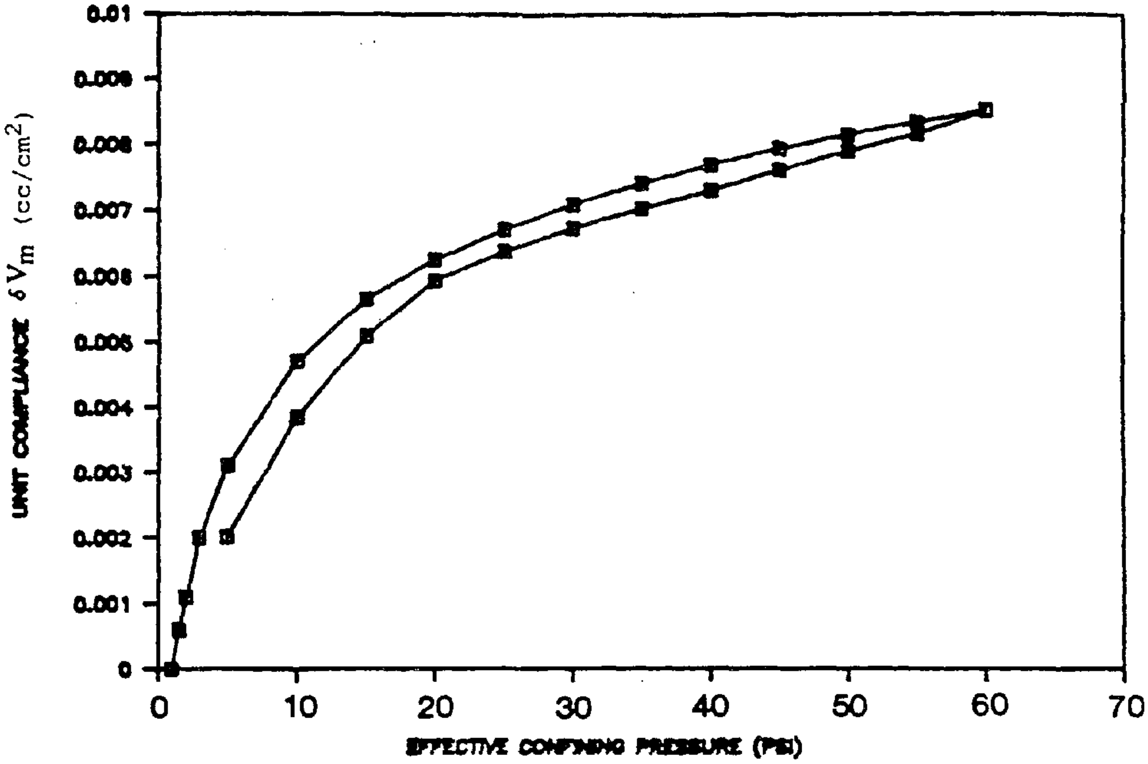


Figure 5.16a: Unit Membrane Compliance vs. Effective Confining Pressure: Ottawa Fine Sand

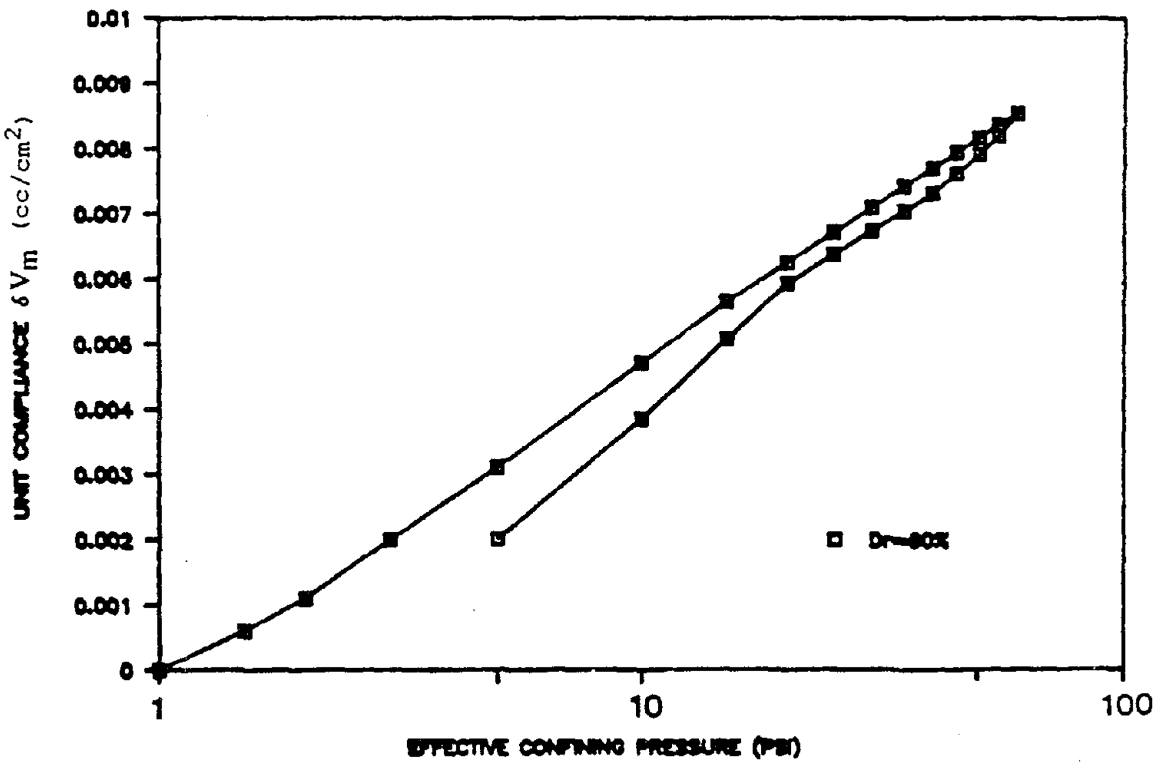


Figure 5.16b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Ottawa Fine Sand

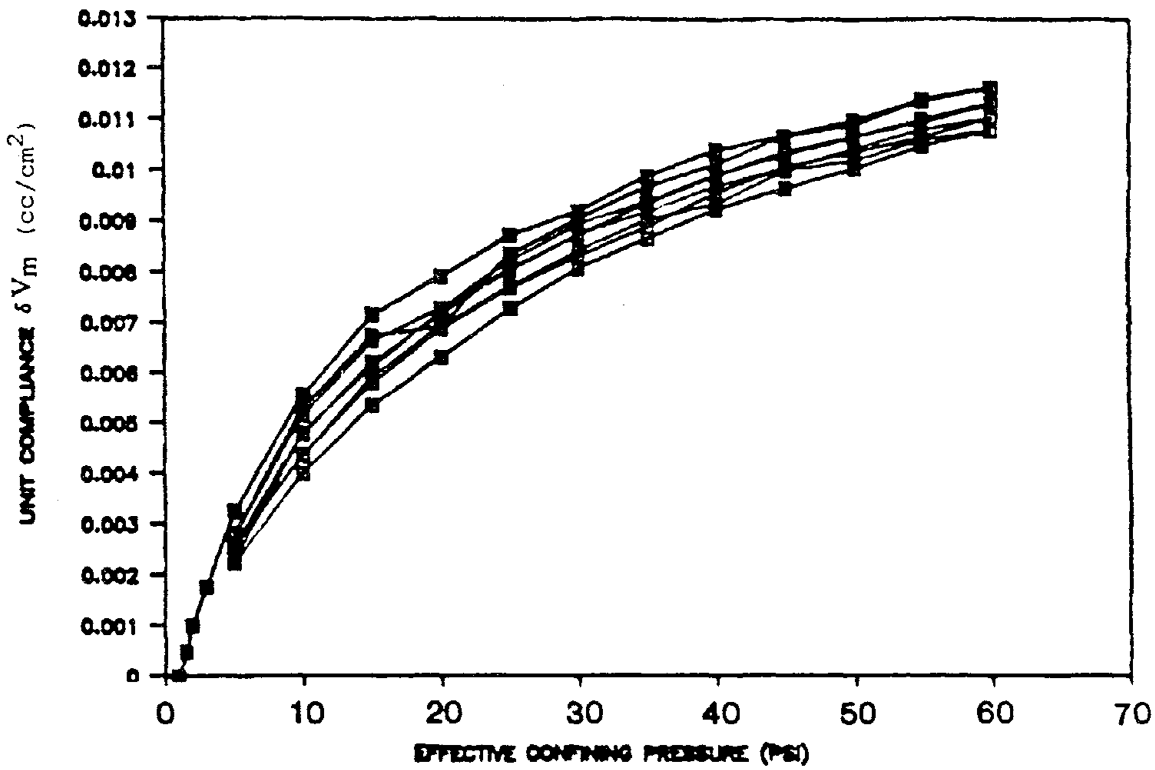


Figure 5.17a: Unit Membrane Compliance vs. Effective Confining Pressure: Ottawa 20-30 Sand

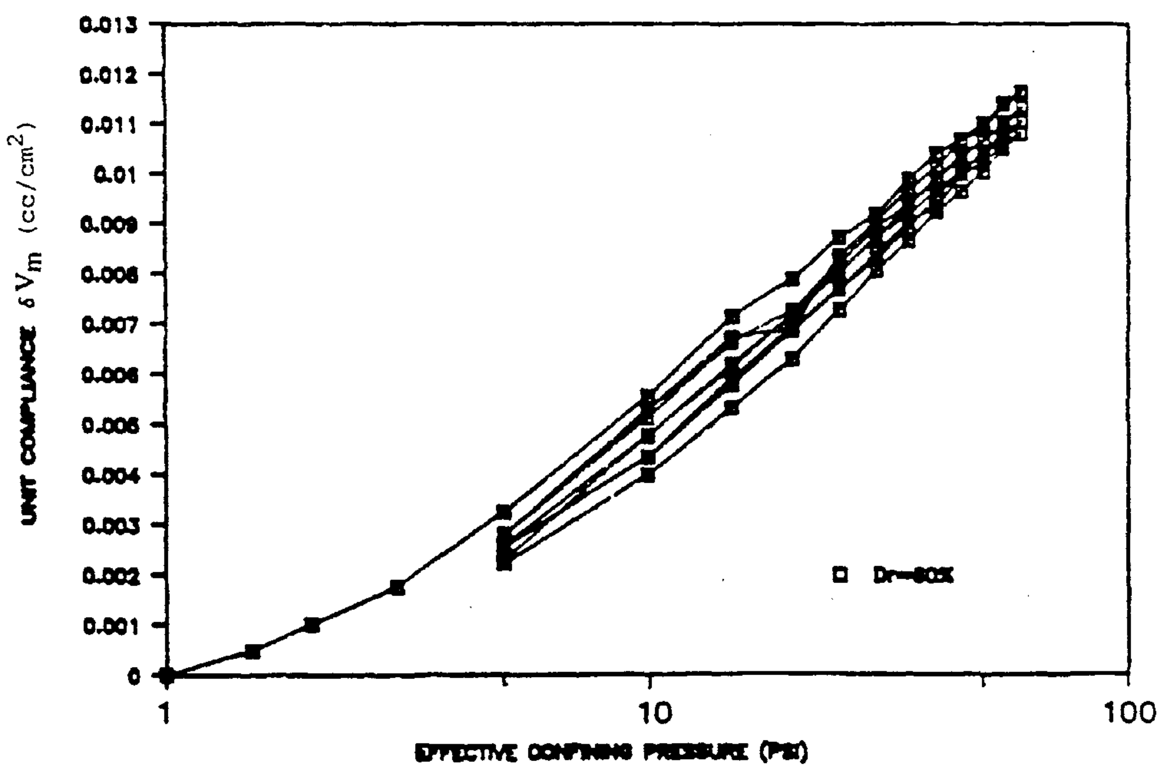


Figure 5.17b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Ottawa 20-30 Sand

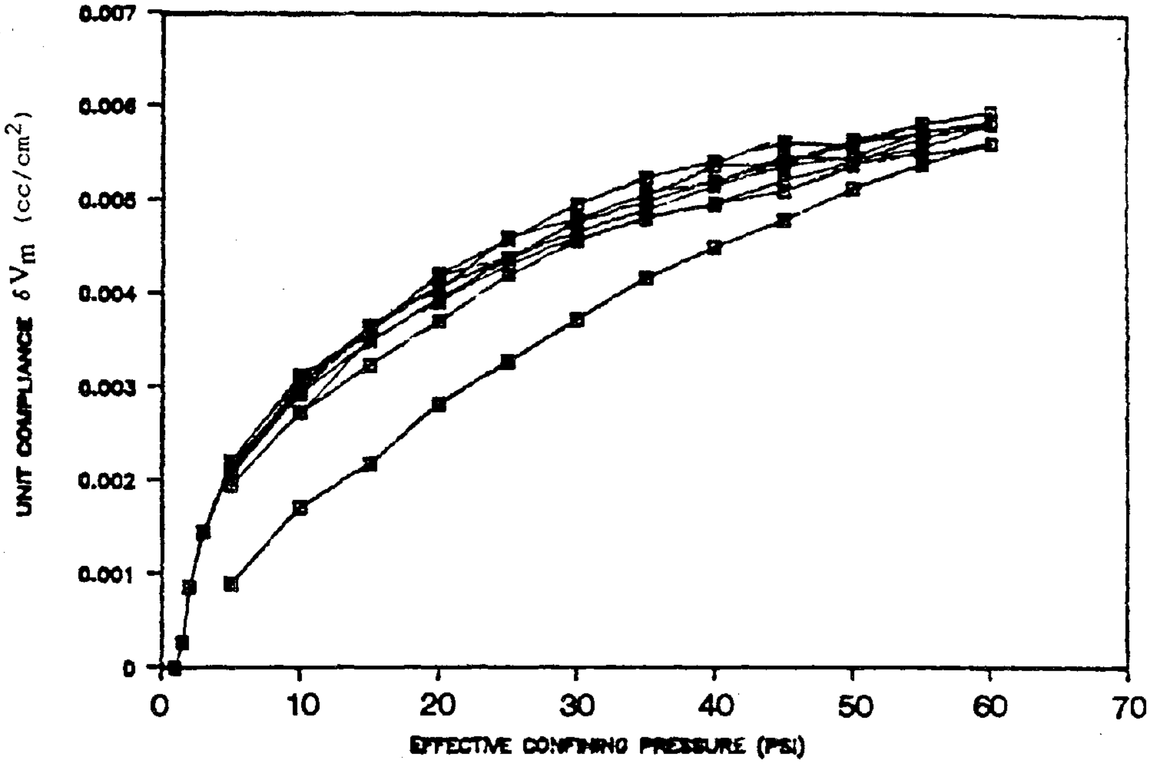


Figure 5.18a: Unit Membrane Compliance vs. Effective Confining Pressure: Monterey "0" Sand

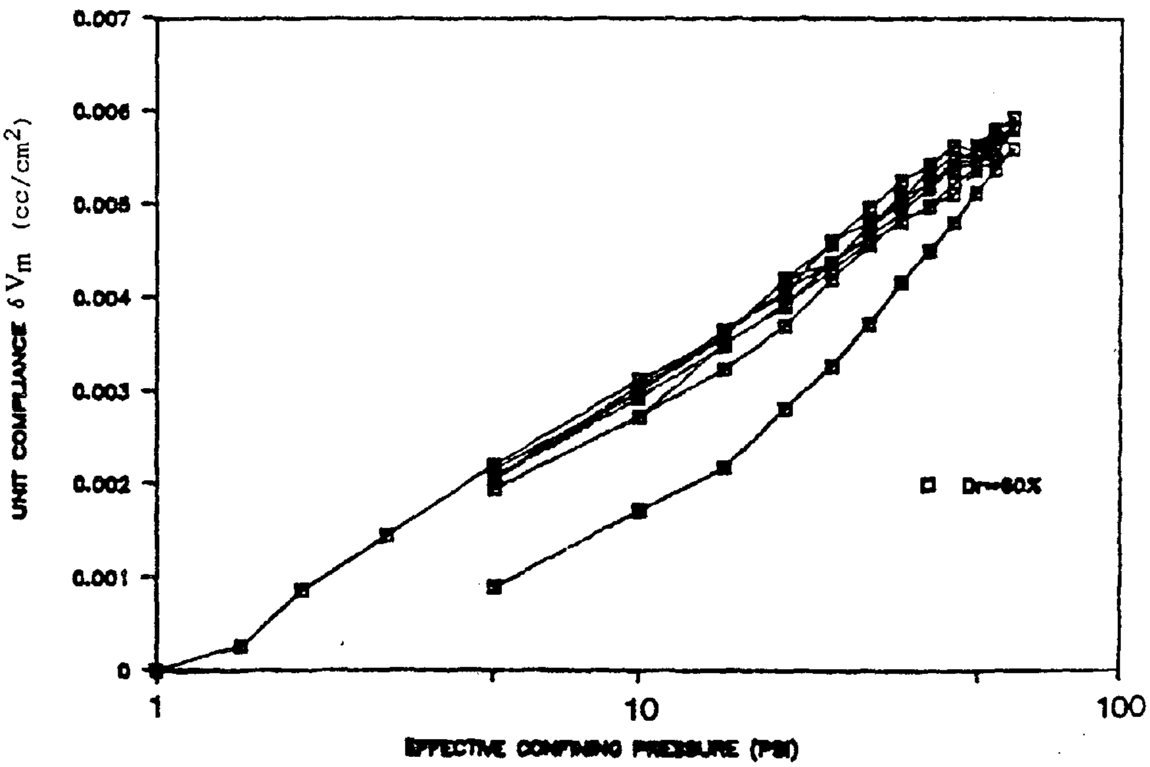


Figure 5.18b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Monterey "0" Sand

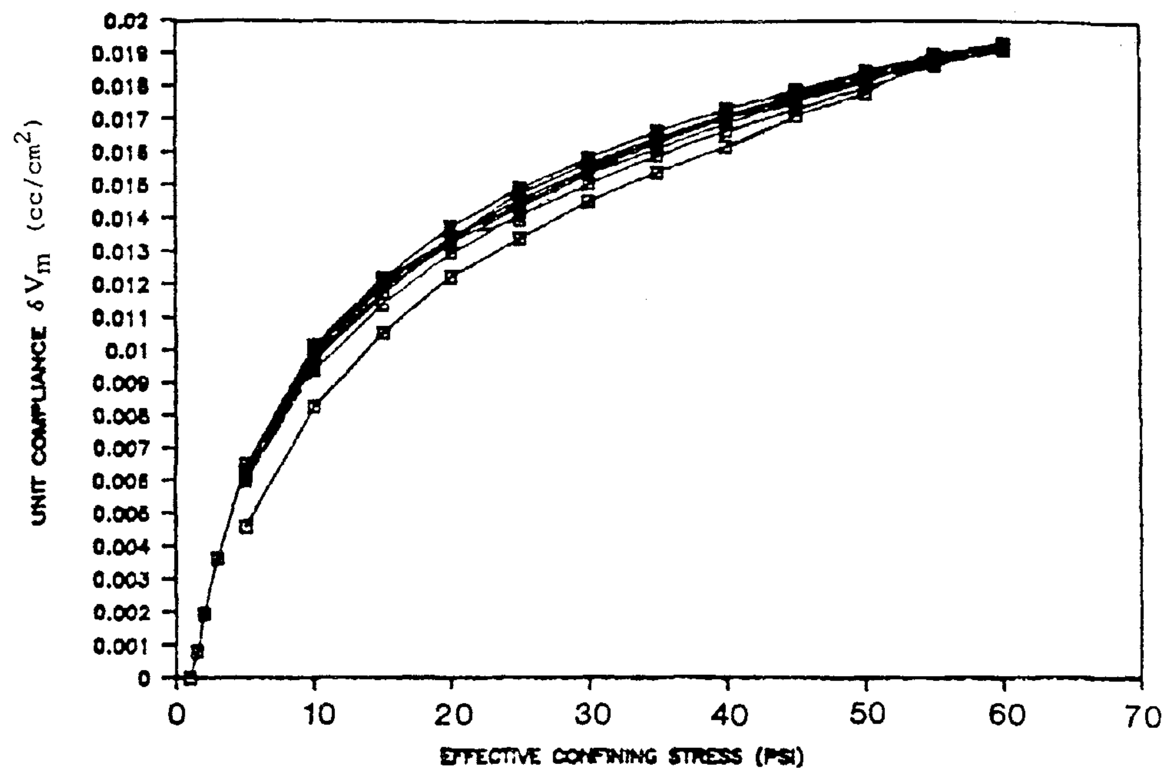


Figure 5.19a: Unit Membrane Compliance vs. Effective Confining Pressure: Monterey 16 Sand

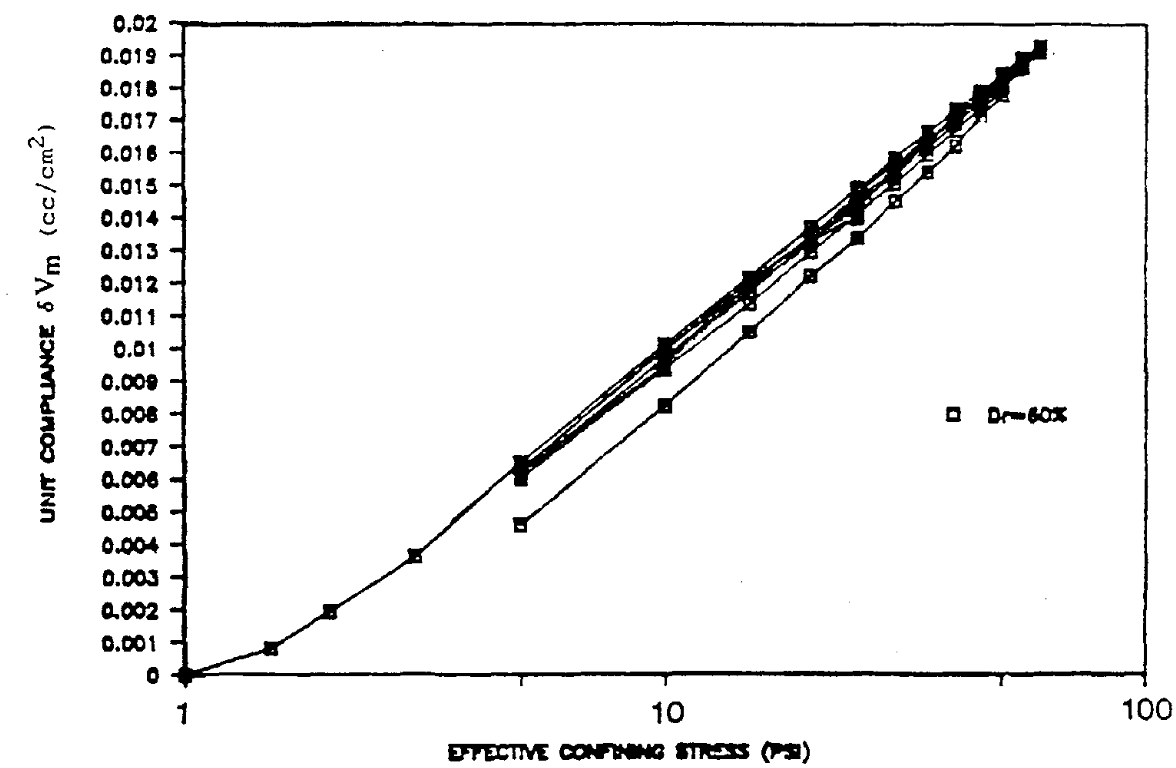


Figure 5.19b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Monterey 16 Sand

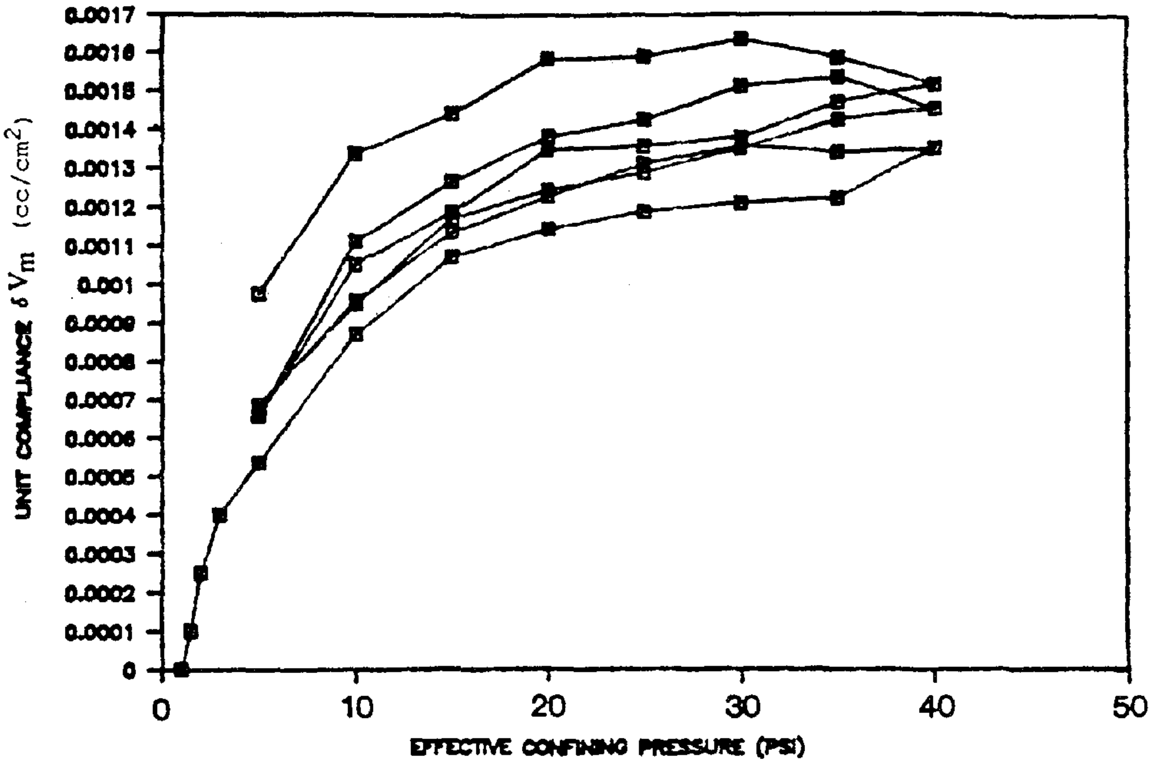


Figure 5.20a: Unit Membrane Compliance vs. Effective Confining Pressure: Gap Graded 1

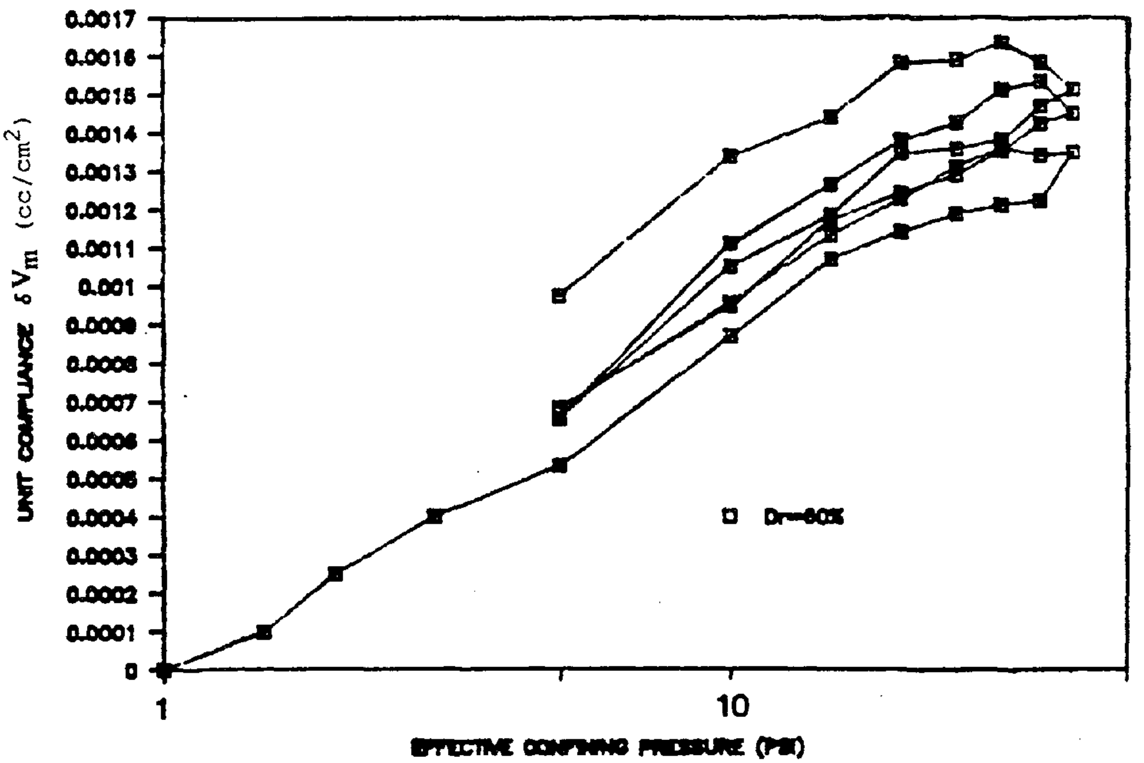


Figure 5.20b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Gap Graded 1

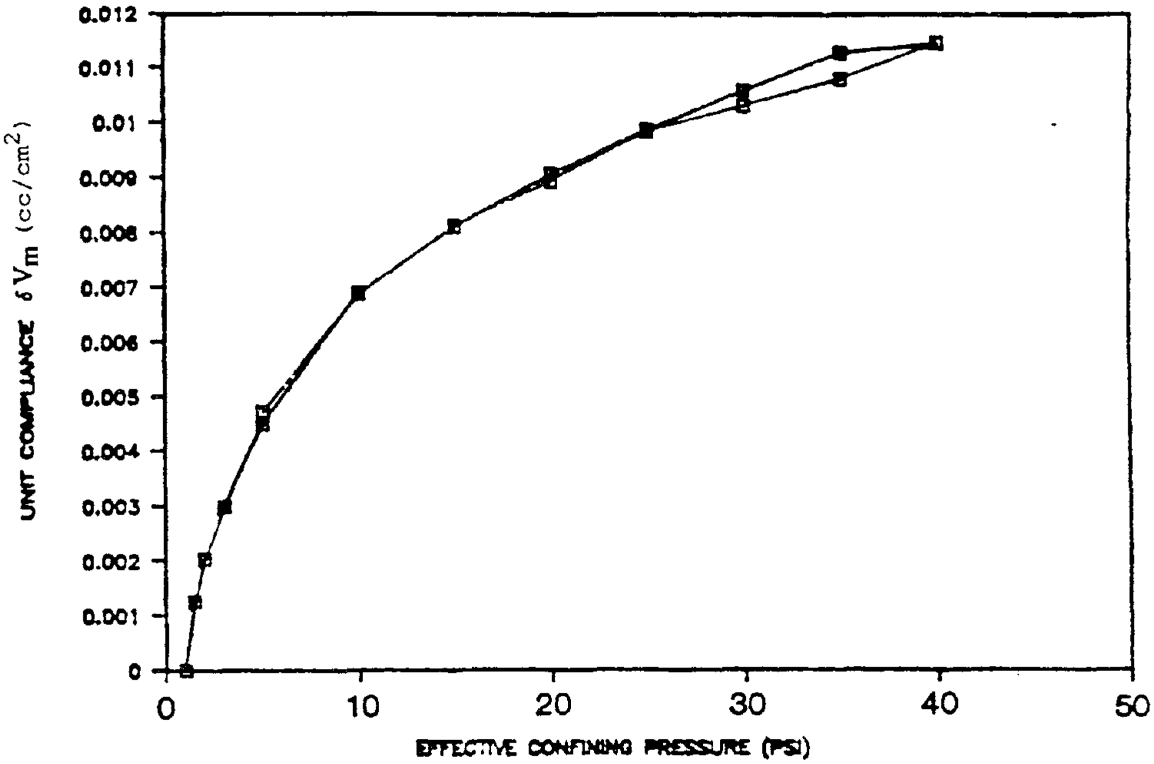


Figure 5.21a: Unit Membrane Compliance vs. Effective Confining Pressure: Gap Graded 2

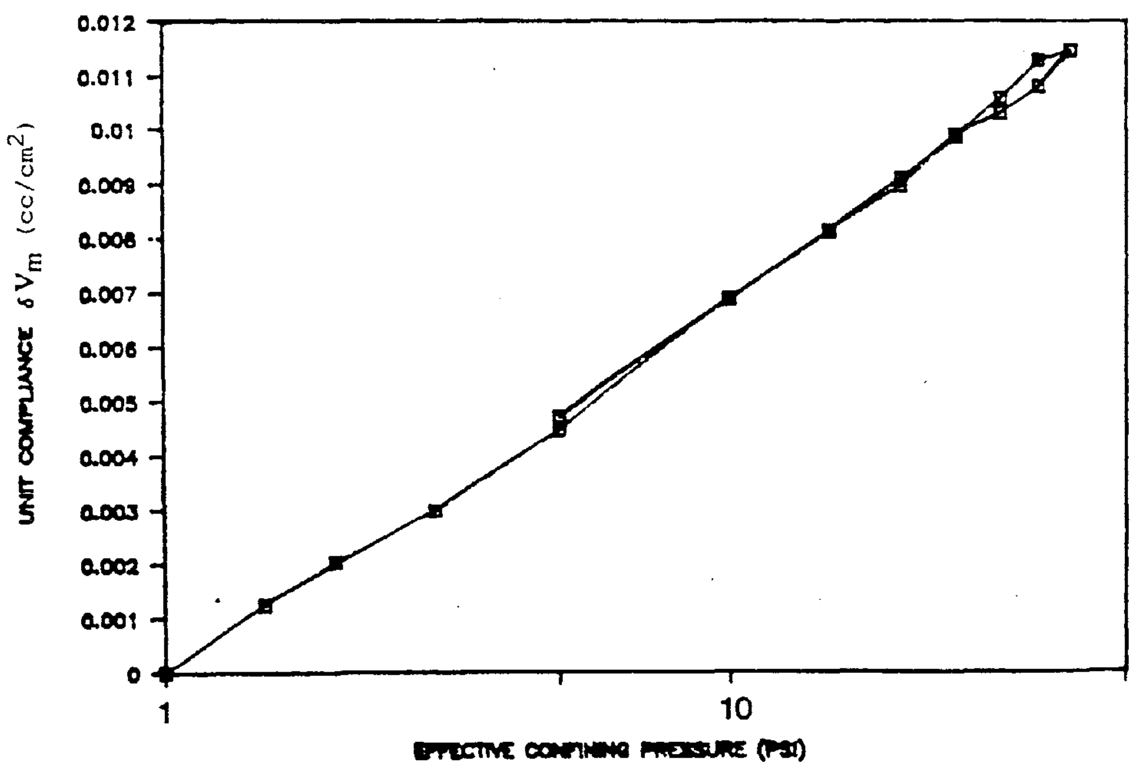


Figure 5.21b: Unit Membrane Compliance vs. Log of Effective Confining Pressure: Gap Graded 2

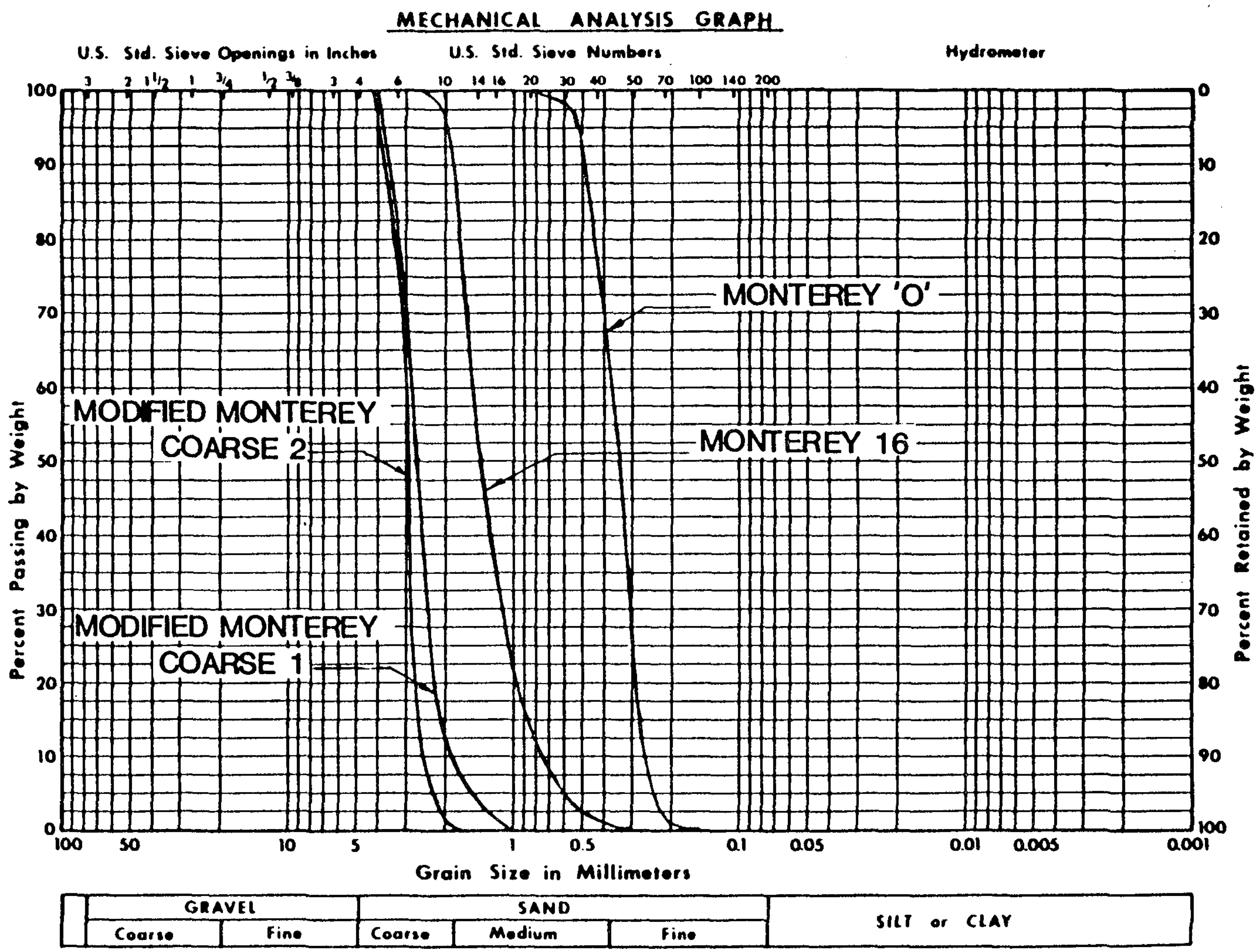


Figure 5.22: Gradations of Soils Tested to Investigate Membrane Compliance as Part of These Studies

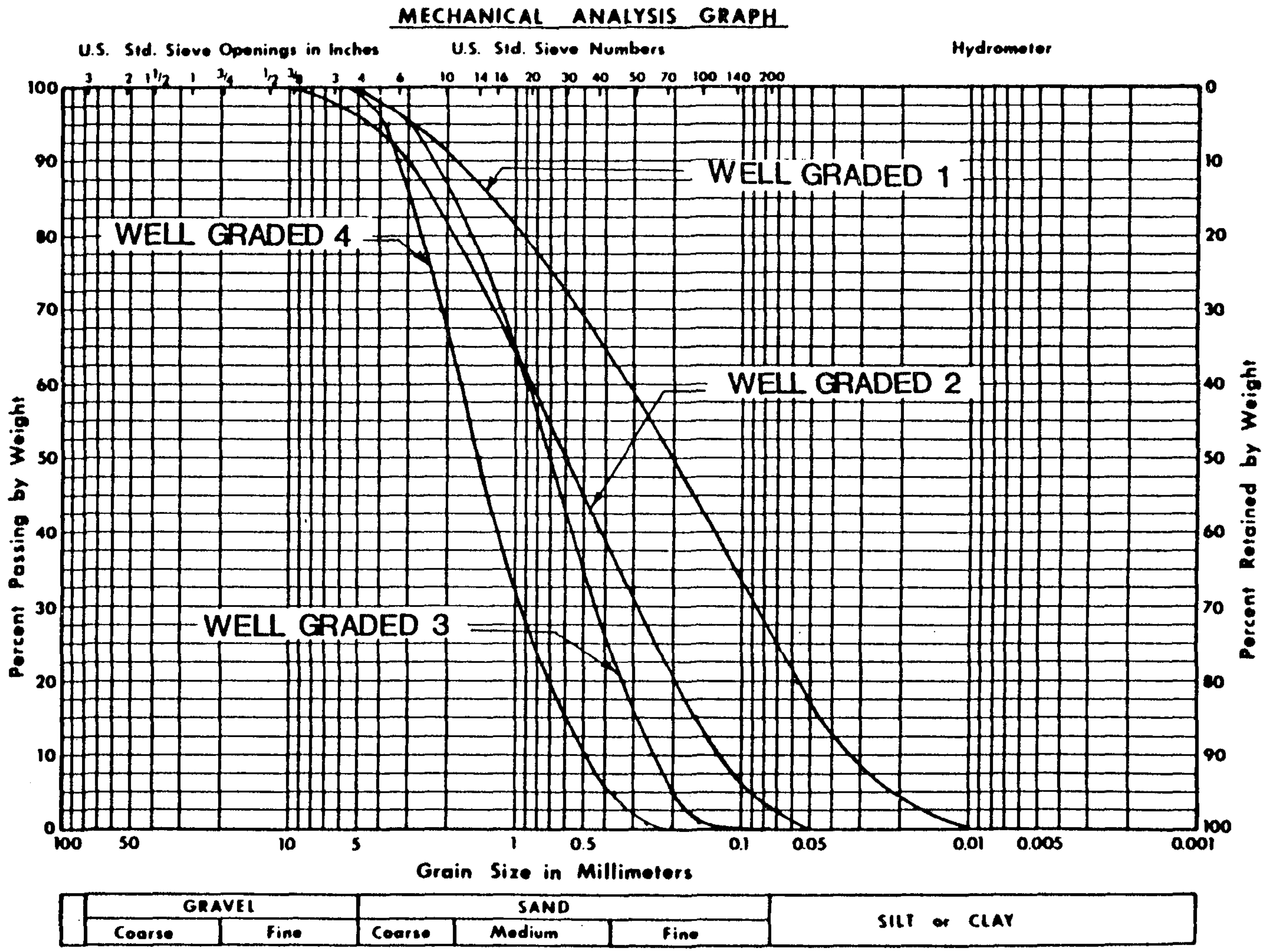


Figure 5.23: Gradations of Soils Tested to Investigate Membrane Compliance as Part of These Studies

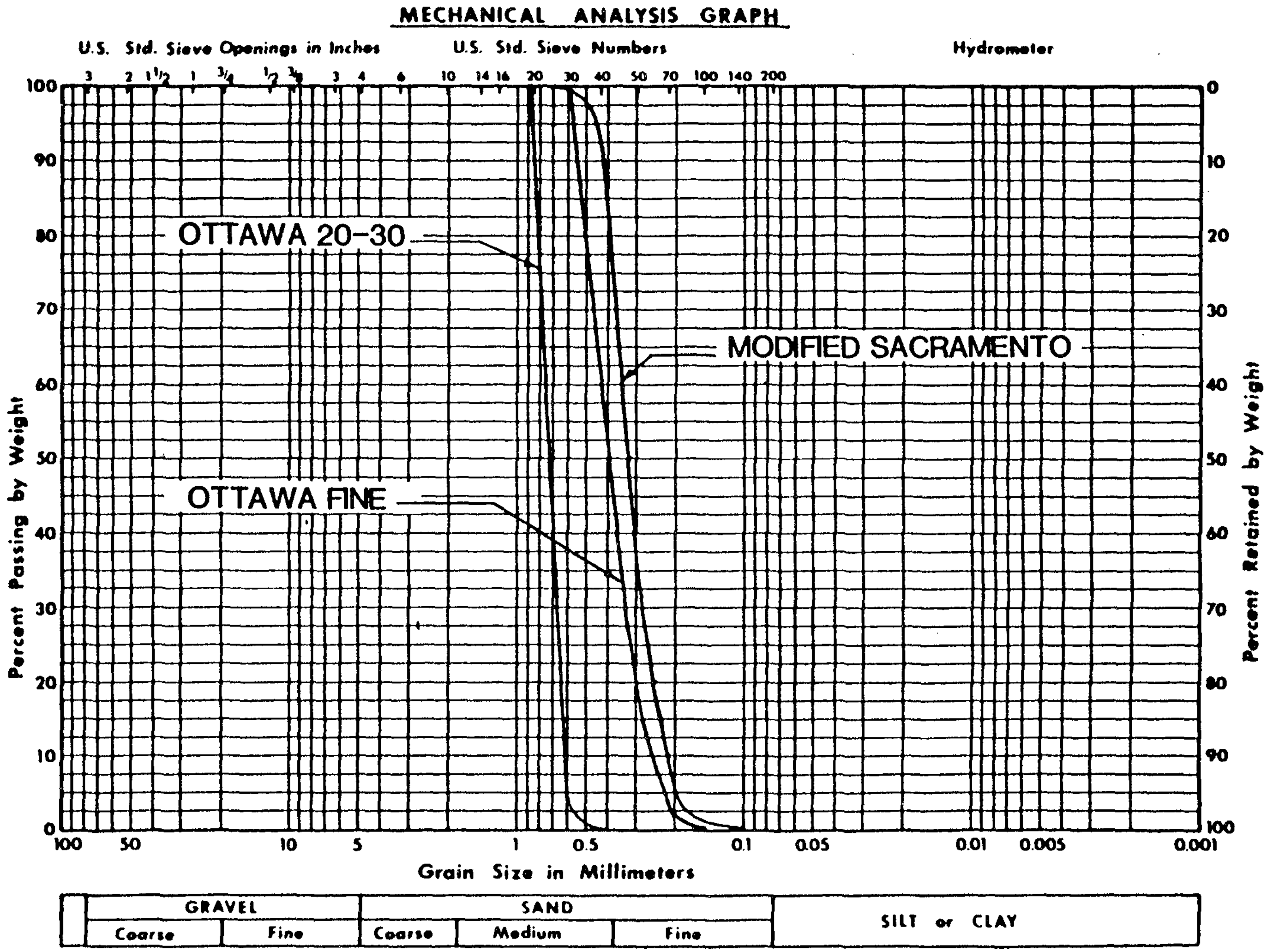


Figure 5.24: Gradations of Soils Tested to Investigate Membrane Compliance as Part of These Studies

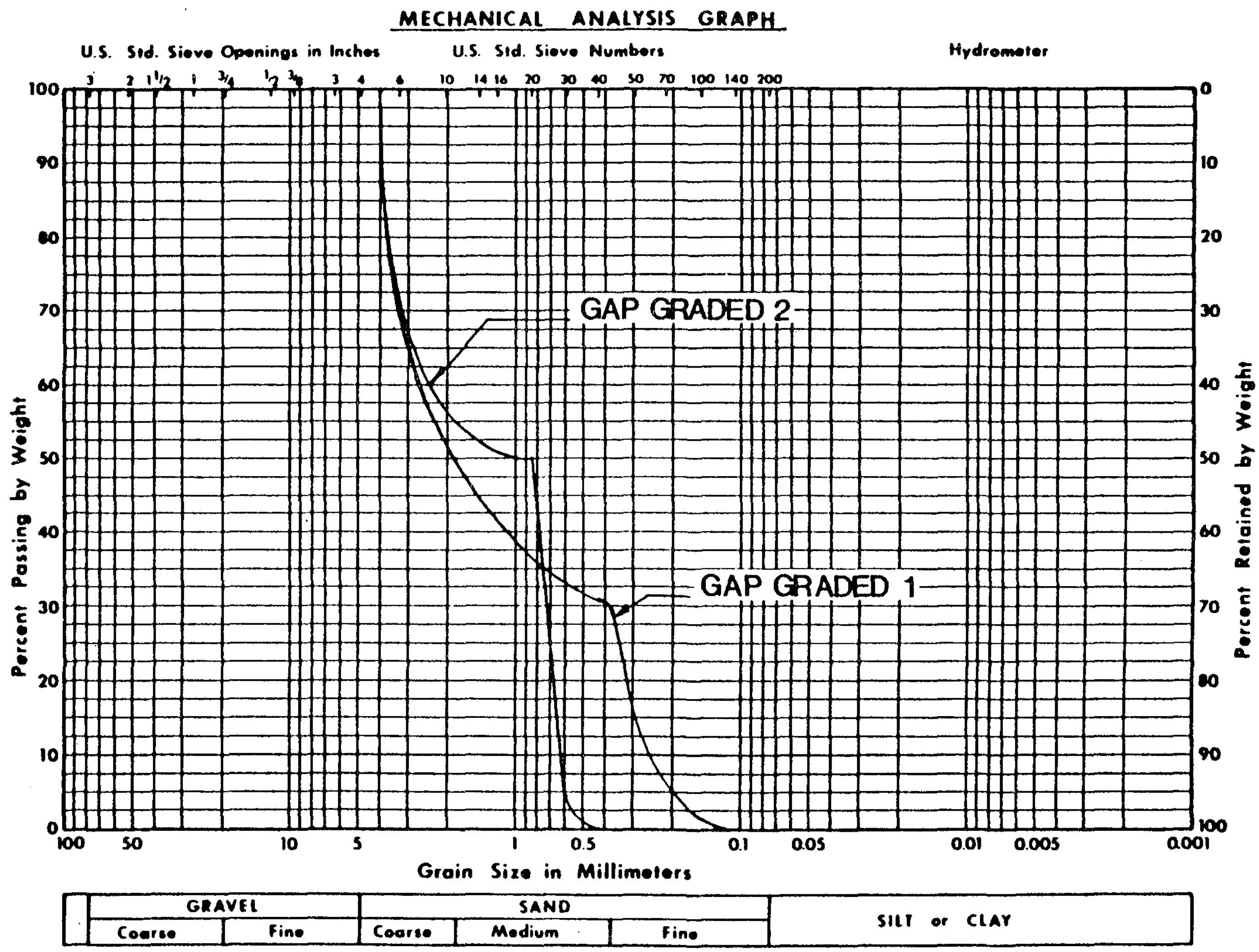


Figure 5.25: Gradations of Soils Tested to Investigate Membrane Compliance as Part of These Studies

representative particle sizes for the gravelly materials spanned three orders of magnitude from medium sands through coarse gravels. Among these were several more broadly graded materials. A listing of these gravelly soils is presented in Table 5.2, with a brief summary description and membrane compliance characteristics evaluated for each. Figures 5.26 through 5.36 show the membrane compliance values measured for these gravelly soils, and Figures 5.37 through 5.41 show the gradations for these materials. Again, all compliance evaluations were plotted as unit membrane compliance (volume change per unit membrane area: in^3/in^2) vs. effective confining stress and $\log_{10}$ of effective confining stress. All of the gravelly soils tested were prepared to $D_R \approx 55\%$, except where otherwise noted.

The results of the compliance measurement tests show very clearly the log-linear relationship between unit compliance and effective stress from which the slope can be taken to give the normalized unit compliance (S) for each material.

A number of additional data points were found in a review of the literature. Those data points for which the compliance measurement techniques were judged likely to provide reasonably accurate results were selected and combined with ours to build up a database with which to generate a relationship between measured compliance values and material particle sizes. Table 5.3 lists those materials tested by previous investigators with a brief summary description and membrane compliance value evaluated for each. Figures 5.42 through 5.47 show the gradations for these additional materials.

A composite plot of the relationship between normalized compliance values (S) as a function of mean particle size (D_{50}) for all of the available data for which the compliance measurement techniques were judged likely to provide reasonably accurate results, is shown in Figure 5.48. The scatter of points in this plot is

Table 5.2: Gradation and Membrane Compliance Characteristics of Gravelly Soils Tested

Soil Name	USCS Classification	D_{10} (mm)	D_{20} (mm)	D_{50} (mm)	$S^{\dagger}$ (cm/$\Delta\log\sigma_3'$)	Figure
Material 1	GP	11	12	14	0.117	5.26
Material 2	GP	15	18	24	0.157	5.27
Material 3	GP	5.9	7	8	0.072	5.28
Material 4	GP	39	47	56	0.406	5.29
Material 5	GP	3.1	4	5	0.0403	5.30
Material 6	GP	4.2	9	25	0.088	5.31
Material 7	GP	3.8	5.4	12.7	0.055	5.32
Material 8	GP	3.1	3.8	4.2	0.04	5.33
Material 9	GP	4.2	5	5.4	0.052	5.34
Material 10	GW	0.23	0.85	8.5	0.0115	5.35
Material 11	GP	3	6.5	19	0.0645	5.36

† = Normalized Unit Compliance; change in volume per unit membrane area per log-cycle change in effective conf. stress (cc/cm^2/$\Delta\log\sigma_3'$).

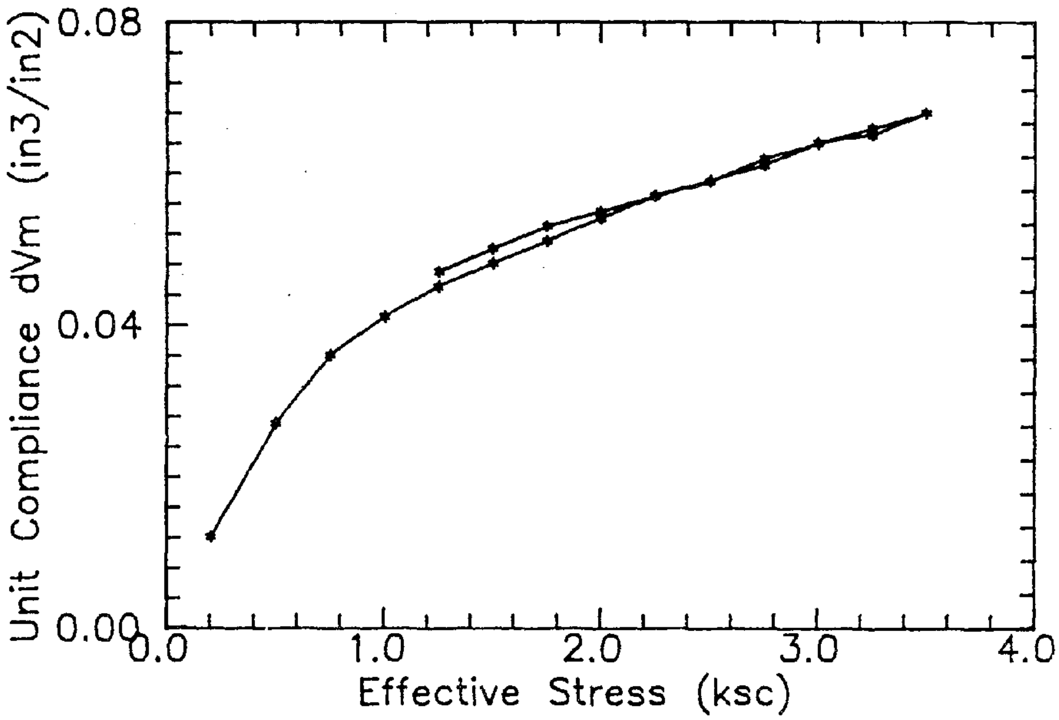


Figure 5.26a: Unit Membrane Compliance vs. Effective Confining Stress: Material 1

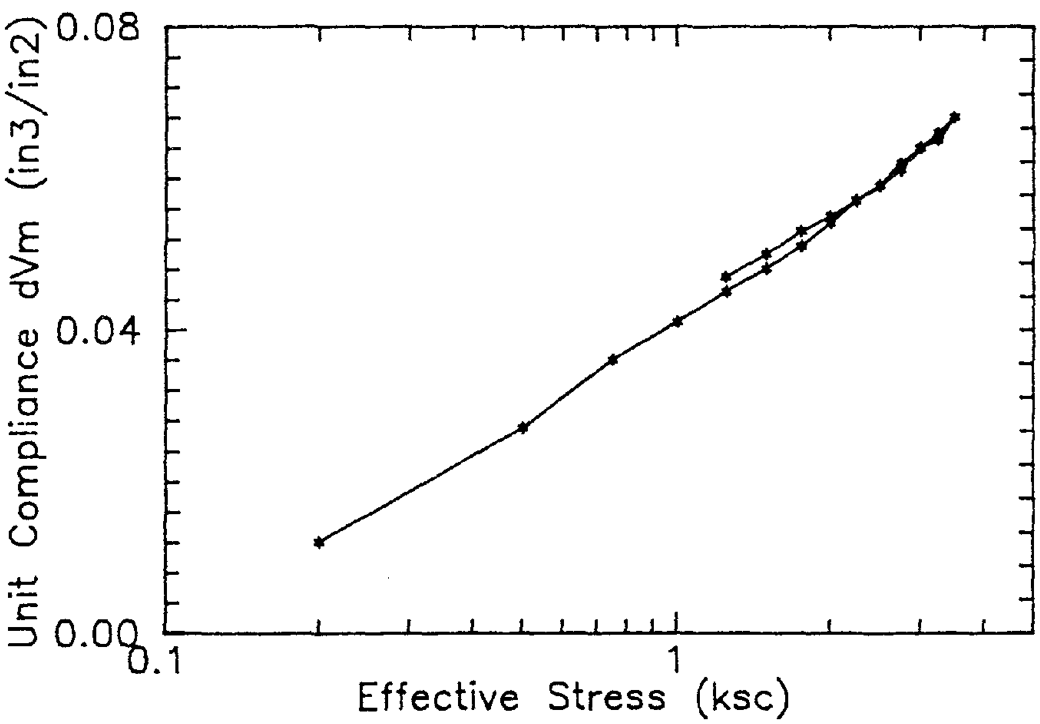


Figure 5.26b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 1

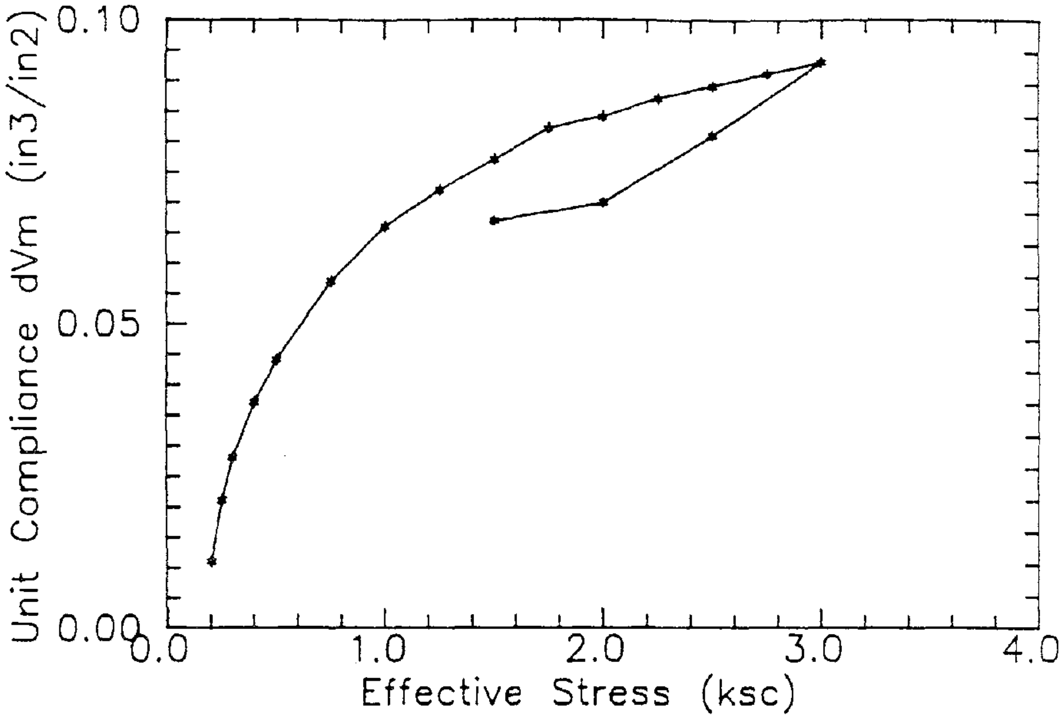


Figure 5.27a: Unit Membrane Compliance vs. Effective Confining Stress: Material 2

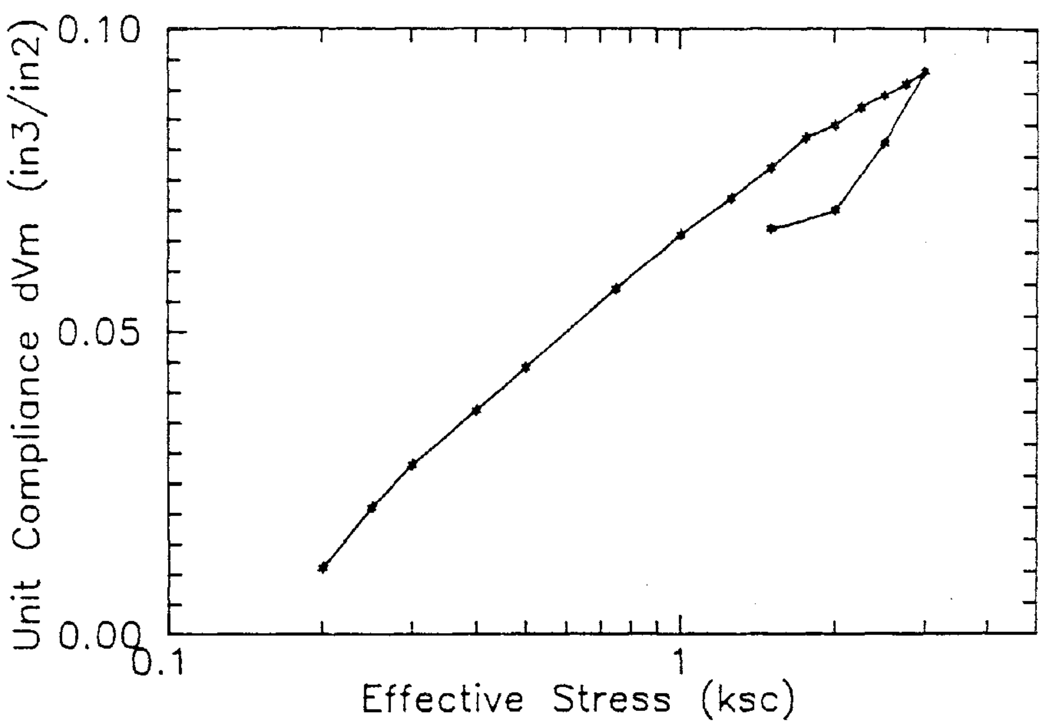


Figure 5.27b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 2

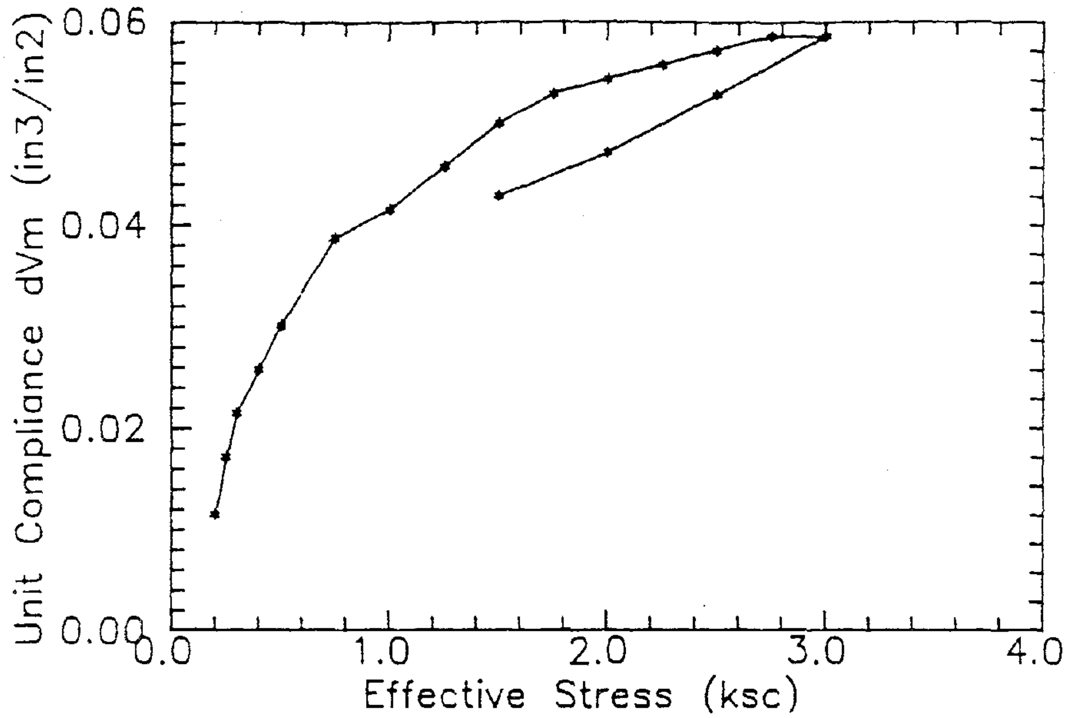


Figure 5.28a: Unit Membrane Compliance vs. Effective Confining Stress: Material 3

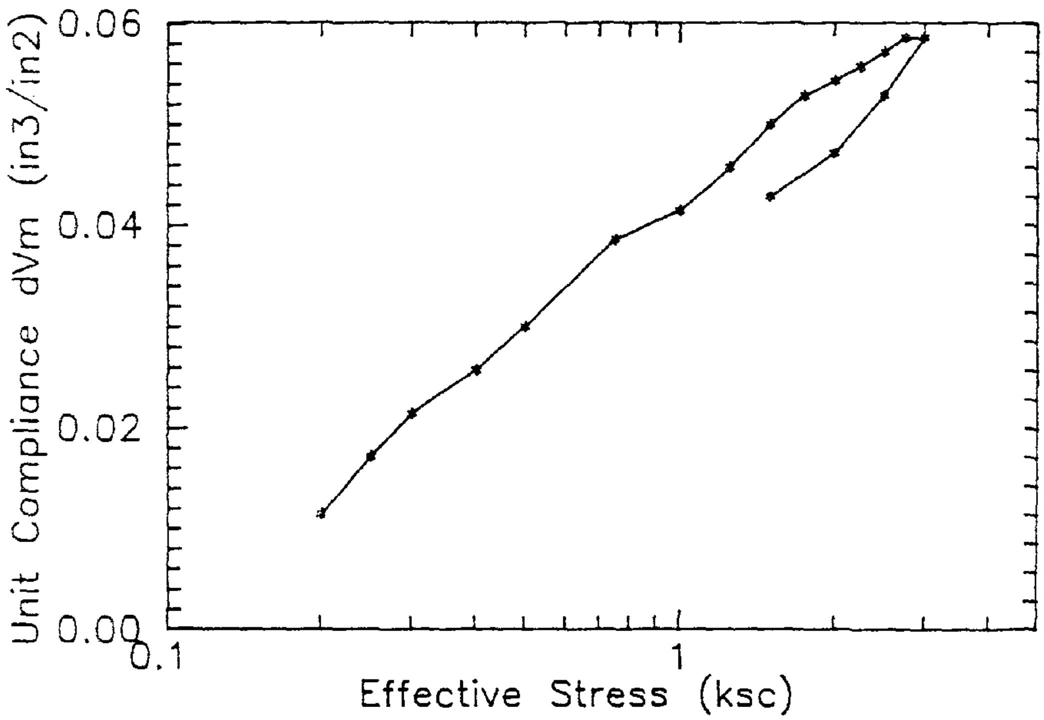


Figure 5.28b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 3

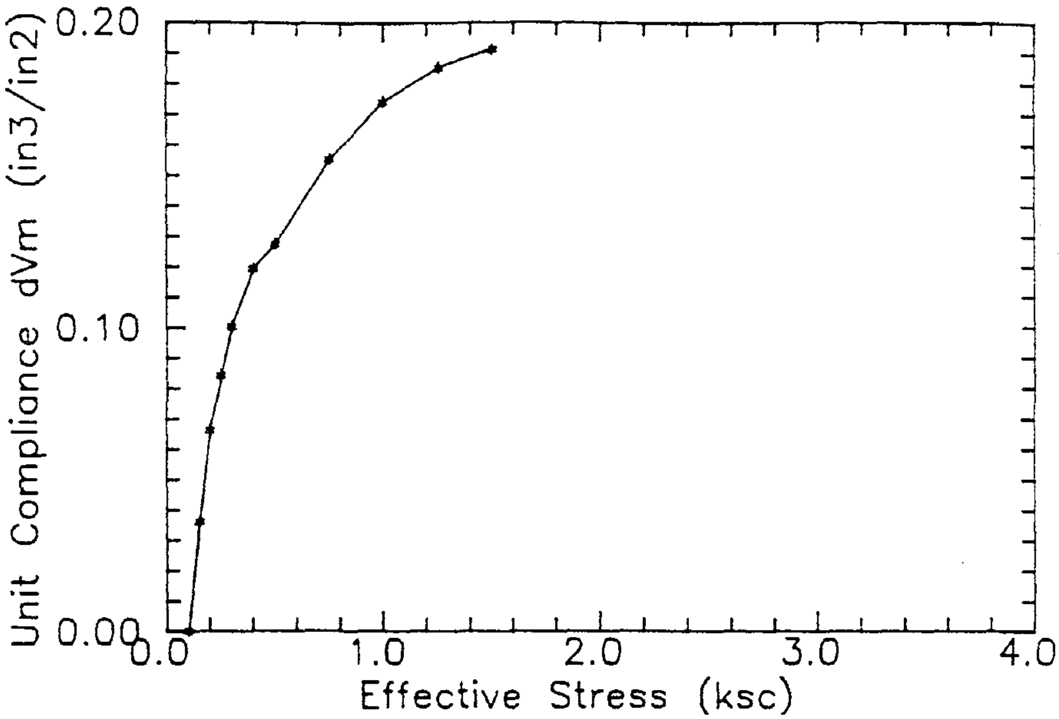


Figure 5.29a: Unit Membrane Compliance vs. Effective Confining Stress: Material 4

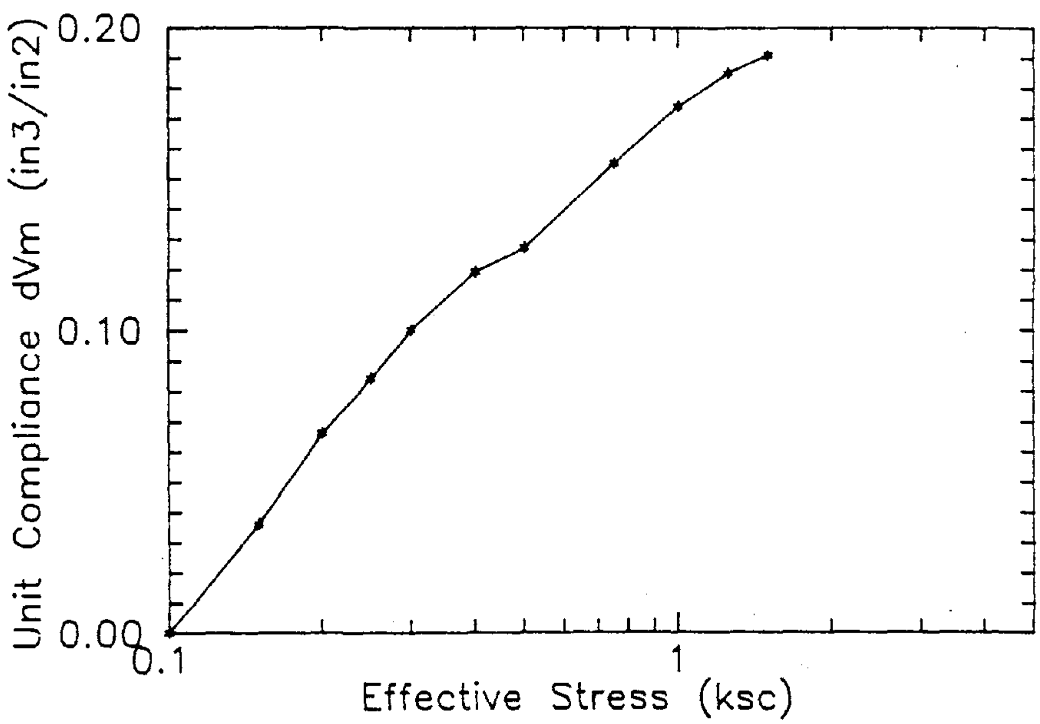


Figure 5.29b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 4

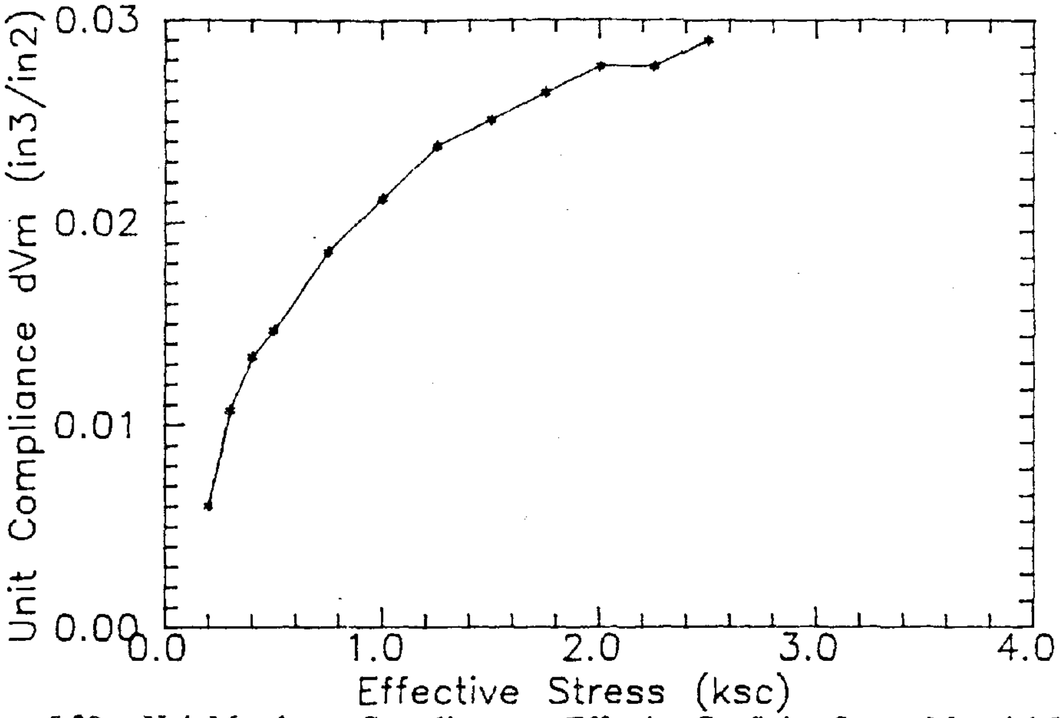


Figure 5.30a: Unit Membrane Compliance vs. Effective Confining Stress: Material 5

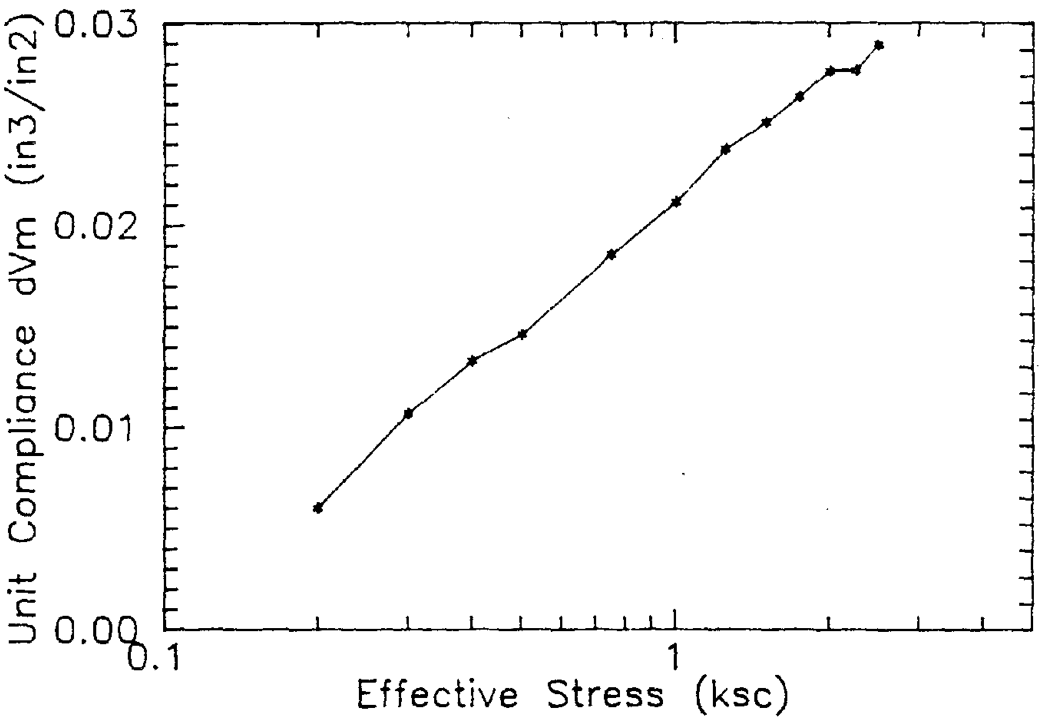


Figure 5.30b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 5

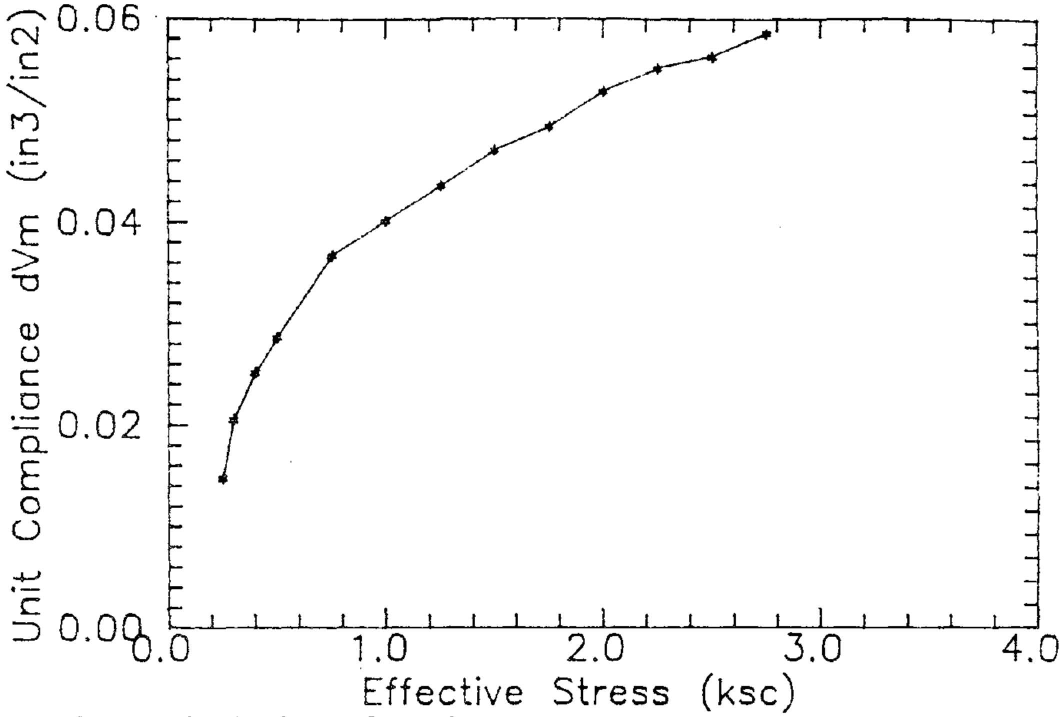


Figure 5.31a: Unit Membrane Compliance vs. Effective Confining Stress: Material 6

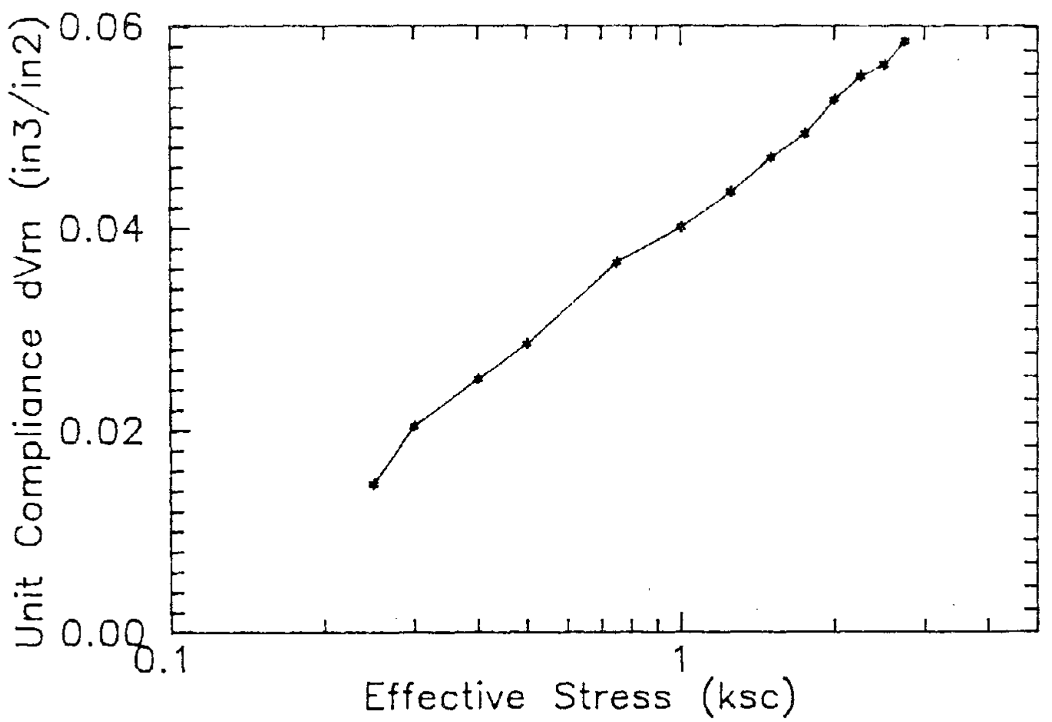


Figure 5.31b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 6

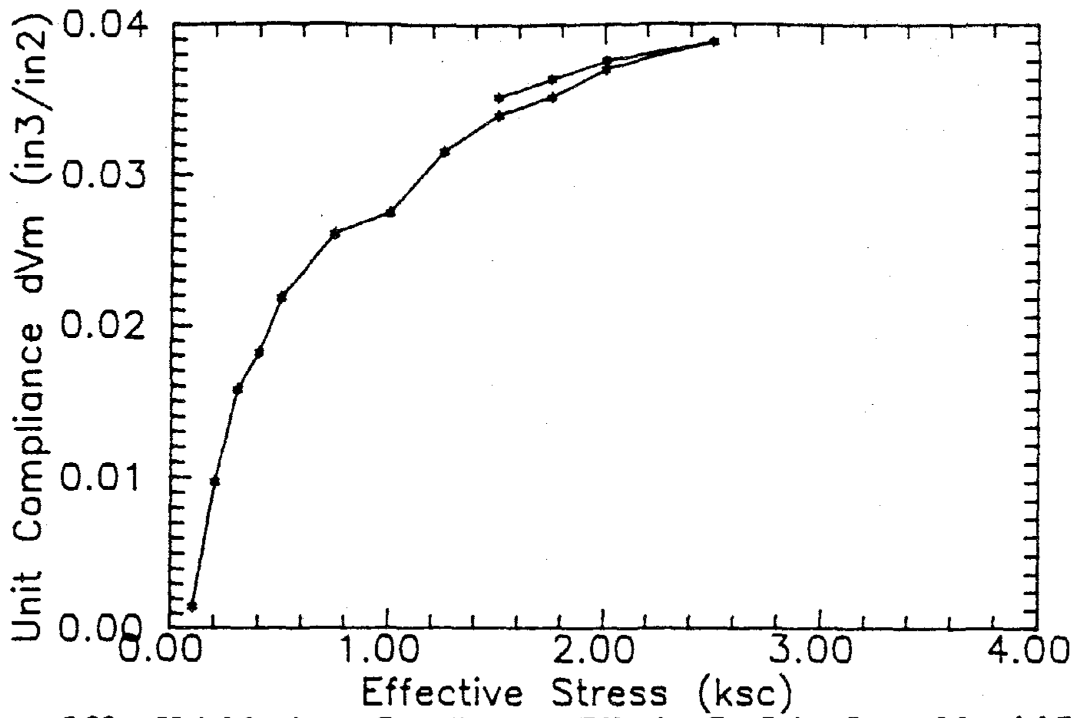


Figure 5.32a: Unit Membrane Compliance vs. Effective Confining Stress: Material 7

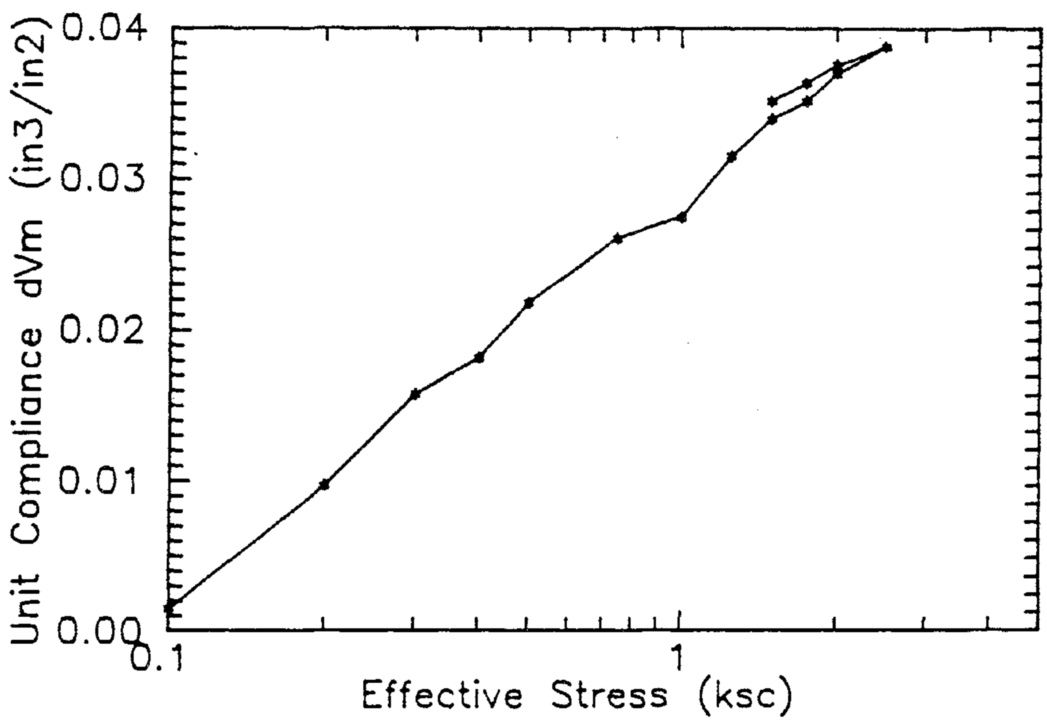


Figure 5.32b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 7

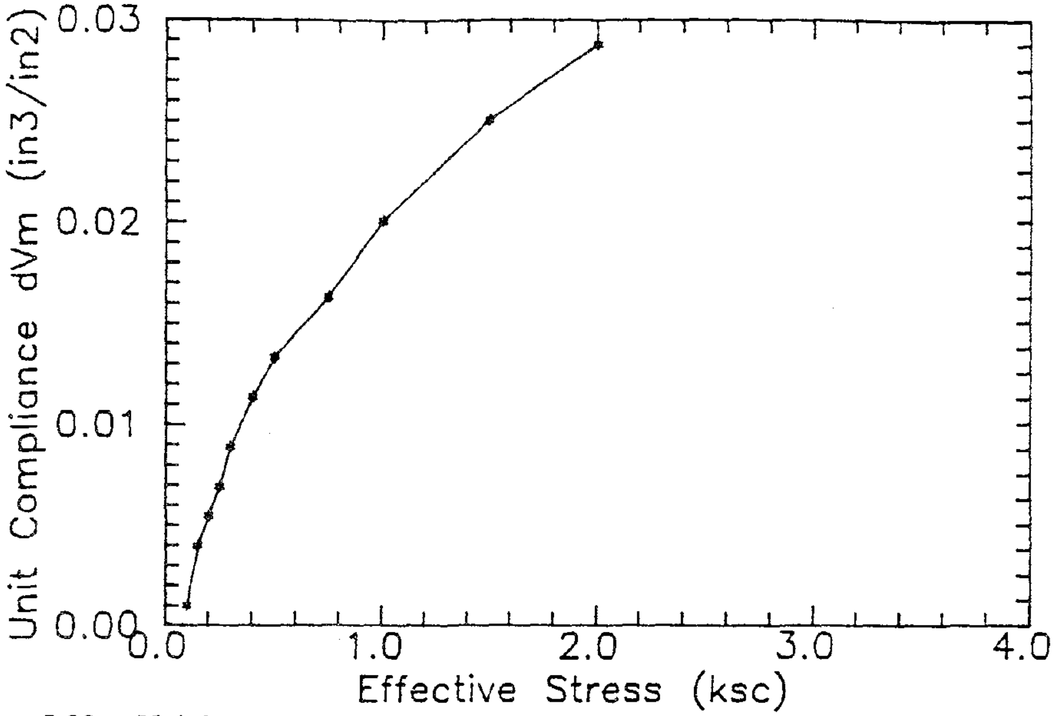


Figure 5.33a: Unit Membrane Compliance vs. Effective Confining Stress: Material 8

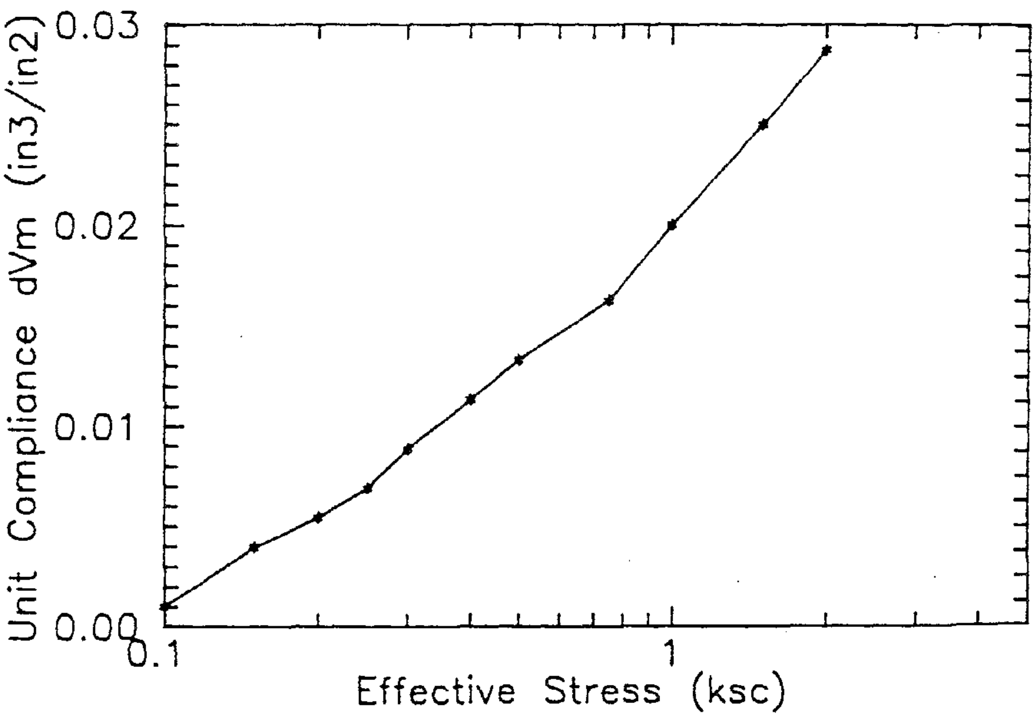


Figure 5.33b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 8

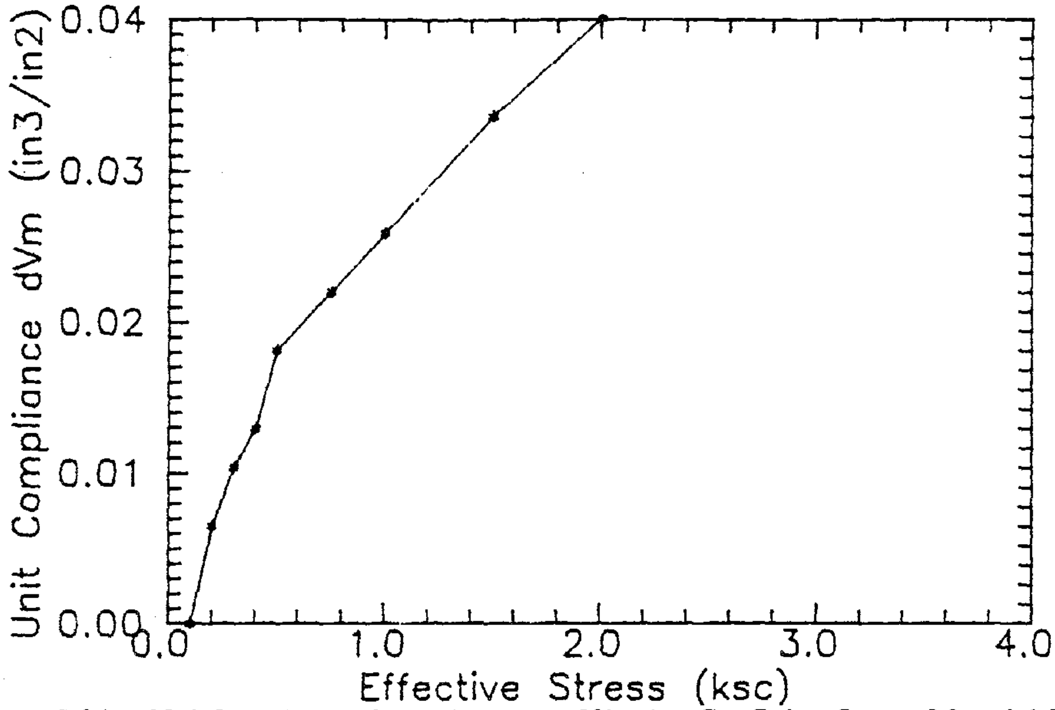


Figure 5.34a: Unit Membrane Compliance vs. Effective Confining Stress: Material 9

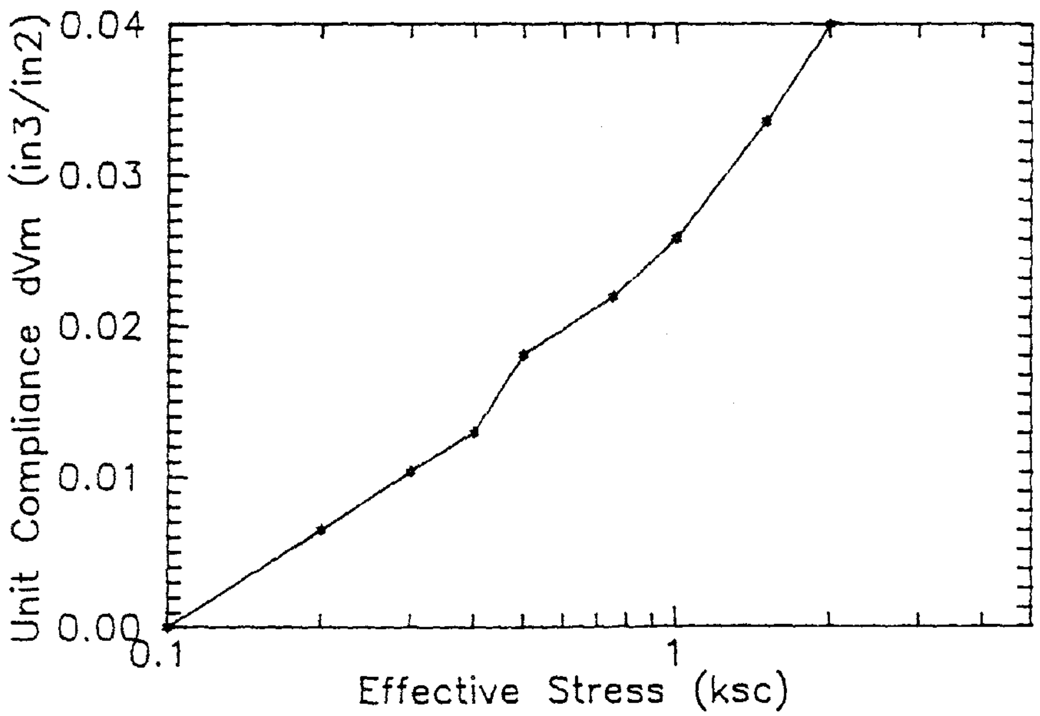


Figure 5.34b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 9

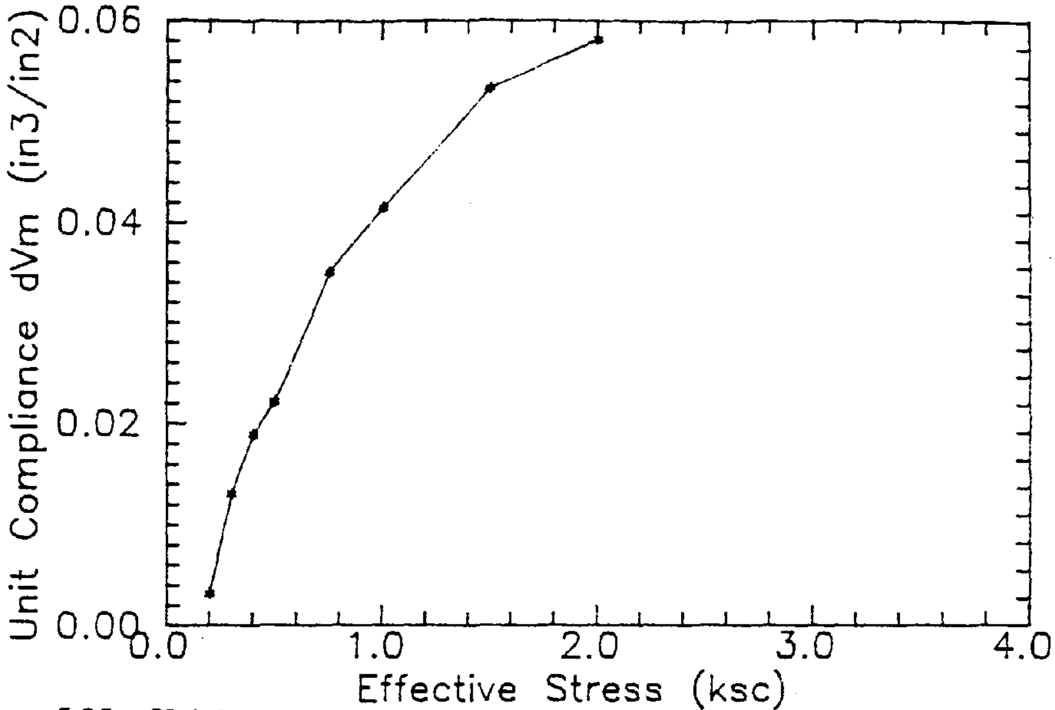


Figure 5.35a: Unit Membrane Compliance vs. Effective Confining Stress: Material 10

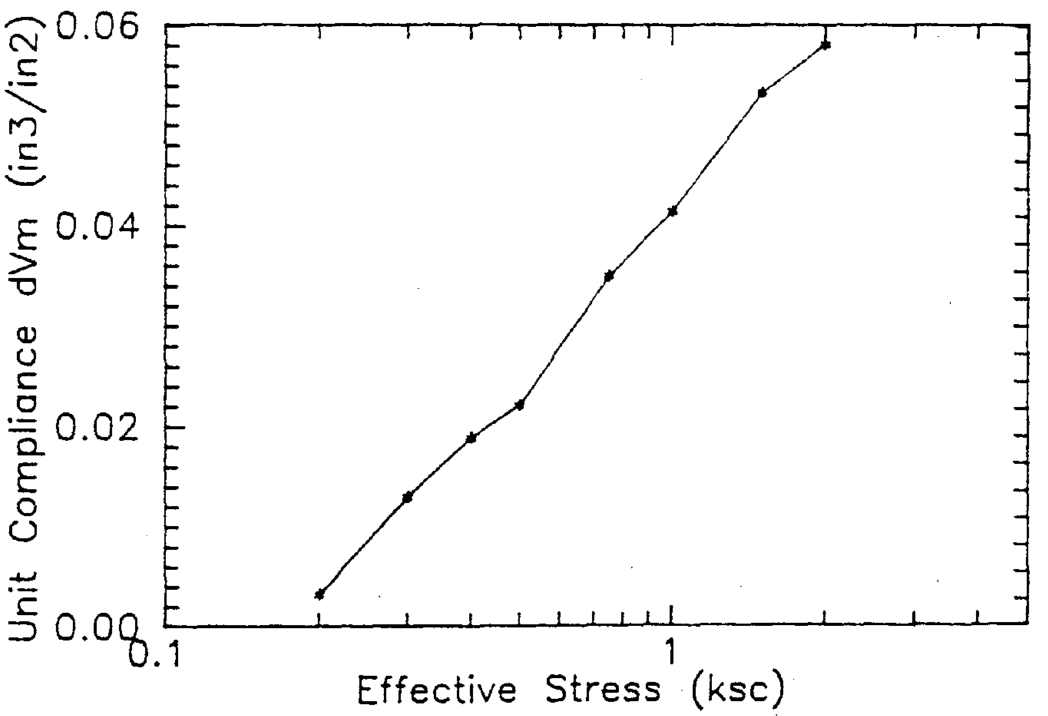


Figure 5.35b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 10

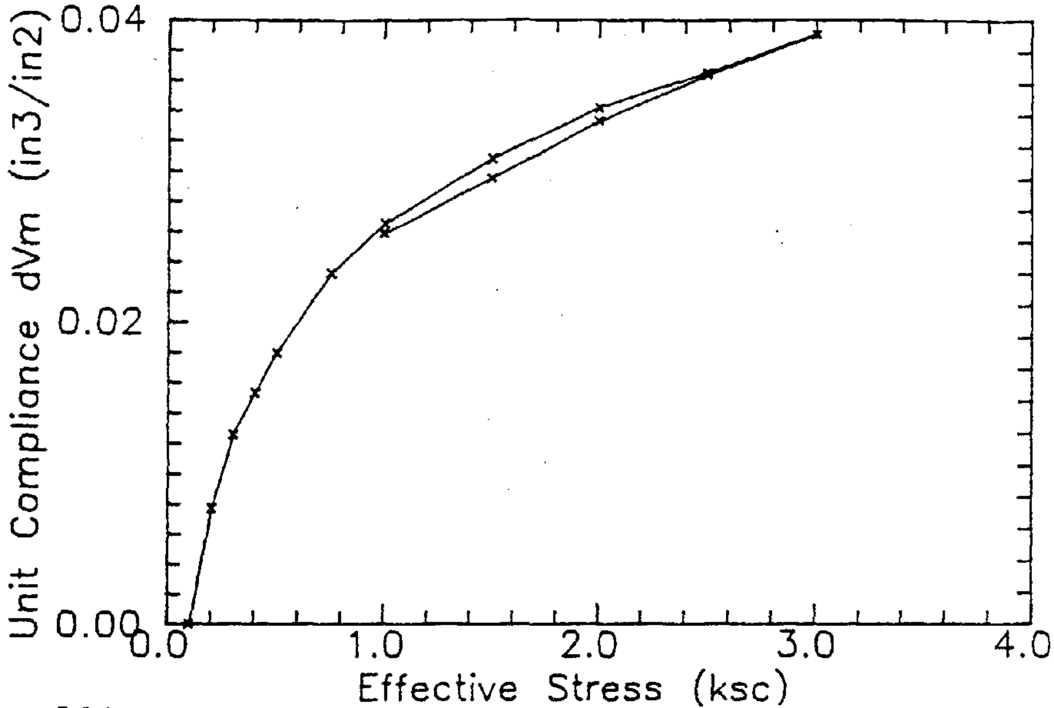


Figure 5.36a: Unit Membrane Compliance vs. Effective Confining Stress: Material 11

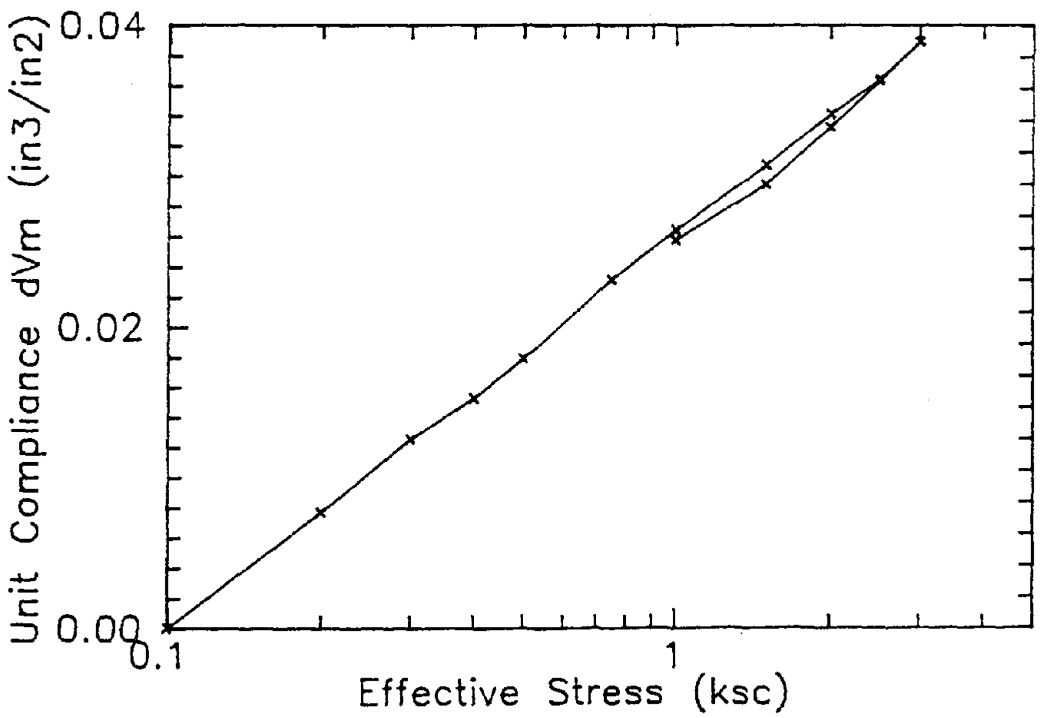


Figure 5.36b: Unit Membrane Compliance vs. Log Effective Confining Stress: Material 11

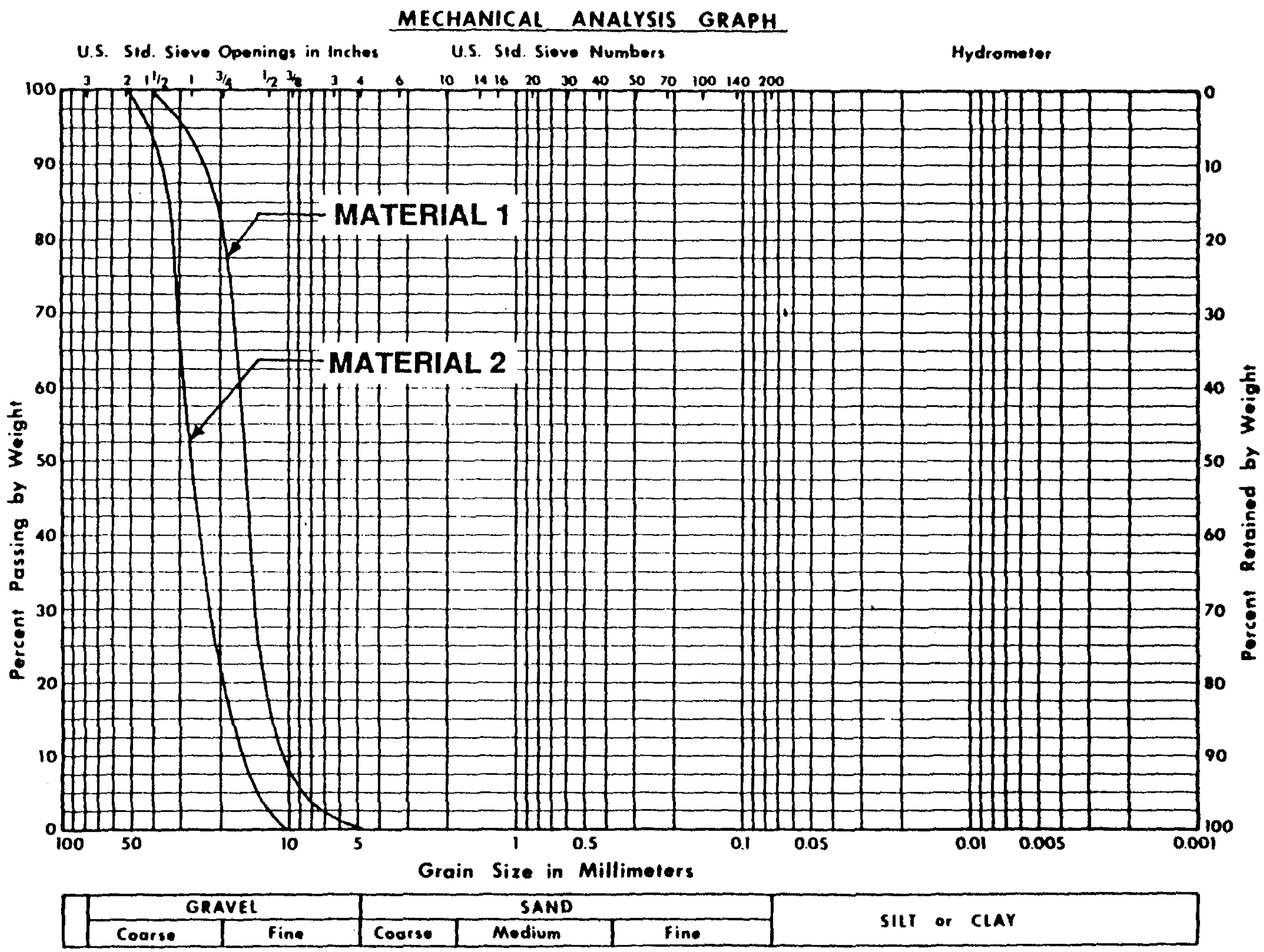


Figure 5.37: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

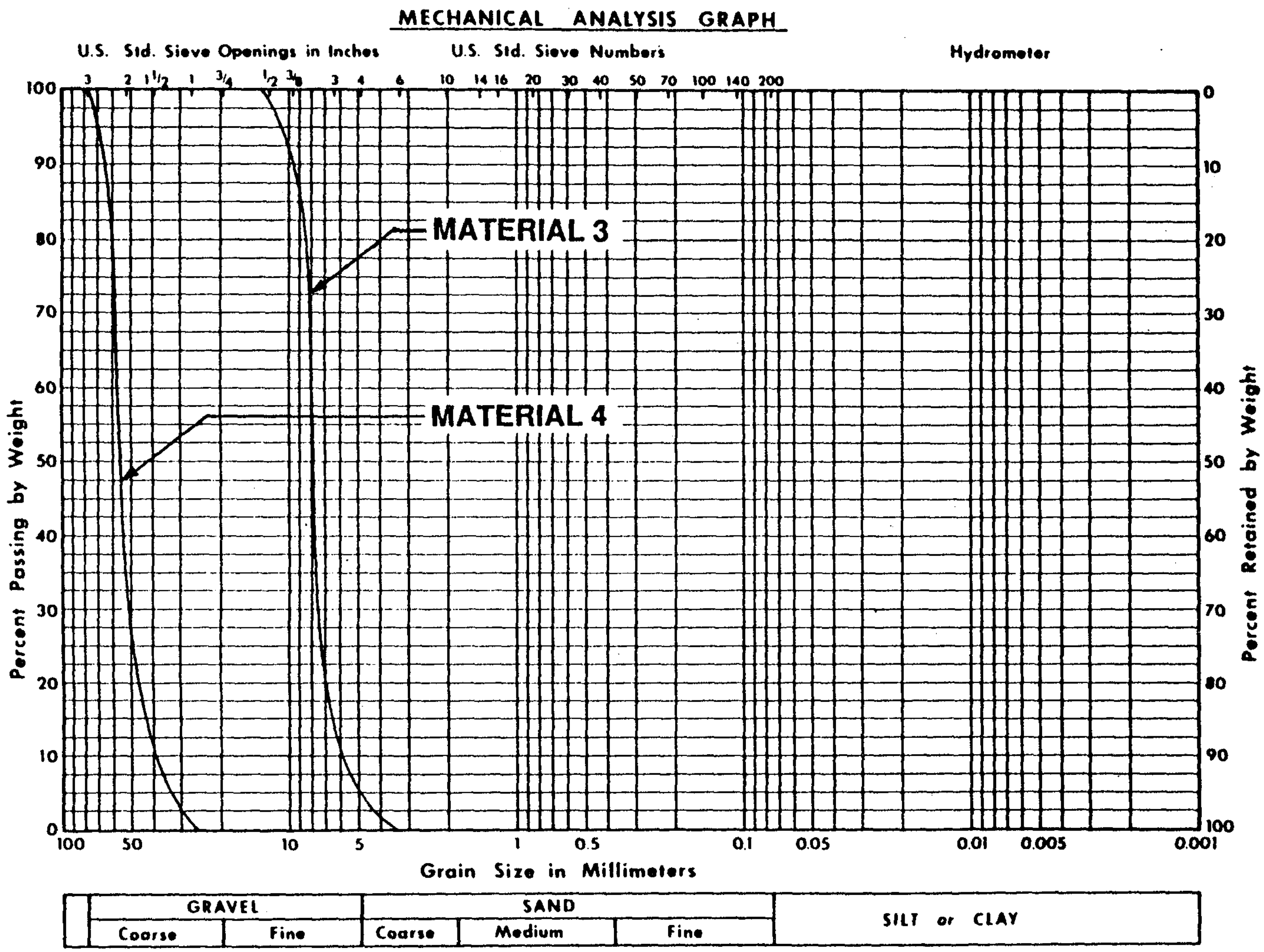


Figure 5.38: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

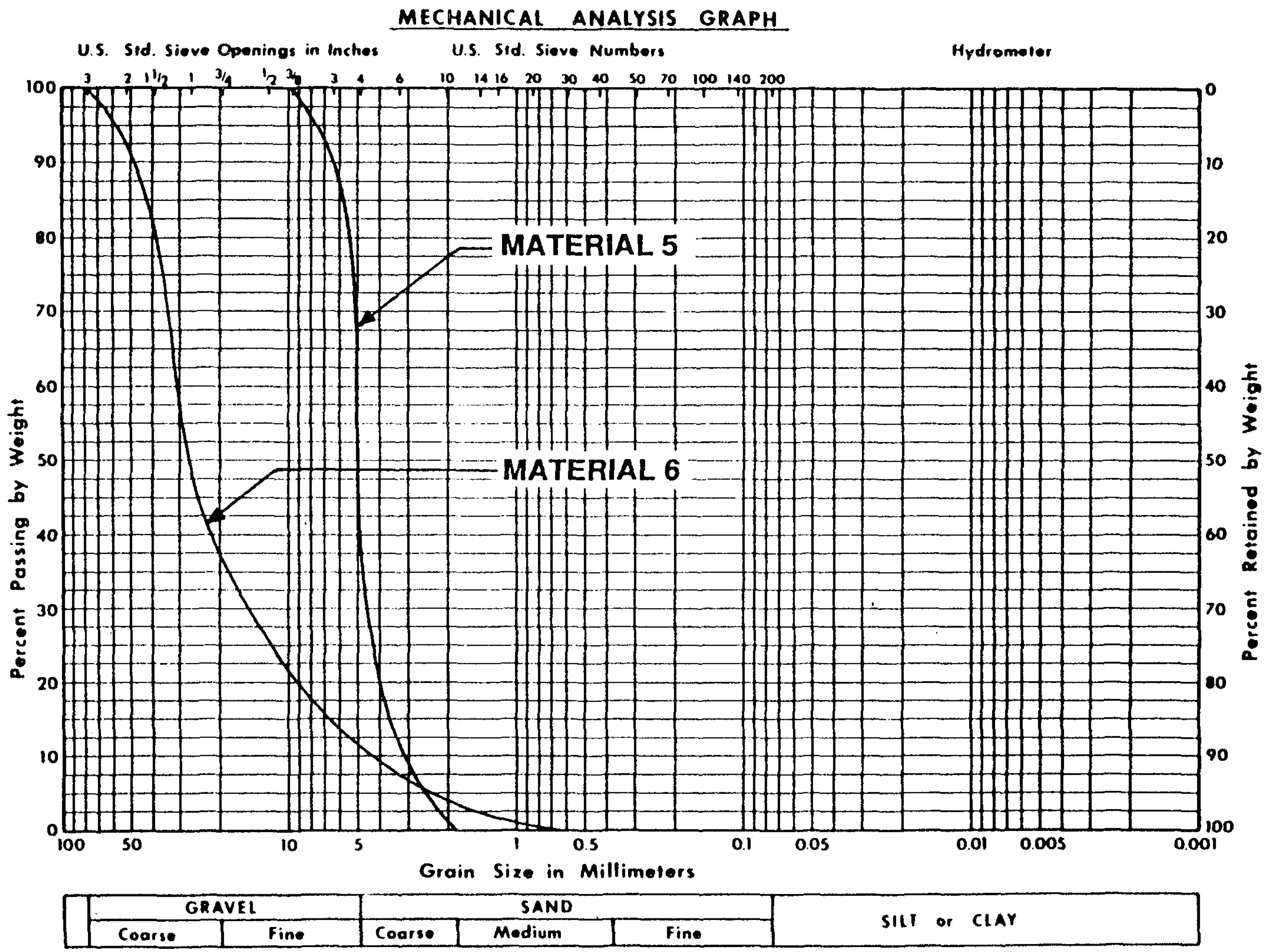


Figure 5.39: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

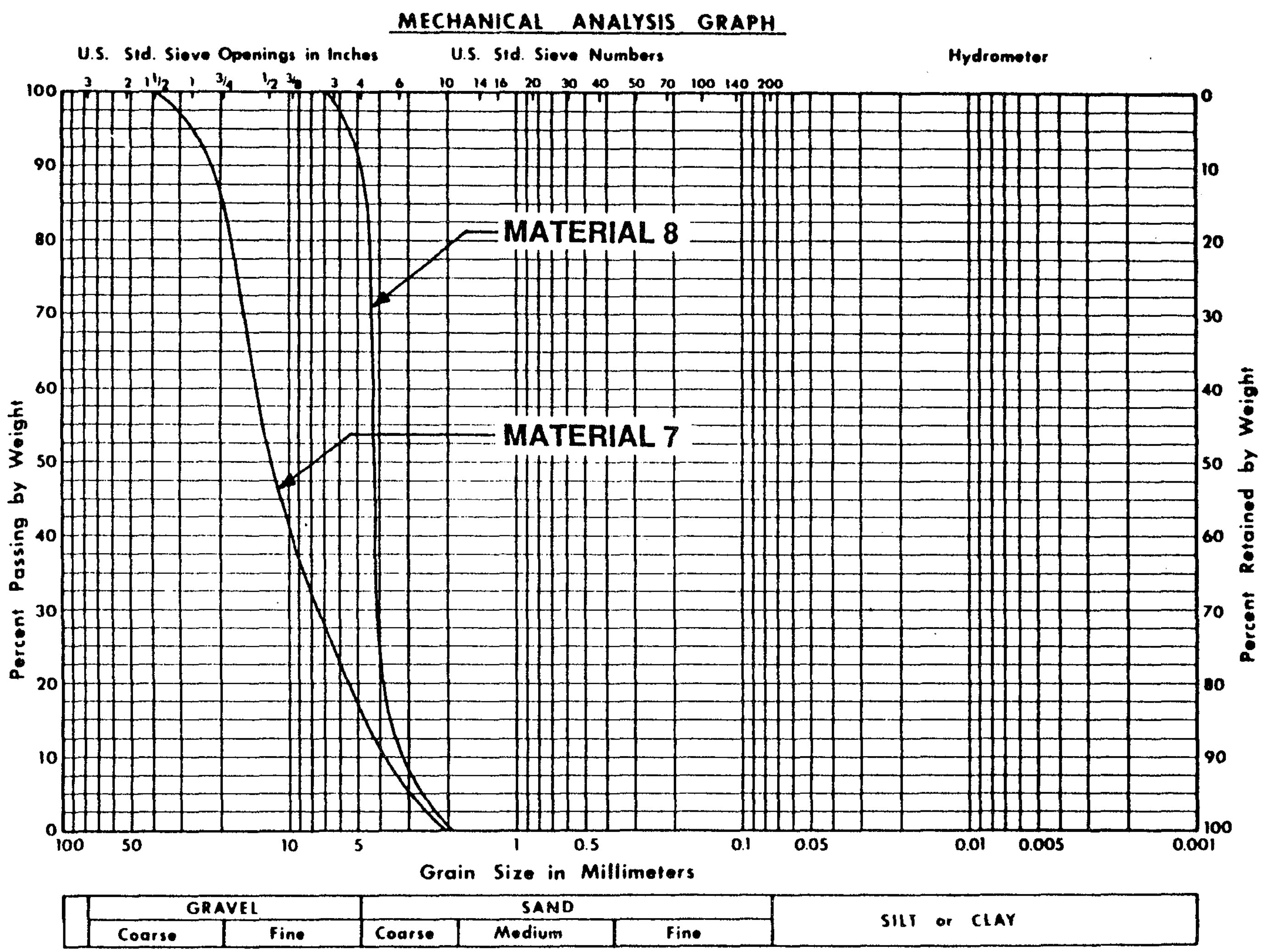


Figure 5.40: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

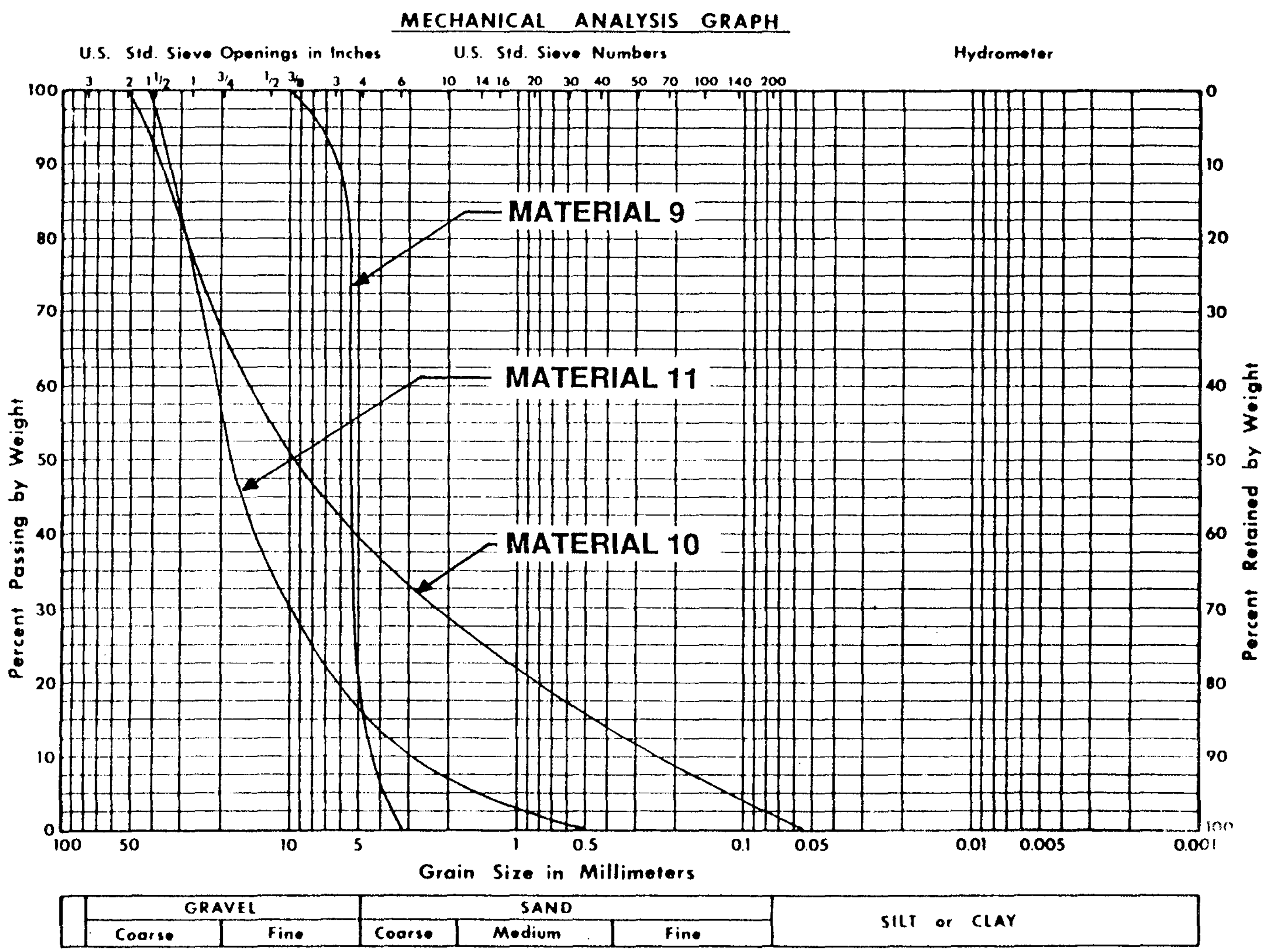


Figure 5.41: Gradations of Soils Tested For Membrane Compliance as Part of These Studies

Table 5.3: Gradation and Membrane Compliance Characteristics of Soils Tested by Selected Alternate Investigators

Soil Name	USCS Classification	D_{10} (mm)	D_{20} (mm)	D_{50} (mm)	S (cm/$\Delta \log \sigma'_3$)
Kiekbusch A	SP	0.3	0.33	0.40	0.0045
Kiekbusch B	SP	0.1	0.14	0.18	0.002
Kiekbusch C	SP	0.07	0.08	0.09	0.0014
Frydman A	SP	0.163	0.17	0.18	0.0027
Frydman B	SP	1.68	1.70	1.85	0.0165
Frydman C	SP	.283	0.29	0.30	0.0045
Steinbach 1	SP	.22	0.23	0.30	0.0035
Steinbach 2	SP	.45	0.51	0.60	0.009
Steinbach 3	SP	.84	1.05	1.50	0.0147
Siddiqi A	GW	.13	0.40	3.8	0.0053
Siddiqi B	SW-SM	.074	0.21	1.4	0.0041
Siddiqi C	SP-SM	.40	0.80	4.0	0.014
Evans	GP	5.5	6.1	6.5	0.056
Hynes	GM	.40	6.0	20.0	0.039

Note: S = Normalized unit membrane penetration; the volumetric change (cc.) due to membrane penetration per square cm. of membrane area per logarithmic cycle of change in effective confining stress (σ'_3)

References:

1. Keikbusch, M. and Schuppener, B. (1977)
2. Frydman, S., Zeitlen, J. G. and Alpan, I. (1973)
3. Steinbach, J. (1967)
4. Siddiqi, F. H., Seed, R. B., Chan, C. K., Seed, H. B. and Pyke, R. M. (1987)
5. Evans, M. D. and Seed, H. B. (1987)
6. Hynes, M. E. (1988)

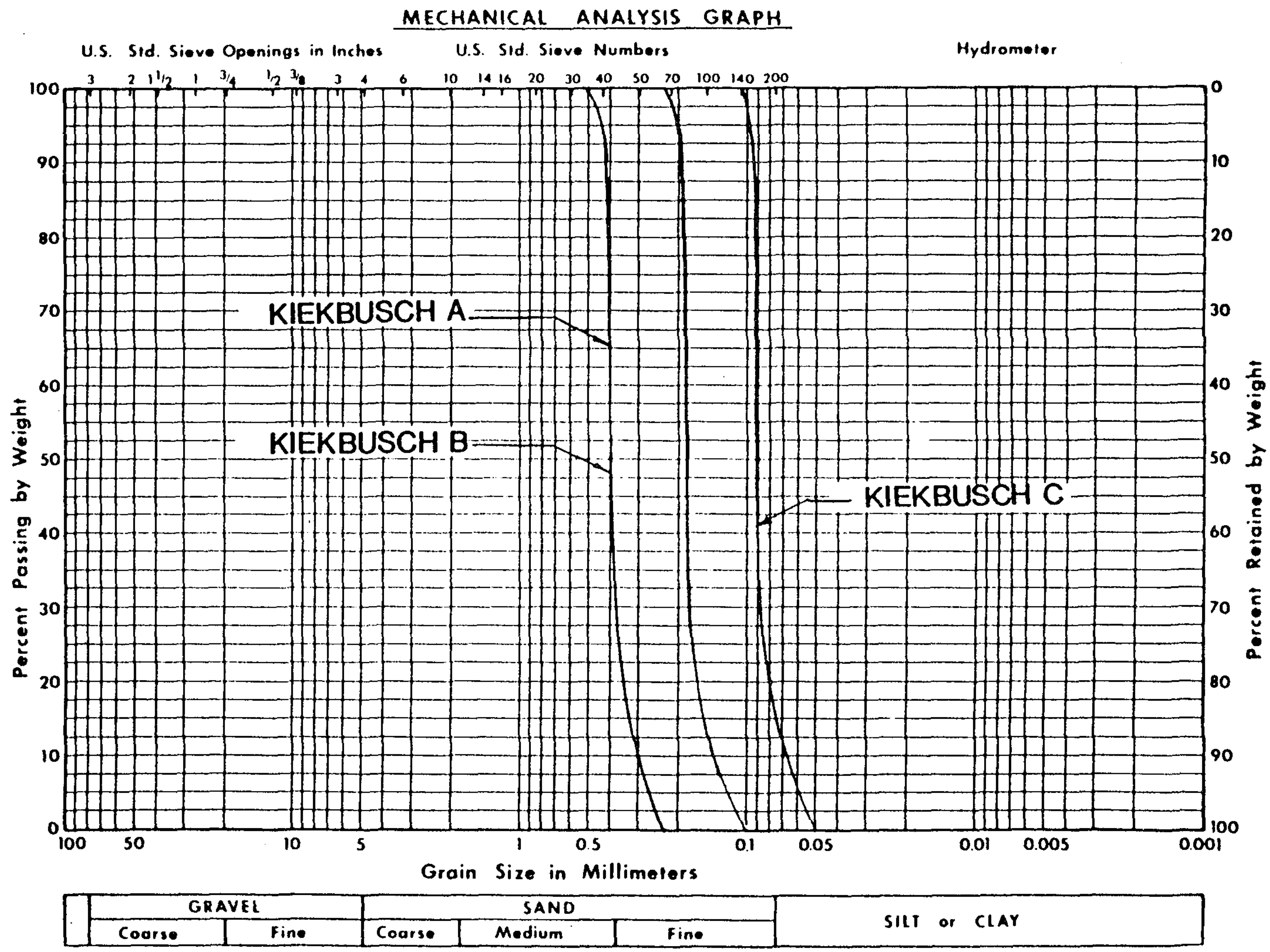


Figure 5.42: Gradations of Soils Tested for Membrane Compliance by Kiekbusch and Shuppener (1977)

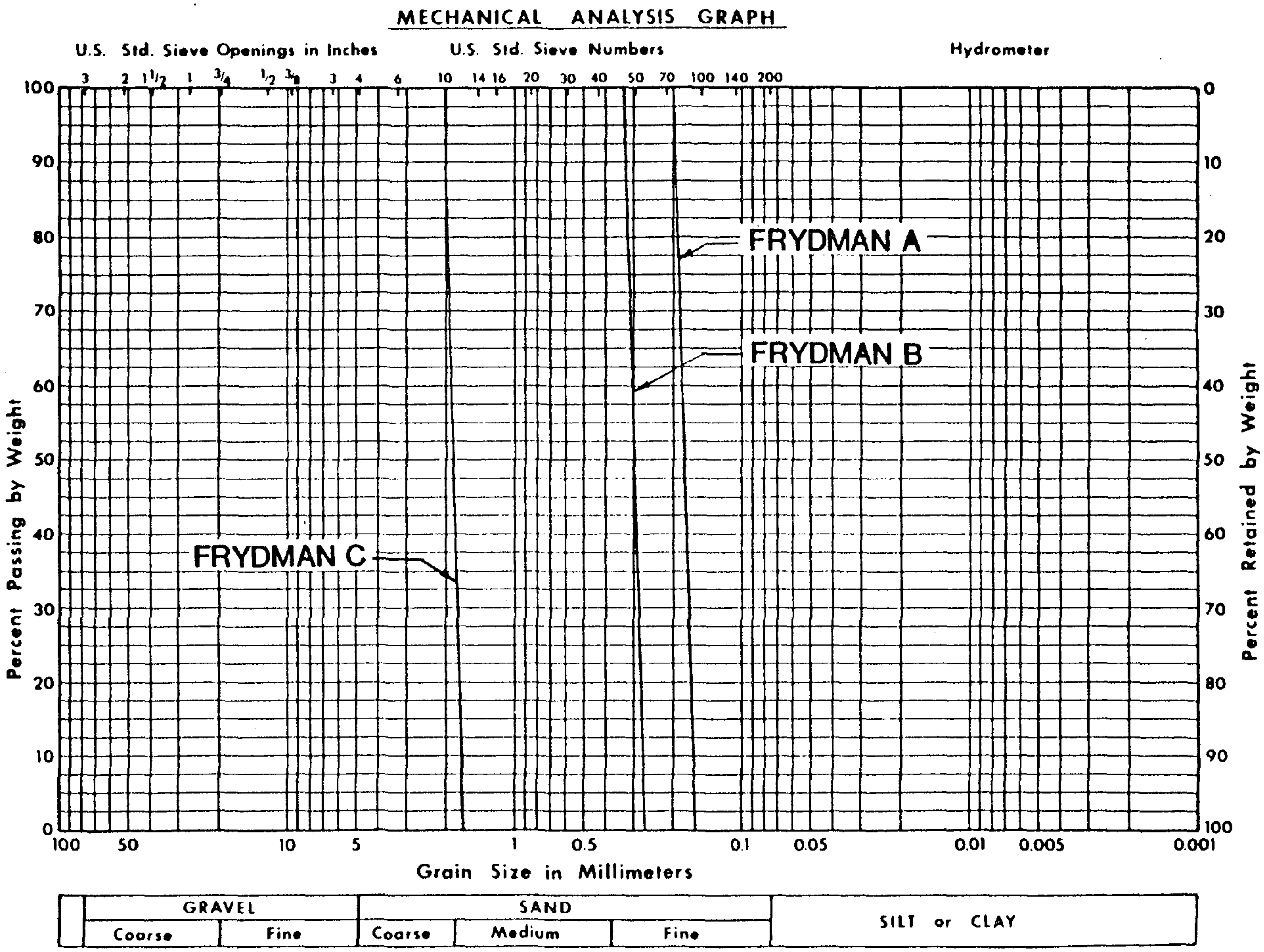


Figure 5.43: Gradations of Soils Tested for Membrane Compliance by Frydman et al. (1973)

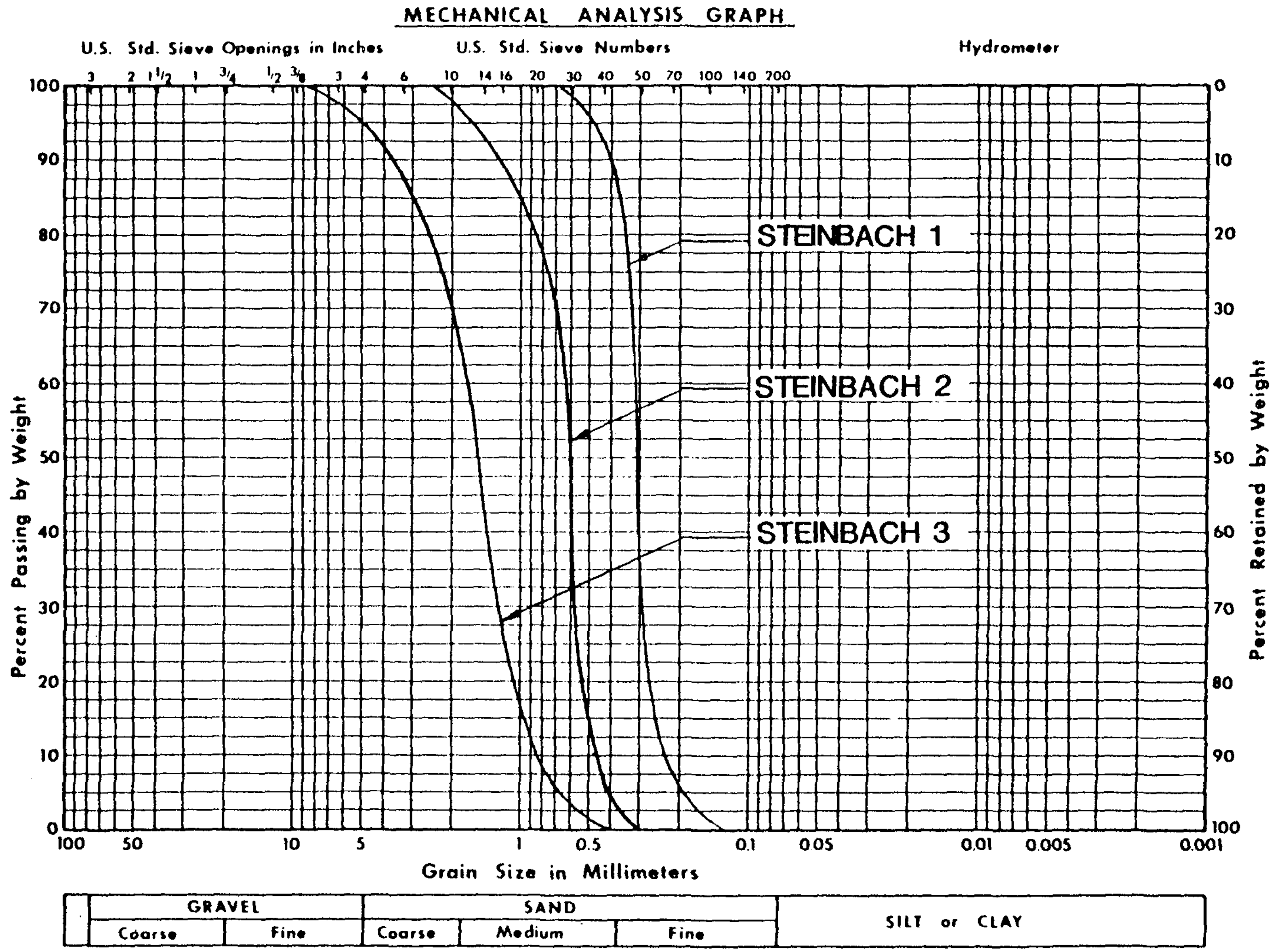


Figure 5.44: Gradations of Soils Tested for Membrane Compliance by Steinbach (1977)

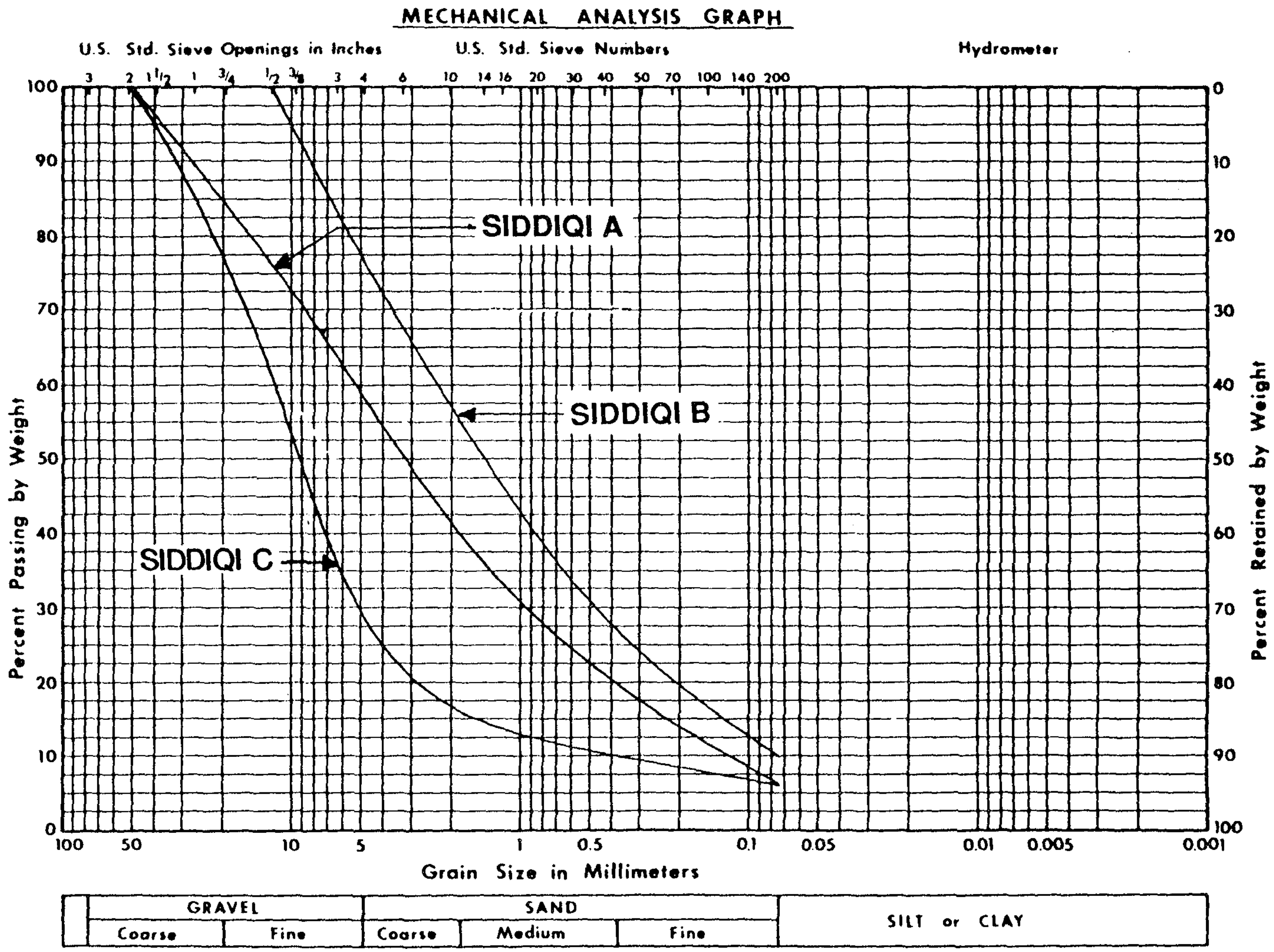


Figure 5.45: Gradations of Soils Tested for Membrane Compliance by Siddiqi (1984)

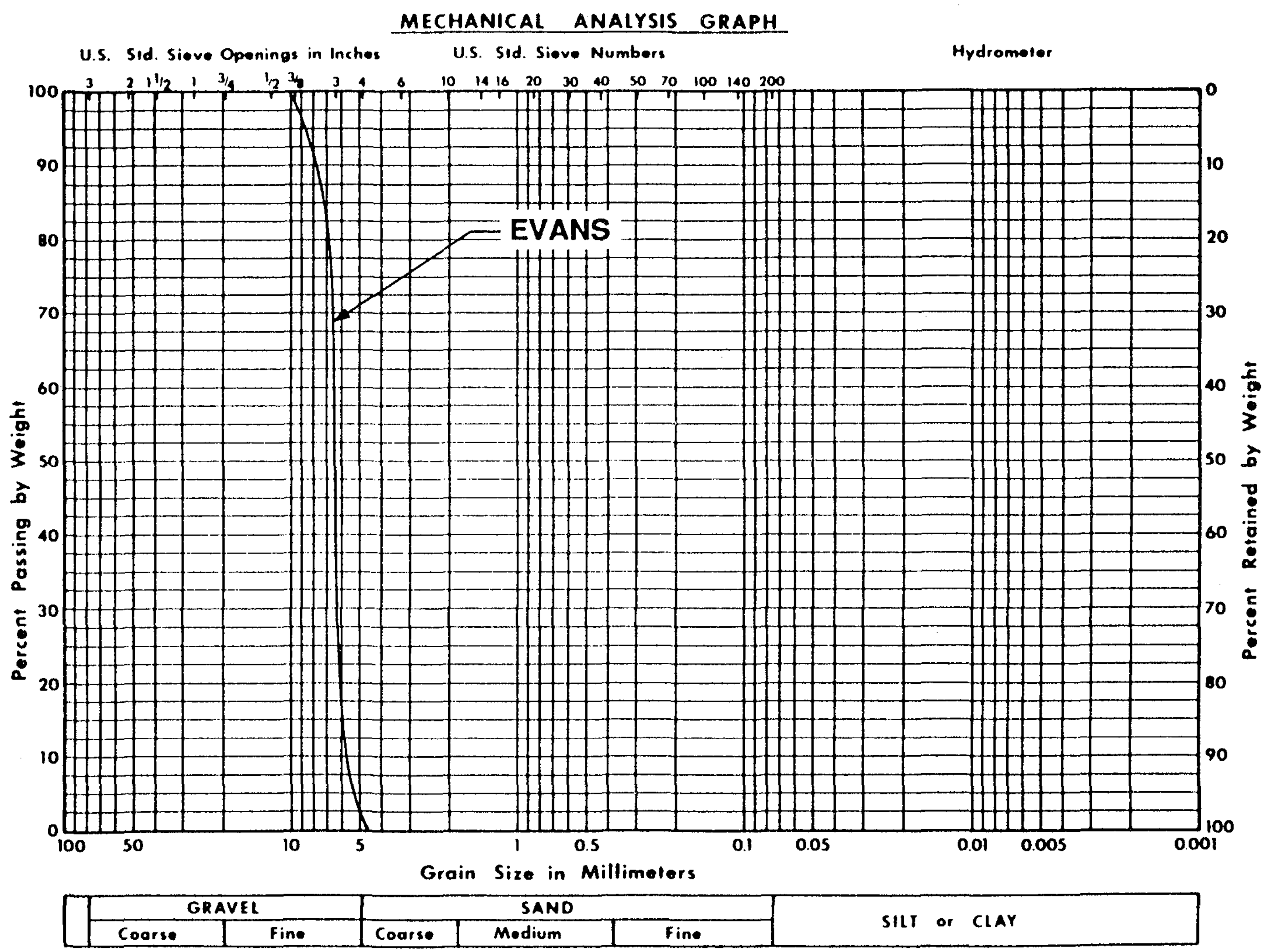


Figure 5.46: Gradations of Soils Tested for Membrane Compliance by Evans (1987)

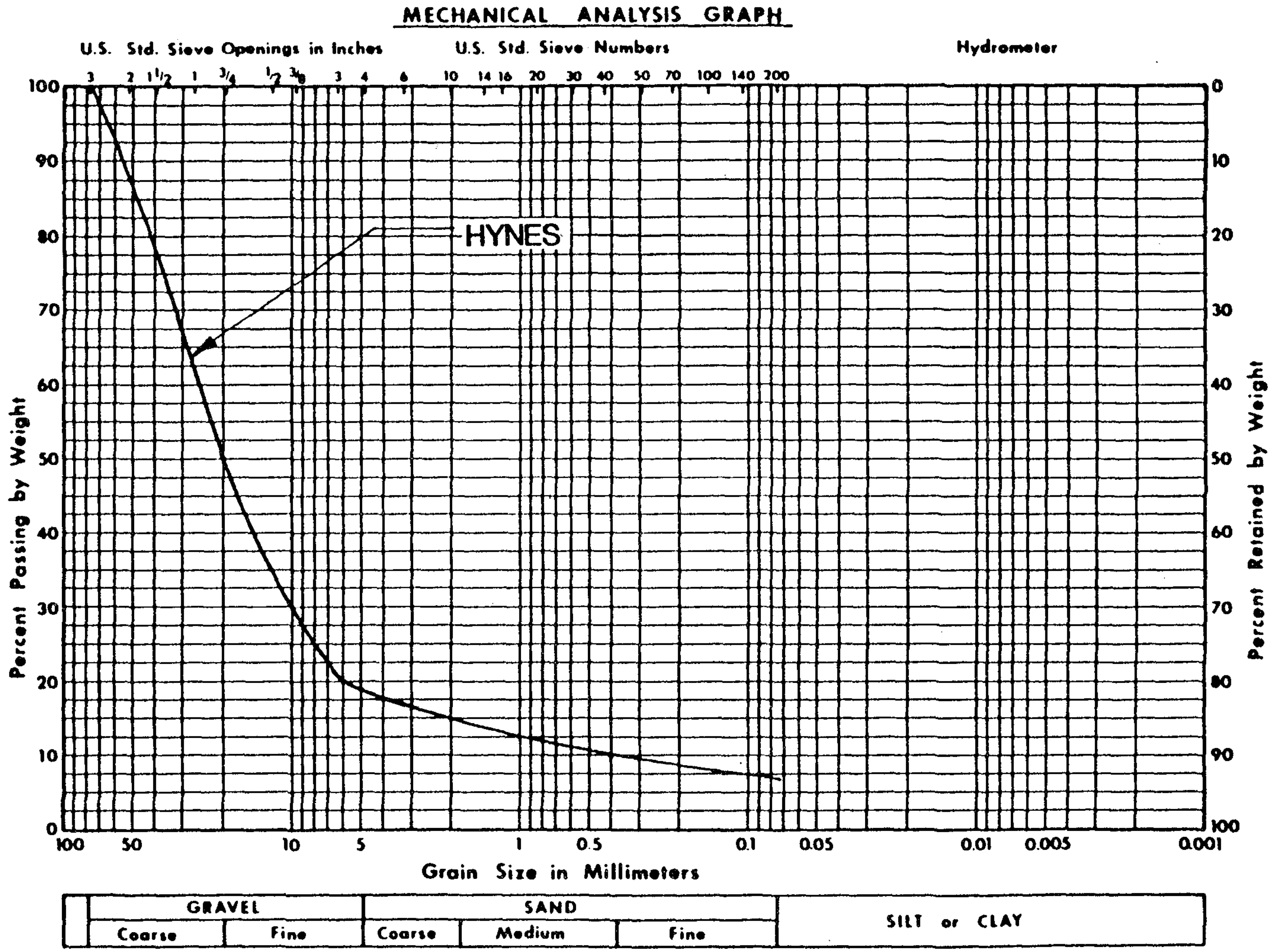


Figure 5.47: Gradation of Soil Tested for Membrane Compliance by Hynes (1988)

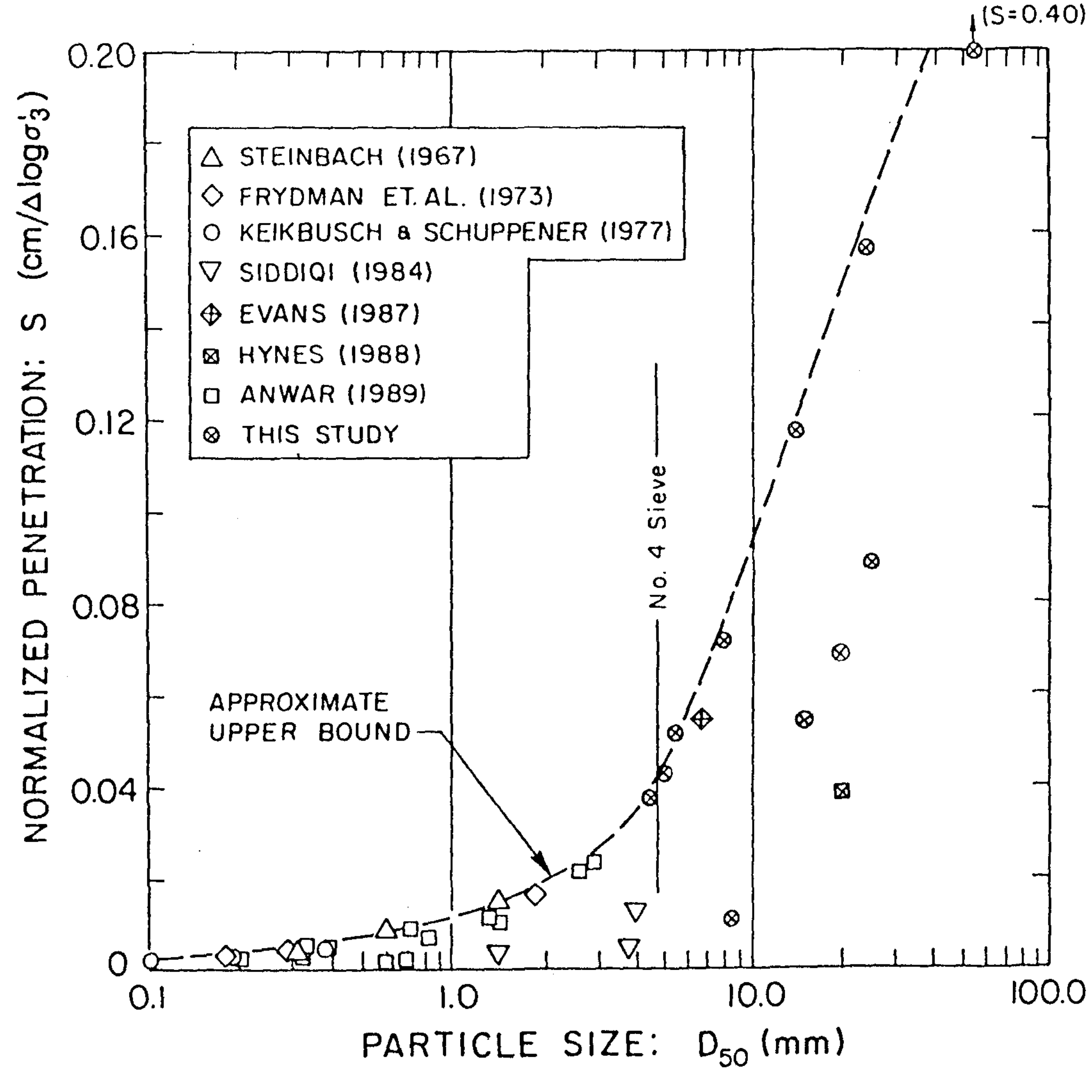


Figure 5.48: Relationship Between Normalized Unit Membrane Penetration (S) and D_{50}

primarily due to the more broadly graded soils which exhibit considerably less penetration than uniformly graded soils due to "finer" particles partially filling the peripheral sample voids between the larger soil grains. A number of particle size indices finer than D_{50} were examined in order to find that which gave the best correlation for all of the available data. It was found from this investigation that D_{20} was the one particle size index with which the membrane penetration characteristics of all different soil types could be best correlated. A composite plot is given in Figure 5.49 showing the relationship between normalized compliance S and D_{20} for all of the same data points that were included in Figure 5.48. This plot shows that by using D_{20} as the "representative" grain size for this wide range of soil types and sizes, a very narrowly banded correlation results demonstrating the significant improvement which can be achieved using the D_{20} grain size for this relationship. The number of data points that have been added to the existing database now gives a clear picture of how the compliance relationship (S vs. D_{20}) should look for soils whose representative grain sizes fall between coarse sand and medium to coarse gravel. This new data fills in the large range of uncertainties which had previously existed.

5.4 Development of a Correlation Between Compliance Characteristics and Material Gradation

An important part of this study was an effort to define a relationship between the normalized unit membrane penetration and material particle sizes for materials whose grain size distributions extended beyond the well documented sand sizes. As previously reported by Seed and Anwar (1986) and Baldi and Nova (1984), the relationship between normalized unit membrane compliance and "representative" grain size does not appear to be strictly log-linear as had been

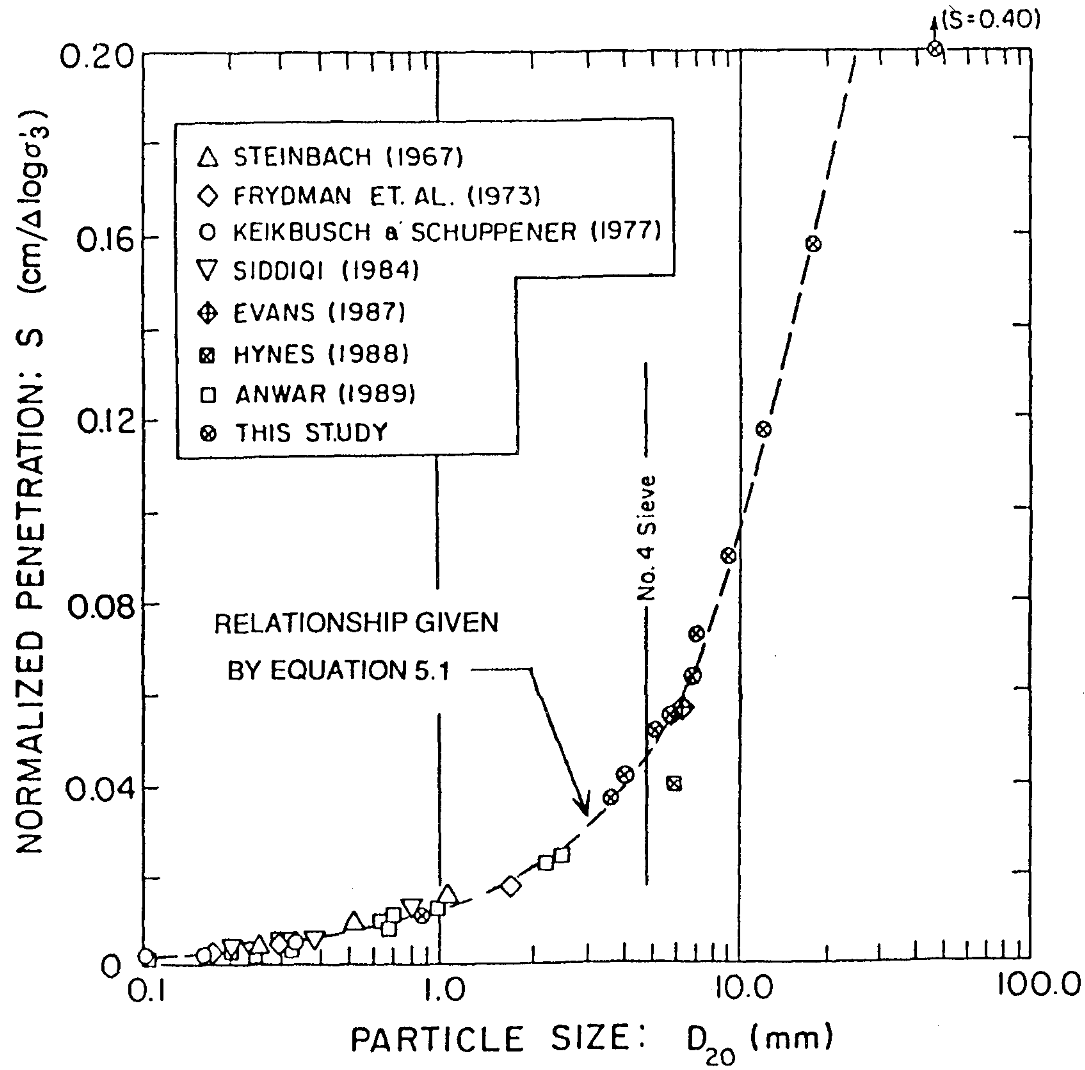


Figure 5.49: Relationship Between Normalized Unit Membrane Penetration (S) and D20

speculated by several earlier investigators, but may be better represented by a curve or multi-log-linear shape.

Seed and Anwar (1986) addressed the question of what part of the gradation curve should be used as a representative grain size for compliance measurement relationships. Historically researchers have always used the median grain size of a material (D_{50}) as the basis for compliance volume relationships. Investigations initialized by Seed and Anwar (1986) and continued as a part of this study have showed that the compliance relationship correlates much better using a finer point of the gradation curves. The particle size indice D_{20} was found to provide the best correlation between material particle size and normalized unit compliance S. Incorporating the available data from previous compliance measurements with those made as a part of this study, an empirical relationship was developed for the database using a polynomial line fitting routine. For all of the data examined, the equation that gave the best "smooth" curve through the data for D_{20} vs. S is:

$$S = 2.029^{3} + 9.248^{3X} - 1.4126\ ^{5X^2} \qquad \text{[Eq. 5.1]}$$

where:

S = Normalized unit compliance; membrane compliance induced volume change per unit area of membrane per log-cycle change in effective confining stress: (cc/cm^2) per log cycle change in σ_3', and

X = D_{20} grain size of a soil in mm.

The curve representing this equation is plotted on Figure 5.49. By using this equation, one would need to know only the gradation of the material in order to make a reasonably accurate estimate of the normalized membrane compliance for any soil. It should be noted that for samples at very high or low relative densities appropriate (though relatively minor) adjustments may have to be made to any estimate made by this relationship. Another possible exception may also be for cases where soils contain an extremely broad gradational "tail" of material finer than D_{20}, as identified by a long tail of the gradation curve. For such soils, normalized unit compliance can be less than that predicted by Eq. 5.1.

CHAPTER 6

IMPLEMENTATION OF A MEMBRANE COMPLIANCE MITIGATION SYSTEM FOR TESTING OF COARSE GRAVELLY SOILS

6.1 Implementation of a Compliance Mitigation System

The technique behind the compliance mitigation method used in this study, originally proposed and tested by Raju and Venkataramana (1980) and Ramana and Raju (1981,1982), is to offset any volume error encountered in undrained tests by injecting to (or removing from) the sample, a volume of water equal to the compliance induced volumetric error. This has been attempted on small scale (2.8-inch diameter) samples in a number of ways. Ramana and Raju attempted this by manual injection using a manually operated system and injecting water at stepwise intervals during tests. Seed and Anwar (1986) used a computer-controlled digital motor driven injector to continuously inject/remove water. Tokimatsu and Nakamura (1986) used a computer-controlled system that incorporated a pneumatic air-pressure system to inject/remove water for these "small scale" tests. The manual system proved to be too slow to be useful, but both of the computer-controlled systems showed promise, and the efficiency and accuracy of the Seed and Anwar system was verified by the tests described in Section 4.2.

A number of attempts were made to control injection with a servo-controlled air pressure system for large-scale samples, but it was found that the air pressure type of system used by Tokimatsu and Nakamura could not efficiently control the relatively large volumes of water that would be necessary to compensate for the membrane compliance errors that would be expected for coarse gravelly soils with sufficient accuracy as to preclude introducing deleterious secondary problems. Of the types of injection methods previously implemented, only the Seed and Anwar type of system appeared to give verifiably continuous and sufficiently accurate (precise) test results utilizing a system suitably adaptable for use with larger scale tests of gravelly materials. The "correct" test results reported by Seed and Anwar using their compliance compensation system were verified by a series of tests performed on the same material using 12-inch diameter samples which exhibited negligible compliance errors for that material, as described previously in Section 4.2.

Accordingly, a large-scale hydraulic piston injector that could be accurately and efficiently controlled by a digital computer program was designed and constructed. The components and implementation of that system is outlined in the following sections.

6.1.1 Components of the Injection-Correction System for Testing of Large-Scale Samples of Coarse Material

6.1.1.1 Compliance Mitigation System Hardware

Figure 6.1 shows a photograph of the computer-controlled injection/removal system that was developed for mitigation of membrane compliance effects during undrained large-scale triaxial testing. A schematic of the injection set-up is shown in Figure 6.2. In addition to the IBM PC-AT microcomputer described previously, the injection/removal system consisted of a series of two hydraulic cylinders mounted on a stiff aluminum channel, so that no relative movement would be permitted between the two. One of the two pistons was driven by a single computer-controlled servo valve. The piston rods of the two cylinders were securely fastened to each other with a threaded compression collar.

The controlling cylinder was a double-acting 1.5-inch I.D. 24-inch stroke heavy duty hydraulic cylinder filled with hydraulic oil under a pressure of

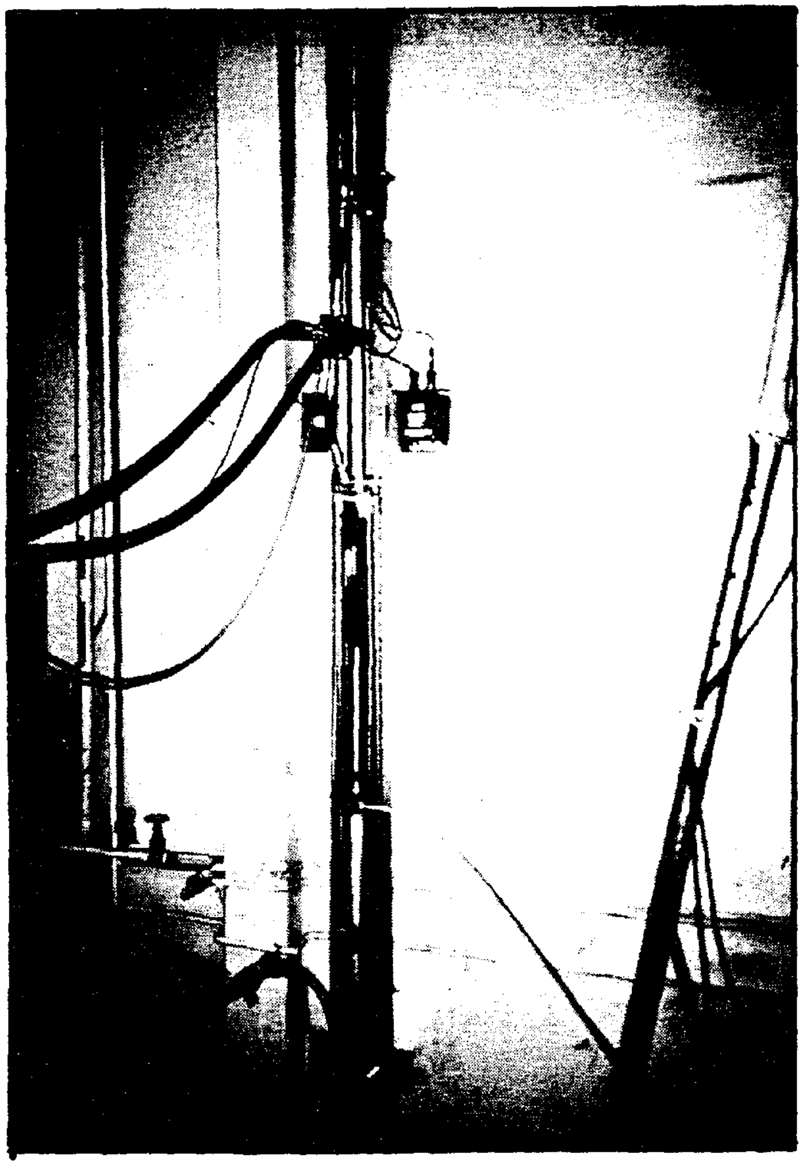


Figure 6.1: Large-Scale Computer-Controlled Injection/Removal System

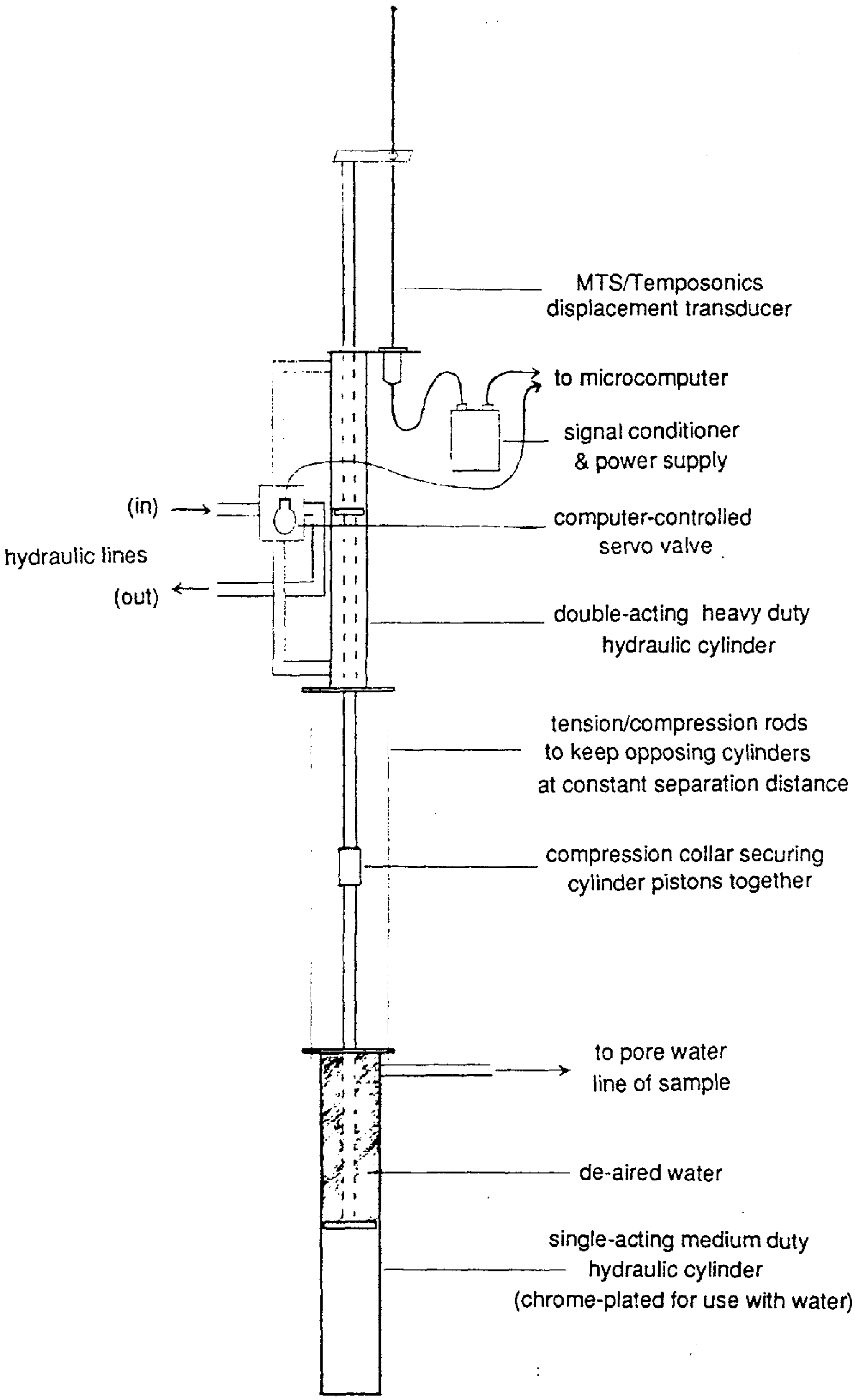


Figure 6.2: Schematic of Large-Scale Injection Set-Up

approximately 2700 psi whose displacement was controlled by the computer-controlled dynamic servo-valve. The motion of the first piston commanded an equal displacement of the piston of the second cylinder, which was a single-acting 2-inch I.D. 24-inch stroke medium duty hydraulic cylinder specially modified with interior chrome plating for use with water. One side of the second cylinder was filled with de-aired water under a pressure equal to that of the back-pressure in the sample at the initiation of the test, and that side of the cylinder holding the water was connected to the pore pressure lines of the sample. Displacements of the cylinder pistons were measured by means of a 24-inch MTS/Temposonics displacement transducer attached to one end of the cylinder configuration with an accuracy of ± 0.004 inches. Injected/removed water volumes were calculated by means of a calibration of the linear displacement of the cylinder pistons with corresponding volumes of water held by the water cylinder. Controlled increments of water volumes could then be injected/removed to/from the sample by allowing specified displacements of the cylinder pistons. The resulting accuracy of the system was found to be ± 0.40 in^3, representing approximately 0.015% of the total soil sample volume tested.

This system requires only a single connection to the sample pore water lines of a triaxial testing system, and could therefore be relatively easily adapted to any existing large scale triaxial system.

6.1.1.2 Compliance Mitigation Program Software

The program implemented to offset the effects of the compliance induced volumetric errors was formulated to run as a part of the ATS system. During the course of this research the "injection program", as it became to be known, was modified a number of times until it satisfied all of the requirements that were demanded of it for a variety of different testing situations. It was important that one single program could be used for all encounterable situations and that it be generic enough to be applied to any different configuration of sample sizes and material particle sizes.

A simplified description of the compliance mitigation algorithm of the final version of the program is presented here. At the initiation of the test, readings are recorded for the current effective confining stress and the position of the injection system cylinder pistons. After a short specified time interval, additional readings of effective stress are taken. The new readings of effective stress are then compared to the previously recorded readings. For any change in effective confining stress, the program calculates the volumetric error induced by membrane compliance based on pre-determined values for the material being tested and the sample geometry, and commands the servo-controlled injection cylinder pistons to move accordingly. The loop continues until termination of the test.

There are a number of variable test parameters that are required by the program in order to be able to properly calculate the volumetric errors to be offset. Among these are the S-value or normalized compliance (volumetric error per unit area of membrane per log cycle change in effective confining stress), sample geometry given in appropriately corresponding units, and the minimum effective stress below which no further injection-corrections will be implemented. This last parameter is necessary to adjust for the non-linearity of the semi-log plot of unit compliance as a function of effective confining stress at very low levels of effective stress. Two additional optional testing parameters were also included in the program to overcome possible irregularities and/or potential problems that might be encountered for different materials or testing equipment. These included: (a) an option to change the rate of injection (time interval between runs of the injection control loop), and (b) a specified percentage of the calculated volumetric error to be injected at one time. The objective of these variables was to protect against sudden pressure surges that might be induced to the system from rapid injection of large amounts of water, thereby leading to overcompensation by the injection system. By slowing the rate of the injections, internal pressures were allowed to equilibrate before the next reading of effective stress was taken. Injecting only a portion of the calculated water volume prevented any over-injection at a single point in the test. The full amount or "correct" volume offset was converged to quickly within the next few iterations. Only very subtle uses of these optional variables is advised so as to maintain "complete and continuous" corrections. With some practice, a careful balance between these two optional variables would "smooth" out the injections without sacrificing the accuracy of the corrections.

One further addition to the software that was written into the program was to average a number of very rapid effective stress readings to "smooth" or eliminate any values of spikes or valleys from the pore-pressure transducer readings which would otherwise result from water-hammer pulses caused by rapid injection surges into a closed test system.

6.1.2 Injection System Specifications

The accuracy to which the system could inject or remove volumes of water to/from the sample, was to within 2cc. The maximum rate at which accurately controlled injections could be performed was on the order of 100cc per 0.5sec. The speed of the injection software loop could be run as fast as once every 3 milliseconds. It was this high degree of accuracy, and the rapid correction loop speed possibilities, that enabled the system to be able to completely and continuously mitigate the effects of membrane compliance throughout undrained testing.

6.2 Tests Performed on 12 Inch Diameter Samples

6.2.1 General

The results of the tests performed on 12-inch diameter specimens as a part of this research program are presented in this chapter. Test data is categorized into groups depending on test type (e.g., monotonic or cyclic loading), and whether or not the injection-correction system was used. Four series of tests were performed: strain-controlled undrained monotonic triaxial load tests performed with and without the effects of membrane compliance mitigated; and stress-controlled undrained cyclic triaxial tests preformed with and without the effects of membrane compliance eliminated by the injection-correction system.

6.2.2 Material Tested

The material used for the comparative tests in this study was a uniformly-graded medium gravel whose grain size distribution is shown in Figure 6.3. A photograph of this material is also shown in Figure 6.4. This material (termed PT-Gravel) was chosen for a number of reasons including its availability and because its characteristic properties were such that some anticipated problems might be avoided. The grains were typically sub-angular to sub-rounded, which was helpful in avoiding tearing of the confining membranes during compaction or testing, and allowed ease in identifying whether or not individual grains were broken during sample preparation or testing. The maximum particle size of this material (1.5 inches) fell well within the limits recommended for testing of these sample dimensions (12-inch dia.), where it has been recommended that a ratio of specimen diameter to maximum grain size be no greater than 6:1 for uniformly graded soils to avoid stress concentration problems that might otherwise invalidate test results. The gradation was coarse enough to provided large enough amounts of volumetric compliance so that compliance effects would significantly affect undrained test

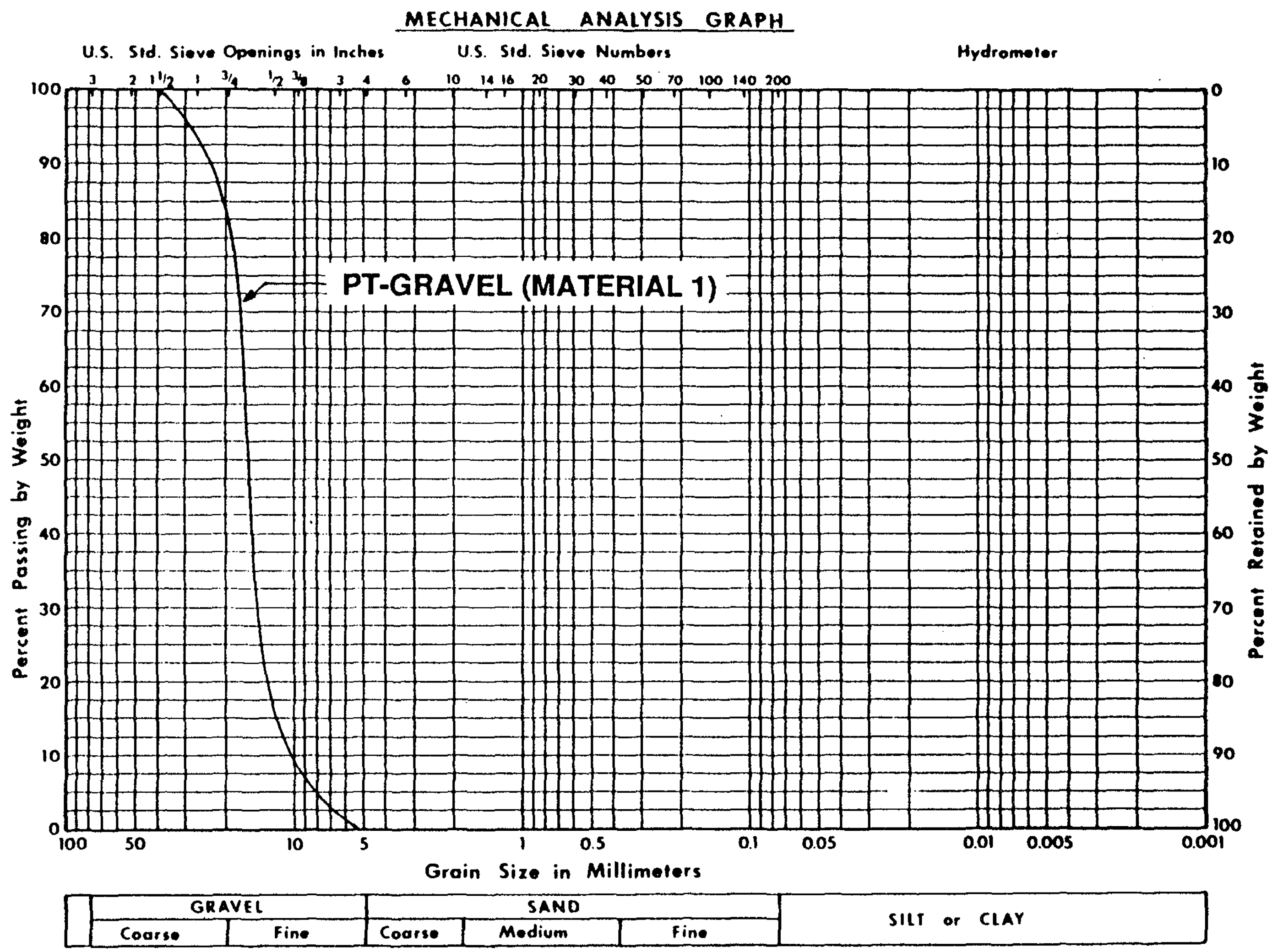


Figure 6.3: Gradation for PT-Gravel Used for Comparative Large-Scale Tests

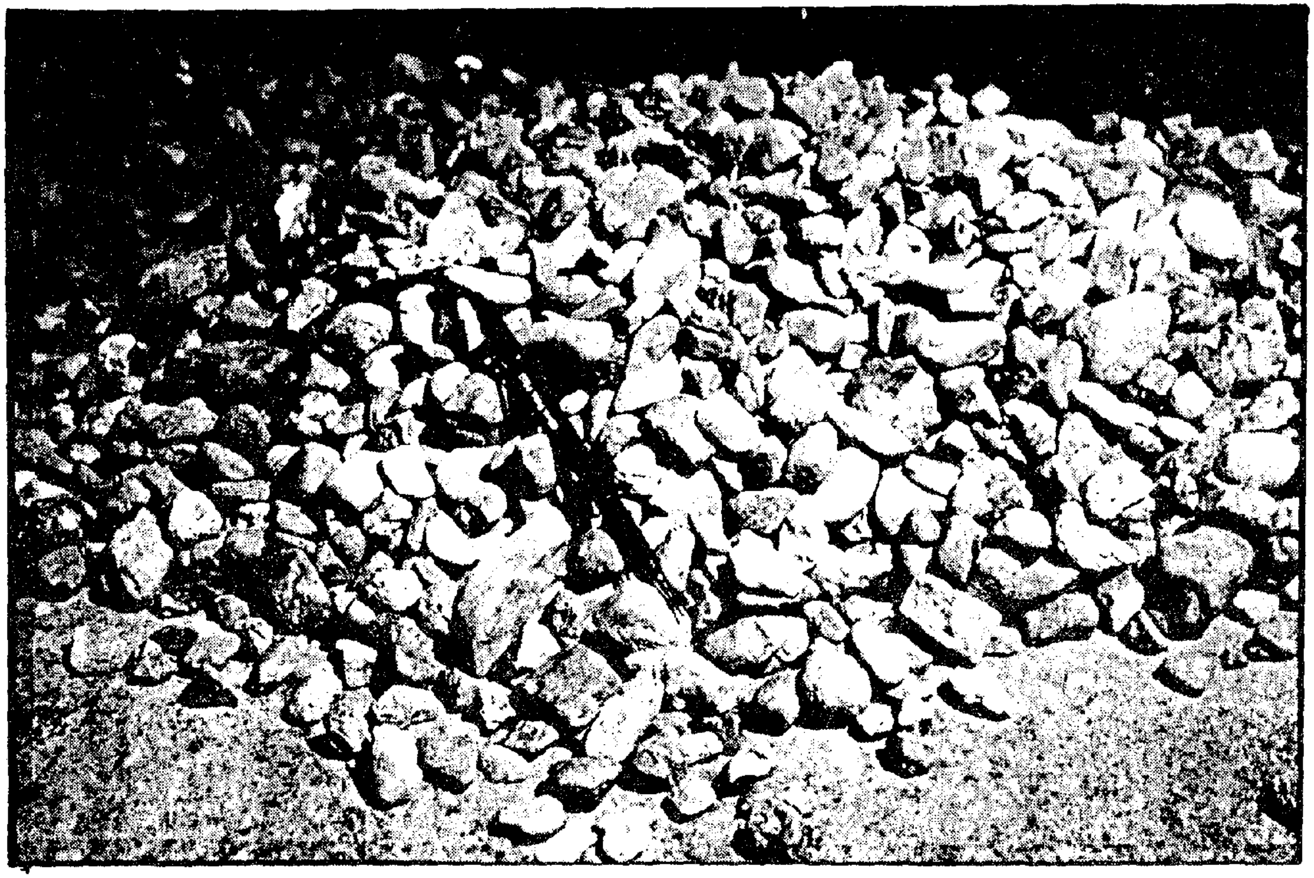


Figure 6.4: Photograph of PT-Gravel

results, and to assure that membrane thickness would be insignificant in affecting membrane compliance. The normalized compliance, or S-value of this material was approximately 0.117 cc/cm^2 per log cycle change in effective confining stress, which was in the middle of the range of the gravels for which compliance measurements were made, and also towards the middle of the range of gravel sizes best suited for the large scale testing apparatus. This amount of volumetric compliance was also large enough so that a high degree of accuracy could be maintained in compliance measurements and water volumes to be injected. Because of these reasons, the material was chosen for these tests as it seemed to be an average, representative material for testing in this size equipment.

6.2.3 Determination of Maximum & Minimum Density for Coarse Soils

The difficulty in evaluating the actual maximum and minimum density values for cohesionless soils has long been recognized. Some of the problems associated with these density determinations become even more evident when dealing with soils so coarse that "true" density values tend to be underestimated due to a lesser volume of soil at the edges of reasonably sized samples.

The gravel used for most of this study gave a wide range of values for minimum density varying with sample size. The variation of measured densities over the range of area/volume (A/V) ratios showed that as sample size is increased, and the ratio of surficial area to sample volume decreases, the deleterious effect of container edge volume errors on soil density determinations is reduced. Using the density values obtained over a range of A/V ratios, true maximum and minimum density values were extrapolated. To report accurate density values at which individual tests were conducted, measured density values needed to be interpreted by means of comparison of the tested sample geometries to curves representing variation of density with A/V ratios. Such curves were developed for the variation of maximum and minimum densities of the PT-gravel for a range of A/V ratios using the ASTM-D standard "large" compaction mold and the 12-inch diameter sample preparation mold used for large-scale tests. These curves are presented in Figure 6.5. In addition to the density tests performed for developing these density "correction" curves, large-scale vibration table tests were also performed, both wet and dry, in order to get a second evaluation of maximum density for the material. The results of the vibration table density tests agreed very well with those values determined by vibrating individual layers in the 12-inch diameter sample preparation mold.

Samples for minimum density tests were prepared by pluviation through standing water and hand placement of grains (previously suggested by Evans and Seed, 1987), but the "wet pluviation" method appeared to give more consistent and lower density values and were therefore ultimately used to determine minimum density. The values reported for maximum and minimum density of the PT-Gravel were 110.0 pcf and 92.2 pcf respectively.

6.3 Test Results

Four series of tests were performed on the PT-Gravel material. Undrained monotonic load tests ($\overline{\text{IC-U}}$) were performed at controlled strain rates so that a clear record of peak and residual strengths and pore water pressures could be collected. One series of tests were performed with the use of the injection-mitigation system, and one without. A series of undrained cyclic strength tests were performed on samples at approximately 50% relative density, to obtain a cyclic strength curve for the material without corrections for compliance errors. A similar series of cyclic tests were performed on samples identically prepared with the injection-correction system implemented.

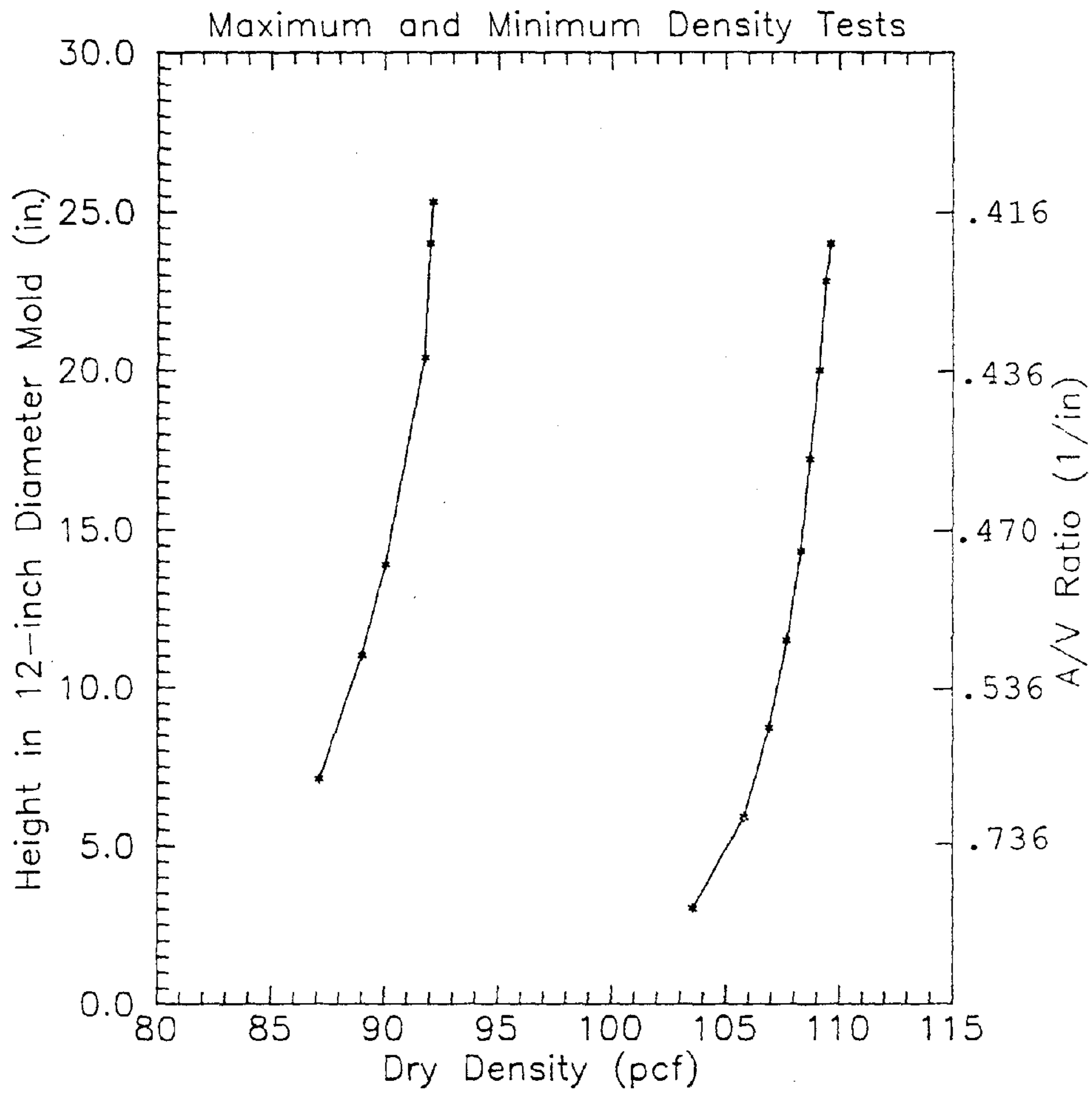


Figure 6.5: Relationship Between Measured Relative Density and Sample Size for PT-Gravel

6.3.1 Results of Undrained Static Load Tests

Table 6.1 presents the results of the undrained static load triaxial tests performed on 12-inch diameter samples of PT-Gravel with and without the use of the injection-mitigation system. The results of the individual tests are given in Figures 6.6 through 6.18. All static load tests were performed as "strain-controlled" tests at a constant axial strain rate of 0.25% per minute. All loads reported for these tests were the deviatoric stresses applied to the samples in excess of the isotropic consolidation/confining stresses. These loads were corrected to represent assumed changes in sample areas during loading based on assumed "right-cylindrical" deformation. Testing was discontinued when a minimum of 12% axial strain was reached. It was found that at significantly greater strain values, an appreciable amount of grain breakage occurred and the membranes had a tendency to tear thus making testing to greater strains of little practical value.

It is interesting to note that for some of the earlier tests in which the injection was unsteady and over-corrected during the initial stages of the tests (e.g. PT-42 and PT-46), the final test results (pore-pressure and deviator stress values at 12% ϵ_a) were essentially equal to those obtained from tests performed for which the injections were "smoothed out". This implies that "correct" steady-state or critical state (residual strength) $\overline{\text{IC-U}}$ test results can be obtained even with a "rough" injection system, as long as the accuracy of the injection correction is adequate once the system has stabilized.

A composite of the results from the two series of static $\overline{\text{IC-U}}$ tests are plotted in Figure 6.19, showing the differences in slope between the uncorrected and "correct" steady state lines for the test material. As expected, for samples whose densities were below critical state, the contractive behavior of those samples generated positive pore-pressures which reduced membrane penetration and

Table 6.1: Testing Conditions: $\overline{IC}$-U Triaxial Tests on PT-Gravel With and Without Membrane Compliance Mitigation

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,i}$ (ksc)	B-Value	$\sigma_{d,f}$ (ksc)	$\sigma'_{3,f}$ (ksc)
PT-54	24	No	2	0.981	5.3	1.51
PT-57	42	No	2	0.979	5.9	1.79
PT-56	54	No	2	0.976	7.25	1.94
PT-66	80	No	2	0.980	8.7	2.2
PT-55	94	No	2	0.976	9.8	2.35
PT-44	17.5	Yes	2	0.980	4.0	1.15
PT-45	38	Yes	2	0.971	5.4	1.58
PT-42	48.5	Yes	2	0.976	6.3	1.78
PT-69	49	Yes	2	0.980	6.4	1.8
PT-47	67.5	Yes	2	0.980	8.25	2.1
PT-64	80	Yes	2	0.979	9.75	2.41
PT-68	95	Yes	2	0.978	11.25	2.75
PT-46	97	Yes	2	0.972	11.0	2.80

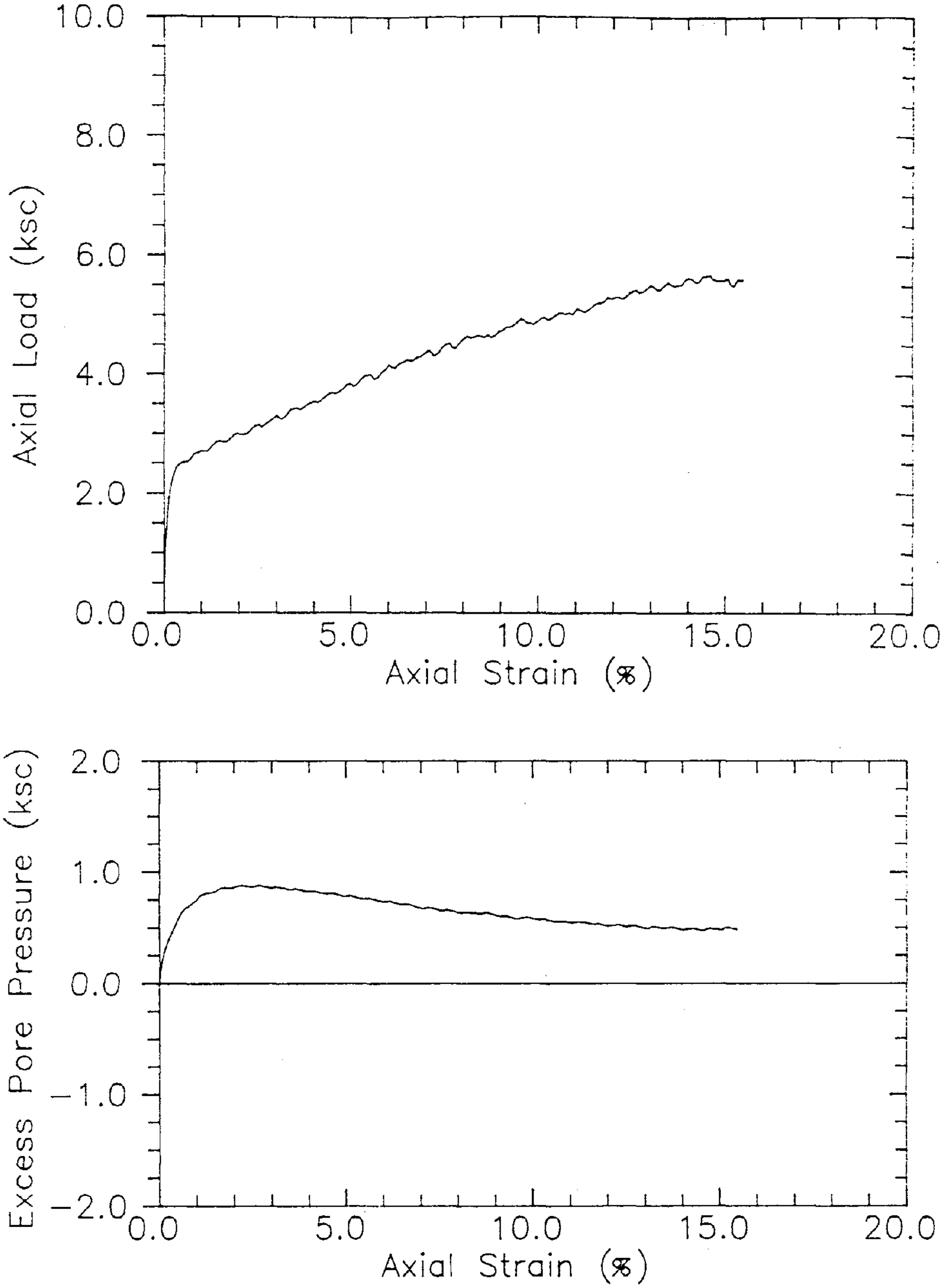


Figure 6.6: $\overline{\text{IC}}$-U Triaxial Test No. PT-54 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 24\%$)

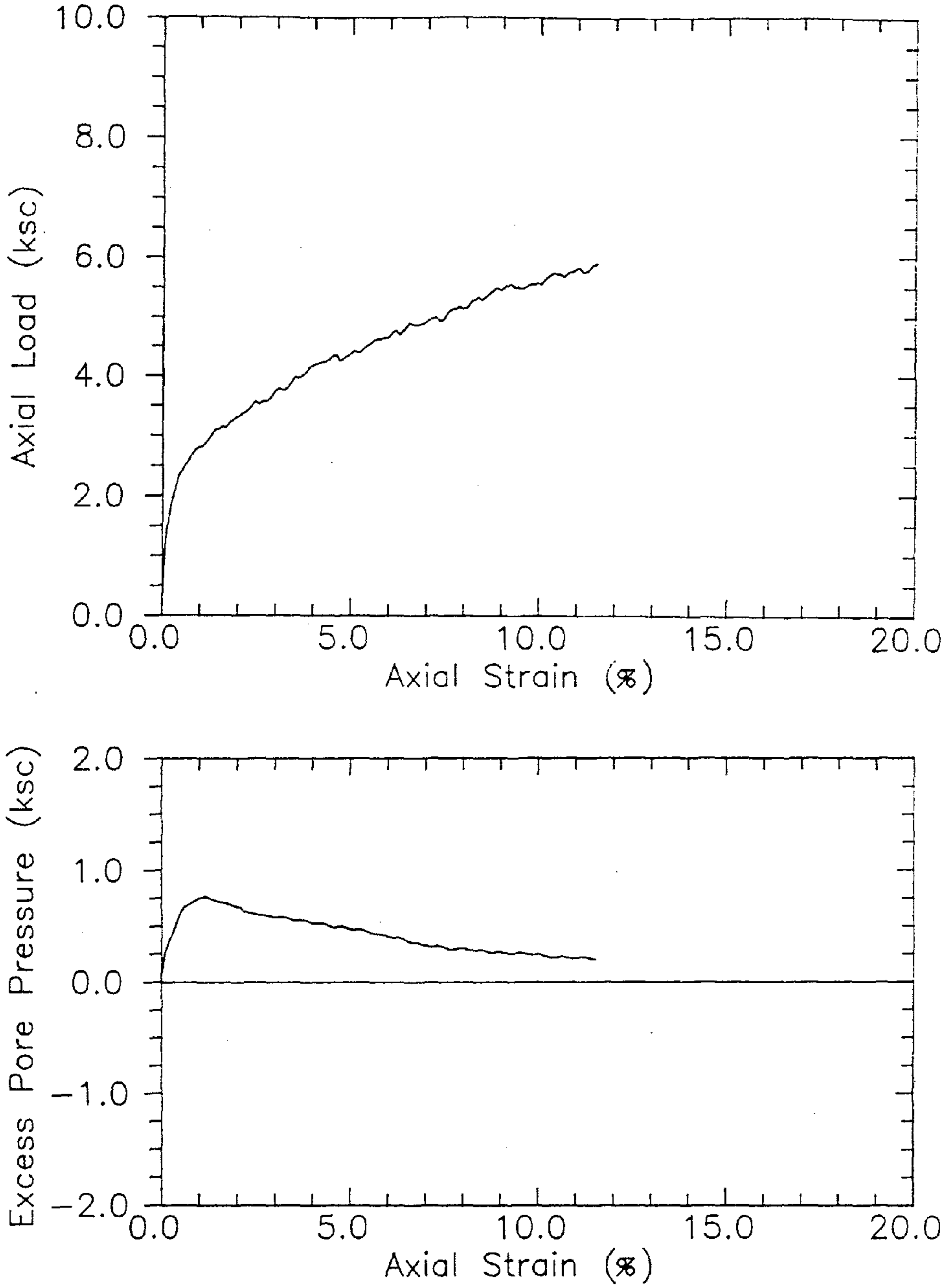


Figure 6.7: IC-U Triaxial Test No. PT-57 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 42\%$)

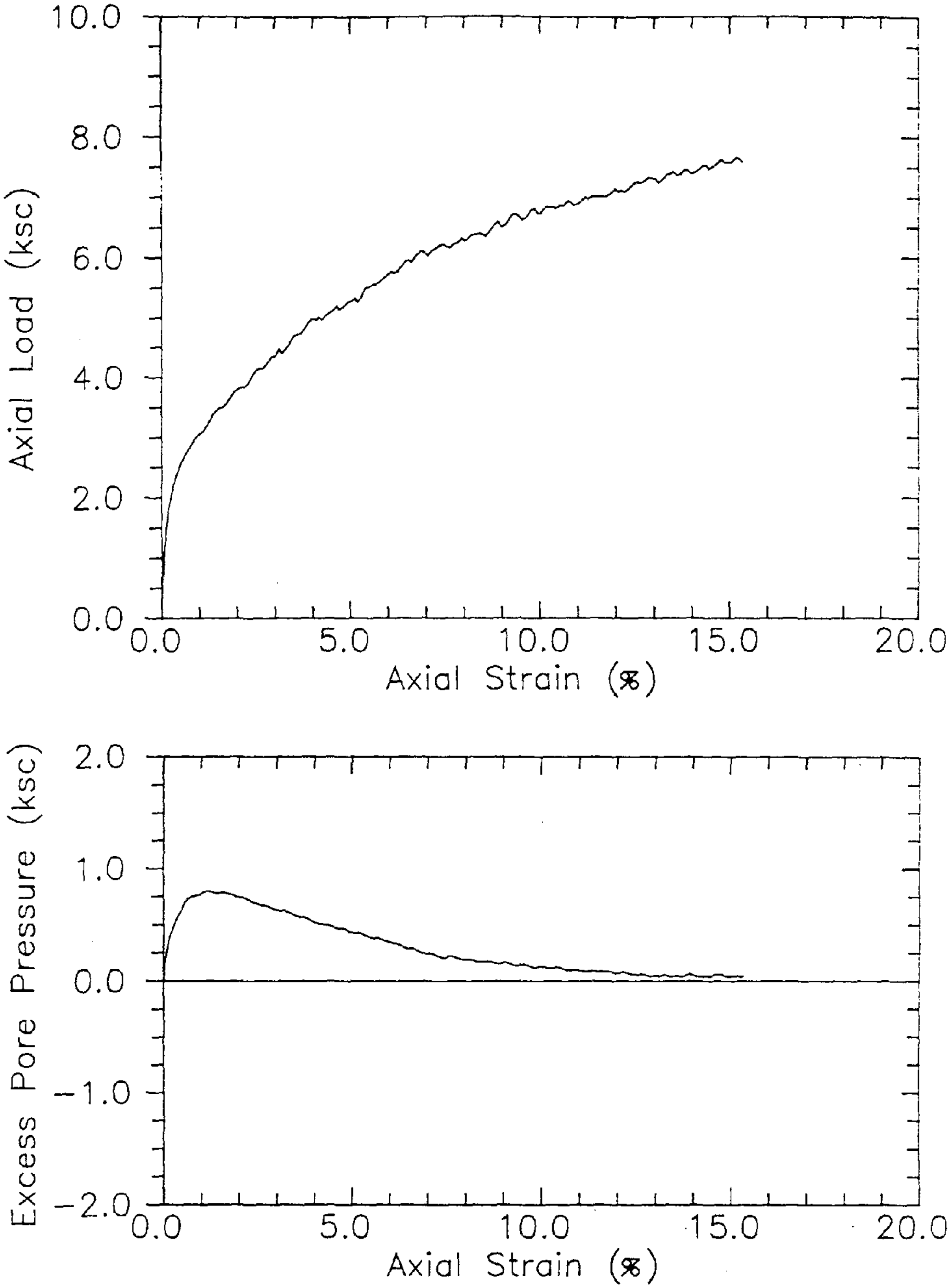


Figure 6.8: $\overline{\text{IC}}$-U Triaxial Test No. PT-56 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 54\%$)

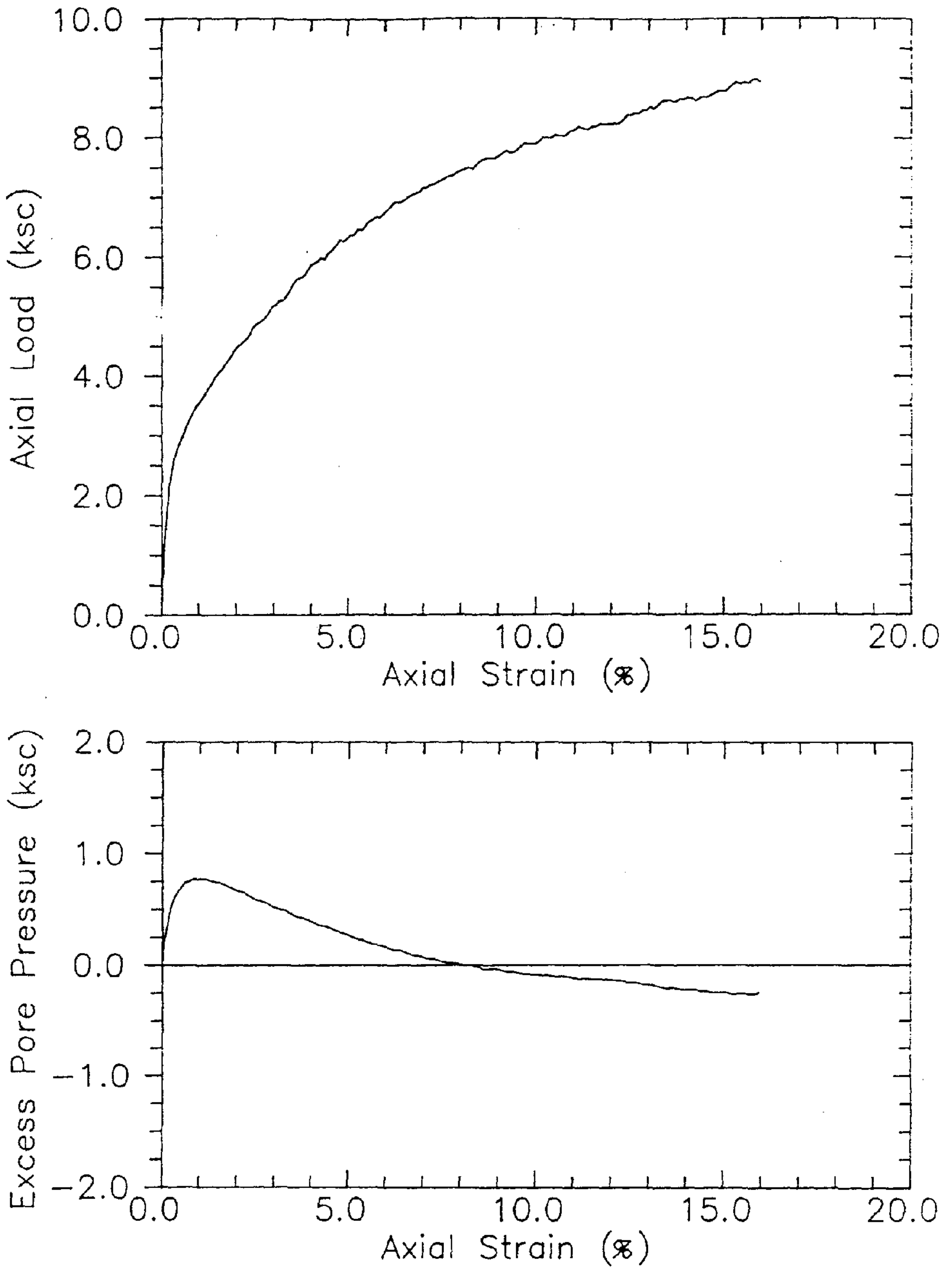


Figure 6.9: <u>$\overline{\text{IC-U}}$ Triaxial Test No. PT-66 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 80\%$)</u>

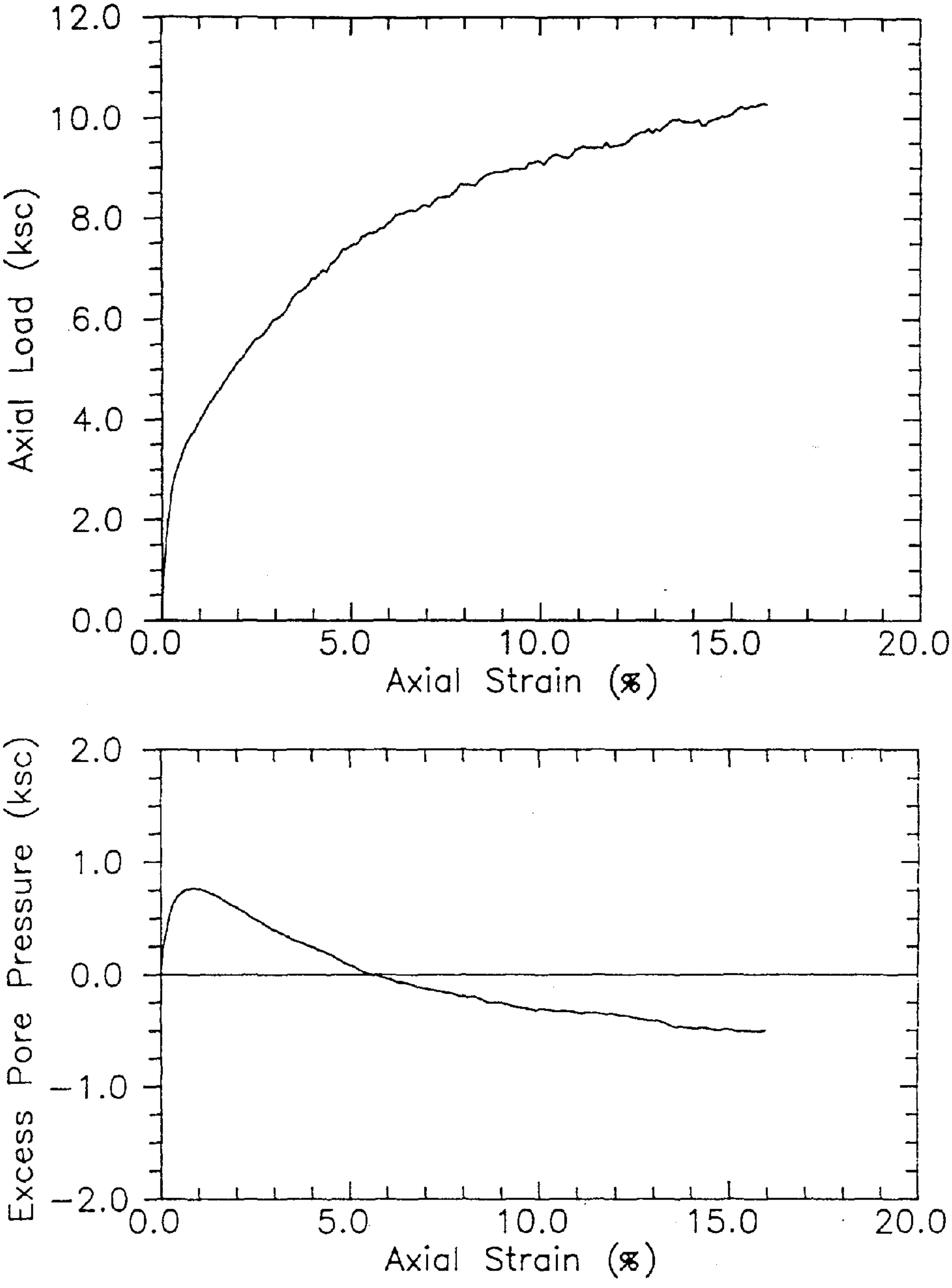
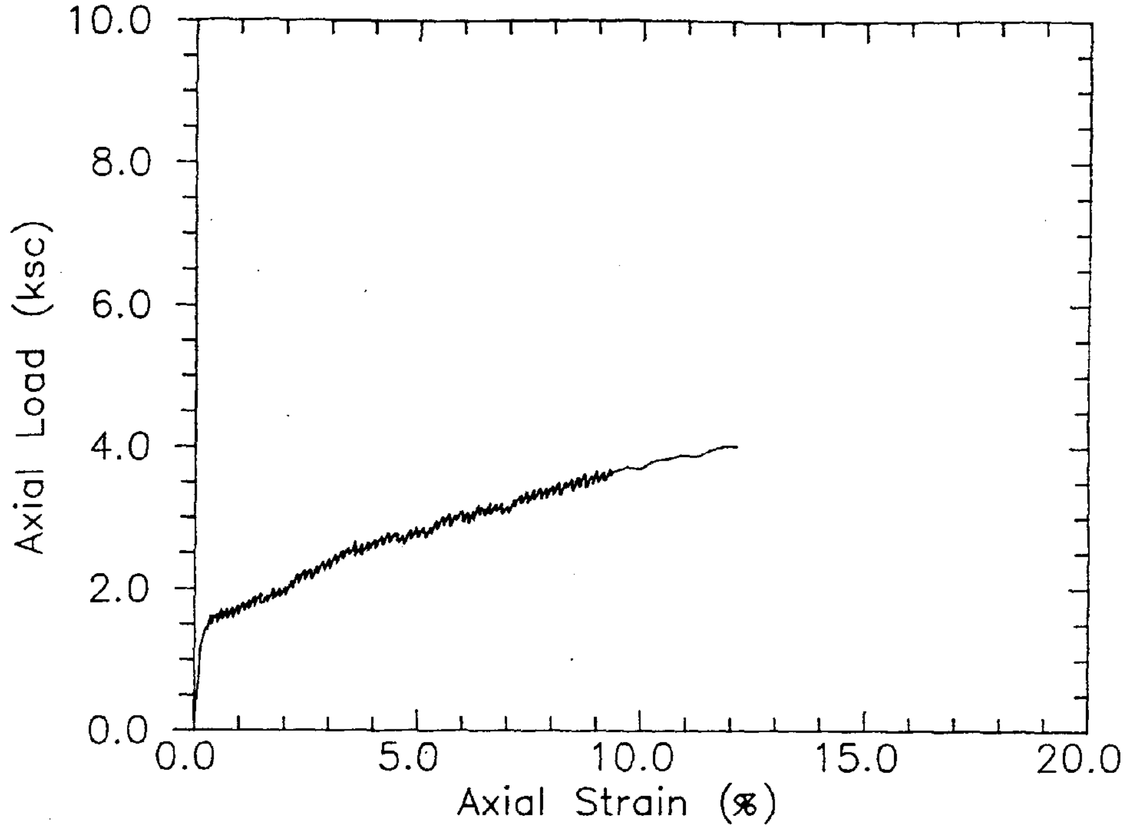


Figure 6.10: IC-U Triaxial Test No. PT-55 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 94\%$)



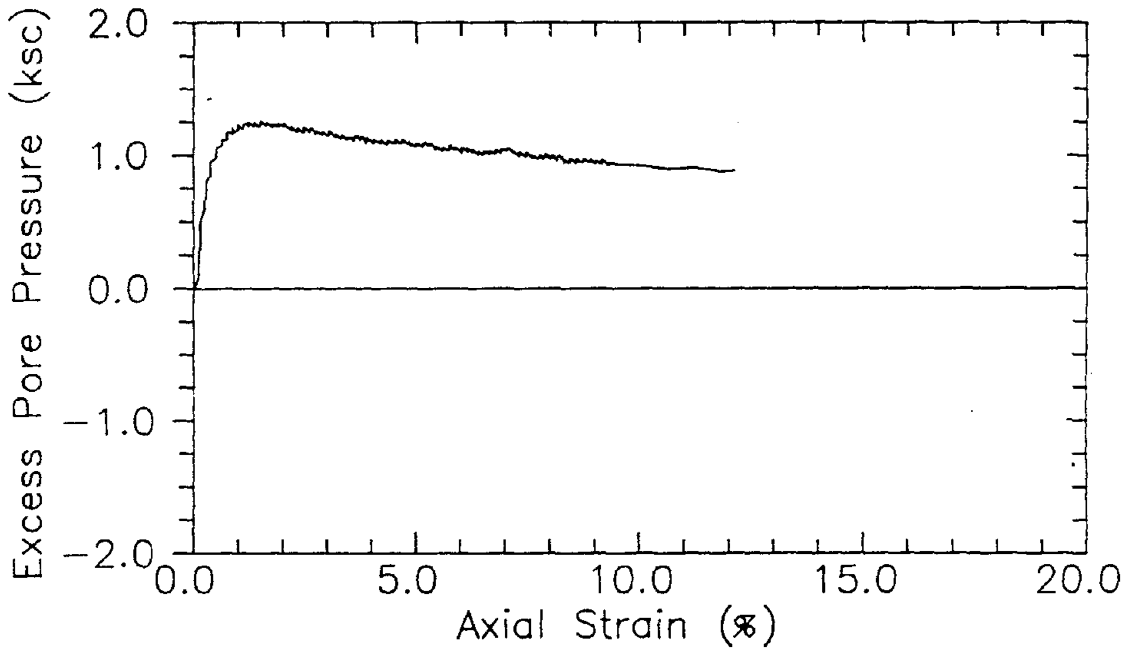


Figure 6.11: $\overline{\text{IC-U}}$ Triaxial Test No. PT-44 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 17.5\%$)

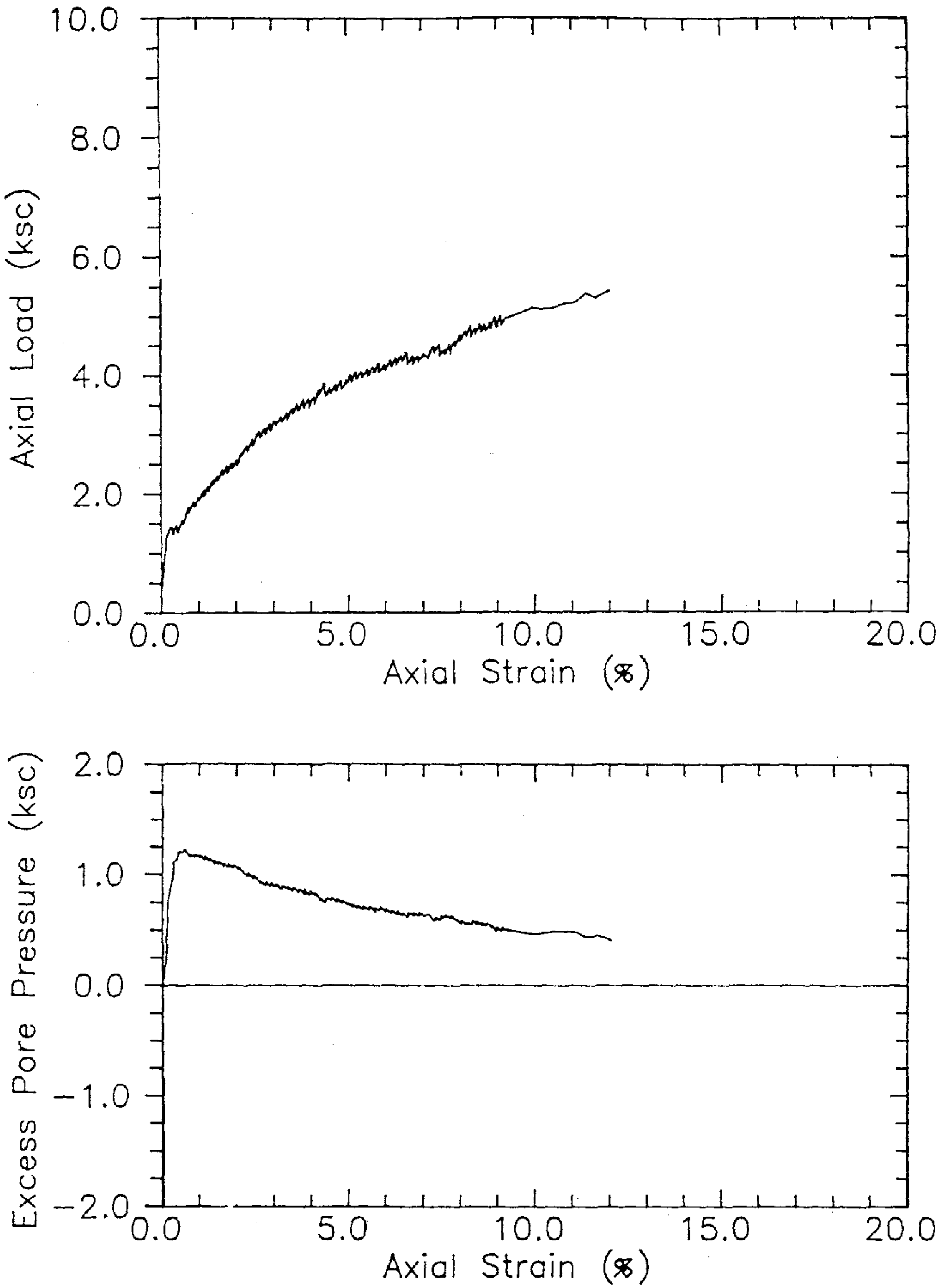


Figure 6.12: $\overline{\text{IC}}$-U Triaxial Test No. PT-45 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 38\%$)

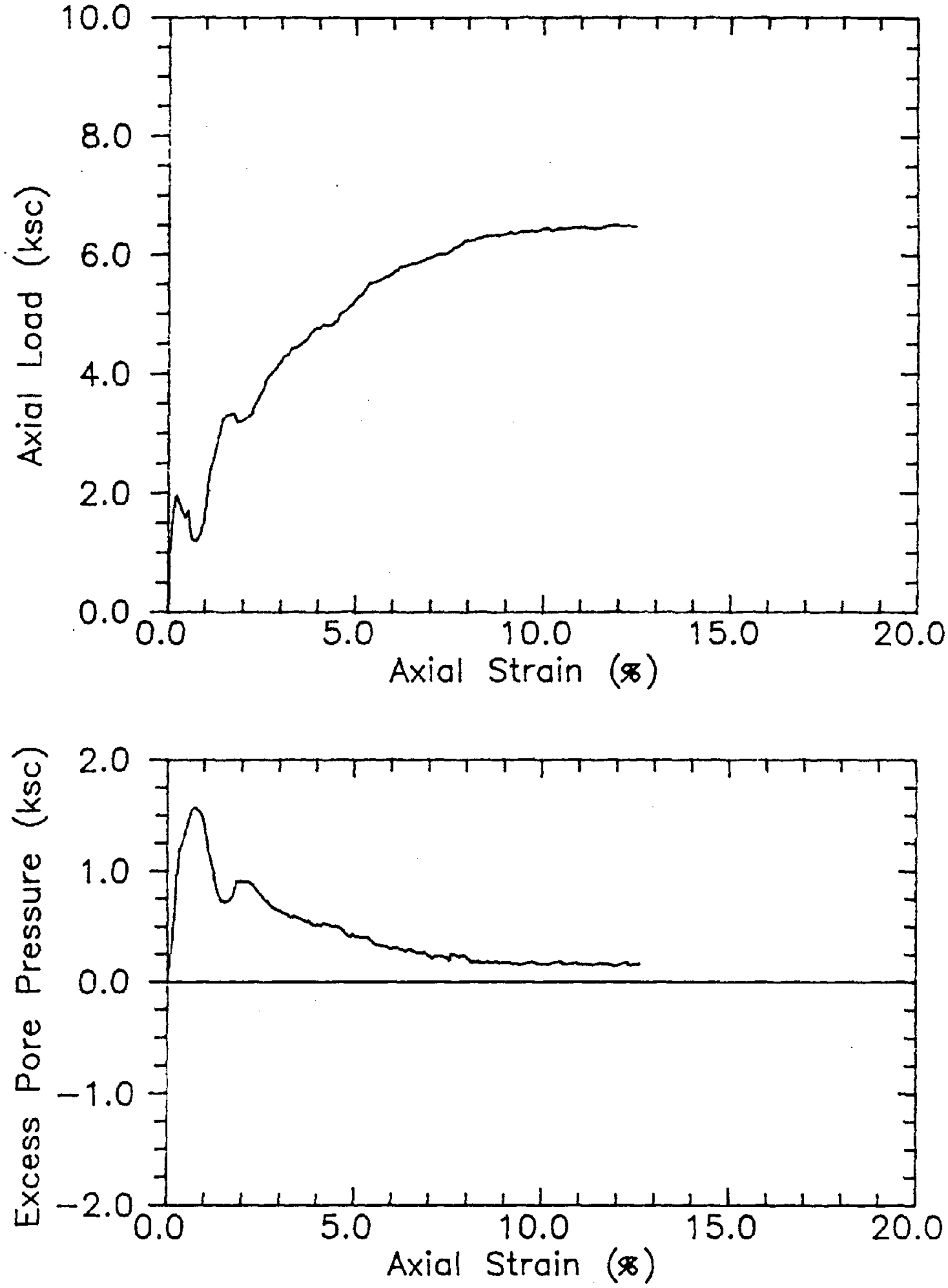


Figure 6.13: $\overleftarrow{\text{IC}}$-U Triaxial Test No. PT-42 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 48.5\%$)

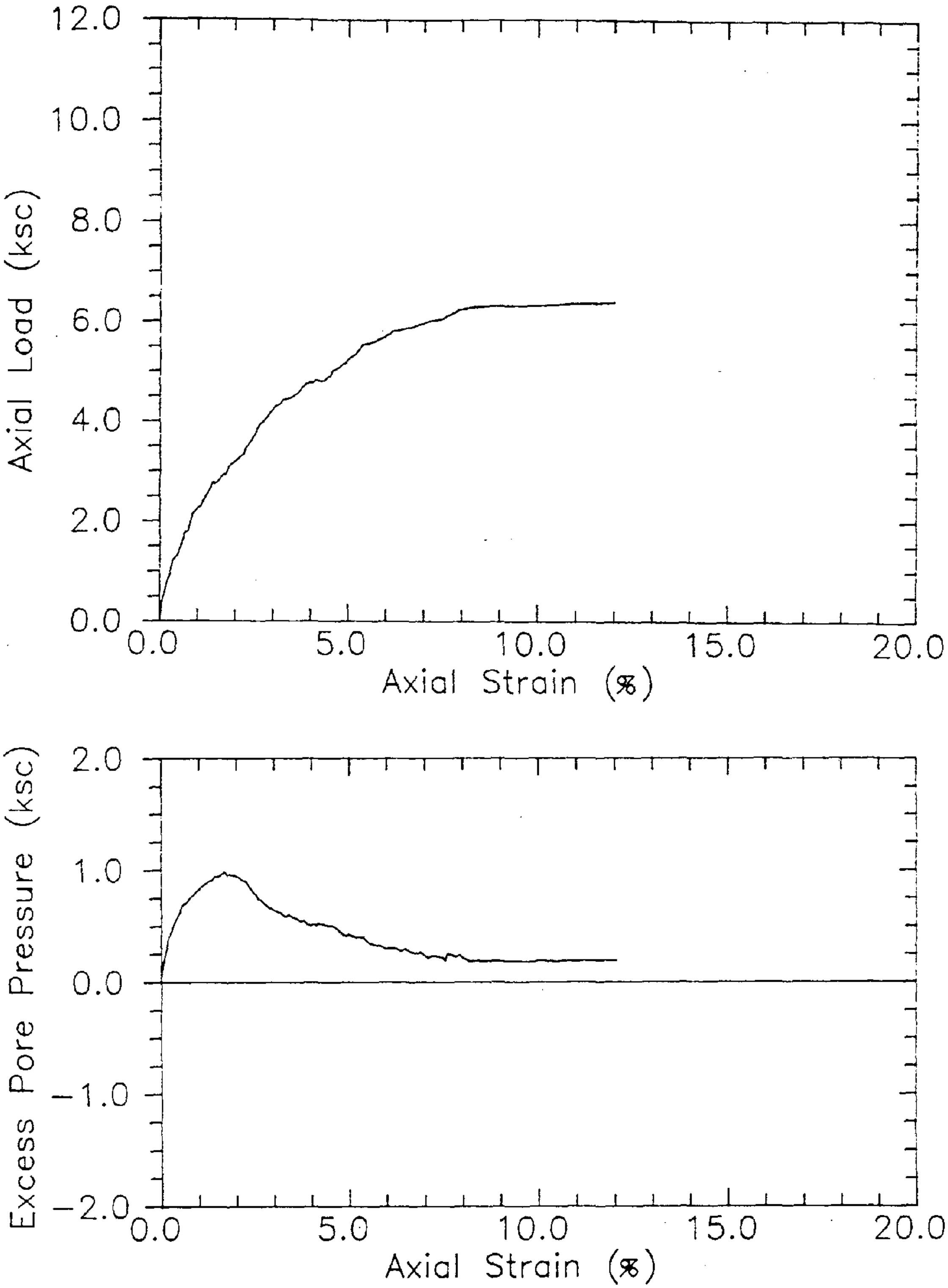


Figure 6.14: $\overline{\text{IC}}$-U Triaxial Test No. PT-69 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 49\%$)

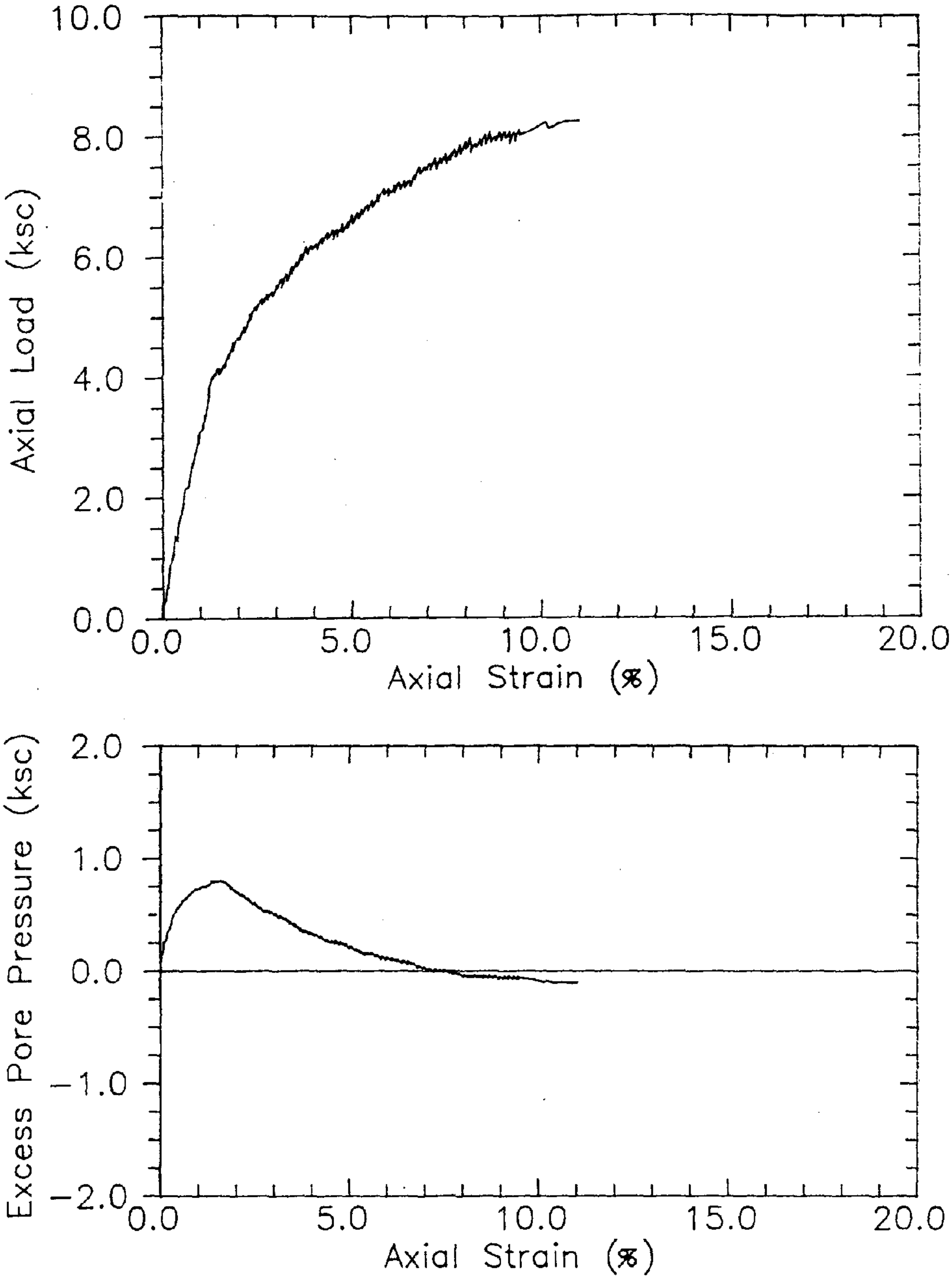


Figure 6.15: $\overline{\text{IC}}$-U Triaxial Test No. PT-47 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 67.5\%$)

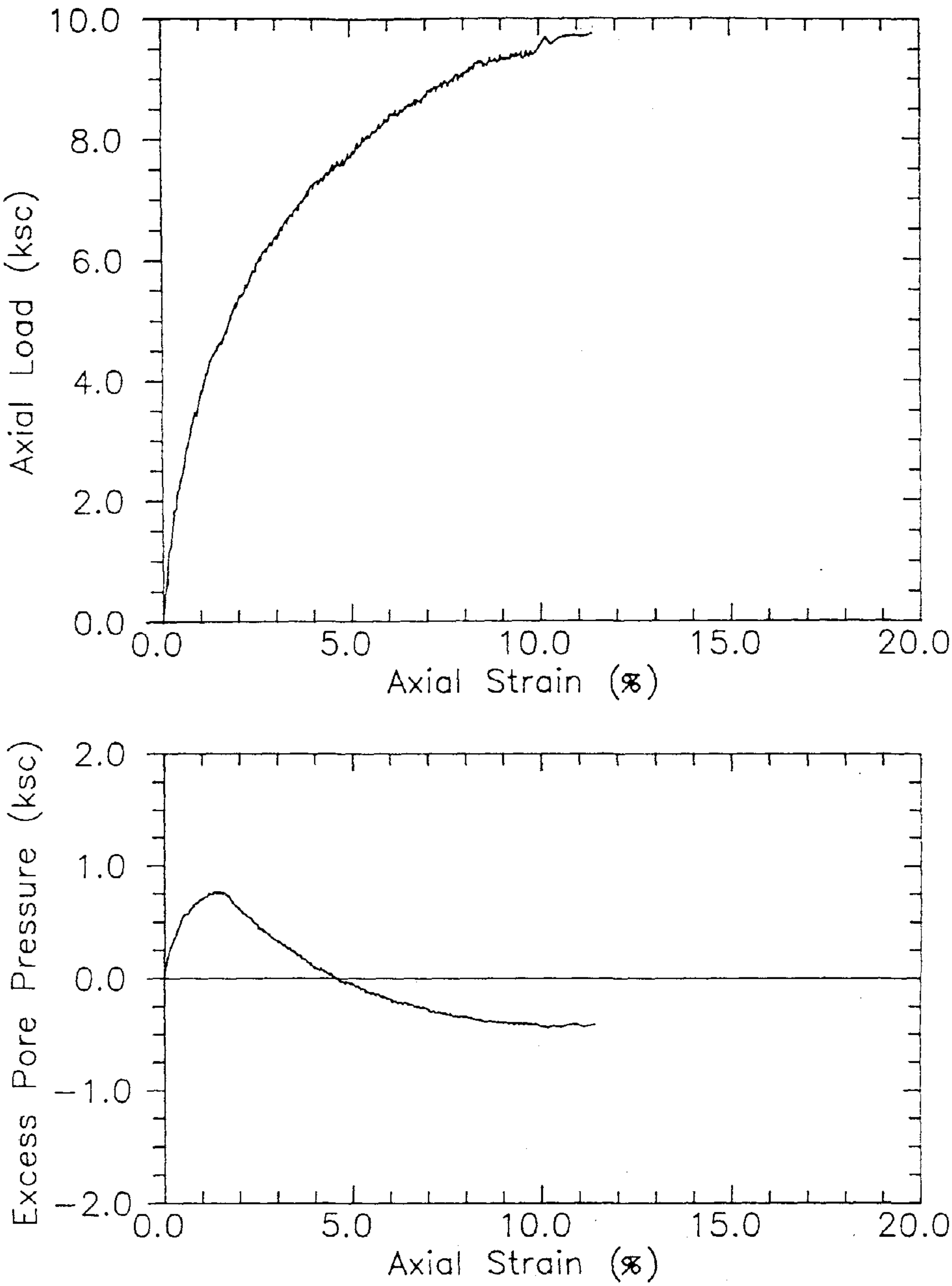


Figure 6.16: IC-U Triaxial Test No. PT-64 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 80\%$)

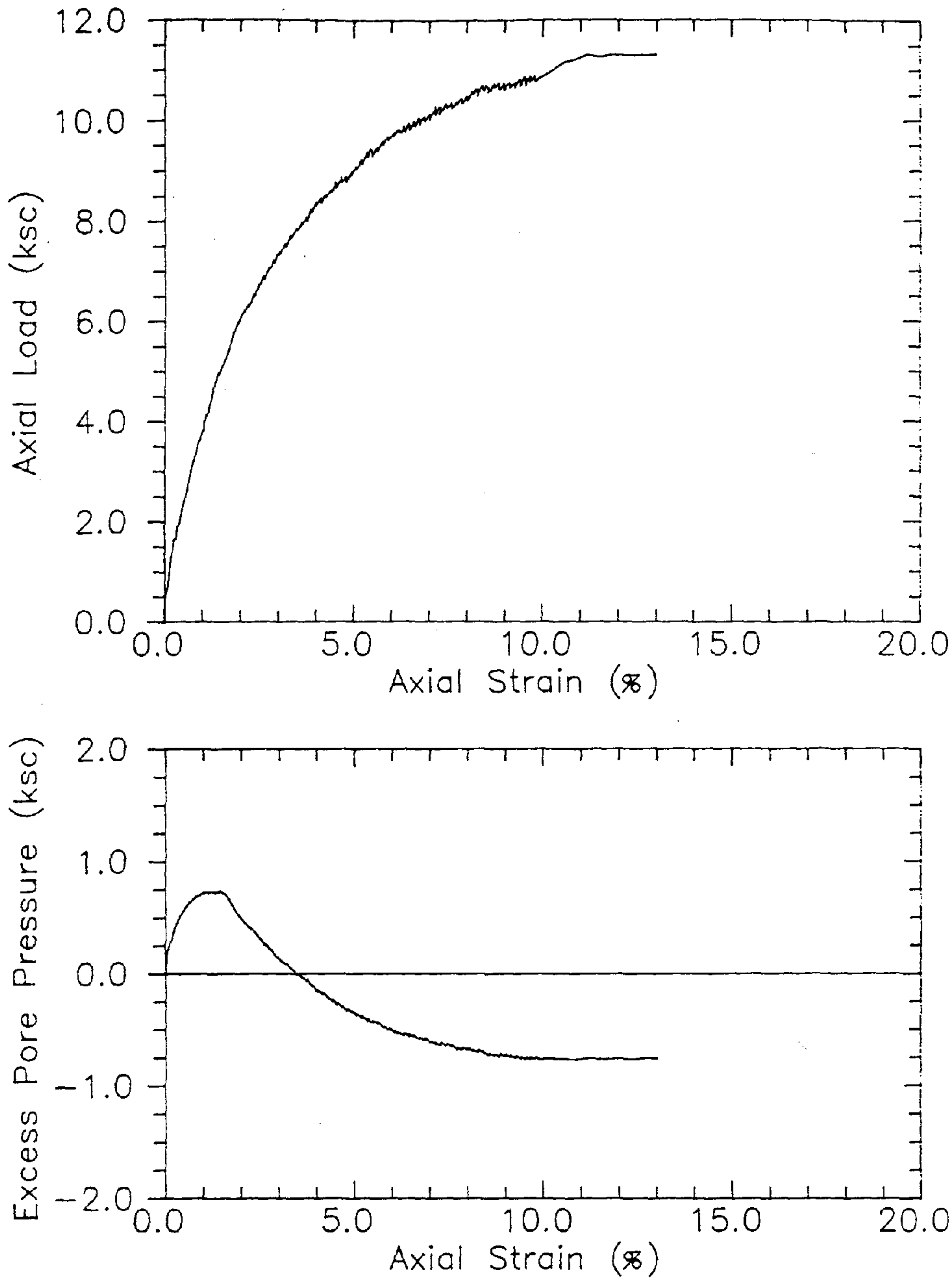


Figure 6.17: IC-U Triaxial Test No. PT-68 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 95\%$)

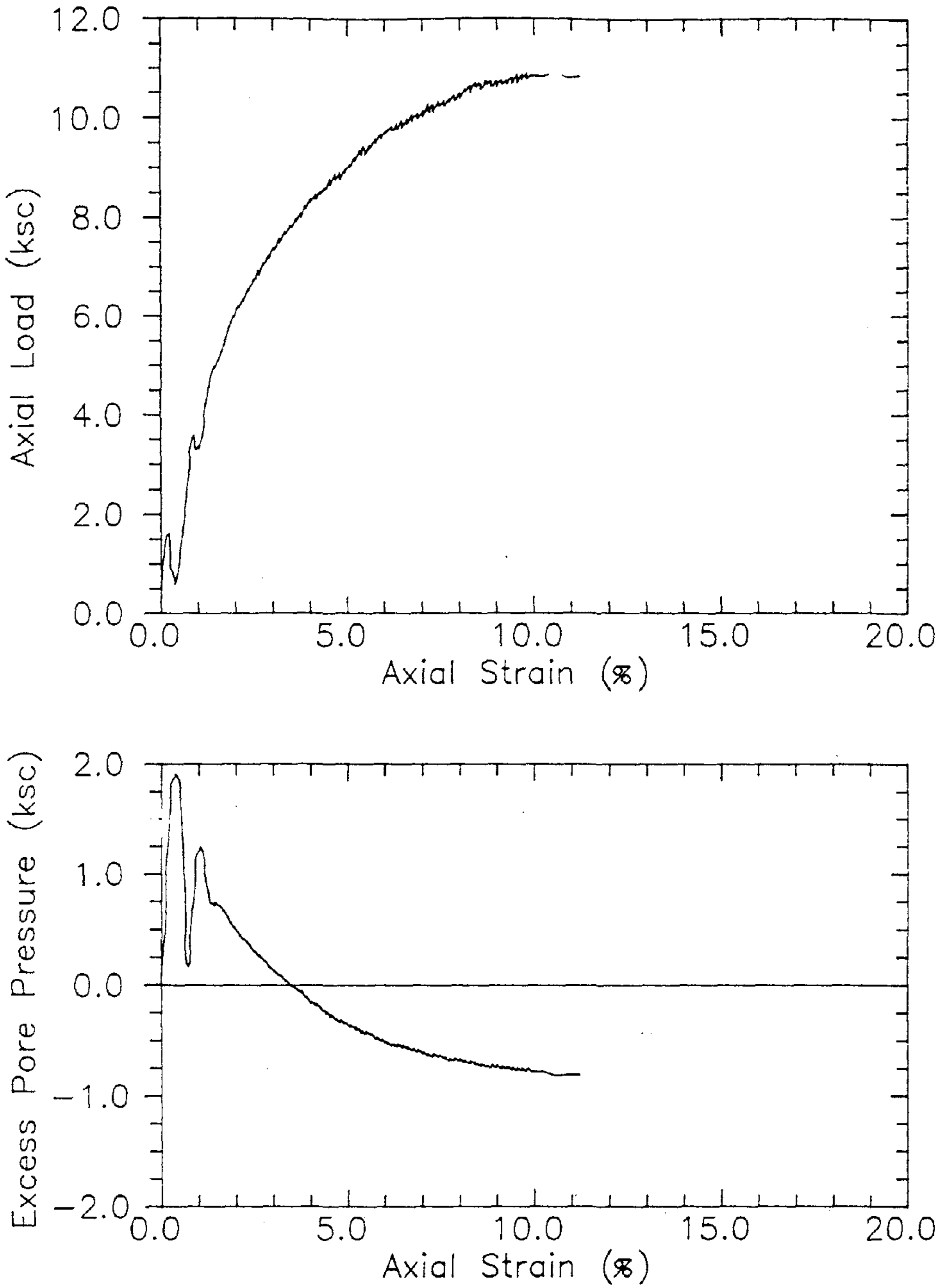


Figure 6.18: $\overline{\text{IC-U}}$ Triaxial Test No. PT-46 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 97\%$)

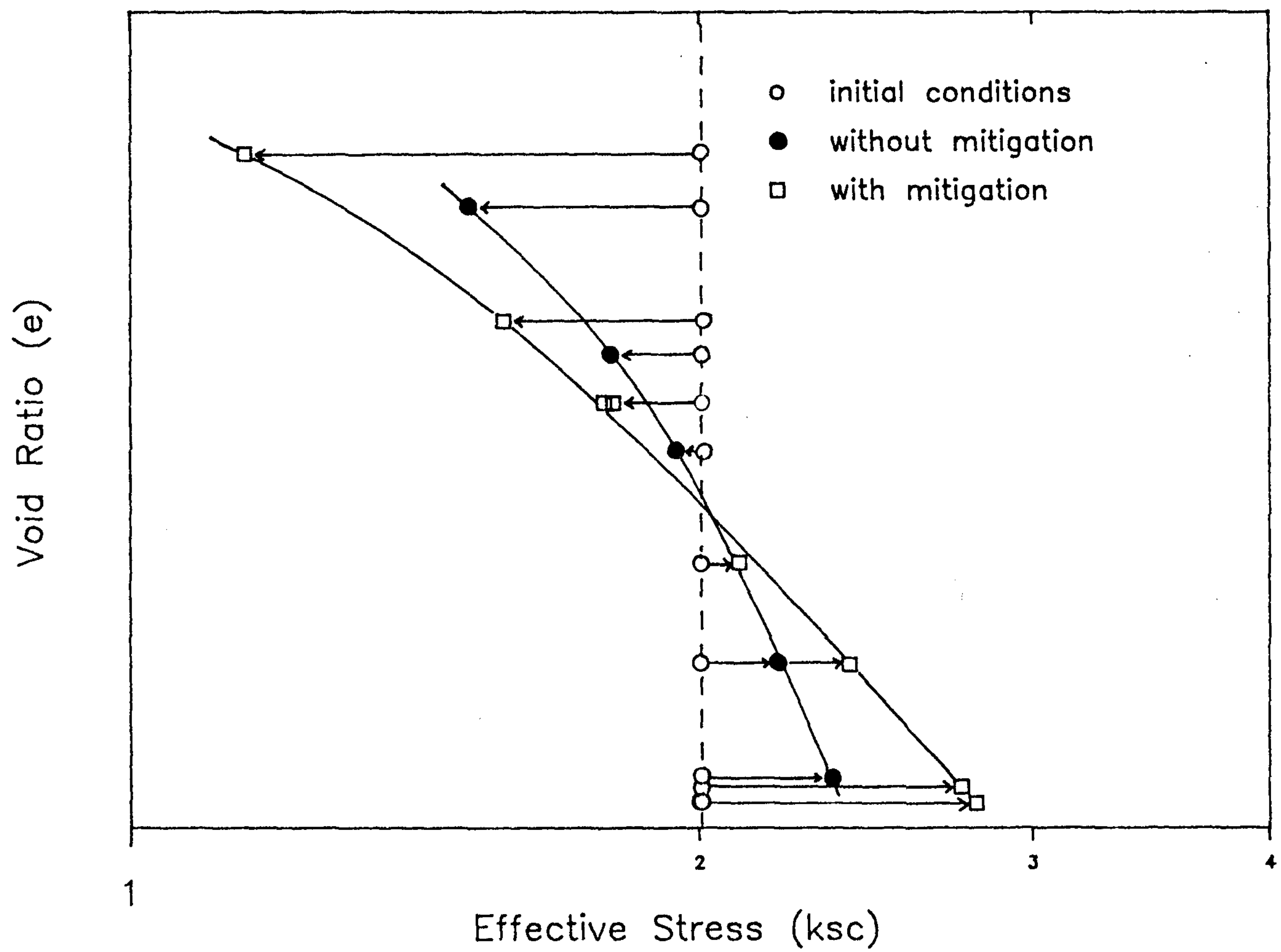


Figure 6.19: Critical-State Plot for $\overline{\text{IC}}$-U Triaxial Tests on PT-Gravel With and Without Membrane Compliance Mitigation

therefore showed residual effective stresses higher than they should, thus representing unconservatively higher static strengths. $\overline{\text{IC-U}}$ tests performed with the injection compliance-mitigation system reduced the residual strengths for the "loose" samples to more representative values. For samples that were constructed at densities higher than critical state the opposite was true. As the "dense" samples without compliance mitigation dilated, negative pore-pressures were generated for which the mitigation system removed water from the samples. The results of the uncorrected tests was to generate overconservative strength evaluations, as higher residual strengths were recorded in the corrected tests.

The importance of these findings becomes apparent when we consider that the use of static or residual strengths is becoming much more widely used as a design parameter for a number of critical earth structures. This becomes most critical for those soils in a relatively loose state as these are the ones most susceptible to very unconservative strength estimates when undrained triaxial tests are used as a basis for strength evaluations in which membrane compliance effects are not accounted for.

An attempt was made to see whether or not the test results obtained while using the injection-compensation system was employed could be replicated by simple mathematical corrections to volume changes based on compliance volume measurements, as was done for the small-scale tests performed by Seed and Anwar (1986). Figure 6.20 shows the results of the $\overline{\text{IC-U}}$ tests with and without injection, and without injection with the simple mathematical adjustment for volume error based on the observed total change in effective confining stress and the predetermined compliance curve (volumetric error as a function of $\Delta\sigma_3'$). It is apparent from this plot that a simple adjustment of the sample volumes to the unmitigated test results does not fully account for the amount of error induced by

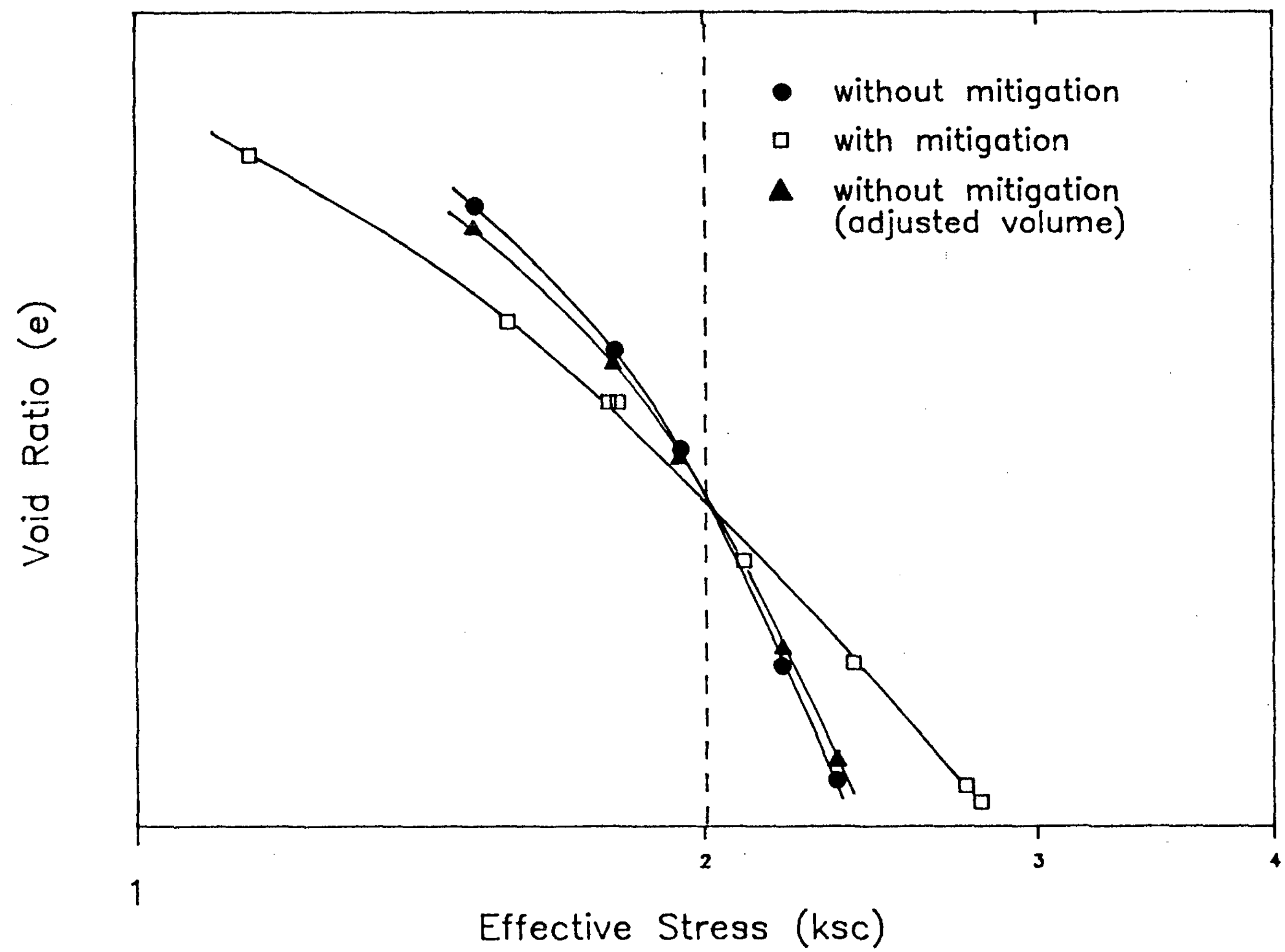


Figure 6.20: <u>Critical-State Plot for $\overline{\text{IC}}$-U Triaxial Tests on PT-Gravel With and Without Membrane Compliance Mitigation and Without Mitigation With Mathematical Adjustment</u>

membrane compliance for these samples. This leads to the conclusion that the testing errors incurred are more complicated and are likely to be the result of the interrelationships between volume change tendencies and pore-pressure generation. Until such a time that a theoretical model can be developed which can accurately model and predict the "correct" results of undrained tests performed on coarser materials, it is suggested that for large-scale testing of coarse gravelly soils a membrane compliance compensation technique is necessary to fully ensure that more accurate and conservative strength estimates are obtained.

6.3.2 Results of Cyclic Triaxial Tests

Table 6.2 presents a listing of the cyclic triaxial tests performed on 12-inch diameter samples of PT-Gravel. Individual results for these tests are shown in Figures 6.21 through 6.33. The cyclic load reported for these tests is the peak deviatoric load above and below the applied isotropic confining stress. The definition of "failure" for cyclic load tests in this study was taken to be $\pm 2.5\% \, \epsilon_a$ (5% double amplitude axial strain). It has been demonstrated that for gravelly materials this corresponds closely to the point at which full pore pressures ($r_u = 100\%$) are developed (Evans and Seed, 1987; Hynes, 1988), which is the definition used here for "initial liquefaction". This correlation has also been observed by this author for tests performed on a gravelly material tested as part of another study, as well as for the PT-Gravel used for these tests. This relationship can be seen in Figures 6.21 through 6.33.

The cyclic tests were load-controlled with dynamic loadings based on the initial sample area calculated at the end of consolidation. Corrections to dynamic loads applied to the specimens for membrane strength were calculated to be considerably less than 1% using an expression suggested by Bishop and Henkel

Table 6.2: Isotropically Consolidated Undrained Cyclic Triaxial Tests on PT-Gravel With and Without Membrane Compliance Mitigation

Test No.	D_R (%)	Membrane Compliance Mitigation	Initial Confining Stress: $\sigma'_{3,i}$ (ksc)	B-Value	CSR ($\sigma_{d,c}/2\sigma'_a$)	No. of Cycles to ± 2.5% ϵ_A
PT-29	51.0	No	2	.980	.350	2.5
PT-30	50.5	No	2	.982	.3075	6
PT-27	49.5	No	2	.976	.2825	10
PT-28	49.9	No	2	.972	.267	22
PT-19	50.8	No	2	.981	.235	*See Note
PT-51	51.8	Yes	2	.980	.346	1.2
PT-50	52.0	Yes	2	.982	.301	2
PT-39	49.4	Yes	2	.980	.275	3
PT-38	51.0	Yes	2	.976	.259	4
PT-53	49.9	Yes	2	.980	.240	4.5
PT-40	51.8	Yes	2	.976	.233	6
PT-67	50.4	Yes	2	.981	.223	16
PT-41	51.5	Yes	2	.979	.215	82

Note: Test No. PT-19 was stopped at 400 cycles with pore pressure ratio of $r_u \approx 0.55$.

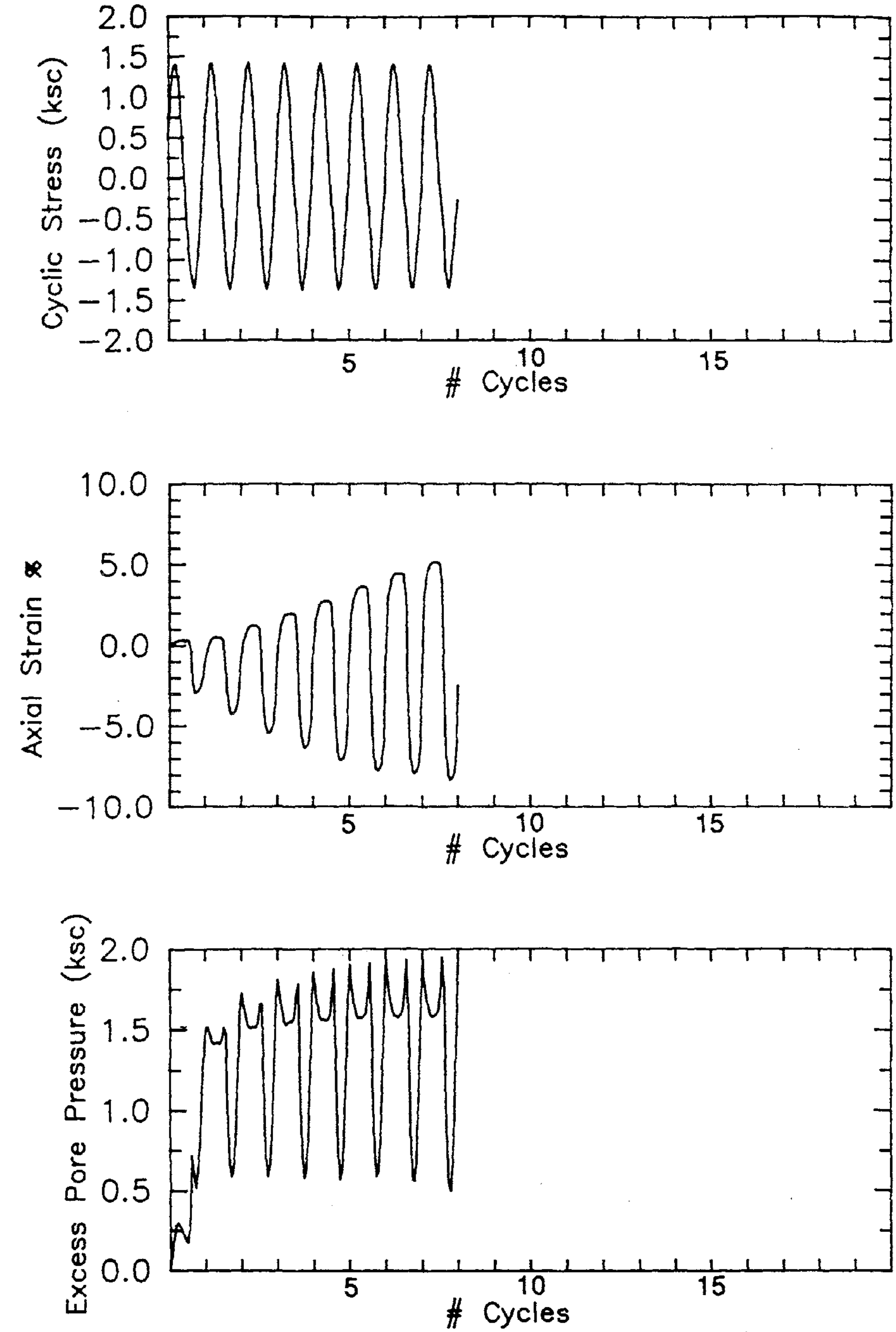
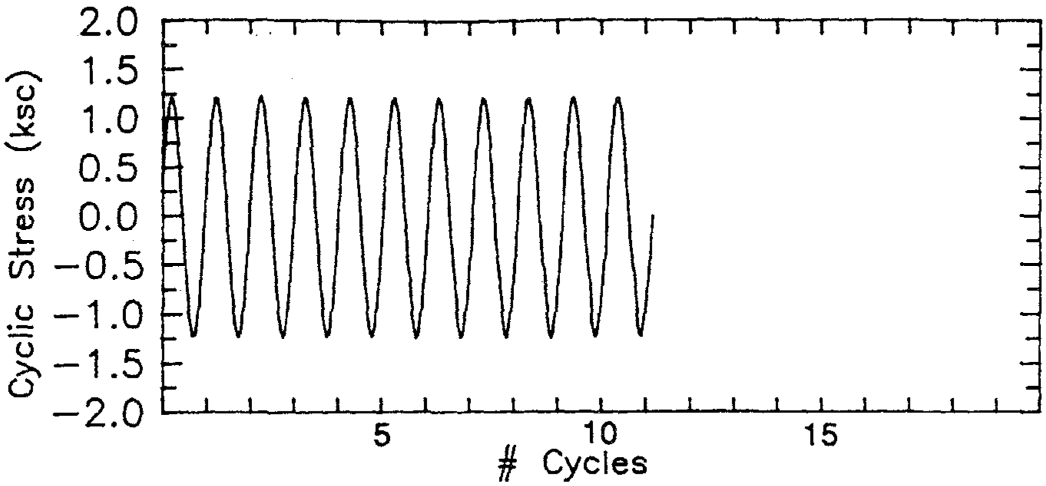
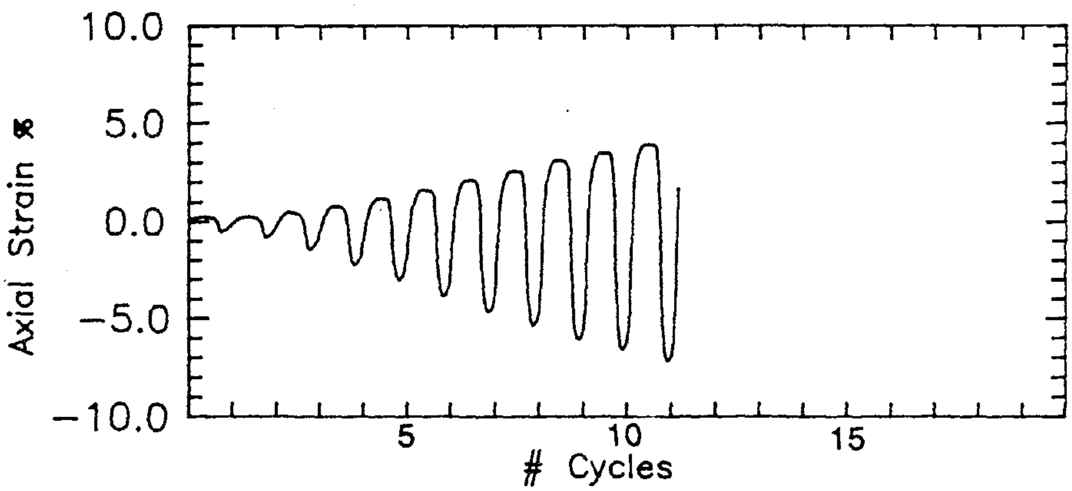


Figure 6.21: Cyclic Triaxial Test No. PT-29 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 51\%$)





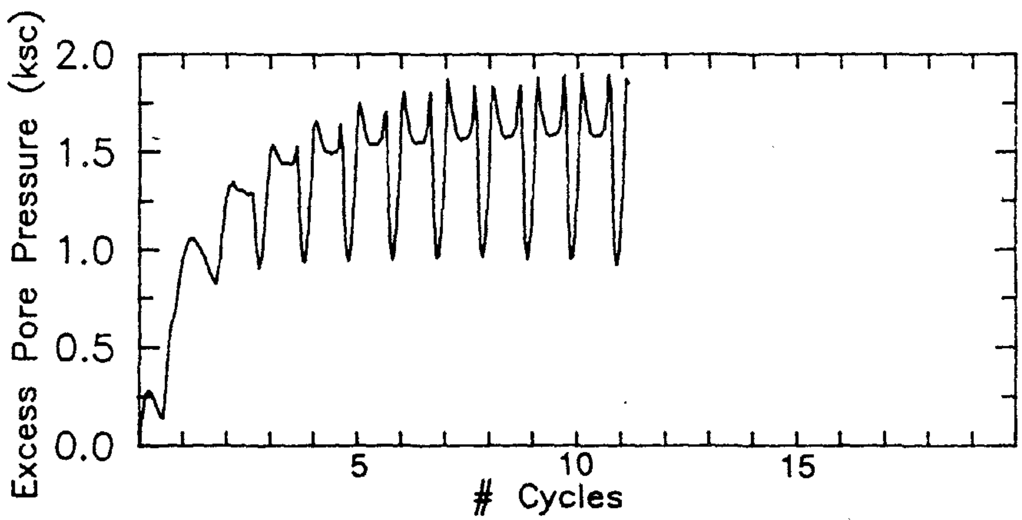


Figure 6.22: Cyclic Triaxial Test No. PT-30 (PT-Gravel Without Membrane Compliance Mitigation, D_R=51%)

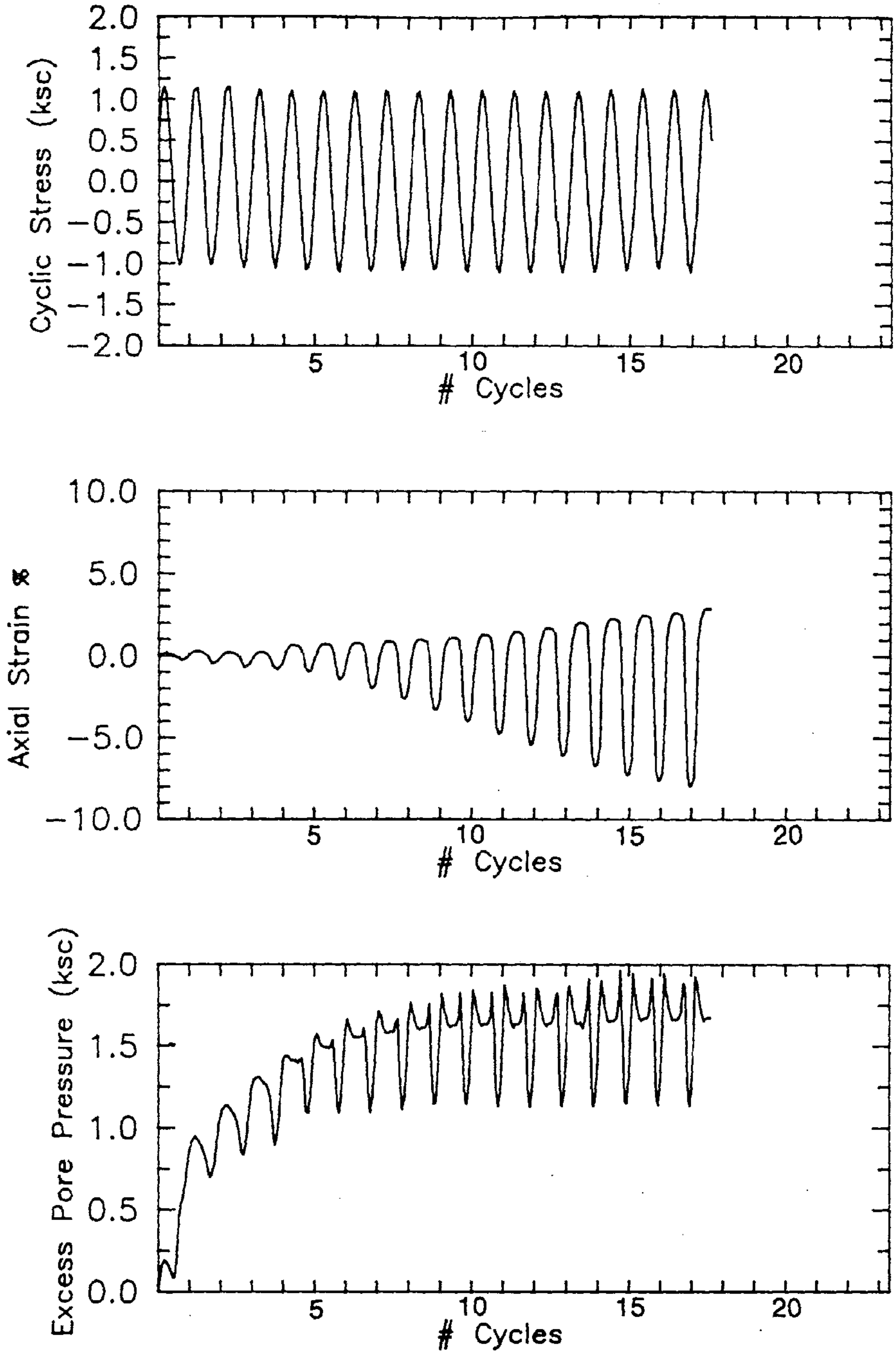


Figure 6.23: Cyclic Triaxial Test No. PT-27 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 50\%$)

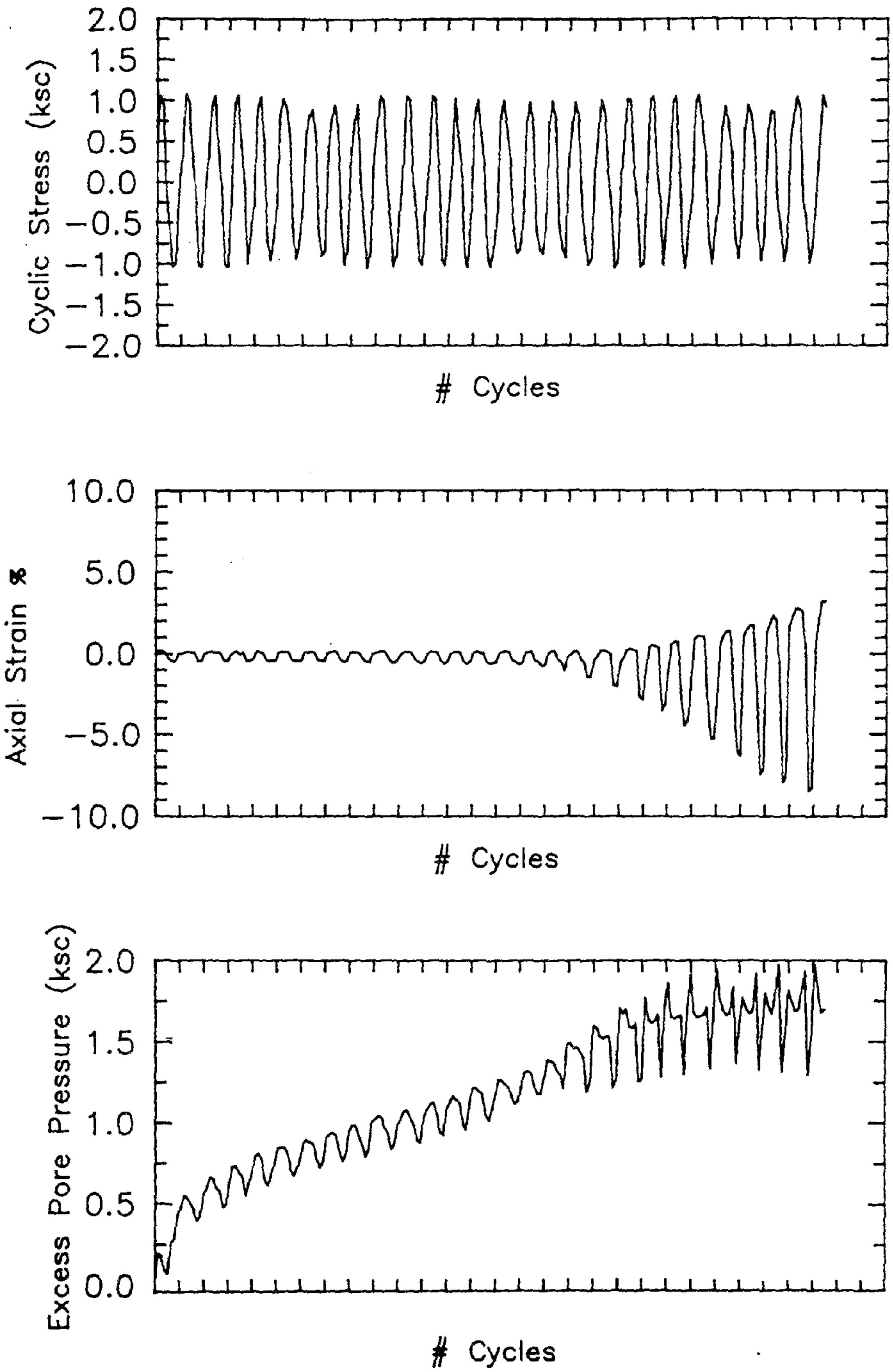


Figure 6.24: Cyclic Triaxial Test No. PT-28 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 51\%$)

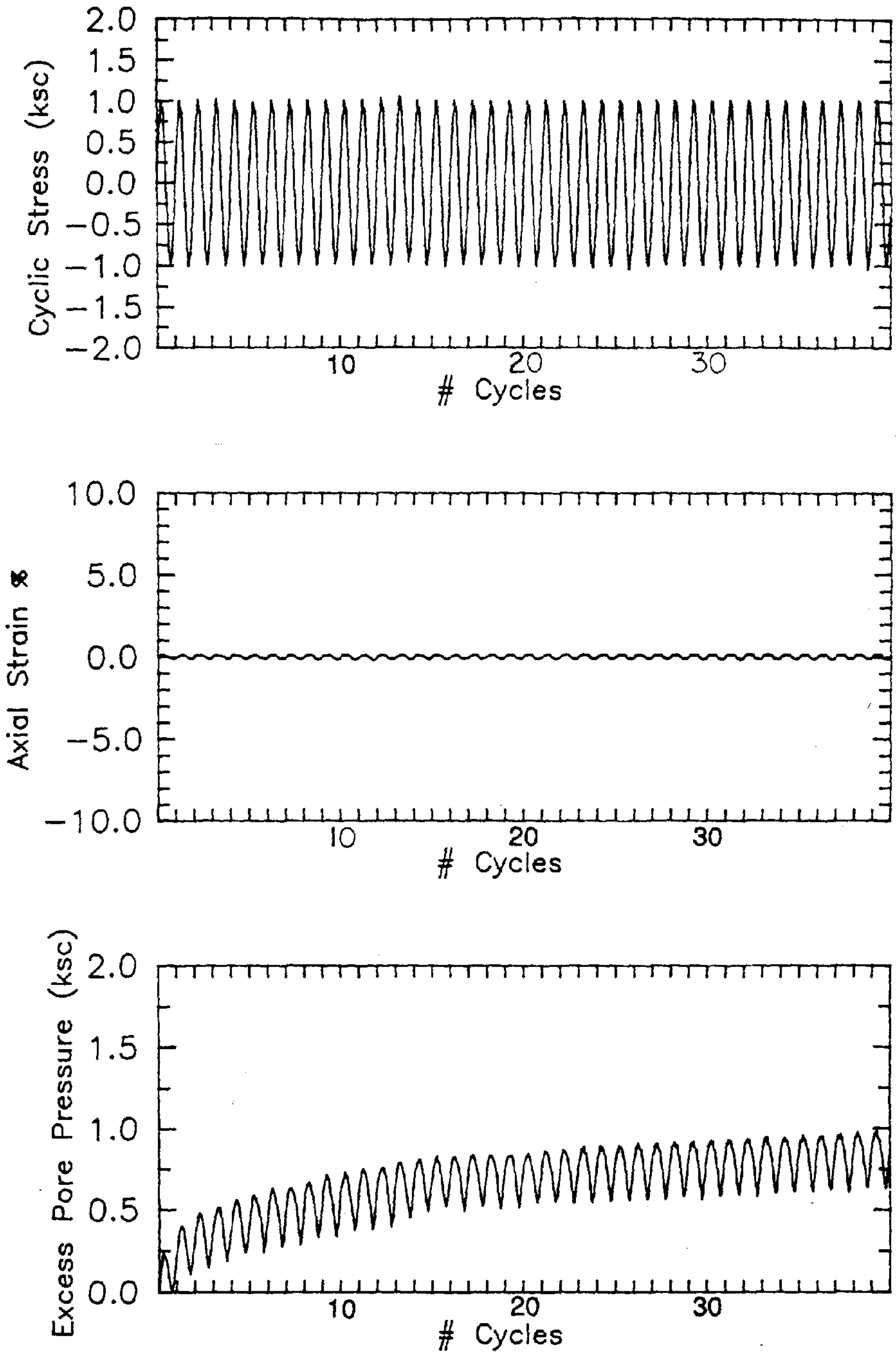


Figure 6.25: Cyclic Triaxial Test No. PT-19 (PT-Gravel Without Membrane Compliance Mitigation, $D_R \approx 51\%$)

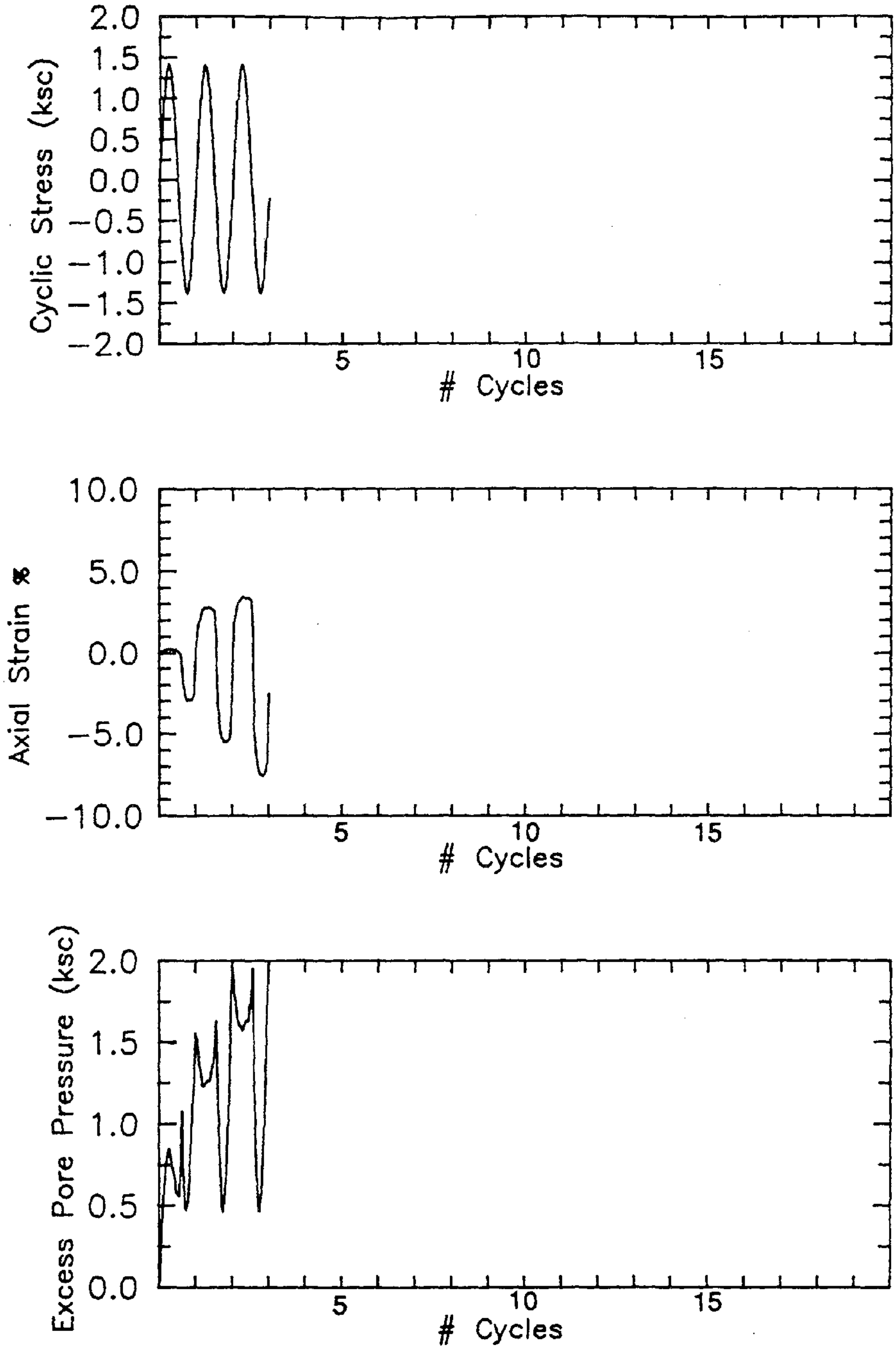


Figure 6.26: Cyclic Triaxial Test No. PT-51 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 52\%$)

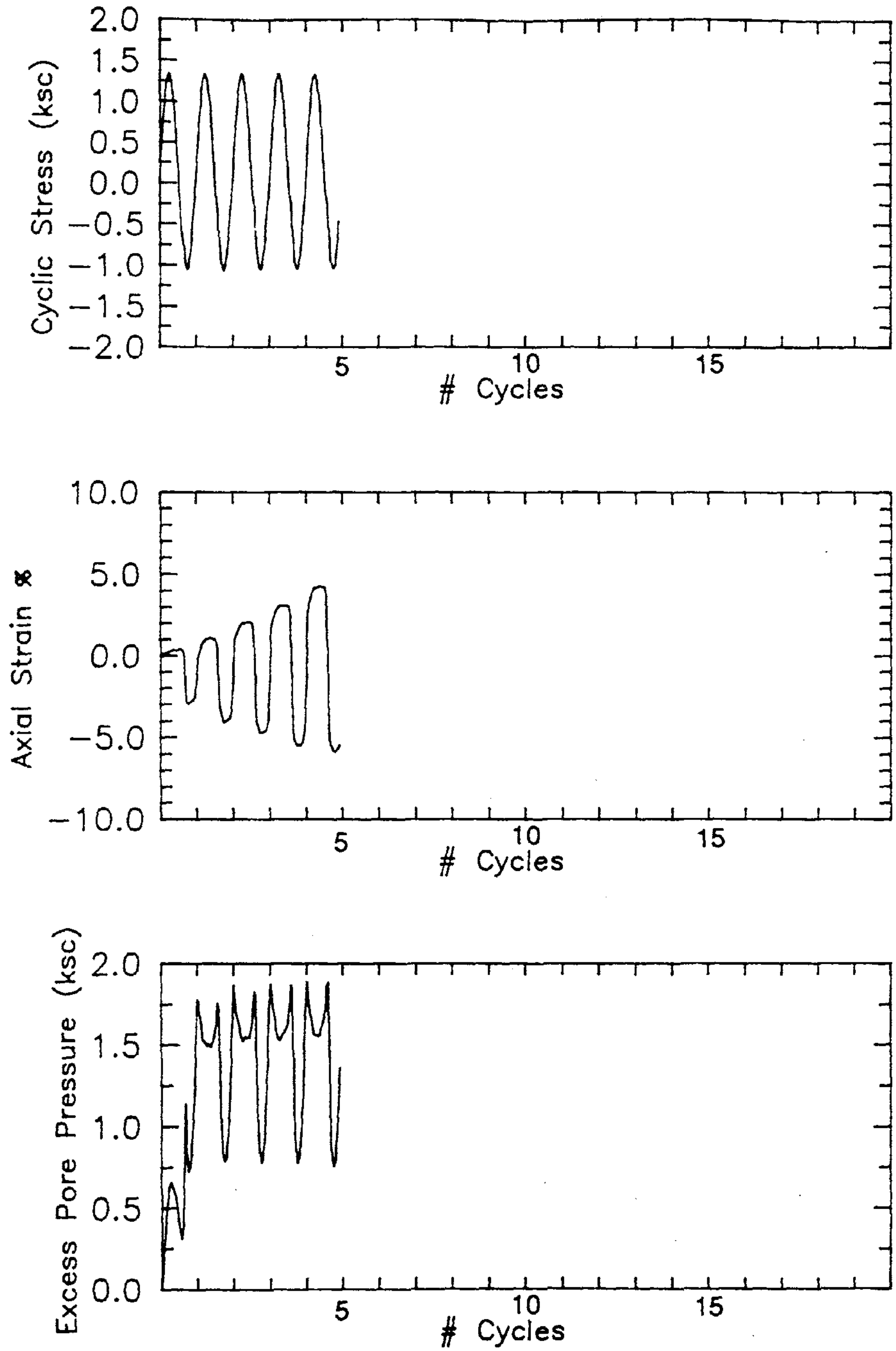


Figure 6.27: Cyclic Triaxial Test No. PT-50 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 52\%$)

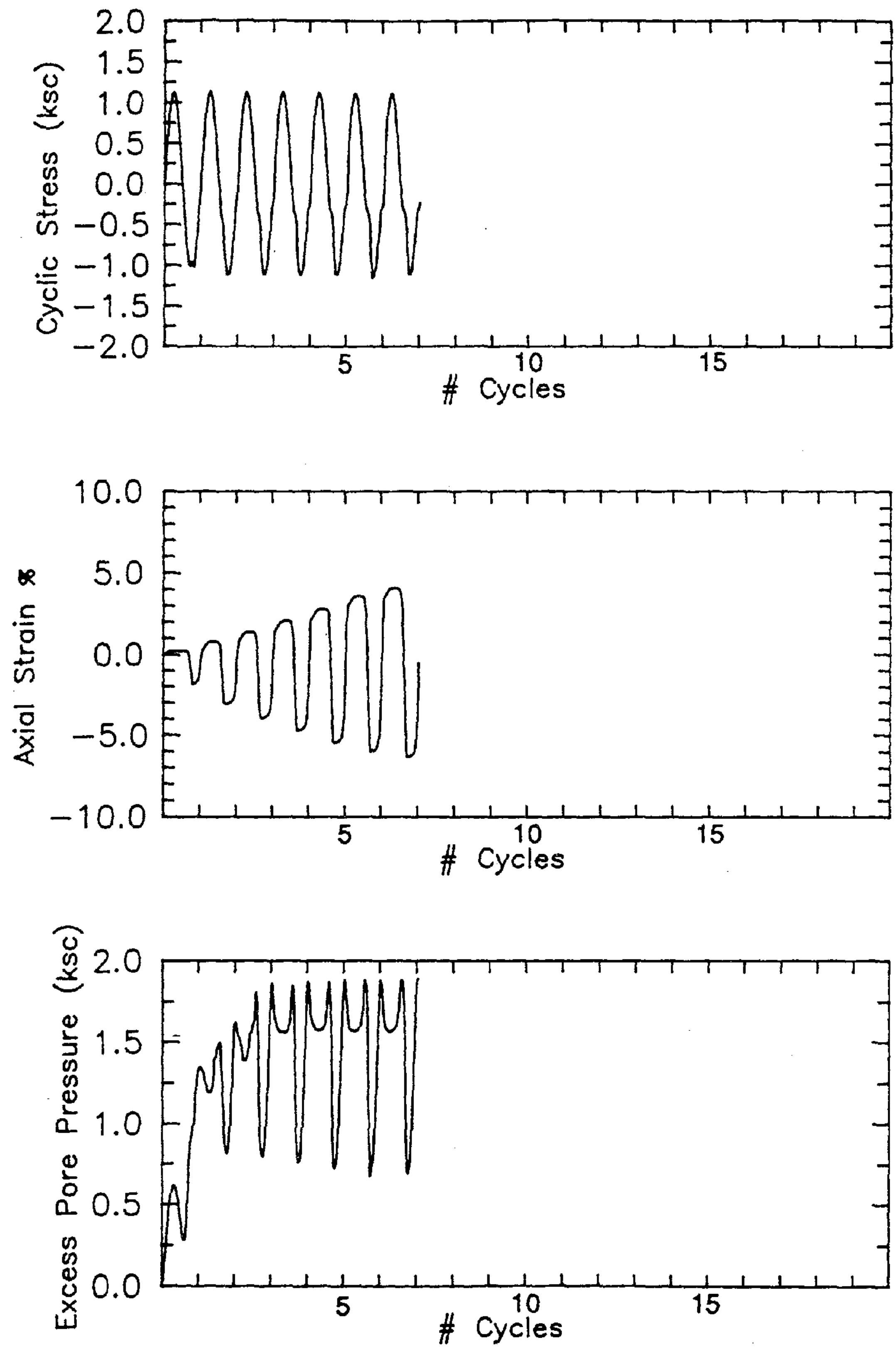


Figure 6.28: Cyclic Triaxial Test No. PT-39 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 49\%$)

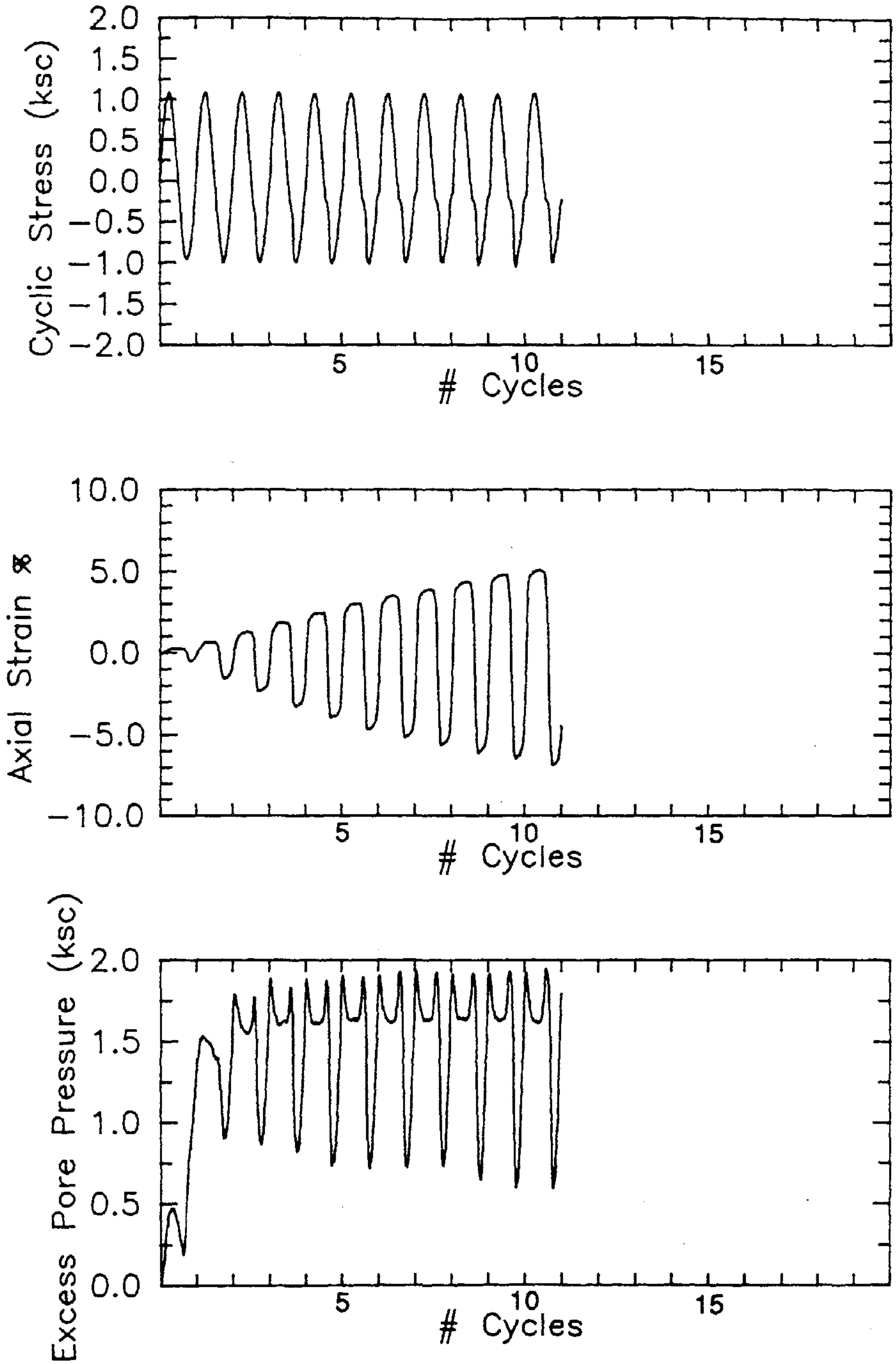


Figure 6.29: Cyclic Triaxial Test No. PT-38 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 51\%$)

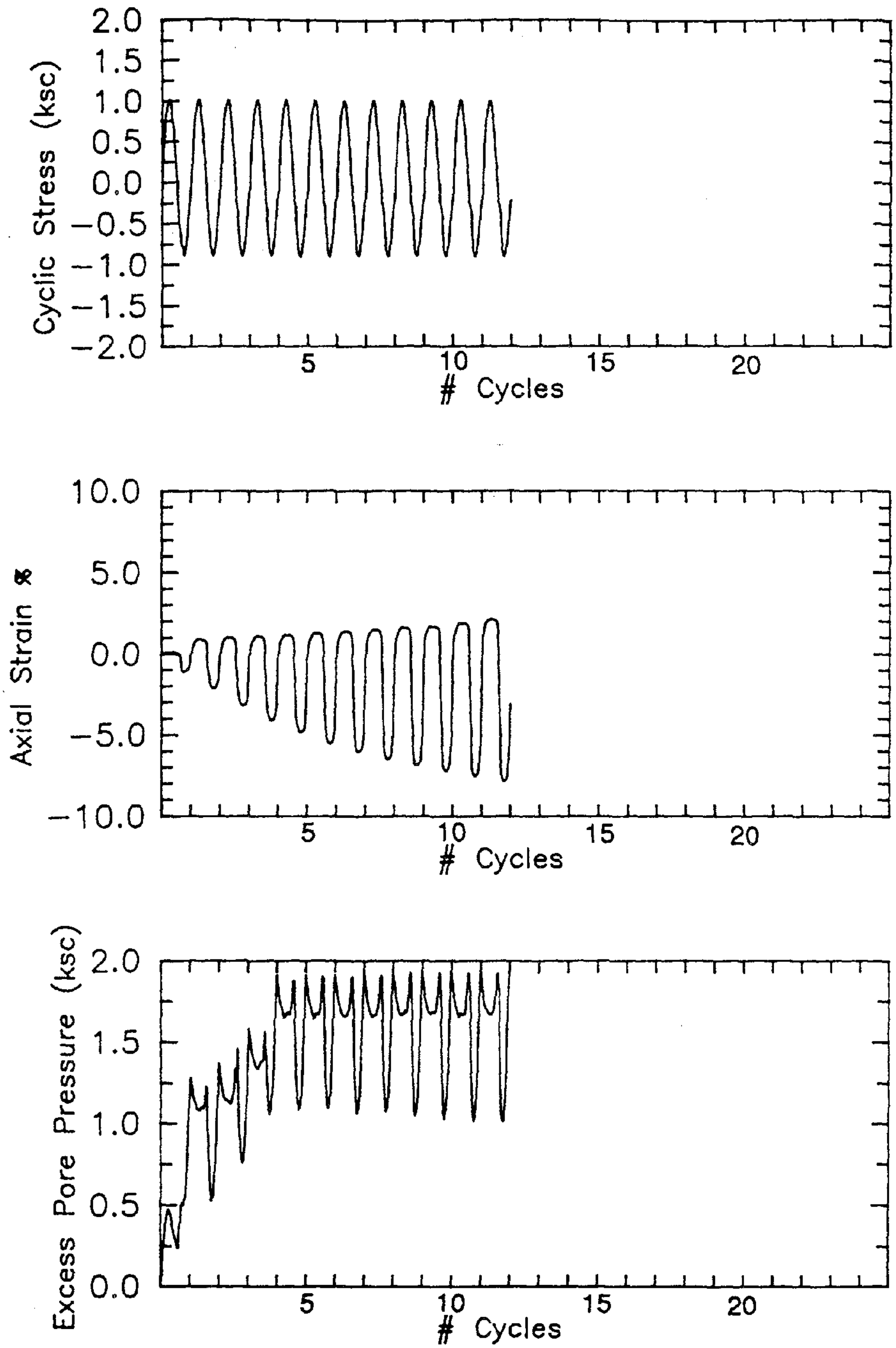


Figure 6.30: Cyclic Triaxial Test No. PT-53 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 51\%$)

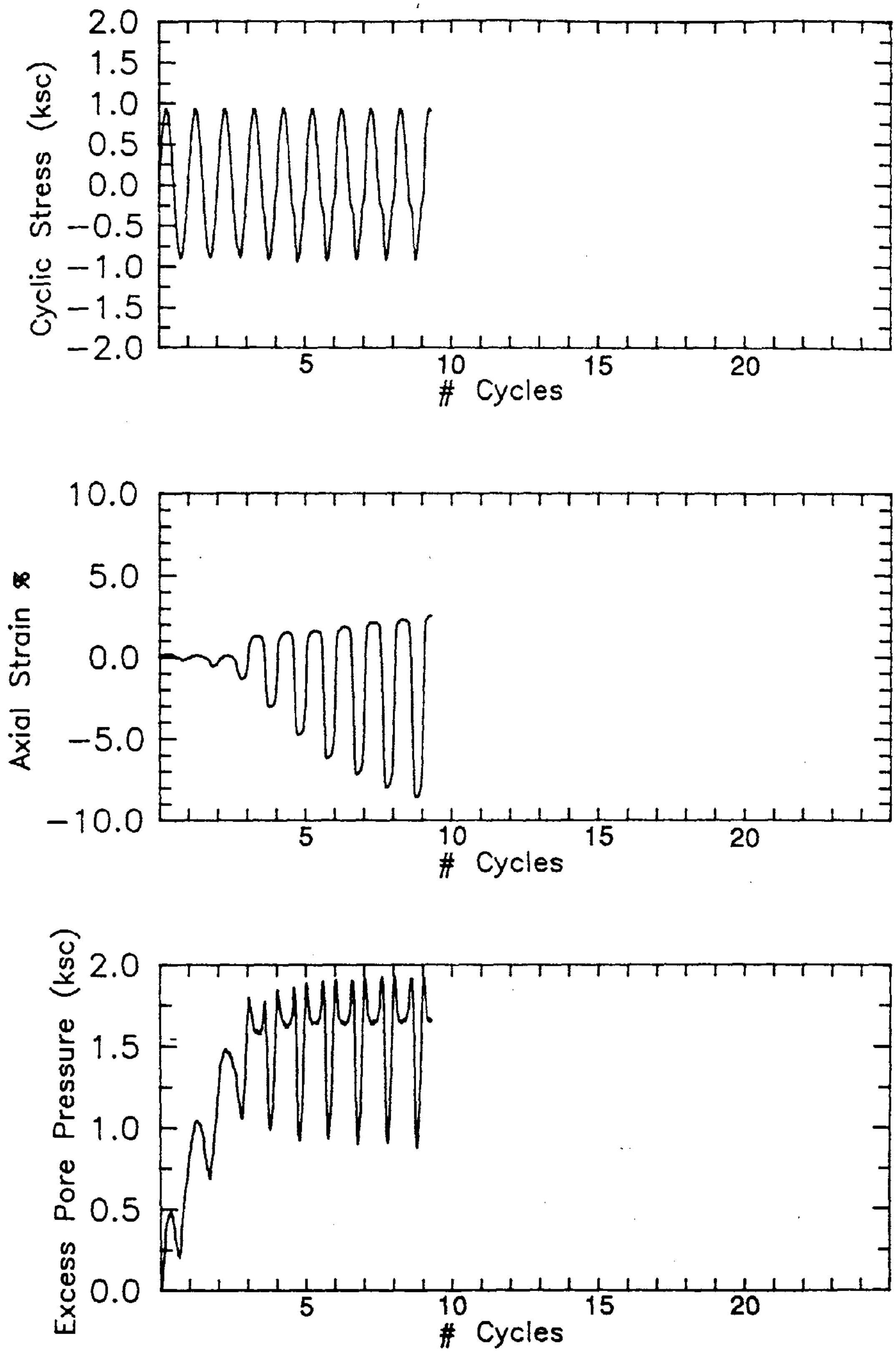


Figure 6.31: Cyclic Triaxial Test No. PT-40 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 52\%$)

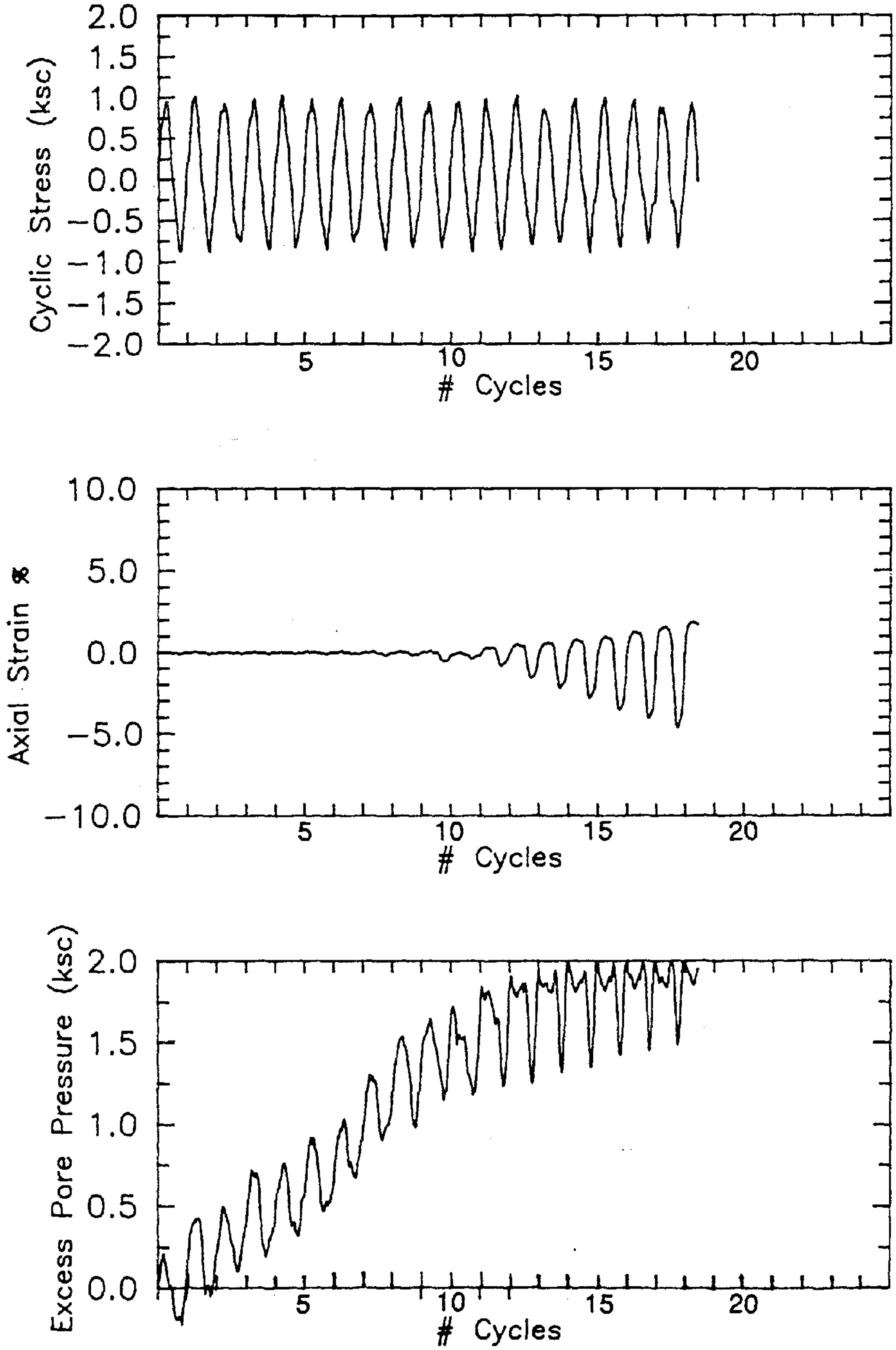


Figure 6.32: Cyclic Triaxial Test No. PT-67 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 50\%$)

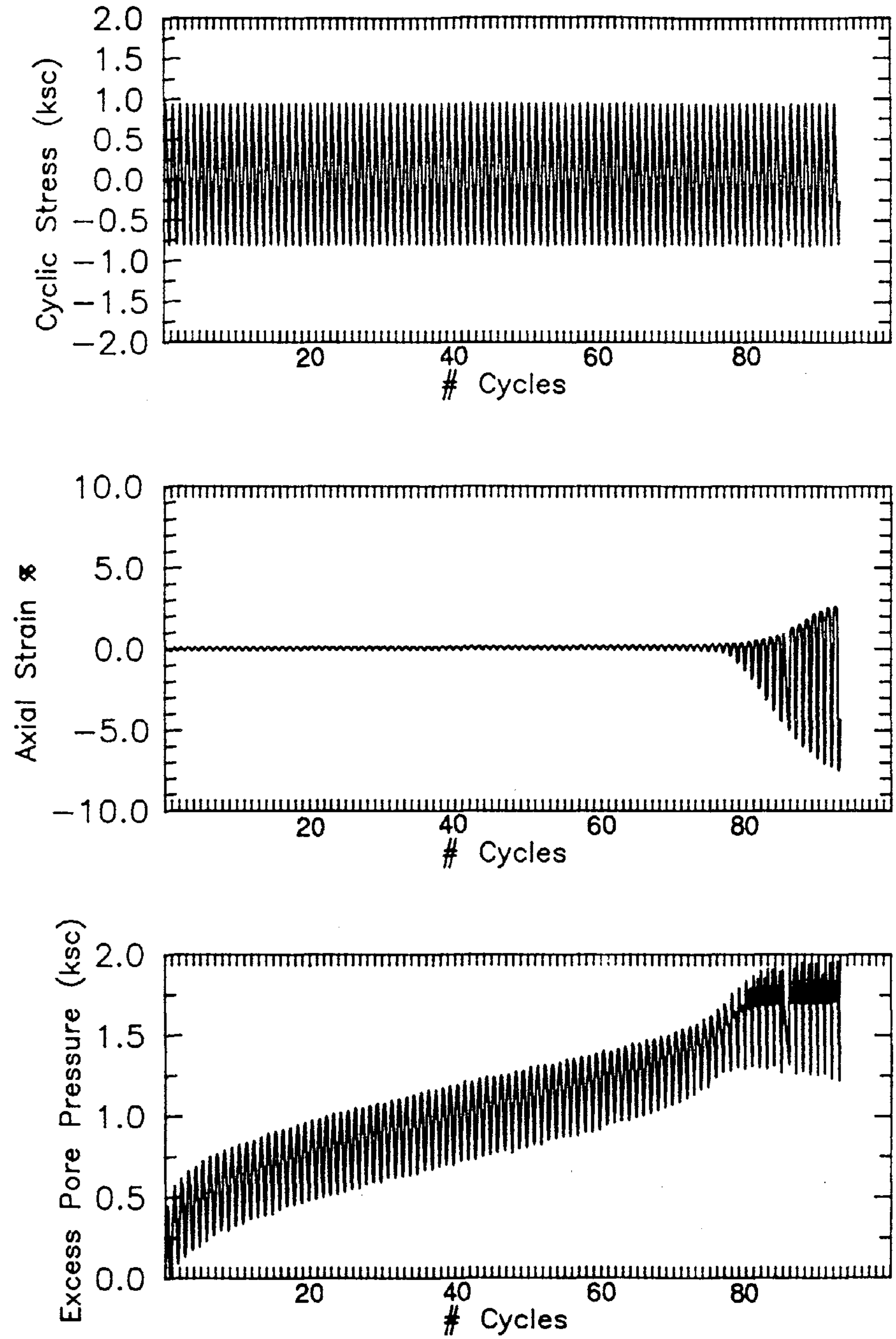


Figure 6.33: Cyclic Triaxial Test No. PT-41 (PT-Gravel With Membrane Compliance Mitigation, $D_R \approx 52\%$)

(1962) using the elasticity modulus determined in laboratory tests by Banerjee et al. (1979).

The rate of cyclic loading for 12-inch diameter samples was typically 1 cycle per minute. This slower rate of loading allowed for more accurate and even loads to be applied to the specimens. Equally important, this also allowed for more uniform distributions of pore pressures generated during undrained testing, and facilitated even injection/removal for compliance mitigation. Previous studies have shown that the cyclic strength of granular materials is not significantly influenced by the frequency of cyclic loading (Lee and Fitton, 1969; Wong et al., 1974; Seed and Anwar, 1986).

A composite plot of the cyclic test results is shown in Figure 6.34 as a pair of cyclic strength curves for the two sets of test data (tests performed with and without injection-mitigation). The cyclic stresses for each test are represented in this plot as cyclic stress ratio or CSR, which is equal to the average single amplitude peak cyclic deviator stress divided by twice the initial effective confining stress. Comparing the results of the cyclic tests performed with and without compliance mitigation, it can be easily seen that the uncorrected tests greatly overestimate the cyclic loading resistance of the gravel at this density. The error, in terms of cyclic stress ratio necessary to induce liquefaction in a given number of cycles, is on the order of nearly 25%. The relative importance of this error in dynamic strength evaluation is realized when anticipated dynamic loadings from possible earthquakes are compared to the cyclic strength curves. Given the geology and potential seismicity for any given location, the maximum expected degree of ground shaking can be derived. This amount of ground shaking may be defined as a dynamic shear stress applied for a certain duration which may be interpreted as a number of equivalent cycles applied in a dynamic test. The cyclic strength curves

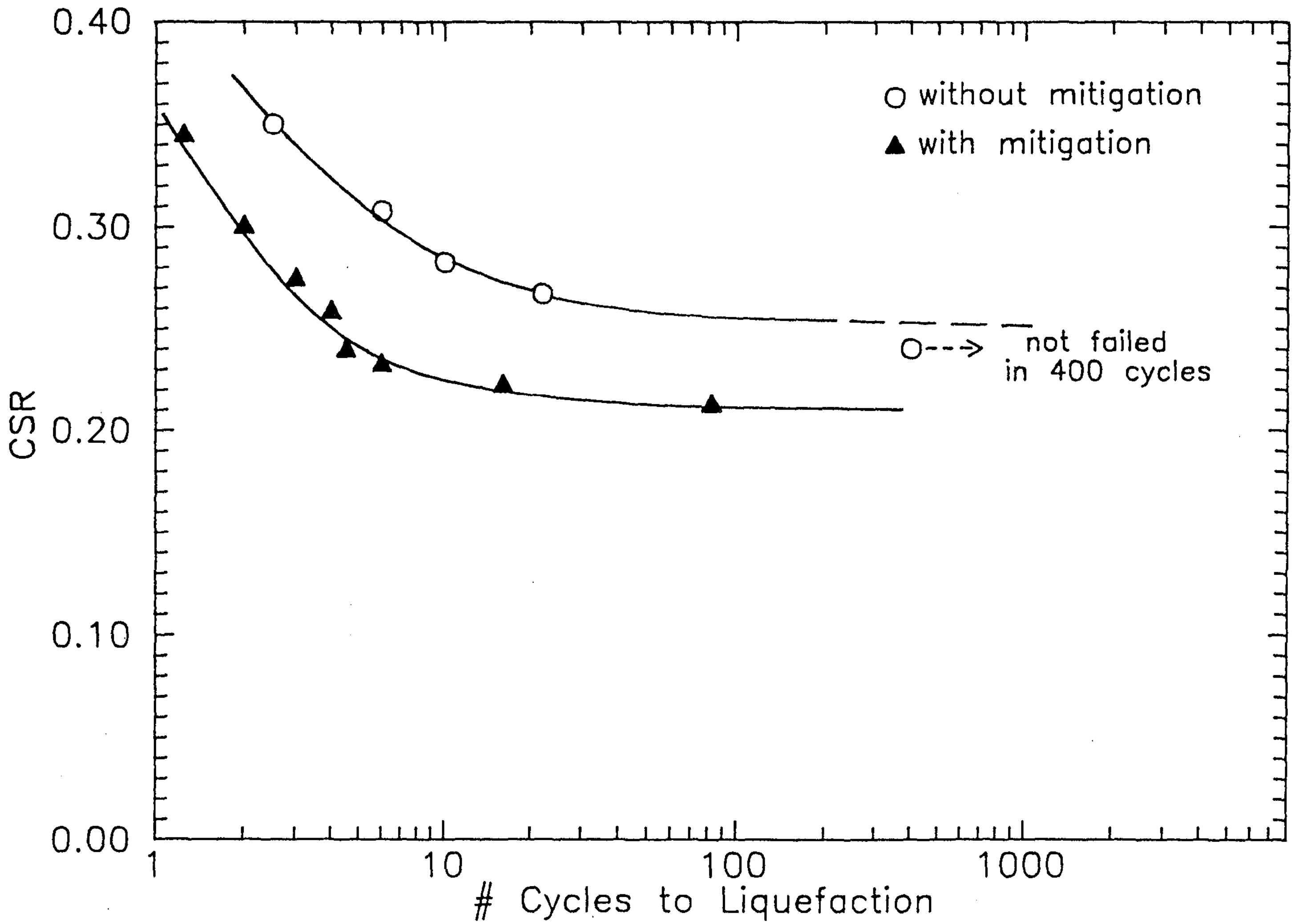


Figure 6.34: <u>Results of Isotropically Consolidated-Undrained Cyclic Triaxial Tests on Samples of PT-Gravel at $D_R \approx 50\%$ With and Without Mitigation of Membrane Compliance Effects</u>

divide the field of applied stresses and duration of loads into two categories. Points that plot above the curve are considered "unsafe" while those below are considered "safe". Furthermore, the minimum acceptable factor of safety for analysis and design of earth structures constructed of or upon this type of material, again in terms of cyclic stress ratio (CSR), is typically on the order of 1.3 to 1.4. Given the error in cyclic strength observed in this study, it can be seen that a great majority of that factor of safety will be consumed by unconservative compliance-induced testing error, making those designs and analyses much more critical. Noting that the material used for the testing performed in this study had a normalized compliance value in the mid-range of gravel sizes, it may be assumed that for coarser materials the unconservative error in cyclic strength may be considerably greater.

Looking at the pore-pressure generation curves for both uncorrected and "corrected" cyclic tests, it appears that the tests performed with the use of the compliance mitigation system generated curves similar in shape to those performed without the injection correction, suggesting that the injection system is giving "corrected" test data representative of the soil behavior.

The accuracy and reliability of the computer-controlled injection correction system devised for use with conventional sized (2.8-inch diameter) samples was verified by the use of large-scale (12-inch diameter) specimens as described in Chapter 4. Unfortunately, this type of experimental verification is unrealistic for the "corrected" large-scale tests due to the necessary size of the samples that would be needed to achieve the same relative increase in sample size as was possible between 2.8-inch and 12-inch diameter samples. In lieu of this type of physically verified accuracy, an analytical check was made by comparing the results of the cyclic triaxial tests performed on 12-inch diameter samples of the PT-gravel with and without the use of the injection-correction system, to the analytical prediction made by Martin et al. (1975) regarding the likely error (in terms of CSR) due to membrane compliance as a function of sample geometry (size and diameter) and soil gradation (D_{50}). The model proposed by Martin et al. predicts the necessary correction of cyclic stress ratio for the stresses necessary to cause liquefaction of the samples in 30 stress cycles as a function of mean grain sizes (D_{50}) of uniformly graded soils, for different sized samples.

The cyclic stress ratio correction calculated for the medium uniformly-graded gravel used in this study is plotted in Figure 6.35. It can be seen from this plot that the experimentally observed results from the tests performed with implementation of the computer-controlled injection correction system are in very good agreement with the analytically predicted correction proposed by Martin et al. This finding provides two mutually complementary benefits. The agreement between these two very different approaches to obtaining "correct" cyclic test results for a material which is prone to significant membrane compliance induced testing errors gives further backing to the accuracy and correctness of the physical testing correction. In addition, the "correct" test results obtained by using the computer-controlled injection correction system, whose fundamental accuracy was verified by virtue of the comparison between 2.8-inch and 12-inch diameter samples of Monterey 16 sand, provide a viable check of the analytical correction prediction.

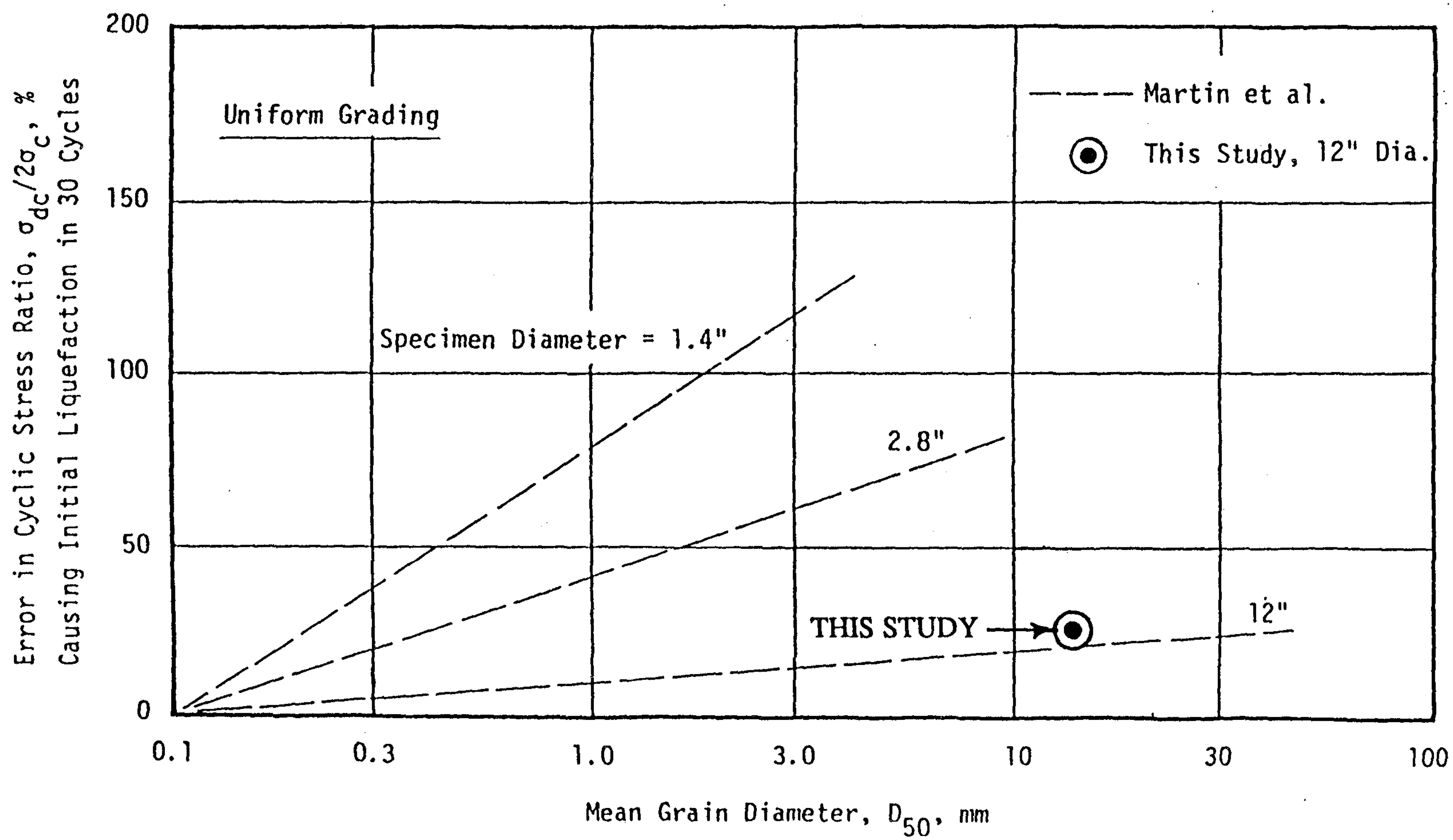


Figure 6.35: Comparison Between Laboratory-Determined Errors in Cyclic Stress Ratio Due to Membrane Compliance as a Function of Mean Grain Size for Various Specimen Diameters vs. Theoretical Error proposed by Martin et al., 1978

CHAPTER 7

RESEARCH SUMMARY AND CONCLUSIONS

The research program has represented the second phase of a two-stage program intended to develop techniques for fully and continuously eliminating the adverse effects of membrane compliance in undrained monotonic and cyclic testing of coarse, gravelly soils. Numerous investigators have worked on various aspects of this problem over the past 32 years, and many of their findings contributed significantly to the results obtained by these present studies.

The first phase of this research effort, reported in detail by Anwar et al. (1989), involved development and implementation of a technique for elimination of membrane compliance effects in testing of conventional ("small scale") 2.8-inch diameter triaxial samples. Based on review of previous work, the methodology selected for development and implementation consisted of: (1) predetermining membrane compliance magnitude as a function of sample geometry and changes in effective confining stress, and (2) use of a computer-controlled system to continuously inject or remove water from the "undrained" sample to precisely offset the volumetric error induced by membrane compliance.

The second phase of these studies, reported herein, consisted of two basic tasks: (1) verification of the accuracy and reliability (or "correctness") of the injection-mitigation techniques developed in the first phase of this research effort, and (2) development and implementation of a testing system using these same techniques to fully and continuously mitigate membrane compliance effects in "large-scale" undrained testing of coarse, gravelly soils. Both of these objectives were successfully achieved, and the resulting testing system appears to be the first to successfully perform accurate, unbiased undrained monotonic and cyclic loading tests on coarse, gravelly soils.

Verification of the small-scale injection/correction methodology developed and reported by Anwar et al. (1989) was achieved by repeating the tests on 2.8-inch diameter samples of sand performed by Anwar et al. with and without mitigation of compliance effects. The repeated tests, however, were performed using 12-inch diameter samples, at which size scale effects rendered potential membrane compliance insignificant for the sandy soil tested. The results of 12-inch diameter monotonic and cyclic tests were found to be in excellent agreement with the corresponding "injection-corrected" tests on 2.8-inch diameter samples, providing strong support for the accuracy of these injection techniques.

As the second stage of these current studies, a large-scale computer-controlled system was then developed and used to mitigate membrane compliance effects in performing undrained tests on 12-inch diameter samples. Both monotonic and cyclic tests were then performed on 12-inch diameter samples of a medium gravel. Essentially identical tests of both types were performed both with and without use of the computer-controlled injection system. The significant differences between the "corrected" and uncorrected test results clearly demonstrated the potential importance of compliance in this type of testing. Moreover, the apparent compliance-induced error in the uncorrected (unmitigated) cyclic tests was found to be in good agreement with that predicted theoretically by Martin et al. (1975), and this was taken as providing additional support for the accuracy and validity of the large-scale injection/mitigation system developed.

A necessary corollary part of both phases of this research effort was the demonstration that: (a) volumetric membrane compliance could be accurately and reliably pre-determined, and (b) that it was a repeatable function of effective confining stress. This too, was successfully accomplished, and it was further demonstrated that compliance magnitude could be strongly correlated to the soil grain size index D_{20}. A considerable number of soils were tested, and a relationship between normalized unit membrane compliance (S) and D_{20} for both sandy and gravelly soils was developed.

The techniques and procedures developed in these studies provide tools for performing large-scale undrained triaxial tests on coarse soils without any adverse impact from membrane compliance. Such tests may be expected to be useful directly in evaluating the undrained loading behavior of such soils, and also for developing data for correlation with large-scale in situ penetration tests (e.g. Becker Hammer) for development of empirical techniques for evaluation of the in situ liquefaction resistance of coarse, gravelly soils.

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