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16. Abstract

This report describes results of a series of laboratory tests performed to understand better the strength properties of highly plastic clays used in the construction of embankments in Texas. Previous studies showed that such embankments failed by sliding many years (10-30) after construction and that the apparent shear strengths in the field were substantially lower than the long-term shear strengths determined on the basis of laboratory tests on compacted specimens. This report presents the laboratory procedures and results for several series of tests in which specimens of compacted Beaumont clay were subjected to repeated cycles of wetting and drying and then sheared to measure the shear strength parameters in terms of effective stresses. Results of the tests show that repeated wetting and drying can produce a significant reduction in the effective stress cohesion intercept.

Factors of safety were calculated using the reduced laboratory strengths for an embankment which had failed. Although the computed factors of safety were still somewhat greater than unity, even using the reduced shear strengths due to wetting and drying, the factors of safety were significantly reduced by the effects of wetting and drying. Although the factors of safety were still somewhat higher than unity, this may have been caused by small amounts of scatter and uncertainty in the data, which is especially critical for the relatively low cohesion values involved (less than 100 psf). Further laboratory testing is needed to more fully reproduce and understand the effects of wetting and drying on the long-term strength properties of highly plastic clays.

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by

Laura E. Rogers
Stephen G. Wright

Research Report 436-2F

Development of Design Practice and Procedures for Determining Embankment Slope Stability
Research Project 3-8-85-436

conducted for

Texas State Department of Highways
and Public Transportation

in cooperation with the
U.S. Department of Transportation
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by the

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The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

PREFACE

The Texas State Department of Highways and Public Transportation has experienced a number of slope failures in embankments constructed of highly plastic clays. The failures typically occur a number of years after construction and are associated with the embankment itself, rather than any feature of the foundation. Stauffer and Wright (1984) studied a number of these failures and, using strength data obtained by Gourlay and Wright (1984), showed that the shear strength measured in the laboratory on compacted specimens was significantly higher than the strength that was developed in the field. In response to this observation Research Study 3-8-85-436 was initiated to understand better the reasons for the discrepancies between the field and laboratory strengths and to develop a rational basis for estimating shear strengths for design, prior to construction.

The first phases of Research Study 3-8-85-436 included laboratory shear tests on both undisturbed specimens and specimens prepared in the laboratory by a variety of means. Results of this first phase of the laboratory testing program are presented by Green and Wright (1986). They found that in all cases the shear strengths measured in the laboratory were generally in excess of those which were apparently developed in the field. As a consequence of Green and Wright's observations an additional series of tests was initiated in which specimens were subjected to various amounts of wetting and drying prior to shear testing. The results of the testing program involving the effects of wetting and drying are presented in this report.

ABSTRACT

This report describes results of a series of laboratory tests performed to understand better the strength properties of highly plastic clays used in the construction of embankments in Texas. Previous studies showed that such embankments failed by sliding many years (10-30) after construction and that the apparent shear strengths in the field were substantially lower than the long-term shear strengths determined on the basis of laboratory tests on compacted specimens. This report presents the laboratory procedures and results for several series of tests in which specimens of compacted Beaumont clay were subjected to repeated cycles of wetting and drying and then sheared to measure the shear strength parameters in terms of effective stresses. Results of the tests show that repeated wetting and drying can produce a significant reduction in the effective stress cohesion intercept.

Factors of safety were calculated using the reduced laboratory strengths for an embankment which had failed. Although the computed factors of safety were still somewhat greater than unity, even using the reduced shear strengths due to wetting and drying, the factors of safety were significantly reduced by the effects of wetting and drying. Although the factors of safety were still somewhat higher than unity, this may have been caused by small amounts of scatter and uncertainty in the data, which is especially critical for the relatively low cohesion values involved (less than 100 psf). Further laboratory testing is needed to more fully reproduce and understand the effects of wetting and drying on the long-term strength properties of highly plastic clays.

SUMMARY

Previous experience has shown that the shear strength of highly plastic clays in compacted earth embankments is much lower in the field than laboratory tests suggest. In order to understand better the reasons for the discrepancies between field and laboratory strengths, procedures were developed for simulating effects of wetting and drying and determining the effects on effective stress shear strength parameters. Specimens with the dimensions of both direct shear- and triaxial-size were compacted and subjected to alternating 24 hour periods of soaking in water and drying in an oven, after which either consolidated-drained direct shear tests or consolidated-undrained triaxial shear tests with pore water pressure measurements were performed. The number of cycles of wetting and drying was varied from one to thirty.

Specimens of direct shear size, especially, revealed a significant reduction in the effective stress cohesion intercept after specimens were subjected to wetting and drying cycles. The reduction in strength explains a significant portion of the discrepancy between field and laboratory strengths for Beaumont clay. Specimens of triaxial size showed negligible reduction in the effective stress shear strength parameters due to wetting and drying. However, this may have been due to the relatively short period of time allowed for wetting and drying and the manner in which the triaxial specimens were confined during wetting and drying. Additional research would be useful to develop more fully appropriate test procedures for subjecting specimens to repeated wetting and drying and to examine the effects of wetting and drying for highly plastic soils other than Beaumont clay.

IMPLEMENTATION STATEMENT

Results presented in this report indicate that repeated wetting and drying can cause significant reduction in the effective stress shear strength parameters for compacted highly plastic clays. Although effects of wetting and drying on undrained shear strengths are well-recognized, the effects of wetting and drying on the effective stress (or "drained") shear strength parameters are not generally recognized by geotechnical engineers. In the instance of embankments constructed of highly plastic clay in Texas, the effects of wetting and drying at least partially explain the discrepancies between the shear strengths measured by conventional procedures employed in geotechnical engineering and the shear strengths which can actually be developed in the field. Although reliable laboratory test procedures do not yet exist to enable the shear strength which can be relied upon in the field to be predicted for long-term stability of embankments constructed of highly plastic clays, it is evident that the cohesion intercept should not be relied upon for design in highly plastic clays like the Beaumont clay when they are subjected to repeated wetting and drying.

Further research is recommended to develop better procedures for simulating in the laboratory the long-term effects of wetting and drying in the field and to determine the effects of wetting and drying on clays other than Beaumont clay. Information from such research is needed to enable rational predictions to be made of fundamental strength properties for embankment design.

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CHAPTER 1. INTRODUCTION

The Texas State Department of Highways and Public Transportation has experienced a number of failures in embankments constructed of highly plastic clays. Typically such failures occur at times ranging from 10 to 30 years after construction. Failures of this type are especially frequent in embankments constructed of the "Beaumont" clay, a highly plastic clay found in southeast Texas. Effective stress shear strength parameters for the "Beaumont" clay have been measured in previous laboratory studies and have been found to correspond to significantly higher strengths than those back-calculated for field conditions at the time of failure. This report presents the results of a laboratory testing program which was initiated in January of 1985 to assess the possibility that the observed discrepancy between laboratory and apparent field strengths of the highly plastic "Beaumont" clay is due to repeated cycles of wetting and drying which occur in the field.

Preliminary studies were necessary to develop procedures for preparation of compacted specimens and subjecting the specimens to repeated cycles of wetting and drying. The procedures and effects of wetting and drying on the moisture content and dry density of specimens are summarized in Chapter 2. Direct shear tests and consolidated-undrained triaxial shear tests were performed on specimens subjected to repeated cycles of wetting and drying; results from direct shear and triaxial shear tests are presented in Chapters 3 and 4, respectively. The effective-stress shear-strength parameters obtained from the direct shear and triaxial tests are compared to each other and to values measured previously in the laboratory for compacted specimens which were not subjected to wetting and drying in Chapter 5. Conclusions are presented in Chapter 6.

CHAPTER 2. PRELIMINARY STUDIES OF EFFECTS OF SAMPLE PREPARATION PROCEDURES AND WETTING AND DRYING

INTRODUCTION

All of the soil tested in this study was obtained from the site of the slope failure at the intersection of Scott Street and I.H. 610 in Houston, Texas. Actually, two soils existed at this site, one a reddish brown clay, the other more grey in color. The soil tested in the current study consisted of what has been termed the "red" clay, which was classified by Gourlay and Wright (1984) as highly plastic (CH under the Unified Soil Classification System) with an average liquid limit of 70 percent and an average plastic limit of 20 percent; the average plasticity index is 50 percent. The specific gravity of the clay is 2.69 as measured by Green and Wright (1986).

Methods of specimen preparation, procedures for subjecting the specimens to wetting and drying, and the effects of the wetting and drying on the moisture content and dry density are presented in this chapter. Specimens of appropriate dimensions for testing in the direct shear devices and in triaxial devices were both considered and are discussed separately in the following sections.

DIRECT-SHEAR-SIZE SPECIMENS

Several series of preliminary tests were performed to establish appropriate procedures for preparing specimens and subjecting them to cycles of wetting and drying. The effects of wetting and drying were first examined on compacted specimens with a height of 0.816 inches and a diameter of 2.5 inches, which were the appropriate dimensions for use in the direct shear device. It was felt that specimens could be more readily set-up and tested in the direct shear device than in the triaxial device after wetting and drying. Specimens could be directly extruded from a trimming mold into the metal box of the direct shear device, while a triaxial specimen would require additional handling and placement in a rubber membrane after trimming. The integrity of specimens after wetting and drying was not known and, thus, the direct shear device was initially selected as the primary device for testing.

At the beginning of the current laboratory testing program, the clay was air dried, pulverized and passed through a #40 U. S. Standard Sieve Series brass sieve. Subsequent soil preparation procedures consisted of mixing the soil to a specific moisture content followed by various procedures intended to break-down large lumps of soil which formed in the mixing process. The breaking down of these lumps was necessary to produce uniform specimens when compacted. Due to the "Beaumont" clay's high plasticity and tendency of the soil to form into lumps when mixed with water, obtaining uniform compacted specimens with a minimum of void space is difficult.

Green and Wright (1986) previously processed the moist soil after mixing by forcing the soil by hand through a #40 brass sieve at a moisture content similar to the one used for later compaction. This method of soil preparation was awkward and time-consuming. Accordingly, additional methods of soil preparation were examined as part of the current study.

Two methods of soil processing after mixing and prior to compaction were developed. The initial method consisted of extruding soil through a perforated aluminum plate, while the second method entailed forcing the clay through a stainless steel screen with the same size openings as a U.S. Standard Sieve Series #40 sieve secured in an aluminum frame fabricated specifically for this purpose.

Extruded Soil Specimens

The first processing procedure consisted of extruding moist soil through a perforated aluminum plate 0.25 inches thick using a hydraulic ram. Prior to extrusion, pulverized dry soil passing the #40 sieve was mixed to a moisture content of approximately 30 percent; through trial and error, it was found that the soil could not be extruded at a moisture content lower than 30 percent. After 24 hours of curing in a humid room, the moist soil was forced through the aluminum plate containing numerous 1/32 inch diameter holes. A hydraulic ram was used to force the soil through the plate; the extruded soil was then cut into strands 0.5 inches in length and allowed to dry to approximately the desired moisture content of 24 percent for compaction.

The size of the soil threads created with the extrusion process was approximately twice as large as that produced by passing moist soil through the #40 sieve. Despite the larger size, the specimens obtained using the extrusion process appeared to be satisfactorily uniform; the specimens did not possess any large voids that are customarily present in specimens compacted with "Beaumont" clay, which has not been processed after mixing with water. Both the time period required and the physical energy necessary for soil preparation were significantly reduced by extruding the wet soil through the plate. However, this method was eventually discarded for reasons which are presented in a later section of this chapter.

Compaction Procedures. Specimens were compacted in a 2.5-inch diameter mold specially designed for compacting direct shear specimens. A diagram of the mold is presented by Green and Wright (1986). Values of dry density (96.3 pcf) and optimum moisture content (24 percent) as determined by Gourlay and Wright (1984), based upon the Texas SDHPT Test Method Tex-113-E, were used as "target" values.

A 2.15-pound hammer with an acrylic cylindrical face 2.4 inches in diameter was utilized to compact the specimens. The soil was compacted in four equal lifts using six drops of the hammer at a height of 12 inches. The compactive effort generated in this manner was 9.66 ft-lb/cu.in. (16,695 ft-lb/cu.ft.). Each lift contained approximately 42 grams of moist soil. The surfaces of each of the first three lifts were scarified prior to the addition of each subsequent lift.

Once compaction was complete, the mold was disassembled and the specimen was placed in a stainless steel ring with an inside diameter of 2.5 inches and a height of 0.816 inches. The top of the specimen, consisting of most of the fourth lift, was then removed by trimming flush with the top of the ring using a knife and wire saw. The soil trimmings were used to measure the as-compacted moisture content. By using essentially three lifts for the final specimen height, the shear plane which would be induced through the middle of the specimen in the direct shear test would not coincide with a lift boundary and produce unrealistic shear strengths. All of the specimens had compacted dry densities and moisture contents which were within 2 pcf and 2 percent, respectively, of the "target" values.

Wetting and Drying Procedures. A special chamber was fabricated to maintain the direct shear specimen's approximate shape during wetting and drying so that it could later be tested in the direct shear device. A diagram of this chamber is shown in Figure 2.1. The chamber consists of an acrylic cylinder with an inside diameter of 2.55 inches and a height of 2.5 inches; this cylinder is secured between two circular acrylic end-plates with three threaded rods and nuts.

Once each specimen was compacted and trimmed, it was removed from the metal ring and placed in the special chamber preparatory to beginning the wetting and drying cycles. A porous stone was placed at each end of the specimen and the entire device was secured. Specimens remained in this chamber throughout the cycles of wetting and drying; they were not removed until they were trimmed, just prior to shear testing.

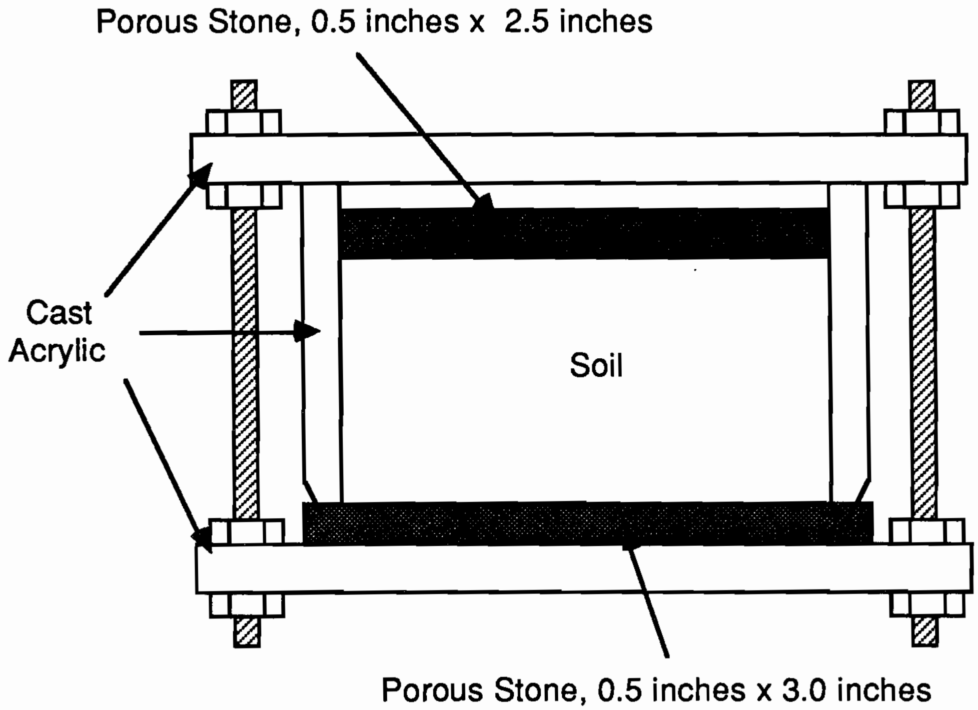

Figure 2.1. Chamber for Subjecting Direct-Shear-Size Specimens to Wetting and Drying

Wetting and drying procedures were developed through experimental trial and error. All wetting and drying to which specimens were subjected began with an initial wetting phase by immersing the specimen in a tub of distilled water. Water could enter the specimen through both ends of the chamber through the two porous stones. The moisture content increased from approximately 24 percent to a final value of approximately 36 percent in 24 hours, representing an average increase in moisture content of approximately 50 percent of the initial value.

After an initial wetting phase, specimens were placed in an oven to dry. The temperature for drying was chosen to be 140 degrees Fahrenheit, which is the temperature at which soil-lime mixtures are dried in the Texas SDHPT Test Method Tex-121-E. The average moisture content after 24 hours of drying was approximately 12 percent, representing an average decrease in moisture content of 50 percent of the initial compaction moisture content.

The approximately 50 percent increase and decrease in moisture content during each of the two 24-hour phases of a wetting and drying cycle was considered to be an adequate change for the purpose of this study and was, thus, selected as the "standard" for the remainder of this study.

In the direct shear tests which were to be performed, it was desirable to have a degree of saturation as close to 100 percent as possible to prevent the presence of significant negative pore water pressures in the specimens. Accordingly, specimens would be set up in a wet condition. Specimens which were not subjected to wetting and drying would be wetted once. Similarly, a specimen subjected to "one cycle" of wetting and drying would be considered to be a specimen which was wetted for 24 hours, dried for 24 hours, and then re-wetted for 24 hours; subsequent cycles consisted of a drying phase followed by a wetting phase.

At the end of the final cycle of wetting and drying for each specimen, the specimen was extruded from the acrylic cylinder and trimmed into the same stainless steel ring used for trimming after compaction. A final weight and moisture content were determined. The dry weight was calculated from this moisture content. The specimen's dry density, degree of saturation and void ratio at the end of wetting and drying were estimated using this weight and the clay's measured specific gravity of 2.69.

Properties after Wetting and Drying. Moisture contents were first examined for six specimens which had been soaked for 24 hours. A range in final moisture contents of 35.6 to 38.1 percent was measured; all measured values are presented in Table 2.1. Significant radial and axial expansion were observed within the specimens. Radial expansion consisted of about 2.5 percent of the initial diameter and was limited to this value (2.5 percent) by the acrylic cylinder in which the

TABLE 2.1. MOISTURE CONTENTS OF SPECIMENS PREPARED FROM EXTRUDED SOIL AFTER ONE 24-HOUR WETTING PERIOD

Specimen Number	Moisture Content
WD1	35.6
WD2	35.7
WD3	38.1
WD4	37.4
WD5	38.2
WD6	37.3

specimen was held during the wetting phase. Axial expansion was much greater and was approximately 20 percent.

Moisture contents were also examined for four specimens at the end of 24 hours of wetting followed by 24 hours of drying. The average final moisture content was 12 percent; final moisture contents are presented in Table 2.2. Vertical cracks had developed within the specimens while some horizontal cracks developed along the compaction lift boundaries.

Additional specimens were subjected to one, two, three, four, six and fifteen cycles of wetting and drying. The as-compacted properties of these specimens prepared by the extrusion process are presented in Table 2.3; final properties after wetting and drying are presented in Table 2.4. Changes in moisture content, dry density, degree of saturation and void ratio were computed from the results in Tables 2.3 and 2.4 and are summarized in Table 2.5.

The initial and final dry densities for each specimen are plotted versus the number of cycles of wetting and drying to which each specimen was subjected in Figure 2.2. Corresponding changes (decreases) in density are plotted in Figure 2.3. While lower final densities were anticipated for those specimens which underwent the larger number of cycles of wetting and drying, the specimens which were subjected to the largest number of cycles (fifteen) actually had some of the highest final dry densities.

Initial and final moisture contents of the specimens are plotted against number of cycles of wetting and drying in Figure 2.4. A nearly constant change (increase) in moisture content, averaging 12.3 percent, was found to occur, regardless of the number of cycles of wetting and drying. Initial and final degrees of saturation are plotted versus the number of cycles of wetting and drying in Figure 2.5. A wide range of both initial and final degrees of saturation may be seen.

Based on the results of the wetting and drying tests on specimens prepared by the extrusion process, no trends in unit weight, moisture content or degree of saturation were observed with the number of cycles of wetting and drying. Significant scatter was also present in the data. Consequently, a second method of specimen preparation was examined.

Screen-Prepared Soil Specimens

The second procedure for preparation of soil for compaction consisted of forcing soil through a #40 sieve in a manner similar to that used by Green and Wright (1986). Eight specimens were compacted of soil prepared in this manner and subjected to one, three and ten cycles of wetting and drying, specifically to determine the effects of wetting and drying on the specimens'

TABLE 2.2. MOISTURE CONTENTS OF SPECIMENS FPREPARED FROM EXTRUDED SOIL AFTER ONE 24-HOUR WETTING PERIOD AND ONE 24-HOUR DRYING PERIOD

Specimen Number	Moisture Content
WD7	14.0
WD8	15.4
WD9	20.1
WD10	18.9

TABLE 2.3. INITIAL PROPERTIES OF SPECIMENS PREPARED OF EXTRUDED SOIL

Specimen Number	Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
WD11	1	24.7	98.2	94.0	0.72
WD12	1	24.4	96.9	89.1	0.73
WD13	2	23.7	96.0	84.6	0.76
WD15	2	24.7	95.2	86.5	0.77
WD14	3	22.2	96.5	80.4	0.75
WD16	4	23.4	95.4	82.3	0.83
WD17	4	24.3	96.2	87.0	0.75
WD18	6	25.3	98.1	95.2	0.72
WD19	15	23.2	95.4	81.5	0.77
WD20	15	26.0	97.5	96.3	0.73

TABLE 2.4. FINAL PROPERTIES OF SPECIMENS PREPARED FROM EXTRUDED SOIL

Specimen Number	Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
WD11	1	34.7	85.1	95.4	0.98
WD12	1	38.5	81.5	97.3	1.07
WD13	2	37.2	81.3	94.0	1.07
WD14	2	39.8	77.3	91.2	1.17
WD15	3	37.4	79.7	90.5	1.12
WD16	4	36.8	81.7	93.7	1.06
WD17	4	35.9	80.7	100.0	1.09
WD18	6	33.2	88.0	97.8	0.92
WD19	15	37.3	84.0	99.9	1.01
WD20	15	34.2	88.6	100.0	0.90

TABLE 2.5. CHANGE IN PROPERTIES OF SPECIMENS PREPARED FROM EXTRUDED SOIL AFTER WETTING AND DRYING

Specimen Number	Number of Cycles	Change in Moisture Content, percent	Change in Dry Density, pcf	Change in Degree of Saturation, percent	Change in Void Ratio
WD11	1	10.0	-13.2	2.2	0.27
WD12	1	14.1	-15.4	8.2	0.34
WD13	2	13.5	-14.7	9.4	0.31
WD14	2	15.1	-17.9	4.7	0.40
WD15	3	15.2	-16.8	10.1	0.40
WD16	4	13.5	-13.7	11.4	0.23
WD17	4	11.6	-15.5	13.0	0.34
WD18	6	7.9	-10.2	2.6	0.20
WD19	15	14.1	-11.4	18.5	0.24
WD20	15	8.2	-8.9	3.7	0.17

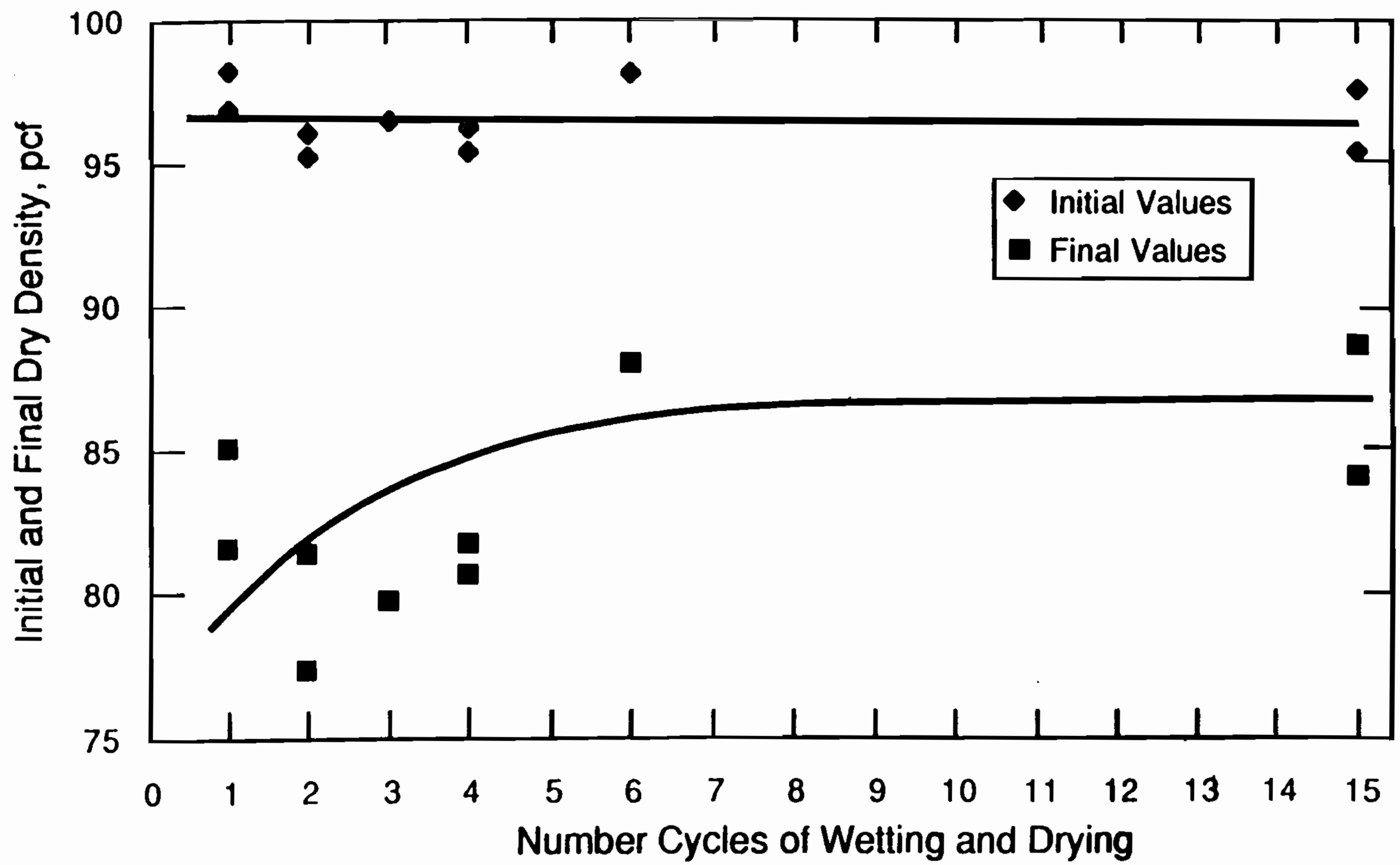

Figure 2.2. Initial and Final Dry Density vs. Number of Cycles of Wetting and Drying, Extruded Soil Specimens

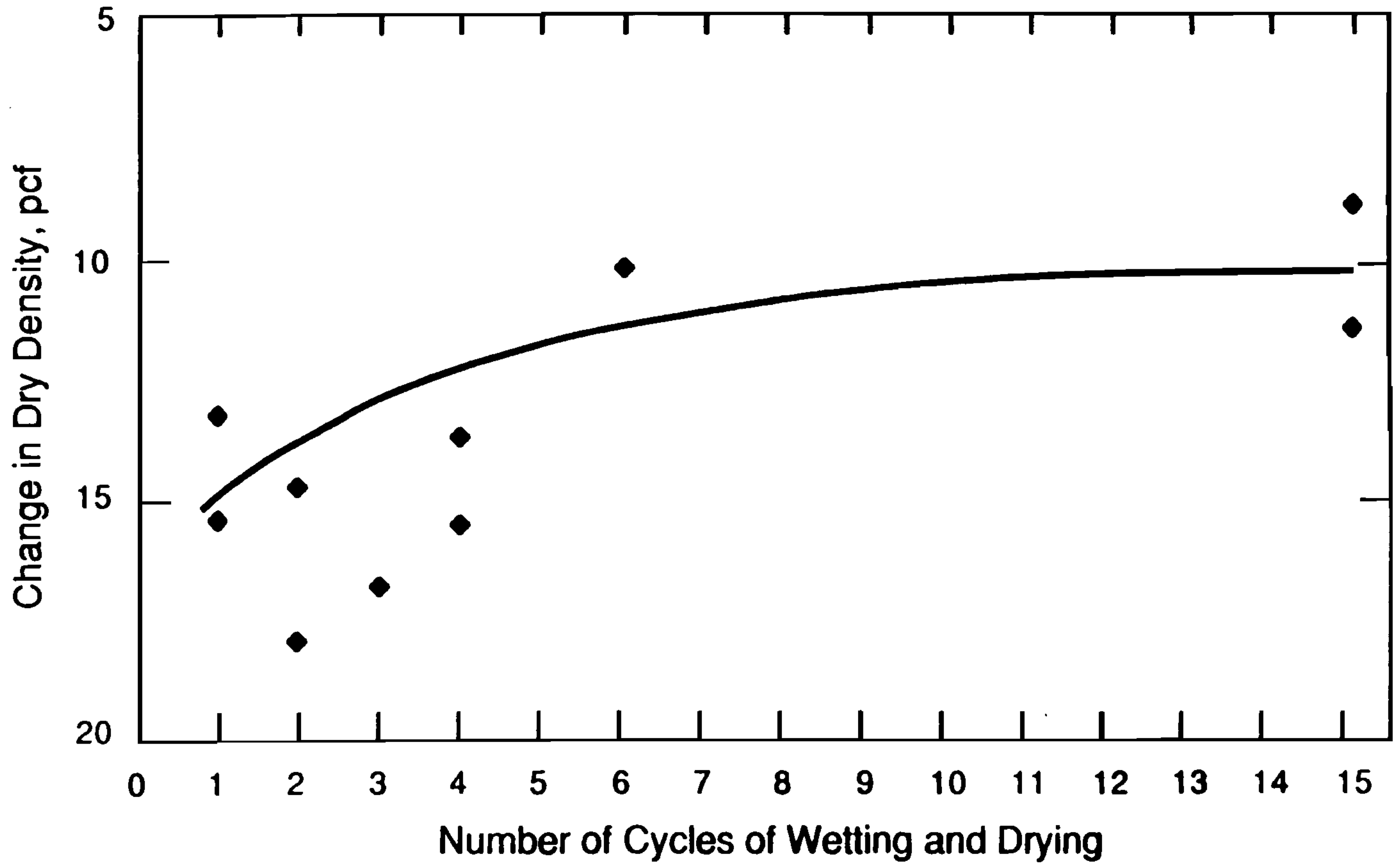

Figure 2.3. Change in Dry Density vs. Number of Cycles of Wetting and Drying, Extruded Soil Specimens

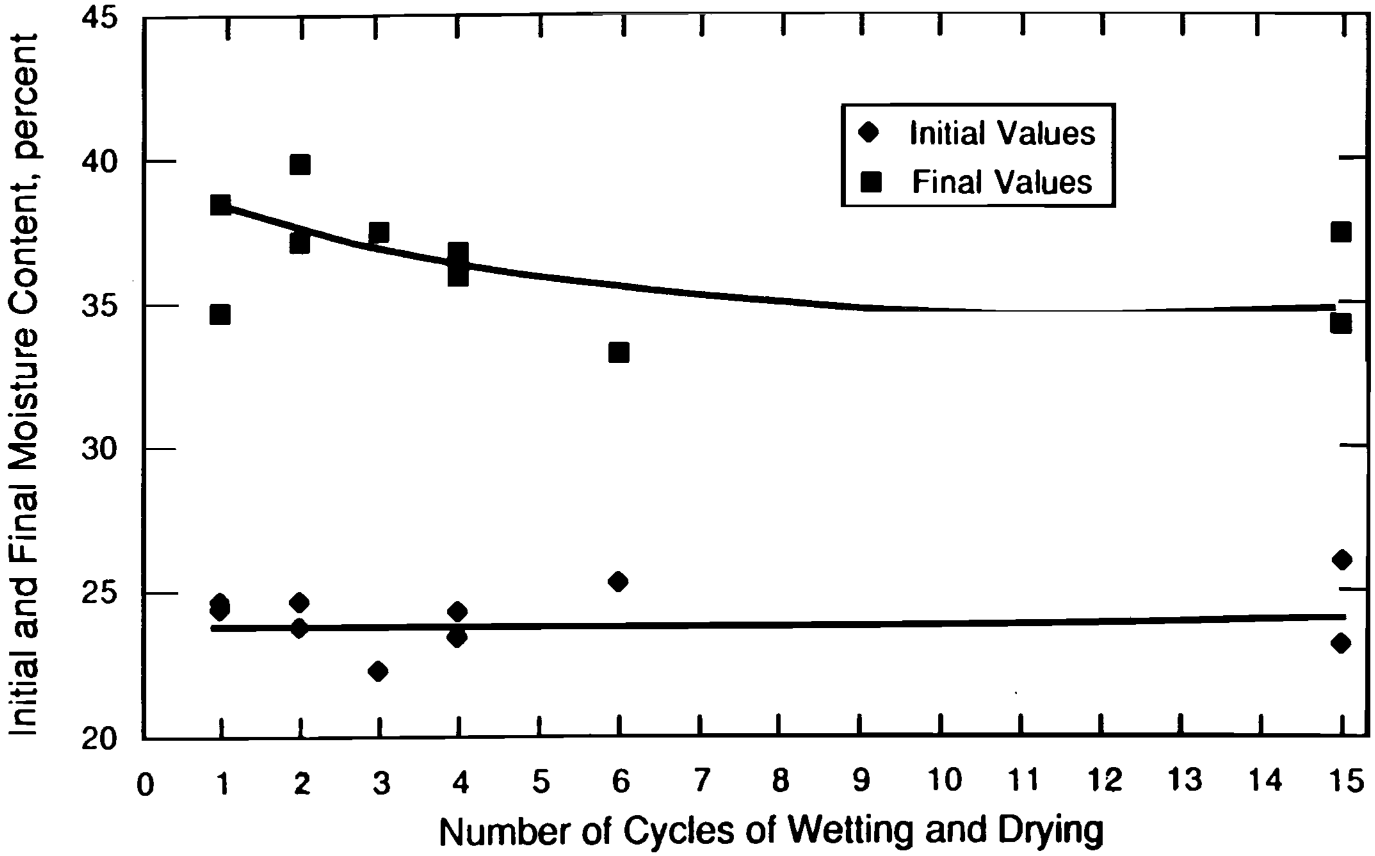

Figure 2.4. Initial and Final Moisture Content vs. Number of Cycles of Wetting and Drying, Extruded Soil Specimens

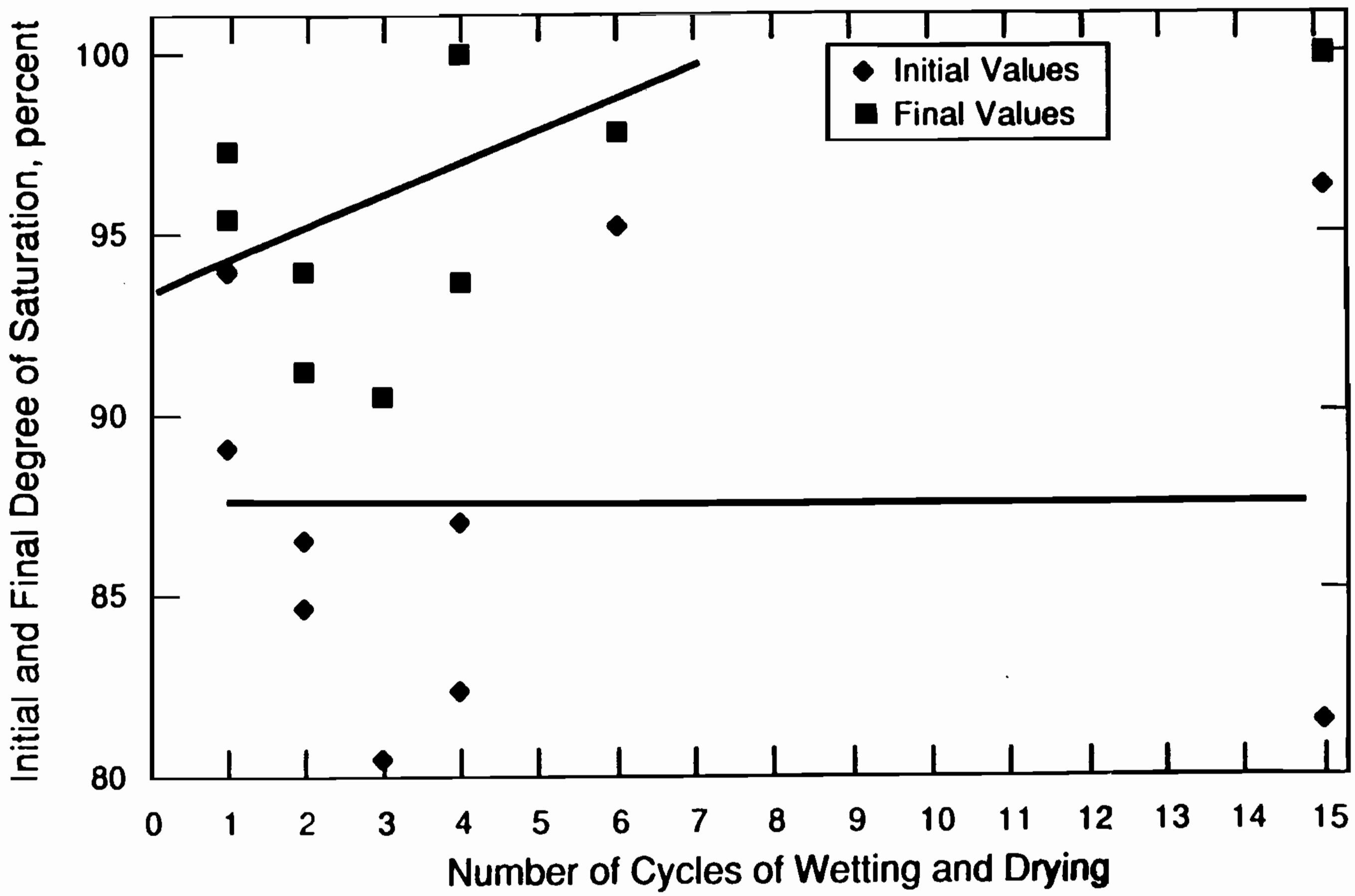

Figure 2.5. Initial and Final Degree of Saturation vs. Number of Cycles of Wetting and Drying, Extruded Soil Specimens

moisture content, dry density and degree of saturation; twenty-two more specimens were subjected to one, three, nine and thirty cycles of wetting and drying and subsequently sheared in direct shear devices. Specimens compacted from the screen-prepared soil were subjected to wetting and drying in the same manner used for the specimens compacted from the extruded soil.

Compaction Procedures. To expedite the process of preparing soil with the second procedure, a special framework was designed and fabricated. The frame is shown in Figures 2.6, 2.7 and 2.8 and consists of two aluminum rectangular frames bolted together securing a sheet of stainless steel screen with 40 openings per square inch supported by three crossbars in the lower frame; the workable surface area for forcing the moist soil through the screen consists of about one-third of a square foot.

When the soil was prepared by forcing through the screen, compaction procedures had to be changed from those used with the extruded soil to achieve the same target dry density of 96.3 pcf; the number of drops of the 2.15-pound hammer was decreased from six to four per lift, and the height of each drop was lowered from 12 to 10.5 inches; 5.64 ft-lb/cu.in. (9739 ft-lb/cu.ft.) of compaction energy was generated in this manner. Specimens compacted with these procedures attained the "target" values of dry density and moisture content.

Properties after Wetting and Drying. The as-compacted properties of the specimens are listed in Table 2.6, properties after wetting and drying are presented in Table 2.7, and changes in properties due to wetting and drying are summarized in Table 2.8.

Specimens which were sheared in the direct shear device following the cycles of wetting and drying were trimmed into a stainless steel ring in preparation for testing. The properties of the specimens at that time were calculated using the wet weight of the trimmed specimen, its volume based upon the ring dimensions, and the total dry weight of solids measured after the direct shear test was completed.

The initial and final dry densities for each specimen are plotted versus the number of cycles of wetting and drying in Figure 2.9. The corresponding change (decrease) in dry density for each specimen is shown versus the number of cycles in Figure 2.10. The densities appeared to decrease uniformly as the number of cycles increased; specimens subjected to the largest number of cycles of wetting and drying experienced the largest decrease in density.

The initial and final moisture contents are plotted versus the number of cycles in Figure 2.11. Final moisture contents were found to have average values of 35.3 percent, 35.4 percent, 35.9 percent, 38.8 percent and 38.0 percent for specimens subjected to 1, 3, 9, 10 and 30 cycles of wetting and drying respectivley. Green and Wright (1986) estimated the moisture content along

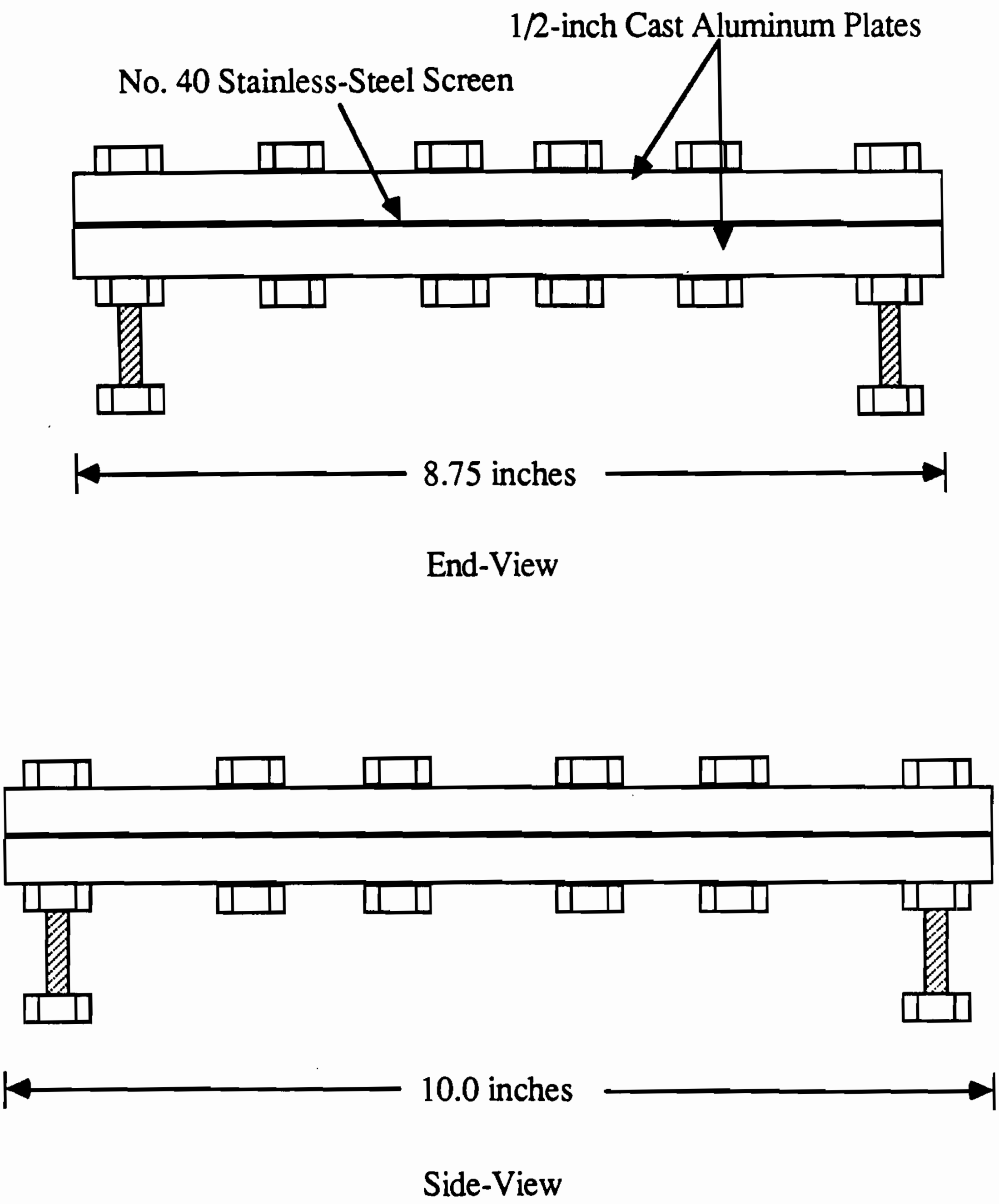

Figure 2.6. Profile Views of Frame Fabricated to Secure Screen for Processing Soil

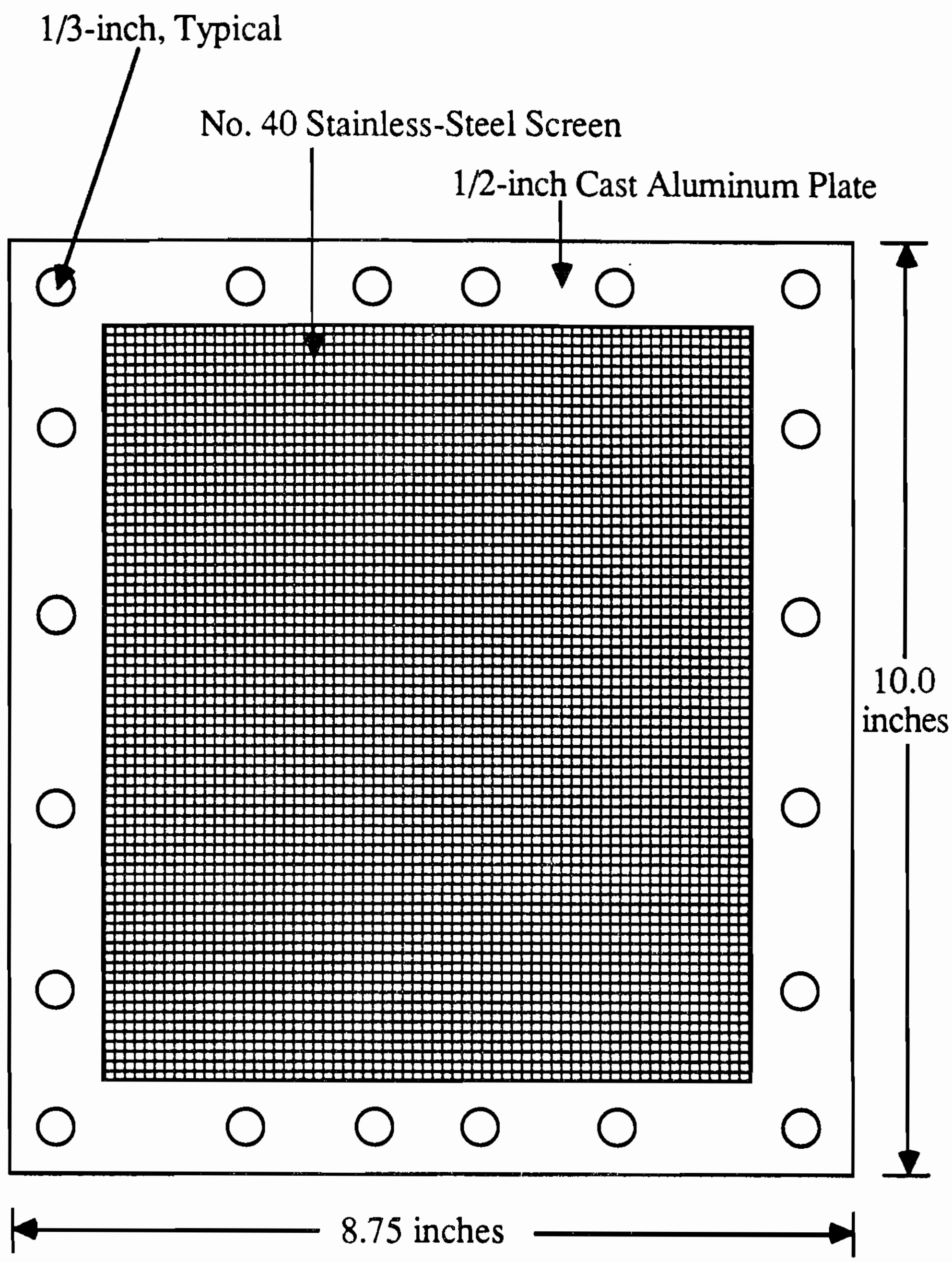

Figure 2.7. Top Plate of Frame Fabricated to Secure Screen for Processing Soil

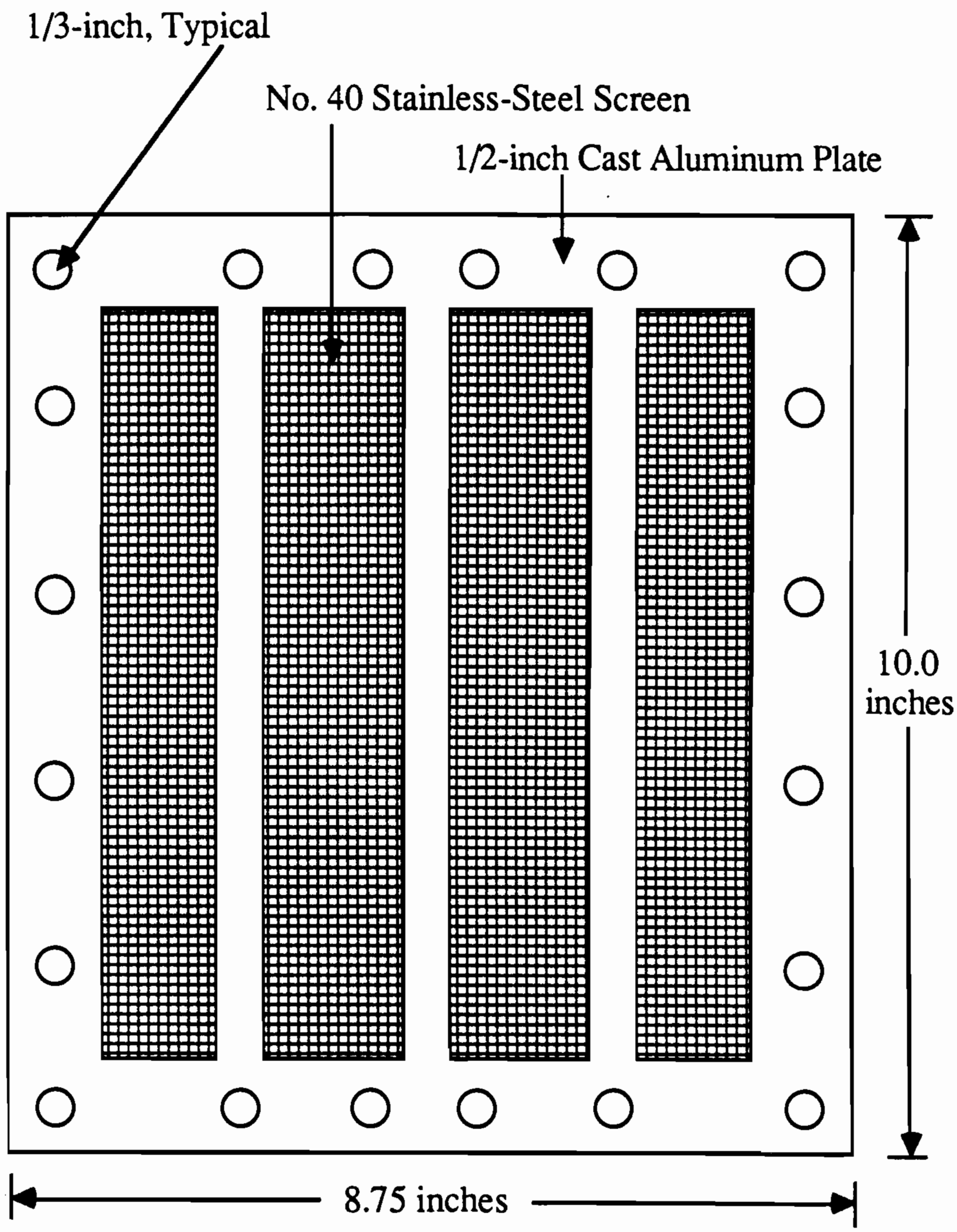

Figure 2.8. Base Plate of Frame Fabricated to Secure Screen for Processing Soil

TABLE 2.6. INITIAL PROPERTIES OF SCREEN-PREPARED SOIL SPECIMENS

Specimen Number	Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
WD21	1	24.8	98.2	93.4	0.72
WD22	1	25.4	96.9	92.7	0.74
WD23	1	23.8	94.5	82.0	0.78
WD24	1	23.9	96.6	86.7	0.74
WD25	3	24.6	94.4	84.6	0.79
WD26	3	24.2	95.8	86.0	0.76
WD27	10	24.0	94.4	82.2	0.79
WD28	10	22.9	95.8	81.5	0.76

(continued)

TABLE 2.6. CONTINUED

Direct Shear Test Number/ Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
DS4W/1	24.6	96.6	89.0	0.75
DS5W/1	23.3	95.4	82.0	0.77
DS6W/1	25.0	95.5	88.2	0.77
DS11W/1	23.9	96.0	85.9	0.75
DS12W/1	23.6	95.9	84.5	0.75
DS13W/1	23.6	95.9	84.5	0.75
DS14W/1	24.5	94.8	85.4	0.77
DS1W/3	24.6	95.0	85.6	0.77
DS2W/3	23.4	96.9	85.5	0.74
DS3W/3	24.0	96.1	85.8	0.75
DS10W/3	25.0	96.7	91.2	0.74

(continued)

TABLE 2.6. CONTINUED

Direct Shear Test Number/ Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
DS15W/3	24.7	94.4	84.7	0.78
DS16W/3	24.2	95.6	81.6	0.76
DS7W/9	25.2	96.8	91.8	0.74
DS8W/9	24.4	97.2	89.6	0.74
DS9W/9	23.6	97.0	86.1	0.74
DS17W/9	24.5	96.6	89.2	0.74
DS18W/9	23.4	97.0	86.0	0.73
DS19W/9	22.4	96.5	81.4	0.74
DS20W/30	23.9	95.5	84.9	0.76
DS21W/30	22.8	94.5	78.6	0.78
DS22W/30	23.8	94.4	82.2	0.78

TABLE 2.7. FINAL PROPERTIES OF SCREEN-PREPARED SOIL SPECIMENS AFTER WETTING AND DRYING

Specimen Number	Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
WD21	1	31.2	91.2	99.2	0.85
WD22	1	31.5	89.3	97.3	0.89
WD23	1	34.1	81.5	86.3	1.07
WD24	1	32.9	86.7	94.1	0.95
WD25	3	34.2	83.7	91.0	1.01
WD26	3	34.5	86.4	97.8	0.95
WD27	10	39.0	79.2	93.2	1.13
WD28	10	38.6	79.7	93.5	1.12

(continued)

TABLE 2.7. CONTINUED

Direct Shear Test Number/ Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
DS4W/1	36.0	85.6	100.0	0.97
DS5W/1	36.2	83.2	95.2	1.03
DS6W/1	35.2	83.2	92.7	1.03
DS11W/1	37.4	83.7	100.0	1.01
DS12W/1	38.4	82.6	100.0	1.03
DS13W/1	37.0	83.9	99.4	1.00
DS14W/1	38.3	81.8	97.8	1.05
DS1W/3	36.8	83.8	98.1	1.01
DS2W/3	34.2	79.7	94.0	0.98
DS3W/3	35.9	82.6	93.0	1.04
DS10W/3	37.9	79.9	92.4	1.10

(continued)

TABLE 2.7. CONTINUED

Direct Shear Test Number/ Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
DS15W/3	38.8	82.6	100.0	1.03
DS16W/3	31.1	91.2	99.4	0.84
DS7W/9	35.0	85.6	97.5	0.97
DS8W/9	34.8	83.7	92.8	1.01
DS9W/9	36.2	81.6	91.7	1.07
DS17W/9	37.6	79.0	89.6	1.13
DS18W/9	35.1	82.6	91.4	1.03
DS19W/9	36.8	81.6	93.6	1.06
DS20W/30	38.8	82.0	99.7	1.05
DS21W/30	38.2	75.1	83.2	1.22
DS22W/30	37.1	77.3	85.1	1.17

TABLE 2.8. CHANGE IN PROPERTIES OF SCREEN-PREPARED SOIL SPECIMENS

Specimen Number	Number of Cycles	Change in Moisture Content, percent	Change in Dry Density, pcf	Change in Degree of Saturation, percent	Change in Void Ratio
WD21	1	6.4	-7.0	5.8	0.13
WD22	1	6.1	-7.6	4.6	0.15
WD23	1	10.3	-13.0	4.3	0.29
WD24	1	9.0	-9.9	7.4	0.21
WD25	3	9.6	-10.7	6.4	0.22
WD26	3	10.3	-9.4	11.8	0.19
WD27	10	15.0	-15.2	11.0	0.34
WD28	10	15.7	-16.1	12.0	0.36

(continued)

TABLE 2.8. CONTINUED

Direct Shear Test Number/ Number of Cycles	Change in Moisture Content, percent	Change in Dry Density, pcf	Change in Degree of Saturation, percent	Change in Void Ratio
DS4W/1	11.4	-11.0	11.0	0.22
DS5W/1	12.9	-12.2	13.2	0.26
DS6W/1	10.2	-12.3	4.5	0.26
DS11W/1	13.5	-12.3	14.1	0.26
DS12W/1	14.8	-13.3	15.5	0.28
DS13W/1	13.4	-12.0	14.9	0.25
DS14W/1	13.8	-13.0	12.4	0.28
DS1W/3	12.2	-11.2	12.5	0.24
DS2W/3	10.8	-17.2	8.5	0.24
DS3W/3	11.9	-13.5	7.2	0.29
DS10W/3	12.0	-16.8	1.2	0.36

(continued)

TABLE 2.8. CONTINUED

Direct Shear Test Number/ Number of Cycles	Change in Moisture Content, percent	Change in Dry Density, pcf	Change in Degree of Saturation, percent	Change in Void Ratio
DS15W/3	14.1	-11.8	15.3	0.25
DS16W/3	6.9	-4.4	17.8	0.08
DS7W/9	9.8	-11.2	5.7	0.23
DS8W/9	10.4	-13.5	3.2	0.27
DS9W/9	12.6	-15.4	5.6	0.33
DS17W/9	13.1	-17.6	0.4	0.39
DS18W/9	11.7	-14.4	5.4	0.30
DS19W/9	12.9	-14.9	8.7	0.32
DS20W/30	14.9	-13.5	14.8	0.29
DS21W/30	15.4	-19.4	4.6	0.45
DS22W/30	13.3	-17.1	2.9	0.39

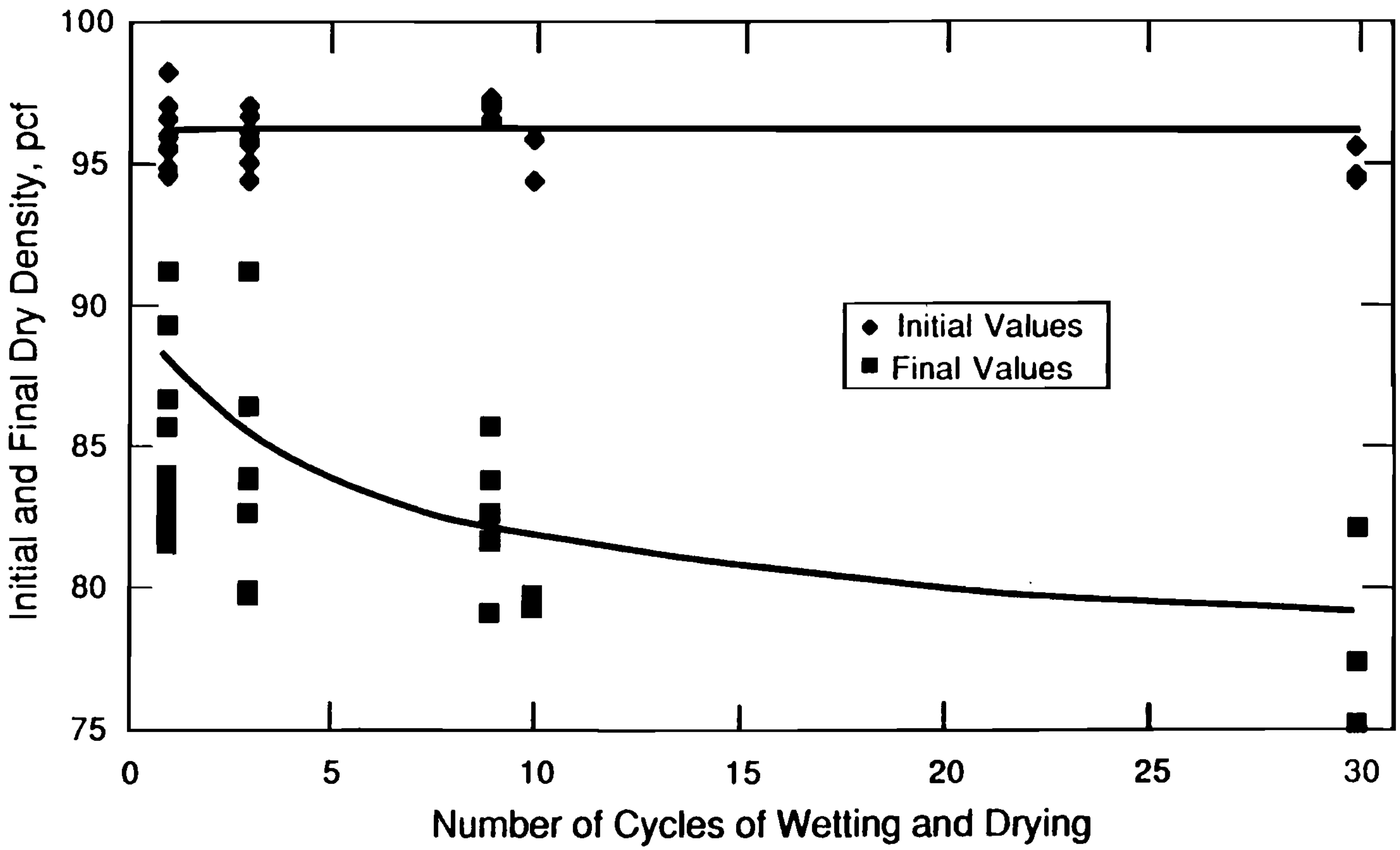

Figure 2.9. Initial and Final Dry Density vs. Number of Cycles of Wetting and Drying, Screen-Prepared Soil, Direct-Shear-Size Specimens

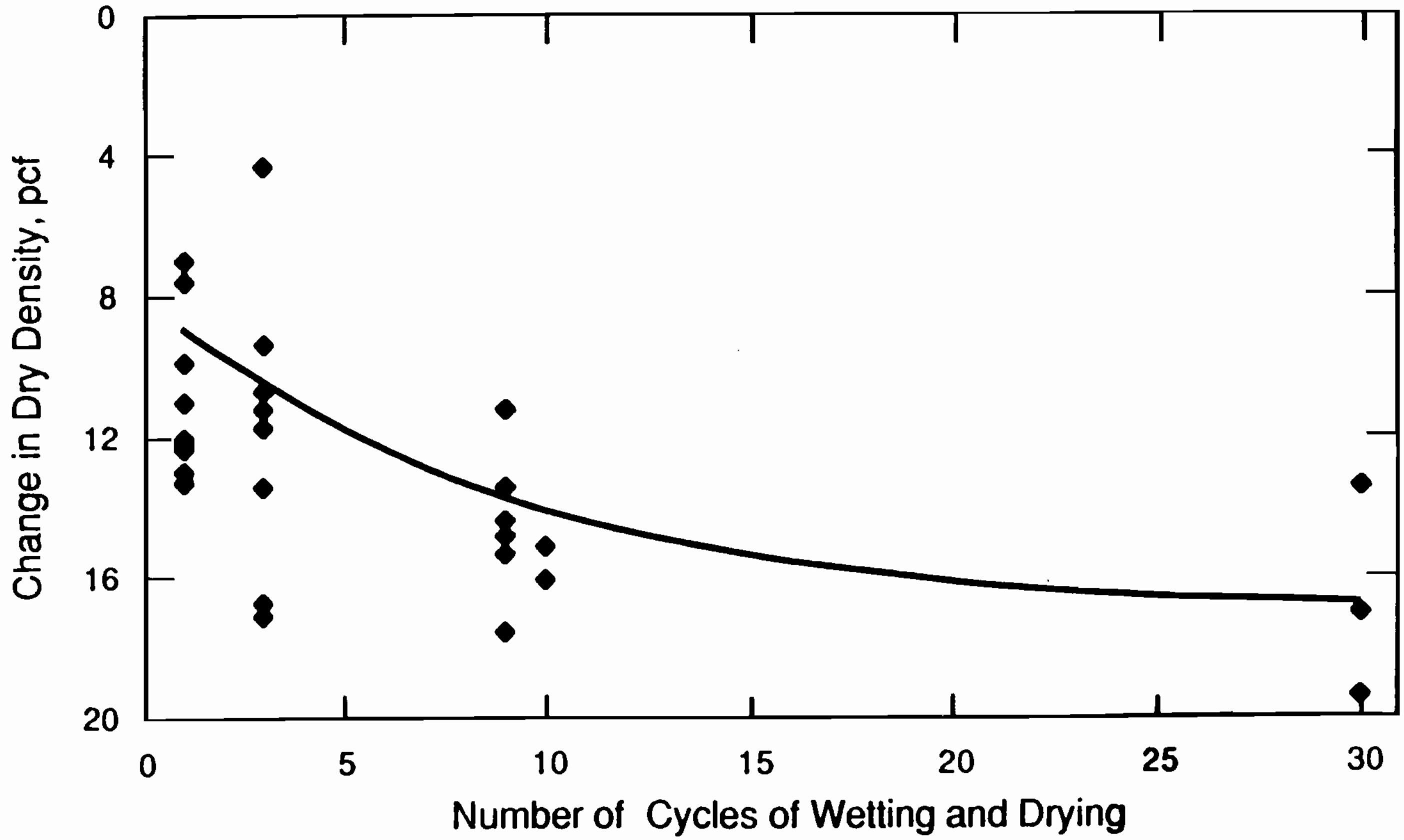

Figure 2.10. Change in Dry Density vs. Number of Cycles of Wetting and Drying, Screen-Prepared Soil, Direct-Shear-Size Specimens

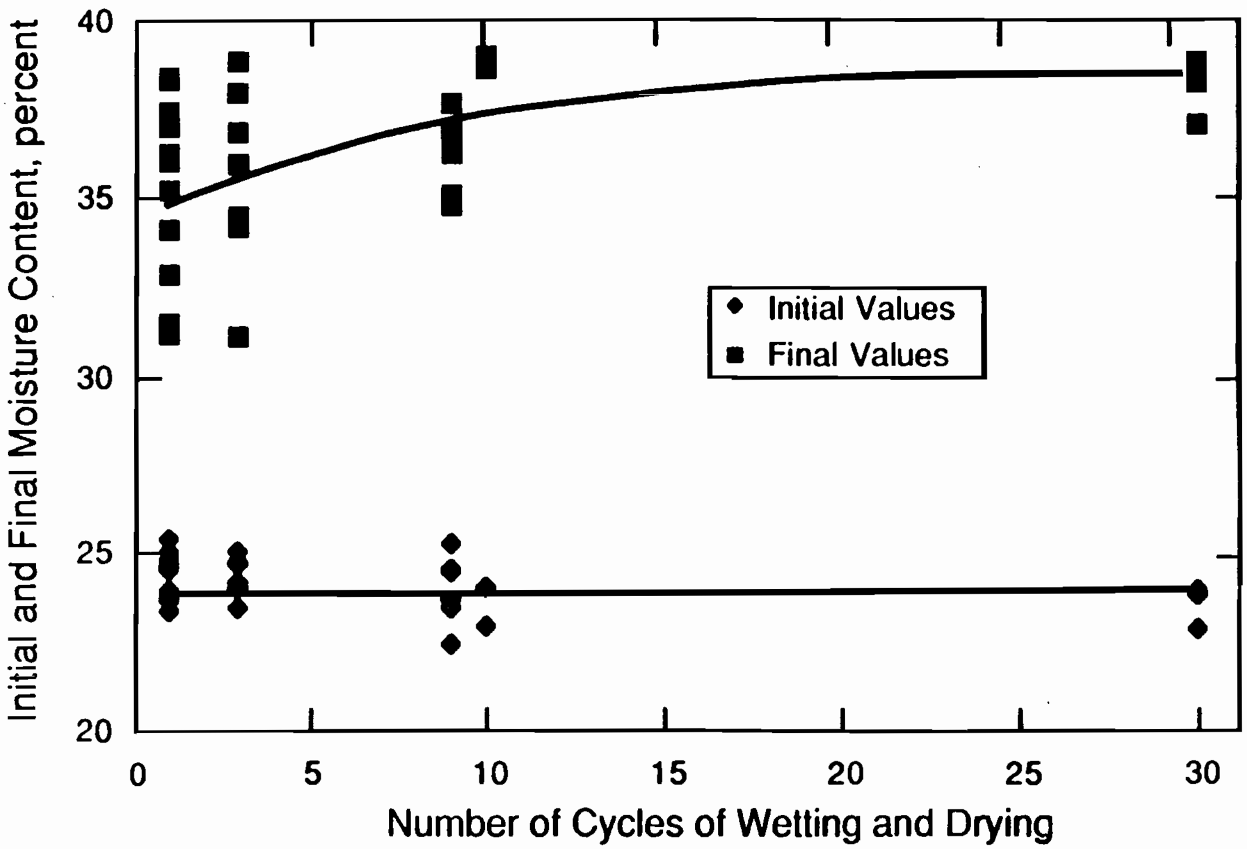

Figure 2.11. Initial and Final Moisture Content vs. Number of Cycles of Wetting and Drying, Screen-Prepared Soil, Direct-Shear-Size Specimens

the failure surface in the Scott Street slide to be approximately 35 percent, which agrees closely with these values.

Initial and final degrees of saturation are plotted versus the number of cycles of wetting and drying in Figure 2.12. The degree of saturation increased uniformly with an increase in the number of cycles to an average value of 94.5 percent.

Specimens compacted from the screen-prepared soil exhibited a more reasonable response to the effects of wetting and drying than those prepared by the extrusion process. Accordingly, the screen preparation method was chosen for subsequent use in this study.

TRIAXIAL-SIZE SPECIMENS

An additional series of tests was performed to determine the effects of wetting and drying on compacted specimens approximately 1.5 inches in diameter by 3.00 inches high. These dimensions are appropriate for specimens used in triaxial testing; specimens used in the preliminary studies of the effects of wetting and drying were prepared using procedures identical to those used for a later triaxial testing program.

Compaction Procedures

Specimens were compacted using soil prepared by the screen method described earlier. Specimens were compacted in a specially designed mold, shown in Figure 2.13. The mold has an inside diameter of 1.5 inches. The height of compacted specimens after removal of the collar on the mold and trimming is 3.3 inches.

A 2.15-pound hammer with an acrylic cylindrical face 1.45 inches in diameter was used to compact the specimens. Soil was compacted in thirteen equal lifts using two drops of the hammer at a height of 8 inches. The compactive effort generated with these procedures is 10.42 ft-lb/cu.in. (18,000 ft-lb/cu.ft.). Each lift contained approximately 17 grams of moist soil. This procedure was found to produce specimens with the "target" density of 96.3 pcf.

Each specimen was removed from the mold after compaction and the top surface was trimmed with a wire saw. The soil trimmings were used to measure the as-compacted moisture content. The specimens were 0.3 inches taller than the required height for the consolidated-

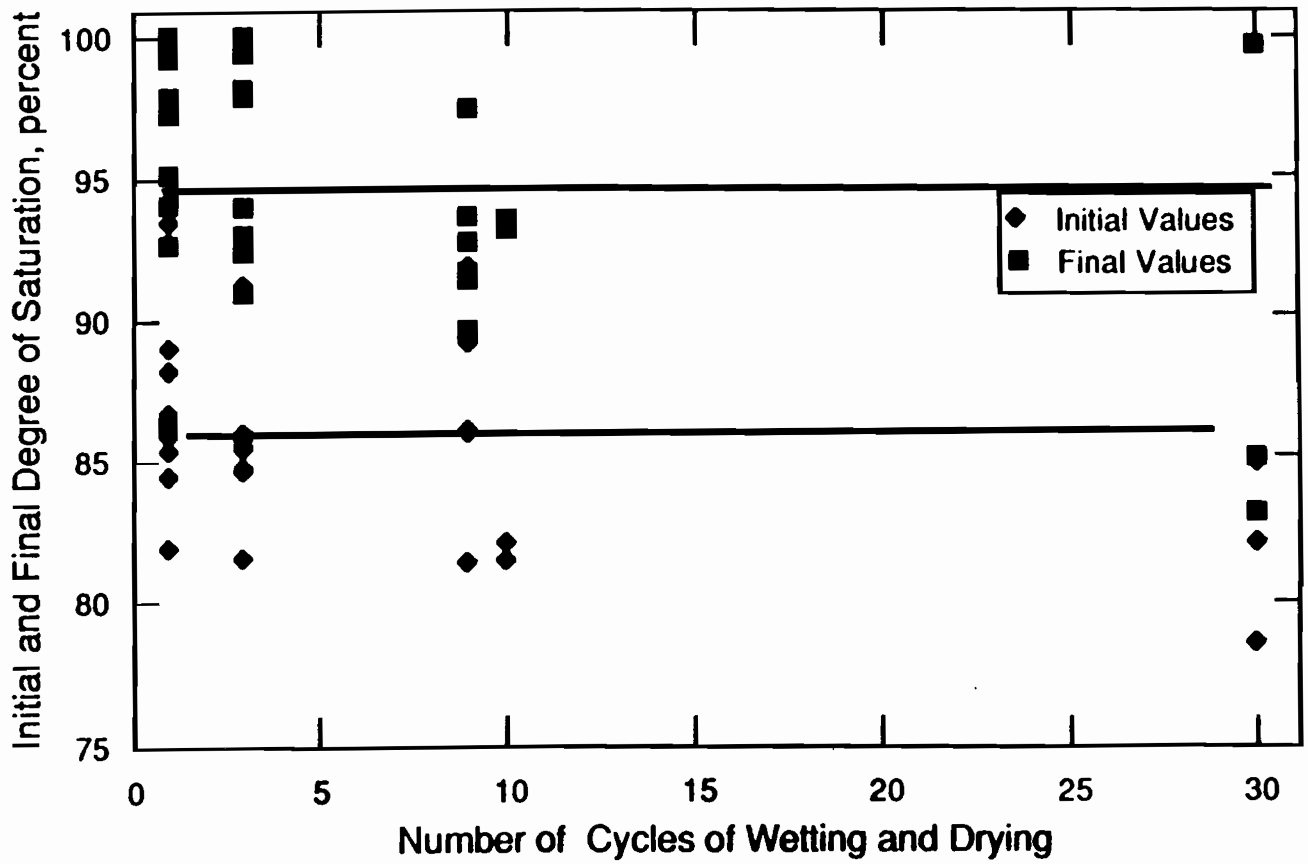

Figure 2.12. Initial and Final Degree of Saturation vs. Number of Cycles of Wetting and Drying Screen-Prepared Soil, Direct-Shear-Size Specimens

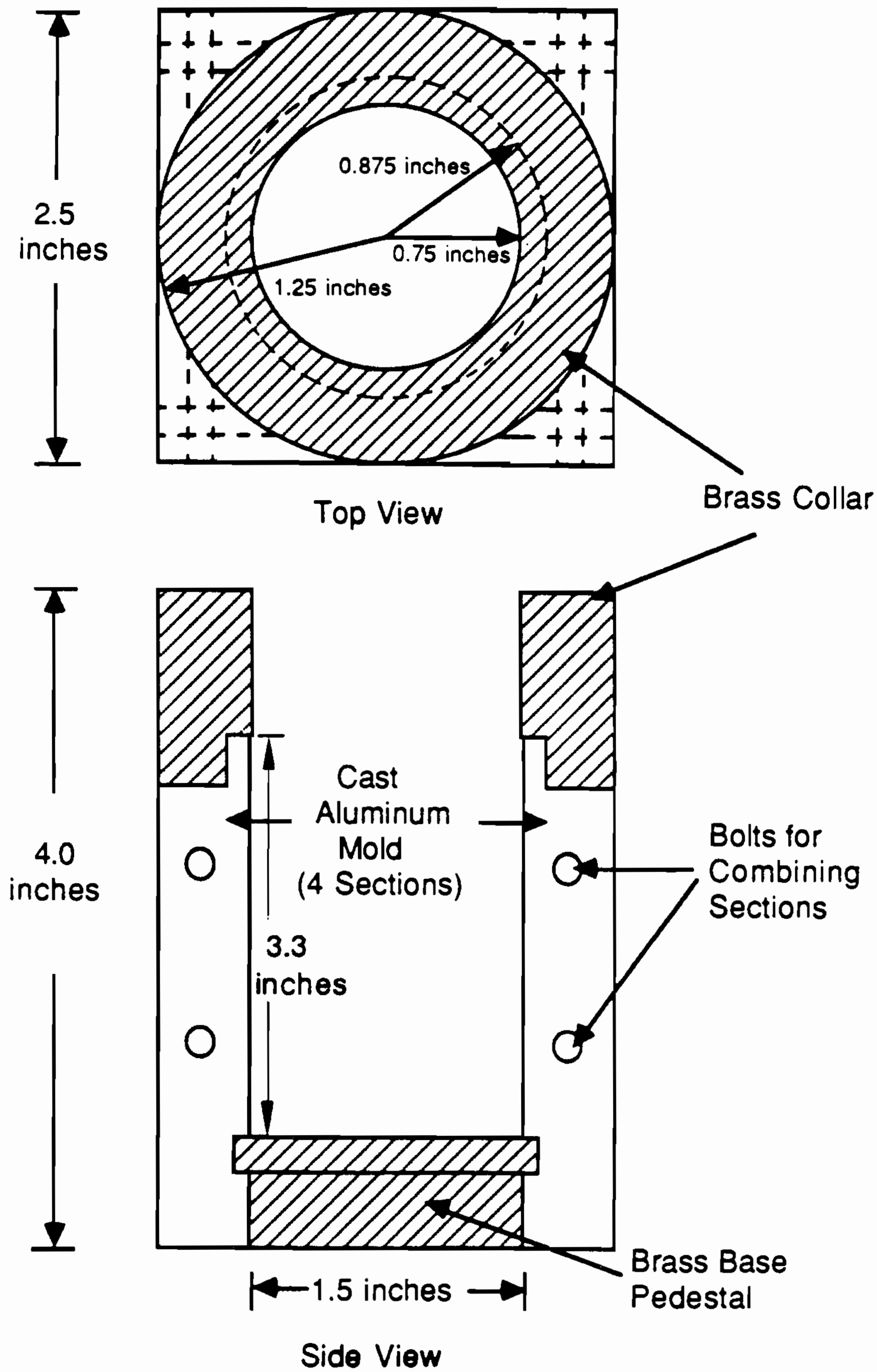

Figure 2.13. 1.5-inch Diameter Compaction Mold

undrained triaxial compression test. Soil within the upper portion of the specimen was removed after wetting and drying and prior to shear testing.

Specimens were placed in an acrylic chamber during the cycles of wetting and drying similar to the chamber used for the 2.5-inch diameter specimens illustrated earlier in Figure 2.1. A diagram of the 1.5-inch diameter chamber is presented in Figure 2.14. The inside diameter of the acrylic cylinder was 1.5 inches and the height was 4.0 inches; the specimens were thereby confined radially but were allowed to expand freely in the vertical direction.

Properties after Wetting and Drying

Specimens were subjected to six, ten and thirty cycles of wetting and drying. Each cycle consisted of a 24-hour wetting period and a 24-hour drying period identical to those used with the direct-shear-sized specimens. An initial trial specimen was subjected to one cycle of wetting and drying consisting of a wetting period, a drying period and a final wetting period. The initial properties of this specimen as well as those of twelve additional specimens which were later sheared with the triaxial device following wetting and drying are presented in Table 2.9. Properties of each specimen after wetting and drying and prior to shear were estimated based upon the volume and moisture content after wetting and drying and the dry weight measured after shear. These estimated properties, along with the measured properties of the trial specimen, are presented in Table 2.10. Corresponding changes in properties due to wetting and drying are listed in Table 2.11.

Initial and final dry densities of each specimen are plotted versus the number of cycles of wetting and drying in Figure 2.15. The corresponding change (decrease) in dry density is plotted versus the number of cycles in Figure 2.16. A relatively wide range of change in density and apparent scatter in the data were observed. No correlation was apparent between the number of cycles and the decrease in dry density of the triaxial specimens.

Initial and final moisture contents are plotted versus number of cycles of wetting and drying in Figure 2.17. The corresponding change in moisture content was found to be relatively constant regardless of the number of cycles; the average change in moisture content was 7.6 percent.

Initial and final degrees of saturation of the triaxial specimens were plotted versus the number of cycles of wetting and drying in Figure 2.18. The increase in saturation level was relatively constant regardless of the length of wetting and drying, although there is significant scatter in the data.

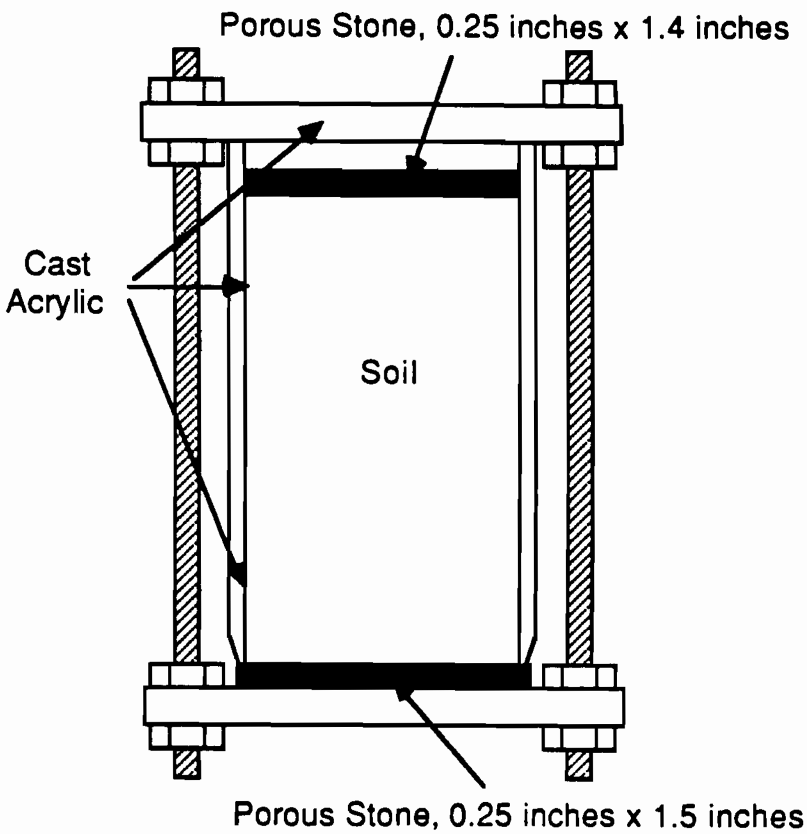

Figure 2.14. Chamber for Subjecting Triaxial-Size Specimens to Wetting and Drying

TABLE 2.9. INITIAL PROPERTIES OF 1.5-INCH DIAMETER SPECIMENS

Triaxial Test Number/ Number of Cycles	Moisture Content, percent	Drv Density, pcf	Degree of Saturation, percent	Void Ratio
Trial Specimen	24.0	98.0	94.5	0.69
6.31/6	25.3	94.4	86.3	0.79
6.30/6	24.1	95.8	86.0	0.75
6.35/6	24.8	95.5	87.9	0.76
B.21/6	23.5	95.6	83.5	0.76
6.28/10	24.3	96.3	87.3	0.75
6.29/10	24.4	96.5	88.0	0.75
B.19/10	23.3	96.8	84.6	0.74
6.34/10	24.1	94.8	84.1	0.77
6.33/30	25.2	96.6	91.2	0.75
6.32/30	23.9	95.5	84.7	0.76
B.20/30	24.3	97.3	89.4	0.73
6.36/30	23.6	96.1	84.9	0.75

TABLE 2.10. FINAL PROPERTIES OF 1.5-INCH DIAMETER SPECIMEN AFTER WETTING AND DRYING

Triaxial Test Number/ Number of Cycles	Moisture Content, percent	Dry Density, pcf	Degree of Saturation, percent	Void Ratio
Trial Specimen	28.3	95.0	100.0	0.76
6.31	31.2	87.3	90.8	0.92
6.30	30.9	92.6	100.0	0.81
6.35	33.1	83.9	100.0	0.76
B.21	30.0	93.9	100.0	0.79
6.28	31.6	90.9	99.6	0.86
6.29	33.0	84.8	90.5	0.98
B.19	30.9	87.7	97.1	0.86
6.34	32.5	90.7	100.0	0.85
6.33	33.0	86.6	94.5	0.94
6.32	31.3	93.6	100.0	0.80
B.20	33.0	85.1	91.3	0.97
6.36	31.0	94.2	100.0	0.77

TABLE 2.11. CHANGE IN PROPERTIES OF 1.5-INCH DIAMETER SPECIMENS DURING WETTING AND DRYING

Triaxial Test Number/ Number of Cycles	Change in Moisture Content, percent	Change in Dry Density, pcf	Change in Degree of Saturation, percent	Change in Void Ratio
Trial Specimen	4.3	-3.0	5.5	0.07
6.31	5.9	-7.1	4.5	0.13
6.30	6.8	-3.2	14.0	0.06
6.35	8.3	-11.6	12.1	0.00
B.21	6.5	-1.7	16.5	0.03
6.28	7.3	-5.4	12.3	0.11
6.29	8.6	-11.7	2.5	0.23
B.19	7.6	-9.1	12.5	0.12
6.34	8.4	-4.1	15.9	0.08
6.33	7.8	-10.0	3.3	0.19
6.32	7.4	-1.9	15.3	0.04
B.20	8.7	-12.2	1.9	0.24
6.36	7.4	-1.9	15.1	0.02

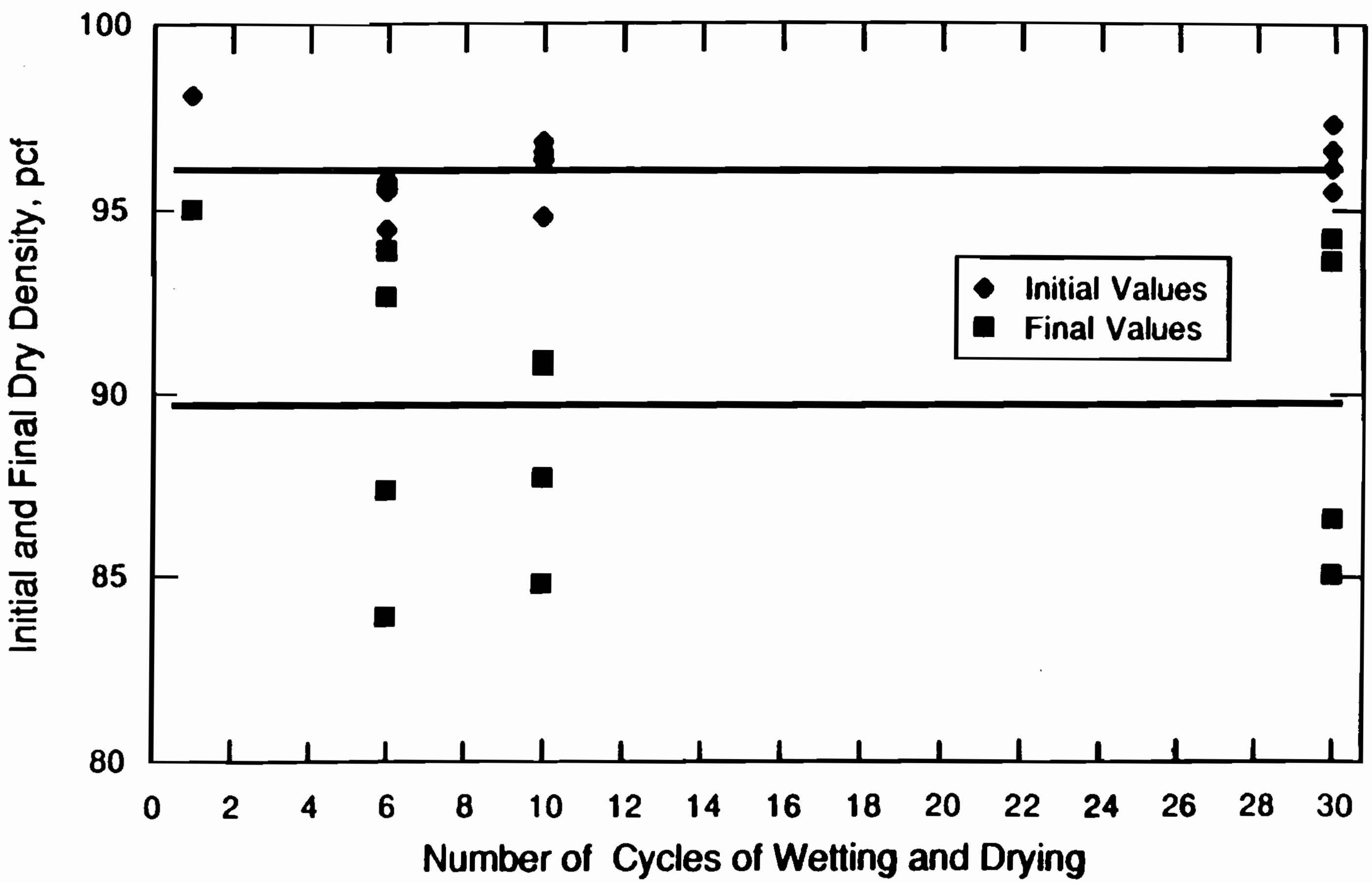

Figure 2.15. Initial and Final Dry Density vs. Number of Cycles of Wetting and Drying, Screen-Prepared Soil, Triaxial-Size Specimens

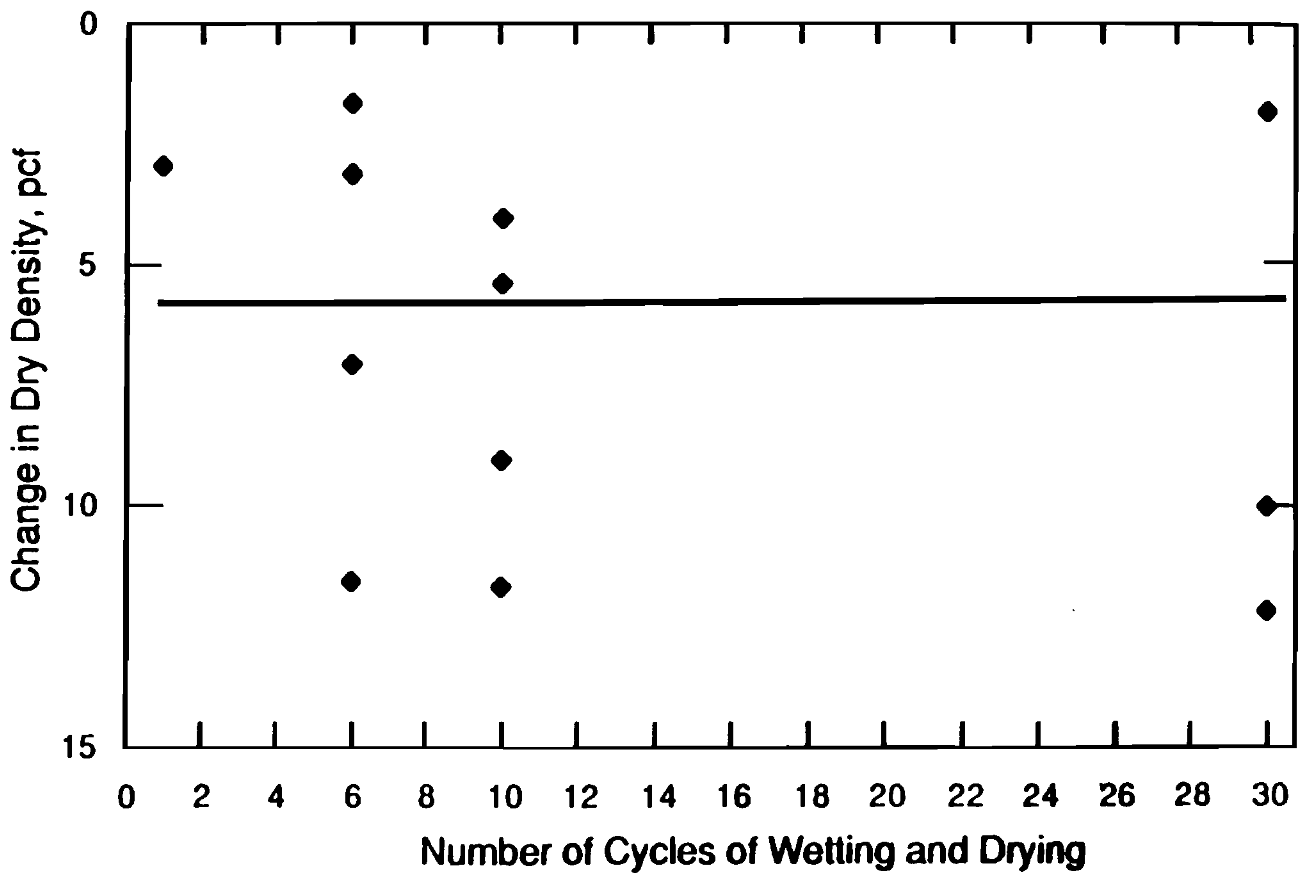

Figure 2.16. Change in Dry Density vs. Number of Cycles of Wetting and Drying, Screen-Prepared Soil, Triaxial-Size Specimens

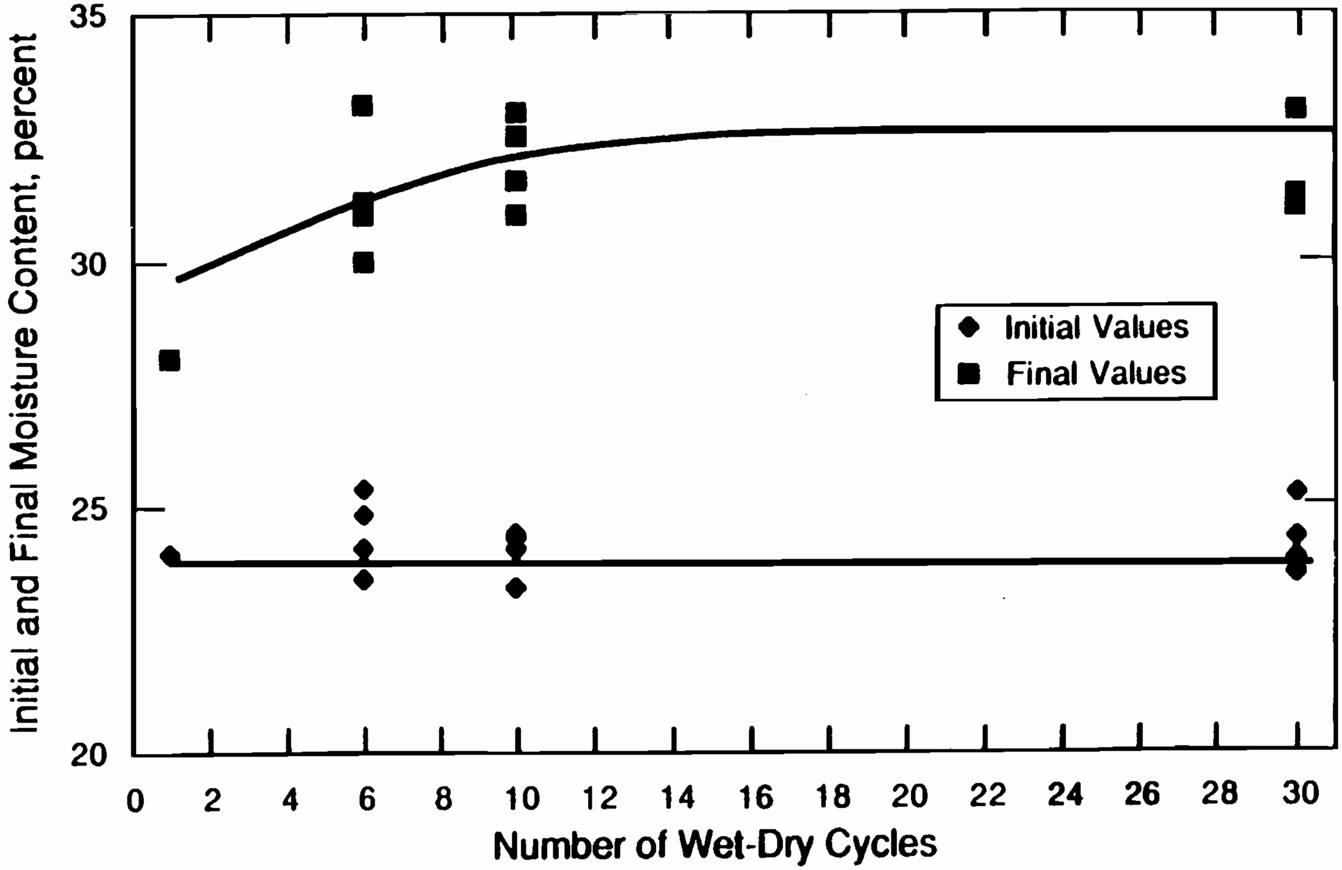

Figure 2.17. Initial and Final Moisture Content vs. Number of Cycles of Wetting and Drying, Screen-Prepared Soil, Triaxial-Size Specimens

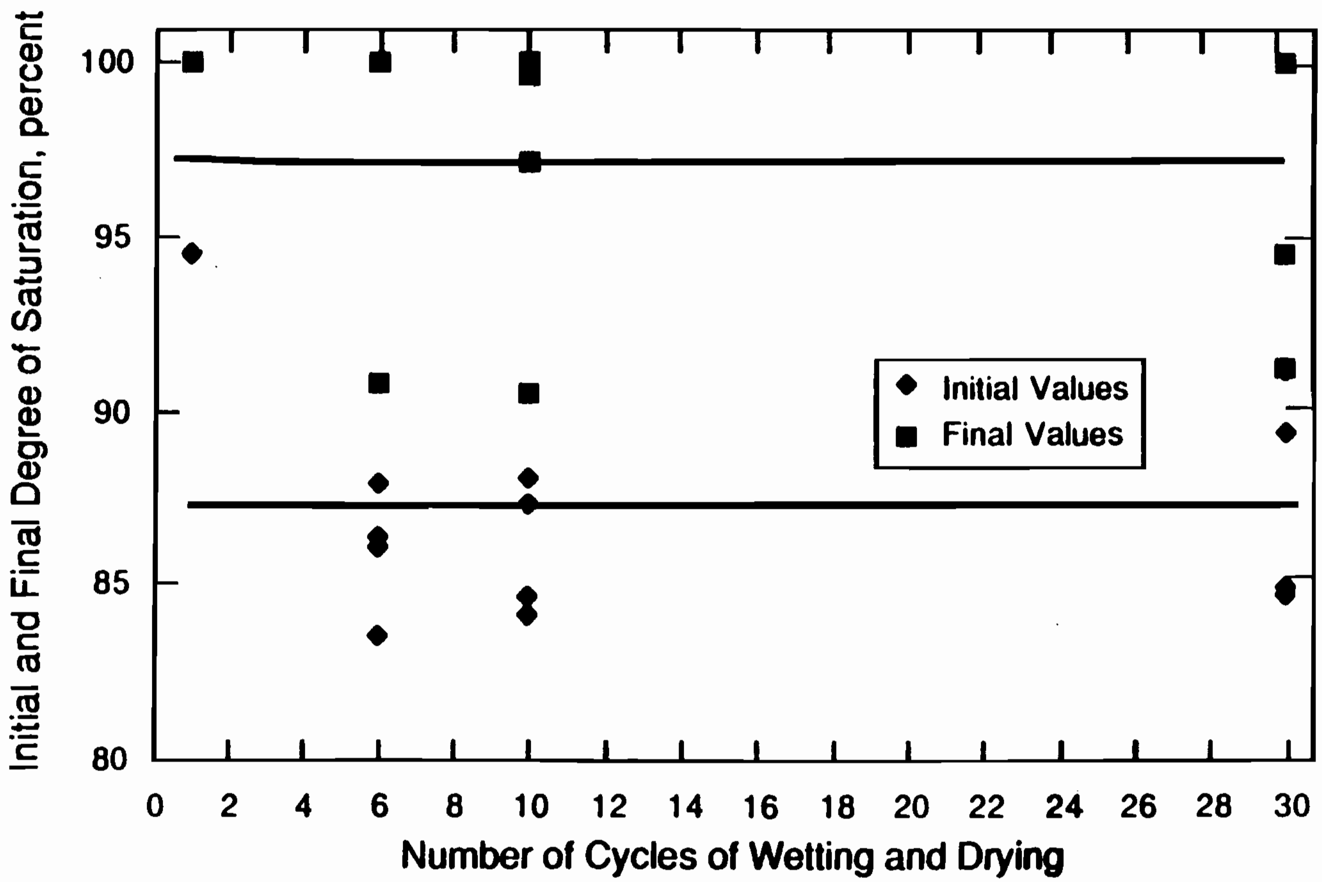

Figure 2.18. Initial and Final Degree of Saturation vs. Number of Cycles of Wetting and Drying, Screen-Prepared Soil, Triaxial-Size Specimens

DISCUSSION AND SUMMARY

Sowers (1979) has suggested that little swell will occur once the soil moisturereaches the plastic limit, or slightly more, equivalent to a water-plasticity ratio, R_w, of about 0.25; the water-plasticity ratio is defined as:

$$R_w = (w - w_{pl}) / P.I. \tag{2.1}$$

where w is the moisture content, w_{pl} is the plastic limit and P.I. is the plasticity index. The plastic limit of the red clay used in this study is about 20 percent, and the plasticity index is 50 percent, (Green and Wright, 1986); the moisture content corresponding to a water-plasticity ratio of 0.25 is 32.5 percent. Direct-shear-size specimens were at moisture contents ranging from 31.1 percent to 39.0 percent after wetting and drying, with an average value of 36.1 percent for all numbers of cycles of wetting and drying combined. The final moisture contents of the triaxial-size specimens were measured to be between 30.0 percent and 33.0 percent, with an average value of 31.8 percent. Thus, the final moisture contents were comparable to those suggested by Sowers. However, there were some significant differences between the specimens of direct shear- and triaxial-size.

Final dry densities are plotted versus number of cycles of wetting and drying in Figure 2.19 for both the direct shear- and triaxial-size specimens. It can be seen that the specimens of triaxial-size consistently possessed a greater dry density after the same number of cycles of wetting and drying than the specimens of direct-shear-size. Initial dry densities of both the direct-shear- and triaxial-size specimens were approximately 96.3 pcf, the "target" value for compaction. Average final dry densities after 10 cycles of wetting and drying were 88.5 pcf and 79.5 pcf for the triaxial-size specimens and direct-shear-size specimens, respectivley; values of 89.9 pcf and 78.1 pcf were measured after 30 cycles.

Final moisture contents are plotted versus number of cycles of wetting and drying in Figure 2.20. Direct-shear-size specimens possessed a greater moisture content than the triaxial-size specimens after a given number of cycles of wetting and drying. The initial moisture content of both the direct-shear- and triaxial-size specimens was approximately 24 percent. After 10 cycles of wetting and drying, the average moisture content of the direct-shear-size specimens was 38.8 percent, while the corresponding value for the triaxial-size specimens was 32 percent. An average moisture content of 38 percent was measured for direct-shear-size specimens subjected to 30 cycles of wetting and drying; triaxial-size specimens possessed an average value of 32.1 percent for the same number of cycles.

The lower final dry densities and higher moisture contents of the direct-shear-size specimens are believed to be due to the lower level of confinement, larger exposed surface area and shorter distances which moisture had to flow in the specimens of direct-shear-size as compared to those of triaxial-size. Specimens of direct-shear-size were also allowed to expand radially during wetting and drying, while the triaxial-size specimens were not given any lateral freedom.

The sufficiency of the 24-hour period of wetting to allow the soil to fully take up water may be assessed by examining the time required to complete primary consolidation. Specimens tested in the direct shear device were consolidated one-dimensionally prior to shear; complete consolidation data are presented in Chapter 3. Values of the time required to complete primary consolidation (t_{100}) ranged from 10 minutes to 5.5 hours. Thus, ample time appears to have been available for the soil in the specimens of direct-shear-size to take up water during the 24 hours of wetting.

Although no one-dimensional consolidation data were available for the triaxial-size specimens, t_{100} can be estimated from Terzaghi's theory of one-dimensional consolidation. According to his theory, the time required for primary consolidation is proportional to the drainage distance squared. Thus, if from 10 minutes to 5.5 hours is required for primary consolidation in a 0.816-inch high (direct-shear-size) specimen, times from 2.7 hours to 90 hours should be required for primary consolidation of a 3.3 -inch high (triaxial-size) specimen. The upper limit (90 hours) exceeds the 24-hour wetting period and may explain why the specimens of triaxial-size exhibited higher final dry densities and lower moisture contents than those of direct-shear-size. The range in times of 2.7 hours to 90 hours overlaps the 24-hour time span used for wetting and drying and may explain some of the scatter in final dry densities and moisture contents of the specimens of triaxial-size.

CHAPTER 3. DIRECT SHEAR TESTS

INTRODUCTION

Twenty-two direct shear tests were performed to measure the effective-stress shear-strength parameters of specimens after repeated wetting and drying. Shear tests were performed on specimens which had been subjected to one, three, nine and thirty cycles of wetting and drying. All specimens were compacted of screen-prepared soil as described in Chapter Two. Shear test procedures and results are presented in this chapter.

TEST PROCEDURES

Two types of direct shear devices were used for testing. A device manufactured by Engineering Laboratory Equipment (ELE Model EL28-009) was used to perform shear tests at vertical effective stresses less than or equal to 3.1 psi; tests run at higher stresses were performed on devices manufactured by CETec, Incorporated (Model DS.306/10). The procedures for running direct shear tests with each type of device are summarized in the following paragraphs.

Set-up and Consolidation - ELE Device

Specimens sheared with the ELE device were trimmed after wetting and drying into a stainless steel ring with a diameter of 2.5 inches and a height of 0.816 inches. They were then extruded from the ring into the direct shear box.

Specimens were consolidated vertically using a single load increment with a loaded hanger. No free water was accessible to the specimen at the onset of consolidation. Distilled water was introduced into the box immediately after the load was applied. A constant level of water was maintained throughout each test. Consolidation data for these specimens tended to be erratic due to seating problems and movement of soil into grooves in the upper and lower confining plates. As a result, the time required to complete primary consolidation could not be determined. However, other specimens sheared at higher vertical stresses on the other direct shear device (CETec) showed that the maximum time for the completion of primary consolidation was approximately 5.5 hours.

Accordingly, a 24-hour time period was allowed for consolidation in the ELE device, which was considered more than adequate to produce full primary consolidation.

Except for the consolidation procedures described above, the other testing procedures in the ELE frame were identical to those used with the CETec frames and are described in further detail in the latter portion of this section.

Set-up and Consolidation - CETec Device

Specimens sheared with the CETec devices were trimmed into a stainless steel ring with a diameter of 2.35 inches and a height of 0.816 inches. Once each specimen was extruded into the direct shear box, it was consolidated with two increments of vertical load. Incremental consolidation was chosen to avoid extruding soil from the box along the plane between the top and base. Specimens were first consolidated under the weight of the hanger to a vertical stress of approximately one-half of the total stress to be applied during shear. Distilled water was introduced into the box immediately after the first load increment was applied. A constant water level was maintained throughout both stages of consolidation and the shear test itself.

Consolidation data for the first load increment were generally poor and, thus, the end of primary consolidation was not readily apparent. Primary consolidation from the first load increment was assumed to be complete after 24 hours, and the second load increment was applied to increase the total vertical stress on the specimens to the level desired for shearing conditions. Good consolidation data were obtained for the second load increment. Specimens were allowed to consolidate under the second load increment for several hours prior to shearing until the time-settlement data indicated that primary consolidation was complete.

Shear

At the end of consolidation, the upper and lower halves of the direct shear boxes on both the ELE and CETec machines were separated 0.05 inches to prevent the development of friction and resulting erroneous data which would be induced by their contact during shear. The rate of shear was set at the slowest rate possible, 0.002 inches/hour, to assure adequate drainage. Calibrated proving rings were mounted on each device to measure the shearing load on each specimen. Dial gauges were also installed on the CETec frames to measure horizontal displacement and monitor the rate of shear. Horizontal deformations on the ELE frame were calculated using a calibrated rate of shear.

Each specimen was sheared to a maximum deformation of at least 0.15 inches. A large number of the specimens were sheared for as long as 5 days to insure deformation beyond the peak shear stress. Shear stress values were calculated based upon the corrected cross-sectional area of the shear plane, where the corrected area represents the reduced area as the two halves of the specimen were displaced. The corrected area was calculated with the following equation:

$$A = 1/90^{o}A_{o}\cos^{-1}(0.5\Delta_{h}/R) - \Delta_{h}R\sin(\cos^{-1}(0.5\Delta_{h}/R)) \qquad (3.1)$$

where Δ_h is the horizontal displacement, R is the radius of the specimen and all angles are in degrees; the derivation of this equation was presented by Green and Wright (1986).

At the completion of each test, each specimen was returned to the point of zero deformation, the water surrounding the specimen was drained and the vertical load was removed. The direct shear device was then disassembled and a final water content was determined.

TEST RESULTS

Test data from the direct shear tests on specimens subjected to wetting and drying were analyzed to estimate the effective-stress shear-strength parameters of the compacted specimens subjected to wetting and drying. Consolidation and shear-deformation versus shear-stress data were examined in detail to assess the shear-strength properties of the "Beaumont" clay under conditions of cyclic wetting and drying.

Consolidation

Two methods were examined for estimating the time required to complete primary consolidation, t_{100}, from the consolidation data. One is based on plotting the decrease of the height of the specimen during consolidation versus the logarithm of time, and the other is based on plotting the change in height of the specimen during consolidation versus the square root of time. The decrease in height during consolidation for the specimen for Test DS12W is plotted versus the logarithm of time in Figure 3.1 and versus the square root of time in Figure 3.2. The t_{100} values

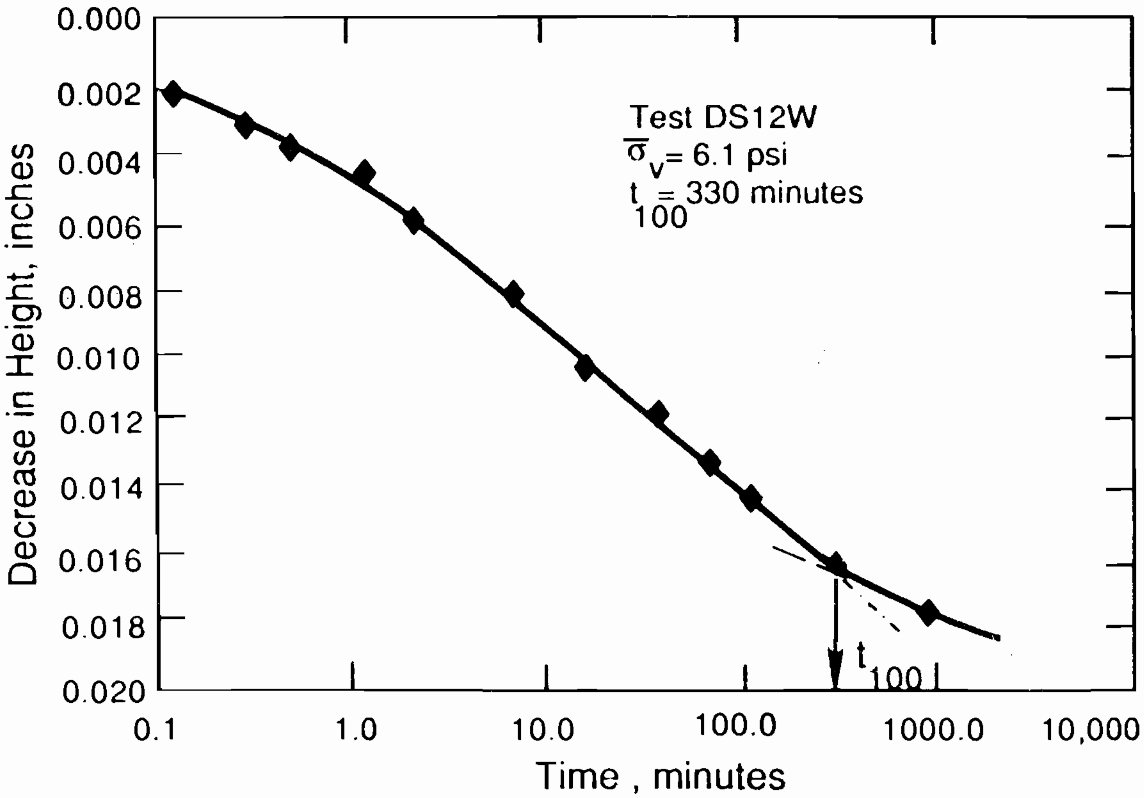

Figure 3.1 Decrease in Height of Specimen in Test DS12W During Consolidation vs. Logarithm of Time, One Cycle of Wetting and Drying

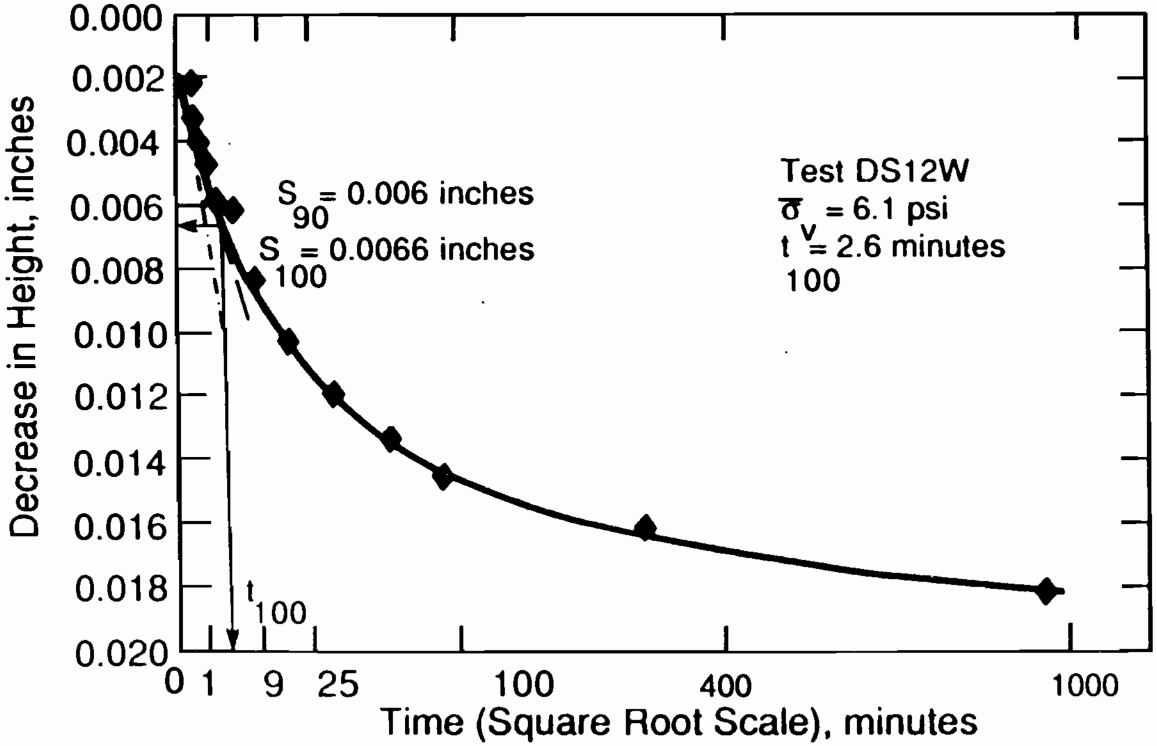

Figure 3.2 Decrease in Height of Specimen in Test DS12W During Consolidation vs. Square Root of Time, One Cycle of Wetting and Drying

were estimated by graphical procedures developed by Casagrande and Fadum (1944) in Figure 3.1, and Taylor (1948) in Figure 3.2. The t_{100} values estimated with the data in these two figures are 330 minutes and 2.6 minutes, respectively. The value of 2.6 minutes found using the square root of time procedure is considered to be unrealistically small. Accordingly, the logarithm of time procedure was selected and used to calculate t_{100} values for consolidation data from the remainder of the direct shear tests. Plots of settlement versus the logarithm of time for those specimens which exhibited good consolidation data are included in Appendix B.

During consolidation, drainage was provided at both ends of each specimen by porous stones. Coefficients of consolidation, c_v, were calculated assuming double drainage from the equation,

$$c_v = (T_{50})(H)^2 / t_{50} \tag{3.2}$$

where T_{50} is the "time-factor" for 50 percent consolidation, (T_{50} = 0.197) H is one-half the average height of the specimen during consolidation and t_{50} is the time at 50 percent consolidation.

Time to Failure

The coefficients of consolidation (c_v) determined from the consolidation data were used to calculate times to failure required to assure adequate drainage. The theoretical equation developed by Gibson and Henkel (1954) was used for this purpose:

$$t_f = H^2 / [2 \times c_v \times (1 - U)] \tag{3.3}$$

where t_f is the theoretical minimum time to failure, H is one-half the specimen height, and U is the degree of excess pore water dissipation corresponding approximately to the degree of drainage (a value of U equal to 0.95 was used).

Values for c_v, t_{100}, and the theoretical minimum time to failure for each test where good consolidation data were available are summarized in Table 3.1.

TABLE 3.1. CONSOLIDATION PARAMETERS FOR DIRECT SHEAR SPECIMENS

Direct Shear Test Number	Vertical Effective Stress, psi	t_{100} min	c_v in^2/min	Theoretical Minimum Time to Failure, min
DS5W	6.0	23	.02435	60
DS8W	6.0	19	.01530	96
DS2W	6.1	82	.01305	117
DS12W	6.1	330	.00217	660
DS15W	6.1	320	.00236	619
DS21W	6.1	64	.01266	106
DS3W	9.0	6	.02340	59
DS9W	9.0	10	.01739	87
DS19W	9.0	150	.00283	458
DS6W	9.1	22	.02021	70

Shear

Final moisture contents, shear deformation - shear stress curves and Mohr-Coulomb diagrams were plotted and examined for the direct shear tests. These data are presented below.

Final Moisture Content. The effective normal stress and final moisture content for the twenty-two direct shear tests are summarized in Table 3.2. The final moisture contents of the specimens sheared are plotted versus the logarithm of vertical effective stress in Figure 3.3. The final moisture content values decreased with increasing effective vertical pressure, as expected.

Stress-Deformation Behavior. A typical plot of shear-stress versus shear-deformation is presented in Figure 3.4 for the direct shear tests performed on specimens subjected to one cycle of wetting and drying. The remaining shear stress versus shear deformation plots are included in Appendix A. Maximum shear stress values and corresponding times to failure at the point where the maximum shear stress first occurred were determined from these plots. A number of the direct shear tests exhibited an increase in shear stress with an increase in deformation to a peak value, followed by a decrease in shear stress with further deformation; the remaining tests exhibited shear stress values which either continued to increase with increasing deformation, or reached a maximum level and never experienced a decrease. The peak or maximum shear stress and corresponding deformation and time to failure are presented in Table 3.3 for each of the direct shear tests. The times to failure ranged from 38.8 hours to 104 hours, and in all cases well-exceeded the theoretical minimum times to failure, ranging from 59 minutes to 11 hours, which were summarized earlier in Table 3.1.

Effective-Stress Shear-Strength Parameters. Separate Mohr-Coulomb diagrams were plotted for each series of direct shear tests performed on specimens subjected to a given number of cycles of wetting and drying. These are shown in Figures 3.5, 3.6, 3.7 and 3.8 for specimens subjected to one, three, nine and thirty cycles of wetting and drying, respectively. Data for Test DS16W (3 cycles - 9.0 psi) were omitted from Figure 3.6 because mechanical problems with the CETec direct shear device were thought to make the data erroneous.

Effective cohesion values and angles of internal friction were determined from linear regression analyses of the data presented in the Mohr-Coulomb diagrams in Figures 3.5, 3.6, 3.7 and 3.8. A standard linear regression analysis on the data for specimens subjected to thirty wet-dry cycles produced a negative cohesion value; consequently, a friction angle was calculated from the test data assuming a cohesion of 0 psf.

TABLE 3.2. FINAL PROPERTIES OF DIRECT SHEAR SPECIMENS AFTER TESTING

Direct Shear Test Number	Number of Wet-Dry Cycles	Effective Normal Stress, psi	Final Moisture Content, percent
DS14W	1	1.3	36.0
DS4W	1	3.1	32.4
DS11W	1	3.1	35.9
DS5W	1	6.0	33.9
DS12W	1	6.1	33.6
DS13W	1	9.0	33.0
DS6W	1	9.1	30.1
DS1W	3	3.1	33.0
DS10W	3	3.1	33.6
DS2W	3	6.1	29.8
DS15W	3	6.1	34.9

(continued)

TABLE 3.2. CONTINUED

Direct Shear Test Number	Number of Wet-Dry Cycles	Effective Normal Stress, psi	Final Moisture Content, percent
DS3W	3	9.0	35.9
DS16W	3	9.0	33.4
DS17W	9	2.2	37.5
DS7W	9	3.1	31.9
DS8W	9	6.0	32.6
DS18W	9	6.1	32.1
DS9W	9	9.0	28.8
DS19W	9	9.0	30.3
DS20W	30	3.1	32.9
DS21W	30	6.1	31.9
DS22W	30	9.0	32.4

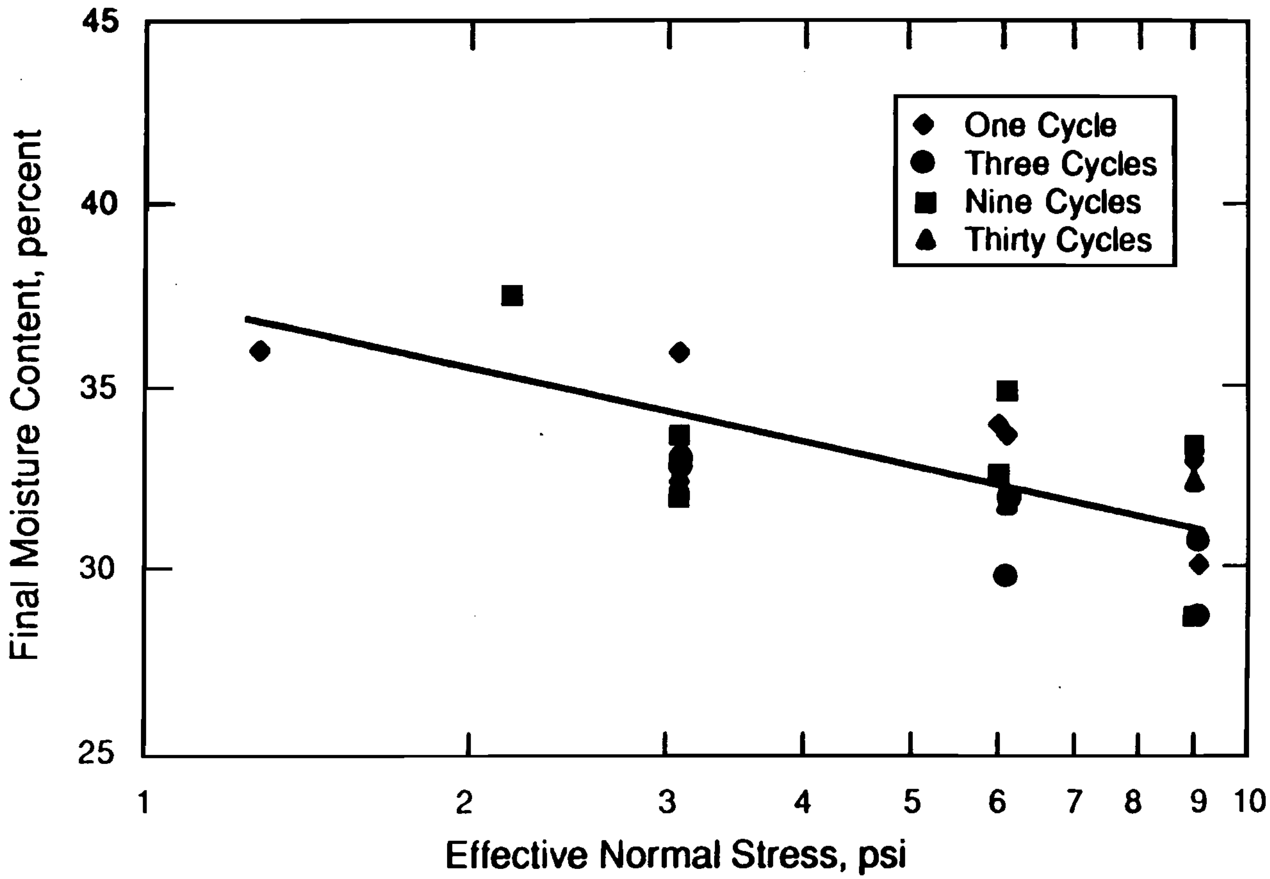

Figure 3.3 Final Moisture Content After Shear vs. Effective Normal Stress for Direct Shear Specimens

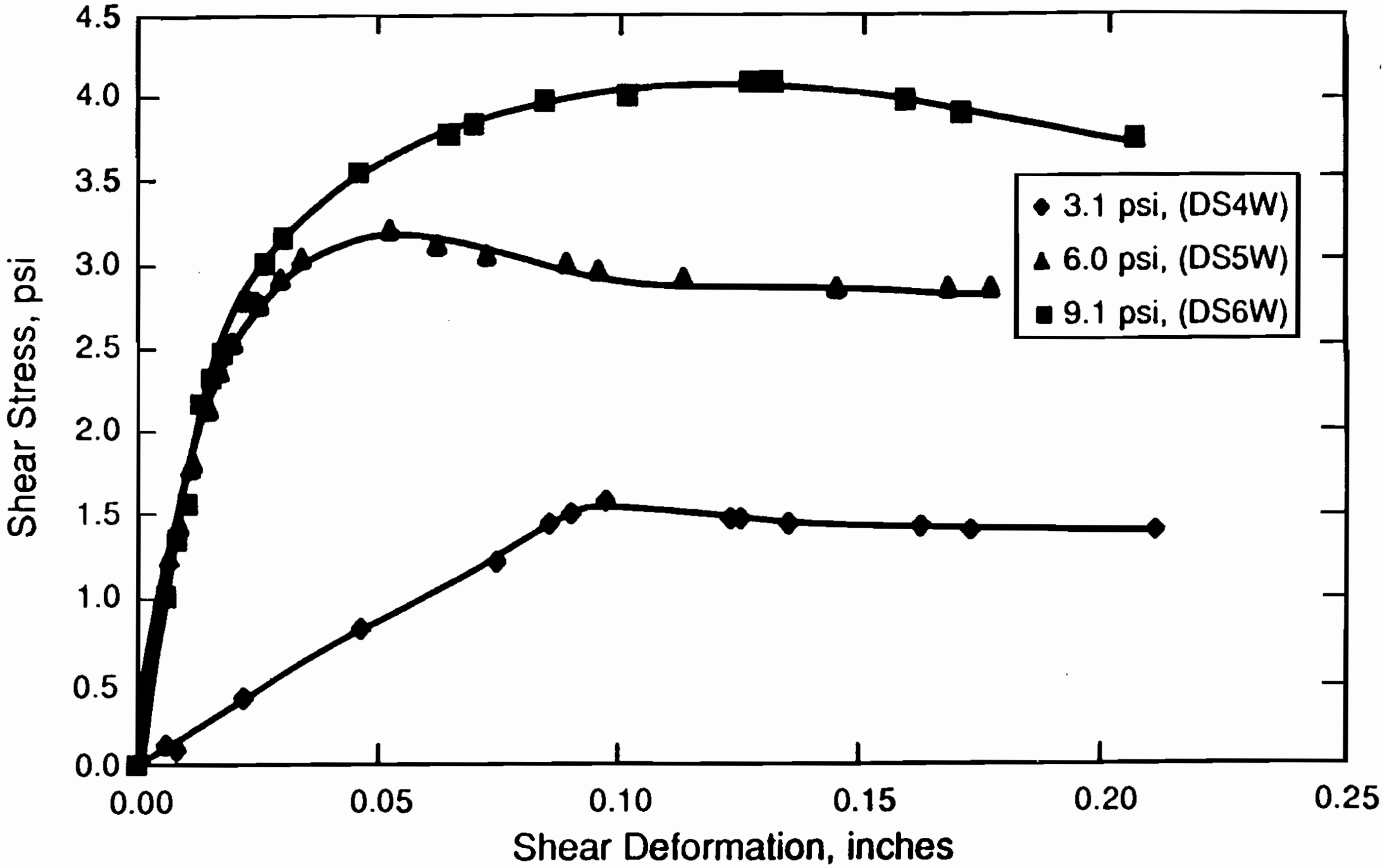

Figure 3.4 Shear Stress vs. Shear Deformation for Direct Shear Tests DS4W-DS6W, One Cycle of Wetting and Drying

TABLE 3.3. STRESS DEFORMATION PROPERTIES

Direct Shear Test Number/ Number of Cycles	Effective Normal Stress, psi	Peak Shear Stress, psi	Deformation at Peak Shear Stress, psi	Time to Failure, min
DS14W/1	1.3	0.66	0.075	2660
DS4W/1	3.1	1.58	0.098	3450
DS11W/1	3.1	1.56	0.083	2920
DS5W/1	6.0	3.21	0.054	2479
DS12W/1	6.1	2.40	0.058	4185
DS13W/1	9.0	4.00	0.079	4980
DS6W/1	9.1	4.08	0.096	2790
DS1W/3	3.1	1.97	0.180	6242
DS10W/3	3.1	2.17	0.154	5423
DS2W/3	6.1	3.72	0.098	3555
DS15W/3	6.1	2.90	0.170	2329

(continued)

TABLE 3.3. CONTINUED

Direct Shear Test Number/ Number of Cycles	Effective Normal Stress, psi	Peak Shear Stress, psi	Deformation at Peak Shear Stress, inches	Time to Failure, min
DS3W/3	9.0	4.98	0.136	4322
DS16W/3	9.0	3.83	0.117	4095
DS17W/9	2.2	1.18	0.154	5428
DS7W/9	3.1	1.56	0.136	4813
DS8W/9	6.0	3.30	0.070	4265
DS18W/9	6.1	2.95	0.098	3271
DS9W/9	9.0	4.53	0.069	7040
DS19W/9	9.0	4.00	0.098	4315
DS20W/30	3.1	1.31	0.075	2637
DS21W/30	6.1	6.10	0.159	5883
DS22W/30	9.0	4.86	0.088	5657

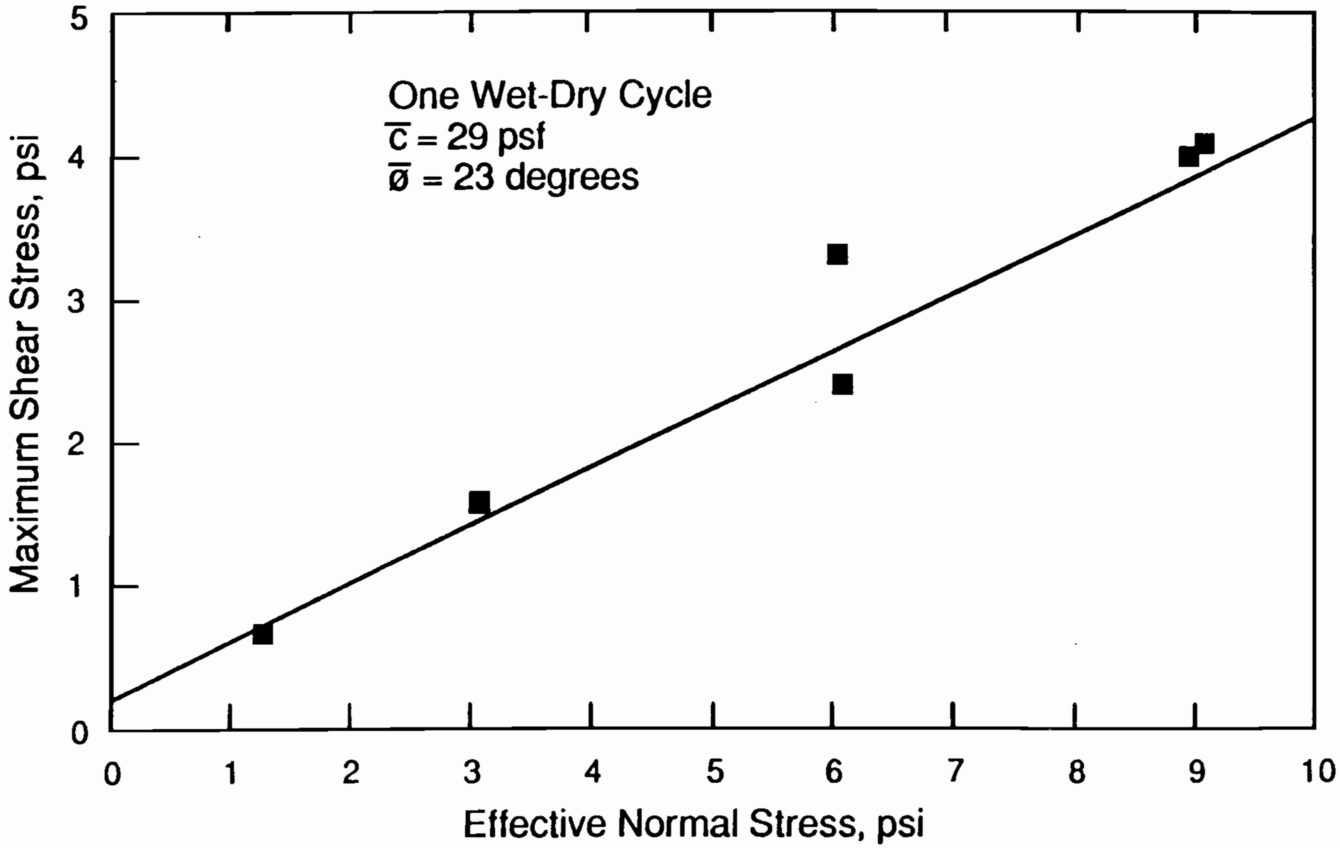

Figure 3.5 Mohr-Coulomb Diagram of Maximum Shear Stress vs. Effective Normal Stress for Direct Shear Tests DS4W-DS6W, DS11W-DS14W, One Cycle of Wetting and Drying

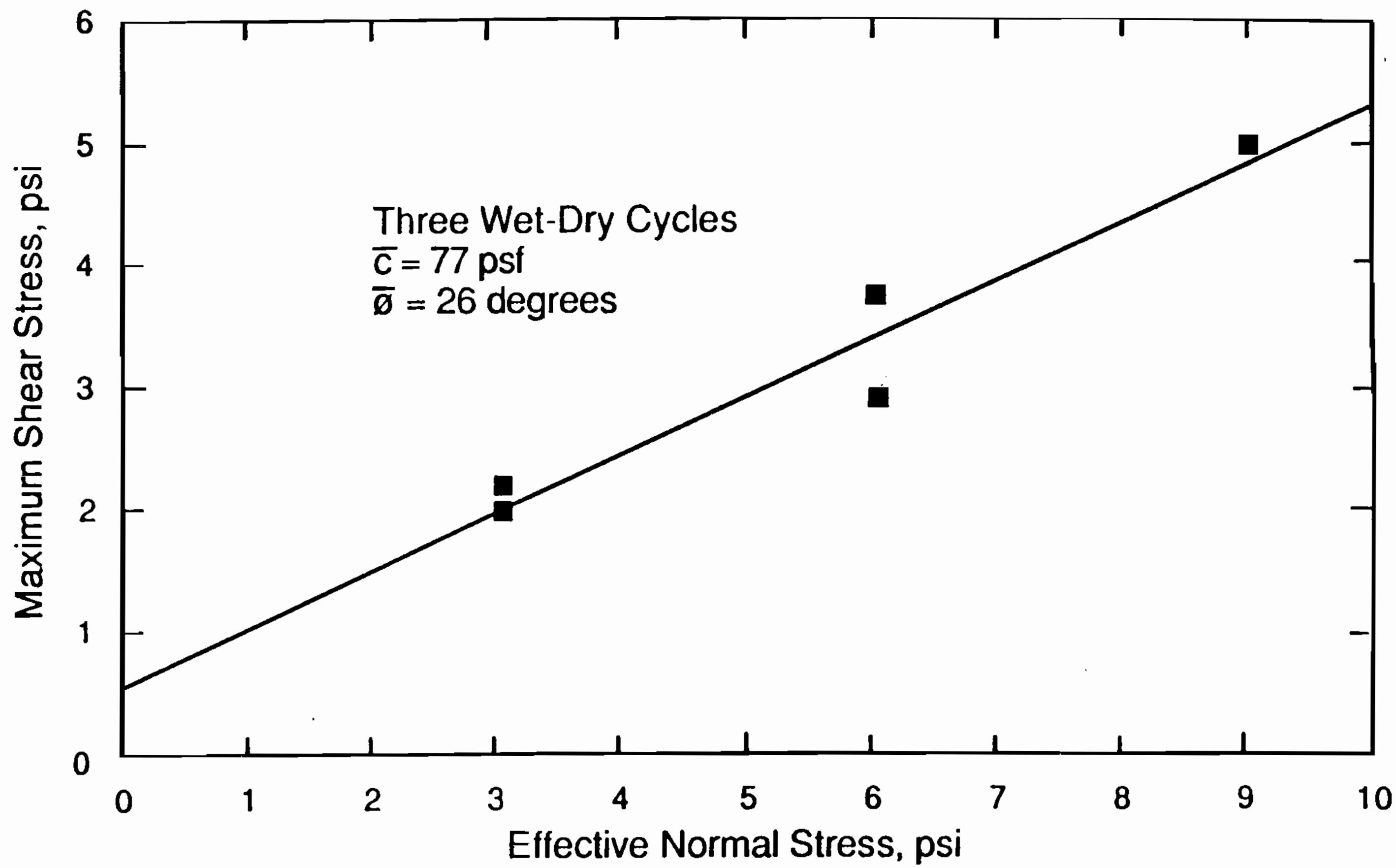

Figure 3.6 Mohr-Coulomb Diagram of Maximum Shear Stress vs. Effective Normal Stress for Direct Shear Tests DS1W-DS3W, DS10W and DS15W, Three Cycles of Wetting and Drying

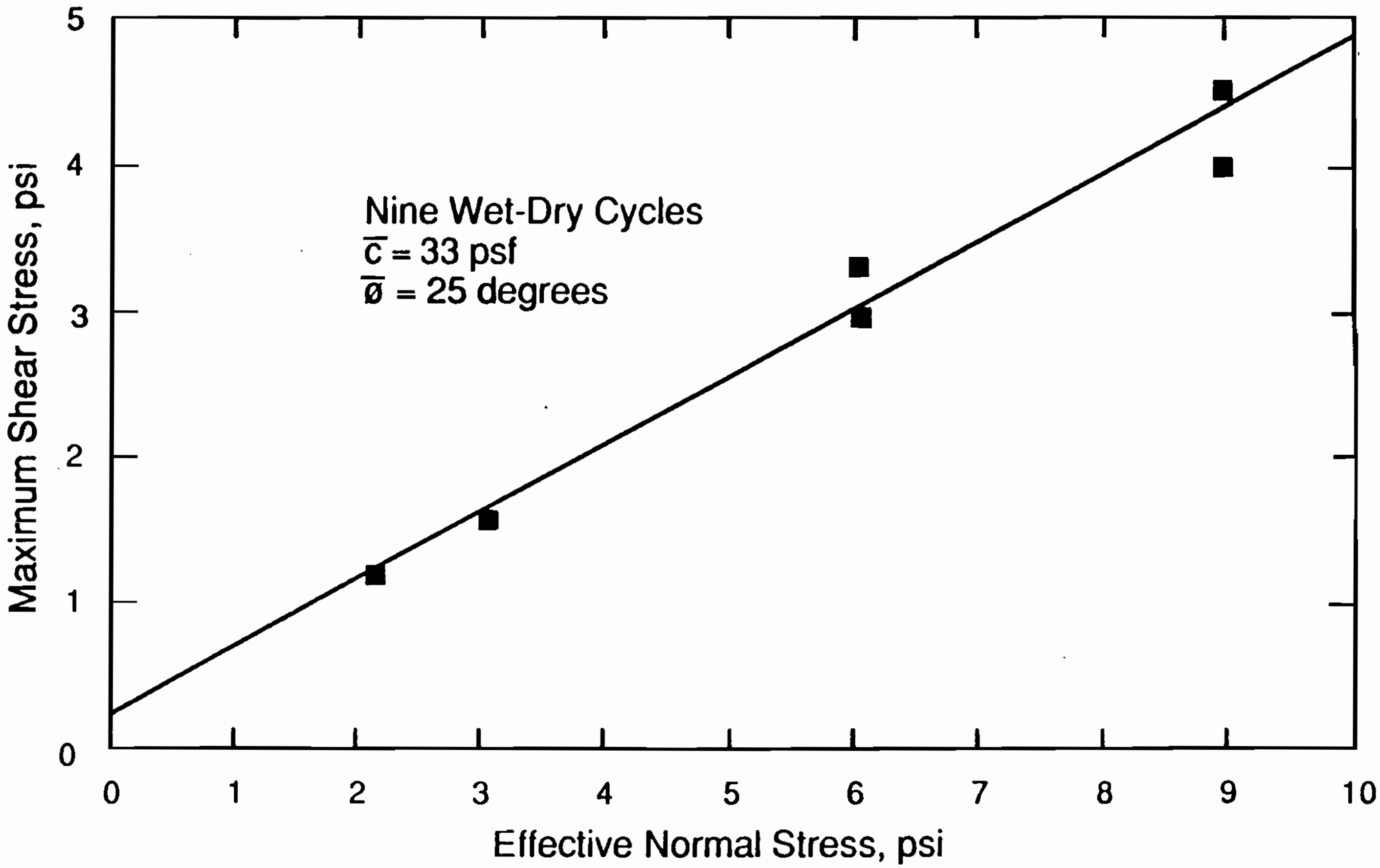

Figure 3.7 Mohr-Coulomb Diagram of Maximum Shear Stress vs. Effective Normal Stress for Direct Shear Tests DS7W-DS9W and DS17W-DS19W, Nine Cycles of Wetting and Drying

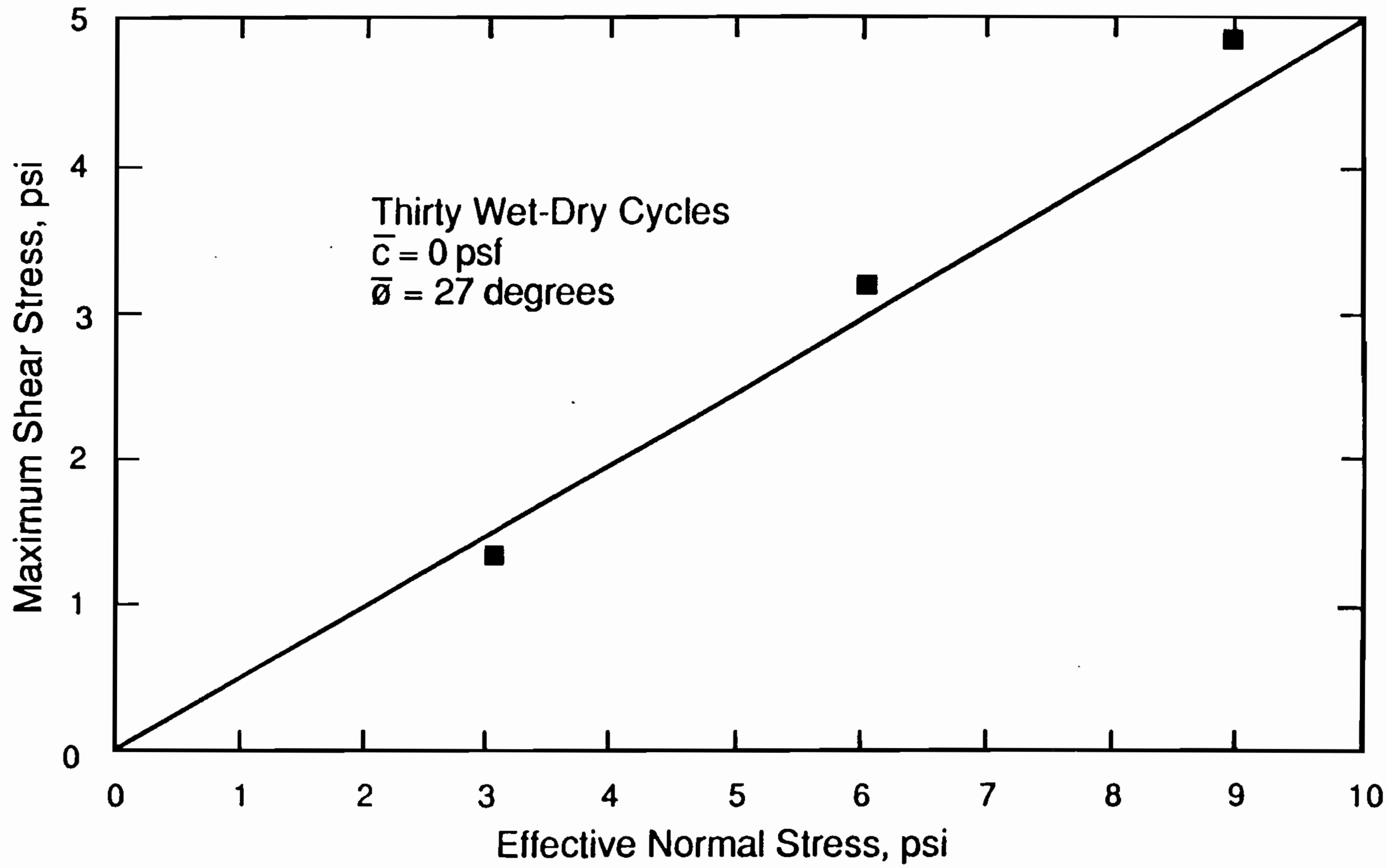

Figure 3.8 Mohr-Coulomb Diagram of Maximum Shear Stress vs. Effective Normal Stress for Direct Shear Tests DS20W, DS21W and DS22W, Thirty Cycles of Wetting and Drying

The effective-stress shear-strength parameters are summarized in Table 3.4. Effective cohesion values of 29, 77, and 33 psf were measured for specimens exposed to one, three, and nine wet-dry cycles, respectively. The effective angle of internal friction values were measured to be 23, 26, and 25 degrees. Further discussion of these results is presented in Chapter 5.

Values of effective cohesion were also calculated assuming a representative friction angle of 21.1 degrees, which corresponds to the peak shear strength measured by Green and Wright (1986) from direct shear tests on compacted "Beaumont" clay. The c values calculated were 61 psf, 147 psf, -14 psf and 126 psf for specimens subjected to one, three, nine and thirty cycles of wetting and drying and are tabulated in Table 3.5. The effective-stress shear-strength parameters measured in the direct shear tests are discussed in further detail in Chapter Five.

TABLE 3.4. EFFECTIVE STRESS SHEAR STRENGTH PARAMETERS DERIVED FROM DIRECT SHEAR TESTS

Number of Wet-Dry Cycles	Cohesion, $\bar{c}$ (psf)	Friction Angle, $\bar{\phi}$ (degrees)
1	29	23
3	77	26
9	33	25
30	0	27

TABLE 3.5. EFFECTIVE STRESS COHESION VALUE FROM DIRECTOR SHEAR TEST DATA ASSUMING A CONSTAT FRICTION ANGLE

Number of Wet-Dry Cycles	Cohesion, $\overline{c}$ (psf)	Friction Angle, $\overline{\phi}$ * (degrees)
1	61	21.1
3	147	21.1
9	-14	21.1
30	126	21.1

*From Green and Wright (1986)

CHAPTER 4. TRIAXIAL SHEAR TESTS

INTRODUCTION

Sixteen consolidated-undrained triaxial shear tests with pore pressure measurement were performed on specimens of compacted "Beaumont" clay which had been subjected to wetting and drying. Six, ten and thirty cycles of wetting and drying were used. Test procedures and results are presented in this chapter.

TEST PROCEDURES

Specimens were compacted with clay prepared with the screen device described in Chapter Two. Following the wetting phase of the final cycle of wetting and drying, each specimen was extruded from the acrylic cylinder used for the wetting and drying. Approximately 0.20 inch of soil was trimmed from the upper portion of the specimen with a wire saw. The top of the acrylic cylinder served as a straight edge to obtain a smooth and level surface on the upper end of the specimen. After trimming the top of the specimen, the remainder of the specimen, which constituted the length to be tested, was extruded from the cylinder. Trimming of the base was attempted on several specimens; however, the clay had a tendency to adhere to both the trimming device and mold resulting in large voids on the surface of the specimen. Accordingly, any further trimming of the specimens was avoided.

Triaxial tests were performed with equipment in two laboratories, one located on the sixth floor and the other in the basement of Ernest Cockrell Jr. Hall at The University of Texas. The triaxial cells and related equipment were also used by Gourlay and Wright (1984) and Green and Wright (1986) and are described by the former. Triaxial specimens possessed ample integrity to be hand-held during transferral from the cylinder used for wetting and drying to the base pedestal in the triaxial cell; however, caution was exercised to reduce possible disturbance to the specimen. The triaxial specimen was placed on top of a porous stone, which was covered with a 1.5-inch disk of filter paper. The porous stone and pore pressure lines leading to the base pedestal of the triaxial cell were saturated with deaired water prior to set-up of the specimen. An acrylic top cap was placed directly on the upper surface of the specimen. A vertical filter paper drain was cut in alternating vertical strips from Whatman No. 1 chromatography paper, moistened and wrapped around the specimen; the paper covered 50 percent of the perimeter of the specimen and overlapped the edge of the porous stone at its base.

An initial membrane was lowered over the specimen using a vacuum membrane expander. The ends of the membrane were sealed to the top cap and base pedestal with rubber o-rings. A second membrane was then placed over the first, and also sealed with o-rings.

The remainder of the procedures for apparatus set-up, back-pressure saturation, consolidation, and shearing of the triaxial specimens were similar to those used for the triaxial tests performed by Green and Wright (1986).

BACK-PRESSURE SATURATION

Periods ranging from one to two weeks were necessary to saturate the soil specimens. Saturation was determined based on measured values of Skempton's (1954) B coefficient. B values were determined using a thirty second response time after an increase in cell pressure. Upon completion of back pressure saturation, B values for the triaxial specimens ranged from 0.97 to 1.0; back pressures ranged from 14.5 psi to 30 psi.

TRIAXIAL CONSOLIDATION

Moisture contents after shearing are plotted versus the logarithm of effective consolidation pressures in Figure 4.1. Tests performed at higher consolidation pressures yielded specimens with lower water contents after shearing as expected.

The volume of water entering and leaving the specimen was measured during saturation and consolidation. Six specimens, consolidated to effective stresses of 3.8 psi and greater, yielded good consolidation data, marked by significant, measurable volume changes in the specimen.

Typical plots of volumetric strain during consolidation versus both the logarithm of time and square root of time are shown in Figures 4.2 and 4.3, respectively, for Test 6.34 at an effective consolidation pressure of 9.2 psi. The remaining tests, which yielded good consolidation data, were Test 6.29 at 3.8 psi, Tests B.19 and B.20 at 9.0 psi, Test 6.36 at 19.8 psi and B.21 at

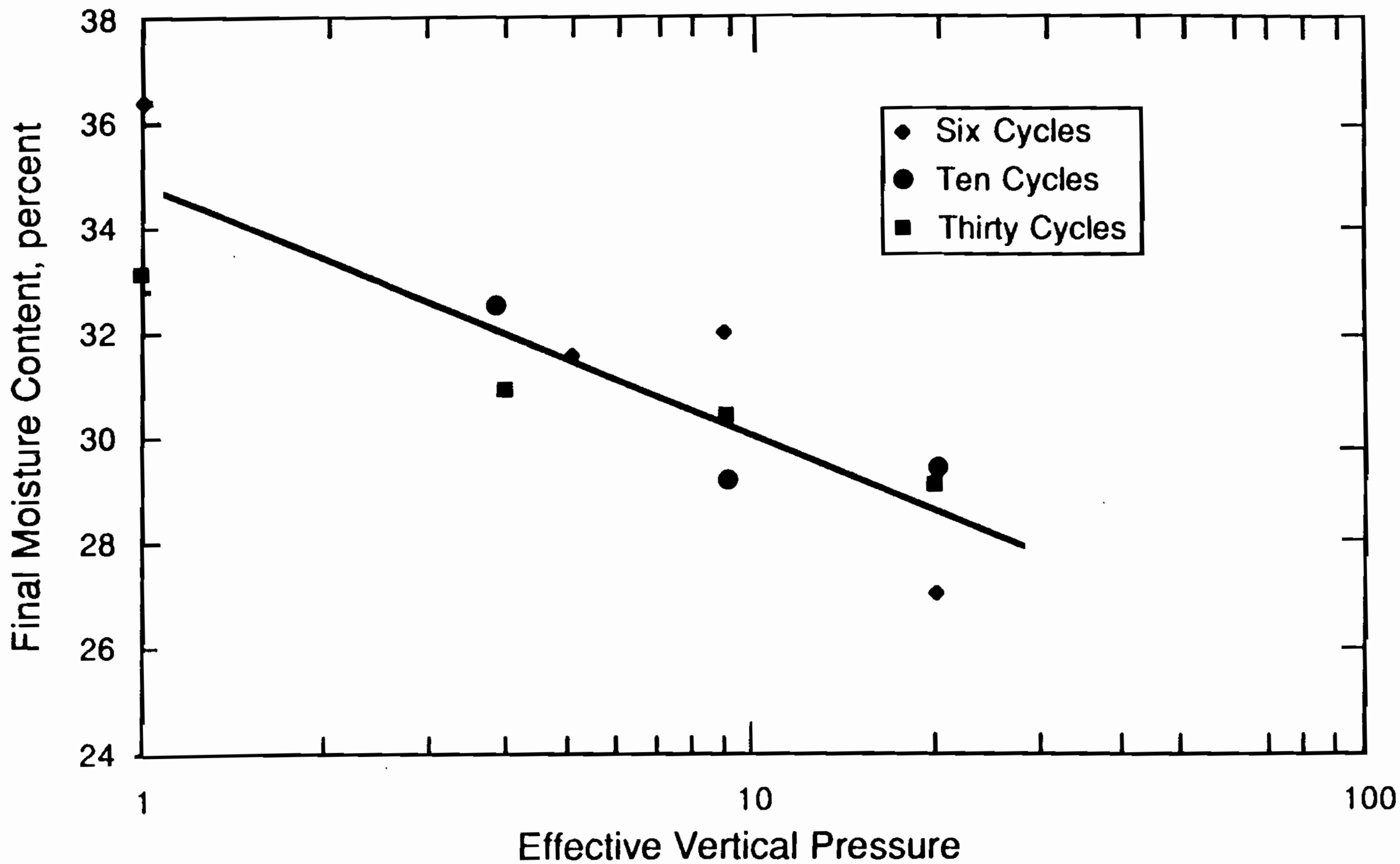

Figure 4.1 Final Moisture Content After Shear vs. Effective Vertical Pressure

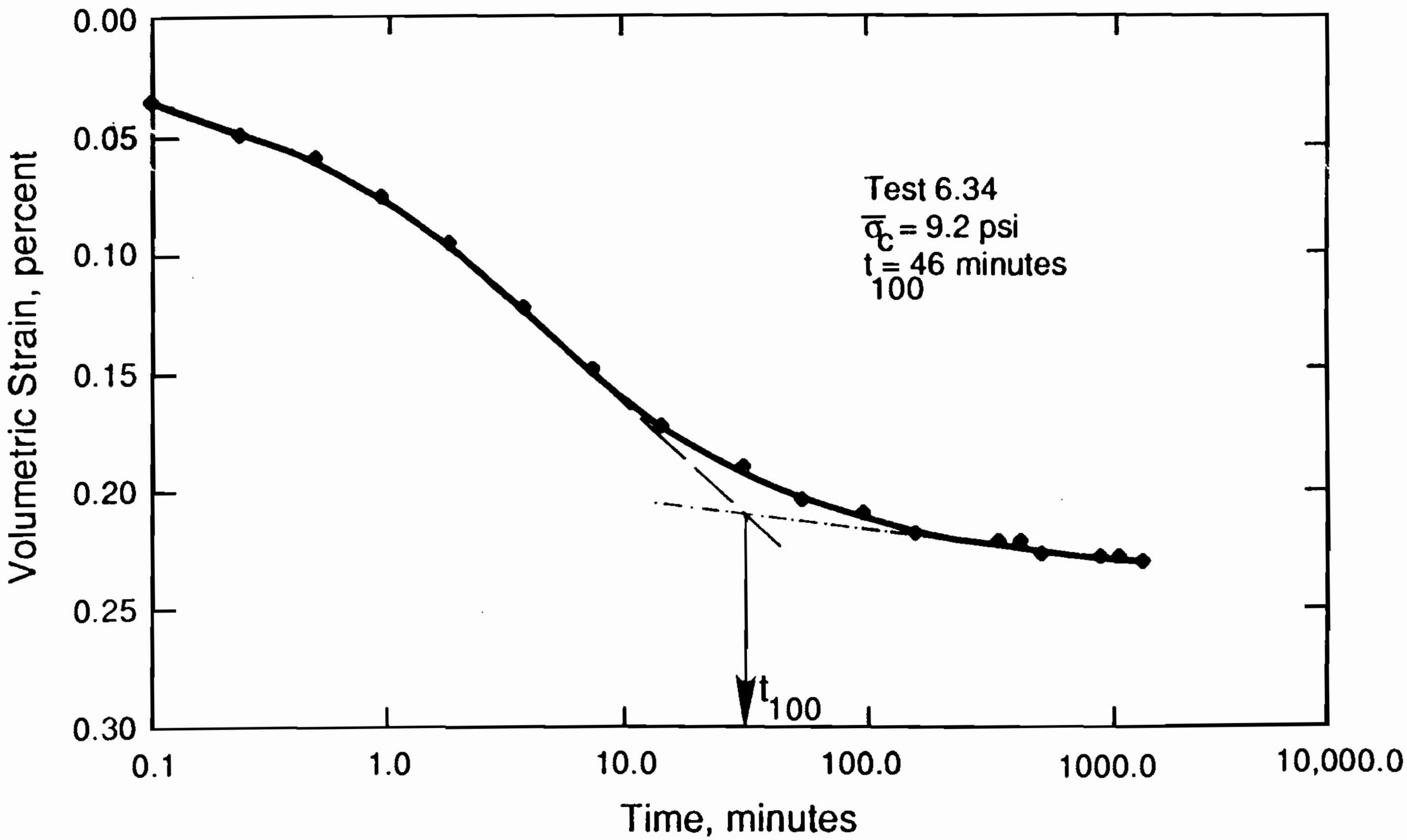

Figure 4.2 Volumetric Strain vs. Logarithm of Time During Final Consolidation of Test 6.34, Six Cycles of Wetting and Drying

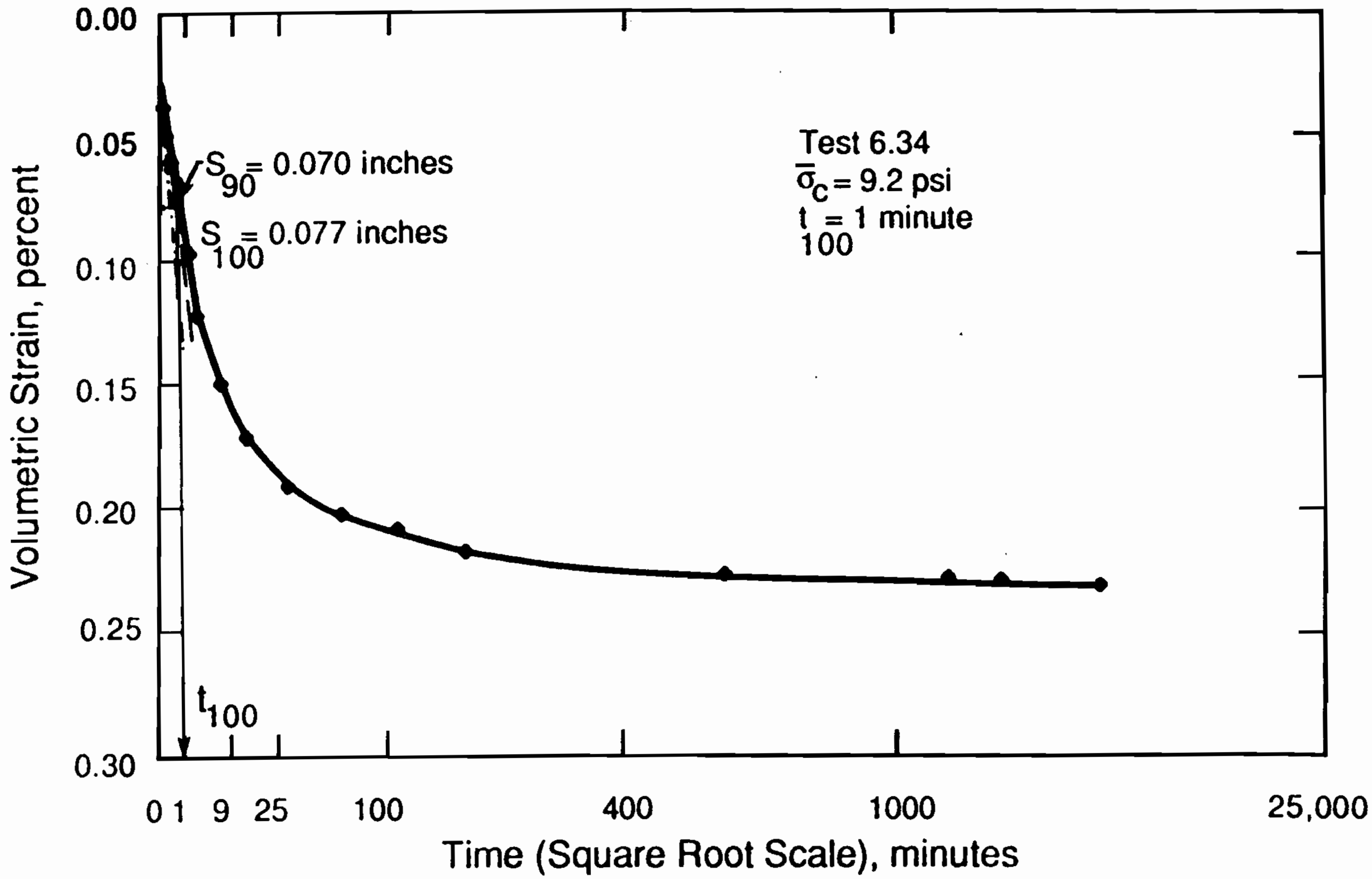

Figure 4.3 Volumetric Strain vs. Square Root of Time During Final Consolidation of Test 6.34, Six Cycles of Wetting and Drying

20.0 psi. Plots of volumetric strain versus the logarithm of time for these tests are included in Appendix B.

The time required to complete primary consolidation, t_{100}, for Test 6.34 was determined using both the logarithm of time and the square root of time methods. The value of t_{100} determined by the logarithm of time procedure was 45 minutes, compared with a value of 1 minute determined by the square root of time method.

The principal interest in determining values of t_{100} was for use in estimating the required times to failure for the subsequent shear of each specimen. The larger the values of t_{100}, the greater the times required for failure and hence the slower the required rate of shear. Because the logarithm of time approach produced longer t_{100} values, the time required for primary consolidation was calculated using the volumetric strain versus logarithm of time plots.

LOADING RATES FOR SHEAR

Deformation rates for shear were estimated and selected to insure pore water pressure equilibration throughout the specimens. A deformation rate for shear of 0.0017 in/hr was used. This rate represents the slowest rate possible on the Wykeham Farrance loading presses used and was judged to be more than slow enough to produce pore pressure equilibration. To verify that the selected rate of shear was adequate, theoretical times to failure were calculated using the equation proposed by Blight (1963):

$$t_f = (0.07)(H)^2 / c_v \qquad (4.1)$$

where c_v is the coefficient of consolidation and H is one-half the height of the specimen during consolidation. (The initial height of each specimen prior to consolidation was used to compute heights.)

Coefficients of consolidation were calculated for those specimens yielding good consolidation data with the following equation presented by Bishop and Henkel (1963):

$$c_v = (\pi)(H)^2 / (81)t_{100} \qquad (4.2)$$

Times required for primary consolidation, coefficients of consolidation, and theoretical minimum times to failure are presented in Table 4.1. Actual times to failure, defined as the time where the effective stress path first became tangent to the Mohr-Coulomb failure envelope, are also included in Table 4.1. All of the actual times to failure (39 hours to 256 hours) greatly exceeded the thereoretical minimum times to failure (1.4 hours to 12.6 hours) calculated with Equation 4.1, demonstrating that the selected loading rate was adequate.

Specimens were sheared for approximately 14 days to axial strains of aproximately 15 percent or greater; Test 6.36, however, was stopped after 7 days with only 9 percent strain due to excessive lateral movements of the top cap and tilting of the specimen. At 15 percent strain, the lateral deformation of the specimens was typically large and irregular; any measured stresses at higher strains would be questionable due to the uncertainty of the cross-sectional area of the specimen.

SHEAR TEST RESULTS

Effective-stress paths and stress-strain data were plotted for each of the triaxial shear tests. The effective stress paths were used to determine appropriate effective-stress shear-strength parameters based upon the points where the effective stress path first became tangent to the failure envelope for each test.

Stress-Strain Response

The principal stress difference, (σ_1 - σ_3), is plotted versus axial strain, ε, for specimens subjected to six, ten and thirty cycles of wetting and drying in Figures 4.4, 4.5 and 4.6, respectively. The curves are corrected for such effects as seating errors. Corrections were also made for the presence of filter papers and membranes using the method established by Duncan and Seed (1965).

The stress-strain curves for the specimens generally indicated a peak with a negligible to moderate decrease in principal stress difference with increasing strain. The axial strain required to achieve the peak principal stress difference generally increased with the effective consolidation pressure. Values of axial strain and peak principal stress difference are summarized in Table 4.2.

TABLE 4.1. CONSOLIDATION PARAMETERS FOR TRIAXIAL SPECIMENS

Triaxial Test Number	Effective Consolidation Pressure, psi	t_{100} min	c_v in^2/min	Theoretical Mininum Time to Failure, min	Time to Failure, min
6.34	9.2	46	.00238	83	2356
B.21	20.0	420	.00021	758	15,350
6.29	3.8	110	.00085	199	2692
B.19	9.0	190	.00046	343	2487
B.20	9.0	100	.00070	181	5217
6.36	19.8	285	.00025	514	3528

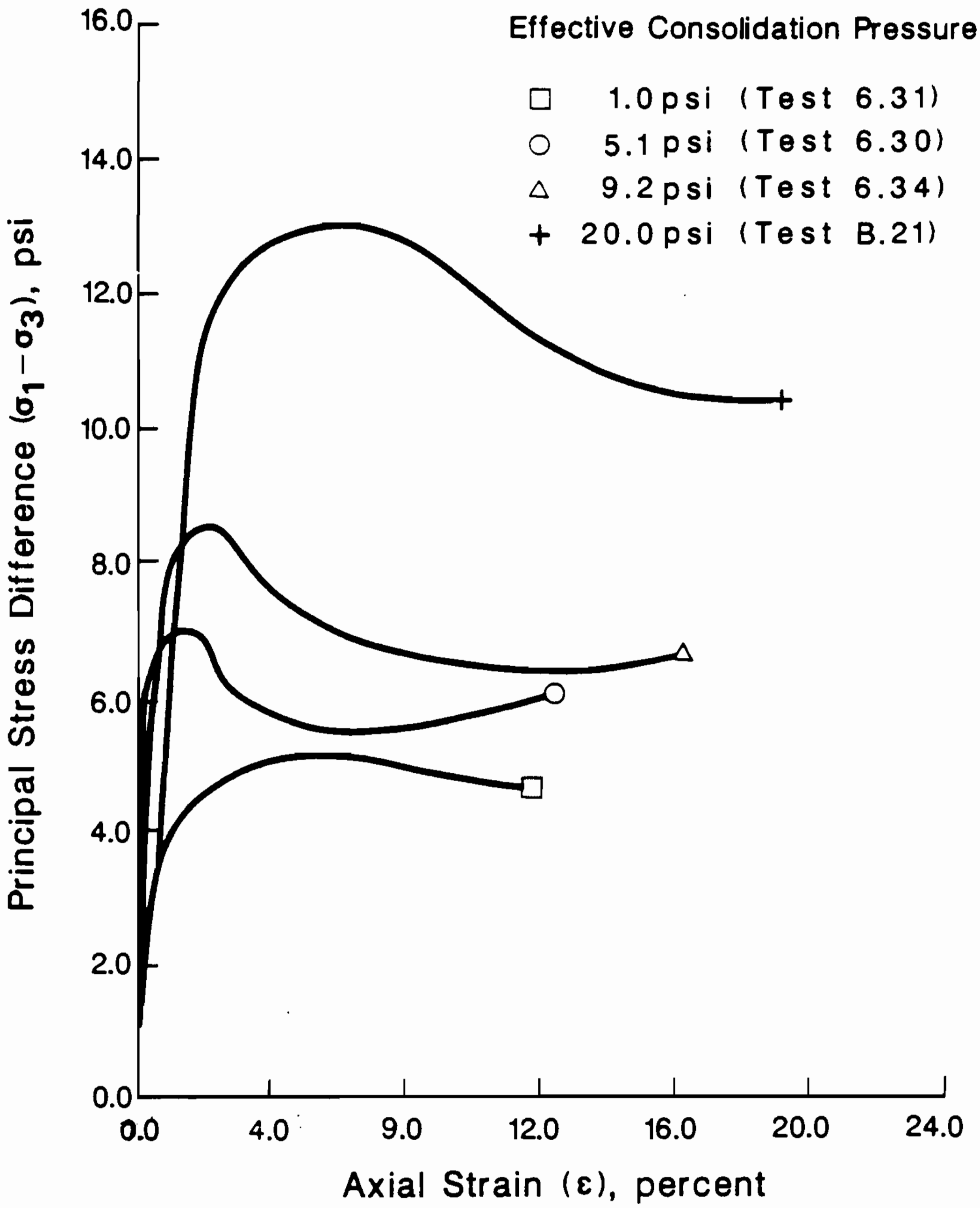

Figure 4.4 Principal Stress Difference vs. Axial Strain for Consolidated-Undrained Triaxial Shear Tests Subjected to Six Cycles of Wetting and Drying

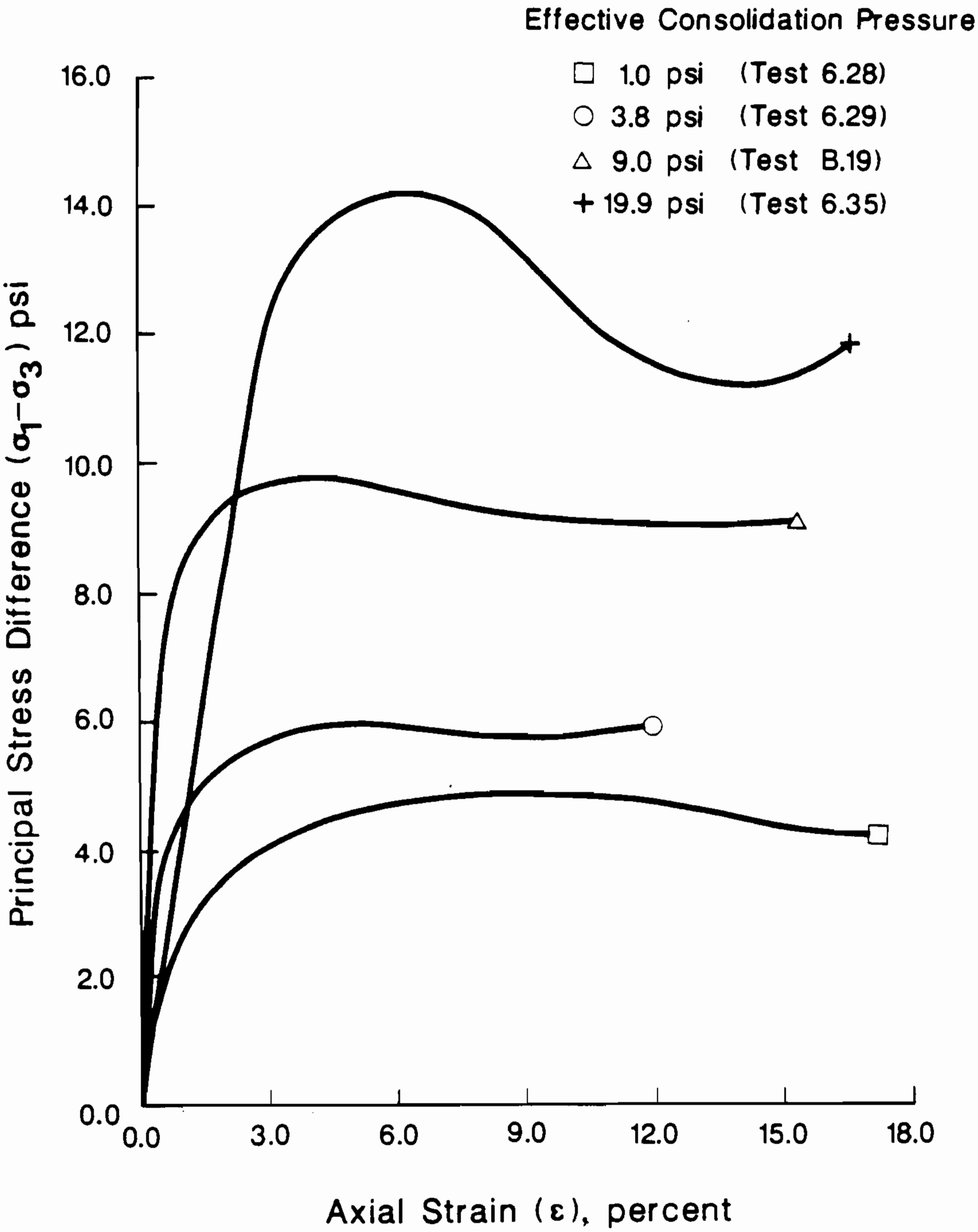

Figure 4.5 Principal Stress Difference vs. Axial Strain for Consolidated-Undrained Triaxial Shear Tests Subjected to Ten Cycles of Wetting and Drying

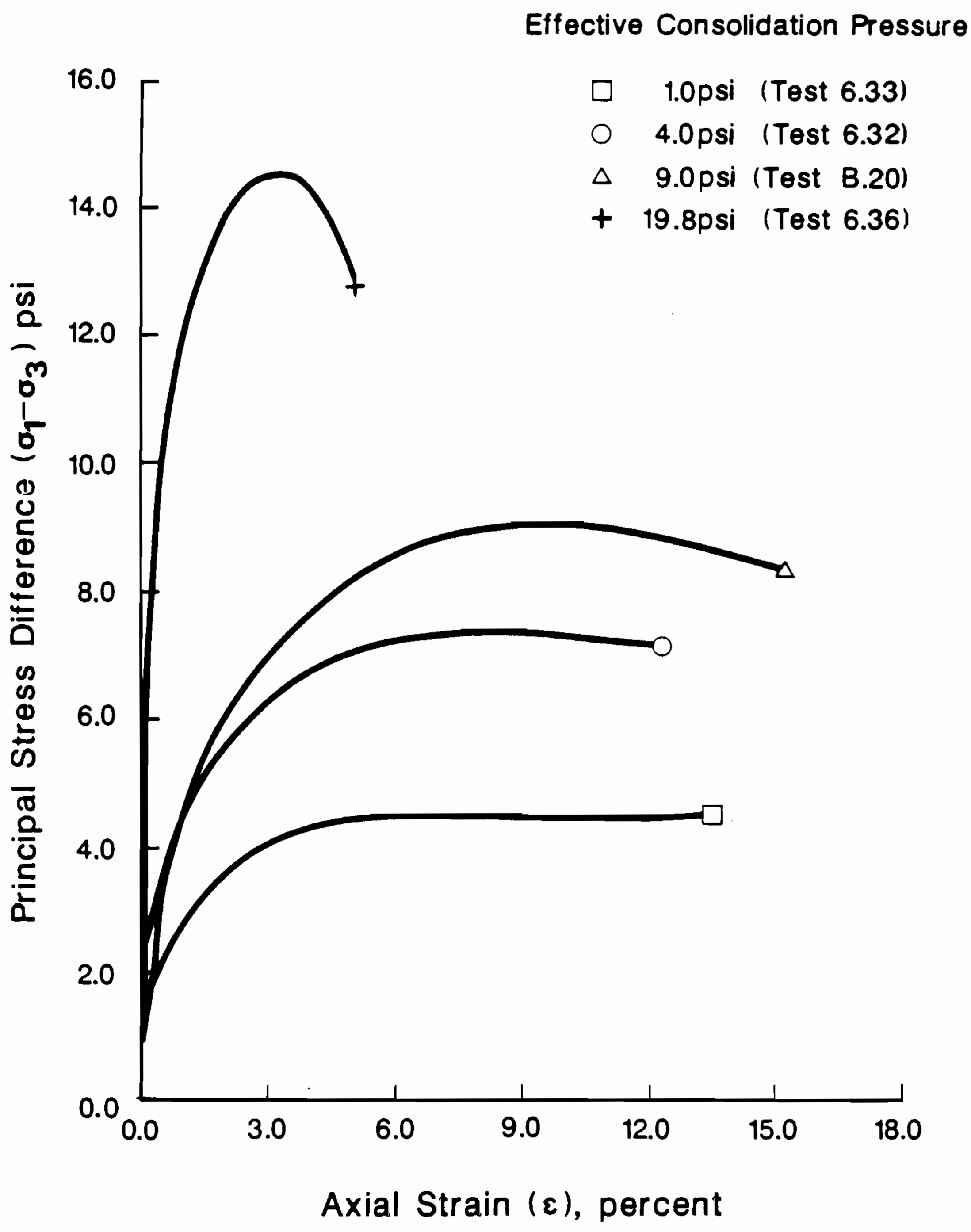

Figure 4.6 Principal Stress Difference vs. Axial Strain for Consolidated-Undrained Triaxial Shear Tests Subjected to Thirty Cycles of Wetting and Drying

TABLE 4.2. AXIAL STRAIN AND PEAK PRINCIPAL STRESSS DIFFERENCE FOR TRIAXIAL TESTS

Triaxial Test Number	Number of Cycles	Effective Consolidation Pressure, psi	Peak $(\sigma_1 - \sigma_3)$ psi	Axial Strain, percent
6.31	6	1.0	4.91	4.18
6.30	6	5.1	7.09	1.36
6.34	6	9.2	8.65	1.95
B.21	6	20.0	11.58	14.44
6.28	10	1.0	4.95	9.55
6.29	10	3.8	5.87	2.74
B.19	10	9.0	9.61	2.10
6.35	10	19.9	13.51	10.10
6.33	30	1.0	4.58	6.12
6.32	30	4.0	7.16	12.10
B.20	30	9.0	9.05	8.85
6.36	30	19.8	14.56	3.32

The smallest axial strain is 1.3 percent which occurred in Test 6.30 with an effective consolidation pressure of 5 psi, while the largest value is 14.4 percent, which occurred in Test B.21 at 20 psi.

Effective Stress Paths

Effective stress paths for the specimens subjected to six, ten and thirty cycles of wetting and drying are presented in Figures 4.7, 4.8 and 4.9 respectively; the stress paths are shown using a modified Mohr-Coulomb diagram with principal stress difference, (σ_1 - σ_3), as the ordinate and the minor principal effective stress, σ_3, as the abscissa.

Straight line failure envelopes are shown in the effective stress paths in figures 4.7, 4.8 and 4.9. The envelopes represent "average" envelopes tangent to the effective stress paths shown. A linear regression analysis was performed using estimated points of stress path tangency to fit the failure envelopes shown. Values of pincipal stress difference, (σ_1 - σ_3), and the minor principal effective stress, σ_3, used in the linear regression analyses are shown in Table 4.3. The corresponding axial strain and elapsed time to failure are presented in Table 4.4.

The point of stress path tangency generally coincided with the point of peak principal stress difference for tests run at effective consolidation pressures greater than 5 psi. However, Test B.20, which was run at 9.0 psi, and tests performed at consolidation pressures of 5 psi or less showed a continuing increase in principal stress differences after first reaching the point of stress path tangency. In all tests, the effective stress paths dropped away from the failure envelopes in the latter stages of shearing, providing clear evidence that the triaxial tests were carried well beyond the point necessary to define the effective stress shear strength parameters.

Shear Strength Parameters

The slope angle and intercept of the failure envelopes shown on the modified Mohr-Coulomb diagrams in Figures 4.7, 4.8 and 4.9 are designated as ψ and d respectively. Values of ψ and d are 46 degrees and 690 psf for specimens subjected to six cylces of wetting and drying, 45 degrees and 720 psf for specimens subjected to ten cycles of wetting and drying and 51 degrees and 639 psf for thirty cycles of wetting and drying. Corresponding effective-stress shear-strength parameters ϕ and c on a Mohr-Coulomb diagram (un-modified diagrams) were calculated from the following equations:

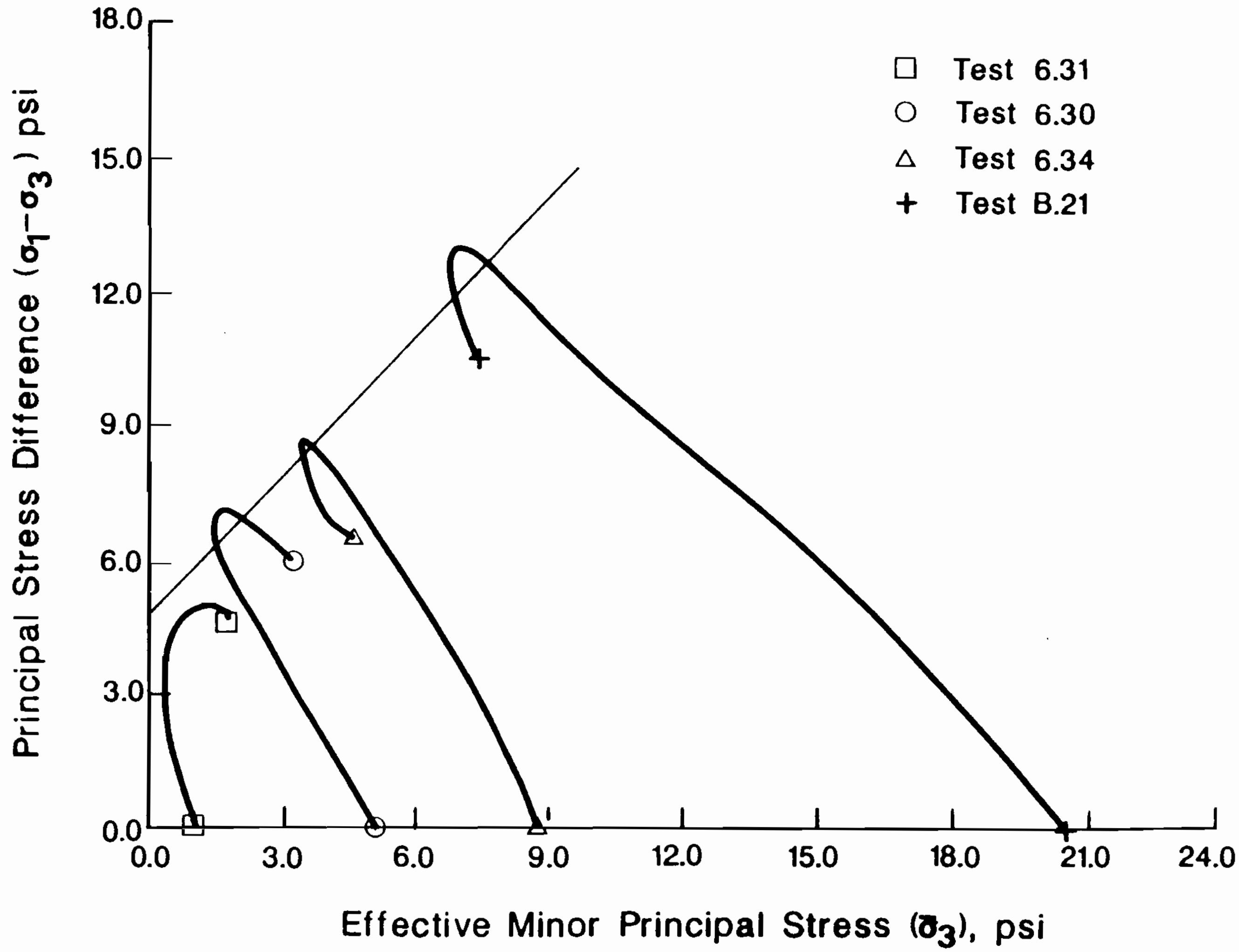

Figure 4.7 Principal Stress Difference vs. Effective Minor Prncipal Stress for Consolidated-Undrained Triaxial Shear Tests Performed on Specimens Subjected to Six Cycles of Wetting and Drying

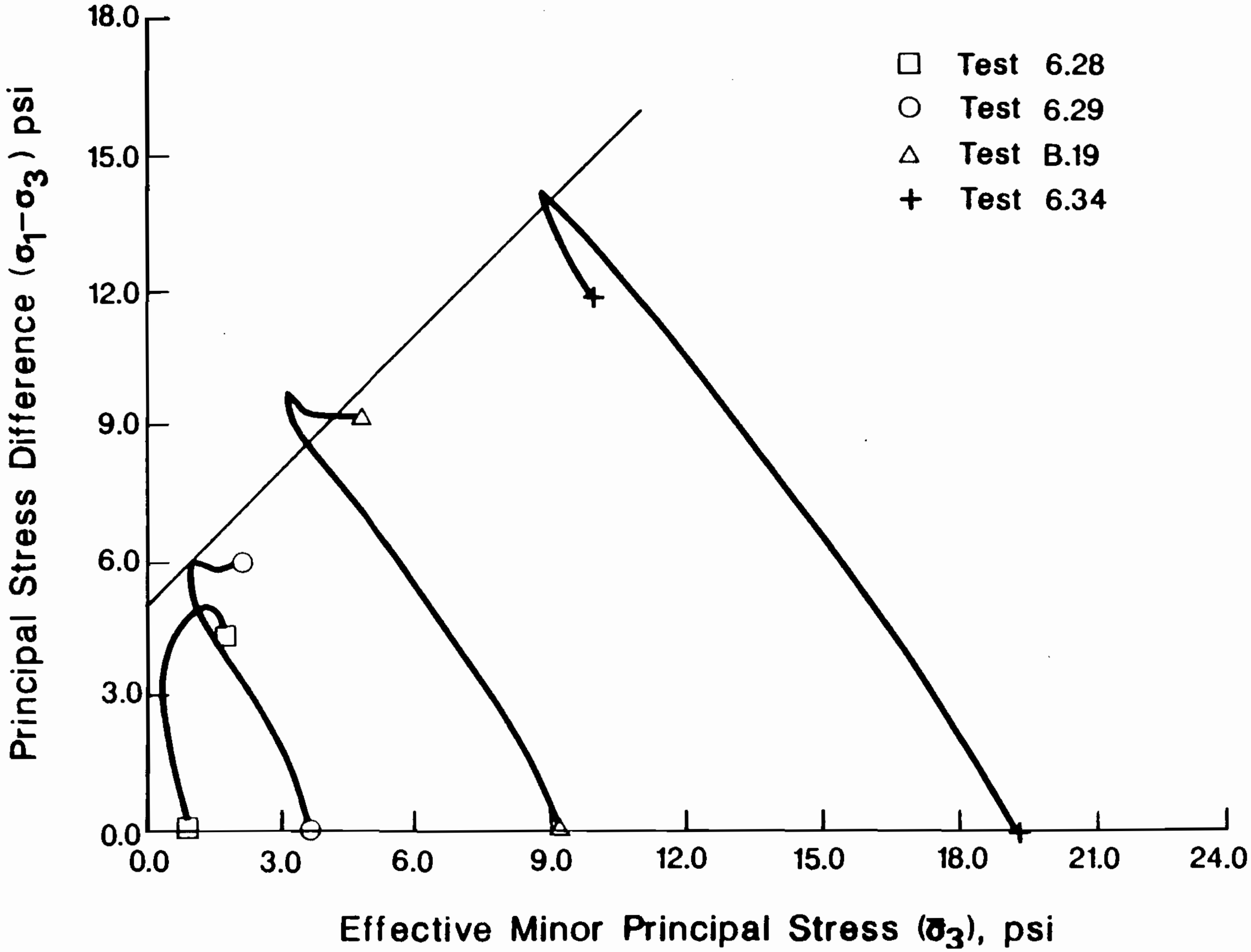

Figure 4.8 Principal Stress Difference vs. Effective Minor Prncipal Stress for Consolidated-Undrained Triaxial Shear Tests Performed on Specimens Subjected to Ten Cycles of Wetting and Drying

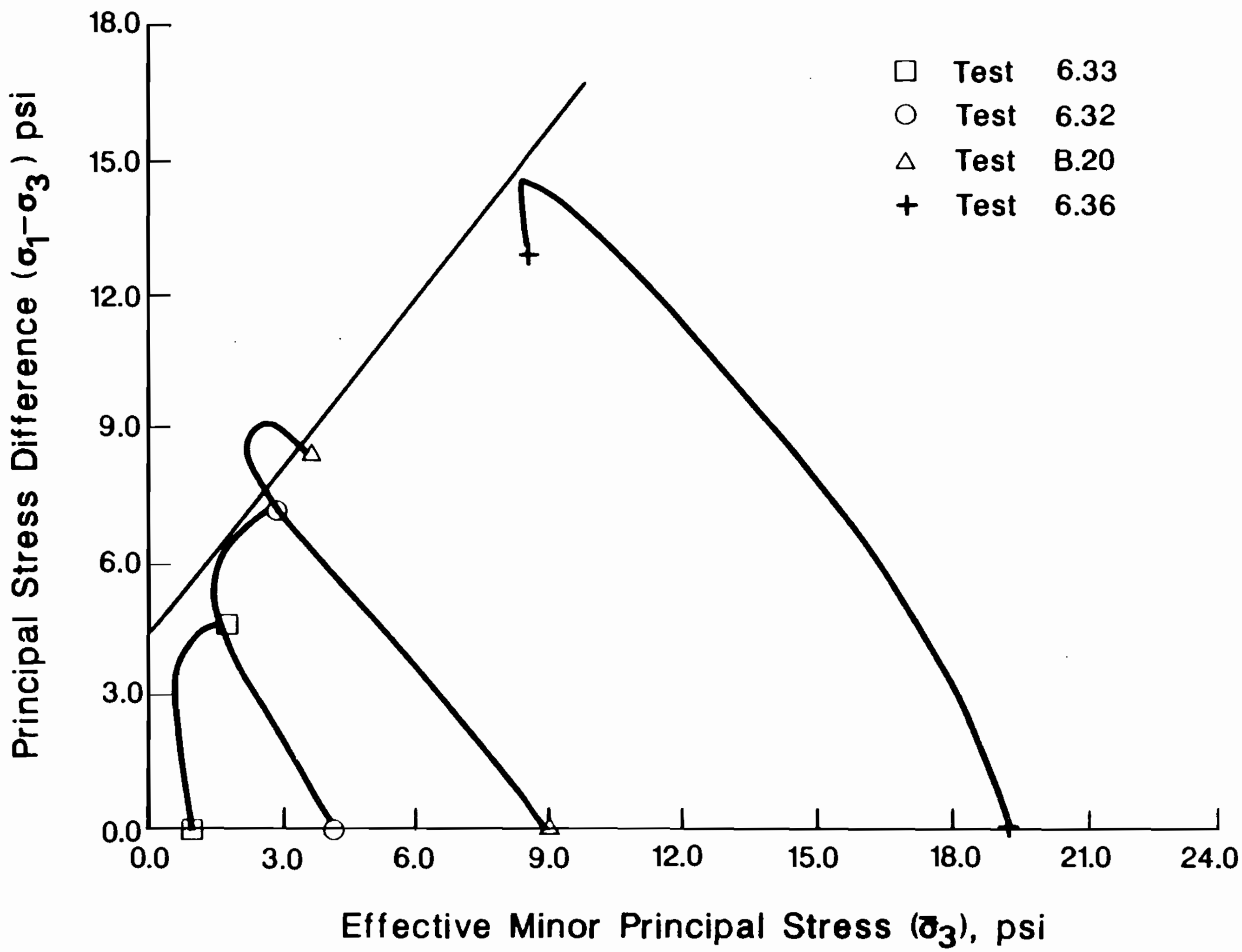

Figure 4.9 Principal Stress Difference vs. Effective Minor Prncipal Stress for Consolidated-Undrained Triaxial Shear Tests Performed on Specimens Subjected to Thirty Cycles of Wetting and Drying

TABLE 4.3. APPARENT POINTS OF STRESS PATH TANGENCY

Triaxial Test Number	Number of Cycles	Effective Consolidation Pressure, psi	$(\sigma_1 - \sigma_3)$ psi	$\overline{\sigma}_3$ psi
6.31	6	1.0	4.61	0.56
6.30	6	5.1	7.00	1.50
6.34	6	9.2	8.64	3.41
B.21	6	20.0	11.58	6.86
6.28	10	1.0	4.78	0.81
6.29	10	3.8	5.78	0.88
B.19	10	9.0	9.61	3.09
6.35	10	19.9	13.52	8.95
6.33	30	1.0	4.22	0.87
6.32	30	4.0	6.74	1.91
B.20	30	9.0	8.75	2.15
6.36	30	19.8	14.56	8.37

TABLE 4.4. AXIAL STRAIN AND ELAPSED TIME TO FAILURE FROM STRESS PATH TANGENCY FAILURE ENVELOPES FOR TRIAXIAL TESTS

Triaxial Test Number	Number of Cycles	Effective Consolidation Pressure psi	Time to Failue, min	Axial Strain, percent
6.31	6	1.0	2214	2.04
6.30	6	5.1	1253	1.14
6.34	6	9.2	2356	2.16
B.21	6	20.0	15,350	14.44
6.28	10	1.0	6306	6.10
6.29	10	3.8	2692	2.50
B.19	10	9.0	2487	2.28
6.35	10	19.9	11,344	10.40
6.33	30	1.0	3526	3.63
6.32	30	4.0	3612	3.48
B.20	30	9.0	5217	6.29
6.36	30	19.8	3528	3.38

$$\phi = (\sin^{-1}) [(\tan\psi) / (2 + \tan\psi)] \tag{4.3}$$

$$c = (d) [(1 - \sin\phi) / 2\cos\phi] \tag{4.4}$$

A friction angle, ø, of 20 degrees and a cohesion, c, of 242 psf were calculated from the data for six cycles of wetting and drying. Values of 20 degrees and 253 psf were calculated for ten cycles of wetting and drying, while values of 22 degrees and 213 psf were calculated from the thirty cycles of wetting and drying test data. A summary of these shear strength parameters is presented in Table 4.5.

The effective-stress shear-strength parameters determined in the triaxial tests showed very few trends toward changes with the number of cycles of wetting and drying, although there was some variation in both cohesion and friction angle from one test series to the next. Additional values for the effective cohesion were calculated assuming a representative value of ø of 21.1 degrees for all tests. The cohesion values were 222 psf, 225 psf and 238 psf for specimens subjected to six, ten and thirty cycles of wetting and drying, respectively. Again, no general trend in values with number of cycles is seen in view of the scatter in the data.

Further comparisons of the effective-stress shear-strength parameters with those for specimens which were not subjected to wetting and drying as well as with those from the direct shear tests are presented in Chapter Five. A comparison is also made with effective-stress shear-strength parameters back-calculated by Stauffer and Wright (1984) for conditions in the field at the time of failure.

TABLE 4.5. EFFECTIVE STRESS SHEAR STRENGTH PARAMETER FROM TRIAXIAL TESTS

Number of Cycles	Cohesion, $\bar{c}$ (psf)	Friction Angle, $\bar{\phi}$ (degrees)
6	242	20
10	253	19
30	213	23

CHAPTER 5. COMPARISON OF SHEAR STRENGTH PARAMETERS

INTRODUCTION

The purpose of this study was to assess the effects of wetting and drying on the effective stress shear strength parameters of compacted "Beaumont" clay. Both triaxial and direct shear tests were performed on specimens subjected to comparable numbers of cycles of wetting and drying. A comparison of the effective stress shear strength parameters and their statistical significance is presented in this chapter for the two types of shear tests. Effective stress shear strength parameters from the shear tests in the current study are also compared to values measured previously for compacted specimens not subjected to wetting and drying and to values back-calculated from actual slope failures.

COMPARISON OF TRIAXIAL AND DIRECT SHEAR TESTS ON SPECIMENS SUBJECTED TO WETTING AND DRYING

Effective stress shear strength parameters c and ø from direct shear tests performed on specimens subjected to one, three, nine and thirty cycles of wetting and drying are summarized in Table 5.1. No significant relationship is apparent between shear strength parameter, c or ϕ, and the number of cycles of wetting and drying. Standard deviations for c and ø obtained for regression analyses are summarized in Table 5.2. The computed probability that the cohesion intercept is not equal to zero is also presented in Table 5.2. The similarity between the magnitudes of the standard deviations (in Table 5.2) and the magnitudes of the cohesion intercepts (in Table 5.1), as well as the relatively low probabilities that c is not equal to zero, indicate that the cohesion might actually be close to zero and could be zero. Methods used to perform the statistical analyses are presented in Appendix C.

Effective stress shear strength parameters c and ø obtained from consolidated-undrained triaxial shear tests performed on compacted specimens subjected to six, ten and thirty cycles of wetting and drying are summarized in Table 5.3. A significant cohesion intercept is exhibited for the triaxial test results shown in this table. The cohesion values were calculated from the intercept (d) of the modified Mohr-Coulomb diagrams. The standard deviation for the values of d and the

TABLE 5.1. EFFECTIVE STRESS SHEAR STRENGTH PARAMETERS DETERMINED FROM DIRECT SHEAR TESTS

Number of Wet-Dry Cycles	Cohesion, $\bar{c}$ (psf)	Friction Angle, $\bar{\phi}$ (degrees)
1	29	23
3	77	26
9	33	25
30	0 (-130)*	27 (33)*

*Values in parentheses from linear regression; other values based on an assumed cohesion value of zero.

TABLE 5.2. STANDARD DEVIATIONS FOR $\bar{c}$ AND $\bar{\phi}$ AND PROBABILITIES THAT $\bar{c} \neq$ ZERO, DIRECT SHEAR TEST DATA

Number of Wet-Dry Cycles	Standard Deviation for $\bar{c}$, psf	Standard Deviation for $\bar{\phi}$, degrees	Probability That $\bar{c} \neq 0$, percent
1	32	2.1	60
3	64	4.3	62
9	38	2.4	56
30	59	0.05	83

TABLE 5.3. TRIAXIAL TEST SHEAR STRENGTH RESULTS

Number of Wet-Dry Cycles	Effective Cohesion, psf	$\bar{\phi}$, degrees
6	242	20
10	253	19
30	213	23

probability that the values of $\bar{d}$ (and also $\bar{c}$) obtained from the linear regression analyses were not zero are presented in Table 5.4. The computed standard deviations for $\bar{d}$ have been converted to equivalent "standard deviations in $\bar{c}$," assuming that the standard deviation of $\tan\psi$ is zero, and are also presented in Table 5.4. The high probabilities that the intercept is not equal to zero, along with the relationship between the magnitudes of the cohesion intercepts and the magnitudes of the equivalent standard deviations, indicate that the cohesion is significantly greater than zero. Methods used for the statistical analyses are presented in Appendix C.

Cohesion values measured in the direct shear test series were consistently smaller than values measured in the triaxial shear tests. In previous laboratory tests, effective stress shear strength parameters were measured for compacted specimens of Beaumont clay which were not subjected to wetting and drying using both triaxial and direct shear devices (Gourlay and Wright, 1984; Green and Wright, 1986). The effective stress shear strength parameters measured using both devices were essentially identical. Thus, the differences in the effective-stress shear-strength parameters measured in the triaxial and direct shear tests on specimens subjected to wetting and drying do not appear to be caused by differences in test apparatus (triaxial vs. direct shear).

The lower cohesion values measured in the direct shear tests are believed to be due to more severe effects of wetting and drying in the specimens of direct shear size. Three significant factors are believed to be responsible for the greater effects of wetting and drying: (1) specimens tested in the direct shear device were confined during wetting and drying by a chamber with a diameter slightly larger than the original specimen, which allowed the specimens to expand laterally; while triaxial specimens were confined such that they could not expand laterally; (2) the 2.5-inch diameter direct shear specimens experienced greater physical degradation due to a greater exposed surface area during wetting and drying; (3) specimens of direct-shear-size possessed a shorter drainage distance through which moisture had to travel during wetting; a 24-hour wetting period was probably sufficient for complete swell to occur in the specimens of direct shear size while it was not adequate for complete swell to occur in specimens of triaxial size.

COMPARISON OF AS-COMPACTED STRENGTH WITH STRENGTHS AFTER WETTING AND DRYING

Effective stress shear strength parameters for specimens of compacted "Beaumont" clay which were not subjected to wetting and drying were measured by Gourlay and Wright (1984) in

TABLE 5.4. STANDARD DEVIATIONS FOR $\bar{c}$ AND $\bar{d}$ AND PROBABILITIES THAT $\bar{c} \neq$ ZERO, TRIAXIAL TEST DATA

Number of Wet-Dry Cycles	Standard Deviation for $\bar{d}$, spf	Equivalent Standard Deviation for $\bar{c}$, psf	Probability that $\bar{c} \neq 0$, percent
6	91	32	97.3
10	138	49	95.6
30	161	53	93.8

consolidated-undrained triaxial shear tests, and by Green and Wright (1986) in direct shear tests. Gourlay and Wright report a cohesion value of 270 psf and a friction angle of 20.0 degrees from triaxial tests; Green and Wright report a cohesion value of 258 psf and a friction angle of 21.1 degrees from direct shear tests. Strengths measured in the current direct shear tests on specimens which were subjected to wetting and drying were significantly lower than those strengths estimated for direct shear specimens not subjected to wetting and drying. In contrast no significant difference was observed in the case of triaxial specimens depending on whether wetting and drying occurred.

COMPARISON OF STRENGTHS AFTER WETTING AND DRYING WITH FIELD VALUES

Stauffer and Wright (1984) calculated a cohesion value of 7 psf and a friction angle of 19 degrees for the field conditions at Scott Street and I.H. 610. They report a range in cohesion values of 7 psf to 14 psf from calculations for all recorded slides in "Beaumont" clay embankments, indicating that the cohesion of the clay at the time of failure is significantly small.

Slope stability computations were performed with the computer program UTEXAS (Roecker and Wright, 1984) for the embankment located at Scott Street and I.H. 610, assuming a slope angle of 21 degrees, a slope height of 19 feet and a unit weight of 120 pcf. The foundation was assumed to have the same properties as the slope material. Factors of safety were calculated using the effective stress shear strength parameters measured with the direct shear device and assuming no pore water pressures. The purpose of these calculations was to compare the factors of safety with the value of unity in the field at the time of failure. Factors of safety are summarized in Table 5.5. Values are shown for specimens which were subjected to wetting and drying as well as those for specimens which were not subjected to wetting and drying by Green and Wright (1986). All factors of safety presented are greater than unity. However, the values calculated based on direct shear tests performed on specimens after wetting and drying are substantially lower; thus, wetting and drying does appear to contribute to at least some significant loss of strength in the field.

Pore water pressure conditions necessary for the factor of safety to be unity were also estimated based on the strengths from the direct shear test for the embankment at the intersection of Scott Street and I.H. 610. The pore water pressures were characterized and expressed by the dimensionless parameter, r_u, defined by Bishop and Morgenstern (1960) as:

TABLE 5.5. COMPUTED FACTORS OF SAFETY FOR EMBANKMENT AT SCOTT STREET AND I.H. G10 BASED ON EFFECTIVE STRESS SHEAR STRENGTH PARAMETERS FROM DIRECT SHEAR TESTS

Number of Wet-Dry Cycles	Cohesion, $\bar{c}$ (psf)	Friction Angle, $\bar{\phi}$ (degrees)	Factor of Safety
0 *	258 *	21.1 *	2.28
1	29.0	23.0	1.39
3	77.0	25.5	1.81
9	33.0	24.5	1.51
30	0.0	27.0	1.34

*Values by Green and Wreight (1986) for compacted specimens not subjected to wetting and drying

$$r_u = u/\gamma z \tag{5.1}$$

where u is the pore water pressure at a point, γ is the total unit weight of soil above the point, and z is the vertical distance from the ground surface to the point.

Values of r_u based on the direct shear tests after wetting and drying and those direct shear tests performed by Green and Wright (1986) without wetting and drying are presented in Table 5.6. The practical upper limit for r_u is 0.5; the value of 0.83 corresponding to specimens with no wetting and drying indicates that the strengths measured for specimens without wetting and drying are probably unrepresentative of strengths in the field. Values based on the direct shear tests on specimens after wetting and drying are within the range of reasonable values, although they appear to be somewhat high for the slopes studied. Actual pore pressure values were probably lower in slopes which failed, while the effective cohesion values may also have been lower than those measured with the direct shear tests after wetting and drying.

TABLE 5.6. DIRECT SHEAR TEST EFFECTIVE STRESS SHEAR STRENGTH PARAMETERS AND PORE PRESSURE COEFFICIENT r_u

Number of Wet-Dry Cycles	Effective Cohesion, psf	$\overline{\phi}$, degrees	r_u
0 *	258	21.1	0.83
1	29.0	23.0	0.29
3	77.0	25.5	0.51
9	33.0	24.5	0.35
30	0.0	27.0	0.22

* Values for the peak shear strength found by Green and Wright (1986) for compacted specimens not subjected to wetting and drying

CHAPTER 6. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Many embankments constructed of highly plastic clays in the Eastern half of the state of Texas have failed a number of years after construction. The shear strength, expressed in terms of appropriate effective stress strength parameters, appears to be much lower in the field than what is measured in the laboratory. Specifically, the cohesion intercept of the effective stress shear strength envelope appears to deteriorate with time. In order to determine if the apparent loss in shear strength with time is due to the effects of repeated wetting and drying the series of laboratory tests described in this report was performed. Effective stress shear strength parameters, c and ϕ, were determined for specimens subjected to various numbers of cycles of wetting and drying using both direct shear and triaxial shear apparatus. Shear strength parameters determined from the direct shear tests indicate a substantial loss in shear strength due to wetting and drying. The strength loss was reflected primarily in the effective stress cohesion intercept with only small variations in the angle of internal friction. No significant effect of the number of cycles of wetting and drying was noted for numbers of cycles ranging from 1 to 30; most changes appeared to occur on the first cycle. Triaxial shear tests showed no significant strength losses due to wetting and drying; however, it is likely that the length of time permitted for each wetting and drying cycle (24 hours) was not sufficient to allow moisture to fully penetrate or escape from the specimens during each wetting and drying cycle, respectively.

Factors of safety were computed using the shear strength parameters determined in this study for an embankment constructed of Beaumont clay which had failed (Scott St. and I.H. 610 in Houston). Although all factors of safety exceeded unity, while they would have been expected to be unity or less for an embankment which failed, the factors of safety based on shear strengths determined in direct shear tests and specimens subjected to wetting and drying were much lower than those for specimens not subjected to wetting and drying. The fact that the factors of safety were not unity may be due to other, yet unexplained factors; however, there was also considerable uncertainty in the cohesion intercept measured in the direct shear tests on specimens subjected to wetting and drying. The small cohesion intercepts which were measured may very well have been zero, which would have further reduced the factor of safety. It is certainly reasonable to expect that wetting and drying might reduce the cohesion intercept to zero based on the available test data.

It is apparent that wetting and drying can reduce the long-term shear strength of highly plastic clays like Beaumont clay. The reduction in strength is reflected by a decrease in the cohesion intercept with only minor changes in the friction angle. Further laboratory testing is recommended to establish if all the observed losses in strength in the field are due to wetting and drying and to establish suitable procedures for measuring the effects of wetting and drying using generally more suitable triaxial shear rather than direct shear tests. However, until such additional testing can be completed it is evident that the effective stress cohesion intercept for highly plastic clay like the Beaumont clay should not be relied upon for design when such soils will be subjected to repeated wetting and drying.

APPENDICES

APPENDIX A

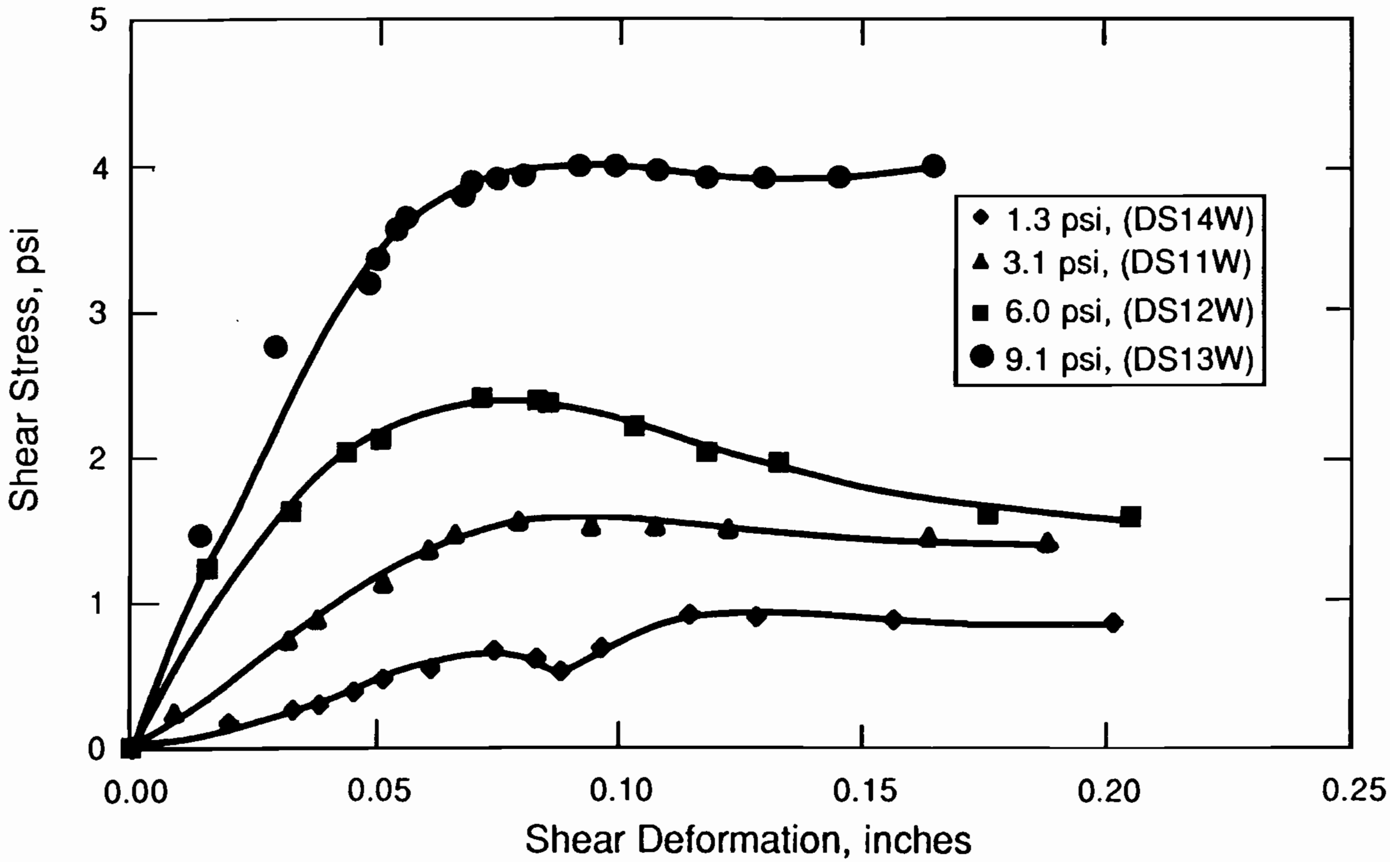

Figure A.1 Shear Stress vs. Shear Deformation for Direct Shear Tests DS11W-DS14W,
One Cycle of Wetting and Drying

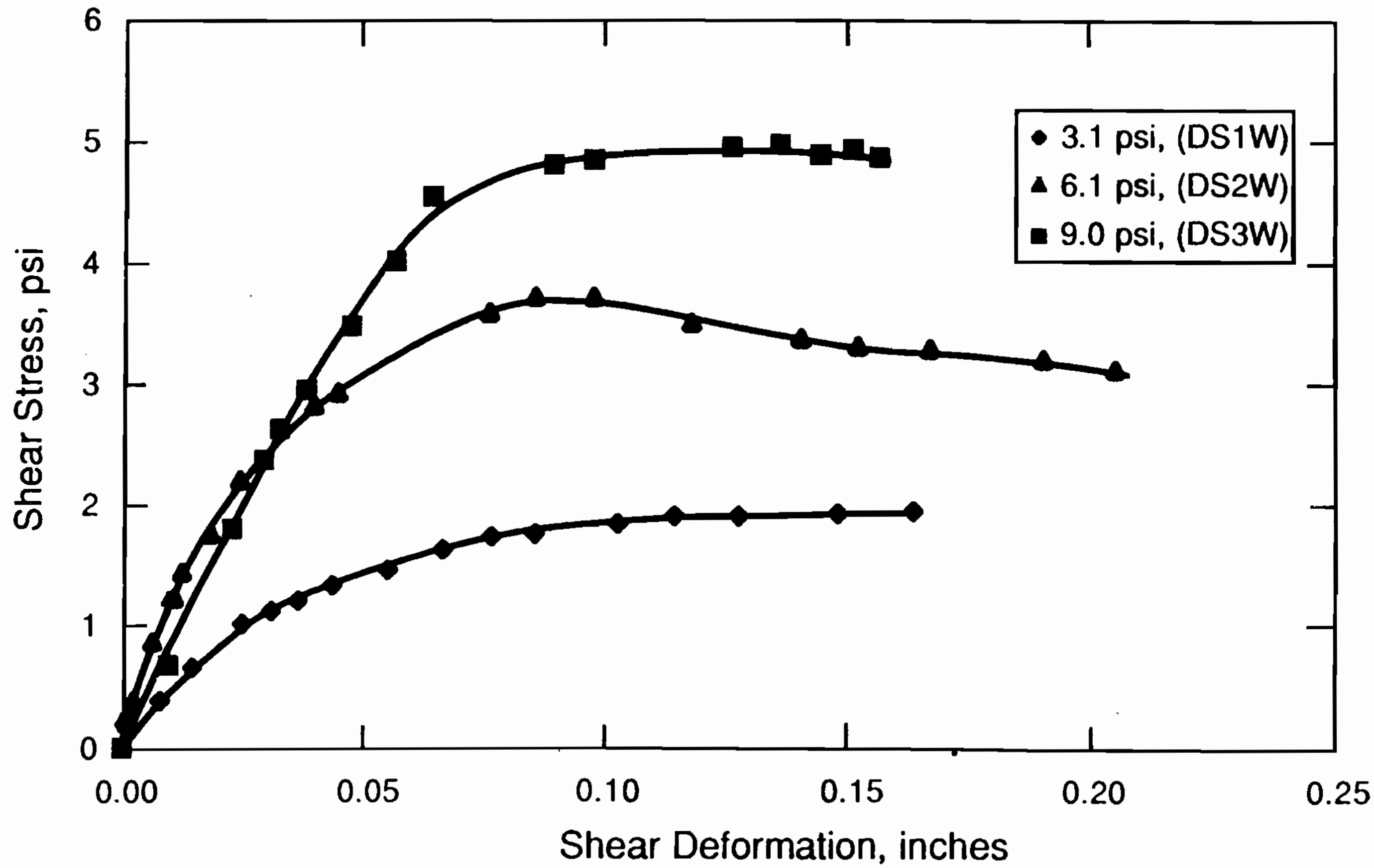

Figure A.2 Shear Stress vs. Shear Deformation for Direct Shear Tests DS1W-DS3W, Three Cycles of Wetting and Drying

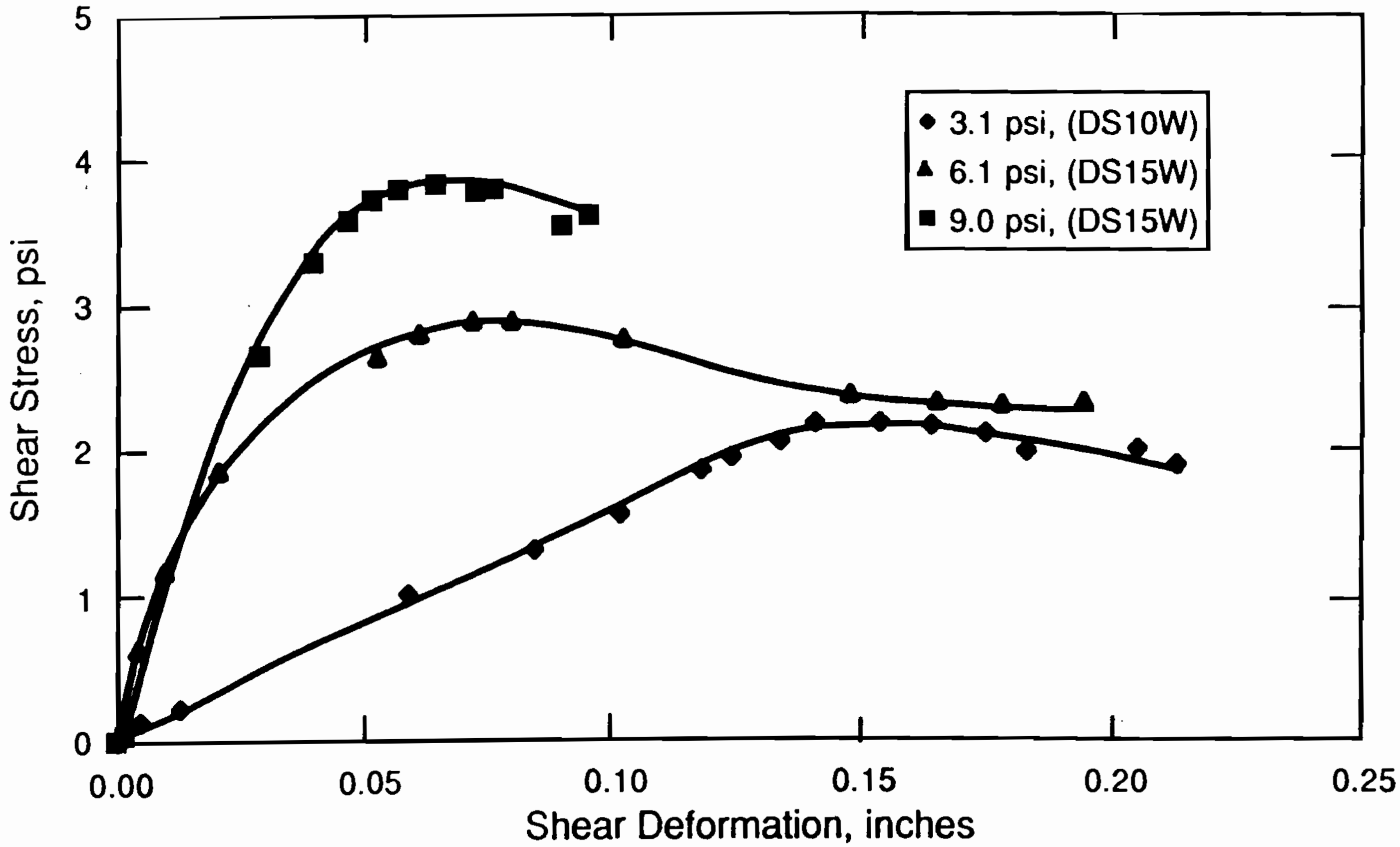

Figure A.3 Shear Stress vs. Shear Deformation for Direct Shear Tests DS10W and DS15W-DS16W, Three Cycles of Wetting and Drying

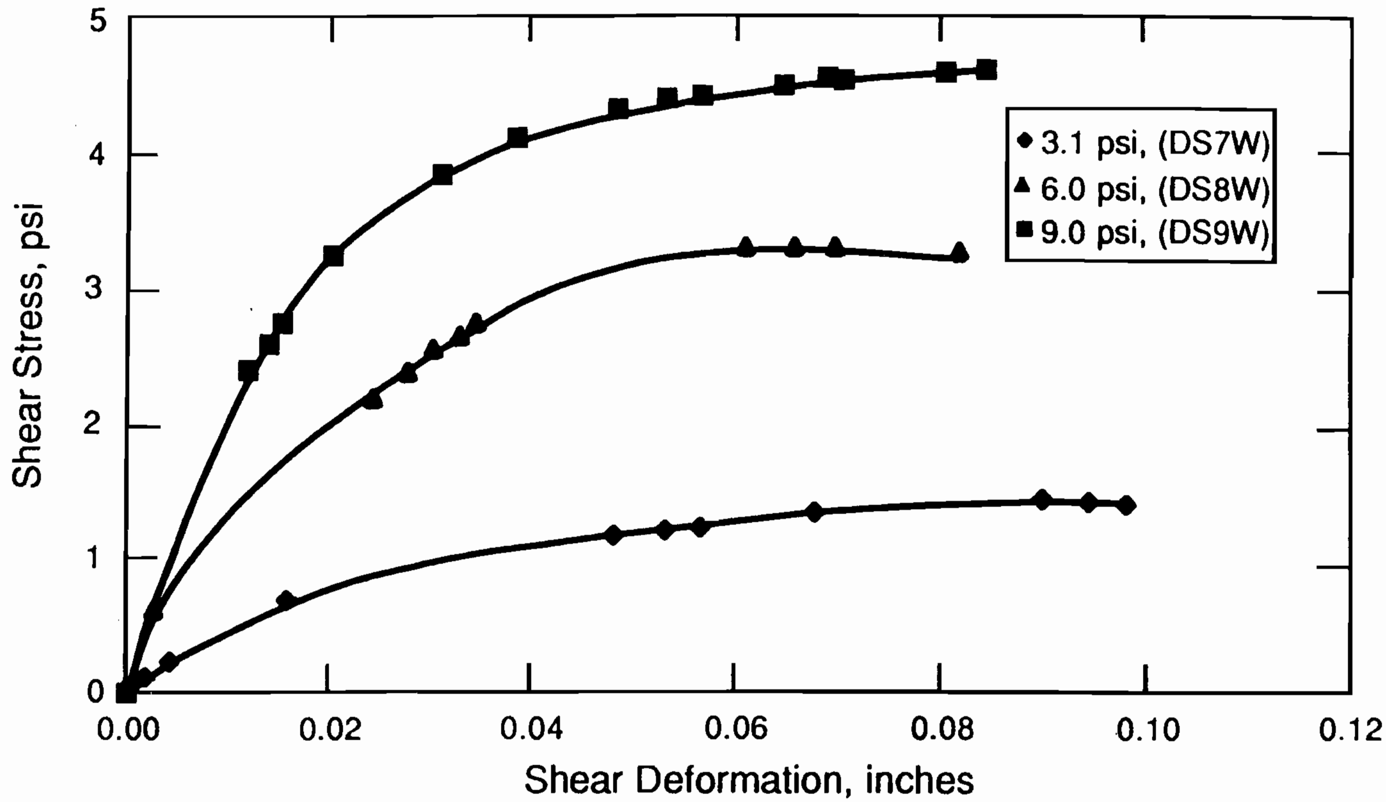

Figure A.4 Shear Stress vs. Shear Deformation for Direct Shear Tests DS7W-DS9W, Nine Cycles of Wetting and Drying

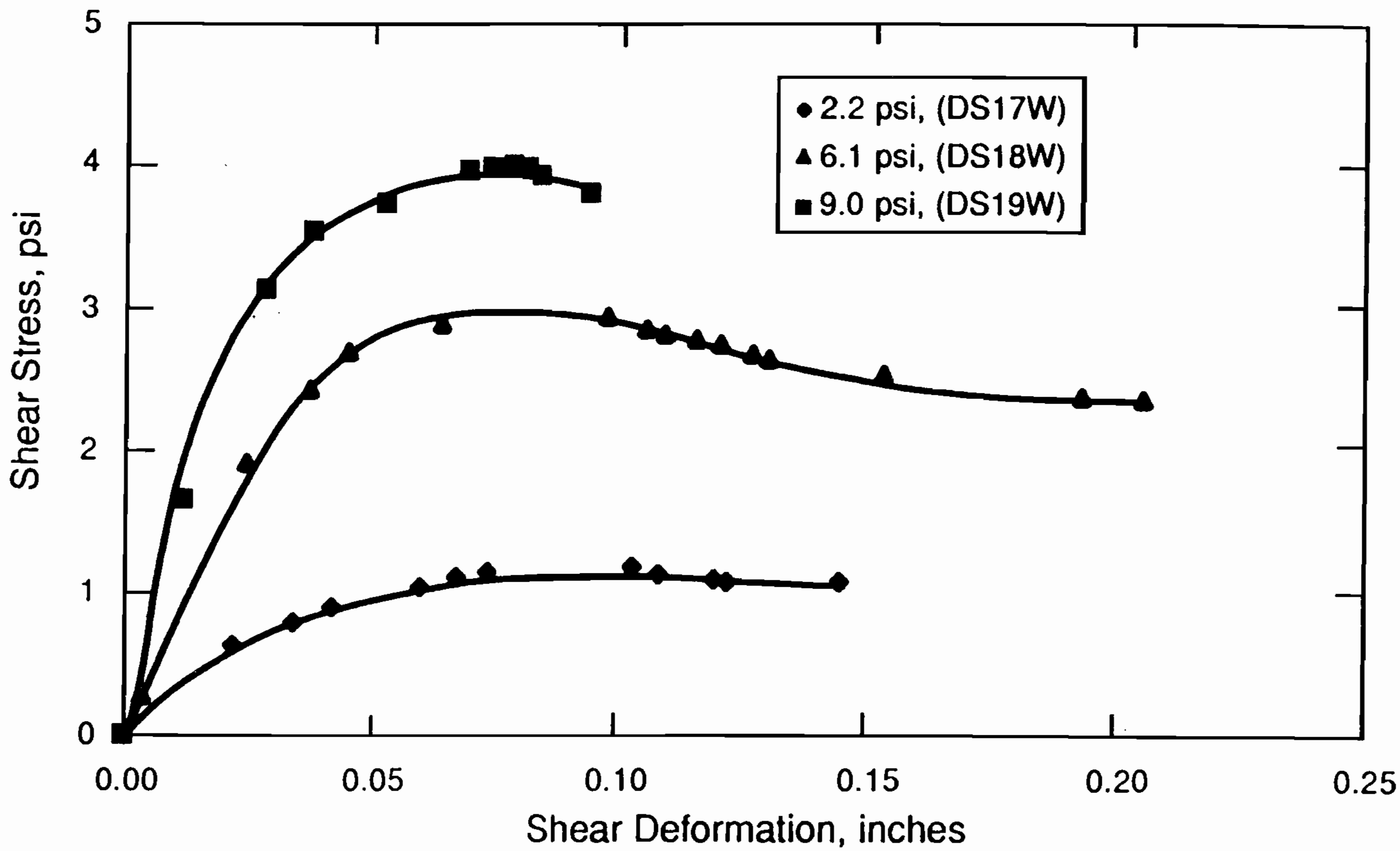

Figure A.5 Shear Stress vs. Shear Deformation for Direct Shear Tests DS17W-DS19W, Nine Cycles of Wetting and Drying

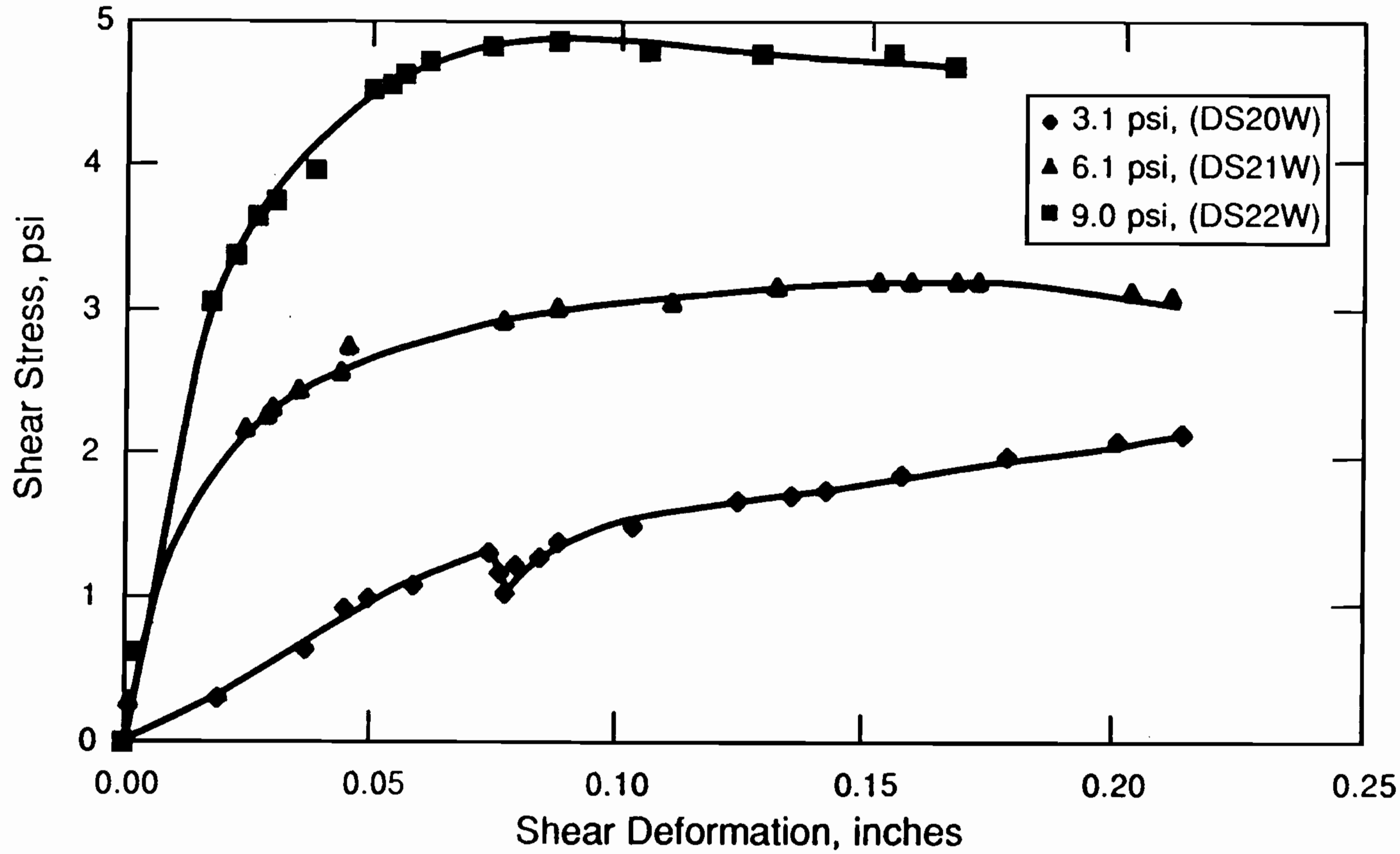

Figure A.6 Shear Stress vs. Shear Deformation for Direct Shear Test DS20W-DS22W, Thirty Cycles of Wetting and Drying

APPENDIX B

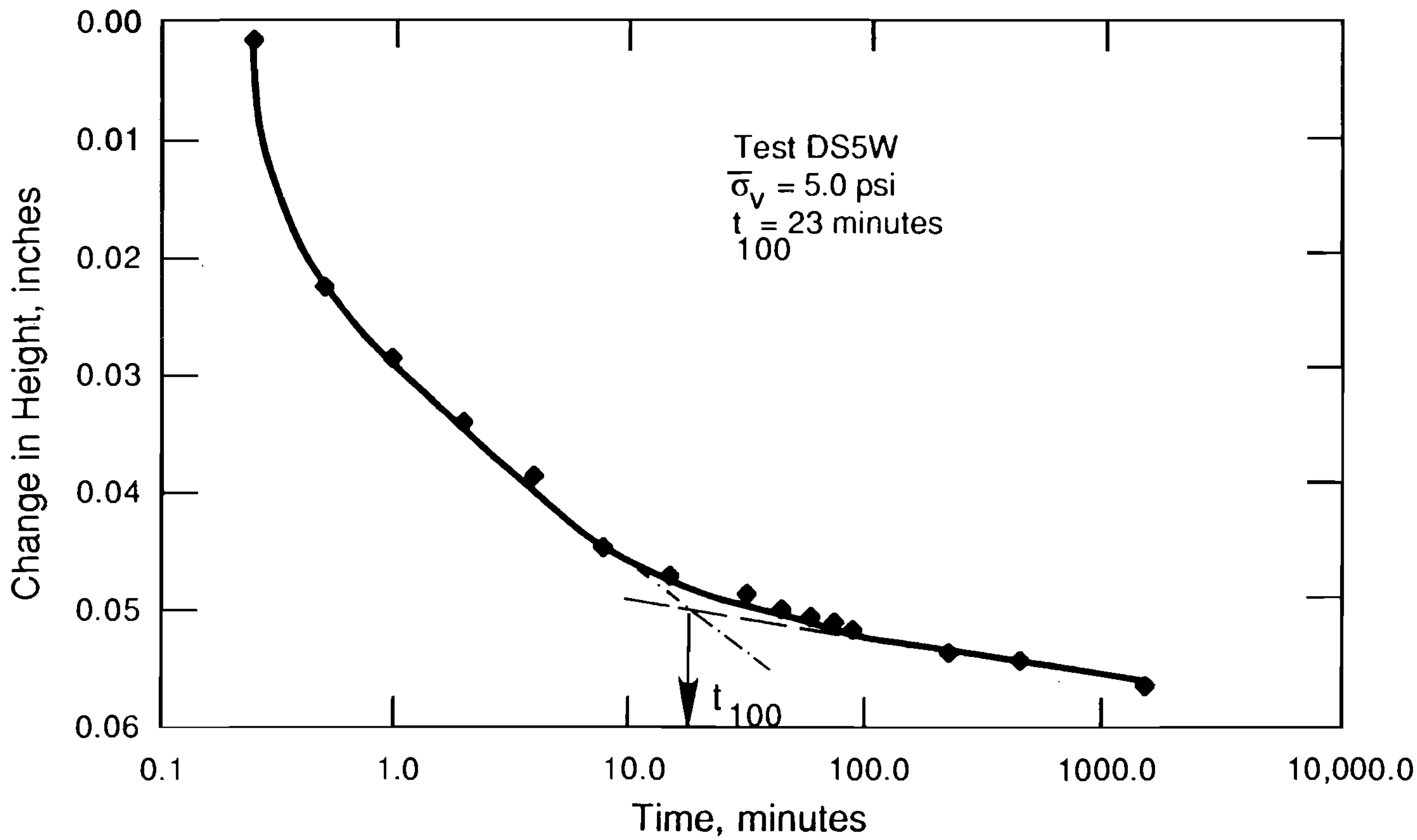

Figure B.1 Change in Height of Specimen in Test DS5W During Consolidation vs. Logarithm of Time, One Cycle of Wetting and Drying

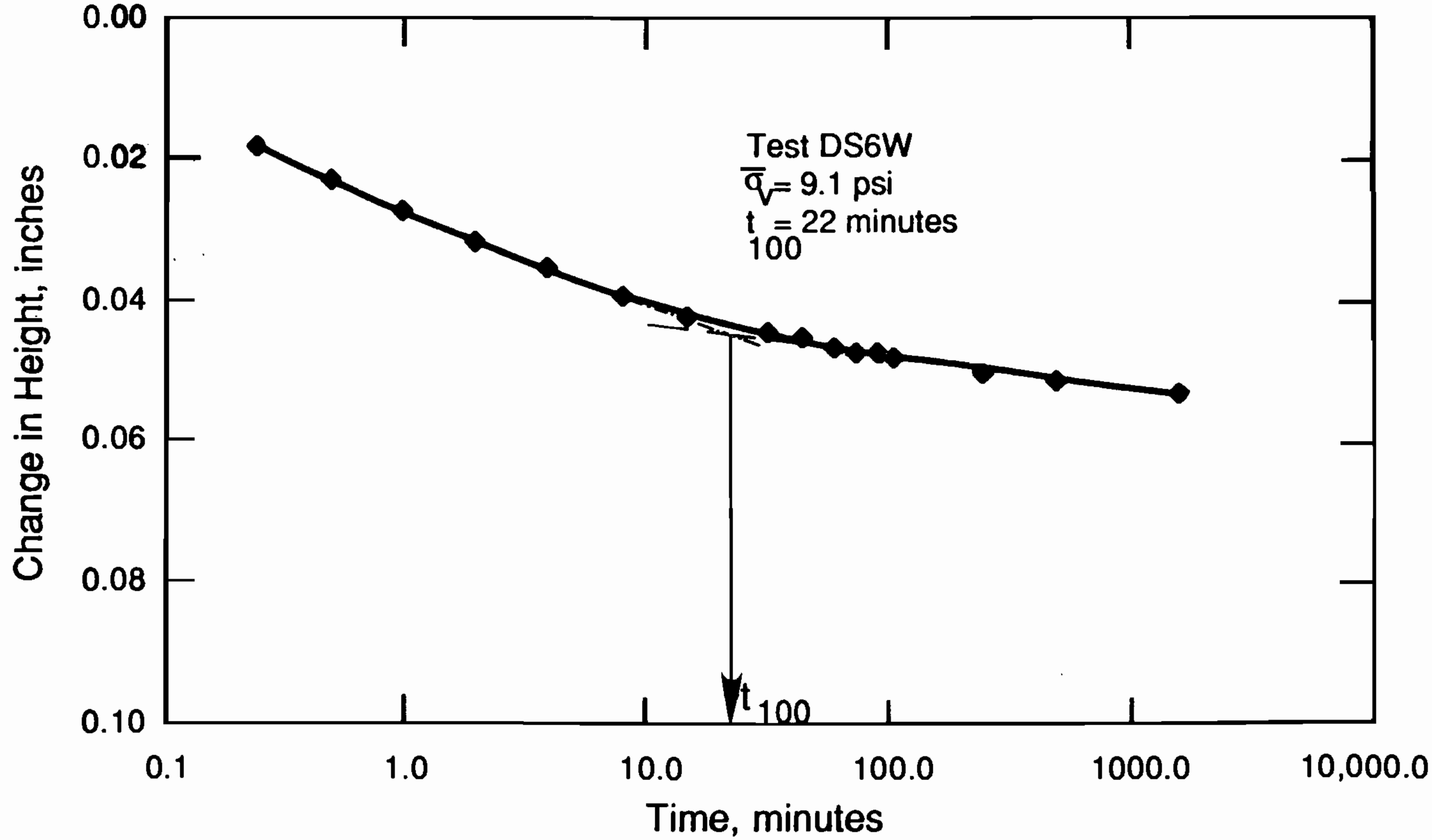

Figure B.2 Change in Height of Specimen in Test DS6W During Consolidation vs. Logarithm of Time, One Cycle of Wetting and Drying

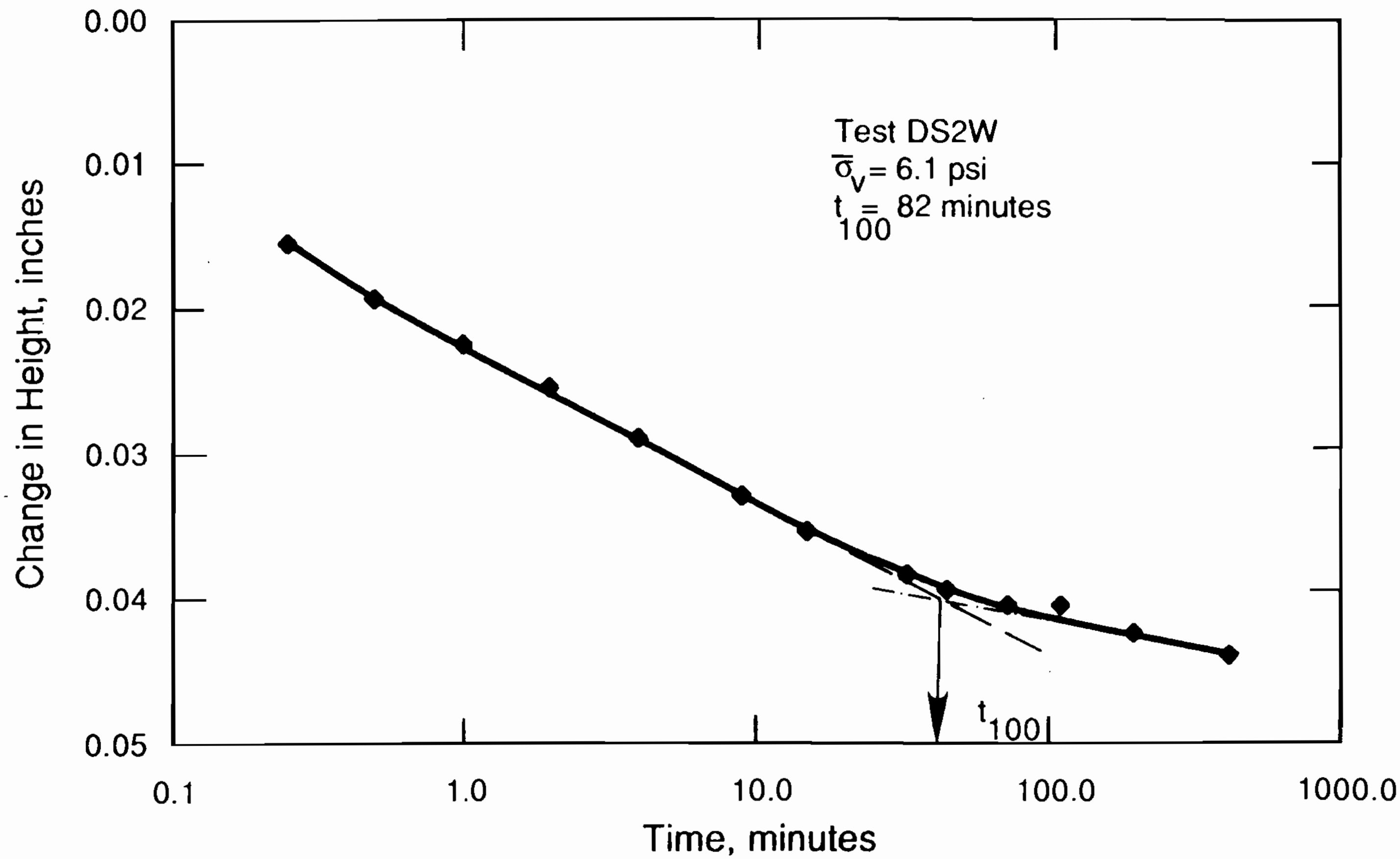

Figure B.3 Change in Height of Specimen in Test DS2W During Consolidation vs. Logarithm of Time, Three Cycles of Wetting and Drying

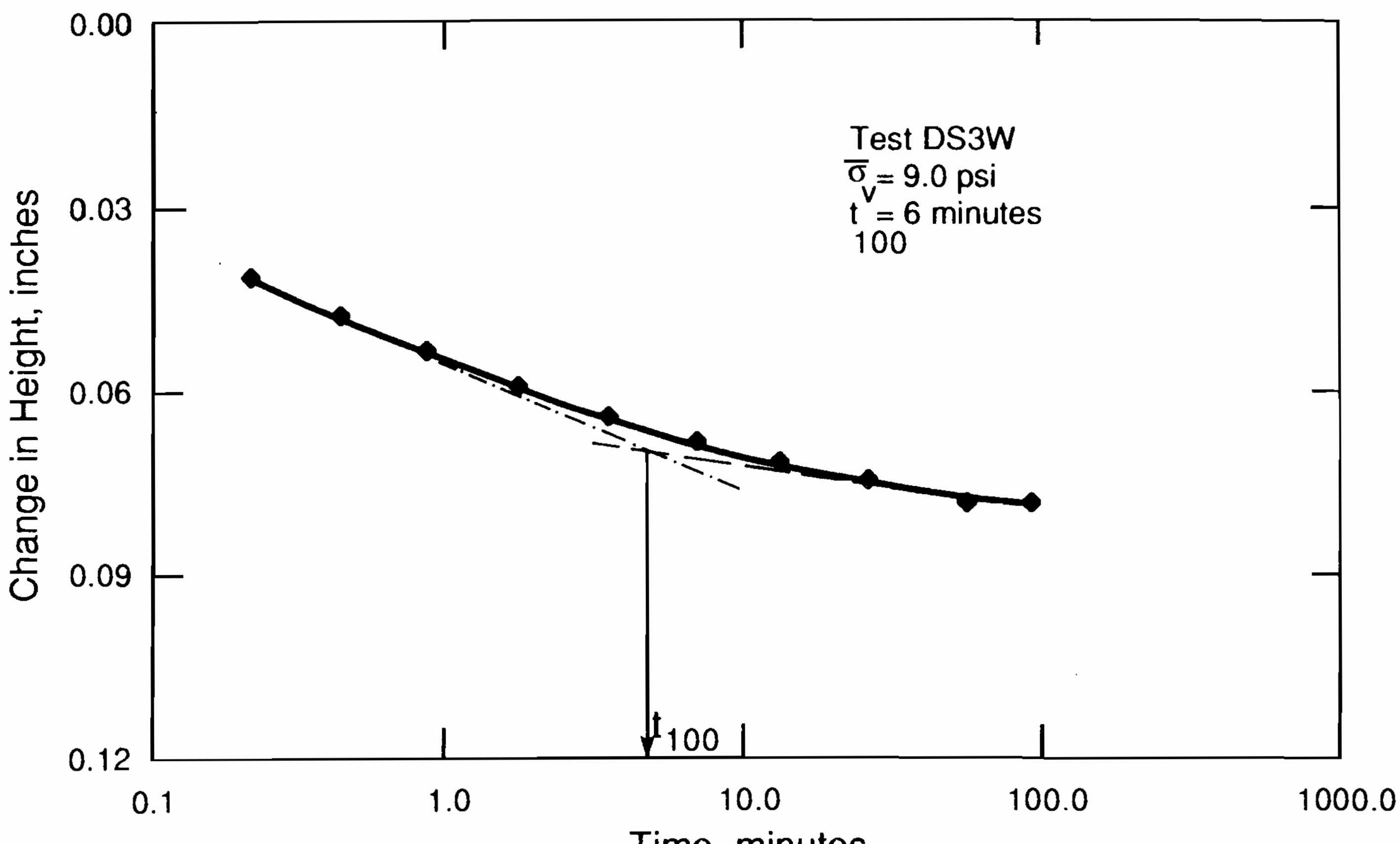

Figure B.4 Change in Height of Specimen in Test DS3W During Consolidation vs. Logarithm of Time, Three Cycles of Wetting and Drying

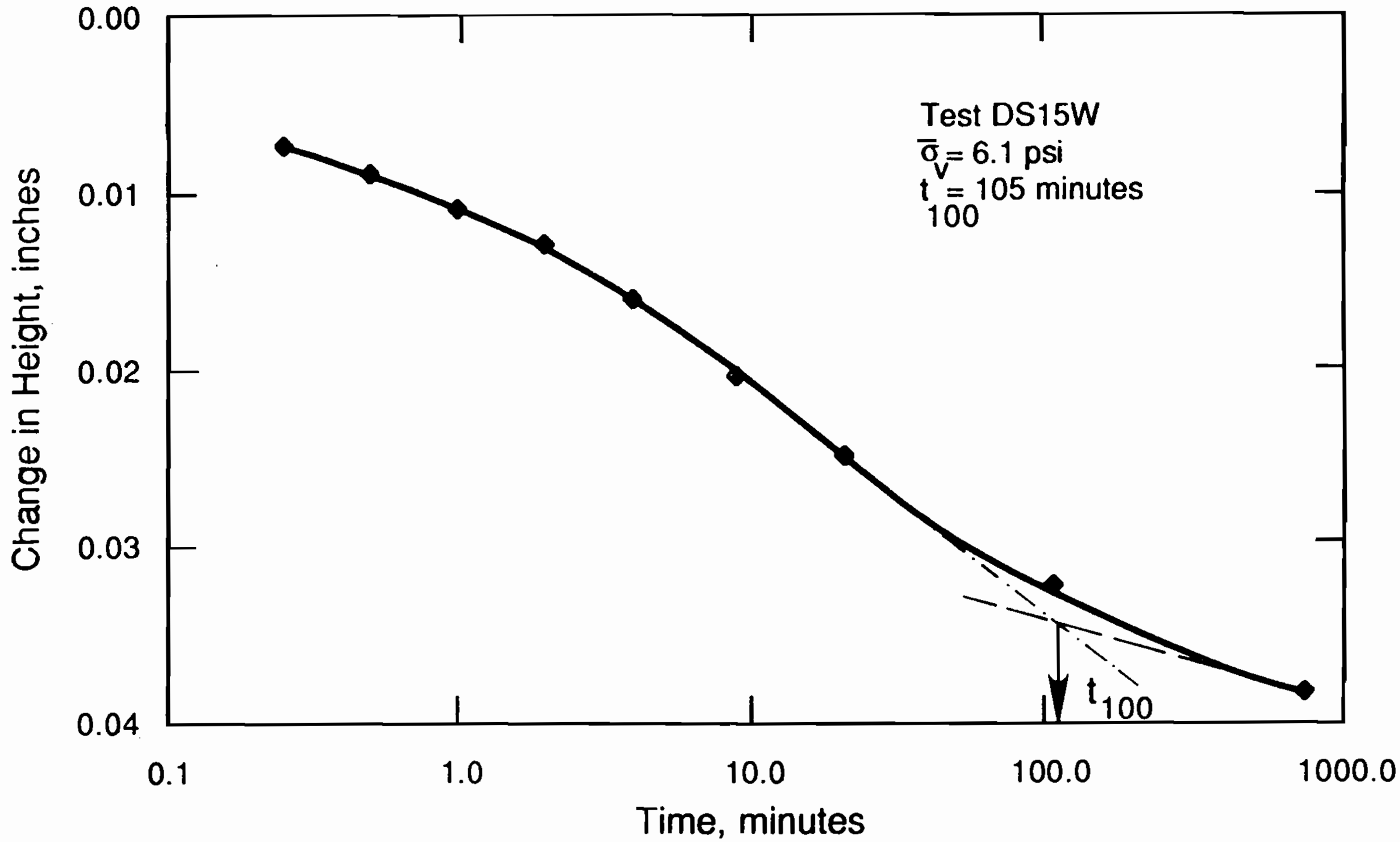

Figure B.5 Change in Height of Specimen in Test DS15W During Consolidation vs. Logarithm of Time, Three Cycles of Wetting and Drying

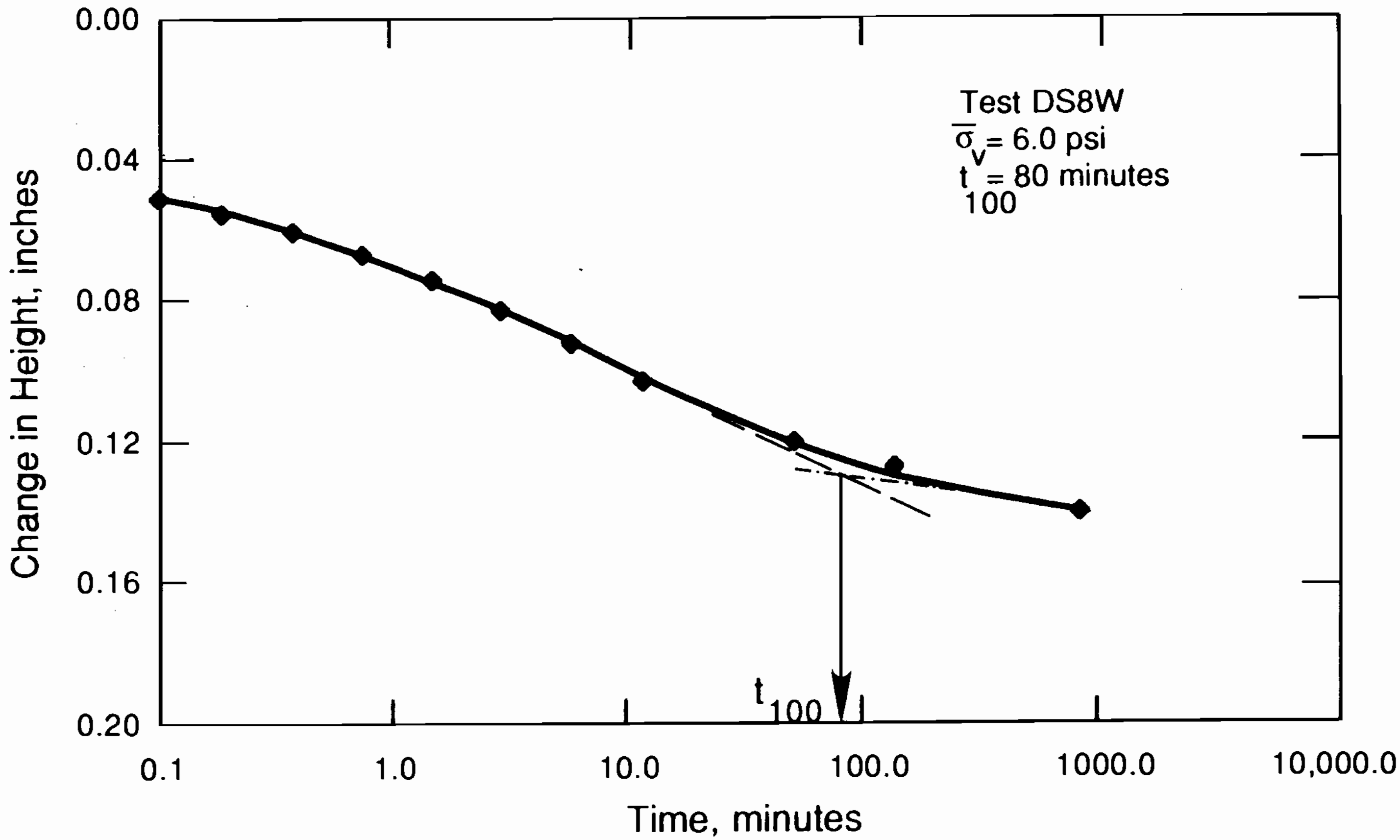

Figure B.6 Change in Height of Specimen in Test DS8W During Consolidation vs. Logarithm of Time, Nine Cycles of Wetting and Drying

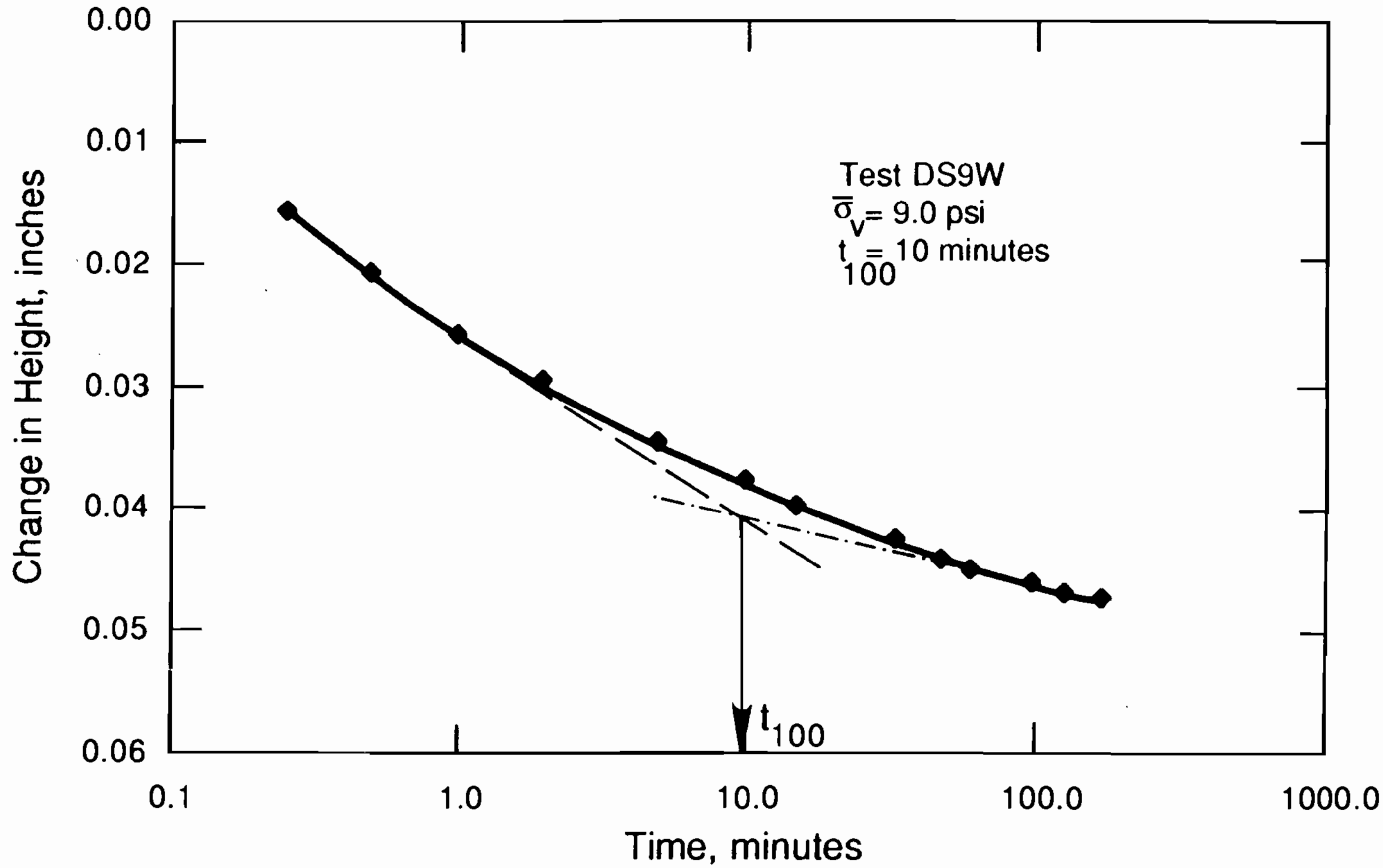

Figure B.7 Change in Height of Specimen in Test DS9W During Consolidation vs. Logarithm of Time, Nine Cycles of Wetting and Drying

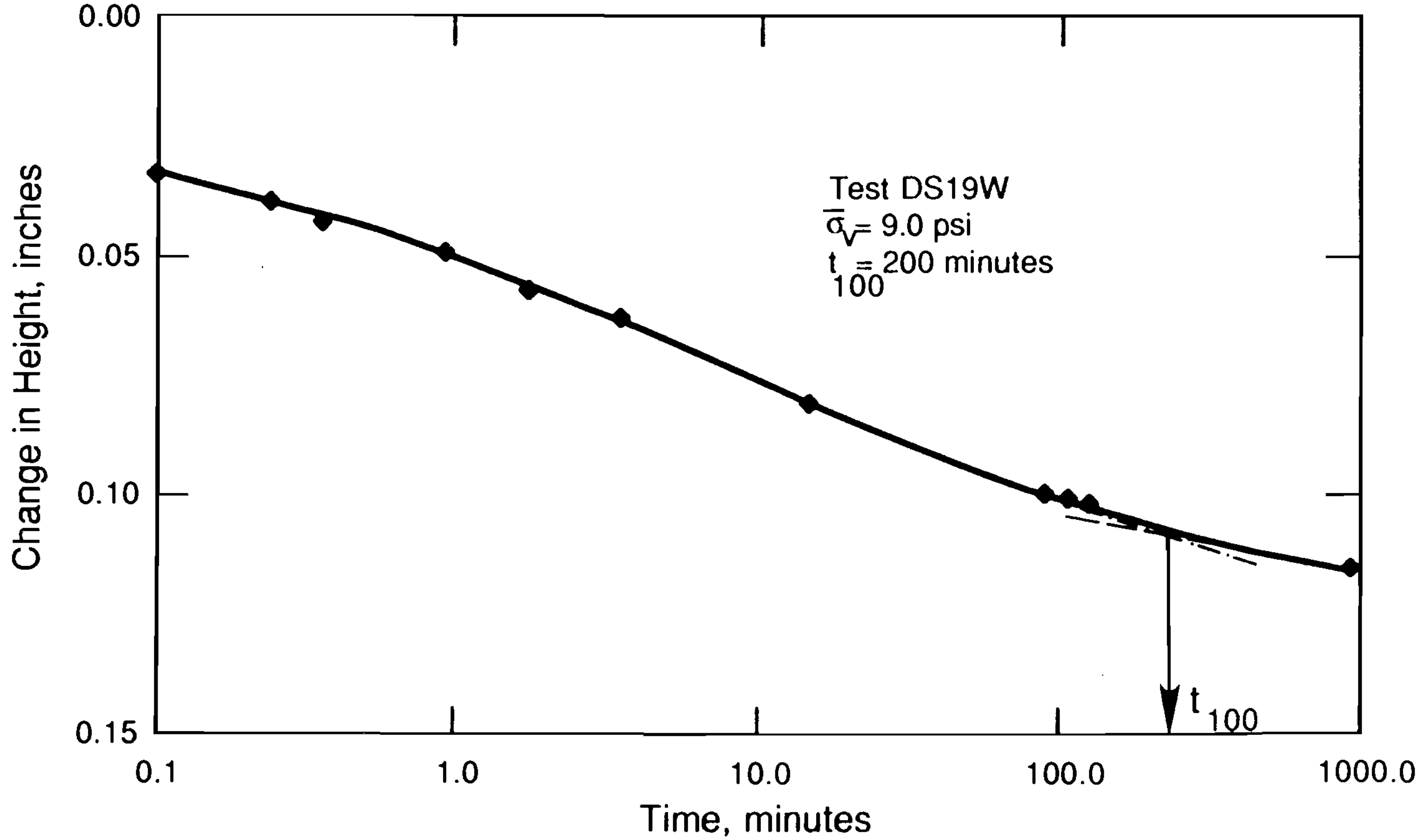

Figure B.8 Change in Height of Specimen in Test DS19W During Consolidation vs. Logarithm of Time, Nine Cycles of Wetting and Drying

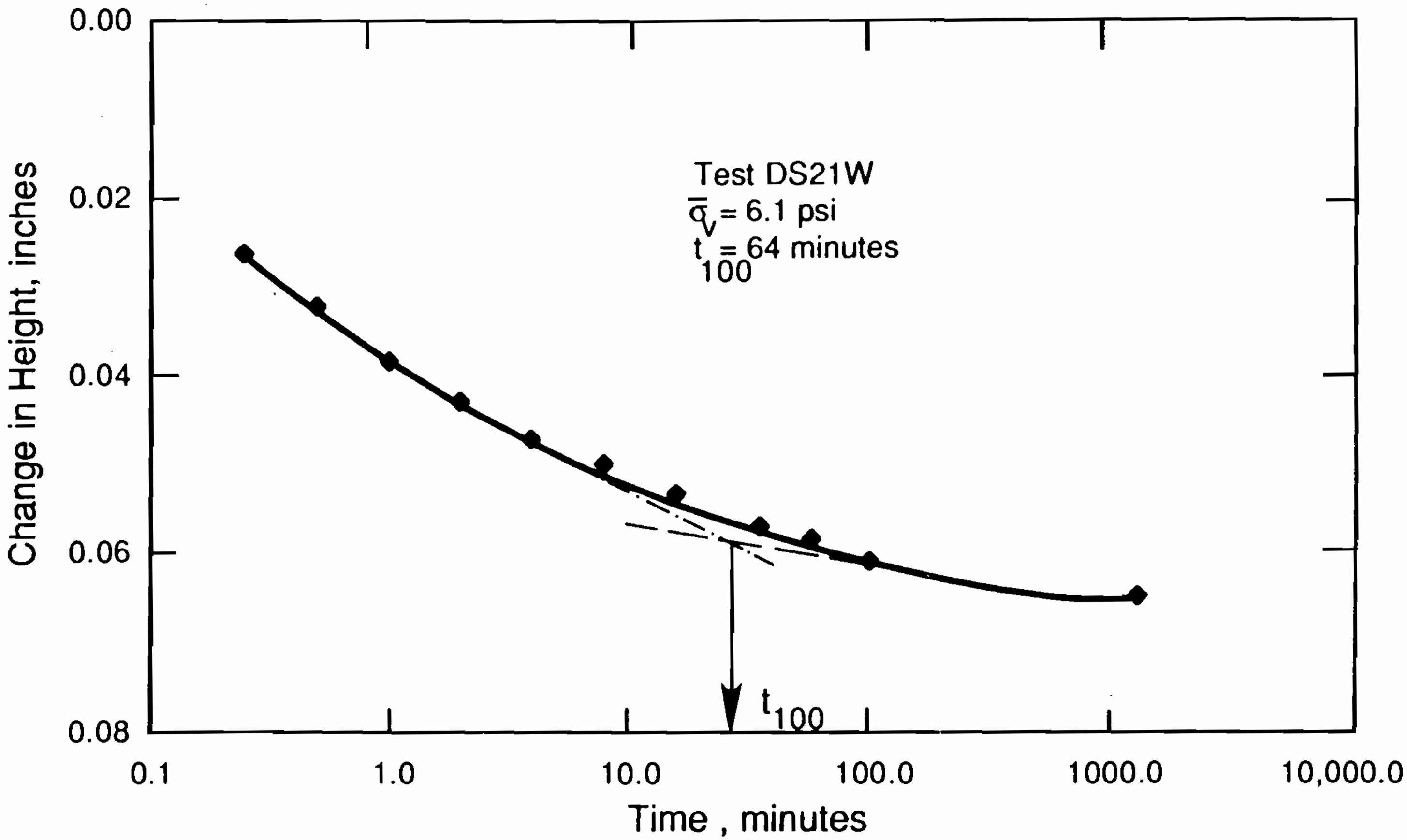

Figure B.9 Change in Height of Specimen in Test DS21W During Consolidation vs. Logarithm of Time, Thirty Cycles of Wetting and Drying

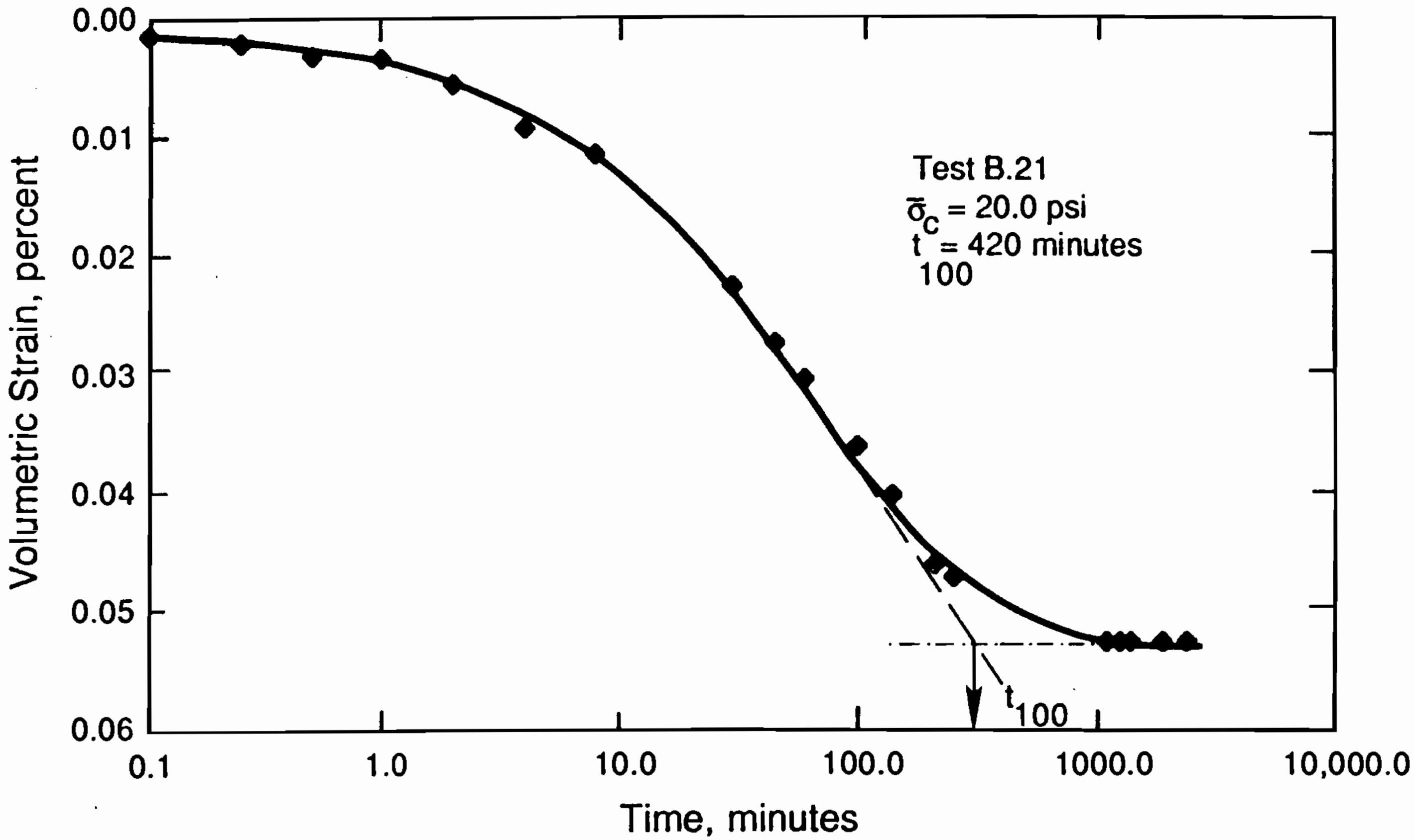

Figure B.10 Volumetric Strain vs. Logarithm of Time During Final Consolidation of Test B.21, Six Cycles of Wetting and Drying

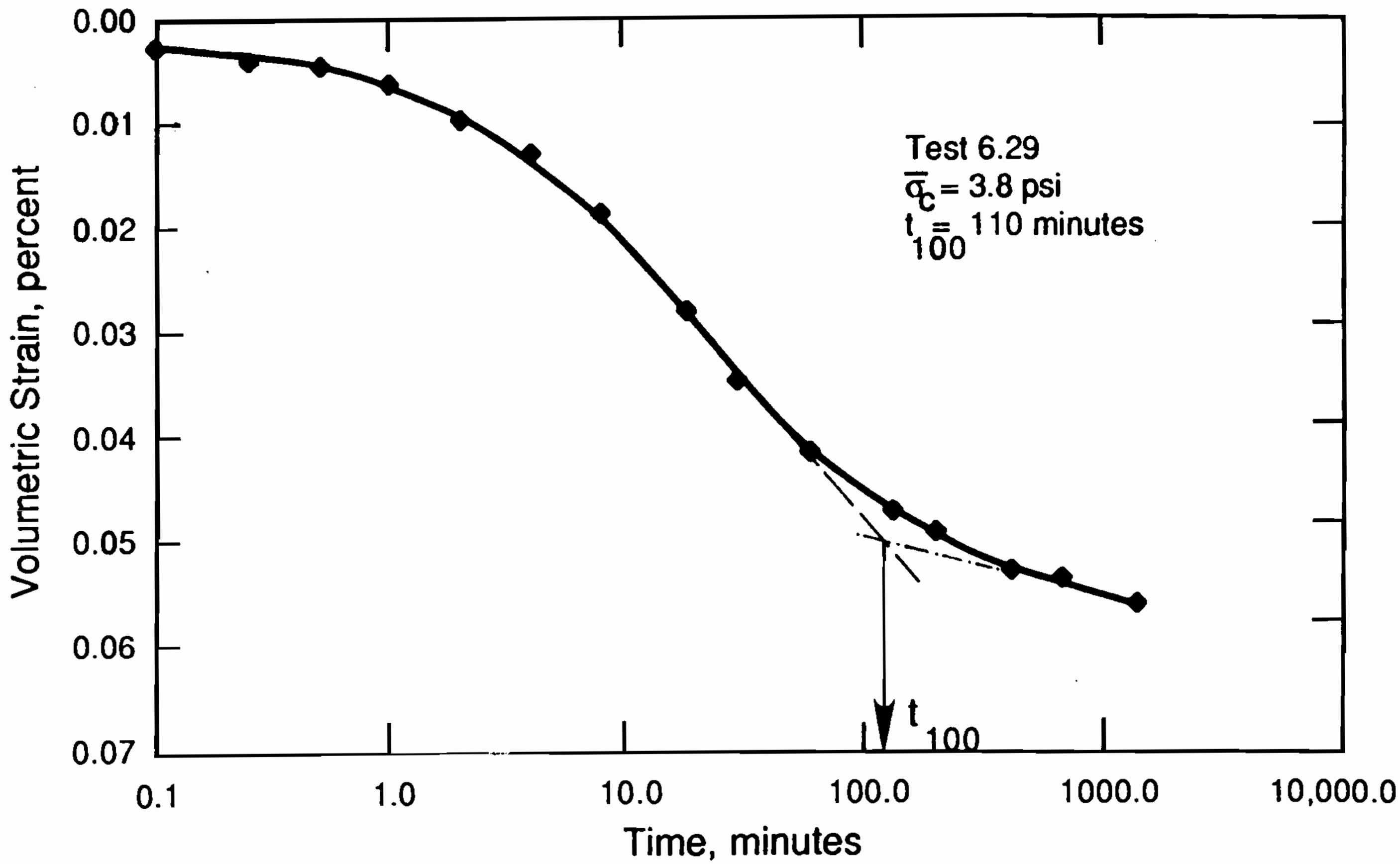

Figure B.11 Volumetric Strain vs. Logarithm of Time During Final Consolidation of Test 6.29, Ten Cycles of Wetting and Drying

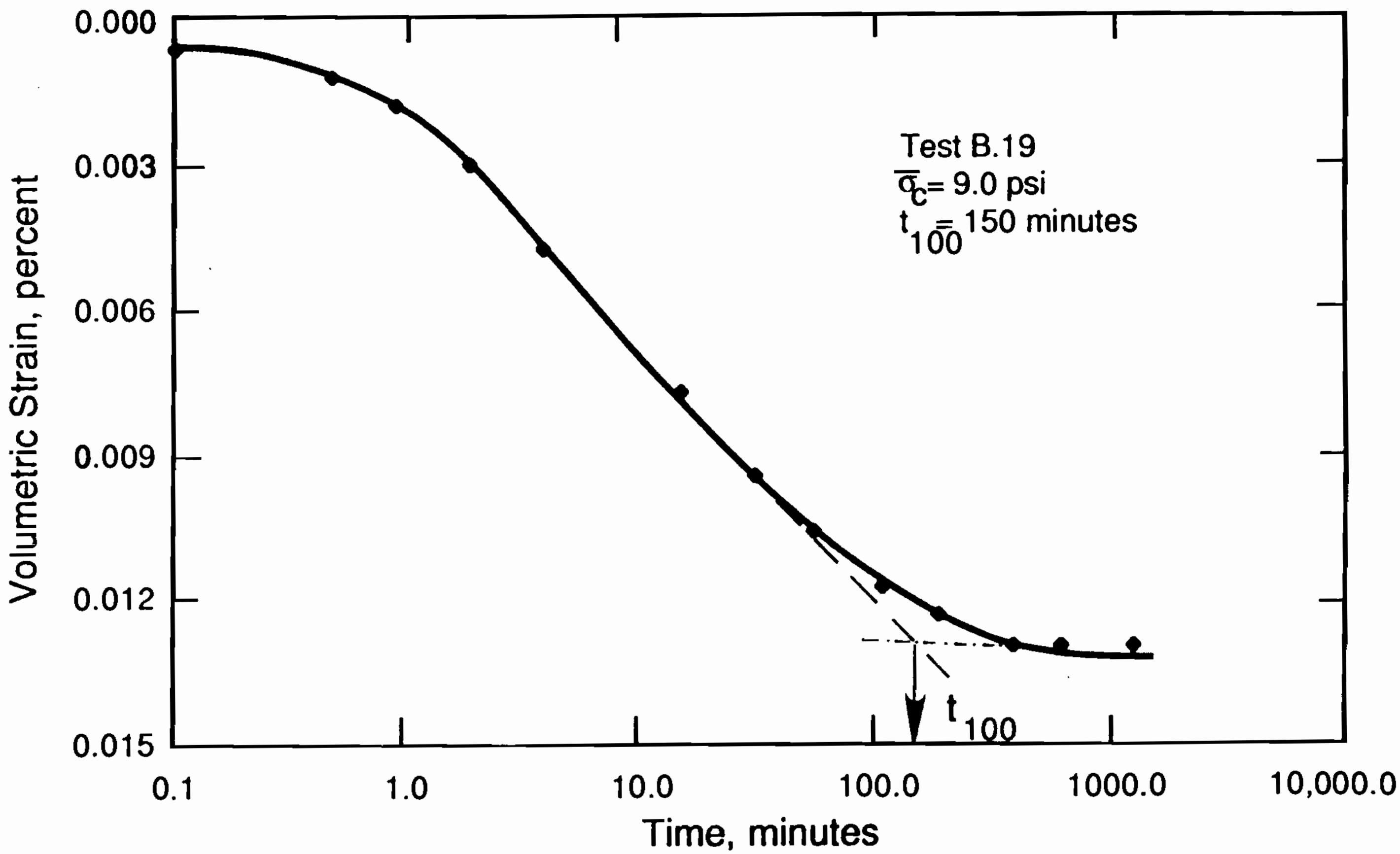

Figure B.12 Volumetric Strain vs. Logarithm of Time During Final Consolidation of Test B.19, Ten Cycles of Wetting and Drying

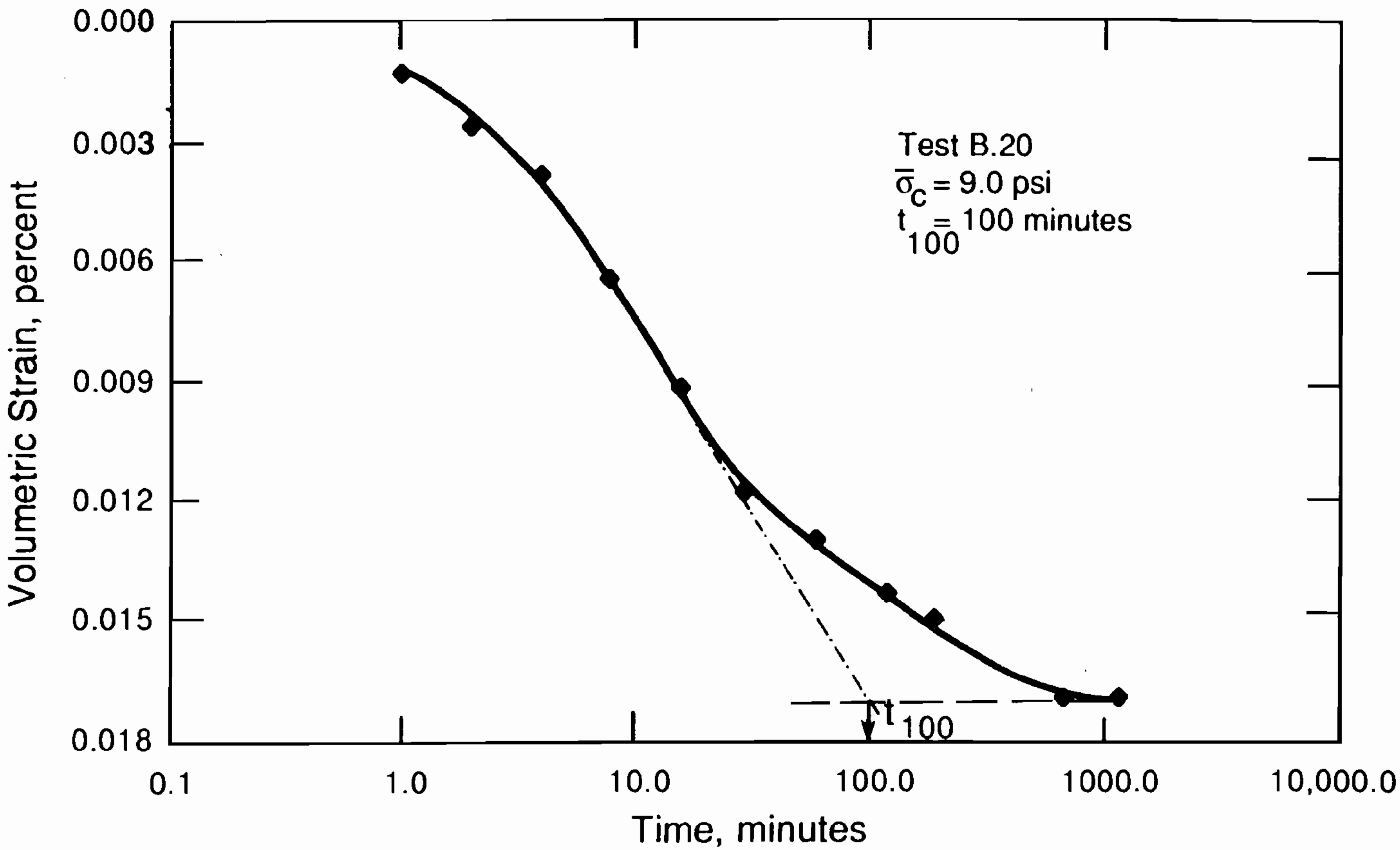

Figure B.13 Volumetric Strain vs. Logarithm of Time During Final Consolidation of Test B.20, Ten Cycles of Wetting and Drying

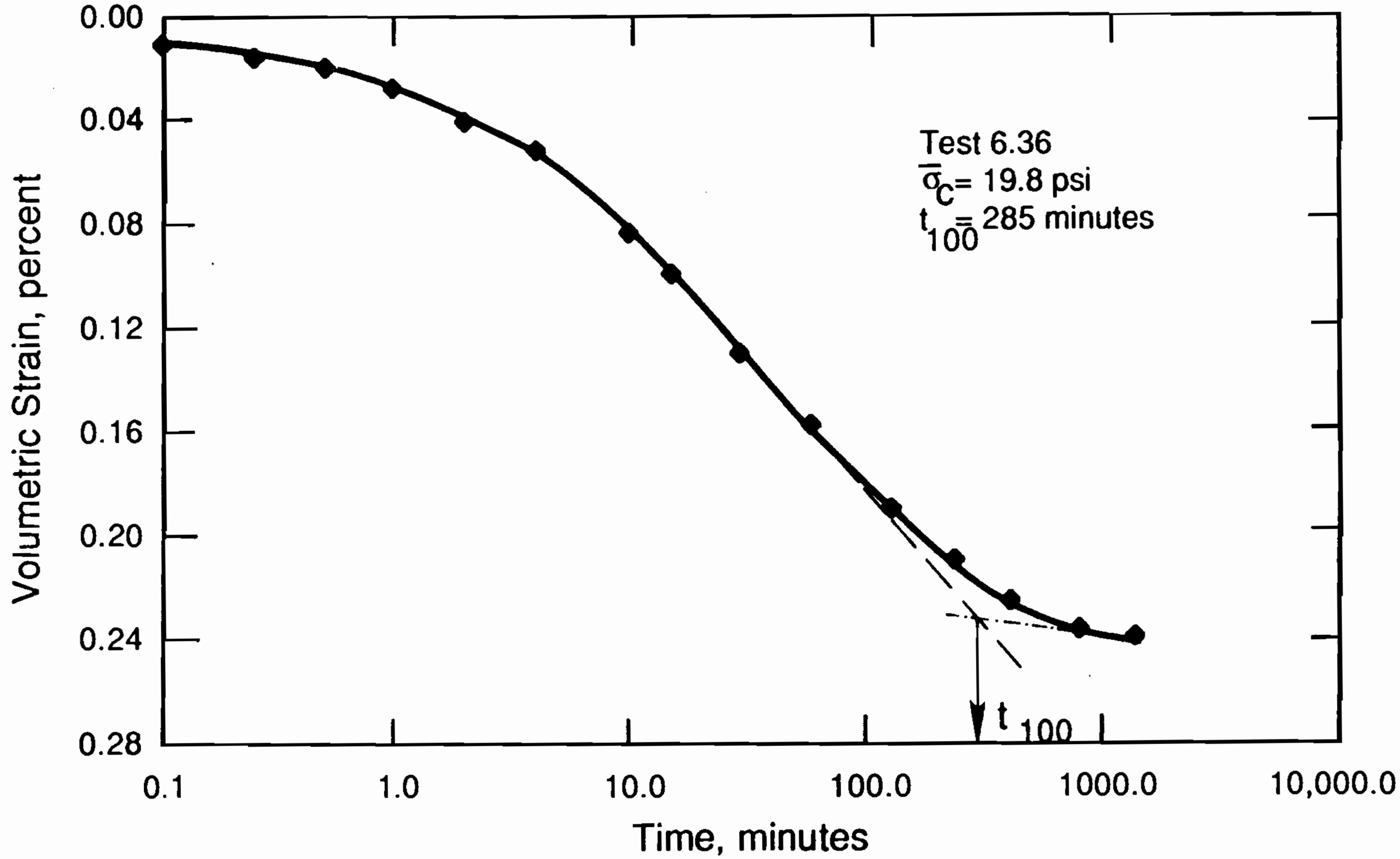

Figure B.14 Volumetric Strain vs. Logarithm of Time During Final Consolidation of Test 6.36, Thirty Cycles of Wetting and Drying

APPENDIX C

Statistical Analyses on Linear Regression Estimates

Standard Deviations

Standard deviations were calculated for both the intercept , c ,and the slope, tanø, from the linear regression performed on the direct shear test data, and the intercept ,d, and an equivalent deviation in c from the linear regression performed on the triaxial test data, assuming a standard deviation of zero for the slope, tanψ. The standard deviations s_c and s_d of the intercepts from the direct shear and triaxial shear test data respectively, were calculated using the following equation:

$$s_c \text{ or } s_d = s_y \sqrt{1/n + x^2/\Sigma(x - x)^2}$$

where

$$s_y = \sqrt{\Sigma\varepsilon_i^2/(n - 2)}$$

where n is the number of data points, x is the mean value of x, and

$$\varepsilon_i^2 = (y - y_i)^2$$

where

$$x = \sigma_v \text{ , } y = \tau \text{ and } y_i = c + \sigma_v \tan\phi \text{ for Direct Shear Test Data}$$

and

$$x = \sigma_3 \text{ , } y = (\sigma_1 - \sigma_3) \text{ and } y_i = d + \sigma_3 \tan\psi \quad \text{for Triaxial Shear Test Data}$$

The "deviation in c" referred to in the text for the triaxial shear test data was calculated assuming that the standard deviation of the slope, tanψ, was equal to zero, and thus the standard deviation of ø was also assumed to be zero. s_c could therefore be calculated as:

$$s_c = s_d[(1\text{-}\sin\phi)/2\cos\phi]$$

Probability That The Intercept ≠ Zero

The probability that the intercept, c, from the direct shear test data was not equal to zero was assessed using a t distribution, where

$$t = c/s_c$$

Values of α representing the probability that the intercept is equal to zero were obtained from t distribution tables. The probability that the intercept, d, from the triaxial shear test data is equal to zero was equated to the probability that the cohesion is equal to zero.

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