COLORADO STATE UNIVERSITY

COMPARISON OF DIRECT SHEAR AND TRIAXIAL TESTS FOR MEASUREMENT OF SHEAR STRENGTH OF SAND

Rahman

THESIS

COMPARISON OF DIRECT SHEAR AND TRIAXIAL TESTS FOR MEASUREMENT OF SHEAR STRENGTH OF SAND

Submitted by

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WE HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER OUR SUPERVISION BY JAMSHED RAHMAN ENTITLED COMPARISON OF DIRECT SHEAR AND TRIAXIAL TESTS FOR MEASUREMENT OF SHEAR STRENGTH OF SAND BE ACCEPTED AS FULFILLING IN PART REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE

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ABSTRACT OF THESIS

COMPARISON OF DIRECT SHEAR AND TRIAXIAL TESTS FOR MEASUREMENT OF SHEAR STRENGTH OF SAND

To ascertain the shear strength parameters of soils for engineering purposes is fundamental to soil mechanics and basic for designing earth-bearing and earth-retaining structures. Direct shear and triaxial tests are the most popular laboratory methods to determine these parameters. The direct shear test is used widely because it is simple and quick. The test has several disadvantages, however. The non-uniform stress-strain behavior, the rotation of principal planes during the test, and the imposition of the failure plane are chief among them. The triaxial test was designed as a possible alternative that eliminates some of these disadvantages.

Direct shear test results are always comparable to those of the triaxial test; the difference usually is negligible from a practical point of view. Researchers have tried to unfold the intricacies involved in the direct shear test especially the complicated stress-strain behavior that a soil experiences during this test. Data, however, are lacking that determine the difference and establish a correlation between the results of the two tests. This study compares the two tests for measurement of shear strength parameters of sand.

Triaxial and direct shear tests were performed on silica sand under the same density and normal stress conditions. Five sets of triaxial tests and 20 direct shear tests each were performed using four different makes of direct shear machines. The results of the direct shear tests were compared with those of the triaxial tests considering the latter as benchmarks. The possible effect of the structural features of the direct shear equipment on results was briefly studied.

The results showed that the shear strengths from direct shear tests are higher than those from the triaxial tests. All four direct shear machines gave cohesion values different from each other and higher than the benchmark value. The Soiltest and Wykeham Farrance machines gave almost the same friction angle that was higher than the benchmark value by 4 degrees. The friction angle value from the ELE machine was higher by 2.7 degrees while those from Clockhouse machine were lower by 4.5 degrees as compared to the benchmark value.

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LIST OF SYMBOLS

Symbol	Represents	Unit
τ	Shear stress	lb/in^2
τ_f	Shear stress at failure, shear strength	lb/in^2
ϕ	Angle of internal friction	degrees
ϕ'	Effective angle of internal friction	degrees
ϕ_m	Maximum friction angle	degrees
σ	Total normal stress	lb/in^2
σ'	Effective normal stress	lb/in^2
σ_f	Normal stress at failure	lb/in^2
c	Cohesion	lb/in^2
σ_1	Major principal stress	lb/in^2
σ_2	Intermediate principal stress	lb/in^2
σ_3	Minor principal stress	lb/in^2
$(\Delta\sigma_d)_f$	Deviator stress at failure	lb/in^2
ε_x	Strain in the x- direction	-
ε_y	Strain in the y-direction	-

Symbol	Represents	Unit
τ_{xy}	Shear stress	lb/in^2
γ_{xy}	Shear strain	-
$\Delta\sigma'_x$	Change in stress in the x-direction	lb/in^2
E'	Young's modulus	lb/in^2
μ'	Poisson's ratio	-
ψ'	Angle of dilation	degrees
K_o	Coefficient of stress at rest	-
e_o	Void ratio	-
C_u	Uniformity coefficient	-
x	Arithmetic mean	-
Σ	Summation of	-
n	Sample size	-
s	Standard deviation	-
s^2	Variance of a sample	-
M	Sample mean	-
γ_d	Dry density	lb/ft^3
γ_R	Relative density	-
D_{10}	Diameter at which 10% of the soil is finer	mm
q	y-coordinate of a stress point on a stress path, defined by $(\sigma_1-\sigma_3)/2$	lb/in^2
p	x-coordinate of a stress point on a stress path,	

Symbol	Represents	Unit
	defined by $(\sigma_1+\sigma_3)/2$	lb/in^2

CHAPTER 1
INTRODUCTION

1.1 General

The laboratory testing of soils to determine their strength and deformation properties is fundamental to soil mechanics. The analysis, design and construction of earth-founded and earth-retaining structures require an accurate measure of the soil strength parameters involved. These parameters are 'cohesion' denoted by 'c' for pure clayey (cohesive) soils and 'angle of internal friction' represented by 'ϕ' for sandy (granular) soils. Since soils in nature are most commonly in the form of mixtures of both cohesive and granular soils, both these strength parameters are usually determined for engineering purposes.

Shear strength parameters can be measured either in the field or in the laboratory. Of the various methods available, the direct shear and triaxial tests are the oldest and the most popular laboratory methods. The triaxial test is popular because of accuracy, precision, stress-strain behavior and ease in evaluation of the principal stresses. The direct shear test can be performed quickly and easily making it practical, economical and used widely by the engineering industry.

The direct shear test, has been criticized by researchers because the state of stress and strain in the soil during the direct shear test is non-uniform and the principal planes and stresses rotate during this test. Another common objection about the direct shear test is that the soil is subjected to a predetermined failure plane. The parameters generated do not have the same accuracy as obtained from the triaxial test where the failure plane is determined naturally during the test and is not imposed.

Regardless of these objections about the validity and accuracy of the direct shear test, the fact that the results obtained from it are comparable to those generated by the triaxial test, provides sufficient ground for the usefulness of the direct shear test from a practical and economic standpoint. Nevertheless, very little technical evidence is available from quantitative studies that satisfactorily answer the following questions:

- How consistent are the results of direct shear tests ?
- How do the results of direct shear tests compare with those generated by the triaxial test ?
- How do the results generated by different makes of direct shear machines compare to each other ?

1.2 Objectives of the Study

The aim of this study is to answer these questions. The study compares direct shear and triaxial compression tests performed on one type of soil (silica sand) remolded at approximately the same density subjected to similar normal stresses in a drained (dry) condition.

Colorado State University has, in the Fu Hua Chen Geotechnical Research Laboratory, four different makes of direct shear testing machines. These include name brands such as Soiltest, ELE, Wykeham Farrance and Clockhouse. Twenty direct shear tests were performed using each. Five sets of triaxial tests were also performed keeping the density and normal stresses the same. Statistical analysis was done and the results are presented in Chapter 5. A discussion of the precision comparison of different direct shear equipment used for this study is also included in Chapter 5. Detailed discussion of results is in Chapter 6; while Chapter 7 summarizes the conclusions drawn from this work.

1.3 Scope for Further Studies

The present study has been limited to shear strength testing on only one soil type remolded at approximately the same density and under the same normal loading. Suggested further studies are:

- testing the same type of soil under a variation of normal loading and density
- testing different types of soils under the same normal load and density,
- testing a variety of soils under a set of different normal loads and densities.

Quantitative studies performed along the above lines will not only assist in ascertaining the extent to which the results generated by the two methods agree but will also be of considerable help in drawing empirical correlations among the results of direct shear and triaxial testing.

CHAPTER 2

LITERATURE REVIEW

2.1 General

The shear strength of a soil is its maximum resistance to shearing stress that the soil mass can offer to resist failure and sliding along any plane. When this resistance is exceeded, failure occurs. Shear strength is usually assumed to be made up of :

(1) internal friction - the resistance due to friction and interlocking of soil particles

(2) cohesion - the resistance due to the forces resulting from the formation of ionic bonds that tend to hold the particles together as a solid mass.

Coarse-grained soils such as sands derive their shear strength almost entirely from intergranular friction; whereas cohesive soils, like clays, depend on the attraction between the clay minerals due to the presence of ionic bonds for exhibiting strength against shear failure.

The shear strength of soils is an important aspect of geotechnical engineering. The bearing capacity of shallow or deep foundations, slope stability analysis, retaining wall design, and indirectly, pavement design, are all affected by the shear strength of soils. Structures and slopes must be stable and secure against total collapse when subjected to maximum anticipated applied loads. Thus, 'limiting equilibrium' methods of analysis are conventionally used for their design, and these methods require determination of the ultimate or limiting shear resistance (shear strength) of soils.

2.2 Shear Strength Theory:

Mohr in 1900 hypothesized that materials fail when the shear stress on the failure plane at failure reaches some unique function of the normal stress on that plane, or

$$\tau_f = f(\sigma_f) \tag{2.1}$$

where τ is the shear stress and σ is the normal stress. Mohr's theory implies that a material fails through a critical combination of normal stress and shearing stress, and not through either maximum normal or shear stress alone (Holtz & Kovacs, 1981; Das, 1985). Though Mohr expressed his findings in the form of a functional relationship, Coulomb in 1776 presented his famous law governing the rupture failure of materials which can equally be applied to soils. Coulomb's law is expressed as

$$\tau = c + \sigma \tan \phi \tag{2.2}$$

where τ = shear strength

c = cohesion

σ = intergranular normal pressure

ϕ = angle of internal friction

$\tan \phi$ = coefficient of friction

The variables c and ϕ are called the strength parameters of a soil. The combination of Mohr's failure envelope and Coulomb's condition of rupture represents the failure criterion commonly used for most soils. The Mohr-Coulomb relationship is shown diagrammatically in Figure 2.1 in which τ is plotted against σ. The principal stresses that cause failure in a soil mass and the resulting normal and shear stresses on the failure plane, can be represented by a Mohr circle, which represent the state of stress for a particular element. Similarly, by varying the magnitude of the principal stresses on different elements of the same soil, we can obtain the corresponding normal and shear stresses at failure.

The representation of stress state at failure for any soil element through Mohr circles is a common practice in soil mechanics. Several circles representing the peak points of the

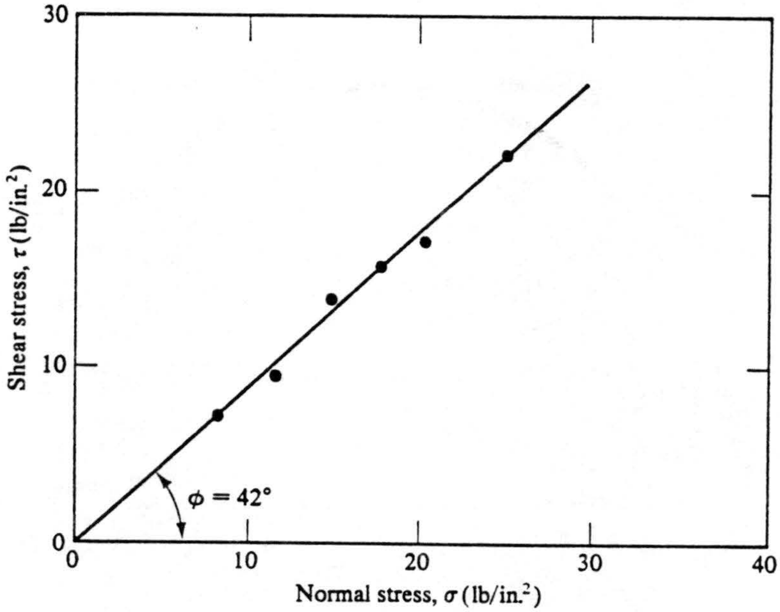


Figure 2.1 Determination of shear strength parameters for a dry sand using the results of direct shear tests (Das, Braja M, 1985)

stress-strain curves for different normal loads are usually plotted on a single diagram to represent the failure condition. A line drawn tangent to the Mohr circles gives the Mohr-Coulomb failure envelope. The Mohr circle for a given state of stress that lies below the failure envelope represents a stable condition for a particular soil under that state of stresses.

Under a wide range of confining stresses, the Mohr-Coulomb envelope is a curved line, because the shear strength contributed by interlocking of particles decreases as the confining stress increases. The reason is that the particles become flattened at contact points, sharp corners are crushed, and particles break. However, the failure envelope is usually considered to be a straight line rather than a curved line (Holtz & Kovacs, 1981).

Considerable experimental evidence has demonstrated that for quartz sand, the Mohr envelope for moderate confining pressures below 150 psi is for all practical purposes a straight line extrapolated through the origin of the Mohr diagram. For some granular materials like calcareous sand, this limiting value could be as low as 75 psi (Lambe, 1969). Sand is regarded as not having any true cohesion, and the absence of a chemical bond is illustrated by considering the fact that the soil will not stand as a cylinder when the confining pressure is zero (Ponce, 1970). Therefore, the failure law for sands can be expressed as

$$\tau_f = \sigma'_f \tan \phi' \qquad (2.3)$$

where σ' and ϕ' represent the effective normal stress and effective friction angle, respectively, of sands tested in a drained (dry) condition.

2.3 Measurement of Shear Strength Parameters

The shear testing of soils is done for two objectives: first, to gain a fundamental understanding of the mechanics of soils and second, to analyze or predict the behavior of a mass of soil under field-loading conditions. To achieve these objectives fully, all the components of stress and strain at any point within the specimen must be determinable (ASTM, 1963). Practically, this requires a reasonably uniform distribution of stress and strain within the specimen, and the control of measurement of stress and strain on the specimen surface. For purposes of engineering problem solving however, it is necessary only that the test results can be used to evaluate the strength under the conditions to which the soil will be subjected in the field.

At present, the principal laboratory tests employed for the measurement of soil shear strength parameters are direct shear and triaxial compression. The basic principles and procedures of these two tests are briefly described below.

2.3.1 Direct Shear Test

In its simplest form, the direct shear apparatus consists of a split square box having a lower frame that is stationary and an upper one that can be moved in a horizontal direction (Figure 2.2). Alternately, the upper half may be fixed and the lower frame may be moved. A soil sample (which may be square or circular in plan) is placed in the box. A normal force is applied to the top of the box after which a gradually increasing horizontal load (shear force) forces the upper frame across the bottom causing the soil to shear along the plane defined by the split between the top and bottom of the shear box. As the displacement of the upper frame increases, the force required to increase the displacement increases and approaches a maximum which is referred to as the 'peak value' (Figure 2.3). Then it decreases, approaches an 'ultimate value' and remains constant. The shear load at failure is divided by the cross-sectional area of the sample to give the ultimate shearing stress. The shear stress values corresponding to a set of normal stress values are found through a number of direct shear tests (usually three), and the results are plotted on a τ versus σ diagram. The best fit line drawn through these points gives the Mohr-Coulomb failure envelope from which the parameters c' and ϕ' can be computed. Figure 2.4 shows failure envelopes for normally consolidated and over-consolidated clays obtained from drained direct shear tests (Das, 1985).

The direct shear test is regarded as stress-controlled if the machine applies the shear force continuously at a suitable rate. On the other hand, if the displacement is increased

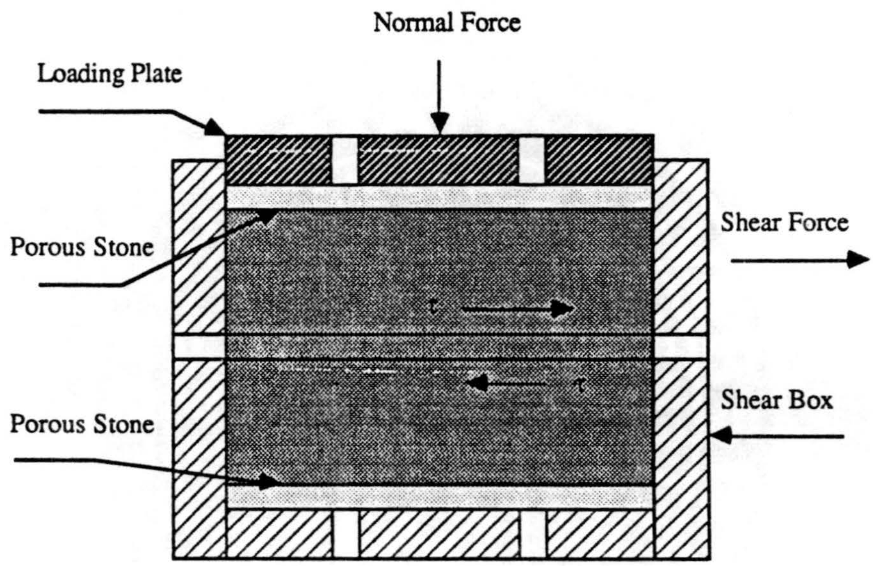


Figure 2.2 Diagram of direct shear test arrangement (Das, Braja M, 1985)

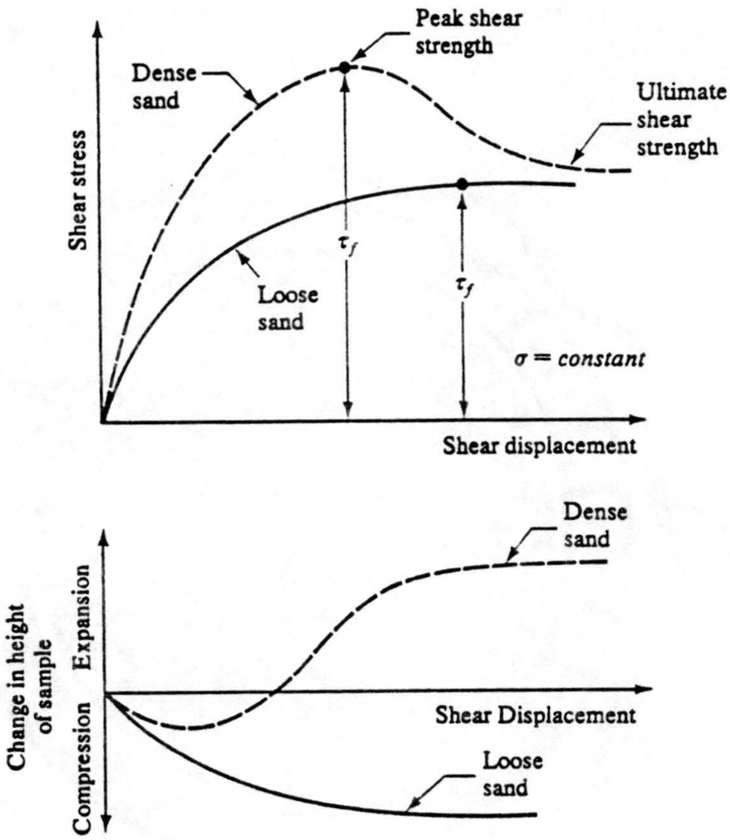


Figure 2.3 Plot of shear stress and change in height of sample against shear displacement for loose and dense dry sand (direct shear test) (Das, Braja M, 1985)

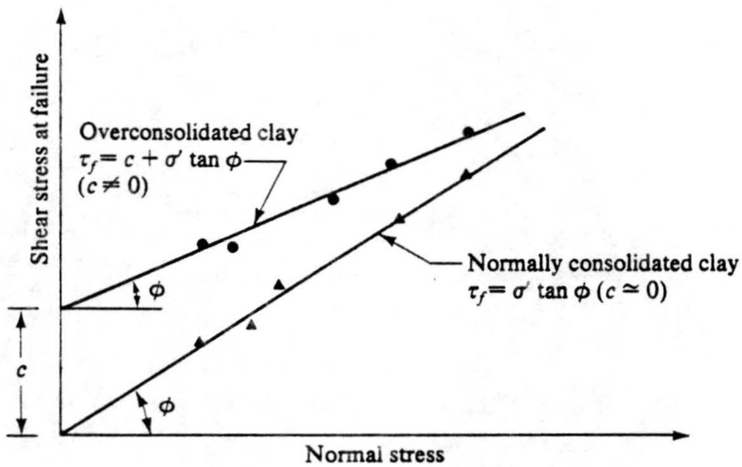


Figure 2.4 Failure envelope for clay obtained from drained direct shear tests (Das, Braja M, 1985)

with constant rate and the force required to produce this constant strain can be measured, the apparatus is said to be strain-controlled (Terzaghi & Peck, 1948).

2.3.2 Triaxial Compression Test

In the early history of soil mechanics, the direct shear test was very popular, but when its serious disadvantages such as the non-uniformity of stresses, the rotation of principal planes during the test, were realized, A.Casagrande at M.I.T. began research on the development of cylindrical compression test. A detailed discussion of the disadvantages of the direct shear test follows in section 2.5.

In the triaxial test, a soil specimen 1.4 in. in diameter and 3 in. in length is used. The sample is encased in a thin rubber membrane and placed inside a plastic cylindrical chamber that is filled with water or glycerine. The sample is subjected to an all-around confining pressure σ_3 by a compression of the fluid in the chamber (Figure 2.5). To cause shear failure in the sample, axial stress is applied through a vertical loading ram. The axial load applied by the loading ram corresponding to a given axial deformation is measured by a proving ring attached to the ram. The normal and shear stresses at failure are computed from a set of triaxial tests and results are plotted in the form of Mohr circles. A line tangent to these circles is drawn which is the Mohr-Coulomb failure envelope (Figure 2.6).

2.4 Genesis of Direct Shear Test

The first shear box was built by Alexandre Collin in 1846 for measuring the shear strength of clay for slope stability analysis. Collin's tests were under zero normal load, undrained and stress-controlled. Bell in 1915 built a direct shear box for undrained stress-controlled tests. Shear strength under different normal loads could be tested in this machine. This was done for the purpose of measuring the shear strength parameters for clay. Until this point, the effective stress concept was not known. But in the early 1920's, Karl Terzaghi introduced the effective stress theory and hence the importance of controlling drainage in shear strength tests was fully appreciated. New designs for the direct shear apparatus began to appear that were able to take the drainage into account. The main

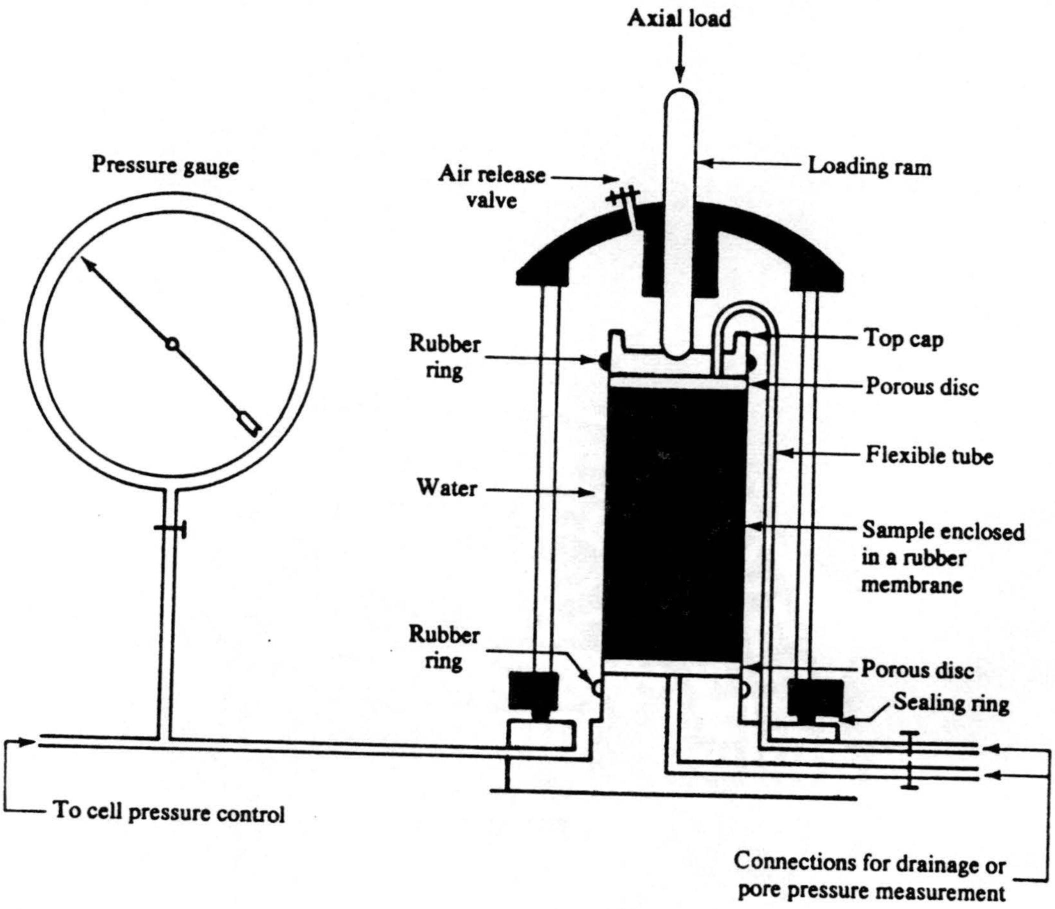


Figure 2.5 Diagram of triaxial test equipment (after Bishop and Bjerrum,1960) (from Das, Braja M, 1985)

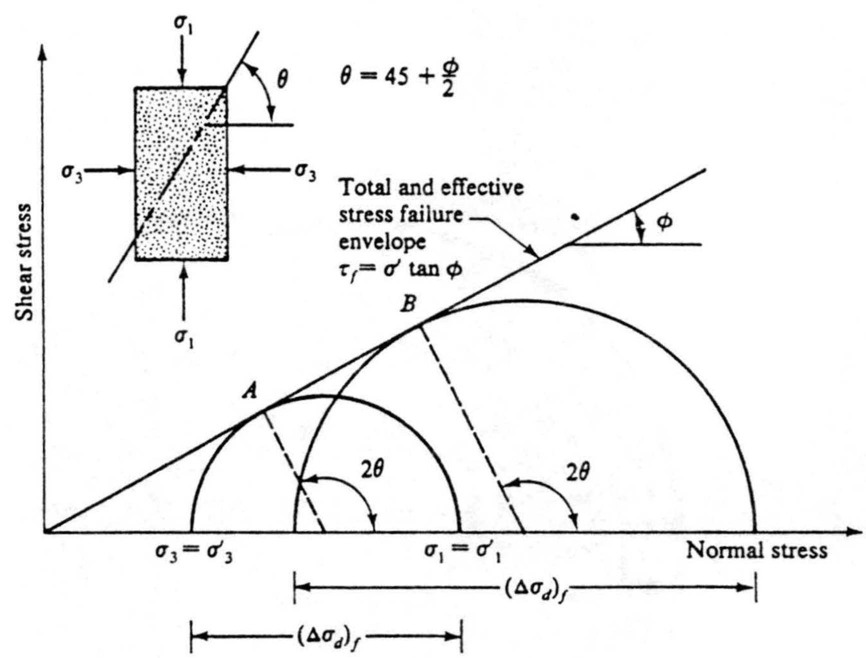


Figure 2.6 Effective stress failure envelope from drained tests in sand and normally consolidated clay (Das, Braja M, 1985)

disadvantage of early shear boxes was that they were stress-controlled, and therefore, did not permit the study of the behavior of the soil beyond the peak shearing resistance.

In 1932, A.Casagrande, a professor at Harvard university, introduced the modern shear box apparatus (square box). Gilboy developed a strain-controlled machine at M.I.T. in 1936 which allowed the behavior of the soil beyond the peak shearing resistance to be studied. A number of improvements were made to the direct shear apparatus in subsequent years. Notable among these were those made by Bishop at Imperial College. The test was received enthusiastically by the industry and has been in use for commercial testing ever since (Matthews, 1987).

Important work was carried out in 1930's for the development of shear strength tests for soils. At this time concern was voiced over the interpretation of direct shear box test data, because of the very complex nature of the distribution of stresses within the specimen. This led to the development of other tests such as the torsional test.

2.5 Stress-Strain Mechanism in Direct Shear Test

The complex nature of stress-strain in the direct shear test has been a question of interest with researchers and several attempts have been made by different groups to find explanations to this problem. These studies have been published in various technical journals. The essence of some of these works will be presented.

2.5.1 Rotation of the Principal Planes

When the soil specimen is placed in the direct shear box, and a normal force is applied but no shear force has yet been applied, the principal stresses are as shown in Figure 2.7. The major principal stress is applied to the top and bottom of the specimen. Although the distribution of this stress is not uniform because of the rigidity of the top and bottom of the box, it is ordinarily assumed to be equal to the normal load divided by the area. The intermediate and minor principal stresses act on the sides of the box. Although these stresses should be equal, their magnitudes are unknown. They might be computed approximately from Poisson's ratio, since the rigid box permits no lateral deformation. As

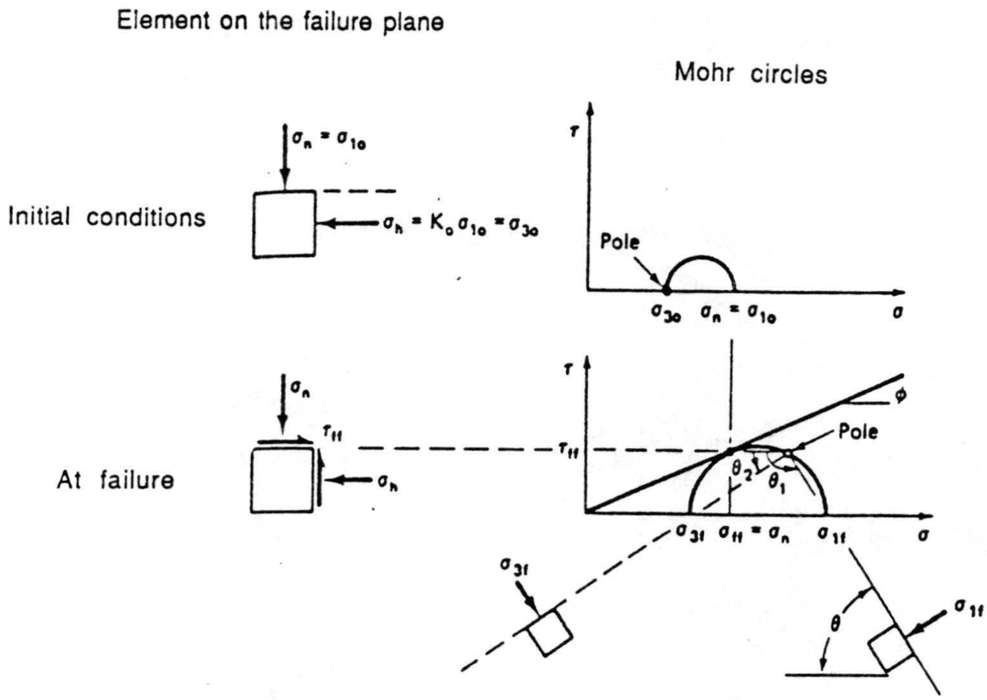


Figure 2.7 Diagram showing the state of stress on an element of soil and the rotation of principal planes during the direct shear test (Holtz and Kovacs, 1981)

the shear force is applied, the major and minor principal stresses and planes rotate.

The principal stresses rotate because at the start of the test, the horizontal plane, which is the potential failure plane, is a principal plane because only normal stress acts and there is no shear stress. But as the test progresses, and the shear force is applied on the horizontal plane, this plane does not remain a principal plane. Therefore, rotation of the principal planes must occur in the direct shear test. The amount of rotation of these planes can be determined by certain simple assumptions (Holtz & Kovacs, 1981).

2.5.2 Non-uniformity of Stress-Strain

The intermediate principal stress continues to act on the sides of the box that are parallel to the direction of the shear. Although vanes or grids and similar irregularities are introduced on the top and bottom surfaces of the box in an attempt to distribute the shear force uniformly, the stresses do not remain uniform. The major and minor principal stresses at failure can be computed from the shear and normal loads if they are assumed to be uniformly distributed. The intermediate principal stress, however, is indeterminate (ASTM, 1963; Jewel, 1987; Airey, 1987).

2.6 Finite Element Analysis of Direct Shear Test

Potts, Dounias, and Vaughan used finite element analysis to study the stress-strain mechanism in a direct shear test. Soil samples were subjected to both simple and direct shear tests. The simple shear test was a reference state for the direct shear box because the stress state and strain in an ideal shear box are uniform. The effects of the non-uniformities of stress introduced by the ends of the direct shear box were then examined using the finite element method and were compared with the reference state. The soil was modeled using an elasto-plastic constitutive law. The effects of initial stress, volume change characteristics and strain softening behavior were also` examined (Potts, Dounias & Vaughan, 1987).

A brief description of the soil behavior in the simple shear test, the finite element analysis of the direct shear box and its results are presented in the following sections.

2.6.1 Simple Shear Behavior

When the top boundary is displaced parallel to the bottom under plane strain conditions, the conditions in a layer of soil are shown in Figure 2.8. The behavior of a typical element is indicated. This form of deformation is defined as simple shear. The soil is subjected to a prescribed shear strain γ_{xy} and is constrained to zero direct strain in the x-direction (ε_x=0). As a result of these boundary conditions, the soil is subjected to a shear stress τ_{xy}, a direct strain ε_y, and a change in stress in the x-direction ($\Delta\sigma'_x$=0).

Although the ideal conditions shown in Figure 2.8 cannot exist in the direct shear box, this ideal state is useful, as a standard to compare predictions for the direct shear test. The soil is idealized as an isotropic linear elastic-perfectly plastic material operating in drained state, with its properties defined by a Young's modulus E', Poisson's ratio μ', angle of shearing resistance, ϕ', cohesion, c', and the angle of dilation ψ'. The behavior of a material with these properties in simple shear has been simulated as a single finite element. As a result of the boundary conditions imposed, horizontal planes remain horizontal and are inextensible (ε_x=0). At ultimate plastic failure, these planes are a set of velocity characteristics; such planes are inclined at +(45^0 - $\psi'/2$) to the direction of the major principal plastic strain increment. In an isotropic elasto-plastic material, the directions of principal stresses and plastic strain increments coincide. At ultimate failure, when all deformation is plastic, any horizontal plane is a velocity characteristic and is inclined at +(45^0 - $\psi'/2$) to the direction of the major principal stress. The Mohr circle of stress for this state is shown in Figure 2.9. From the geometry of the circle, the stress equation is

$$\frac{\tau}{\sigma'n} = \frac{(\sin\phi'\cos\psi')}{(1-\sin\phi'\sin\psi')} \tag{2.4}$$

This equation reduces to $\tau / \sigma'_n = \sin \phi'$ if $\psi' = 0$ and $\tau / \sigma'_n = \tan \phi'$ when $\psi' = \phi'$.

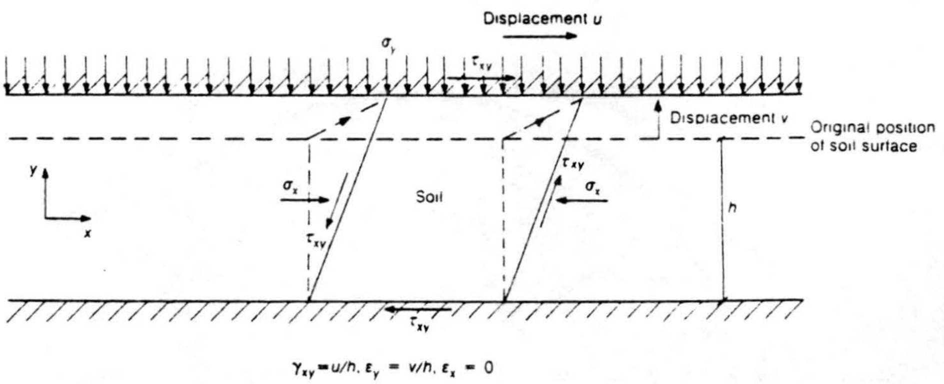


Figure 2.8 Diagram showing behavior of an element of soil in the simple shear test (Potts, Dounias and Vaughan, 1987)

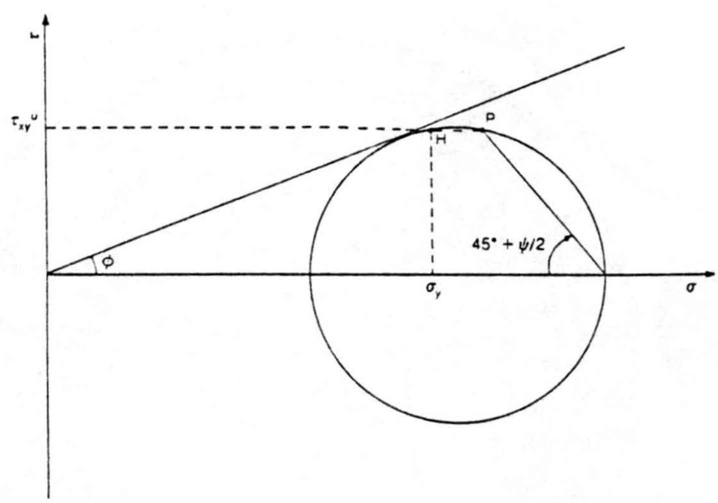


Figure 2.9 Mohr's circle of stress at ultimate conditions (Potts, Dounias and Vaughan, 1987)

This relationship indicates the dependence of the ultimate shearing resistance on both ϕ' and ψ.

2.6.2 Analysis of Direct Shear Box

The direct shear box has been analyzed using the finite element method. The mesh used is shown in Figure 2.10. It contains 150 eight-noded isoparametric elements. The box is 60 mm long by 20 mm deep of two equal halves, which is the common size.

Two approaches were adopted to simulate the initial stresses in the sample before shearing. In the first approach, a very small isotropic initial stress was applied on the sample whereas the vertical load was applied as a single force on the center of the top cap. This exactly simulates what happens in the conventional test. In the second approach, higher values of K_o, such as might arise in an over-consolidated sample, were modeled by giving the sample the appropriate uniform value of initial stress. Both approaches were used for $K_o = 0.43$ and they gave the same result (Potts, Dounias & Vaughan, 1987).

2.6.3 Non-Strain-Softening Behavior

Predictions for both analyses A and B are shown in Figure 2.11. Both analyses have $K_o = 0.43$. Analysis A has an angle of dilation, $\psi = 0$ and analysis B has $\psi = \phi = 35^0$. The predictions for ideal simple shear are shown for comparison. A close agreement was found between the results for the direct shear box and for ideal simple shear test. When $\psi = \phi$, the same ultimate strength of $(\tau_{xy} / \sigma_y)_u = \tan \phi$ is recovered. However, when $\psi = 0$ the ultimate strength obtained from the direct shear box is 4.5 % higher than the value of $(\tau xy / \sigma y)_u = \sin 35^0$ given by simple shear. During initial loading, the direct shear box shows a 10-20 % stiffer response than simple shear.

Contours of stress level S and local shear strain ε_{xy} are presented in Figure 2.12 for analysis with $\psi = 0$, and for the three stages a, b and c of analysis A as indicated on

Figure 2.11.

The stress level S represents the mobilized proportion of the strength that is currently available at the normal stresses which are operating. It varies from zero at

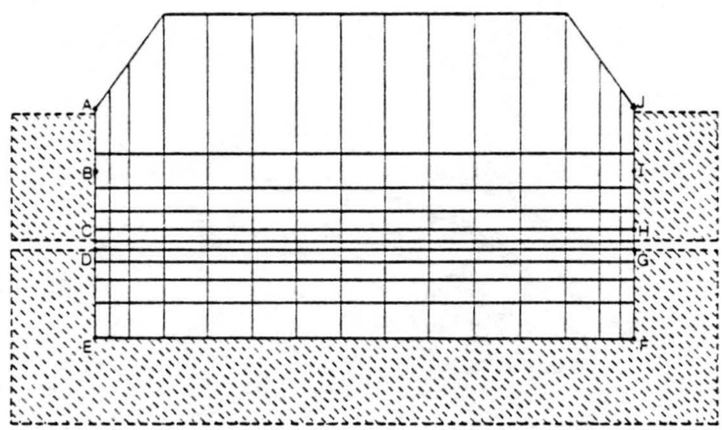


Figure 2.10 Finite element simulation of the direct shear box (Potts, Dounias and Vaughan,1987)

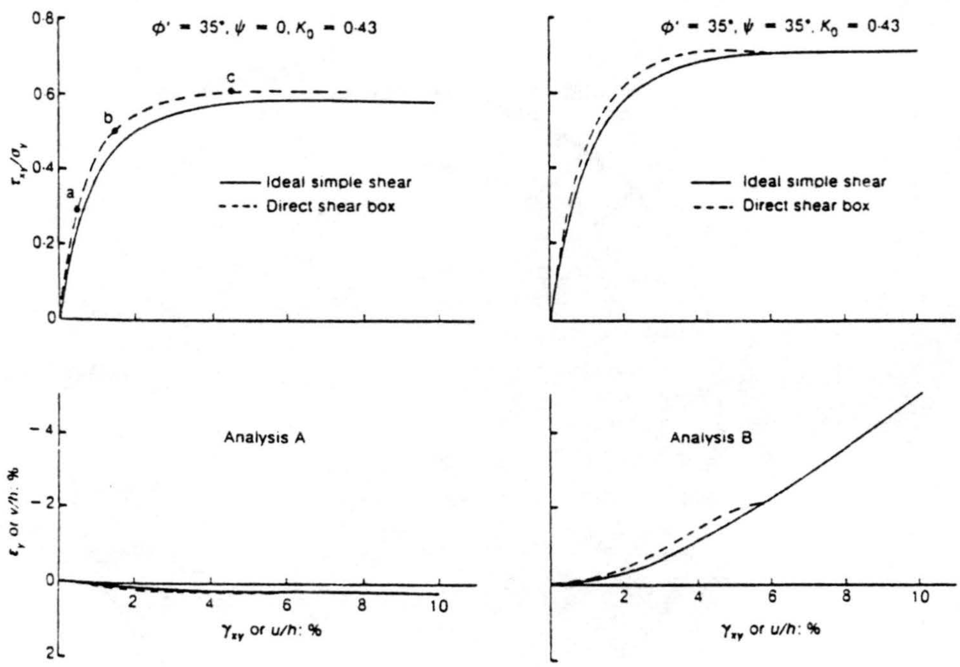


Figure 2.11 Comparison of direct shear box and ideal simple shear load-displacement curves: the effect of dilation angle ψ

isotropic stress to unity when the stress state is on the yield (failure) surface. Both sets of contours indicate that highly stressed zones propagate from the edge of the box at an early stage of loading. These zones are oriented at approximately 45^0 to the horizontal. With further loading these zones grow and rotate and eventually reach the top and bottom surfaces of the box. As failure is approached, they rotate until, when the ultimate shearing resistance is reached, they lie close to the horizontal plane across the center of the box.

Despite the good overall agreement between the load-displacement behavior for the direct shear box and ideal simple shear, the analyses confirm that the local state of stress and strain within the direct shear box are far from uniform. The effects of these non-uniformities in combination with strain softening in the soil might be expected to cause progressive failure (Potts, Dounias & Vaughan, 1987).

2.6.4 Strain-Softening Behavior

Stresses and strains are highly non-uniform within the direct shear box and thus progressive failure might occur if the soil strain softens. This would reduce the peak strength observed in the box below the true strength of the soil if it were tested under conditions of uniform stress and strain. To investigate this possibility, further analyses have been performed in which the soil has been modeled as a strain-softening material.

Two types of softening have been examined: first, from dilation in a dense soil, and secondly, from particle orientation in a clay. To model the first type of softening, a form of the modified Cam clay computational model has been used, with the soil assumed to be over-consolidated. For the second type of softening , the simple elasto-plastic model used for non-strain-softening predictions has been extended to allow the strength parameter ϕ' to reduce with straining. The modified Cam clay model used employs a Mohr-Coulomb hexagon for the shape of the yield surface in the deviatoric plane of constant mean effective stress. Thus the angle of shearing resistance, ϕ', was independent of the intermediate principal stress. However, the plastic potential adopted plots as a circle in the deviatoric

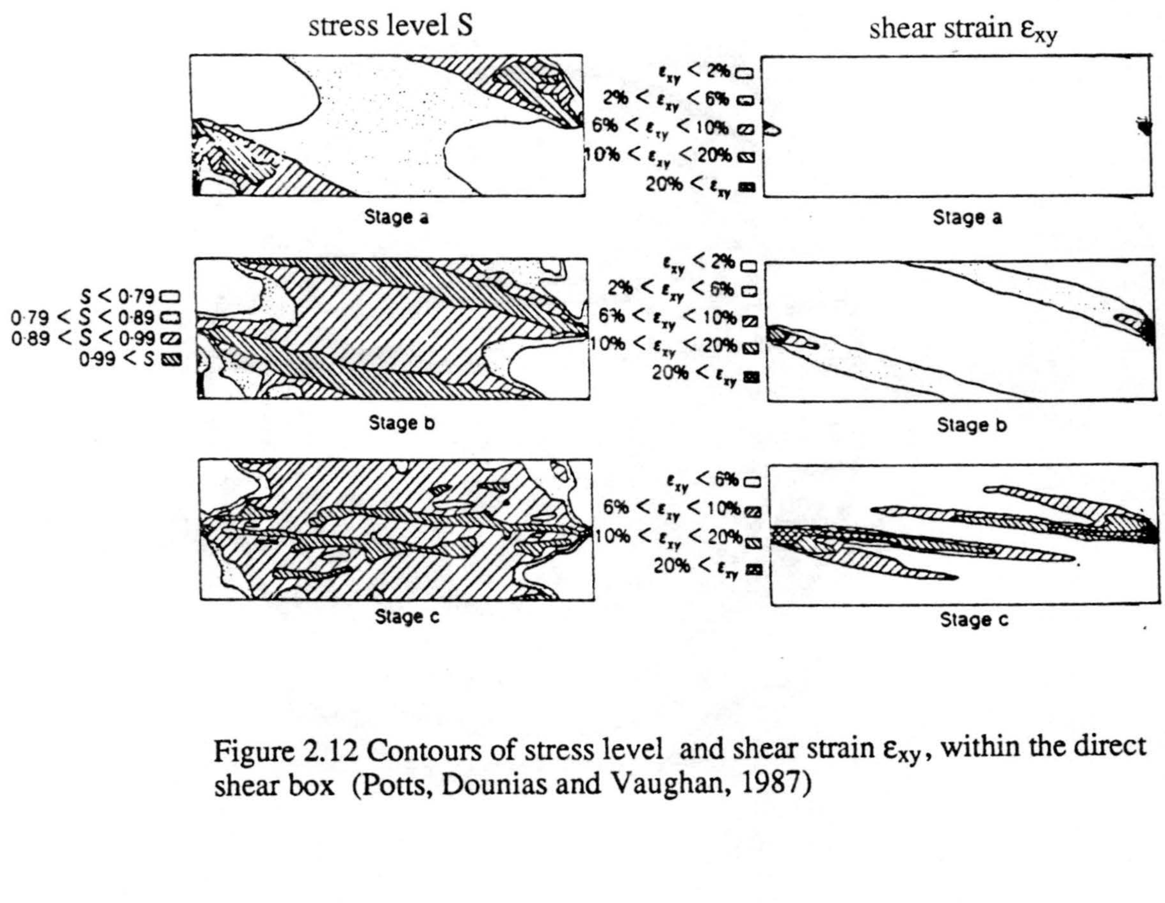


Figure 2.12 Contours of stress level and shear strain ε_{xy}, within the direct shear box (Potts, Dounias and Vaughan, 1987)

plane and this resulted in ultimate conditions (critical state) with $\sigma_2' = (\sigma_1' + \sigma_3') / 2$ in plane strain.

From these analyses, little difference was found between the stress-strain behavior plots before the peak, although the direct shear box again shows slightly stiffer behavior. The peak resistance in the direct shear box is only 2.0 % less than the simple shear prediction. Thus, despite the non-uniform conditions within the direct shear box, the predicted effect of progressive failure is very small.

For the second form of strain softening, that due to particle orientation, the simple elasto-plastic model used previously has been modified. The same parameters have been used to describe the behavior up to peak. A plastic strain of 5% is then allowed at constant strength.

Two analyses were performed. In the first analysis with the severe strain-softening assumption, the direct shear box predicts the strength in simple shear correctly. In the second analysis, the direct shear box overpredicts the strength in the simple shear by 1.7%.

An interesting finding of this study was that the results of the direct shear box model indicated brittle stress-strain behavior in some cases, but the soil model used did not include any brittleness. This suggests that brittle behavior may occur in the direct shear test as a result of the mode of deformation and may not always be a fundamental soil behavior .

2.6.5 Rotation of Top Cap

The effect of top cap rotation in the direct shear test was also analyzed. The rotation of top cap had little influence on stress-strain behavior. However, the tilting of the cap had a significant effect on both the peak and ultimate strength. The resistance of top cap movement as a result of the tilting action will cause the measured stress ratio to be greater than that acting on the soil and increase the non-uniformity of the stress distribution. These factors may be responsible for the strengths obtained from direct shear tests being higher than those obtained from simple shear tests (Figure 2.13) (Potts, Dounias & Vaughan, 1987).

2.7 Direct Shear vs Triaxial Compression Tests

In practice the box-shear apparatus has several inherent disadvantages. Foremost among these are the change in area of the surface of sliding as the test progresses, the unequal distribution of the shearing stresses over the potential surface of sliding, and the rapidity with which the water content of saturated samples of many types of soils changes as a result of a change in stress.

As the horizontal displacement of the upper frame increases, the area of contact between the upper and lower half of the sample decreases. Therefore, even with strain-controlled apparatus, reliable information concerning the ultimate shearing resistance of the sample cannot be obtained. Furthermore, the shear failure does not take place simultaneously at every point of the potential surface of sliding. It starts at the two edges and proceeds toward the center. Therefore, the peak value of the shearing resistance indicated by the test results is lower than the real peak value (Head, 1982). The direct shear test, is now objected to because

1. The area of the specimen changes as the test progresses but the shear strength is calculated by dividing the peak stress by the initial area of the specimen.
2. The actual failure surface is not planar, as is assumed or as was intended from the way the shear box was constructed; nor is the shearing stress uniformly distributed over the failure surface, as is also assumed.
3. Uncontrollable rotation of principal planes and stresses occurs during the test.
4. Values of modulus of elasticity and Poisson's ratio cannot be determined.

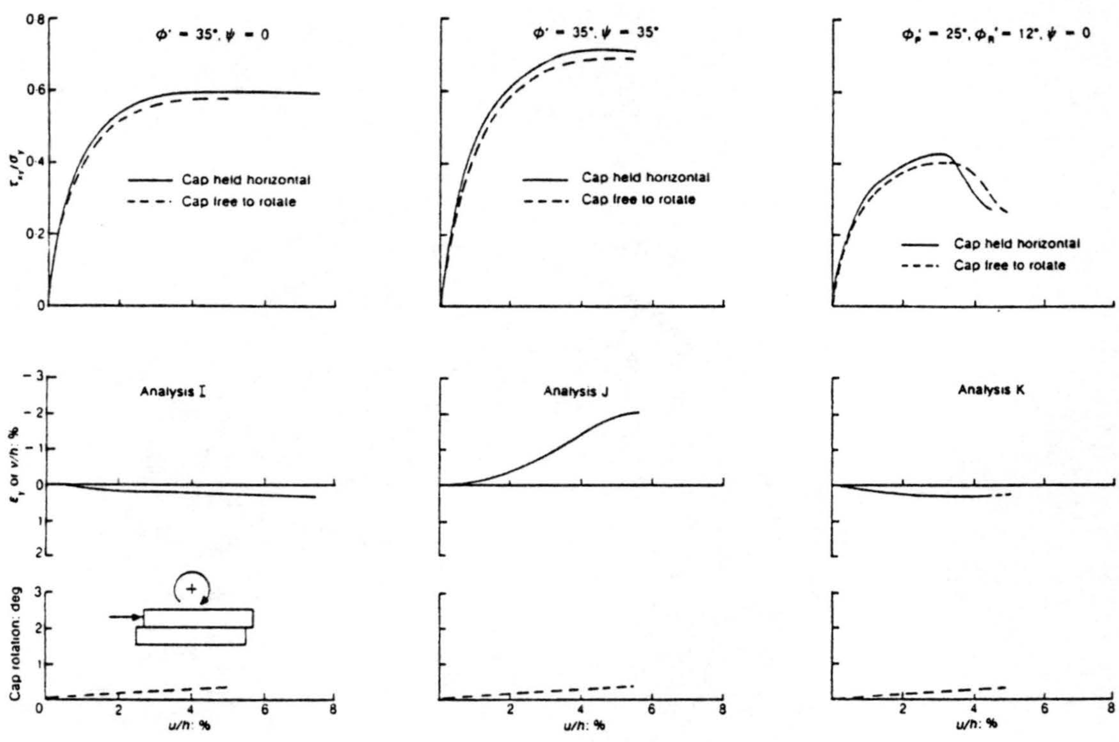


Figure 2.13 Load-displacement curves for the direct shear box with and without restraint of cap rotation (Potts, Dounias and Vaughan, 1987)

Direct shear is considered to be a simple and quick test and hence quite economical. The triaxial test on the other hand is much more complicated than the direct shear test and is more versatile. A wide range of stress states can be controlled in two or three directions and both loading or unloading can be imposed either to evaluate basic phenomena or to simulate field conditions. The resulting deformations can be measured. Seepage, drainage, and pore pressure can be controlled. The principal stresses do not rotate. The failure plane is not imposed and can occur anywhere. Although both theoretical and experimental investigations have found the stresses and strains to be non-uniform, they are assumed to be uniform for practical purposes.

The approximately 150 year successful history of direct shear test provides sufficient proof that the industry could not discard it completely and has been looking at it as a useful, economical and quick laboratory tool to measure the shear strength parameters of engineering soils. One of the other reasons for relying on this test is that the results are compatible with those of the triaxial test.

2.8 Taylor's Work

Donald W. Taylor carried out research at M.I.T. on shearing properties of sands. In this study (Taylor, 1939), direct shear and triaxial tests were performed on sands. Four types of sands were tested: (i) Ottawa standard sand, (ii) Sand A, (iii) Sand B, and (iv) Sand BW. The shear strength properties that were compared in this study included: (i) maximum angle of internal friction, (ii) shape of the shearing-stress versus shearing-strain curve and, (iii) critical void ratios. The comparison of maximum friction angles is:

The maximum friction angle depends on the void ratio of the sample and, to a smaller degree, on the pressure. A fairly good conception of the agreement between the two methods can be obtained from results at the loosest and densest state and such results for all four sands are compiled in Table 2.1. For dense Ottawa sand at low pressures, the disagreement is greatest and equals about 4 degrees. Other differences do not exceed about 2 degrees and on the average are equal to about 1 degree. The friction angles tend to be larger by the direct shear method, especially for the dense state. The friction angle as determined by direct shear tests varies with the size of sample and conditions of testing.

Comparisons of results from different makes of direct shear machines have shown variations fully as large as those appearing in Table 2.1. The friction angles by the two methods are not as close in all cases as would be desirable from the research point of view. The ultimate friction angle as determined by the direct shear machine is a constant and equals 26.7 degrees for Ottawa sand , while for sand A, B, and BW, it equals 33 degrees.

Taylor has expressed the following views regarding the direct shear tests:

> " The question of the relative value of results from the two types of tests has been the subject of much recent discussion. One school of thought swings to an extreme and claims that the direct shear test is ready for discard. The author's beliefs on this question are the following:
> The value of the cylindrical (triaxial compression) test has been proven, nevertheless it has undesirable features and cannot yet be considered a perfect method. The simplicity and practical features of the direct shear test must be recognized and the amount of use made of this type of test in the future should be determined by the dependability of results which it can produce; enthusiasm for a new method is never sufficient grounds for rejecting an older, established method unless it has been definitely proven that the older method is inferior."

Table 2.1 Maximum friction angle comparisons
(From D.W. Taylor)

Soil Type	Stress σ_3, psi	Initial Void Ratio, eo	Maximum Friction Angle, ϕ_m (degrees) By Direct Shear Test	By Cylindrical Test	Difference
Ottawa sand	20	0.65	28.5	26.7	1.8
		0.55	34.1	30.3	3.8
	60	0.65	27.2	26.4	0.8
		0.55	31.1	30.3	0.8
Sand A	15	0.75	33.9	35.1	-1.2
		0.60	41.7	40.5	1.2
	30	0.75	33.5	33.8	-0.3
		0.60	39.9	39.3	0.6
Sand B	18	0.85	33.8	33.8	0.0
		0.60	42.8	41.9	0.9
	34	0.85	33.2	31.6	1.6
		0.60	42.0	40.7	1.3
Sand BW	30	0.85	33.8	34.0	-0.2
		0.65	44.4	41.8	2.6
	60	0.85	32.7	32.2	0.5
		0.65	42.6	39.6	3.0

CHAPTER 3

LABORATORY PROGRAM

A detailed laboratory testing program was developed and executed for this study. This chapter presents the details of the laboratory testing program and describes (i) the type and properties of the material tested, (ii) the type and extent of different laboratory testing performed, (iii) the different types of equipment used, and finally, (iv) the procedure for each type of the tests.

3.1 Properties of Testing Material

The material tested was silica sand from Colorado Springs, Colo. The sand is composed of two types of particles, the white quartz particles with a hardness of 7.0 and reddish particles of potassium feldspar ($KalSi_3O_8$) origin with hardness 6.0-6.5 (values assigned by visual examination). Some of the reddish particles had their color changed slightly due to weathering. A gradation analysis was performed. The gradation curve is presented in Fig 3.1. The sand is uniformly graded with a uniformity coefficient, $C_u = 2.0$. The particles mostly are of sub-rounded shape; however, a small percentage of angular particles is also present. The specific gravity is 2.64. A random sample of 1 kg found that 676 gm by weight are white quartz particles and 324 gm are reddish particles of potassium feldspar origin.

3.2 Testing Program

A detailed laboratory testing program was evolved and carried out. Both triaxial compression tests and direct shear tests were performed. The basic principles of these two

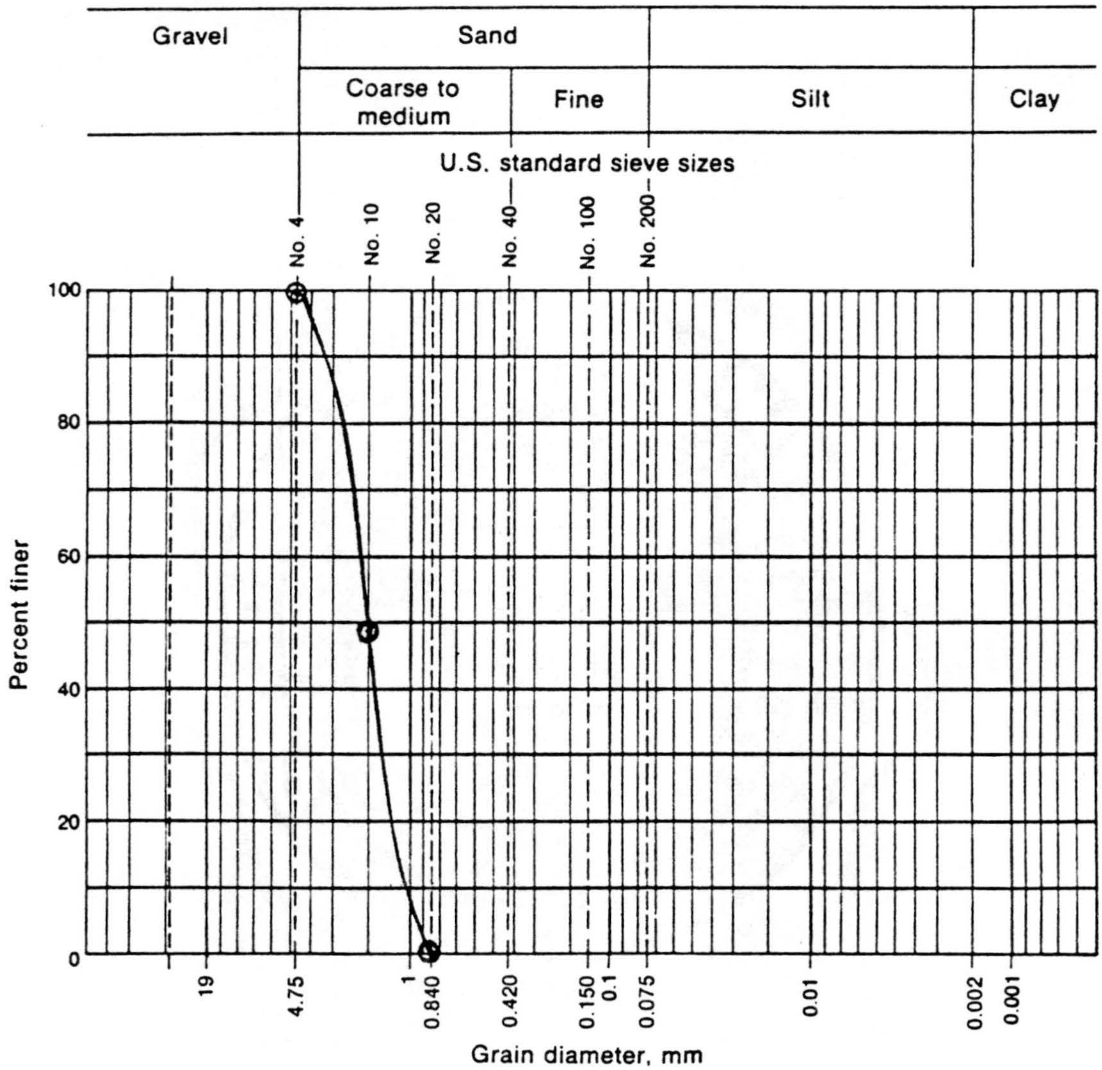


Figure 3.1 Mechanical Analysis graph

tests have been described in detail in chapter 2. The following sections describe the number of each of the two types of tests done, the type of equipment used in the laboratory testing, the magnitudes of normal and confining stresses applied, and detailed procedures of the tests performed.

3.2.1 Triaxial Tests

Five groups of triaxial tests were performed in all. Two were done as multi-stage tests and three were performed on three individual samples, each remolded at approximately the same density in each case. These samples were 3.0" long and 1.4" in diameter and were sheared at confining pressures of 20, 30 and 40 psi respectively, in a dry state.

3.2.2 Direct Shear Tests

A total of 140 direct shear tests were performed using four different makes of direct shear test machines i.e. 35 tests on each machine. For each machine, twenty of the tests were performed for a normal stress of 54 psi and 15 were done under a normal stress of 29 psi each.

3.2.3 Proving Rings

Three proving rings of different capacities were used for the triaxial and direct shear tests. The Soiltest machine is equipped with a proving ring of 500 lb capacity. This was used for the direct shear tests on the Soiltest machine and the triaxial tests. The Wykeham Farrance machine has a proving ring of 1000 lb capacity. This was used for direct shear tests on this machine as well as the Clockhouse machine. Similarly, the ELE machine is equipped with a proving ring of 600 lb capacity which was used for direct shear testing on this machine.

3.2.4 Normal Stresses for Direct Shear Tests

To minimize the test variables, direct shear tests were performed under two normal stresses only i.e. 54 and 29 psi which are the minimum requirement for defining a straight line. Carrying out tests under normal stresses that fall in the mid-range of the proving ring capacity is another factor that ensures accuracy of stress application. The normal stress of 54 psi was selected keeping this idea in mind.

3.3 Procedure of Triaxial Compression Test

All triaxial testing was performed in the graduate research geotechnical laboratory located in the Weber building at CSU (Figure 3.2).

3.3.1 Preparation of Test Specimens

One end of a rubber membrane (0.012" x 8" x 1.4") was fitted over the pedestal of the triaxial machine which was lubricated for tightness with high-vacuum grease. A dry porous stone was placed on top of the pedestal. A split mold (also called split former) fitted with a vacuum connection was clamped to the pedestal. The mold enclosed the membrane. The vaccum when applied, created suction along the inside walls of the mold which held the membrane tightly in contact with the inner wall of the split mold. Sand was first weighed and then placed in this mold up to a level that would give a specimen of minimum 3.0" height. A cap was placed above the sand, and a vacuum of 2 to 3 psi was applied. This kept the sand in a vertical position after the split mold was removed. After removing the mold, two 'O' rings, were fitted to the top and bottom of the specimen. This was done with the help of a membrane stretcher. The specimen diameter and height were carefully measured with a vernier calliper and a pi tape. The diameter was measured at three points: at the specimen top, middle and bottom. An average of these three values gave the mean diameter of the specimen. Similarly, the height was measured at three different points along the circumference of the specimen to obtain the mean height of the specimen. The volume of the specimen was then calculated. Since the weight of the sand used in preparing the specimen was known, the density of sand could be calculated and recorded. The specimen was then ready for testing (Head, 1982).

3.3.2 Density of the Test Specimens

Preparing sand samples of nearly identical density and ensuring a uniform density throughout the specimen require both care and skill. The most popular method for making

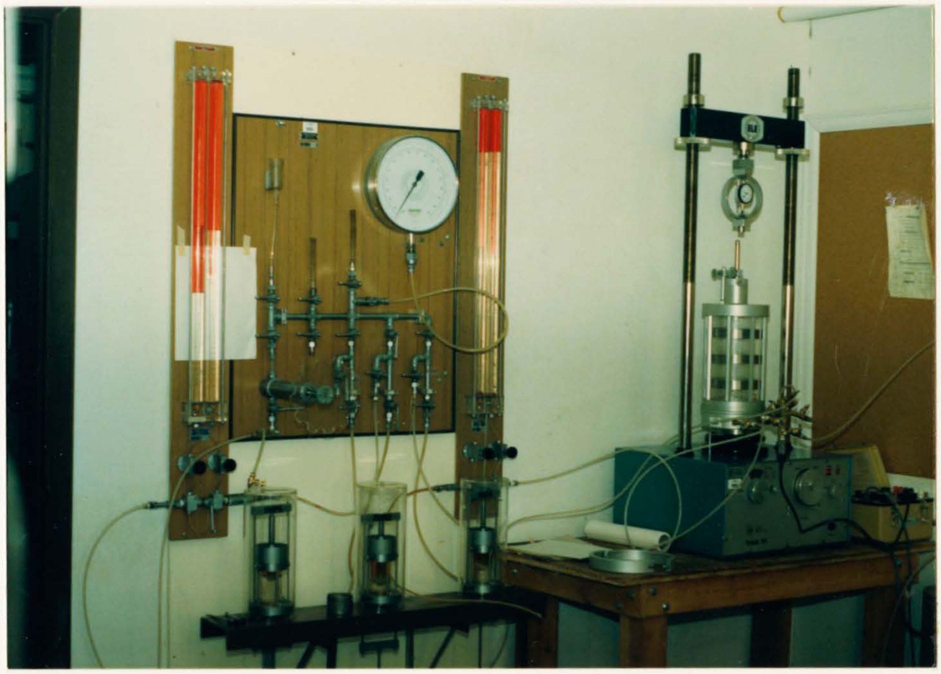


Figure 3.6 Triaxial test apparatus

sand samples is the one described by Head in his Manual of Soil Laboratory Testing vol 2, 1982.

In this method, a mold for the specimen is made, which encloses the membrane and holds it in a stretched position. The procedure is described in detail in section 3.3.1. A weighed quantity of sand is poured into the mold through a funnel fitted with a length of rubber tubing. To obtain a loose specimen of low density, continuous rapid pouring from a small drop, which should be kept constant by steadily raising the funnel, should be used. A loose specimen should not be subjected to shock or vibration. A higher density may be obtained by pouring sand at a slower rate from a higher drop. This method requires continuous agitation of sand with a glass rod because the sand gets stuck in the tubing. However, if the sand is allowed to drop freely from a certain height, the particles arrange themselves differently than if the particles are forced to drop. The author carried out experiments with varying diameter of tubing as well as with different size of opening in the base of the funnel. A Plexiglas cylinder and the upper half of the direct shear box (Soiltest) were used as molds. The Plexiglas cylinder was of 1.5" internal dia. whereas the diameter of the shear box was approximately 2.5 inches. A free drop of sand due to gravity could be ensured only with a rectangular (5/16" x 3/4") slot in the funnel base. The slot in the base of the funnel was made first in a round piece of plastic which was then fitted in the funnel base. As is shown in Figure 3.3, a stiff tubing made of poly-vinyl and 1" dia. was used. This funnel allowed the sand to drop freely and continuously and the tubing did not choke. A number of density tests were performed with the help of this funnel by (i) varying the length of rubber tubing attached to the funnel and keeping the tubing length constant and (ii) varying the height of drop of grains. The results of the density testing with the direct shear box as a mold are shown in Figure 3.4 and those with the Plexiglas cylinder are shown in Figure 3.5.

Increasing the length of tubing would not affect the density of the specimen to a considerable extent. The height of drop measured from the lower end of the tubing actually

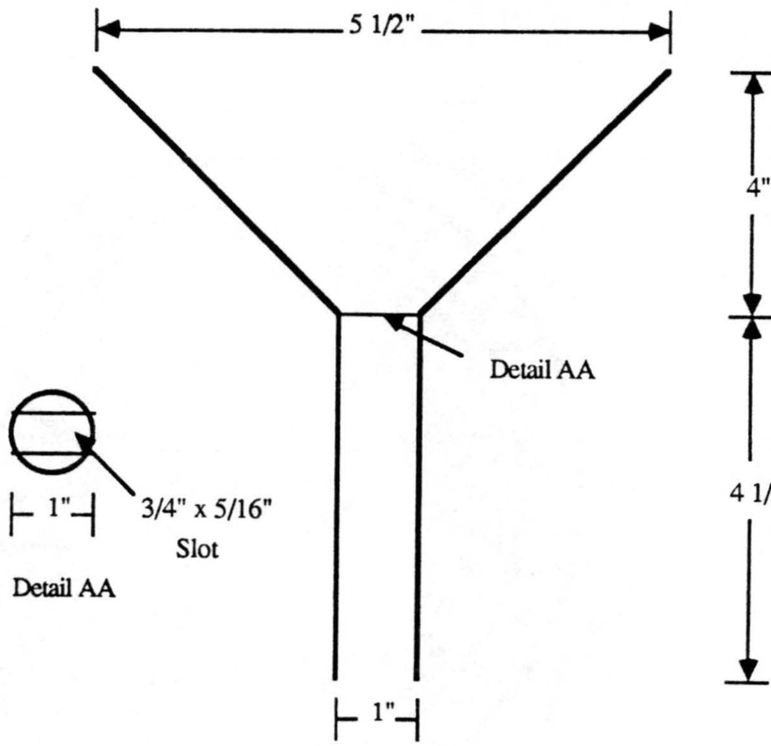


Figure 3.3 Diagram of plastic funnel used for preparing test specimens
(Not to scale)

Fig 3.4 Drop of grain vs density of sand
using direct shear box as mold

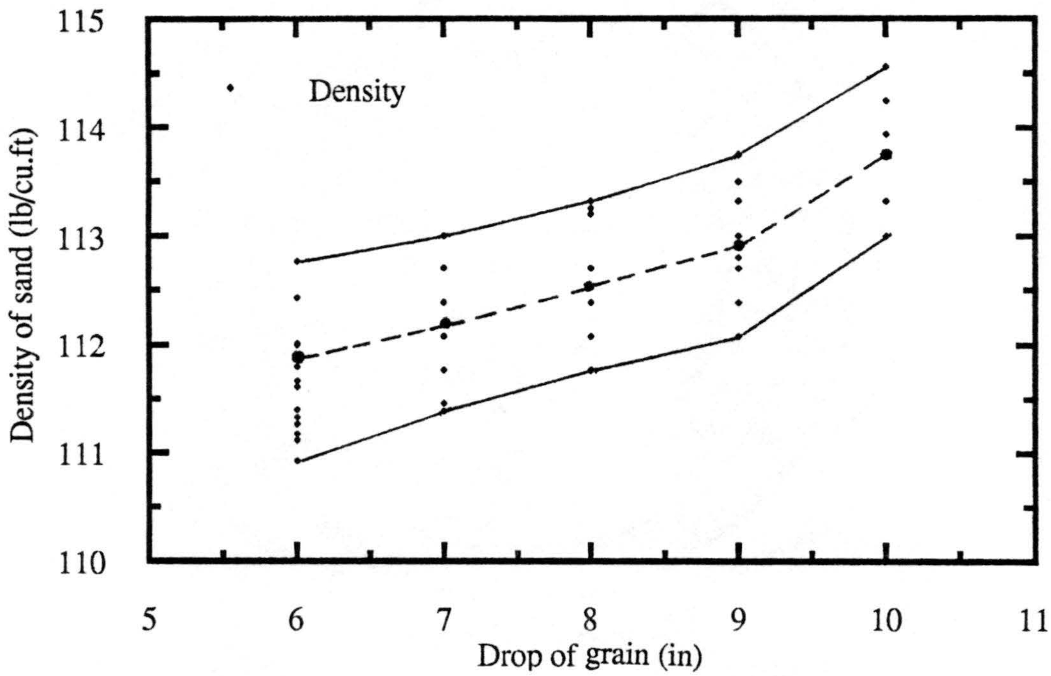
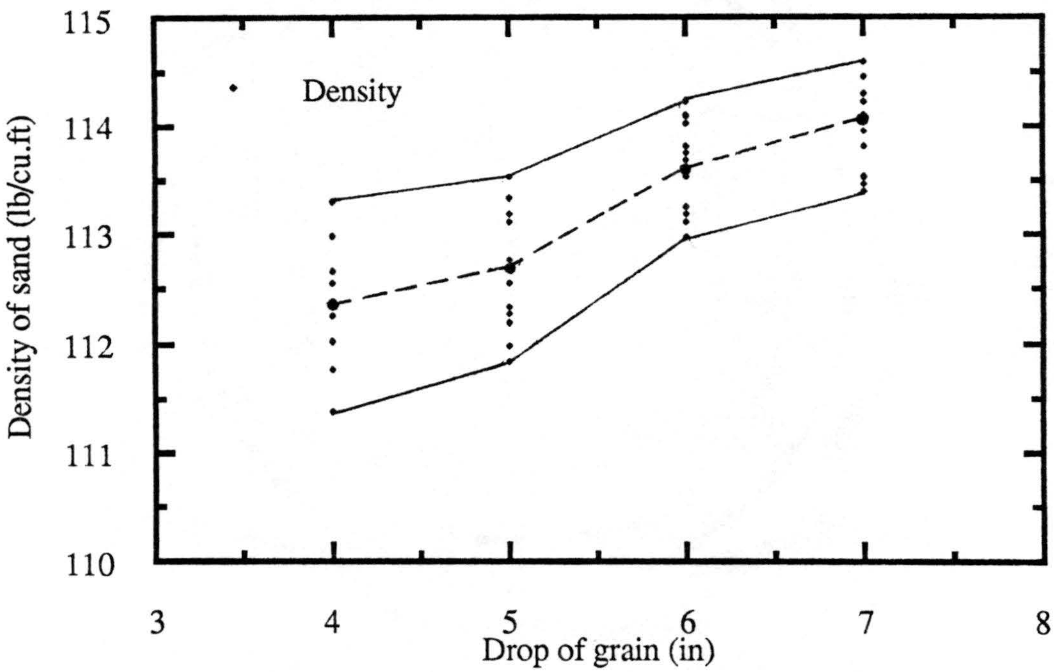


Fig 3.5 Drop of grain vs density of sand
using Plexiglas cylinder as mold



affects the density. To achieve a high density, therefore, the base of the funnel has to be kept at a considerable height from the mold. Similarly, a lower density can be achieved by keeping the lower end of tubing close to the mold.

The author conducted a number of triaxial tests on specimens prepared by the above method. Dropping sand from a constant height through a funnel would not give the same density every time in actual testing. This was indicated by the inconsistent results obtained from the trial triaxial tests. Therefore, the following steps were followed in making the test specimen and the results obtained were found to be quite consistent.

1. First, the amount of sand was weighed that would fit in a fixed volume of the mold to give the desired density.
2. The sand was split into three portions (by weight).
3. Each portion of the sand was allowed to drop freely through the funnel fitted with tubing. This allowed the sand particles to arrange and settle naturally in the mold.
4. Each portion was fitted into one third of the available volume by tamping the sand slowly with a steel rod.
5. The degree of tamping in each layer is varied to ensure that the bottom layer gets the least compactive effort while the top layer gets the greatest. This will take into account the undercompaction idea suggested by Richard Ladd (Ladd, 1978).

This method gives a density uniform to the maximum extent. However, obtaining almost the same density in each specimen is difficult even with this method. In this study, the densities achieved varied by 1-3%. To further minimize the effect of variability in density on test results, two sets of the triaxial tests were carried out as multi-stage tests.

3.3.3 Relative Density Tests

Relative density testing on the testing material was performed by the Empire Laboratories, Inc . of Fort Collins. The dry method of relative density test was adopted in accordance with ASTM procedures D 4253, D 4254. The minimum density of the material was 93.7 pcf, and the maximum density was 108.4 pcf. Figure 3.6 shows the results of the relative density tests.

3.3.4 Void Ratios

The void ratios of test specimens for all tests ranged between 0.54 to 0.57.

3.3.5 Application of Confining Pressure

A Plexiglas chamber or cell is placed around the specimen and tightly clamped along the base of the triaxial machine. The connection of the cell and base is made leak-proof by applying a layer of high-vacuum grease to the inside face of the clamp. The cell is then filled with de-aired water. Pressure is applied to the water in the cell which confines the pressure acting around the specimen. This pressure is increased slowly and gradually. Sudden pressure may result in rupturing the rubber membrane. When the confining pressure is equal to about 4 to 5 psi, the suction or vacuum is brought to zero because the confining pressure keeps the specimen standing by itself. By opening the drainage valve connected to the base of the specimen, the specimen inside the membrane is brought to atmospheric pressure. The confining pressure is now increased in small increments, up to the desired level.

3.3.6 Application of Deviator Stress and Shear Failure

When the desired level of the confining pressure has been achieved, an axial load is applied at the top of the specimen by lifting (straining) the specimen against a steel loading ram. The load is transferred to the specimen through a steel proving ring fitted with a dial gauge, which indicates the value of deviator stress. Readings are recorded at constant intervals of strain until failure occurs. The failure is reached when the strain either remains constant or the change is very small.

After the failure has been achieved, a final observation of the drainage valve is made. If water drops are coming out of this valve, this indicates that the membrane has ruptured during the test and the water is flowing through the soil specimen. This warrants

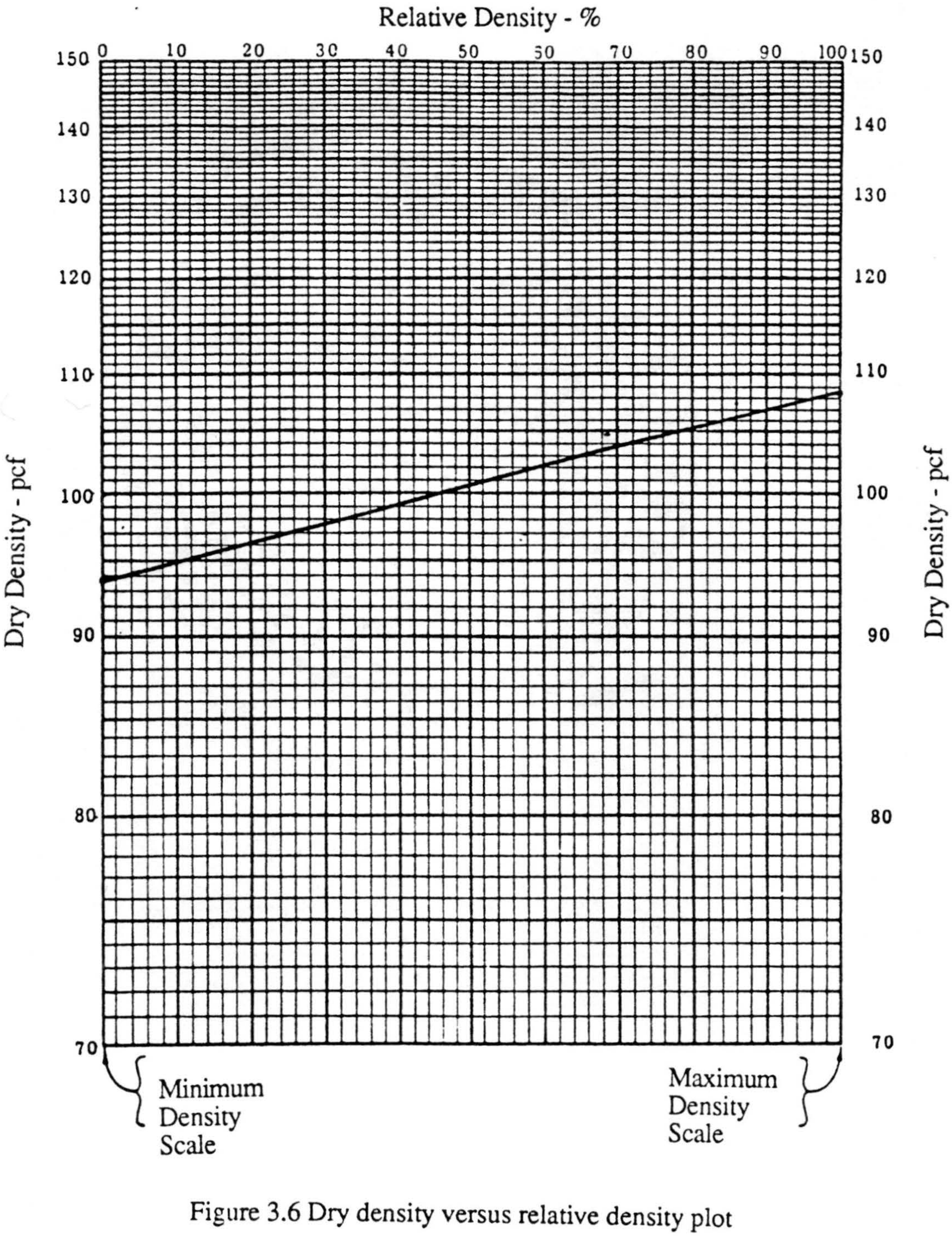


Figure 3.6 Dry density versus relative density plot

the repetition of the whole test. The machine is stopped, the confining pressure is withdrawn in decrements, the cell water is drained out, the soil specimen is dismantled and the failure plane is studied. The sand from the specimen is collected and weighed again to double check the density.

3.3.7 Multi-stage Triaxial Testing

In the multi-stage tests, the axial load is applied to the specimen under the first cell pressure until the load-strain curve indicates that failure is imminent. This is called the first stage or simply stage 'A'. The soil is not allowed to fail under this pressure, rather the test is stopped before the load-strain curve reaches a maximum value. The cell pressure is then increased to the second value of desired confining pressure (stage B), and compression of the sample is resumed. The test is stopped for a second time when the load-strain curve indicates that the maximum value is about to be reached. The cell pressure is increased to the third value and compression is continued until the sample finally fails. The load-strain values are noted after the failure has been achieved (Head, 1982 & 1986).

3.4 Procedure of Direct Shear Test

The procedure of the direct shear tests followed in this study conforms to ASTM D 3080-72 and procedures outlined in Lambe (1951), and Bowles (1970).

3.4.1 Equipment

The direct shear testing was done in the Fu Hua Chen Geotechnical Engineering Laboratory located at CSU. This laboratory is equipped with four makes of direct shear testing machines manufactured by the Soiltest Inc, Clockhouse, Wykeham Farrance, and ELE (Figures 3.7 through 3.10). For convenience in identification of these machines, henceforth, they will be refered to through their manufacturer name.

A total of 35 tests were performed on each of the direct shear testing machines, 20 tests under a normal stress of 54 psi and another 15 under a normal stress of 29 psi. Though the general principle and procedure of direct shear test in each case is the same, a description of the dimensions, shape, and weight of test specimens, in each machine is given in the following sections. The test specimens for all the direct shear tests were formed following the procedure outlined in section 3.3.2.

3.4.1.1 Soiltest

Each sand specimen tested in this machine was 2.5" dia. with a cross-sectional area of 4.91 in^2. The thickness of the specimen was 1.0" and was achieved in two layers. The lower layer was 0.25" thick and the upper layer was 0.75" thick. The specimens were prepared by first weighing 34 gm (0.075 lb) of sand for the first layer and pouring it into the mold through a funnel (described in section 3.3.2). The sand was then tamped with an aluminum rod until the desired thickness was achieved. Similarly, 102 gm (0.225 lb) of sand was weighed and poured into the mold in the same way for the second layer. Some tamping was done to obtain specimens of 1.0" total thickness.

3.4.1.2 ELE

The sand specimens were 2.5" dia. with a cross-sectional area of 4.9087 in^2. The height of each specimen was 1.0". Each specimen was prepared into two layers, a lower 0.25" thick layer and an upper 0.75" thick layer. The lower layer required 32 gm (0.07 lb) of sand whereas the upper layer required about 100 gms (0.220 lb). The reason for the slight difference between the weights of sand required for this machine as compared to Soiltest machine is that the grid plates have ridges of different dimensions in each case.

3.4.1.3 Wykeham Farrance

The shear mold in this machine is square and measures 2.5" x 2.5". The specimen height was 1.25", prepared in two layers. The lower layer was 0.75" thick whereas the upper layer was 0.5" thick. In the lower layer, 126 gm (0.28 lb) of sand was poured and 83 gm (0.18 lb) of sand was poured in the top layer.

3.4.1.4 Clockhouse

The dimensions of the mold are 2.5" x 2.5" with a cross-sectional area of 6.25 in^2. Specimens of 1.5" thickness were used in this machine. The lower layer in each specimen was 1" thick and the upper layer was 0.5" thick. A weight of 169 gm (0.37 lb) of sand was

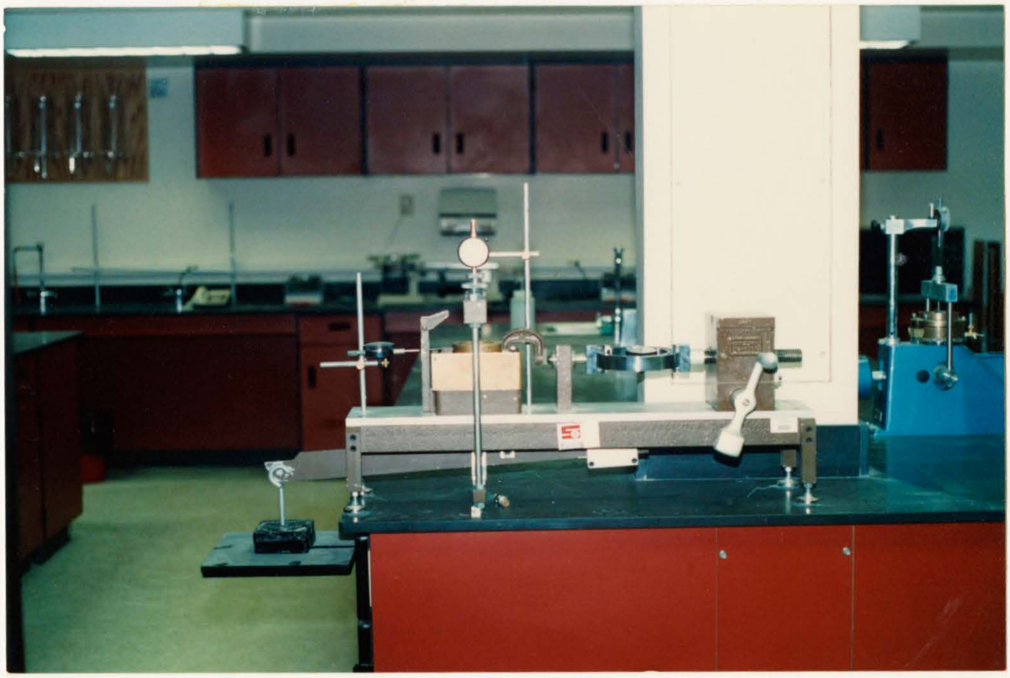


Figure 3.7 Direct shear test machine (Soiltest)

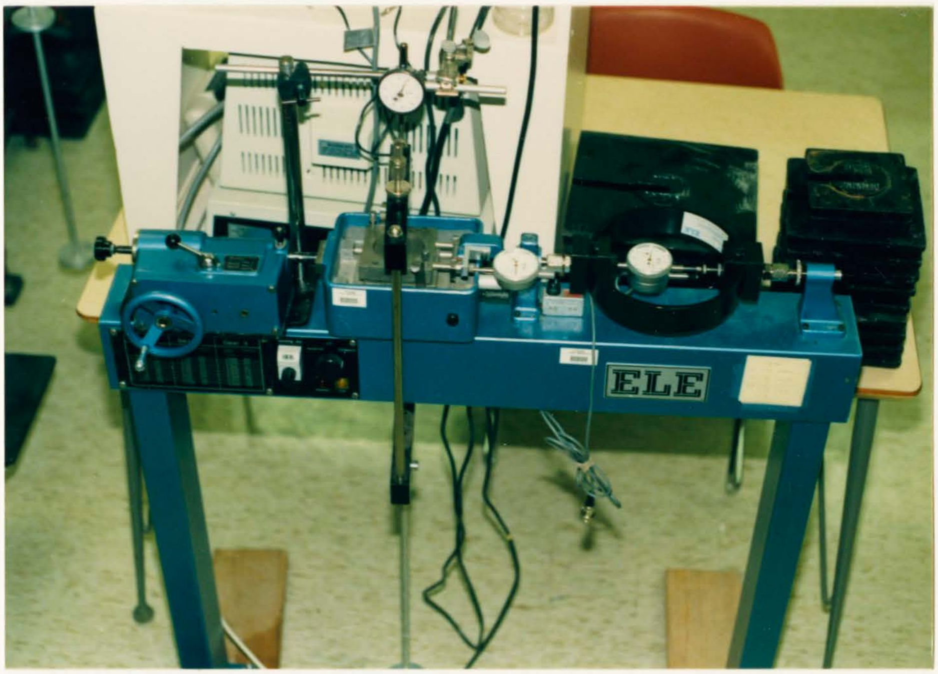


Figure 3.8 Direct shear test machine (ELE)

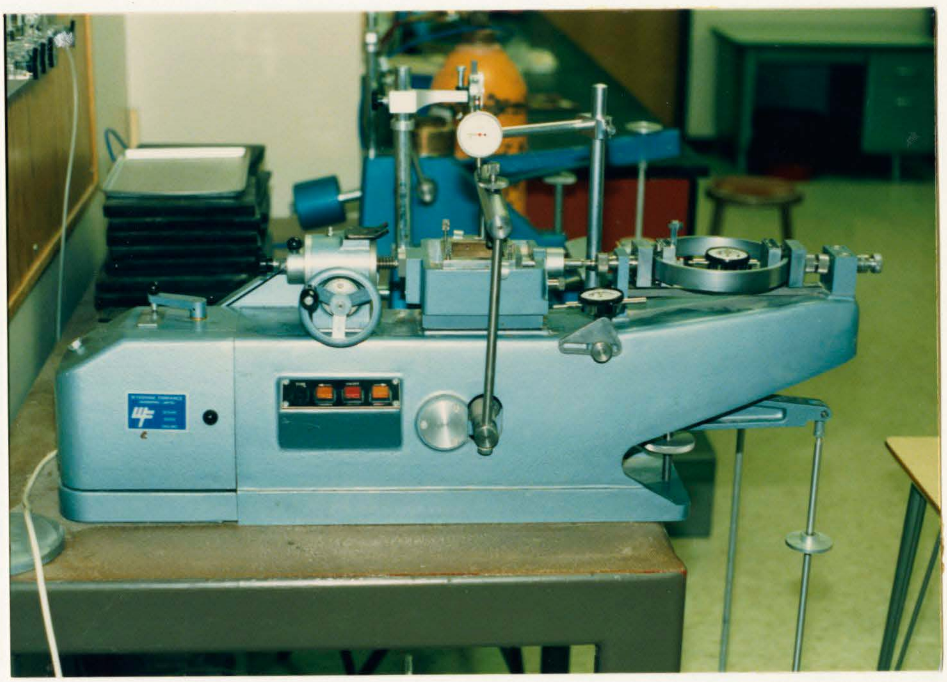


Figure 3.9 Direct shear test machine (Wykeham Farrance)

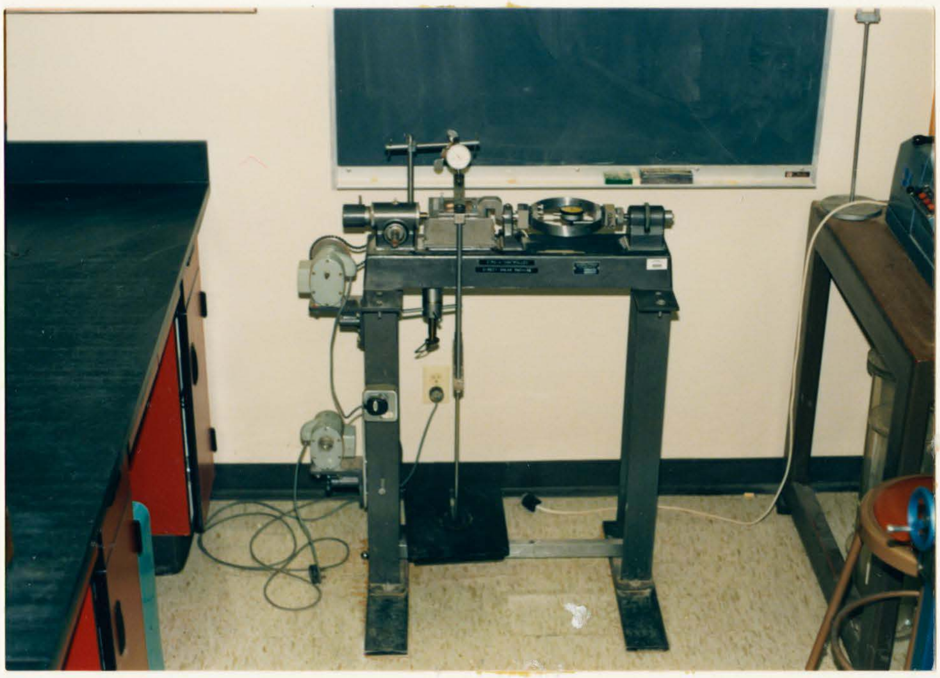


Figure 3.10 Direct shear test machine (Clockhouse)

required for the lower layer and the upper layer required 83 gm (0.18 lb) of sand. All test specimens were prepared by pouring sand through a funnel and then tamping it.

CHAPTER 4

STATISTICAL ANALYSIS

4.1 General

Statistical analysis was performed on the data obtained from the triaxial as well as direct shear tests. The direct shear data include the shear stress values corresponding to the normal stress values of 29 psi and 54 psi. To study the dispersion and distribution of the data, measures of central tendency and dispersion were calculated. A description of these analyses is presented below.

4.2 Measures of Central Tendency

Two types of measures of central tendency were calculated: the mean and median.

4.2.1 The Arithmetic Mean

The arithmetic mean is the most commonly used measure of central tendency. The arithmetic mean, $\overline{x}$ is calculated by summing all the values in the sample and then dividing the total by the number of observations in the sample. Thus for a set of n values x_1, x_2, x_3, ..., x_n, in the sample

$$\overline{x} = \frac{x_1 + x_2 + ... + x_n}{n} \qquad (4.1)$$

$$\overline{x} = \frac{\sum_{i=1}^{n} x_i}{n} \qquad (4.2)$$

where $\overline{x}$ = sample arithmetic mean and n = sample size

Three properties of the arithmetic mean are very important: (i) the sum of the deviations about the mean is zero, that is

$$\sum_{i=1}^{n} (x_i - \bar{x}) = 0 \tag{4.3}$$

(ii) the computation of the mean is based on every observation in the sample, and hence its value is greatly affected by any extreme value or values and, (iii) the sum

$$\sum_{i=1}^{n} (x_i - \theta)^2 \tag{4.4}$$

is minimum when $\theta = \bar{x}$. As can be seen from Table 4.1 thru 4.3, the mean calculated for shear stress values for triaxial as well as different machines under the same value of normal stress do not deviate much from the values of observations comprising these samples. This indicates the consistency and closeness of observed values (Berenson, 1979).

4.2.2 Median

The median is a measure of central tendency or the middle of an ordered sequence of value. Half of the observations in a set of data are below and half of the observations are above the median. The median has three interesting characteristics. First, the calculation of the median value is affected by the number of observations, not by the magnitude of any extreme(s). Second, any observation selected at random is just as likely to be above or below the median . Third, the summation of absolute differences about the median is a minimum, that is ,

$$\sum_{i=1}^{n} \left| x_i - \theta \right| \tag{4.5}$$

is minimized when θ = median. The median values for the test data are given in Tables 4.1 thru 4.3.

4.3 Measures of Dispersion

Dispersion is another characteristic which describes a set of data. Dispersion may be defined as the amount of variation, scatter, or spread in the data. Two types of measures of dispersion were calculated: the standard deviation and the average absolute deviation from the median.

4.3.1 The Standard Deviation (STD)

The standard deviation of a sample is the square root of the variance of a sample. The variance of a sample is ,

$$s^2 = \sum_{i=1}^{n} \frac{(x_i - \bar{x})^2}{n-1} \tag{4.6}$$

Thus , the standard deviation is given by

$$s = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n-1}} \tag{4.7}$$

The standard deviation is a common measure of the scatter about the mean.

4.3.2 The Average Absolute Deviation from Median (AAD)

The average absolute deviation from median (AAD) is calculated by taking the summation over the absolute differences of the median from all observations in a set of data and dividing this value by the number of observations in a sample.

$$AAD = \frac{1}{n} \sum_{i=1}^{n} |x_i - M| \tag{4.8}$$

where M = sample median

Table 4.1 summarizes the measures of location and deviation for the triaxial tests data. Tables 4.2 and 4.3 presents the same for the direct shear tests data of the four machines. Table 4.4 summarizes the results of the statistical analysis.

4.4 Regression Analysis

Regression analysis is used for the purpose of prediction. Of the many equations that can be used to predict values of one variable, y, from given values of another variable, x, simplest and most widely used is the linear equation in two unknowns, or

$$y = a + bx \tag{4.9}$$

where 'a' is the y-intercept (the value of 'y' for x=0) and 'b' is the slope of the line. Ordinarily, the values of 'a' and 'b' are estimated from given data, and once they have been determined, a value of 'x' can be substituted in the equation to calculate the corresponding predicted value of 'y'. Linear equations are useful and important not only because many relationships are actually of this form, but also because they often provide close approximations to relationships which would otherwise be difficult to describe in mathematical terms. When the data from a sample are plotted on x-y plot, a line called 'best-fit' line is drawn through all the data points. The criterion used most commonly for defining a 'best-fit' is called the method of least squares. This method requires that the line fitted to the data be such that the sum of the squares of the vertical deviations of the points from the line is a minimum.

Simple linear regression analysis along the above lines, was performed on data obtained from both triaxial and direct shear tests. For the triaxial tests, the data were plotted on a p-q diagram and a linear regression line was fitted through these points. The main purpose of this analysis was to ascertain the value of cohesion c' and friction angle ϕ' for the testing material. Regression analysis for the direct shear tests was done by plotting the normal stress values on the x-axis and the corresponding shear stress values on the y-axis. A simple regression line was then fitted through the data. The results of the regression analyses are presented in Chapter 5.

Table 4.1 Triaxial shear tests: Shear stress values corresponding to normal stresses of 29 and 54 psi

Test set #	Shear stress values for $\sigma = 29$ psi	Shear stress values for $\sigma = 54$ psi
1	28.29	52.60
2	28.84	52.88
4*	28.21	52.72
5	29.56	54.76
Mean	28.72	53.24
Median	28.56	52.80
Standard Deviation	0.61	1.02
AAD**	0.47	0.58

* The data from test set # 3 were discarded because of inconsistency with the rest of the data.

** AAD stands for Average Absolute Deviation from Median

Table 4.2 Direct shear tests: shear stress values for normal stress = 29 psi

	ST	ELE	CH	WF
	34.44	36.67	29.50	41.44
	34.02	37.08	29.60	38.72
	35.25	34.84	29.50	39.36
	35.45	36.87	28.50	40.48
	34.01	35.45	28.60	40.00
	33.82	36.87	28.70	44.48
	34.22	35.65	28.60	43.20
	33.61	35.45	28.70	40.00
	34.63	35.24	29.60	39.36
	33.21	36.06	29.50	40.96
	33.41	34.63	29.40	36.96
	34.02	35.45	29.70	36.96
	33.82	35.85	29.60	37.60
	33.41	34.84	28.60	41.50
	34.02	34.63	29.80	40.75
Mean	34.09	35.70	29.19	40.12
Median	34.02	35.45	29.50	40.00

Table 4.3 Direct shear tests: shear stress values for normal stress = 54 psi

	ST	ELE	CH	WF
	64.58	64.78	46.24	65.27
	64.58	62.64	50.40	69.00
	62.34	61.93	49.60	64.69
	62.95	61.93	48.16	70.00
	64.38	61.93	49.92	70.38
	63.15	63.56	49.44	70.00
	63.97	60.50	49.76	69.88
	62.95	62.34	45.44	67.00
	61.52	64.98	50.24	73.24
	61.11	64.58	48.48	70.25
	62.74	61.11	50.24	62.90
	64.17	60.91	51.52	74.98
	61.32	61.93	49.44	65.40
	61.52	64.58	49.76	72.74
	61.11	63.15	50.32	71.38
	61.93	64.98	48.64	67.88
	62.54	62.34	50.40	72.50
	61.73	61.73	53.92	65.14
	62.95	62.13	53.44	72.37
	61.93	61.32	56.64	74.74
Mean	62.67	62.67	50.10	69.49
Median	62.64	62.23	49.82	70.00

Table 4.4 Summary of measures of location and dispersion

DATA	MEAN	MEDIAN	STD	AAD
TR29*	28.72	28.56	0.61	0.47
TR54	53.24	52.80	1.02	0.58
EL29**	35.70	35.45	0.84	0.66
EL54***	62.67	62.23	1.43	1.12
ST29	34.09	34.02	0.64	0.45
ST54	62.67	62.64	1.17	0.97
CH29	29.19	29.50	0.50	0.41
CH54	50.10	49.82	2.48	1.60
WF29	40.12	40.00	2.12	1.59
WF54	69.49	70.00	3.51	2.77

*TR29 represents shear stress data related to triaxial tests for a normal stress of 29 psi. Similarly TR54 represents shear stress data from triaxial tests for a normal stress of 54 psi.

**EL29 represents data obtained from direct shear testing performed on ELE machine under a normal load of 29 psi. Likewise, ST, CH, WF stand for the Soiltest, Clockhouse and Wykeham Farrance direct shear testing machines, respectively.

***EL54 represents data obtained from direct shear testing performed on ELE machine under a normal load of 54 psi.

STD = Standard Deviation

AAD = Average Absolute Deviation from the Median

CHAPTER 5

PRESENTATION OF RESULTS

5.1 Triaxial Shear Tests

Five sets of triaxial shear tests were performed in all. In triaxial test sets #1, 2, and 3, three different specimens of sand were tested each for a single value of confining pressure. The data obtained through each set can be represented by three Mohr circles drawn for peak stresses corresponding to confining pressures of 20, 30 and 40 psi. Triaxial test sets # 4 and 5 were performed as multi-stage tests. In each multi-stage test, a single specimen was tested under three different confining pressures of 20, 30 and 40 psi in three stages.

Among the five sets of triaxial tests, set # 3 gave inconsistent and higher failure stresses as compared to the remaining four tests. A higher value of cohesion was also observed. Because this set of data falls so far from all others and is inconsistent with what was expected, the data from this test were not considered for evaluation and comparison of the test results. The result, however, is included in this report for the purpose of documentation. The Mohr diagrams for the five sets of triaxial tests are presented in Figures 5.1 through 5.5. A summary of the test results is presented in Table 5.1.

5.2 Direct Shear Tests

The direct shear tests were performed for normal stress of 29 and 54 psi. For each machine, twenty tests were performed under a normal stress of 54 psi and another fifteen tests were carried out under a normal stress of 29 psi. The results of direct shear testing is presented on x-y plots. A regression line was fitted through these points which represents the Mohr-Coulomb envelope. A summary of the direct shear test results is presented in

Table 5.2. The plots of these results can be seen in Figures 5.6 through 5.9. As is apparent from these plots, the data from the Soiltest and ELE show a small but comparable amount of scatter, whereas the data for Wykeham Farrance and Clockhouse show a scatter significantly wider compared to the other two machines. Regression analyses of the triaxial test data are presented in Figures in 5.10 through 5.19.

5.3 Precision Comparison of Direct Shear Test Machines

Though the basic working principle of all the four direct shear machines used for this study is essentially the same, the different structural features of these machines need to be considered when ascertaining the degree of precision that can be attached to the results generated by each of these machines. The most important structural features that seem to affect the test results are: (i) the automation or manual operation of the direct shear machine; (ii) the arrangement for the application of the horizontal stress; (iii) the arrangement (if one exists), for pinning down the top-cap to the upper half of the shear box (henceforth it will be referred to as the 'upper shear ring') thus allowing or preventing the rotation of the top-cap during the direct shear test (the rotation of the top-cap may influence the test results) and finally, (iv) the type of arrangement for the application of the normal stress on the specimen. A discussion of the accuracy and precision of each machine follows.

5.3.1 Soiltest

This machine is manually operated and does not ensure the application of a uniform or constant rate of strain. Some effort and care is required to revolve the handle with a constant speed to ensure a uniform stress application. The shear stress is applied to the upper half of the shear box which is provided with a 'swan-neck' loading yoke. This arrangement ensures the application of the horizontal load along the shear plane thereby avoiding application of a force couple. The machine allows the pinning of the top-cap to the upper shear ring which allows control of its rotation during the test. The normal load can be applied through a lever loading arm. This arrangement is not objectionable from a technical point of view but it requires some care to keep the lever arm in a horizontal position during the test to avoid the application of an eccentric normal load. One serious drawback of this machine is that the load is applied directly to the specimen while the weights are being placed, which may result in vibrations disturbing the specimen.

5.3.2 ELE

This machine is motor-driven and is strain-controlled thus ensuring a constant strain during the test. Though the top-cap cannot be pinned to the upper shear ring there is little chance of rotation because the top-cap fits tightly in the upper shear ring. The normal load can be applied through a vertical load hanger and ensures an accurate application of the load to the specimen. The arrangement for the application of horizontal load through a swan-neck loading yoke allows the load to be applied a little higher than the shear plane level. This may allow the application of a small moment on the shear plane.

5.3.3 Wykeham Farrance

This machine has some disadvantages. First, the swan-neck loading arrangement is missing. Also the horizontal rod connecting the proving ring and the shear box passes through a bearing that uses friction which may contribute to the horizontal load. Another disadvantage is the lack of the arrangement for pinning the top-cap to the upper shear ring and hence rotation of the same may occur during the test. A subtle advantage of this machine over the others, however, is that this machine is provided with an arrangement that allows the loading lever arm to rest on a beam support jack while weights are being loaded to it. This facility ensures that no load is applied to the sample vertically until the beam support jack is released with the weight on the hanger; neither does the beam without weights exert any load on the sample. The machine also has the advantage of being motor-driven and strain-controlled.

5.3.4 Clockhouse

This machine has these advantages: (i) it is motor-driven ; (ii) the normal load can be applied through a vertical hanger to a certain limit beyond which a loading lever arm must be used, and (iii) the horizontal stress is balanced through a swan-neck loading yoke. The disadvantages of this machine include: the absence of an arrangement for fixing the top-cap to the upper shear ring and the friction that the bearing may exert on the horizontal rod that applies the load to the shear box which may contribute to the shear stress.

Table 5.1 Summary of triaxial tests results

Test No.	Dry Density γ_d (lb/ft^3)	Relative Density γ_R (%)	Cohesion c' (lb/ft^2)	Angle of Internal Friction ϕ' (Degrees)
1	105.3	80.0	0.093	44.5
	105.5	80.5		
	105.2	80.0		
2	106.0	82.0	0.155	44.2
	105.3	80.0		
	105.5	80.5		
3	105.3	80.0	1.183	43.5
	105.2	80.0		
	105.4	80.5		
4	105.4	80.5	-0.04	44.4
5	105.7	81.0	0.055	45.3

Table 5.2 Summary of direct shear tests results

Make of Test Machine	Dry Density γ_d (lb/ft^3)	Relative Density γ_R (%)	Cohesion c' (lb/ft^2)	Angle of Int. Friction ϕ' (degrees)
Soiltest	105.0 - 106.0	79.0 - 82.0	0.64	48.9
ELE	105.0 - 106.0	79.0 - 82.0	4.41	47.2
Wykeham Farrance	105.0 - 106.0	79.0 - 82.0	5.97	49.5
Clockhouse	105.0 - 106.0	79.0 - 82.0	3.92	40.5

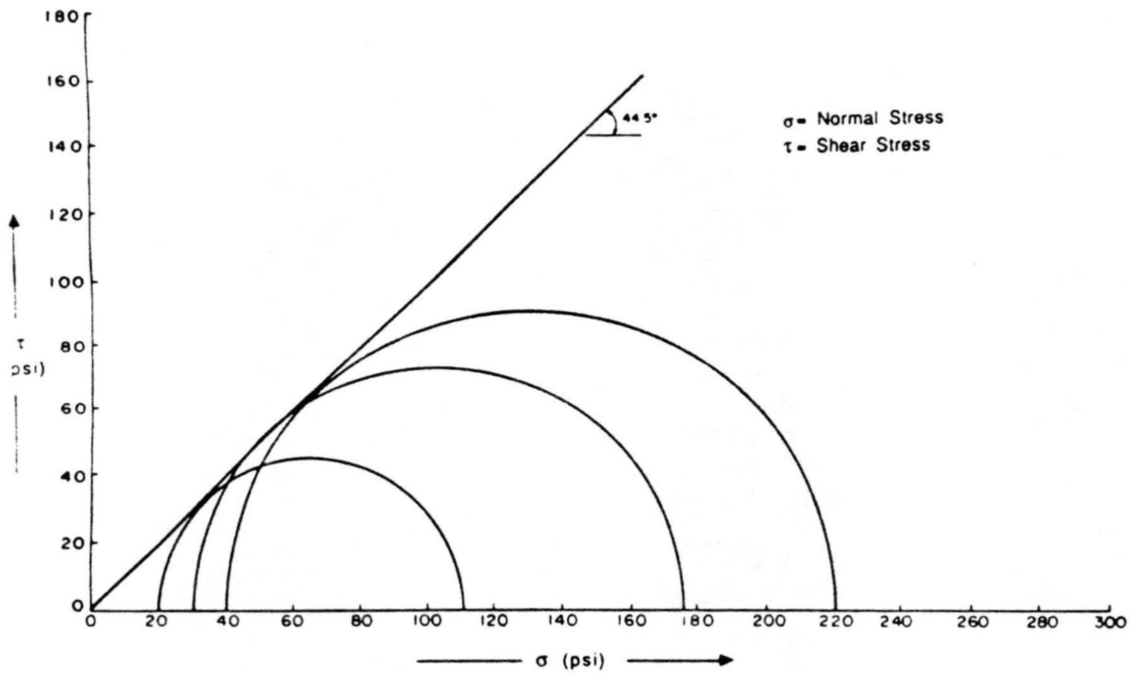


Figure 5.1 Results of triaxial test set #1

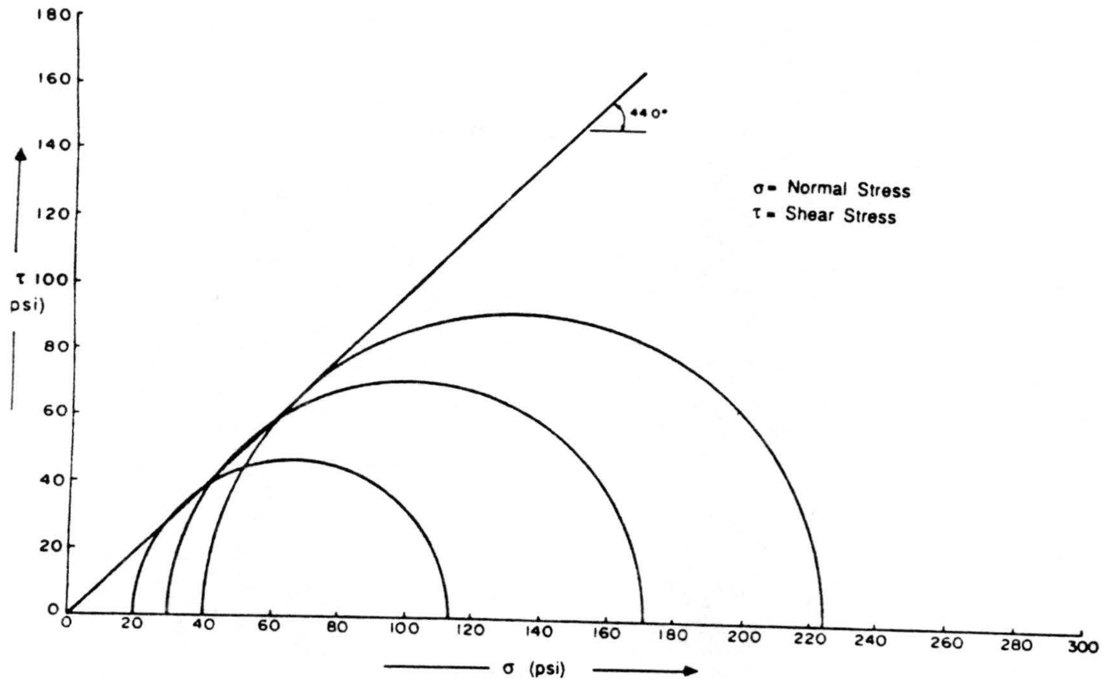


Figure 5.2 Results of triaxial test set #2

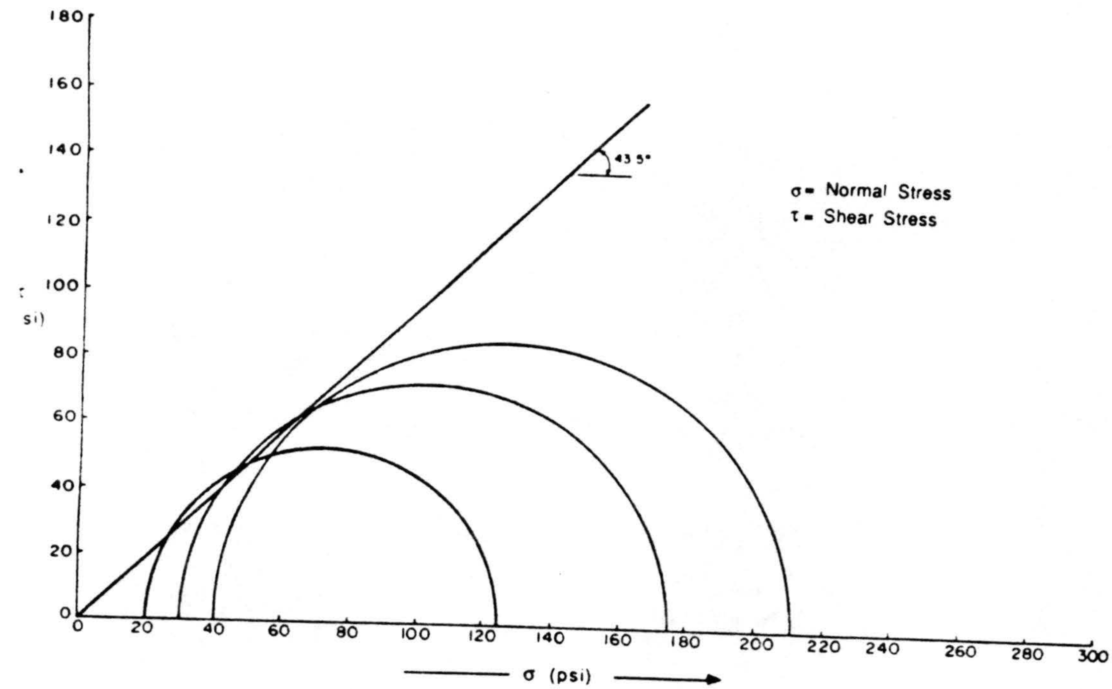


Figure 5.3 Results of triaxial test set #3

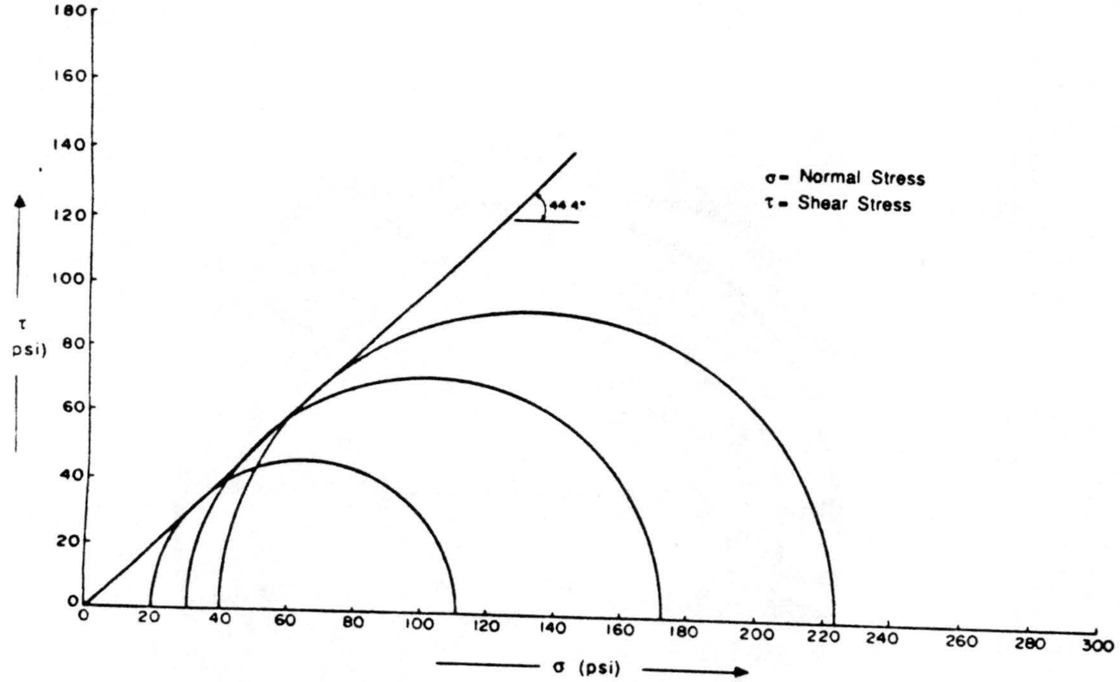


Figure 5.4 Results of triaxial test set #4 (multi-stage)

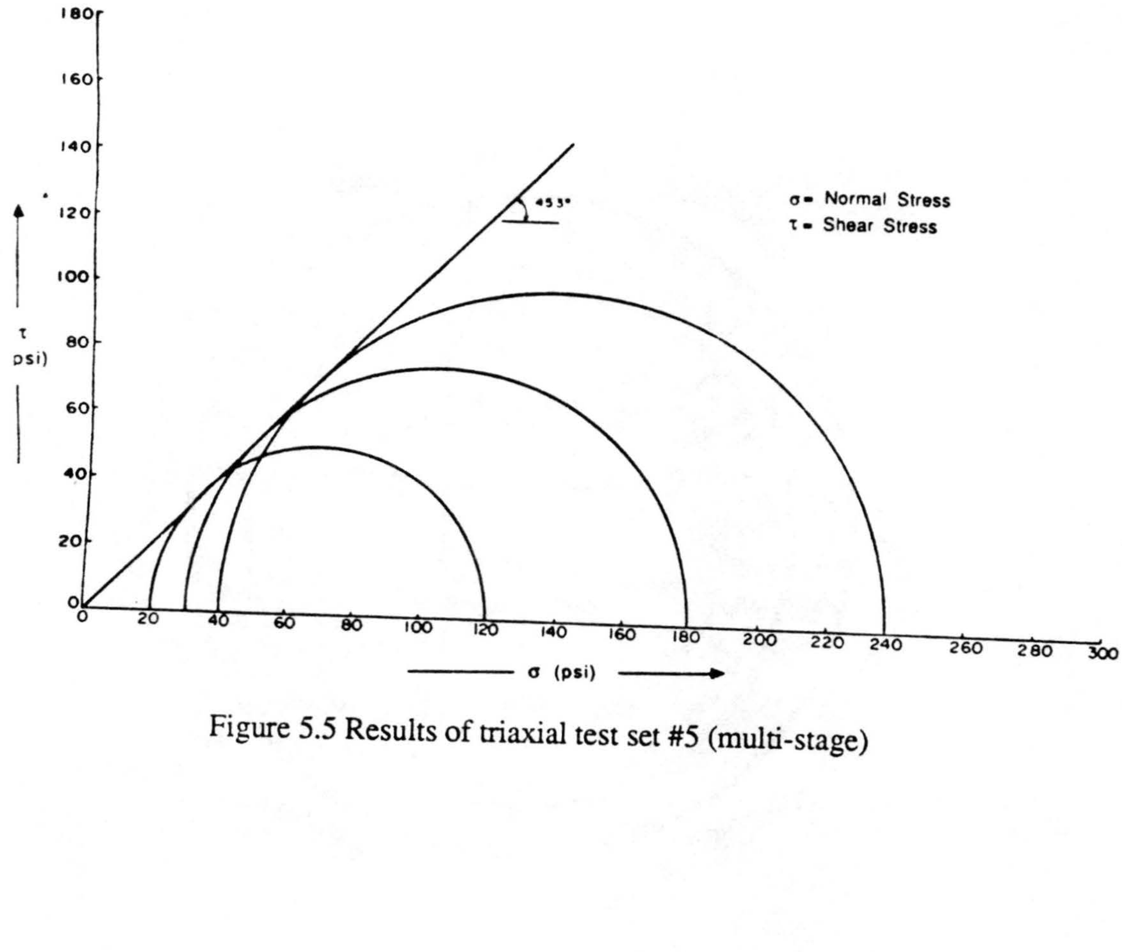


Figure 5.5 Results of triaxial test set #5 (multi-stage)

Fig 5.6 Normal stress vs shear stress
(Soiltest)

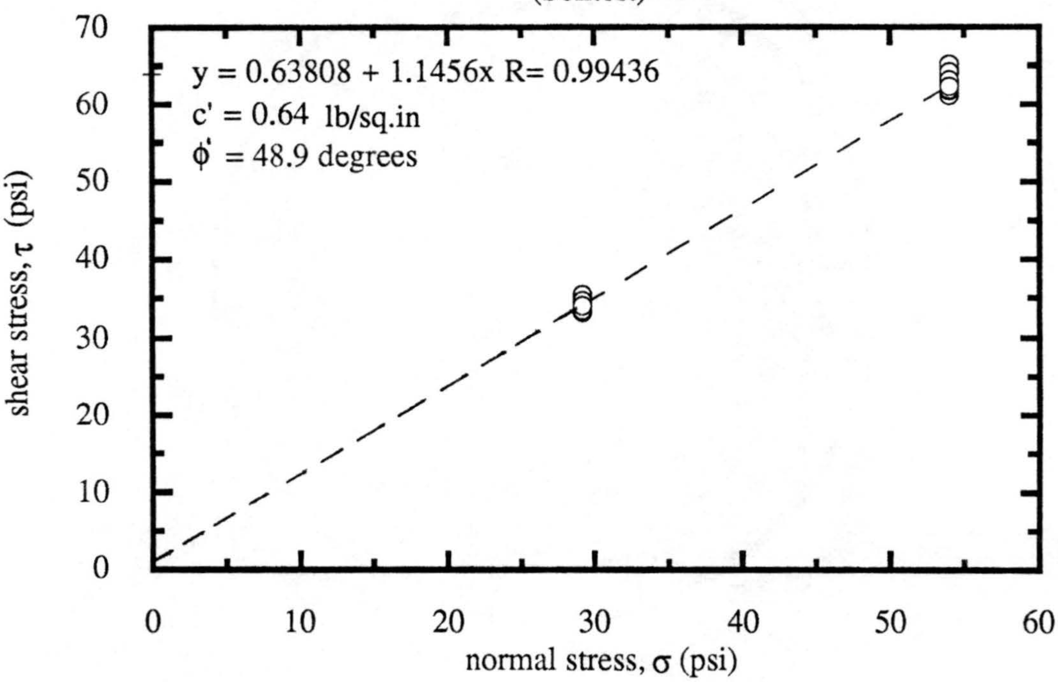


Fig 5.7 Normal stress vs shear stress
(ELE)

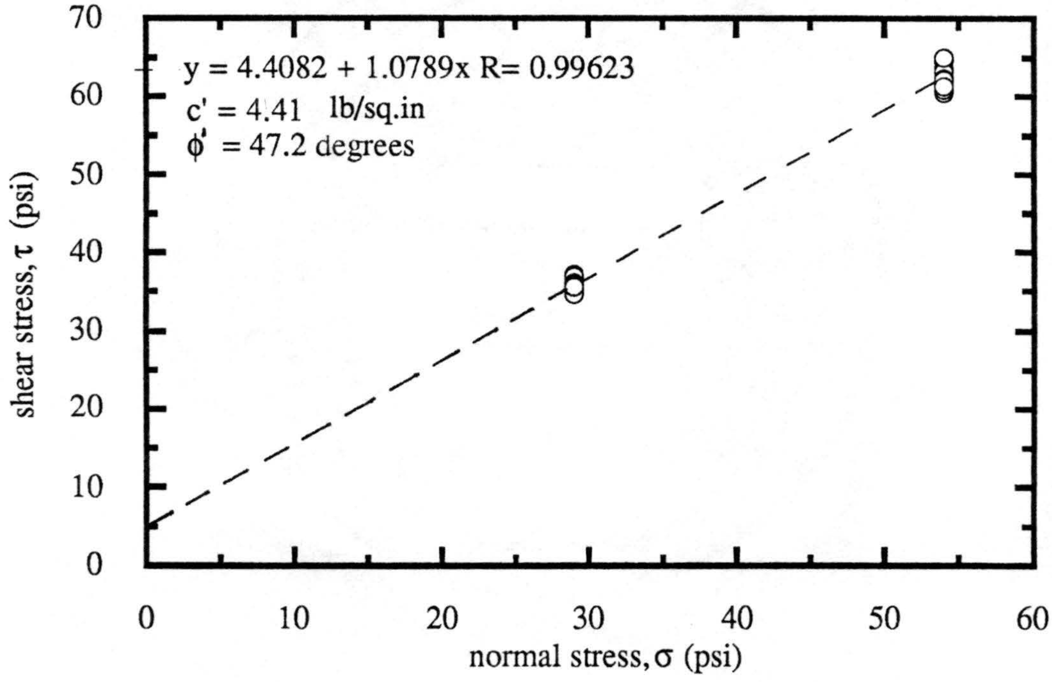


Fig 5.8 Normal stress vs shear stress
(Wykeham Farrance)

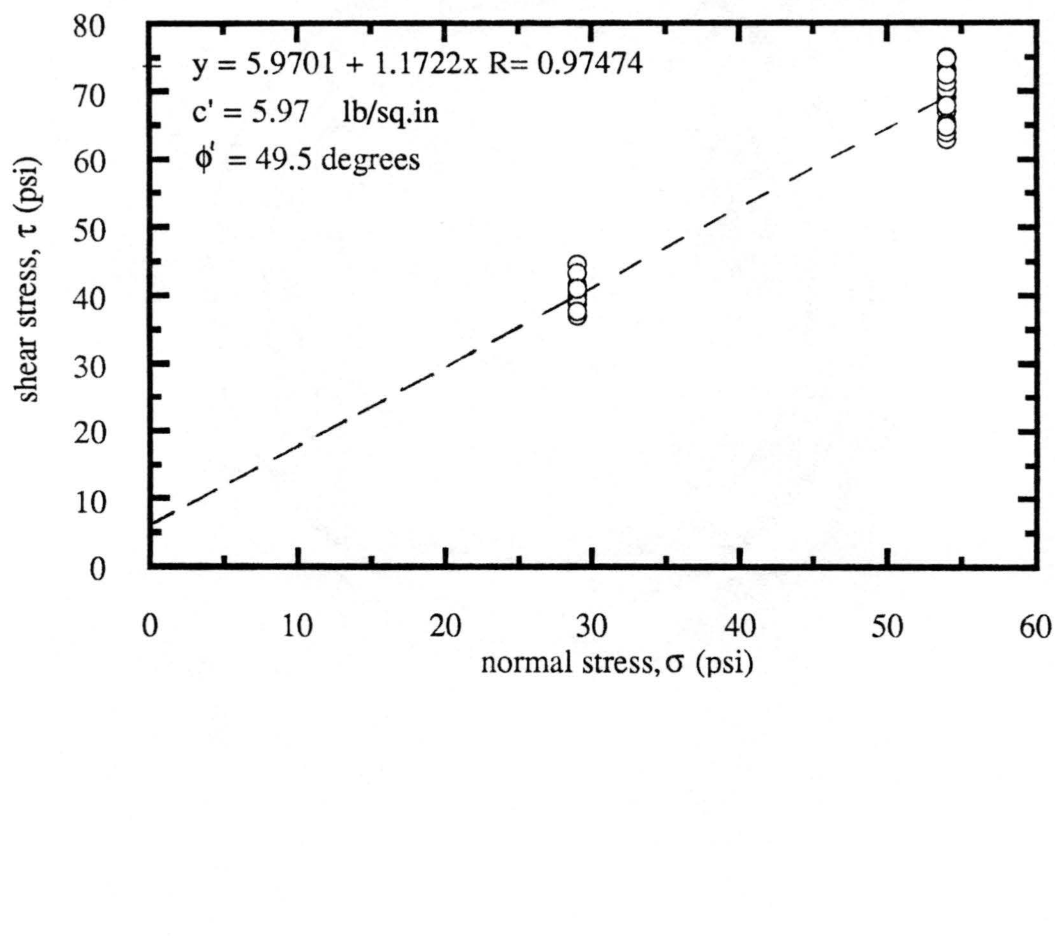


Fig 5.9 Normal stress vs shear stress
(Clockhouse)

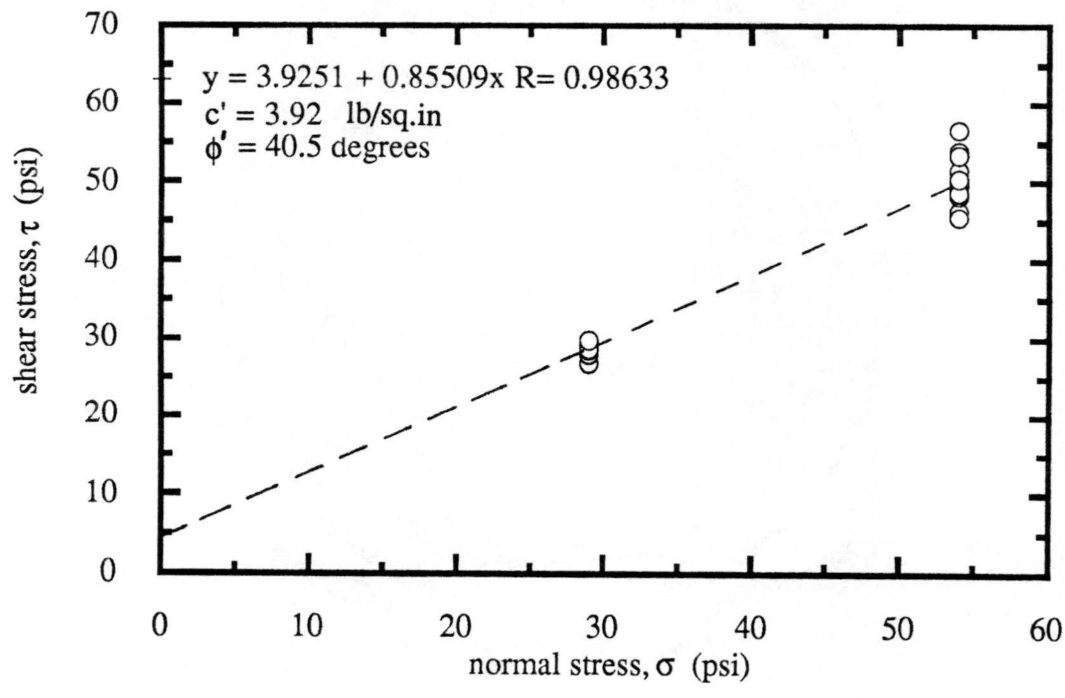


Fig 5.10 Triaxial test set # 1
(Regression analysis)

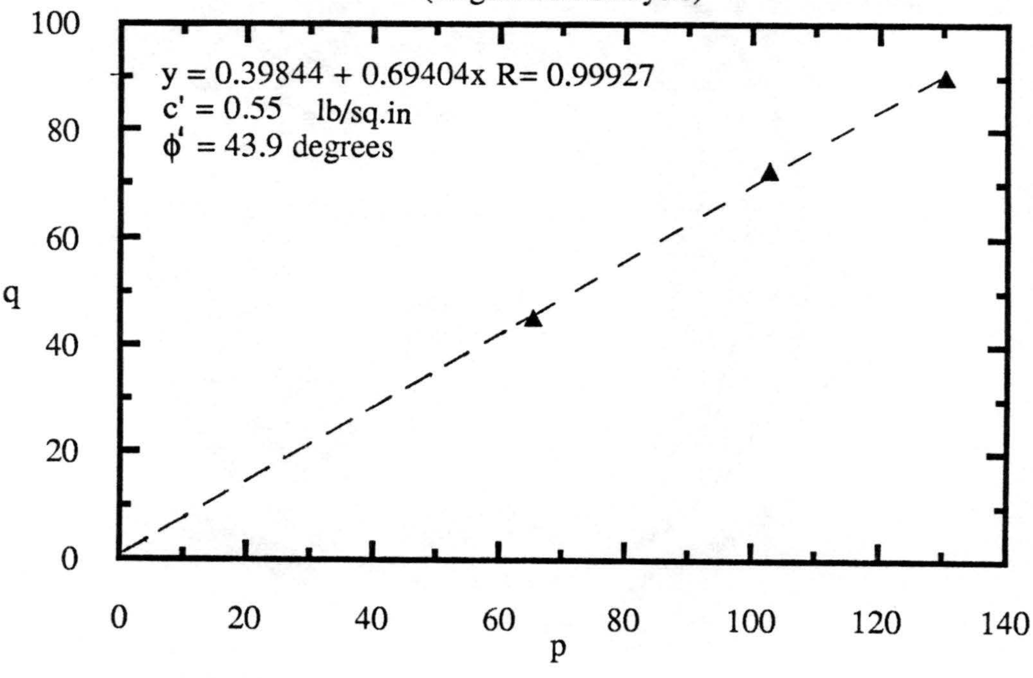


Fig 5.11 Triaxial compression test set # 2
(Regression analysis)

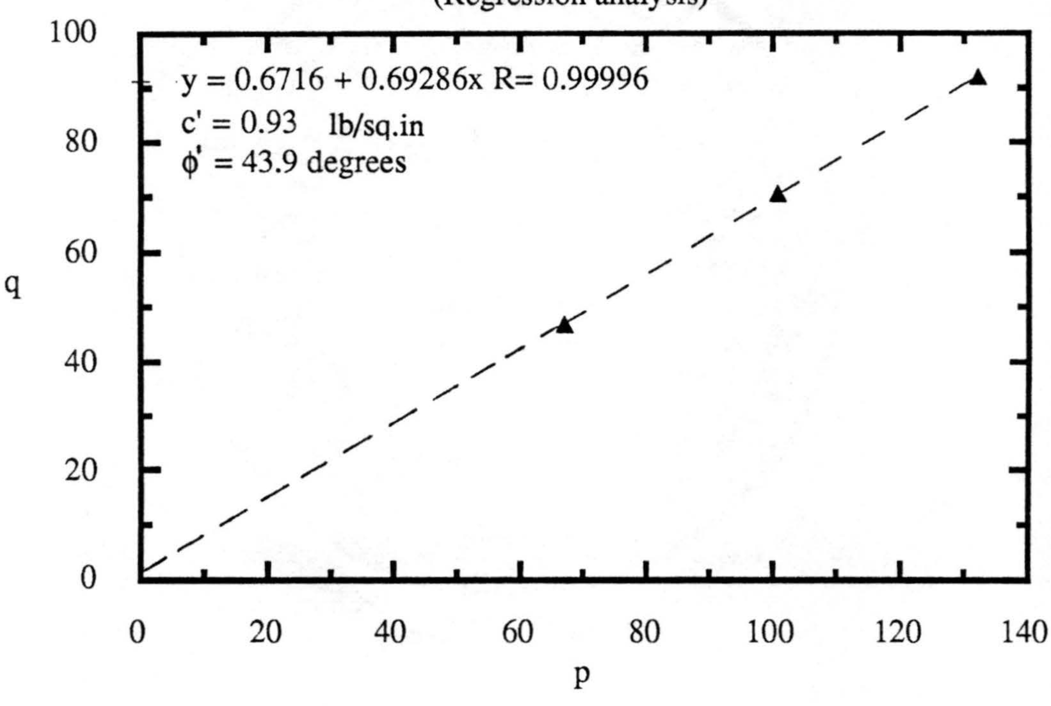


Fig 5.12 Triaxial compression test set # 3
(Regression analysis)

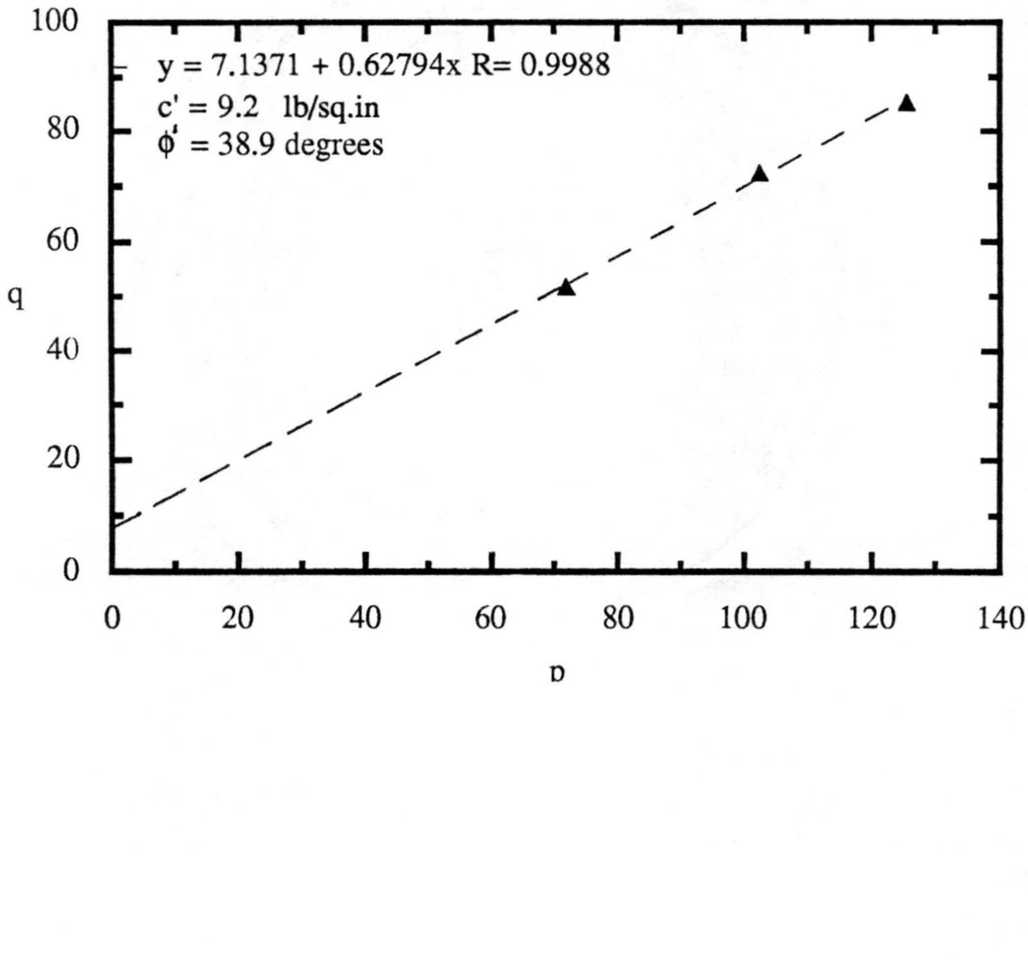


Fig 5.13 Triaxial compression test set #4
(Regression analysis)

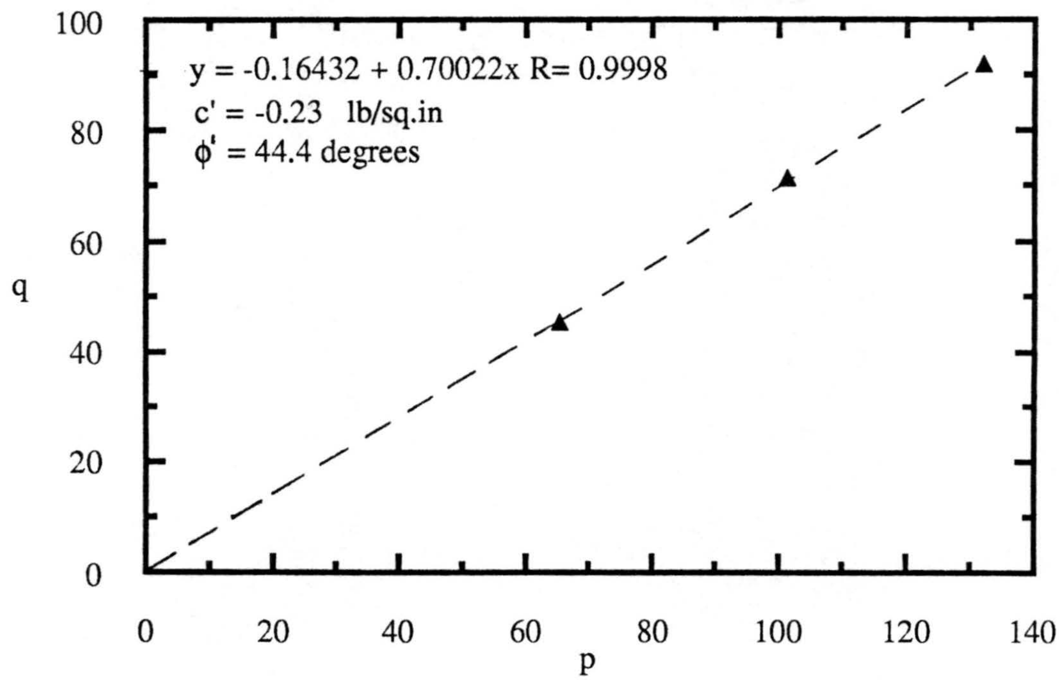


Fig 5.14 Triaxial compression test set # 5
(Regression analysis)

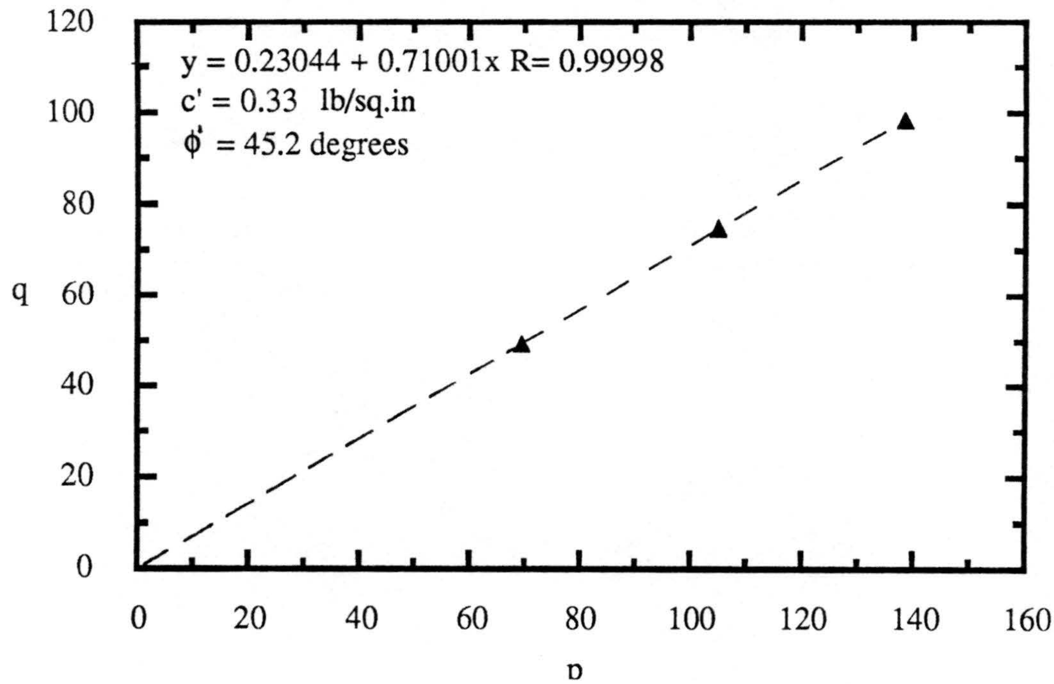


Fig 5.15 Triaxial compression test sets #1,2
(Regression analysis)

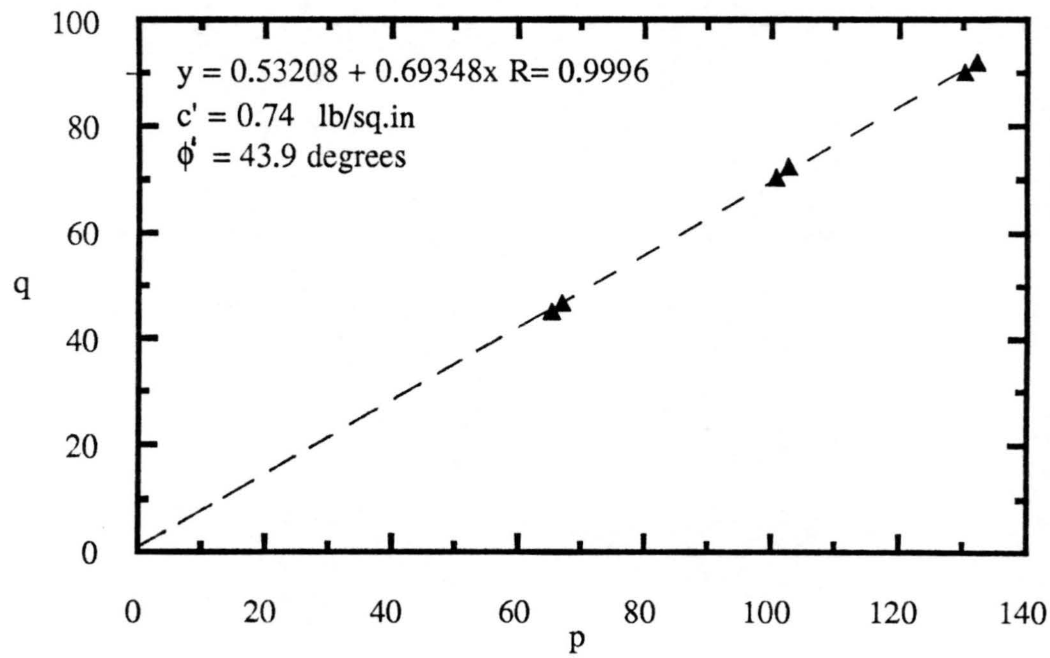


Fig 5.16 Triaxial compression test sets # 4,5
(Regression analysis)

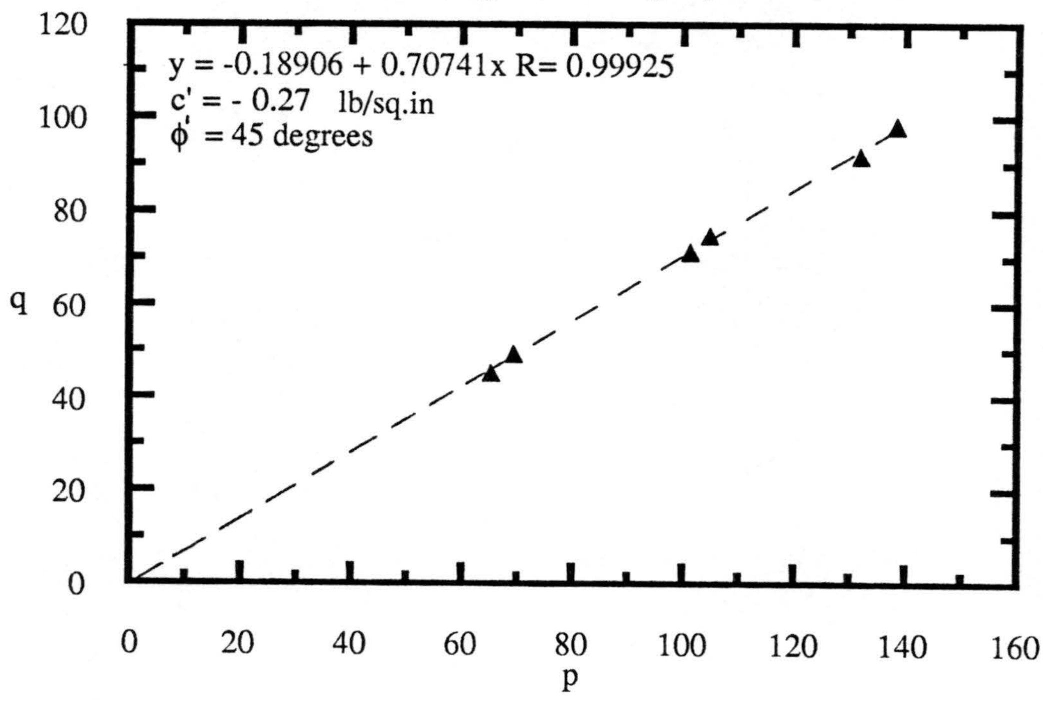


Fig 5.17 Triaxial compression test sets # 2,4,5
(Regression analysis)

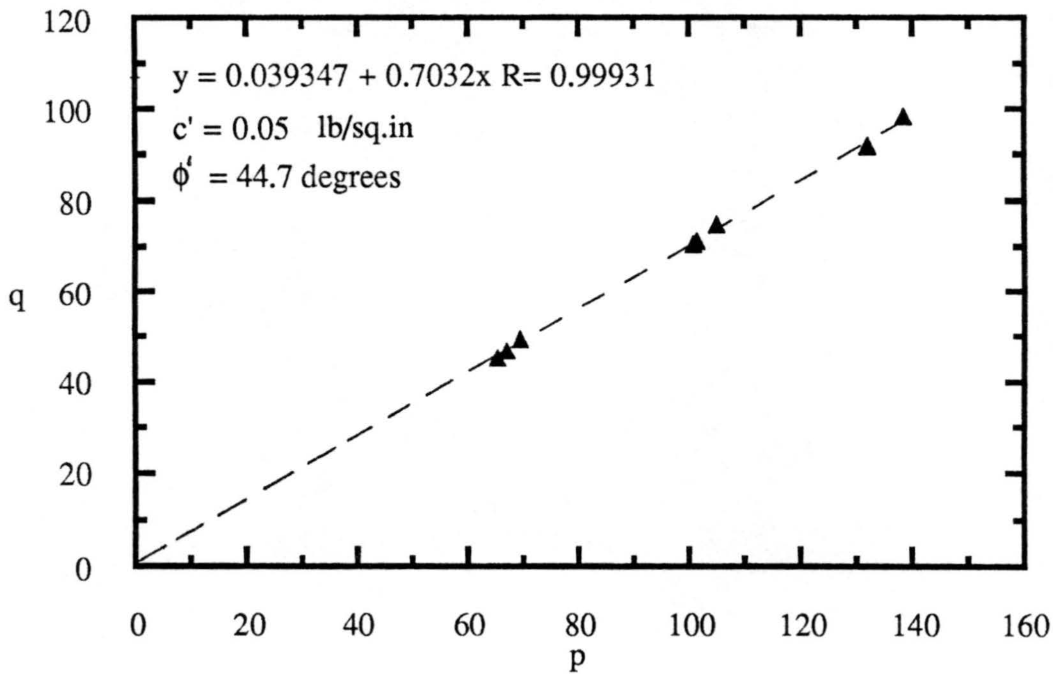


Fig 5.18 Triaxial compression test sets # 1,4,5
(Regression analysis)

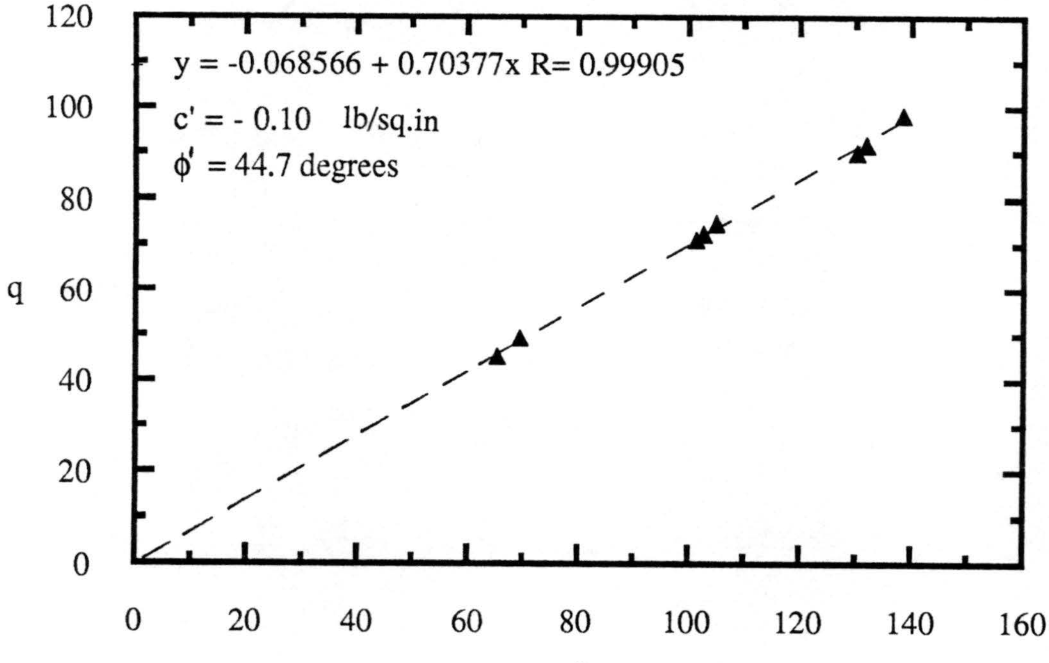
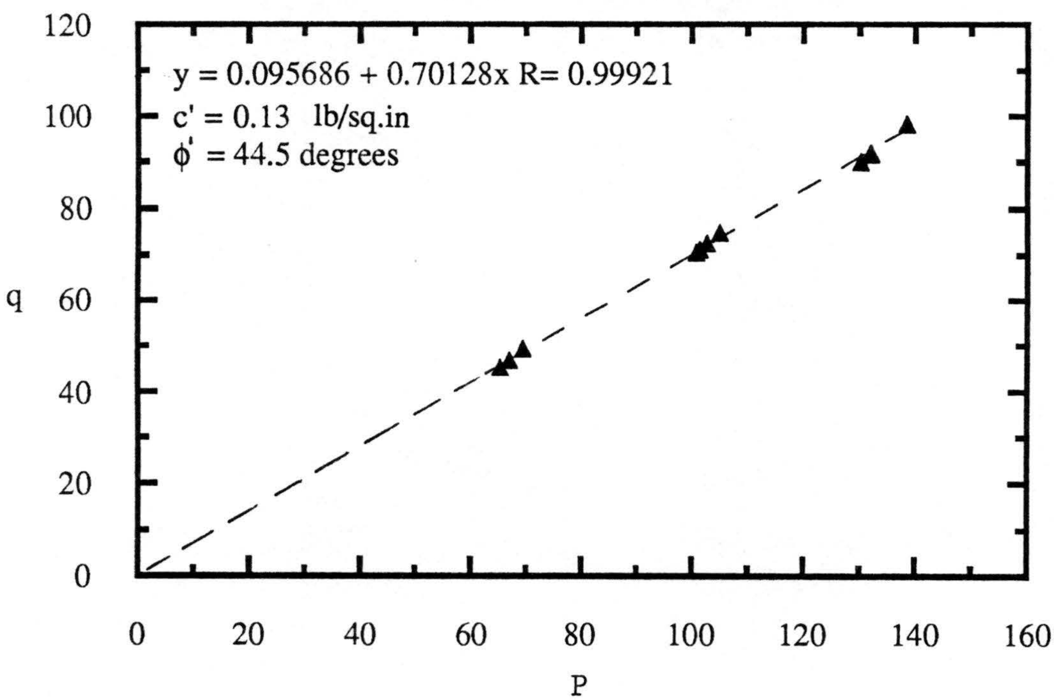


Fig 5.19 Triaxial compression test sets # 1,2,4,5
(Regression analysis)



CHAPTER 6

DISCUSSION OF RESULTS

The results of triaxial and direct shear tests were presented in Chapter 5. These results will be analysed in detail in this chapter with emphasis on the following:

- The values of c' and ϕ' obtained from triaxial tests.
- The value of c' and ϕ' from the direct shear tests, and their comparison with those from triaxial tests.
- The values of c' and ϕ' from direct shear tests on four machines and their comparison with each other.

6.1 c' and ϕ' Values from Triaxial Tests

The triaxial test results can be presented in two forms: (i) by plotting the Mohr circles and drawing a line tangent to these, giving the Mohr-Coulomb envelope defined by the equation

$$\tau = c' + \sigma \tan \phi' \qquad (6.1)$$

and (ii) by plotting the Mohr circles and drawing a line connecting the points of maximum shear stress. This is called a p-q plot and gives the K_f -line represented by the following relationship (Lambe, 1969)

$$q_f = a + p_f \tan\alpha \qquad (6.2)$$

The values of friction angle and cohesion can then be computed from the following relationships:

$$\sin \phi = \tan\alpha \qquad (6.3)$$

and $$c = a / \cos \phi \tag{6.4}$$

The triaxial test data have been presented in both formats. In the first mode the data were presented in Figures 5.1 through 5.5. A value of ϕ' ranging between 44.2 and 45.3 degrees was obtained through test sets # 1, 2, 4, and 5. These tests also gave a value of cohesion c' that ranged between -0.04 and 0.15 lb/ft^2. As was already pointed out in chapter 5, the results of test set # 3 are not in line with those of the remaining tests in that the friction angle ϕ' was found to be lower and cohesion higher as compared to the corresponding values from the rest of the test sets. This test set was discarded as an 'outlier' .

Regression analyses of the triaxial tests were presented in Figures 5.10 thru 5.19 using p-q plots. Figure 5.19 presents the regression analysis of the triaxial test sets # 1, 2, 4, 5 which gives a value of ϕ' equal to 44.5 degrees and cohesion c' equal to 0.13 lb/ft^2. Since these values of friction angle and cohesion were obtained from the regression analysis of all the bonafide triaxial tests, these will be used as reference values (benchmarks) with which the direct shear test results will be compared.

The important factors that affect the c' and ϕ' values of a soil are the void ratio or relative density, particle shape, grain size distribution, particle surface roughness, the degree of saturation, intermediate principal stress, particle size, and over-consolidation or pre-stress. However, void ratio, related to the density of the sand, is the most important factor that influences the strength of sands. As a general rule, the higher the relative density (or the lower the void ratio), the higher the shear strength (Holtz & Kovacs, 1981).

A. Casagrande tried to assign values of angle of internal friction to different soils on the basis of the above mentioned physical characteristics. Table 6.1 summarizes his findings (Holtz & Kovacs, 1981). In comparison with this Table and keeping in view the physical characteristics such as void ratio, particle size and uniformity coefficient, C_u, of the test material for this study and the fact that the sand was clean containing no fines, a

Table 6.1 Angle of Internal Friction of Cohesionless Soils
(From A. Casagrande)

No.	General Description	Grain Shape	D_{10}	C_u	Loose		Dense	
				(mm)	e	ϕ	e	ϕ
1	Ottawa standard sand	Well rounded	0.56	1.2	0.70	28	0.53	35
2	Sand from St. Peter sandstone	Rounded	0.16	1.7	0.69	31	0.47	37
3	Beach sand from Plymouth, MA	Rounded	0.18	1.5	0.89	29	-	-
4	Silty sand from Franklin Falls Dam site, NH	Subrounded	0.03	2.1	0.85	33	0.65	37
5	Silty sand from vicinity of John Martin Dam, CO	Subangular to subrounded	0.04	4.1	0.65	36	0.45	40
6	Slightly silty sand from the shoulders of Ft. Peck Dam, MT	Subangular to subrounded	0.13	1.8	0.84	34	0.54	42
7	Screened glacial sand, Manchester, NH	Subangular	0.22	1.4	0.85	33	0.60	43
8	Sand from beach of hydraulic fill dam, Quabbin Project, MA	Subangular	0.07	2.7	0.81	35	0.54	46
9	Artificial, well-graded mixture of gravel with sands No.7 and No.3	Subrounded to subangular	0.16	68	0.41	42	0.12	57
10	Sand for Great Salt Lake fill (dust gritty)	Angular	0.07	4.5	0.82	38	0.53	47
11	Well-graded, compacted crushed rock	Angular	-	-	-	-	0.18	60

value of ϕ' ranging between 44.2 and 45.3 degrees as obtained from the triaxial tests is reasonable.

Schmartman presented a chart (Figure 6.1) that determines friction angles for sands on the basis of relative densities (Merrit, 1983 ; Hunt, 1984). As can be seen from this chart, uniform sands can have a ϕ' value equal to 42 to 43 degrees for a relative density range of 79-82 %. Since the chart gives only empirical values of friction angle for various materials without any mention of the physical properties, the slightly higher value of ϕ' given by the triaxial tests than those given in Figure 6.1 is reasonable.

6.2 c' and ϕ' Values from Direct Shear Tests

The data from the direct shear tests can be presented in the following two ways. The first is by plotting the values of the shear stress against the corresponding values of normal stress on a τ vs σ plot and drawing a single best fit line (linear regression line) through these points. This 'best fit' line represents the Mohr-Coulomb failure envelope that gives only one value each of c' and ϕ'. These values for shear strength parameters are average representative values for the silica sand obtained from a particular machine.

The second method is by plotting the values of shear stress vs normal stress and drawing individual lines of failure. This generates possible Mohr-Coulomb failure lines and a range of ϕ' and c' values.

Both of these modes of presenting results are significant and were used for comparing the results of the direct shear tests with the benchmarks.

The results of the direct shear tests are in Table 5.2. Figures 5.6 through 5.9 present the regression analyses of the data obtained individually from each of the four direct shear test machines. The triaxial test results have been plotted on a normal versus shear stress plot in Figure 6.2. Figures 6.3 thru 6.6 present the results of the direct shear tests of each machine superimposed on Figure 6.2 for the purpose of comparison.

6.2.1 Soiltest

The plot of the regression analysis can be seen in Figure 5.6. The value of c' was found to be 0.64 lb/ft^2 and ϕ' equal to 48.9 degrees. Compared to the benchmarks, c' is

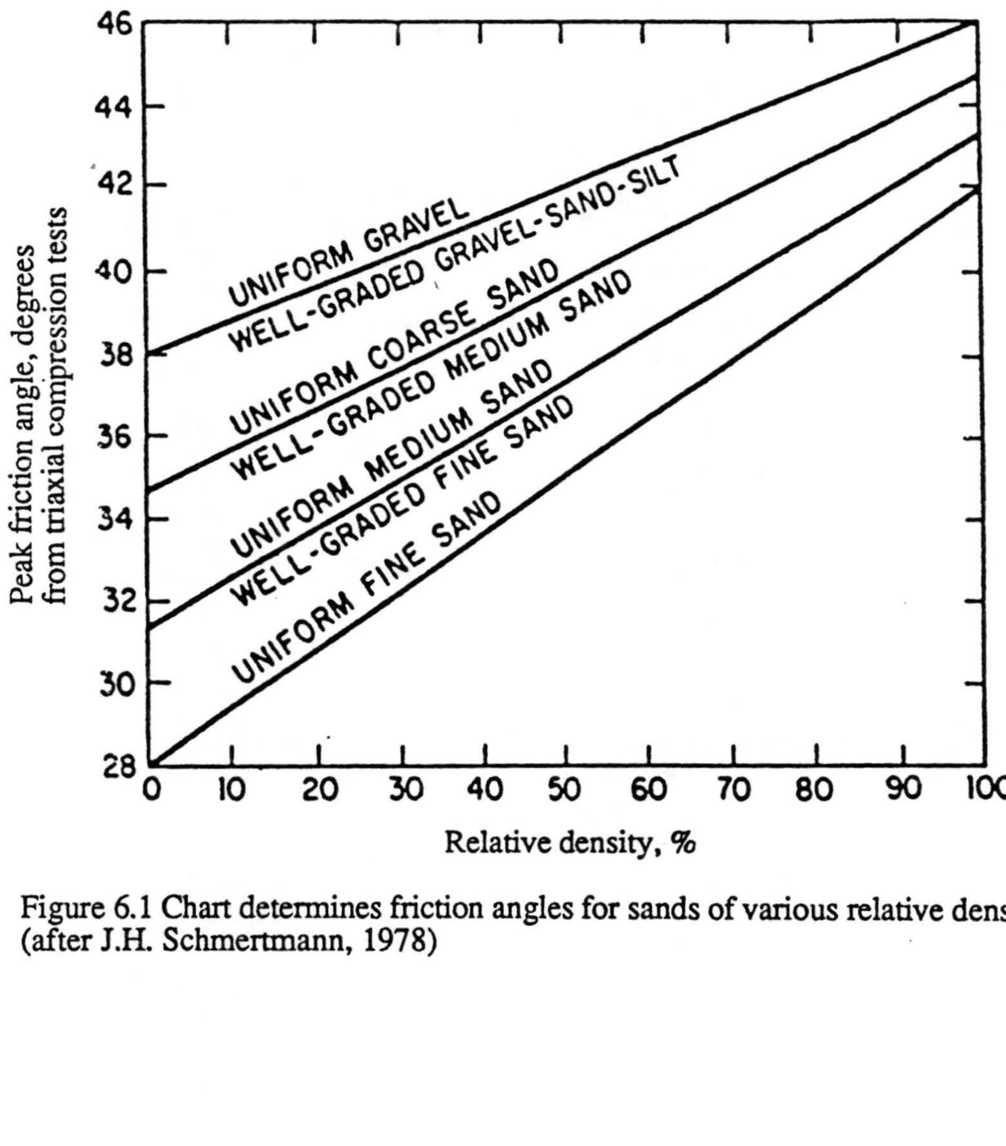


Figure 6.1 Chart determines friction angles for sands of various relative densities (after J.H. Schmertmann, 1978)

slightly higher but ϕ' is higher by as much as 4.4 degrees. A comparison with the triaxial test results is shown in Figure 6.3. This figure shows that the shear strength from the Soiltest is higher than that from triaxial tests.

6.2.2 ELE

The results are shown in Figure 5.7. The Mohr-Coulomb envelope gives a value of cohesion equal to 4.41 lb/ft^2 and the value of friction angle equal to 47.2 degrees. The cohesion c' is significantly higher than the benchmark value. The value of ϕ' is also higher than the benchmark value by 2.7 degrees. Figure 6.4 presents the Mohr-Coulomb envelopes as obtained from the direct shear tests on this machine and the triaxial tests. The shear strength from this machine is higher than that of the triaxial tests.

6.2.3 Wykeham Farrance

The results from this machine are shown in Figure 5.8. The c' value of 5.97 lb/ft^2 and ϕ' equal to 49.5 degrees are both significantly higher than the benchmark values. These values are the highest among the results of this study and have the greatest difference as compared to the benchmarks. Figure 6.5 shows the comparison of the Mohr-Coulomb envelopes. This machine also gives higher shear strength as compared to the triaxial tests.

6.2.4 Clockhouse

The results of testing on this machine are shown in Figure 5.9. The value of cohesion c' was found to be 3.9 lb/ft^2 and a friction angle equal to 40.5 degrees. In this case, the cohesion c' is higher and ϕ' is significantly lower than the benchmark values. Figure 6.6 presents a comparison of the two Mohr-Coulomb envelopes. The shear strength from this machine is higher from the triaxial tests for small values of the normal stress and it decreases as the normal stress increases.

6.3 Comparison with Taylor's Work

The findings of Taylor's study (section 2.7) indicate that the direct shear tests give a higher value of shear strength particularly the friction angle as compared to the triaxial tests. As was seen in the preceeding sections, the outcome of this study proves to be in

Figure 6.2 Normal stress vs shear stress
(Triaxial compression test sets # 1,2,4,5)

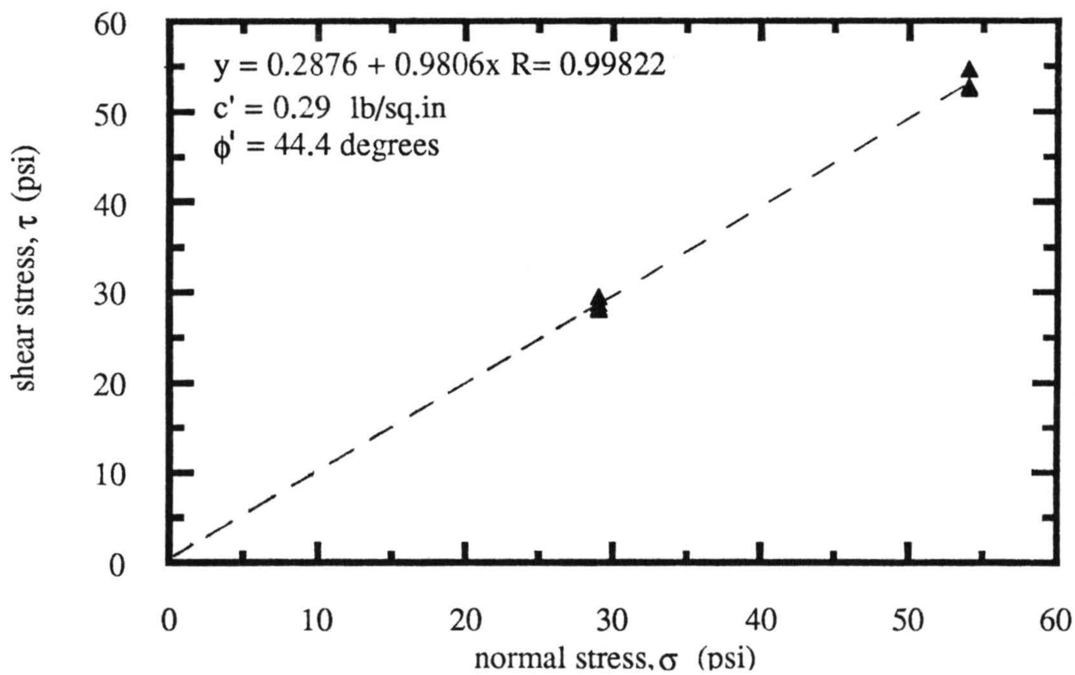


Fig 6.3 Comparison of triaxial and direct shear test data (Soiltest)

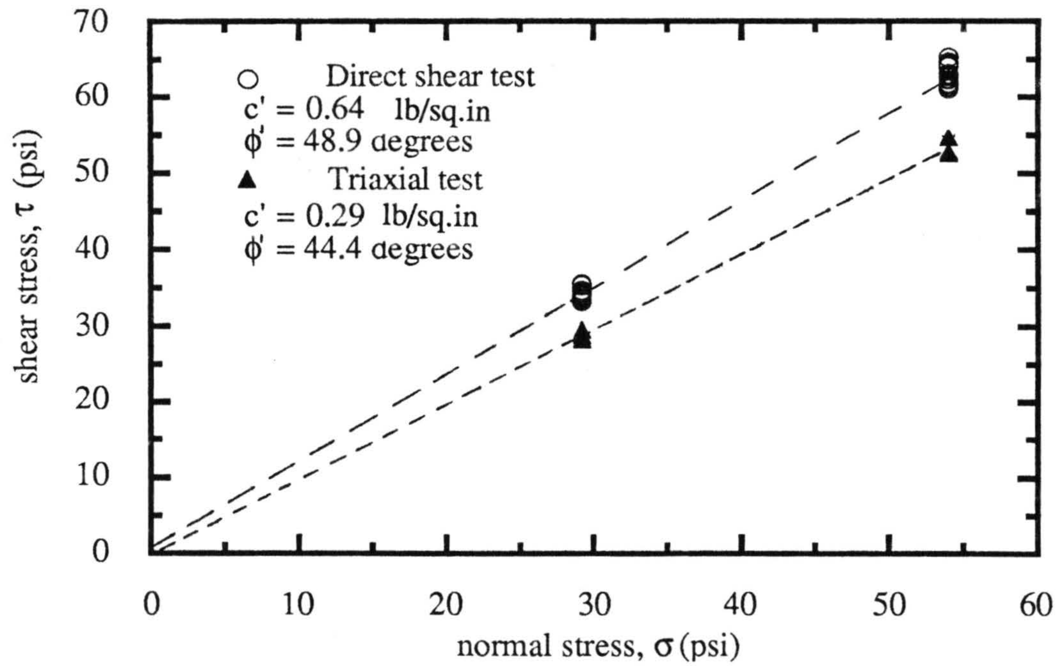


Figure 6.4 Comparison of triaxial and direct shear test data (ELE)

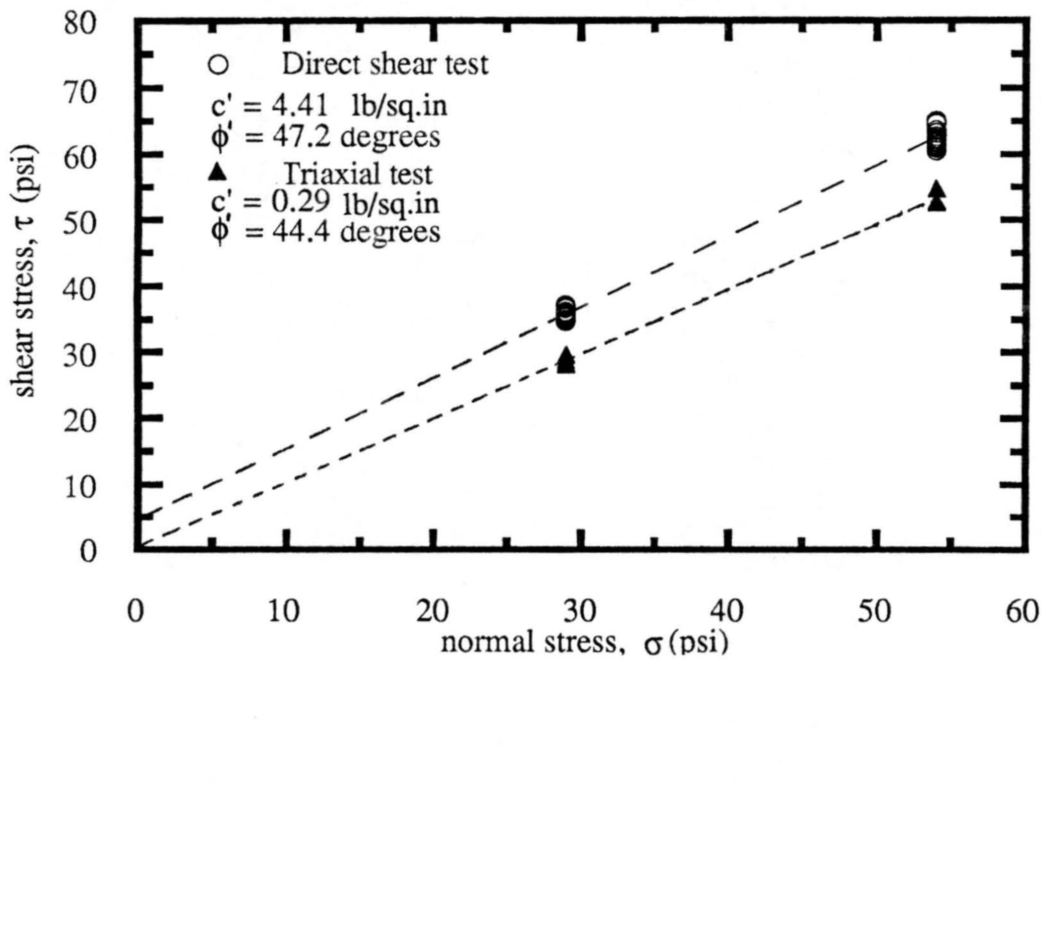


Fig 6.5 Comparison of triaxial and direct shear test data (Wykeham Farrance)

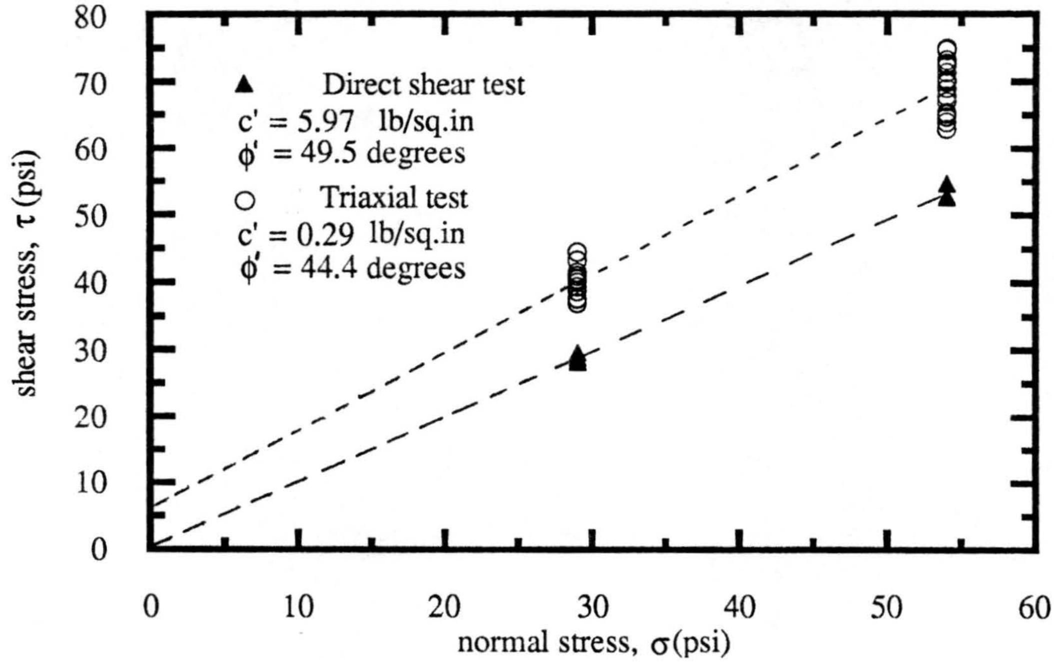
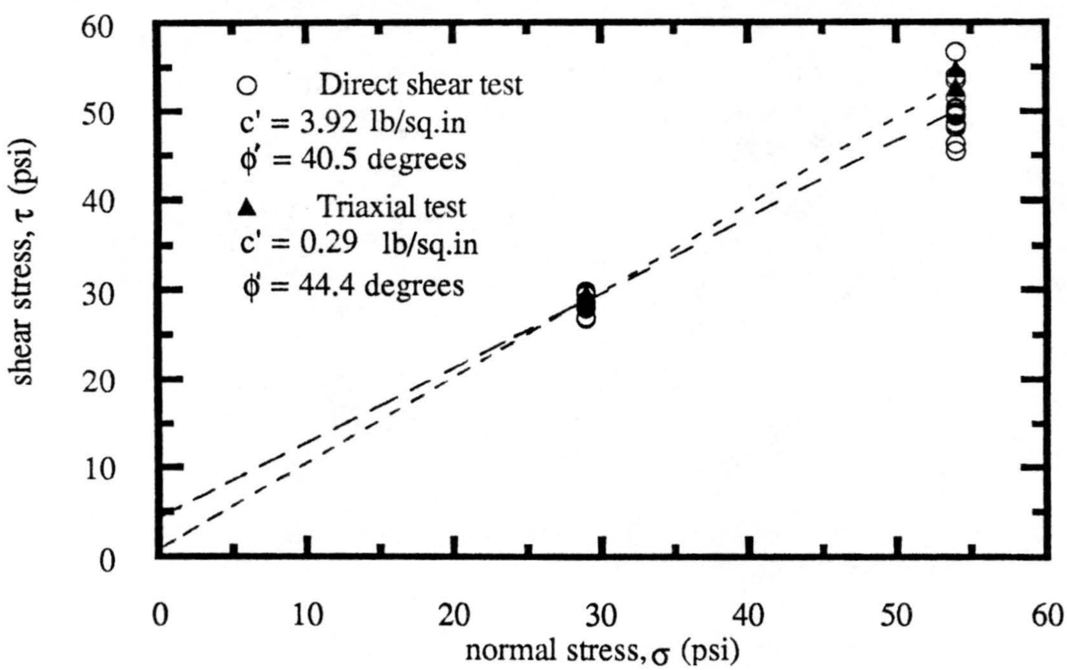


Fig 6.6 Comparison of triaxial and direct shear test data (Clockhouse)



line with the findings of Taylor's work except in the case of Clockhouse, which gives a lower value of friction angle.

6.4 Relationship Between Triaxial and Direct Shear Test Results

The findings of this study indicate the following:-

- The value of cohesion c' from direct shear tests was found to be generally higher than that of the triaxial tests.
- The value of friction angle ϕ ' is higher in case of three out of four machines
- The shear strength value from the Clockhouse machine was greater than the triaxial tests up to a certain limit of normal stress beyond which it decreased.

An empirical correlation between the results of the triaxial and direct shear tests is difficult to establish. However, results of individual direct shear test machines can be corrected with those of the triaxial tests after extensive testing on each machine is done under a variation of the testing material, relative density and normal loads.

6.5 Structural Features of the Equipment and Shear Strength Values

The different makes of the direct shear test equipment do not necessarily generate the same shear strength parameters even if the testing material, normal loads and densities are kept constant. One of the reasons for this may be the difference in the structural features of the equipment which were discussed briefly in section 5.3. The most important of these features are the swan-neck, the arrangement for controlling the rotation of the top cap and the bearing that supports the horizontal rod through which the shear stress is applied. The following sections discuss the direct shear test results in light of these features.

6.5.1 Swan-neck Arrangement for Horizontal Load

Out of the various differences in construction, the most important one appear to be the presence or absence of the 'swan-neck' arrangement for application of the horizontal load. As shown in Figure 6.7, the horizontal load applied at the center of the upper shear ring results in a horizontal force as well as a moment along the shear plane (the middle of the soil sample). If, however, the load is applied through a swan-neck attached on the upper shear ring, the horizontal load is applied directly along the shear plane and the generation of a moment is eliminated.

The direct shear test machines used in this study were examined in light of the above fact. It was found that (i) the Soiltest has an appropriate swan-neck, (ii) the ELE does have a swan-neck but the load is still applied slightly higher than the shear plane thus producing a small moment along the shear plane, (iii) the Wykeham Farrance does not have any such arrangement at all, and (iv) the Clockhouse has an appropriate swan-neck.

The influence of the swan-neck in the direct shear equipment, however, cannot be ascertained from this study because the Soiltest and Wykeham Farrance machines gave almost the same friction angles (49.0 degrees) despite the fact that the former has a swan-neck but the latter does not. Similarly, the value of the friction angle from the Clockhouse is significantly lower than that from the Soiltest and at the same time both of these machines have swan-necks that seem to have structural similarity to a great extent.

6.5.2 The Effect of Rotation of Top Cap

The effect of the rotation of the top cap during the direct shear test was studied by Potts, Dounias and Vaughan (1987) when they analysed the direct shear test through the finite element method. The findings of the study indicated that the freedom of the top cap to rotate during the test gave slightly less stiff pre-peak load-displacement behavior (Figure 2.13) and had an influence on the peak shearing resistance. In two of the three analyses, the freedom caused a reduction of 3% while in the third one, the peak strength was reduced by 5% which was also 5% lower than the simple shear prediction. The differences were quite small, and the evidence limited. However, the results suggested that there may be a marginal advantage in promoting freedom of rotation of the top cap when testing soils with slight strain softening and in restraining it when testing highly brittle soils such as plastic clays.

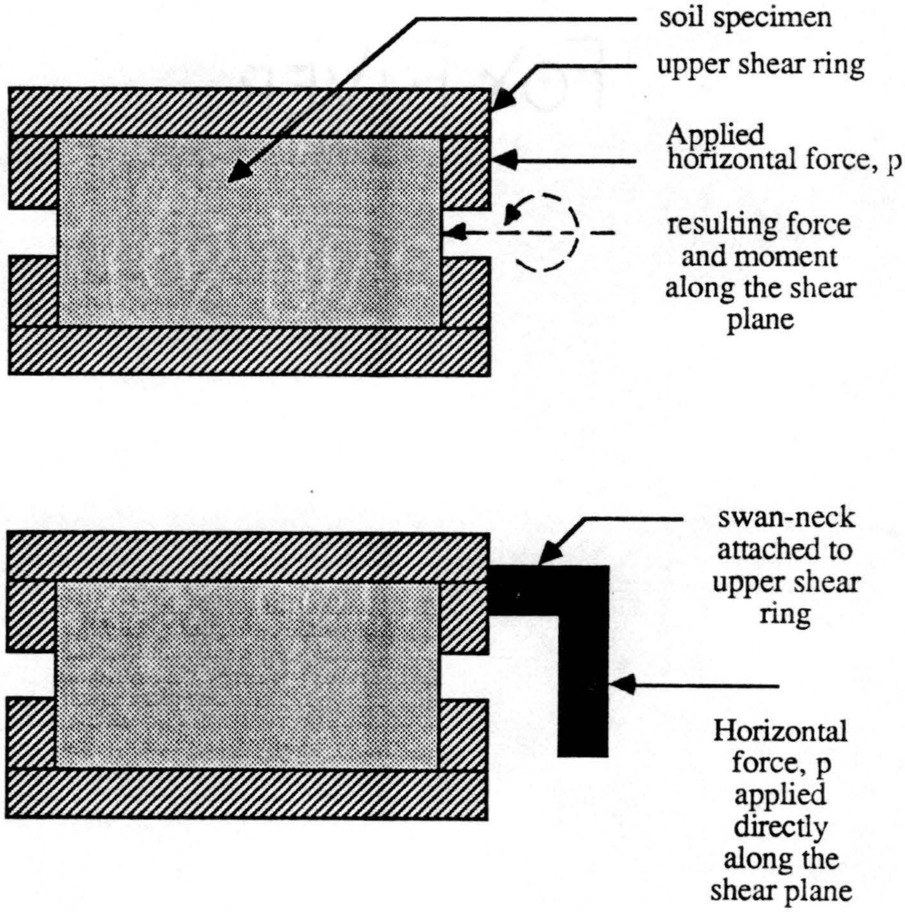


Figure 6.7 Diagram showing schematic of direct shear box with and without a swan-neck attachment

The Soiltest machine has an arrangement for pinning down the top cap to the upper shear ring that eliminates the rotation of the top cap. In ELE, the top cap sits tightly in the upper shear ring preventing it from rotation. Both in Wykeham Farrance and Clockhouse, however, the cap has a chance to rotate slightly during the test.

The results of the shear tests if viewed in light of the top cap rotation do not conclude whether it contributed to the strength or not. Both Soiltest and Wykeham Farrance machines gave the same values of friction angle yet in the Soiltest machine no rotation was observed while slight rotation was observed in the case of the Wykeham Farrance. Similarly, if the results of Wykeham Farrance are compared with those from the Clockhouse, the rotation was observed in both these machines yet the friction angles obtained from these differ by almost 9 degrees. The results of Soiltest and ELE are not the same despite the fact that no rotation of the top-cap was noticed in both these machines.

6.5.3 The Effect of Bearing on Direct Shear Test Results

Another structural feature that can affect the results to a degree is the presence of a bearing that supports the shaft which applies the horizontal stress to the shear box. The bearing can apply friction on the shaft thus resulting in a higher value of the shear stress. The Wykeham Farrance and Clockhouse have this feature. The effect of the bearing on the results can not be ascertained from this study because the Wykeham Farrance gave a higher value of friction angle whereas the Clockhouse gave the lowest value among all the machines. From the values of the cohesion given by these two machines, the bearing might have contributed to the value of cohesion.

6.6 Qualitative Analysis of the Direct Shear Machines

Based on the results of this study, a preliminary qualitative analysis of the direct shear data can be made on the basis of the following two criteria:

(i) The general agreement between the Mohr-Coulomb envelopes as determined from the results of direct shear tests with those of the triaxial tests.

(ii) The scatter in the data which is indicated by statistics such as the standard deviation and the AAD.

A judgement regarding the machines can be made with reference to Table 4.4 and Figures 6.2 thru 6.6.

Table 4.4, shows the Soiltest machine gave the lowest values of standard deviation (0.64 and 1.17). The values of AAD are also the lowest for this machine (0.45 and 0.97). These values clearly indicate the smallest scatter in the data as compared to the other three machines and indicates the dependability of the Soiltest machine. This machine also provides the best correlation to the Mohr-Coulomb envelopes obtained in the triaxial tests (Figure 6.3). However, the friction angle is higher than that of the triaxial tests by almost 4 degrees .

Soiltest machine is followed by ELE in meeting the two criteria. Clockhouse satisfies only the second criterion, giving very low scatter at low normal stresses and higher scatter at higher values of normal stresses as compared to the other three machines. If judged on the basis of the first criterion, however, the trend of the Mohr-Coulomb envelope is very dissimilar to that of the triaxial tests as well as those of the other three direct shear machines which makes the behavior of this machine anamolous. This machine should be studied further.

Wykeham Farrance satisfies the first criterion but has shown a very high degree of scatter in the data (STD = 2.12 and 3.51) and (AAD = 1.59 and 2.77). These numbers make this machine to be the least dependable among the four used for this study.

CHAPTER 7

OBSERVATIONS, CONCLUSIONS AND RECOMMENDATIONS

7.1 Observations

1. The degree of variability or scatter is smaller for the triaxial tests than was found in the direct shear test data.
2. The value of cohesion c' obtained from the direct shear tests was found to be significantly higher than that of the triaxial tests.
3. Three out of four direct shear test machines, (Soiltest, ELE, and Wykeham Farrance) gave a significantly higher value of ϕ' as compare to that obtained from the triaxial tests.
4. The values of c' and ϕ' vary from machine to machine.
5. The Soiltest machine gave a value of cohesion which was the closest to that generated by the triaxial test machine.
6. The values of friction angle obtained from the Soiltest and Wykeham Farrance machines were found to be sufficiently close to each other, the difference being only 0.5 degrees.
7. The values of cohesion from ELE and Clockhouse machines are sufficiently close.

7.2 Conclusions

1. The way stresses are applied to the soil specimen in the triaxial compression and the direct shear tests are completely different.

Finding an explanation for the higher shear strength values from direct shear tests is therefore difficult.

2. The value of the shear strength from triaxial test and different direct shear machines are close but the difference could be significant within different engineering tolerances.

3. The structural features of the direct shear test equipment such as the swan-neck attachment, rotation of the top cap and the presence of the bearing that supports the shear stress rod, may have played a role in the higher values of the shear strength obtained from the direct shear tests especially in the case of the Wykeham Farrance machine. The extent of that contribution, however, can not be ascertained quantitatively at this point.

4. The behavior of the Clockhouse machine is different from the other machines in that the shear strength values obtained from this machine were greater than those of the triaxial tests for low values of normal stresses and smaller for high values of normal stresses. This behavior is anomalous as compared to the other three machines.

5. If judged on the basis of qualitative criteria such as scatter in the shear values and the extent of the similarity of the trend of the Mohr-Coulomb envelope in comparison with that of the triaxial tests, the Soiltest machine was found to be the best whereas the Wykeham Farrance was the worst. Greater dependability can therefore, be attached to the Soiltest machine for its shear strength results.

7.3 Recommendations

To understand the results of direct shear tests and to establish some sort of definite empirical correlation between the results of the direct shear and triaxial tests for practical reasons, further studies are needed. They should generate enough data that will help in investigating the amount of difference in the results of direct shear and triaxial tests. Specific suggestions include:

1. Carrying out direct shear tests with a different range of relative density using the material of this study. The value of relative density can be adopted as 25%, 50%, and 100% while keeping the normal stresses the same as were used in this study. This will help in ascertaining the effect of variation in relative density on the friction angle and cohesion.
2. Testing the same type of soil under a variation of normal stresses and relative density.
3. Testing different types of soils under the same normal stresses and relative density.
4. Carrying out tests on a variety of soils under a set of different normal stresses and relative densities.

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