LABORATORY TEST TECHNIQUES FOR DETERMINING
THE CONSTANT-STRUCTURE ENVELOPE OF SOILS

By

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NOTATION

a	=	zero intercept of Rowe's stress path plot
A_m	=	area of membrane
A_s	=	area of sample
c	=	cohesion term in Coulomb's equation
c_e	=	Hvorslev effective or 'true' cohesion
c_f	=	cohesion corrected for dilatancy, interparticle cohesion
c_o	=	zero cohesion intercept
CSE	=	Constant Structure Envelope
d	=	diameter of sample
e	-	void ratio
e_c	=	void ratio, after consolidation
e_f	=	final void ratio, after completion of CSE-test
e_i	=	initial void ratio, before consolidation
e_s	=	void ratio, after compression
E	=	extention modulus of rubber membrane
G_s	=	specific gravity of soil solids
h	=	height of sample
I_o	=	bond strength
I_ε	=	effective stress independent component of mobilized shear resistance
I_{max}	=	maximum value of effective stress independent component of mobilized shear resistance
IDS	=	Independent Dependent Strain

LL	=	liquid limit
PI	=	plasticity index
PL	=	plastic limit
r_s	=	radius of sample
R	=	multiple correlation coefficient
s	=	standard error of estimate
S_c	=	degree of saturation, after consolidation
S	=	final degree of saturation, after completion of CSE-test
S_i	=	initial degree of saturation, before consolidation
S_s	=	degree of saturation, after compression
t_m	=	thickness of membrane
u	=	pore pressure
V	=	volume of CSE-test specimen
W_c	=	water content, after consolidation
W_f	=	final water content, after completion of CSE-test
W_i	=	initial water content, before consolidation
W_s	=	water content, after compression
α	=	constant in constant-structure envelope equation
β	=	constant in constant-structure envelope equation
$\delta\varepsilon_1$	=	increment of major principal strain, positive in compression
$\delta\varepsilon_3$	=	increment of minor principal strain, positive in compression
ε	=	strain
ε_{at}	=	vertical strain during testing
ε_v	=	volumitric strain during testing

κ = slope of c_e vs σ'_e line

σ = total normal stress

σ_c = consolidation pressure

σ_f = normal stress on failure plane

σ_r = rebound pressure

σ' = effective normal stress = $(\sigma - u)$

σ'_e = equivalent effective consolidation pressure

σ'_{oct} = effective octahedral stress

$\bar{\sigma}'_t$ = average normal effective stress at any strain between the two points of common Mohr circle tangency in an IDS test

σ'_1 = major principal effective stress

σ'_3 = minor principal effective stress

$\Delta\sigma_{am}$ = membrane correction for axial stress

$\Delta\sigma_{hm}$ = membrane correction for horizontal stress

τ_f = shear stress on failure plane

ϕ = internal friction term in Coulomb's equation

ϕ_e = Hvorslev effective or 'true' angle of internal friction

ϕ_f = angle of internal friction corrected for dilatancy, interparticle friction

ϕ_μ = physical angle of friction between the minerals

ψ = slope of Rowe's stress path plot

Abstract of Thesis Presented to the Graduate Council
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LABORATORY TEST TECHNIQUES FOR DETERMINING THE
CONSTANT-STRUCTURE ENVELOPE OF SOILS

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December, 1971

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This research project involves the development of a laboratory test technique for obtaining the constant-structure envelope of soils. A constant-structure envelope is defined as a Mohr envelope which, in terms of effective stress, for any strain, shows the variation of the maximum shear resistance that can be offered by a particular constant soil structure.

The concept of the constant-structure Mohr envelope has been introduced by Dr. Schmertmann (1966) to explain the effective stress dependency of the I component which had initially been thought independent of effective stress.

A satisfactory test technique was found for a cohesionless glass bead material and a remolded kaolinite clay. The writer could not develop a test for a material with artificial bondings.

Good correlation was obtained between Schmertmann's theoretical equation for a constant-structure envelope and the experimental results. The soil parameters I_o-bond strength, α and β may be determined from the experimentally developed constant-structure envelopes.

CHAPTER I

INTRODUCTION

This research project concerns mainly the development of a laboratory test technique for obtaining a constant-structure Mohr envelope for saturated soils. A constant-structure envelope is a Mohr envelope expressed in terms of effective stress at any strain and shows the variation of the maximum shear resistance that can be offered by a particular soil structure at that strain. Soil structure is defined as the geometrical arrangement of particles and internally generated forces. From a constant-structure Mohr envelope it is possible to separate the shear resistance into an effective stress independent and a dependent component as defined by Professor Schmertmann (1963).

The equation for the constant-structure envelope is given by

$$I = I_o + \alpha\sigma' - \beta\sigma' \ln \sigma' \qquad (1\text{-}1)$$

based on the linearity between σ' and I_{max} (Schmertmann, 1966a).

The concept of the constant-structure Mohr envelope was originally introduced by Professor Schmertmann (1966) to explain the effective stress dependency of the I_m component of the shear strength of a soil. Prior to this work, the existance of such an envelope was hypothetical and inferred from other data. Ho (1971) showed that such an inferred constant-structure envelope can be obtained at each strain.

The bond strength, I_o is defined as the shear strength of a soil when the effective stress is zero and can be determined as the τ intercept on the $\sigma'=0$ axis of the constant-structure Mohr envelope. The bond strength is some function of the non-effective stresses or bonds between particles and of the extremely complex geometry of particle spacing and contacts, in other words, a function of the soil structure. There has been much careful research and many proposals suggesting that soil bonding may be an adhesive cementing, and/or some special behavior of the water molecules near the soil particle surfaces, and/or do to various other electrochemical forces that are believed to be significant. A knowledge of the magnitude of this bond strength may help to identify its causes.

The knowledge of the shear strength of a soil is of great importance to the practice of soil engineering. Among the most frequent and most serious problems which a soil engineer faces are those of stability, i.e., those requiring knowledge of the shear strength of soil.

Herein a test technique is developed whereby it is possible to determine directly a constant-structure envelope from a single sample. This will be a great help in the investigation of the mechanics of shear strength, which, as mentioned, occupies such an important position in the field of soil mechanics. The test technique is characterized herein as the constant-structure envelope test (CSE-test).

Three types of materials are used in this investigation, i.e., kaolinite (cohesive), glass beads (cohesionless) and glass beads with hydrocal (artificial bonds). A test technique was developed, tested and is reported. The experimentally produced constant-structure envelope is fitted to Schmertmann's theoretical equation by a least square fit and the bond strength I_o and the soil characteristics α and β determined. These values are compared with the ones determined indirectly by Ho (1971) and reasonable agreement is obtained.

CHAPTER II

REVIEW OF PREVIOUS WORK

In 1773 Coulomb published the paper, "Essai sur une Application des Regles de Maximis et Minimis a Quelques Problemes de Statique, Relatifs a L'Architecture", in which he dealt with the earth pressure theory and introduced in words the first relationship between shear and normal stress for a soil at failure. This relationship has been used to evaluate an equation, known as Coulomb's failure criteria, which states.

$$\tau_f = c + \sigma_f \tan \phi \qquad (2\text{-}1)$$

where τ_f is the total shear resistance, c is called the cohesion, and ϕ the angle of internal friction. Figure II-1 illustrates this linear equation. It was thought that c and ϕ were constant for a given soil and could be determined by simple tests. Ever since its appearance, this law of Coulomb has served as the basis for most stability computation of soil masses or scientific investigations of shear strength problems.

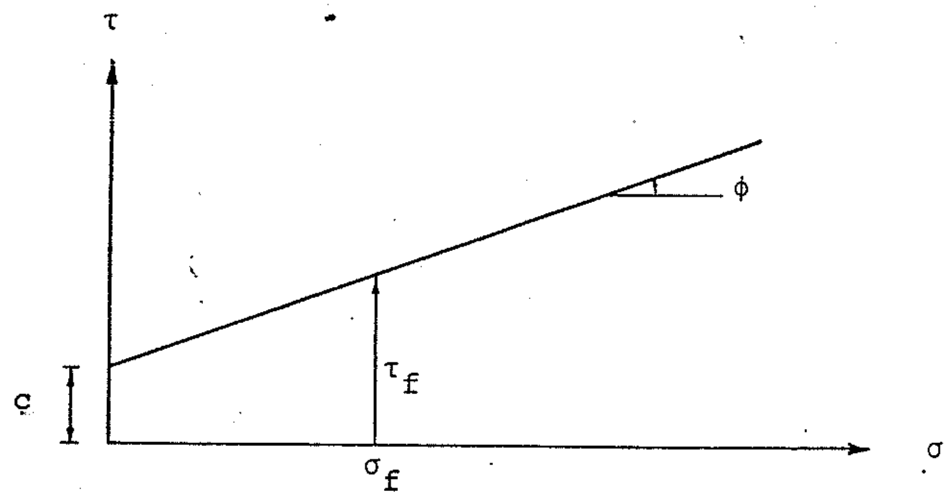


Figure II-I. Coulomb Failure Criteria

In spite of the uncomplicated form of this equation a determination of the soil coefficients introduced by Coulomb creates a number of difficulties, especially where cohesive soils are concerned. A determination of these coefficients demands a knowledge of the physical properties of soils and an understanding of the phenomena which determine their strength.

In 1882, Mohr proposed a more general criterion of failure based on shear stress. He assumed that at failure the shear stress, τ_f and the normal stress on the failure plane, σ_f are uniquely related,

$$\tau_f = f\,(\sigma_f) \tag{2-2}$$

This relation is represented on a shear stress versus normal stress diagram by the envelope of all possible Mohr circles of stress for the material. When the relation of the Mohr criterion is linear, it becomes identical with the Coulomb criterion, as shown in equation (2-1). Thie is referred to as the Mohr-Coulomb failure criterion.

Early investigators using direct shear apparatus found that the test results were greatly influenced by the rate of shearing and by the initial water content of a given soil. In the 1920's Terzaghi pointed out that effective stress, σ', controls the deformation behavior of saturated soils and it is equal to the total stress, σ, minus the pore water pressure, u

$$\sigma' = \sigma - u \qquad (2\text{-}3)$$

The Mohr-Coulomb criterion for saturated soils is therefore more accurate when expressed in terms of effective stresses

$$\tau_f = c' + (\sigma_f - u) \tan \phi' \qquad (2\text{-}4)$$

Figure II-2 illustrates this linear equation for the plane of envelope tangency, at failure, during a triaxial compression test. This equation remains one of the most controversial in soil mechanics in the sense that the parameters c' and ϕ' are not unique.

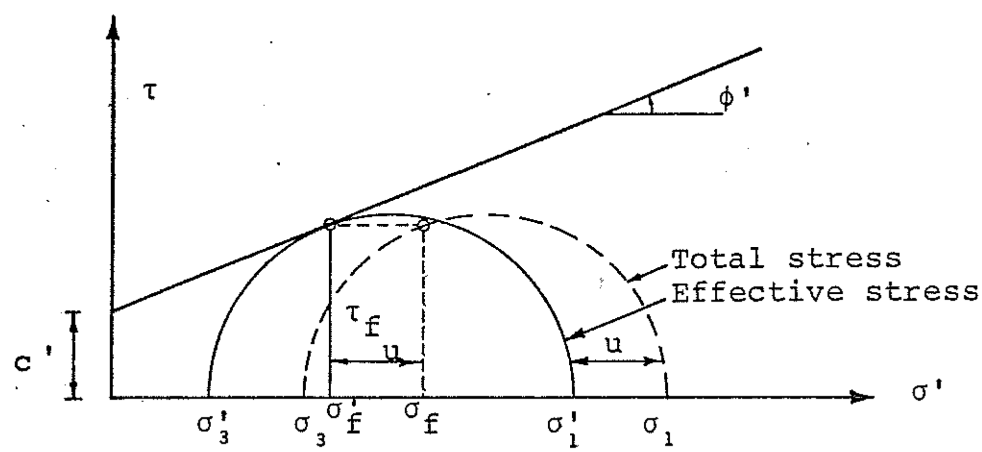


Figure II-2. Mohr-Coulomb Effective Failure Criteria.

Real progress in the knowledge of fundamental shear strength properties was not made until systematic investigations were carried out at Terzaghi's Soil Mechanics Laboratory in Vienna in order to clear up some of the more important questions concerning shear strength. On the basis of experimental work carried out during the years 1934-37 on two remolded clays, Hvorslev (1937) showed that the shear strength τ_f could be expressed by the equation

$$\tau_f = c_e + \sigma_n' \tan \phi_e \qquad (2\text{-}5)$$

where c_e and ϕ_e, the effective cohesion and effective angle of internal friction, were parameters characterising the soil. The cohesion was shown to depend only upon the water content of the soil at failure while the angle of internal friction was found to be approximately a constant at different water contents. These parameters were thought to be fundamental and became therefore known as the 'true' components of soil strength.

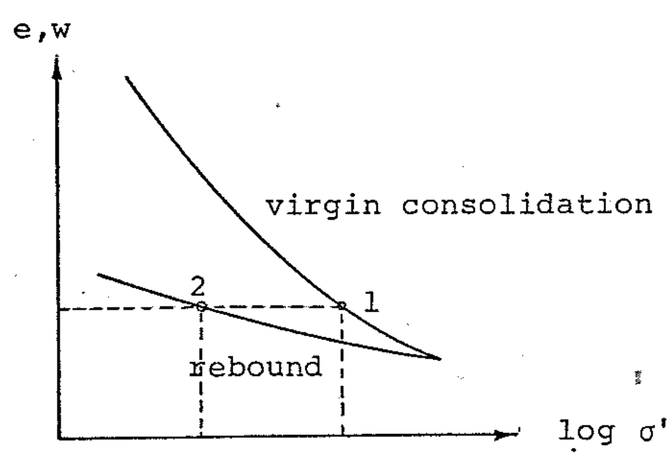


Figure II-3. Consolidation Curve

Terzaghi suggested that c_e and ϕ_e were to be determined from the common tangent at failure for two samples with the same void ratio. One sample is normally consolidated and the other overconsolidated. Figure II-3 and II-4 show the determination of Hvorslev's effective strength components.

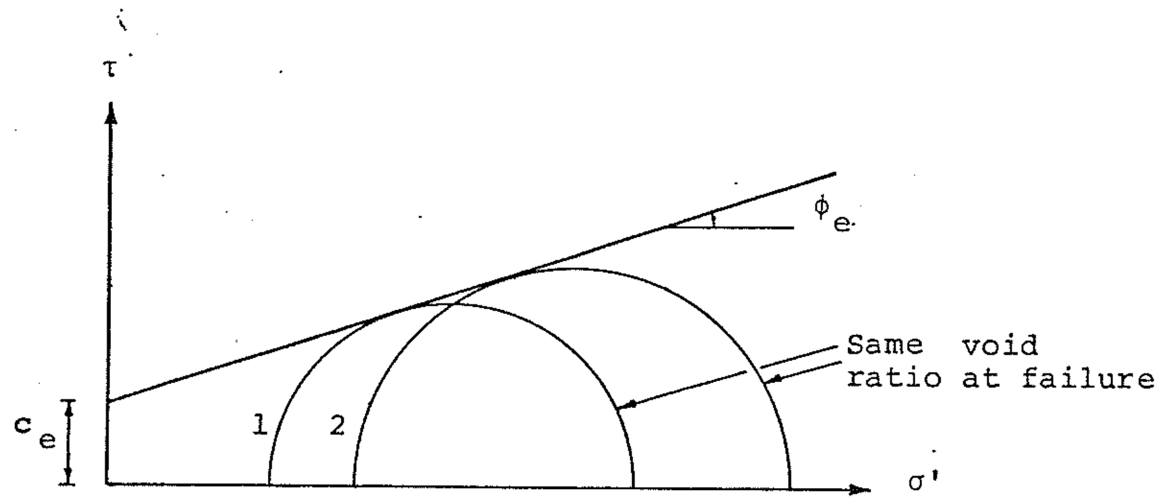


Figure II-4. Hvorslev's Effective Strength Components.

Hvorslev found a linear relationship between the intercept on the σ' = 0 axis and the equivalent virgin consolidation pressure, σ'_e, for clays remolded at high water contents,

$$c'_e = \sigma'_e \ \kappa \tag{2-6}$$

The equivalent virgin consolidation pressure is defined as the pressure on the virgin consolidation curve, which corresponds to the acutual void ratio (water content). κ is a 'coefficient of cohesion'.

Subsequent work by Peynircioglu (1939) indicated that ϕ_e depended to some extent on the water content at failure.

Bjerrum (1954) found that for clays remolded at low water contents, a zero cohesion intercept, 'origin cohesion', c_o exists

$$c_e = c_o + \kappa\sigma'_e \tag{2-7}$$

Bjerrum worked with normally and overconsolidated clays and considered the 'true' cohesion to result from the attractive forces between clay particles which are not in direct contact and the resistance of water shells around the clay particles.

Hvorslev's so called 'true' strength components are seldom used in practical problems. Yet, the components are important for an understanding of the basic failure mechanisms of soils. Their mere existence has been a point of support for the continued use of the component philosophy of soil strength behavior. However,"it should be pointed out that some prominent authors (Lambe, 1958), working with the physico-chemical fundamentals of soil behavior, have expressed doubt about the usefulness and wisdom of separating soil strength into cohesion and friction components," (Schmertmann and Osterberg, 1960).

Hvorslev's effective stress components, c_e and ϕ_e, were based on strength behavior at different effective stresses, but at the same void ratio on the failure plane at the point of failure. However, equal void ratio does not mean the same structure and thus it is not a sufficient description of the soil. These components therefore, are not truly fundamental. For example, an undisturbed sample and a remolded sample of the same sensitive clay may have the same void ratio, but their structures are totally different.

The importance of relating the fundamental components of shear strength to the soil structure was realized almost simultaneously in the early 1960's by Professor Rowe (University of Manchester) and Professor Schmertmann (University of Florida). The mathematical and experimental relations of shear strength and structure developed by these investigators supplanted Hvorslev's concept of 'true' strength components. The experimental procedure developed by Schmertmann is called the IDS-test, and the mathematical concept proposed by Rowe is known as the Stress Dilatancy Theory.

Rowe, together with many other investigators, separated the total shear resistance measured by shear tests into energy and strength components. He suggested that the structure be allowed to change when comparing tests, but included a correction for the effects of any such change. This is a method of obtaining the fundamental components by correcting for the structural change and, therefore, gives the true components of shear strength.

Rowe separates the effective stress dependent components into one of volume change (dilatency) and one of effective friction. By considering individual particles in contact, at sliding, he derived the energy ratio, E_r for cohesionless soils,

$$E_r = \frac{\sigma_1' \delta\varepsilon_1}{2\sigma_3 \delta\varepsilon_\sigma} = \tan^2(45° + \frac{\phi_\mu}{2}) \qquad (2\text{-}8)$$

The following notation is used (after Rowe):

σ_1' = major principal effective stress.

σ_3' = minor principal effective stress.

$\delta\varepsilon_1$= increment of major principal strain, defined as positive in compression.

$\delta\varepsilon_3$= increment of minor principal strain, defined as positive in compression.

ϕ_μ = physical angle of friction between the minerals.

For a soil in elastic deformation at small strains, the change in volumetric strain, $\delta\varepsilon_v = \delta\varepsilon_1 + \delta\varepsilon_2 + \delta\varepsilon_3$. For a sample in the triaxial cell the above equation reduces to

$$\delta\varepsilon_v = \delta\varepsilon_1 + 2\delta\varepsilon_3 \qquad (2\text{-}9)$$

Rewriting this volume change in the form

$$\frac{2\delta\varepsilon_3}{\delta\varepsilon_1} = 1 + \frac{\delta\varepsilon_v}{\delta\varepsilon_1} \qquad (2\text{-}10)$$

he defined the quantity $(1 + \frac{\delta\varepsilon_v}{\delta\varepsilon_1})$ as the dilatency factor, which is easily measured in the triaxial cell.

Substituting (2-10) in (2-8) he obtained the basic stress dilatency equation for cohesionless soils

$$\frac{\sigma_1'}{\sigma_3'} = (1 + \frac{\delta\varepsilon_v}{\delta\varepsilon_1}) \tan^2(45 + \frac{\phi_\mu}{2}) \qquad (2\text{-}11)$$

In the above equation, ϕ_μ is replaced by ϕ_f in the general case, where the soil is not very dense.

He extended the concept to cohesive materials by introducing the interparticle cohesion term, c_f and derived to the following equation:

$$\left(\frac{\sigma_1}{1+\frac{\delta\varepsilon_V}{\delta\varepsilon_1}}\right) = \sigma_3' \tan^2(45+\phi_f/2)+2c_f \tan(45+\phi_f/2) \qquad (2\text{-}12)$$

where ϕ_f and c_f are corrected for stress dilatency.

For separation of the interparticle components, c_f and ϕ_f, as a function of strain, it is convenient to plot stress paths with $\left(\frac{\sigma_1'}{1+\frac{\delta\varepsilon_V}{\delta\varepsilon_1}} + \sigma_3'\right)$ as abscissa and $\left(\frac{\sigma_1'}{1+\frac{\delta\varepsilon_V}{\delta\varepsilon_1}} - \sigma'\right)$ as the ordinate. The parameters are then computed from

$\sin\phi_f = \tan\psi$

$c_f = (a/2) \sec\sigma_f$

Figure II-5 shows the procedure

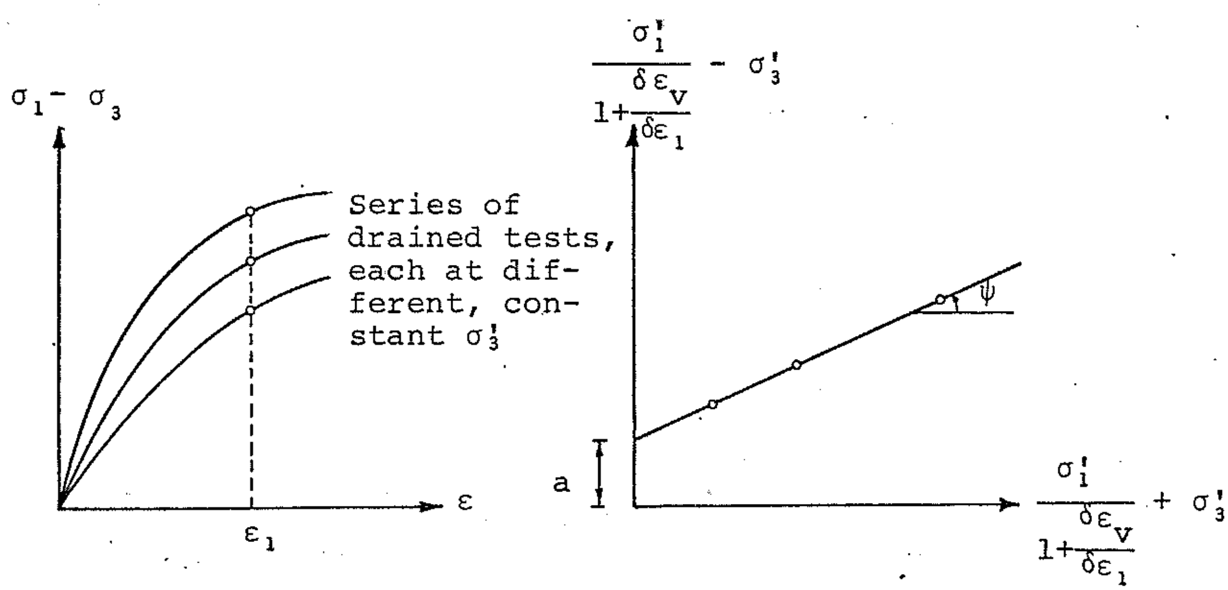


Figure II-5. Definition of Strain Path by Stress Dilatancy Method.

The interparticle cohesion, c_f, is a measure of the bond strength of the soil and can be compared with I_o from the IDS-test. Their difference lies in the effect of dilatancy, which is small (Ho, 1971).

The theory and technique introduced by Schmertmann and Osterberg (1960) to separate shear resistance into components at any stage of deformation will be discussed in the next chapter. This forms the basis of the development of the CSE-test.

CHAPTER III

IDS-I_o THEORY AND DEFINITIONS

Schmertmann's Theory

Professor Schmertmann was the first to realize that any fundamental cohesion and friction component must vary with strain and he devised a test procedure to investigate such variation. Schmertmann states that, in general, even though a soil has the same void ratio it does not necessarily mean equal structure and therefore equal cohesion, and that soil structure is a function of stress history, effective stress, strain and time. He realized that though Hvorslev's parameters were obtained from 2 or more samples at equal void ratio, they were not at equal structure. He introduced two generalized components of shear strength. At any strain level the shear strength is expressed as a combination of two components, one dependent and proportional to the effective stress, and the other independent of the effective stress, i.e.

$$\tau_\varepsilon = I_\varepsilon + D_\varepsilon \tag{3-1}$$

where

I_ε = independent component

D_ε = dependent component.

The definitions of D_ε and I_ε are illustrated in Figure III-1.

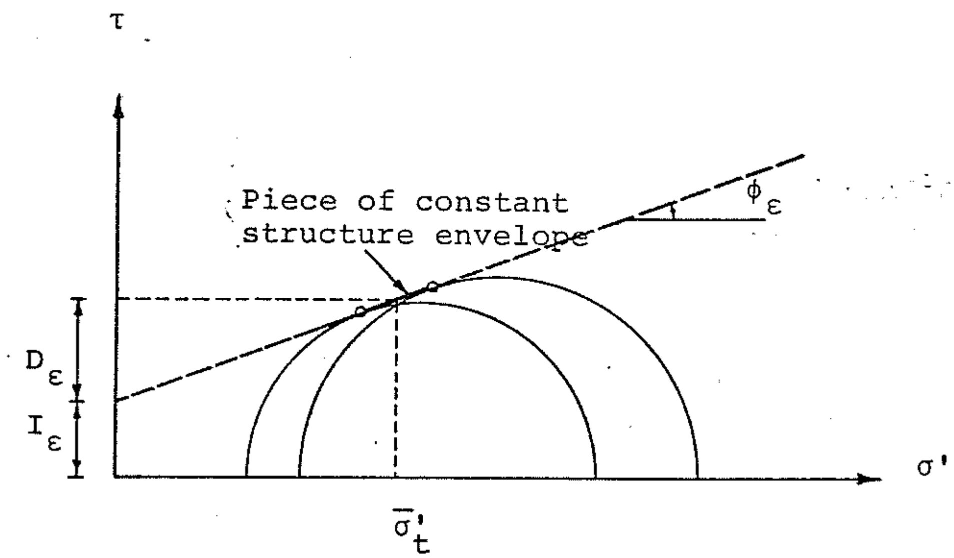


Figure III-1. I_ε and D_ε Components.

Schmertmann invented his curve-hopping technique to separate shear resistance into I_ε and D_ε components at any strain. The separation requires the fitting of a tangent to two Mohr circles having the same structure. The zero tangent intercept on the shear stress axis gives the I_ε component while the slope, tan ϕ_ε, is a measure of the shear resistance sensitivity to a change in effective stress at the particular soil structure. The D_ε component is then determined as the difference between the total shear resistance and I_ε.

$$D_\varepsilon = \tau - I_\varepsilon = \overline{\sigma}'_t \tan \phi_\varepsilon \qquad (3\text{-}2)$$

where $\overline{\sigma}'_t$ is the average normal effective stress at the tangential points to the two Mohr circles.

To assist in defining bond strength, Schmertmann introduced the concept of an imaginary constant-structure envelope. This is defined as a Mohr envelope expressed in terms of effective stress and shows the maximum shear resistance that can be offered by a particular, constant soil structure, including strain. He then defines bond strength as the shear resistance of the soil when the constant-structure envelope intersects the τ-axis at zero effective stress. The constant-structure envelope is shown in Figure III-2.

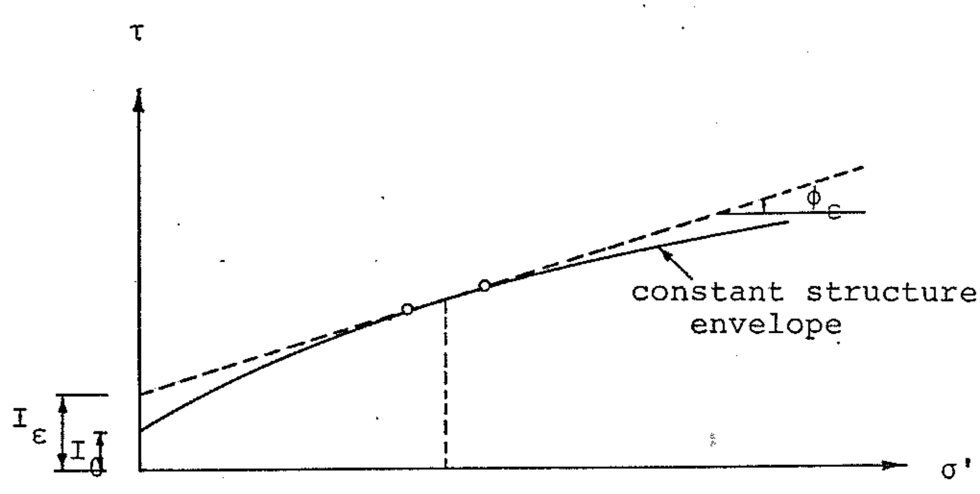


Figure III-2. Constant-Structure Envelope.

He further showed that this bond strength, I_o, could be determined by the zero intercept on the I-axis of the linear relationship between I_{max} and $\overline{\sigma}'_t$ data. I_{max} was defined as the maximum I_ε value. I_ε was found to develop its maximum value consistently at low strain. Ho extended this procedure to determine I_o from linear relationships of I_ε and $\overline{\sigma}'_t$ at different strain values.

A review of the IDS-procedure is illustrated in Figures III-3a, 3b, and 3c.

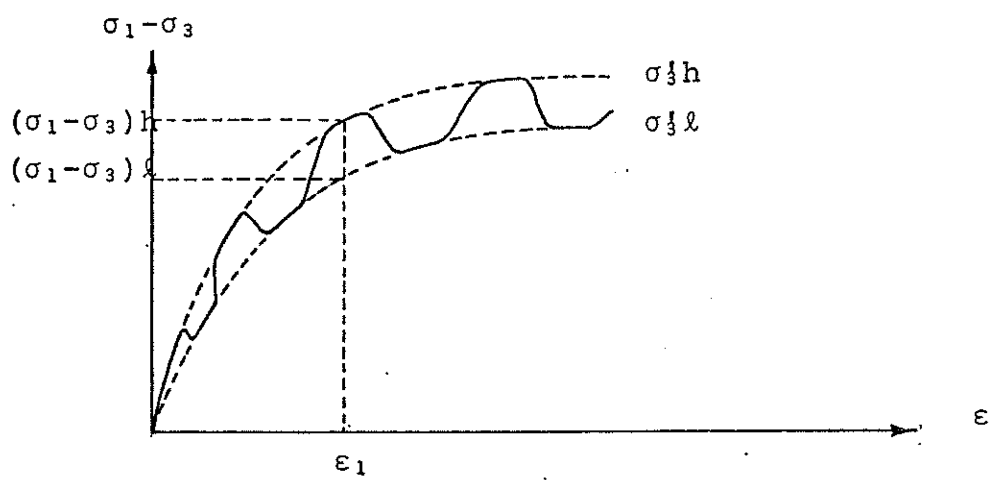


Figure III-3a. Strain Path from one IDS-test using Curve-hopping Technique and Constant Pore Pressure (Triaxial test with σ'_3 = constant).

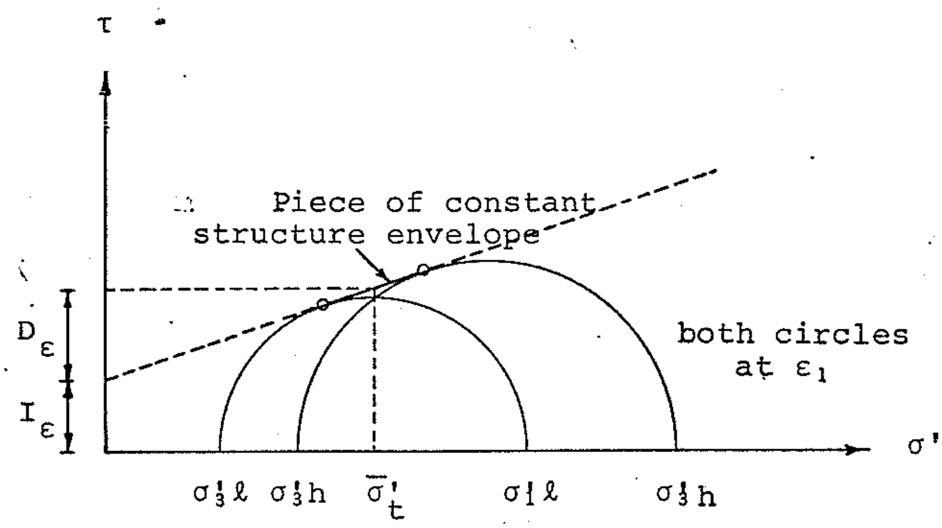


Figure III-3b. Fitting of Tangent to Two Mohr Circles with Same Structure, and Determining of the I_ε and D_ε Components.

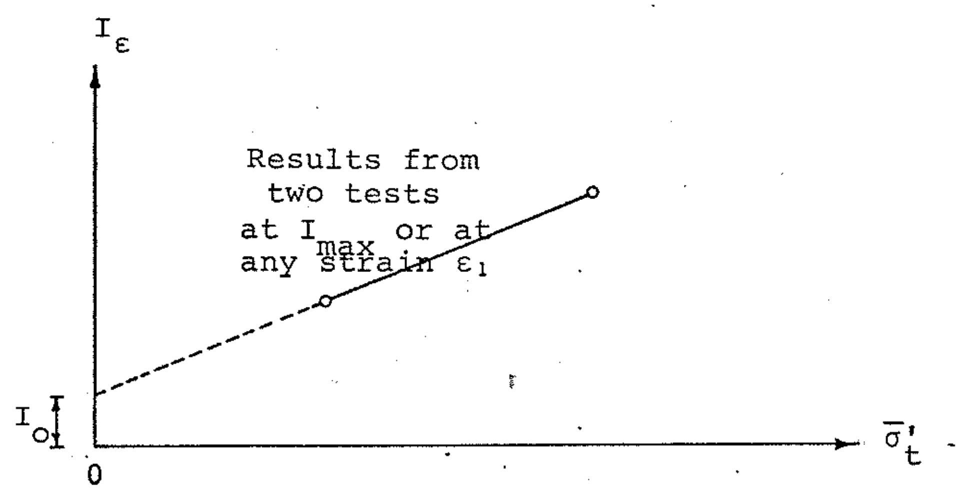


Figure III-3c. I_ε vs $\overline{\sigma}'_t$ Plot and Determining of the Bond Strength, I_o.

Figure III-3. Illustration of IDS-Test.

Schmertmann and Osterberg (1960) and Schmertmann (1963) give a complete description of the IDS-test.

Schmertmann (1966a) demonstrated the stability of the I_{max} values which he found to be a linear function of the magnitude of effective stress. This indicates that the I value must include some other component besides the bond strength between particles. The introduction of the constant-structure envelope helps to explain this paradox. The curvature of the constant-structure envelope results in the contribution to the I_ε component. (Figure III-2 shows this effect.)

Based on the experimentally proven linear relationship between I_ε and $\bar{\sigma}'_t$, an equation for the constant-structure Mohr envelope can be derived from the differential equation,

$$\frac{d\tau}{d\bar{\sigma}'_t} = \frac{\tau - I_O - \beta\bar{\sigma}'_t}{\bar{\sigma}'_t} \tag{3-3}$$

giving the solution as

$$\tau = I_o + \alpha\sigma' - \beta\sigma' \ln\sigma' \tag{3-4}$$

where I_o = bond strength

α = soil structure parameter varying with strain, stress ratio, density, etc.

β = soil constant varying seemingly with grain size only.

Knowledge of this relationship has been a great help in the understanding of the shear strength phenomenon. Determination of the bond strength, I_o, using the described IDS-test procedure has the decisive disadvantage that it requires at least two good tests with identical materials. This requirement of duplicate specimens has been a disadvantage in the investigation of bond strength, especially with undisturbed soils. Schmertmann (1966b) proposed a one specimen test procedure for determining I_o based on previous knowledge of range of β values.

This problem can also be avoided if a constant-structure Mohr envelope can be constructed from a single test. This project concerns the development of such a test, which for different effective stresses defines the maximum shear resistance that can be offered by the particular soil structure, keeping structure (including strain) nearly constant during the test.

The main philosophy in the test procedure is that after drained loading of a sample, it is possible to increase the pore pressure (decrease the effective stresses) in steps. After allowing for equilibrium, the strength of the sample is measured. The increase of pore pressure can be done at a nearly constant strain of the sample, which might mean only a slight, perhaps negligible, change of the soil structure.

Review of Definitions

Before going further into the test procedure which will be discussed in Chapter V some definitions will be outlined and analyzed. This is done in order to familiarize the reader with the terminology and avoid any uncertainty regarding the definitions which will be used throughout this thesis.

Soil Structure

The term soil structure refers to the orientation and distribution of particles in a soil mass and the forces between adjacent soil particles (Lambe and Whitman, 1969).

The particulate nature of soils distinguishes soil mechanics from solid mechanics and fluid mechanics. Thus the resistance to shear deformation of a soil mass is controlled by the interaction between adjacent particles, or soil structure. The two extremes in soil structures are flocculated- and dispersed-structure. In the flocculated-structure, the soil particles are edge to face and attract each other. A dispersed-structure has parallel particles which tend to repel each other. In general, an element of flocculated soil has a higher strength, lower compressibility and higher permeability than the same element of soil at the same void ratio in a dispersed state.

A given soil structure is a main function of the arrangement of the particles. Alterations which introduce displacement between particles tend to change the structure, even at the same void ratio. Therefore, soil structure varies significantly with strain. Other factors which may effect the soil structure are effective stress, void ratio, temperature, time, etc.

Constant-Structure Envelope

A constant-structure envelope is a Mohr envelope which in terms of effective stress for any strain defines the variation of the maximum shear resistance that can be offered by a particular, constant soil structure. Schmertmann proposed an equation for the constant structure Mohr envelope.

$$\tau = I_o + \alpha\sigma' - \beta\sigma' \ln \sigma'$$

Bond Strength

The bond strength, I_o, is the shear resistance a given soil structure is able to mobilize when the effective stress is zero. This resistance must result from one or more forms of internally generated forces of attraction between particles. This bonding between particles is an important factor in the strength, and therefore, engineering behavior of soil. Hence, it has been the subject of considerable research, supplemented by extensive investigation in the field of colloid science.

α

The α-parameter is a dimensionless constant in the constant-structure Mohr envelope equation. α is dependent on stress history.

β

The β-parameter is a dimensionless constant in the constant-structure Mohr envelope equation. The investigation done in this field by Schmertmann (1966) and supported by data developed by Ho, (1971) suggests that β is a function of the sphericity of shape of the soil grain, i.e., the more angular the particles, the higher the β value. Ho found that β varies from about zero for glass beads to 0.20 for the same remolded kaolinite as used herein. For glass beads plus hydrocal a higher β value of up to 0.30 was found, but this dropped sharply with an increase of strain as the bonds are broken.

CHAPTER IV

SOIL, PREPARATION AND EQUIPMENT

Soil and Preparation

The scope of this investigation included the development of a test technique for obtaining a constant-structure Mohr envelope for three different, artificial, saturated materials--a cohesionless soil, a cohesive soil and a material with artificial bondings were investigated. A description of the different materials follows.

Cohesionless Sample (Glass Beads)

Physical properties:

Specific gravity, $G_s = 2.43$

Shape of particles = spherical

Range of particle sizes = .074 to .105 mm

Minimum Void Ratio (estimated) = 0.49

Maximum Void Ratio = 0.79

The glass beads were obtained from Microbeads Div., Catophote Corporation, Jackson, Mississippi. The material as received was sieved three times for 15 minutes each to separate the fraction between sieve no. 140 and 200 (.074 and 0.105 mm), which was used for the testing. Figure IV-1 shows the used glass bead material.

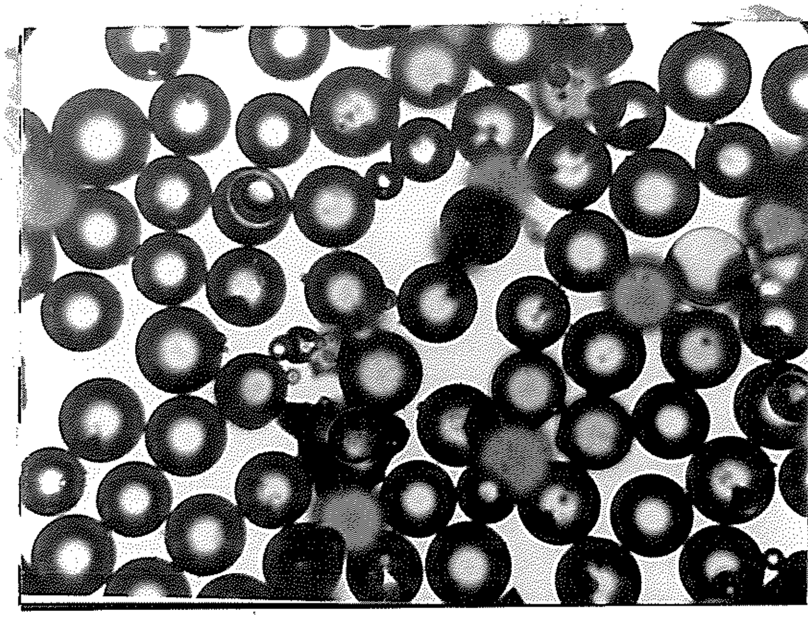


Figure IV-1. Glass Bead Material, 150 x Magnification.

Preparation of samples

Each sample was prepared in the triaxial cell immediately before the test. From the batch of glass beads between 0.074 and 0.105 mm in diameter, the amount necessary for one sample (130 g) was measured. This portion was then saturated with distilled water. The entrapped air was removed by 10 minutes of boiling accompanied by continuous agitation. The glass beads were then ready for forming of the sample. The saturated beads were placed in a split mold lined with a rubber membrane (Sheik rubber prophylactics with a thickness of 0.06 mm), and vibrated in five layers by a 'Burgess' vibro-tool. Figure IV-2 shows a stage of the preparation.

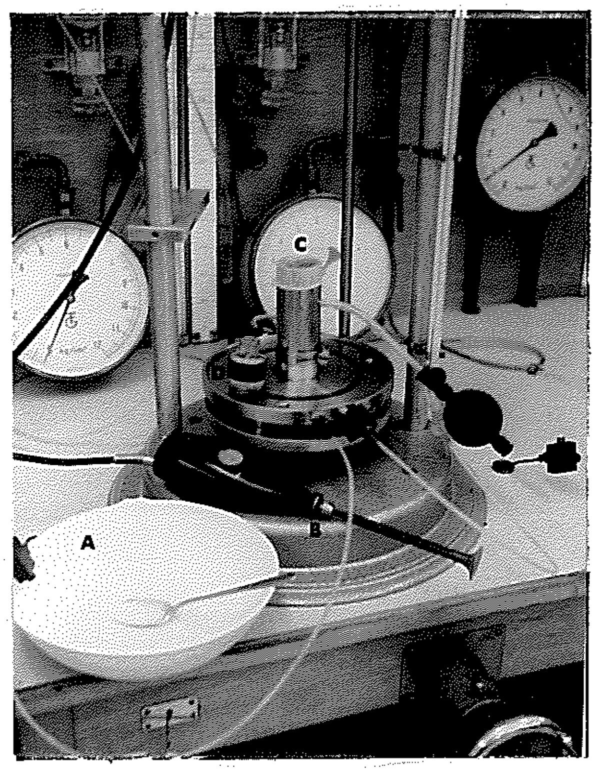


Figure IV-2. Preparation of Glass Beads Sample. A= Dish with saturated glass beads; B= "Burgess" vibrotool; C=Split mold with membrane; D=Top cap.

The last layer of the sample was vibrated with a surcharge weight of 600 grams after placing a top cap in position to eliminate necking of sample at the top when a small vacuum was applied. The vacuum keeps the sample standing after the split mold was removed. The dimensions of the sample were then measured with vernier calipers to 0.001 cm. The sample was sealed with another membrane and six 0-rings with unstretched inside diameter of 3.1 cm, three around both bottom and top cap. To assure a good seal of the sample, a film of grease was placed between the caps and the membrane as well as between the two membranes. Before putting the glass beads in, a boiled and saturated porous filter stone was placed on the pedestal. Frictionless platens were placed at the top and bottom of the sample. The top frictionless platen consisted of Dow Corning 4 compound silicone grease smeared between a rubber membrane and the cap. The bottom one consisted of two rubber membranes with the same grease in between and 4 holes in order to allow drainage of the sample.

After setting up the sample, the cell was assembled and the hydrostatic consolidation pressure was applied.

Cohesive Sample (Kaolinite)

Physical properties:

Specific gravity, G_s = 2.609

Liquid Limit, LL = 59.4%

Plastic Limit, PL = 29.6%

Plastic Index, PI = LL-PL = 29.8%

Less than 0.002 mm = 71%

Activity, PI/% < 0.002 mm = 0.42.

The Kaolinite was delivered in powdered form from Edgar Plastic Kaolin Company, Edgar, Florida. Kaolinite clay, the most important and common two-layer mineral encountered by the engineer, consists of silica and alumina sheets and most of its particles have a hexagonal shape. Kaolinite is a very pure white clay with an earthy smell and a smooth and greasy feel.

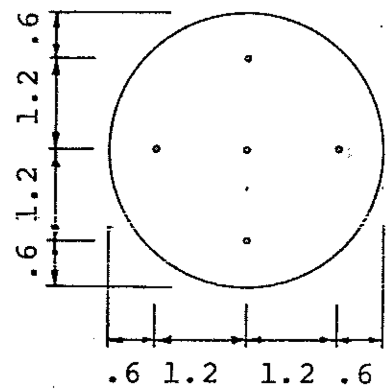
Preparation of sample

The powdered kaolinite was mixed with distilled water to a predetermined water content of about 40 percent and then left overnight. The clay and water mixture was then passed 5 times through a 'Vac-Aire' extruder to distribute the water evenly. The machine extruded the kaolinite in a circular bar with a diameter of 3.58 cm. It was cut to lengths of 10 cm, wrapped in wax paper and dipped into warm wax 4 times to prevent air coming in direct contact with the sample, causing evaporation and/or molding. The samples were numbered in ascending order as they came out of the machine. Water content samples were taken every five samples to check the uniformity of the batch. The water contents of the samples decreased slightly during extrusion and, therefore, with increasing sample number. All Kaolinite samples used in this project were from a batch of 44 samples produced in July, 1971.

The setting up of the kaolinite sample is given in the following procedure. The wax was removed from the sample, which then was placed in a 10 cm mold. With the aid of a template and a needle, 0.18 cm in diameter, 5 holes were punched vertically through the specimen. Doubled red wool yarn, saturated with water, was pulled through the pre-punched holes and served as drains in order to quicken the consolidation and the distribution of pore pressure. The geometry of the drains is shown in Figure IV-3.

The sample was placed in an 8 cm mold and trimmed to a length of 8 cm. The equipment used for the preparation is shown in Figure IV-4. The sample was weighed in a Mettler scale, ± 0.005 g, and the height and diameter measured with vernier calipers, ± 0.0005 cm, before it was placed on a filter stone on the pedestal in the triaxial cell. As for the glass bead sample, the connection between sample and caps was made frictionless by aid of membranes and grease. The sample was sealed with two membranes with grease between the three O-rings each around the pedestal and top cap.

After setting up the sample the cell was assembled and the hydrostatic consolidation pressure applied.



All dimensions are in cm.

Figure IV-3. Configuration of Drains.

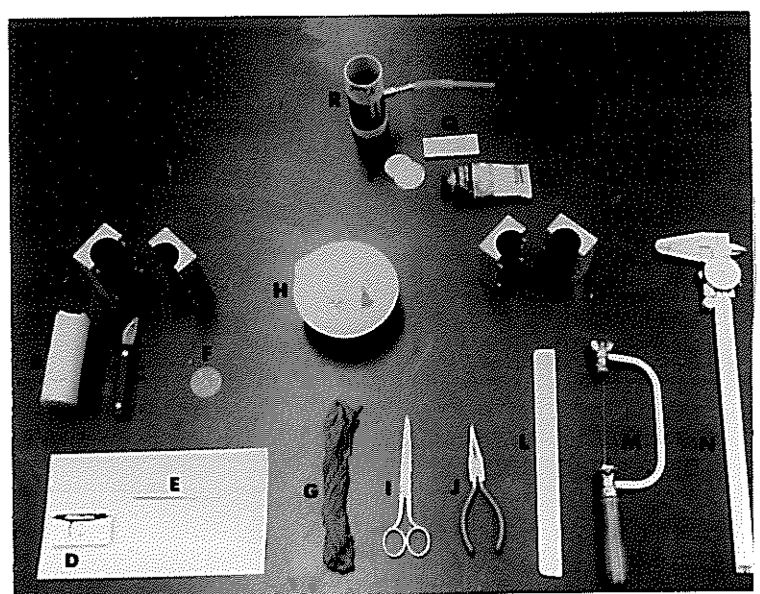


Figure IV-4. Equipment for Preparation of Kaolinite Sample. A=10 cm mold; B=Wax sample; C= Knive; D=Razer blades; E=0.18 cm needle; F=Template; G=Wool for drains; H=Saturated wool drains; I=Scissors; J=Pliers; K= 8 cm mold; L=Straightedge; M=Sample trimming devise; N=Vernier calipers; O=Silicone grease; P=Cut top and bottom membranes; Q=Membrane; R=Membrane in stretcher.

Sample with Artificial Bondings (Glass Beads plus Hydrocal)

Ho (1971) experimented with different materials before he decided on hydrated calcium sulphate, hydrocal, (gypsum $CA_2SO_42(H_2O)$), for the artificial bonding media. The same mixture was used in this project so it would be possible to compare the results. The glass beads and hydrocal were mixed in the dry in the proportion of 140 grams to 14 grams and then 27 cm^3 of distilled water added for producing one sample. The wet mixture was then immediately compacted with a vibro-tool in a split mold which had been lined with Saran wrap to prevent sticking of the sample to the sides of the mold. The sample was compacted in five layers and was allowed to set for half an hour before the mold was removed. Figure IV- 5 shows the equipment and materials used.

The sample was prepared at least 14 days before it was used in order to reduce any strength increase with time. A few samples were tested dry in an unconfined compression apparatus. The unconfined compression strength was approximately 10 kg/cm^2.

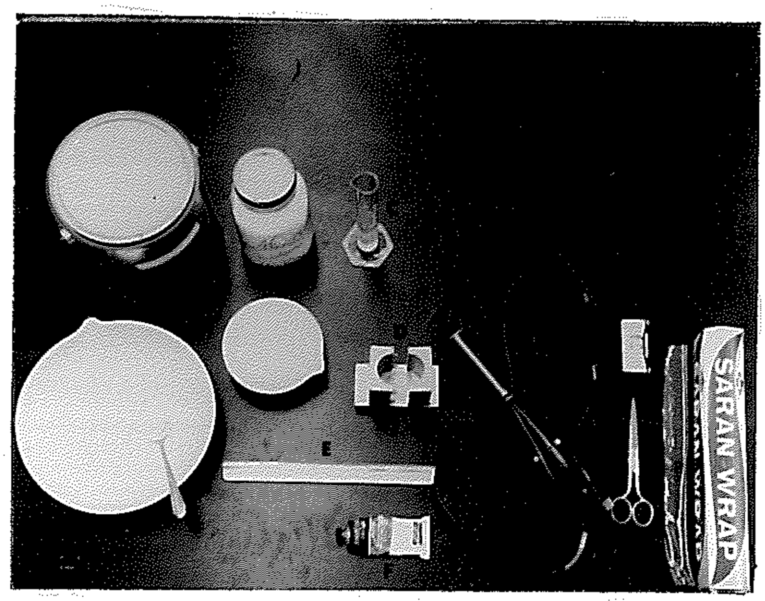


Figure IV-5. Equipment for Preparation of Glass Beads Plus Hydrocal Sample. A=Glass beads (140 g in dish); B=Hydrocal (14 g in dish); C=Distilled water; D=Split mold; E=Straightedge; F=Silicone grease; G=Vibrotool; H=Razer blades; I=Scissors; J=Saran wrap

The dry specimen was measured and installed in the triaxial cell and sealed with two membranes with grease between and three O-rings around pedestal and top cap. A frictionless connection of membranes and grease was established between sample and caps.

A vacuum of 8.5 inches of mercury was slowly applied to the top of the sample while water was admitted from the bottom. The water was supplied from a burette. Water was flushed through the sample for half an hour and after that period of time no more air appeared in the lines. Care was exercised to prevent cavitation of the bottom of the sample from the inrush of water. This method permitted Ho (1971) to obtain a degree of saturation of about 88 percent without backpressure. Figure IV-6 shows the set-up for the saturation process.

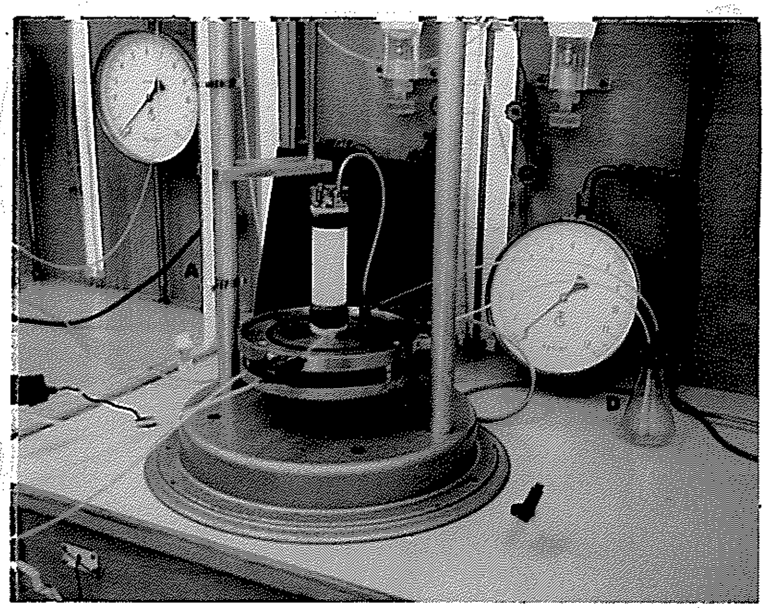


Figure IV-6. Saturation of Glass Beads plus Hydrocal Sample. A=Burette with water supply; B= Sample; C=Top cap with vacuum line; D= Bottle for collecting of water.

After saturating the sample, the cell was assembled and the hydrostatically consolidation pressure applied.

Equipment

The 'Geonor' triaxial equipment used in this research was developed by Norwegian Geotechnical Institute and has been described by Andresen and Simons (1960) among others. The modifications and changes described below were made to the triaxial equipment to give a either better precision or to allow possibilities for obtaining supplementary information.

Vertical Load Measurement Device

Strain gage load cells, which are hooked up through a bridge to a strain indicator, are used to measure the piston load instead of the normal proving rings. For the cohesive soil, a 100-lb capacity Thwing-Albert load cell with a sensitivity of 0.007496 kg/microinch/inch was used, while for cohesionless soils, a 200-lb capacity load cell with a sensitivity of 0.15108 kg/microinch/inch was employed. The readings on the strain indicator were reproducible to ± 1 division = ± 1 microinch/inch. The advantage with the load cells is that they deflect much less than the conventional proving rings.

In a few of the pilot tests, the load cells were connected to an electric recorder which was calibrated 5 kg/inch and 10 kg/inch respectively. The recorder gave the advantage of a continous registration of the load with time.

Figure IV-7 and IV-8 show the set-up with the strain indicator and with the recorder.

Vertical Deformation Measurement Device

The vertical deformation of the specimen was measured by the relative movement between the top of the piston and the bottom of the cell. This method of measuring the deflection eliminated the effects of cell deformation due to variation in cell pressures. A dial gage with an accuracy of 0.0001" = 0.000254 cm per division deflection was used. Figure IV-9 shows the arrangement.

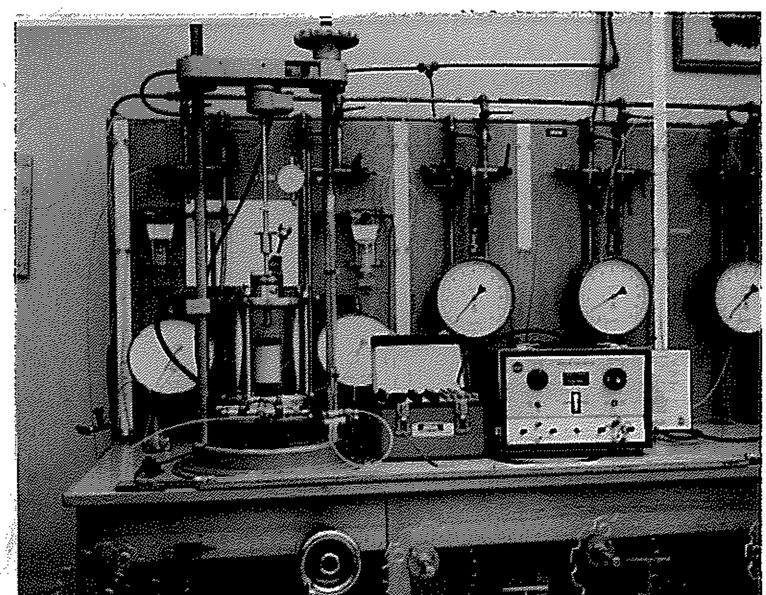


Figure IV-7. Test Set-up with Strain Indicator.

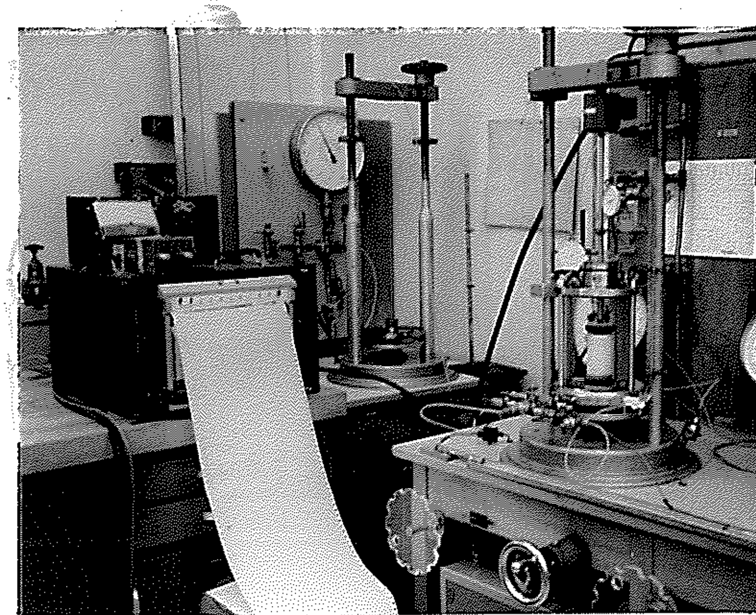


Figure IV-8. Test Set-up with Electric Recorder.

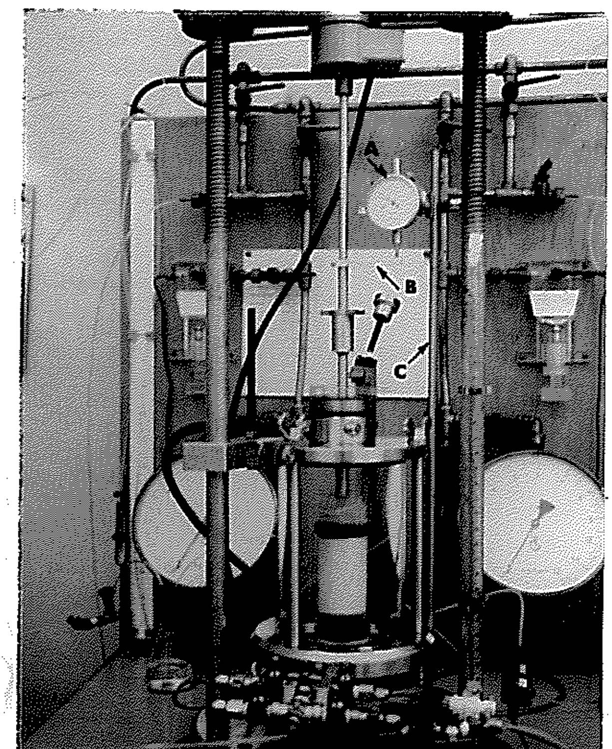


Figure IV-9. Vertical Deformation Measuring Instrument. A=Dial gage; B=Dial holder; C=Rod fastened to cell bottom.

Volume Change Measurements Device

In some of the preliminary tests, the volume changes due to increase in pore pressure were measured. This was done by a sensitive device designed by K. H. Ho at the University of Florida. The device consisted of an air bubble locked up in a 46 inch long glass capillary with an internal diameter of 1.0 mm, connected to two pairs of Circle Seal valves which, by closing one pair and opening the other pair, reverse the movement of the air bubble in the glass capillary without changing the movement of the water into or out of the sample. The device can be read to the nearest 0.01 inch and the calibration factor between bubble displacement and volume change was 0.0238 cm^3/inch. Figure IV-10 shows the device.

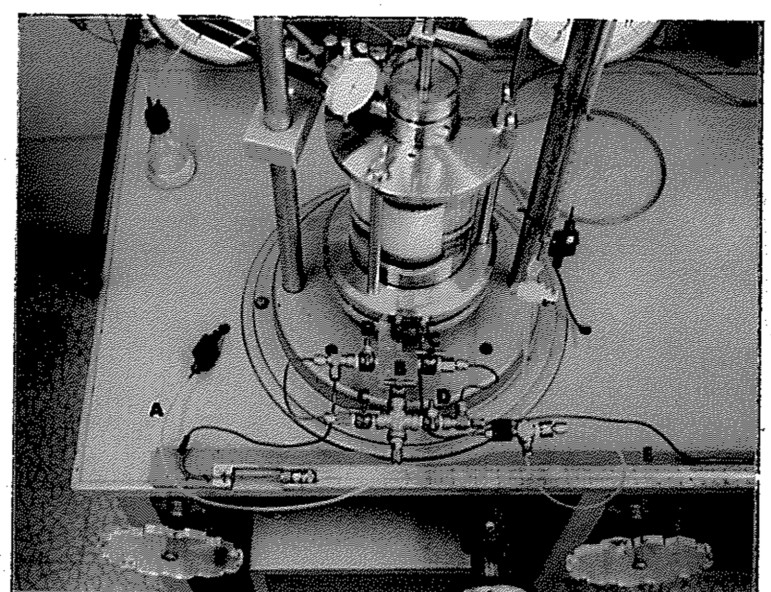


Figure IV-10. Volume Change Measuring Instrument. A=Main pore pressure line; B=Direct connection to sample; C-C=Corresponding pair of valves (opened); D-D=Corresponding pair of valves (closed); E=Glass capillary tube.

Constant Pressure Devices

Fluctuation in cell pressure due to volume changes of the sample or the cell, leakage, temperature variation or movement of the piston within the triaxial cell, were prevented by the displacement of the piston in the constant pressure cell described by Andresen and Simons (1960). The piston was checked for verticality before each test and during testing the table was frequently tapped in order to eliminate any friction.

The pore pressure was kept constant, during most of the test, by aid of a self-compensating mercury control (Bishop mercury pot). The principles and design of this system was described in detail by Bishop and Henkel (1964). The constant pore pressure could be controlled within ± 0.1 psi = ± 0.007 kg/cm^2 accuracy. The important thing was to keep the pressure difference, σ_3 - u constant.

Correction of Measured Data

When working with delicate equipment like the 'Geonor' triaxial cell where small quantities are measured, it is important to make corrections for the unwanted unavoidable effects which may influence the results. In this project, the following corrections were made.

Area Correction

In the kaolinite samples, 5 holes were punched with a 0.18 cm diameter needle for inserting of double wool drain. The 5 holes take up an area on 0.13 cm^2 which amounts to 1 1/4 percent of the cross section area for an average sample with 3.60 cm diameter. In calculation of the stresses, the area considered was reduced by the area of the holes. The area was also corrected for volumetric and vertical strain assuming that the sample deforms as a right circular cylinder with the aid of lubricated platens.

Rubber Membrane Correction

The rubber membrane added extra strength to the soil specimen. A method for calculation of the stress correction due to this effect was used and was based on elasticity principles and the following assumptions (Bishop and Henkel, 1964):

1) the membrane, when held against the sample by the

cell pressure, was capable of taking compression.

2) the sample deformed as a right cylinder.

The membrane correction per membrane for axial stress was given by

$$\Delta\sigma_{am} = -(2/3)E\ [1+2\varepsilon_{at} - \sqrt{\frac{1-\varepsilon_v}{1-\varepsilon_{at}}}\ ]\ \frac{A_m}{A_s(1-\varepsilon_v)}$$

and the correction for horizontal stress by

$$\Delta\sigma_{hm} = -(2/3)E\ [2+\varepsilon_{at} - 2\sqrt{\frac{1-\varepsilon_v}{1-\varepsilon_{at}}}\ ]\ \frac{2t_m}{r_s(1-\varepsilon_v)}$$

where:

E = Extension modulus value of the rubber membrane = 12.38 kg/cm^2

ε_{at} = Vertical strain during testing

ε_v = Volumetric strain during testing

A_m = Area of membrane

A_s = Area of sample

t_m = Thickness of membrane

r_s = Radius of sample.

Weights of Plunger and Rod

Weights of plunger and rod are accounted for in the computation of the deviator stress by taking the difference between the load cell reading at any strain and the initial reading before starting the test and after consolidation and applying of backpressure.

In cases where the samples are dead loaded with the hanger system the weights are included with the calculation of the hanger load.

Vertical Deformation

The measured vertical deformation had to be corrected for deflection on the top cap, ball joint, and frictionless connection under load. Ho (1971) obtained this correction by replacing the sample with an aluminum dummy which was loaded as in the actual test. He found a deformation correction which was linear with a slope of 0.0004 cm/kg.

Friction

The triaxial cell was equipped with a rotating bushing to reduce the plunger friction. Much care was taken for proper alignment of the triaxial assembly in order not to introduce any horizontal forces on the piston. The lubricated piston fall freely through the bushing under its own weight and when no horizontal forces acted on the plunger the friction was negligible (Andresen and Simons, 1960).

Temperature

A correction for the changes in temperature was not made since the effects on a complex system like this test arrangement are unknown. The effects are minimized by keeping a constant temperature (± 0.5 °C) in the laboratory.

CHAPTER V

TEST PROCEDURE

General

It is necessary to measure the shear resistance of the soil for different effective stress conditions without changing the structure of the soil in order to evaluate a constant-structure Mohr envelope. The fact that the effective stress equals the total stress minus the pore pressure makes it possible to establish different effective stress situations in a triaxial sample by changing the pore pressure in the specimen and keeping the cell pressure constant.

The test procedure evaluated during this research consisted of the following main steps: (see Figure V-1)

1. Hydrostatic consolidation (B to C)
2. Drained loading and waiting for equilibrium (C to D)
3. Increase of pore pressure in steps and waiting for equilibrium after each step (D to E,..., F to G)

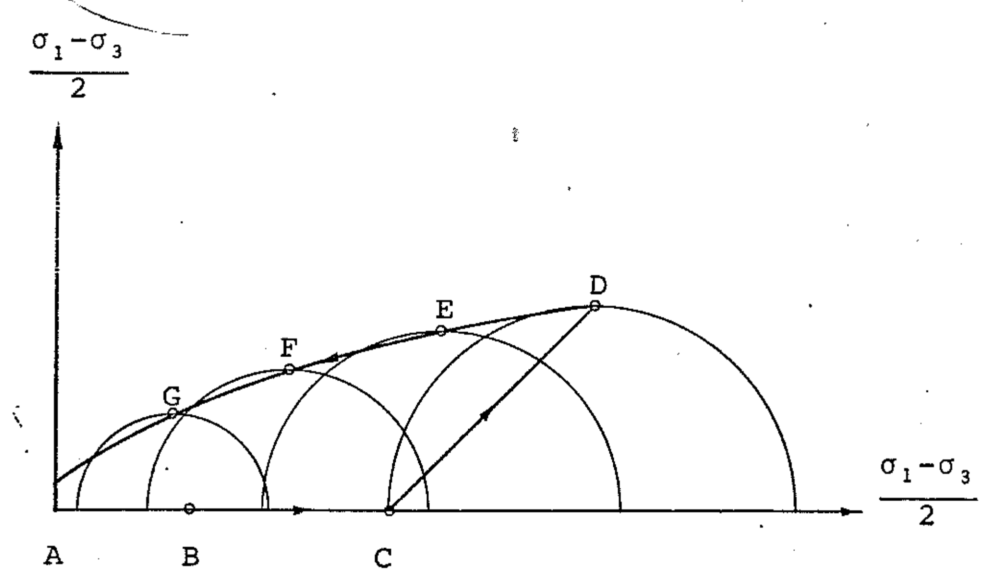


Figure V-1. Effective Stress path for CSE-test. B-C= Hydrostatic consolidation; C-D= Drained loading; D-G=Increasing of pore pressure.

In Figure V-1, the circles represent a two dimensional stress condition of equilibrium for the structure at the strain obtained by load to point D, because only a slight change in strain occurs during Step 3. An envelope formed by points of tangency to the Mohr circles representing stresses at a point at a specific value of strain is known as a constant-structure Mohr envelope.

By measuring the load after equilibrium condition has occurred after each pore pressure increment, the mean effective stress, $(\sigma_1'+\sigma_3')/2$, and shear stress, $(\sigma_1'-\sigma_3')/2$ can be computed and the Mohr circles drawn. The Mohr envelope which is tangent to the circles can then be constructed graphically.

The parameters in the equation for the constant-structure envelope

$$\tau = I_o + \alpha\sigma' - \beta\sigma'\ln\sigma' \qquad (5\text{-}1)$$

are obtained by a least square fit to the points of tangency and can be compared with the ones determined by Ho (1971) for the same soils.

Equipment

Performance of a CSE-test required a simple triaxial cell where both cell and pore pressure or the pressure difference, σ_3-u could be controlled and maintained constant automatically. The volume change equipment used in some of the tests in this research was unnecessary. It was found that the volume change in these soils during pore pressure increase with regard to area change had a negligible effect on the computation of stresses. The volume change during hydrostatic consolidation and loading was measured in an open burette connected to the bottom of the sample.

Glass Beads

Preliminary Tests

The first tests run on glass beads samples were conducted dry and with air as the pore pressure media. The sample was consolidated hydrostatically to σ_c and then loaded in drained compression to about 1/2 percent strain. After attaining an equilibrium condition where there was no further change in load with time, the pore pressure could be increased. It took more than 24 hours to reach a condition which could be considered to be satisfactory. The real test procedure then consisted of increasing the pore pressure in steps and waiting for equilibrium after each increment. When there was no further load change with time, the load on the sample was measured. The time for obtaining equilibrium for each such step was also around 24 hours.

A single test with 5 pore pressure increments required almost a week and it proved impossible to keep the sample dry for this amount of time with the available equipment. The sample became partially saturated and consequently the control of the effective stresses was lost. Water condensed from the air and/or penetration of water through the membranes might have been possible. Another disadvantage of using air as pore pressure media was that it became more difficult to measure the volume changes that occurred during consolidation, loading and testing. Due to these disadvantages, the dry sample and air system was dropped in favor of a saturated sample and water as pore pressure media. The investigation of bond strength done by Ho (1971) was also done with saturated samples.

Using a saturated sample speeded the test considerably. Approximately eight hours were necessary to obtain a satisfactory equilibrium condition both after straining and after each pore pressure increment.

Final Test

For tests to be performed on saturated samples and for further testing of cohesionless materials a procedure was adopted, which gave reproducible results that compared well with the ones determined by Ho. The procedure consisted of the following steps:

1. Flush water through all lines and connections to remove any air that might be trapped.

2. Set-up sample (see procedure on page 25).

3. Hydrostatic consolidation. Measure height and volume change.

4. Drained compression to about 1/2 percent strain or any other desirable strain. No back pressure was used in this series of tests. Measure height and volume change. A strain rate of close to 1/2 percent/hour is suitable.

5. Wait for equilibrium. A satisfactory equilibrium criteria determined and used in this research was that the change in load per hour should be less than or equal to 1 percent of the load change during the first minute after the compression was stopped. It was necessary to allow 8 hours for the glass beads. Measure load, height and volume change with time.

6. Close drainage to burette.

7. Increase pore pressure in steps and wait for equilibrium after each increment. The equilibrium criteria described in step 5 was used. A new pore pressure increment was made when the change in load per hour due to the previous increment was less than or equal to 1 percent of the change during the first minute after that increment. It was necessary (for the glass beads) to allow nearly 8 hours. Measure load and height change with time. It was desirable to have the last pore pressure value close to the cell pressure and thereby obtain a point on the stress path as near to origin as practicable. A typical plot of change in load with time for a high and low $\Delta\sigma_3'$ range is shown on Figure V-2.

8. Stop the test by closing the pore pressure line. Release pore pressure to burette. After putting approximately 20 cm of water suction on the sample, decrease cell pressure, empty and dismantle cell. Measure dimensions of sample before it was placed carefully in a dish for drying and weighing.

9. Calculate the test results, mean stress and shear stress, with the necessary corrections. In this project the computations were performed on a Wang 700 calculator. A copy of the program with description is given in Appendix C.

10. Draw Mohr circles to an appropriate scale and estimate best common tangent to the set of circles, and thereby obtain a constant-structure Mohr envelope.

11. Make best fit of the equation $\tau = I_o + \alpha\sigma' - \beta\sigma'\ln\sigma'$

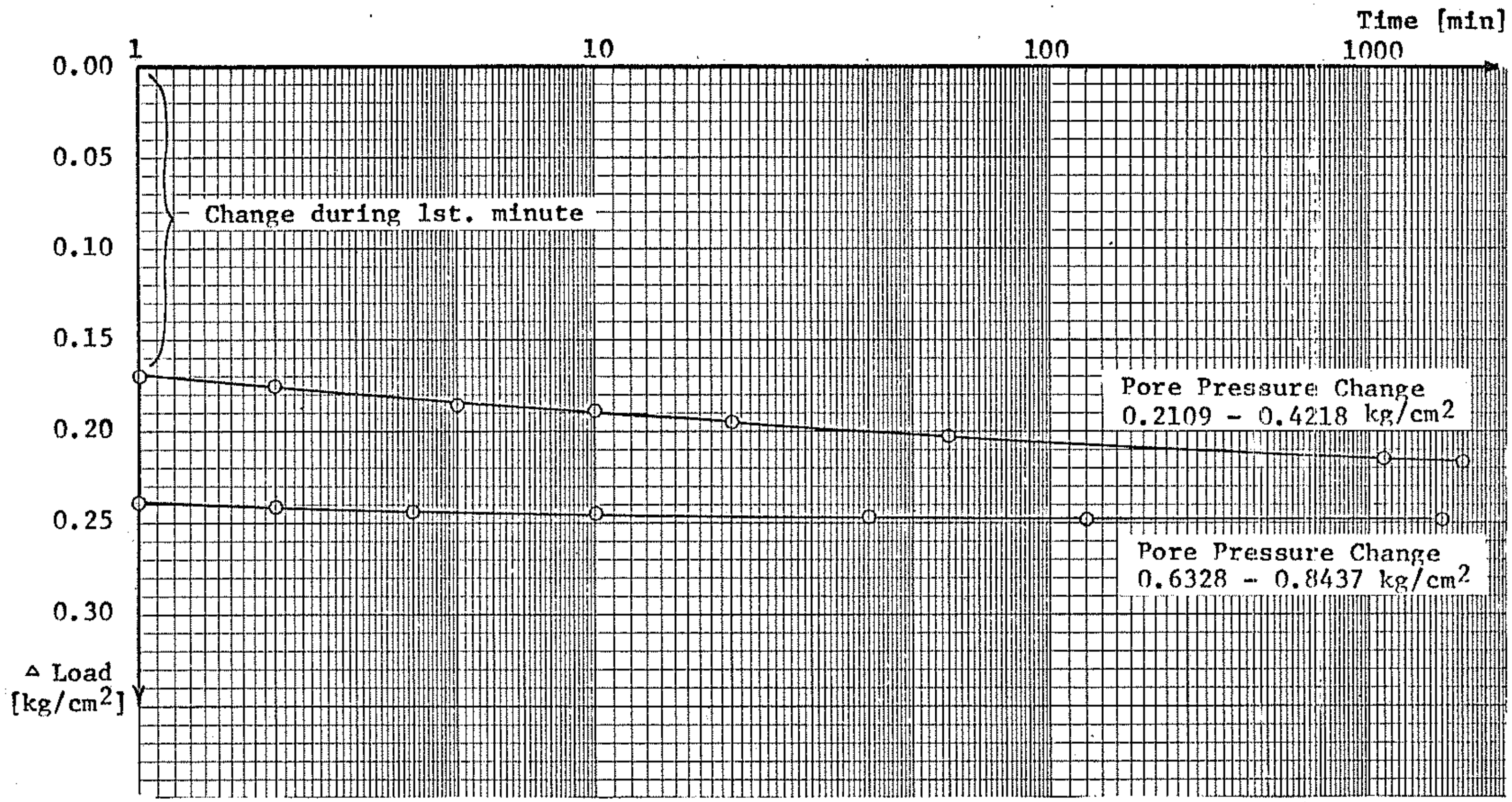


Figure V-2. Load Change versus Time for Glass Beads (Test No.29).

to the points of tangency and determine the soil parameters, I_o, α and β. In this project, the least square fitting was done with the aid of the Wang 700 calculator. A copy of the program with description is given in Appendix D.

The test data are recorded as shown in Appendix A, which contains data from 2 tests performed on glass beads using the procedure described above.

Kaolinite

Preliminary Test

The kaolinite samples were saturated and all tests were run with water as the pore pressure media. It was desired to perform the drained loading stage of the test through a dead weight system instead of compressing it to a predetermined strain. This method of loading shortened the time interval between end of loading and occurrence of equilibrium. The constant stress was allowed to act until a satisfactory equilibrium condition occurred between vertical deformation and time. The 'Geonor' triaxial cell can be equipped with a hanger system and the sample loaded with dead loads. The hanger system used in these tests had a lever arm ratio of 1 to 5 and is shown on Figure V-3.

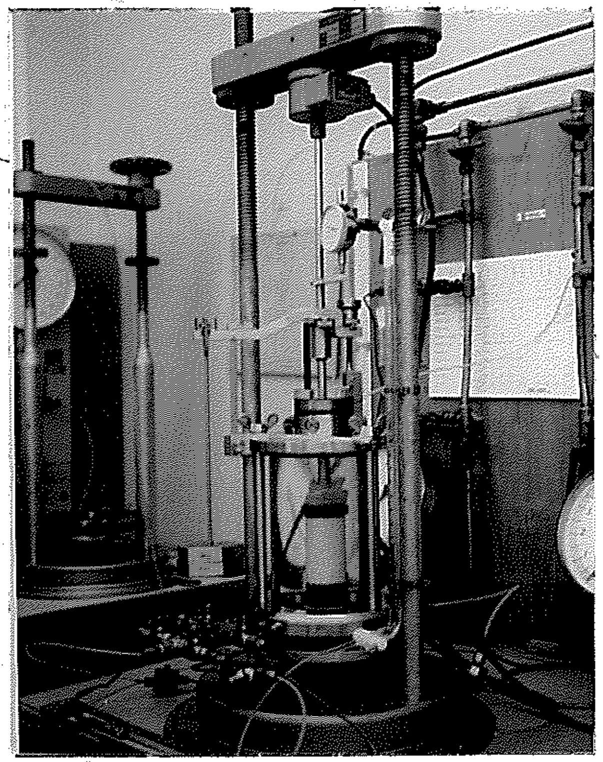


Figure V-3. Loading with Hanger System.

After reaching equilibrium, the load was transferred from the hanger system to the load cell and the sample was ready for stage 3 of the test, i.e., increase of pore pressure. This part of the test was performed as for the glass bead sample, only it was necessary to allow longer time for equilibrium condition to take place. The time required for equilibrium is a function of, among other things, permeability, soil structure and σ_1' and σ_3'.

Slightly overconsolidated samples were used (overconsolidation ratio=2), and the drained, axial compressive loading of the samples was made such that all the planes were prestressed by the hydrostatic consolidation compared to any stress condition appearing during the drained compression test. Figure

V-4 shows this effect, which reduces the time to equilibrium to 24 hours or less. Normally consolidated samples of kaolinite required at least 36 hours to reach the same difinition of equilibrium.

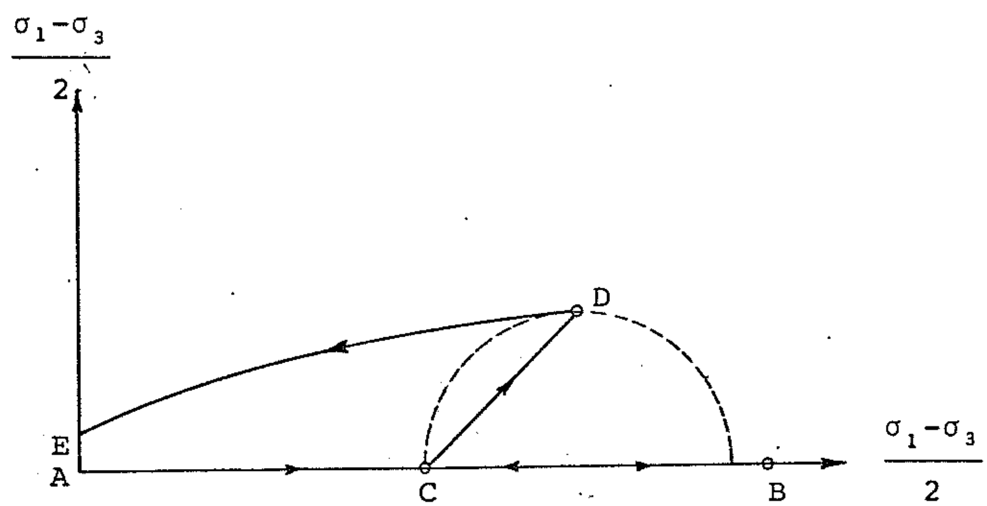


Figure V-4. CSE-test Stress Path. A-B=Consolidation; B-C=Rebound; C-D=Loading; D-E=Increase of pore pressure.

When working with a clay material, the problem of swelling may be encountered. A consolidation curve for the kaolinite is shown on Figure V-5.

In the described test where the sample height is kept constant while the effective stresses are decreased, the swelling effect will cause an additional unwanted strain of the sample.

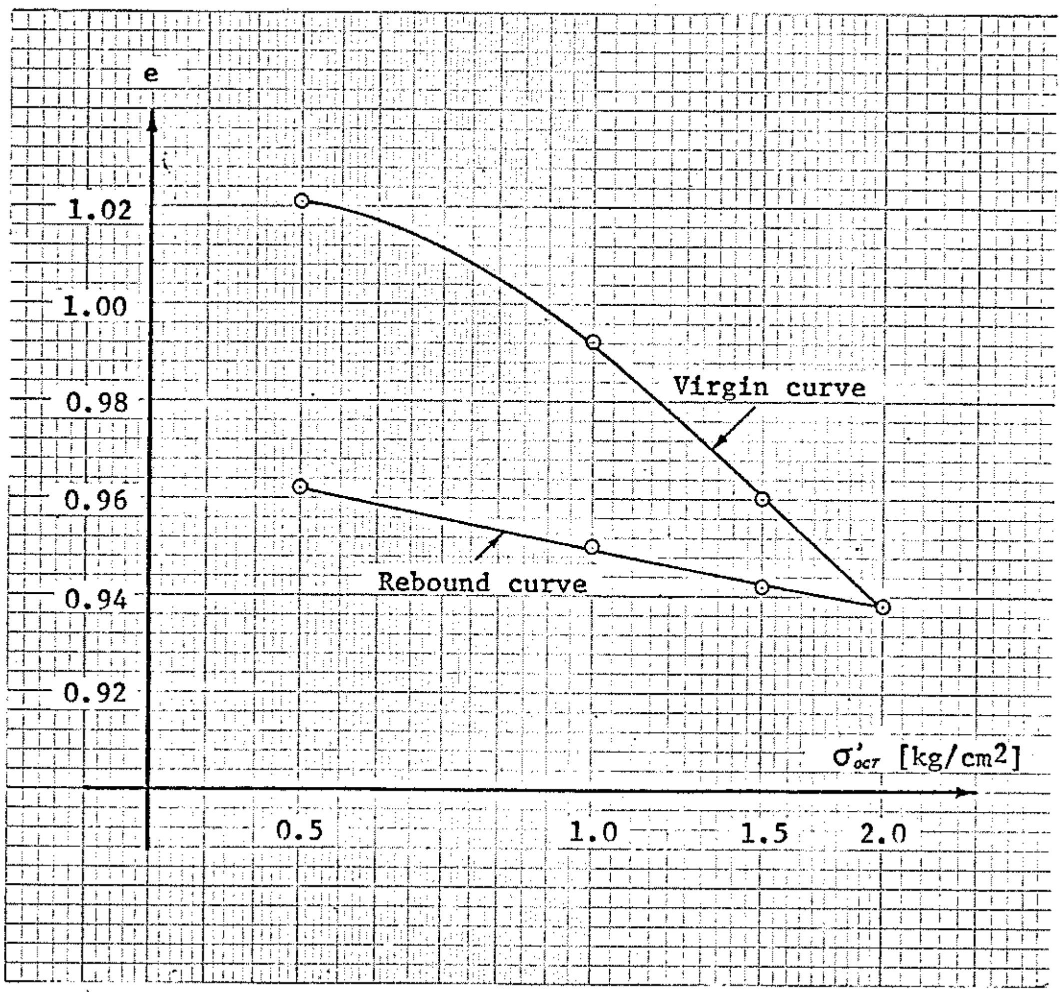


Figure V-5. Hydrostatic Consolidation Curve.

Two unsuccessful attempts were made to take account of the swelling, though for kaolinite, which has a low swell index, it might have a minor effect.

Attempt 1: A so-called "swell test" was performed before the CSE-test in order to correct it for swelling effects. The main object in the swell test was to establish stress conditions in the sample similar to that encountered during the CSE-test and to permit measurement of the height change of the specimen due to the increase of pore pressure (increase of volume).

In the swell test, the sample was hydrotatically consolidated to σ_c, rebounded to σ_r and reconsolidated to σ_{rc}, where

$$\sigma_{rc} = \frac{3\sigma_r + x(\sigma_c - \sigma_r)}{3} \tag{5-1}$$

where $x = \dfrac{(\sigma_1' - \sigma_3')}{(\sigma_c - \sigma_r)}$

In the CSE-test for which the swell test is done, the sample was consolidated to σ_c, rebounded to σ_r and loaded to a deviator stress, $(\sigma_1' - \sigma_3') = x(\sigma_c - \sigma_r)$. Following stress path takes place:

Consolidation: $\sigma_1' = \sigma_2' = \sigma_3' = \sigma_c = \sigma'_{oct}$

Rebound: $\sigma_1' = \sigma_2' = \sigma_3' = \sigma_r = \sigma'_{oct}$

Loading: $\sigma_2' = \sigma_3' = \sigma_r$

$$(\sigma_1' - \sigma_3') = x(\sigma_c - \sigma_r)$$

which gives

$$\sigma_1' = x\ \sigma_c + (1-x)\ \sigma_r$$

The octahedral stress at the start of the constant-structure envelope is

$$\sigma'_{oct} = \frac{3\sigma_r + x(\sigma_c - \sigma_r)}{3} \qquad (5\text{-}2)$$

In the swell test, the following stress path takes place:

Consolidation: $\sigma_1' = \sigma_2' = \sigma_3' = \sigma_c = \sigma'_{oct}$

Rebound: $\sigma_1' = \sigma_2' = \sigma_3' = \sigma_r = \sigma'_{oct}$

Reconsolidation: $\sigma_1' = \sigma_2' = \sigma_3' = \sigma_{rc} = \sigma'_{oct}$

For the swell test to have the same octahedral stress as the CSE-test, the reconsolidation pressure σ_{rc} is given by (5-1).

After reconsolidation to σ_{rc}, the pore pressure was increased in steps, the sample allowed to swell and after occurrence of equilibrium the height change was measured. A plot of strain versus octahedral stress was made.

It should now be possible in the real test before each pore pressure increment to correct for the distance between piston and bottom cap in the cell according to the change in octahedral stress and the curve plotted from the swell test.

This method had the disadvantages that the change in octahedral stress had to be predicted for determining the height change of the sample, and that one swell test was required for each CSE-test. This together with difficulties in performing the swell test and the uncertainty in whether the octahedral stress was determining the swell seemed to disqualify this method as too difficult and uncertain.

Attempt 2: In this method, the sample also was consolidated to σ_c, rebounded to σ_r and the pore pressure then increased in steps. After equilibrium occurred the force due to the swell pressure was measured by the load cell. It was the intention to subtract this swell pressure from the deviator stress measured during the CSE-test. Due to the small magnitude of the swell pressure (0.001-0.01 kg/cm^2) it was impossible to conduct good tests with the available equipment.

Final Test

Due to the difficulties with performing a swell test, the plan to correct for swelling was dropped and the following test procedure was accepted as satisfactory for evaluating a constant- structure envelope of a cohesive material with low swell index:

1. Flush water through all lines and connections to remove any air that might be trapped.

2. Set-up sample (see procedure on page 28).

3. Hydrostatic consolidation to σ_c. Measure height and volume change.

4. Rebound to σ_r. Measure height and volume change.

5. Drained compression with dead loads to a deviator stress less than $(\sigma_c-\sigma_r)$. No back pressure was used in this

series of tests. Measure height and volume change.

6. Wait for equilibrium. A satisfactory equilibrium criteria determined and used in this research was that the change in load per hour should be less than or equal to 1 percent of the total load change. For the kaolinite with 5 internal drains, it was necessary to allow 24 hours. Measure height and volume change.

7. Transfer load from hanger system to load cell.

8. Increase pore pressure in steps and wait for equilibrium after each increment. Due to swelling of the kaolinite after the increase of pore pressure, the equilibrium criteria for glass beads had to be modified to the same criteria as used under step 6. A new pore pressure increment was made when the change in load per hour due to the previous increment was less than or equal to 1 percent of the total change caused by that increment. For the kaolinite samples, it was necessary to allow 24 hours. Measure load and height change with time. As for the cohesionless material, it was desireable to have the last pore pressure value as close to the cell pressure as practicable and thereby obtain a point on the stress path near the origin. A typical plot of change in load with time for a high and low $\Delta\sigma_3'$ range is shown on Figure V-6.

9. Stop test by closing pore pressure line. Release pore pressure to burette. Decrease cell pressure, empty and dismantle cell. Remove sample. Weigh and measure sample before placing in oven for drying.

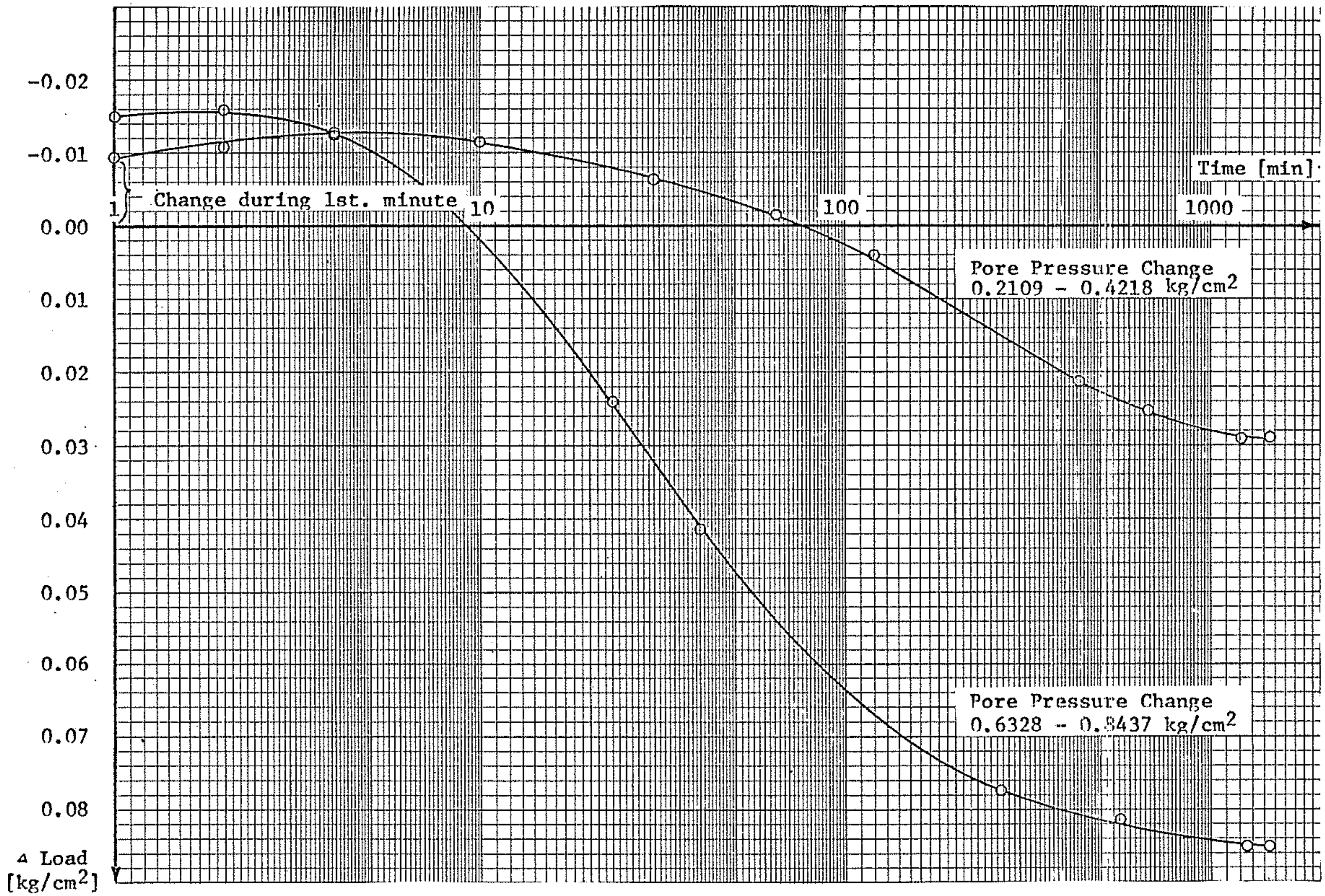


Figure V-6. Load Change versus Time for Kaolinite (Test No.28).

10. Calculate test results with corrections mentioned in Chapter IV.

11. Draw Mohr circles and estimate each common tangent graphically.

12. Fit the equation, $\tau = I_o + \alpha\sigma' - \beta\sigma' \ln\sigma'$ to points of tangency and determine soil parameters.

The test data are recorded as shown in Appendix B which contains data from 3 tests performed on kaolinite after above described procedure.

Glass Beads plus Hydrocal

Preliminary Tests

A single test was run dry. As with the glass beads it was not possible to keep the sample dry during such a long term test and the idea was dropped. Here again an advantage with the saturated specimens was that the results are directly comparable with Ho's tests. The pore pressure media does have some effect on the test results.

By running the tests on saturated samples the problem of getting the sample completely saturated was encountered. Ho obtained 88 percent saturation by suction of water under a small vacuum. By using a back pressure of 4 kg/cm^2, it was possible to bring the degree of saturation to 99 percent.

The tests were performed in a similar manner as the glass beads material except a back pressure of 4 kg/cm^2 was applied after consolidation and before compression. The first problem with this material was met at this point. The time necessary for reaching equilibrium after applying back pressure seems to be infinite concerning volume change.

One test was started without regard to the equilibrium after applying back pressure. The sample was compressed to about 1/2 percent strain and then the writer waited for equilibrium to occur. After 24 hours there was still no sign of equilibrium and the load continued to drop.

Check Test

A so-called check test was now performed. The load cell was connected to an electric recorder with less accuracy than the strain indicator but with the advantage of a continuous recording of the load. The chart paper moved at the rate of 2 inches/hour. The calibration factor for the load cell used with the glass beads plus hydrocal samples was 10 kg/inch.

After consolidating the sample to 1 kg/cm^2, a back pressure of 4 kg/cm^2 was applied and then 72 hours were allowed for equilibrium. After the 72 hours, there was still a movement of water into the sample with the speed of 0.006 cm^3/hour.

The specimen was compressed to approximately 1/2 percent strain and the load change recorded. After compression, the sample was left under constant strain until there was no further change in load with time.

After 22.5 hours the load carried by the strength of the sample decreased from 23.5 kg at the strain where compression was stopped to a value of 4.0 kg. Though the load was still decreasing, the pore pressure was now increased by 0.570 kg/cm^2 to 4.570 kg/cm^2 and the load change recorded. The load increased immediately and then started to decrease slowly. It took 36 hours before the load dropped back to the strength of the sample before the pore pressure increased and even then the load was still slowly decreasing.

This slow reaction, which made the test technique impractical, may have been caused by the sample not being properly saturated and/or an unstable chemical condition that occurred in the sample and/or the special soil structure.

It was not possible to adopt the test procedure introduced with the glass beads sample to the glass beads plus hydrocal specimens and determine a constant-structure envelope. Due to lack of time, the problem of a suitable CSE-test on this material was left unsolved.

CHAPTER VI

EXPERIMENTAL RESULTS

This chapter presents the results of a few CSE-tests performed according to the procedures described earlier. For each test, the Mohr's circles were constructed. The graphically estimated constant-structure envelope (solid line) as well as the least square fitted (dotted line) constant-structure envelopes were drawn. Two tests were conducted on glass beads and three on kaolinite. The results are summarized in Tables 1 and 2 on the next page.

The soil parameters, I_o, α and β were found by a least square fit of the equation $\tau = I_o + \alpha\sigma' - \beta\sigma'\ln\sigma'$ to the experimental results. The multiple correlation coefficients R are given in the tables. The bond strength, I_o was also determined as the intercept of the τ-axis by graphical extrapolation of the constant-structure envelope (solid line).

Water contents, degree of saturation and void ratios, as they were recorded during the test procedure, are shown in Appendix E.

The results are discussed and compared with the ones found by Ho (1971).

Table 1

Test Results for Glass Beads

Test No.	σ_c kg/cm²	$\bar{\varepsilon}$ %	$\Delta\varepsilon_{max}$ %	Graph. I_o kg/cm²	Least Square Fit I_o kg/cm²	α	β	R
24	1	0.52	0.15	-0.005	-0.013	0.605	0.076	0.9992
29	1	1.00	0.10	-0.010	-0.015	0.628	0.040	0.9999

Table 2

Test Results for Kaolinite

Test No.	σ_c kg/cm²	σ_r kg/cm²	$\bar{\varepsilon}$ %	$\Delta\varepsilon_{max}$ %	Graph. I_o kg/cm²	Least Square Fit* I_o kg/cm²	α	β	R
12	4	2	3.82	0.07	0.048	-0.007	0.443	0.151	0.9997
28	2	1	2.22	0.15	0.020	0.011	0.320	0.134	1.0000
31	2	1	0.87	0.10	0.010	0.010	0.260	0.077	0.9997

*Neglecting first point after drained comp~~arison~~ression, whcih was always low for unknown reasons.

Glass Beads

The results of two CSE-tests showed small negative values for the bond strength, I_o. The β-parameter was low, which indicated that the constant-structure envelope was near a straight line with the slope α. Assuming β equals zero, the least square line gives a correlation coefficient above 0.9975 for both tests. α was found to be approximately equal to 0.6. The same general results were found by Ho (1971). He performed only constant-σ_3' IDS tests on the glass bead material.

Kaolinite

The results of three CSE-tests showed that the bond strength I_o, appeared to be very small and even negative in the test where no point on the stress path was obtained near the origin. The nature of the constant-structure envelope equation tends to draw the envelope against small shear strength values for effective stresses near zero. Therefore, it was important for the last pore pressure increment to come as near the cell pressure as possible. $\sigma_3' = 0$ when pore pressure equals cell pressure. Figure VI-1 shows the constant-structure envelopes from test No. 28 near the origin. The data suggests from graphical curvefitting that I_o decreases with decreasing strain.

The α-parameter, which is an indicator of the slope of

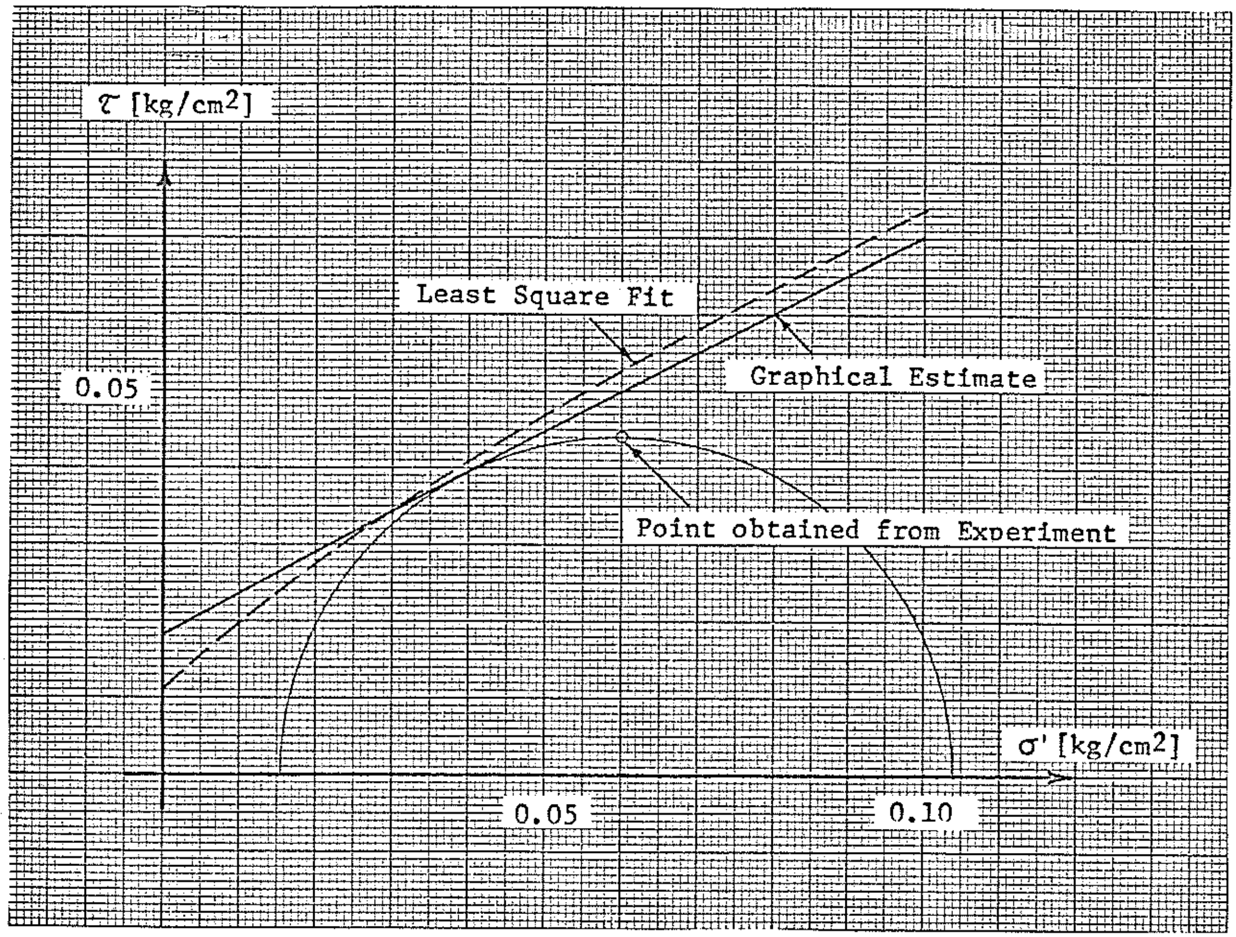


Figure VI-1. CSE near Origin from Test No.28.

the constant-structure envelope, varied between 0.25 and 0.45 and increased with increasing confining pressure and strain.

The β-parameter, which is a measurement of the curvature of the constant-structure envelope, varied between 0.07 and 0.15 and appeared also to increase with increasing strain. The β-parameter falls outside the range reported by Schmertmann (1966b) of 0.15 to 0.20. Ho reported $\beta = 0.05$ to 0.35.

When plotting the stress path and fitting the envelope, it was found that one of the first two points after compression did not fit in a smooth curve with the others. Since the first test point was better defined and the second one inexplicably low, the curve was made to fit the first point.

Ho performed three different series of tests. A constant-σ_1', a constant-p' and a constant-volume series. He worked with both normally and overconsolidated samples. The overconsolidated samples had an overconsolidation ratio of 3, 6, and 12.

None of Ho's tests are directly comparable with the CSE-tests which may be considered as a constant-σ_3 test. The samples used in the CSE-tests all had an overconsolidation ratio of 2, but from an overall viewpoint the α and β soil parameters found here were in the same range as those determined by Ho (1971). The bond strength, I_o appeared to be lower than found in earlier research. However Ho also obtained small and even negative values for I_o in some of his test on overconsolidated kaolinite.

Table 3 Results from Test No. 24

TEST NO. 24	STAGE NO.			
PT NO.	STRAIN	PORE PRESS	MEAN STRESS	SHEAR STRESS
1	.00460	.00000	1.95720	.95720
2	.00473	.35160	1.35242	.70402
3	.00504	.56260	.91306	.47566
4	.00525	.70320	.61292	.31612
5	.00565	.84390	.31117	.15507
6	.00612	.98450	.01880	.00330

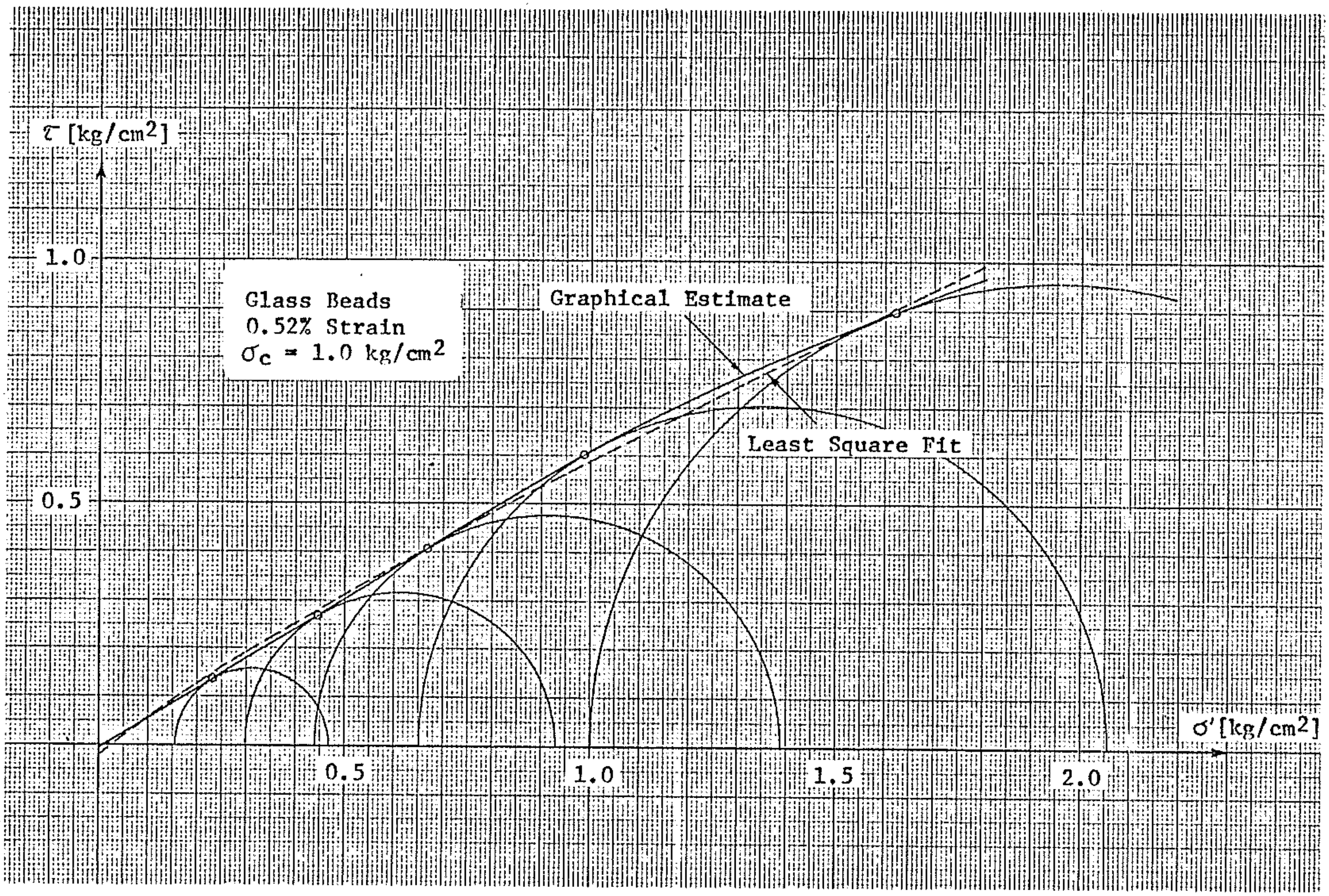


Figure VI-2. Constant-Structure Envelope from Test No.24.

Table 4 Results from Test No. 29

TEST NO. 29	STAGE NO.			
PT NO.	STRAIN	PORE PRESS	MEAN STRESS	SHEAR STRESS
1	.00951	.00000	2.05690	1.05690
2	.00961	.21090	1.65573	.86662
3	.00971	.42180	1.22712	.64892
4	.01012	.63280	.77679	.40959
5	.01040	.84370	.31680	.16049
6	.01048	.98420	.02262	.00681

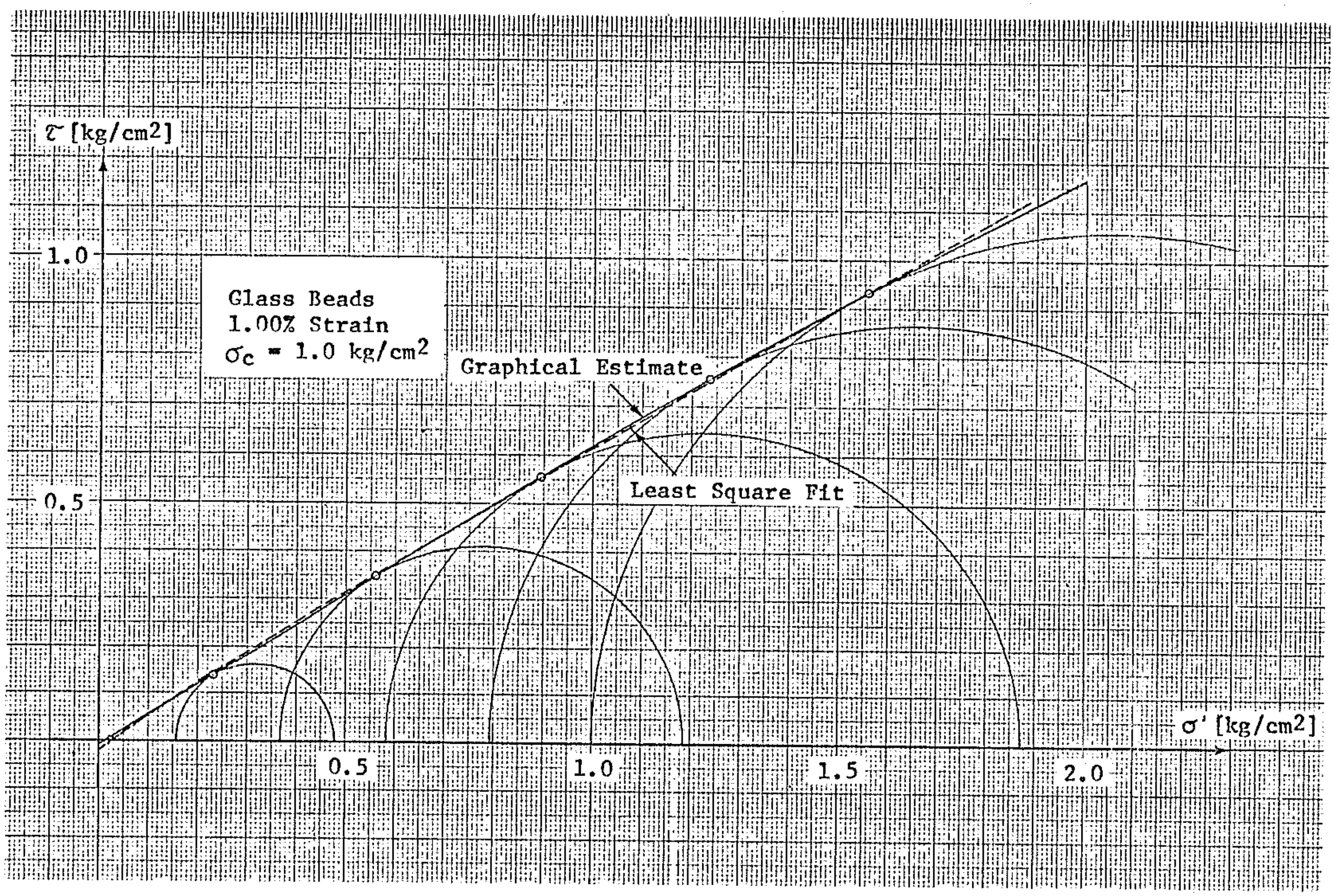


Figure VI-3. Constant-Structure Envelope from Test No.29.

Table 5 Results from Test No. 12

TEST NO. 12	STAGE NO.			
PT NO.	STRAIN	PORE PRESS	MEAN STRESS	SHEAR STRESS
1	.03784	.00000	2.79206	.79200
2	.03806	1.00000	1.54603	.54597
3	.03823	1.30000	1.17343	.47337
4	.03849	1.60000	.71656	.31650
5	.03852	1.90000	.23667	.13661

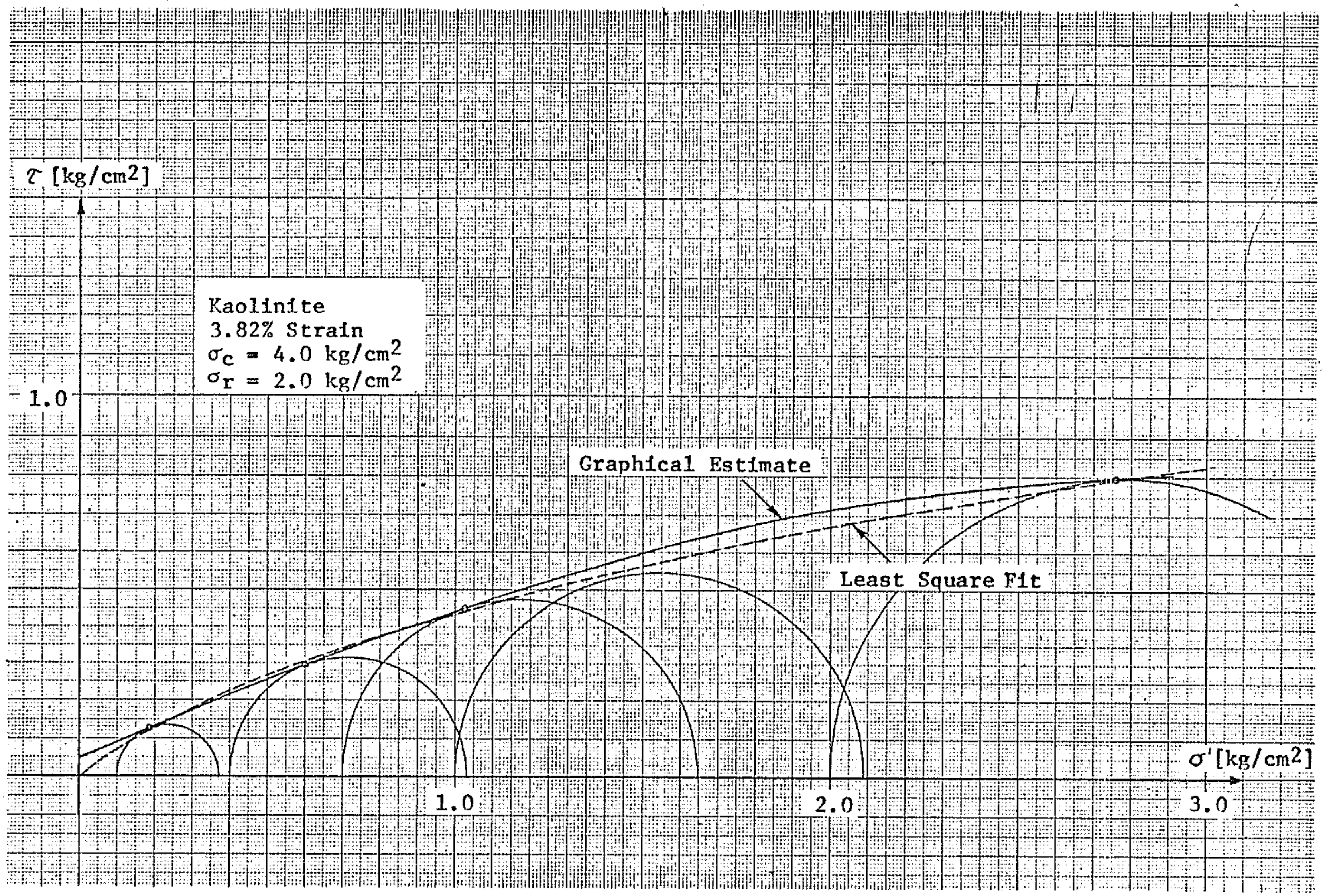


Figure VI-4. Constant-Structure Envelope from Test No.12.

Table 6 Results from Test No. 28

TEST NO. 28	STAGE NO.			
PT NO.	STRAIN	PORE PRESS	MEAN STRESS	SHEAR STRESS
1	.02134	.00000	1.38977	.38975
2	.02169	.21090	1.11936	.33024
3	.02204	.42180	.87945	.30123
4	.02210	.63280	.60182	.23460
5	.02219	.84370	.30580	.14948
6	.02291	.98430	.05967	.04395

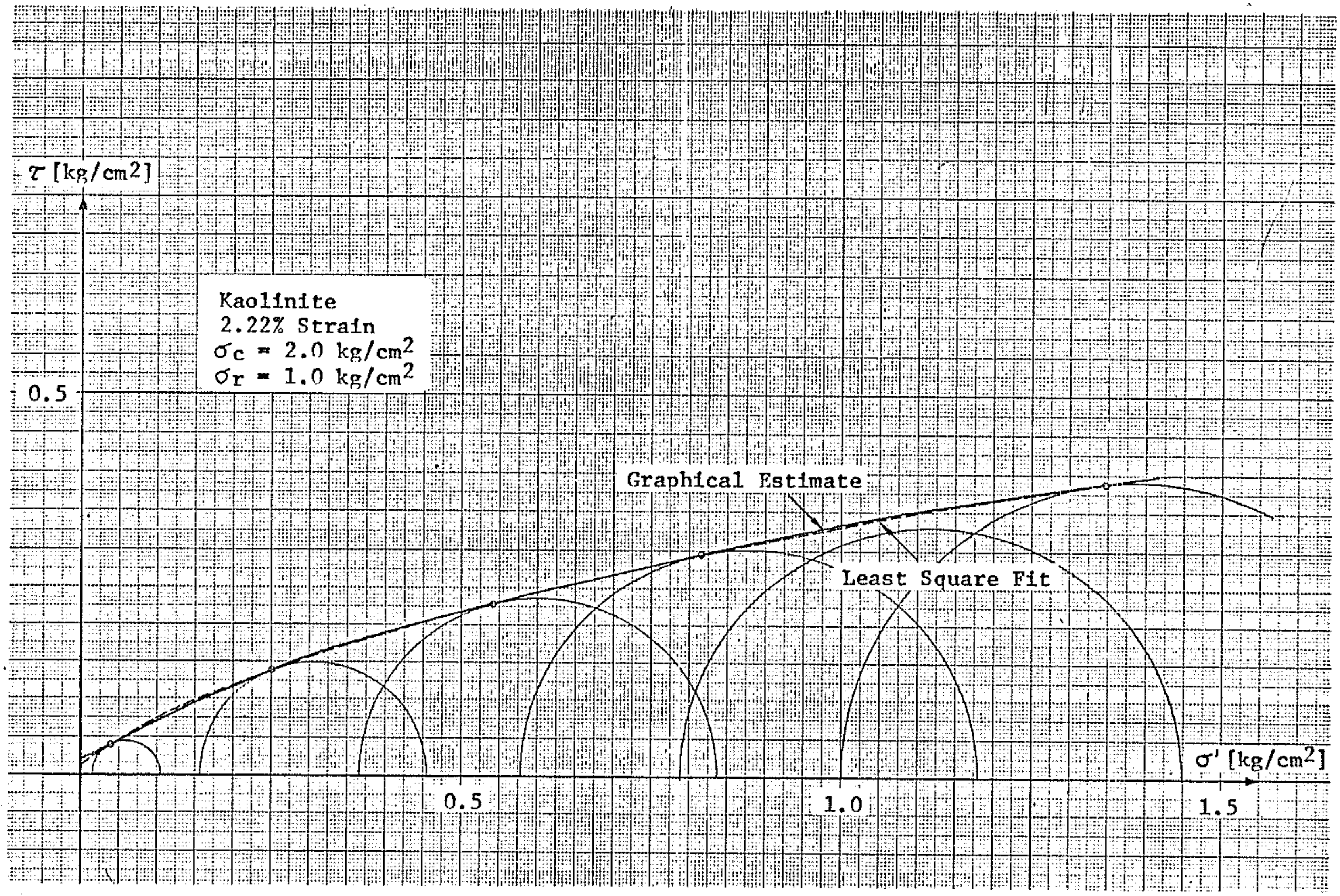


Figure VI-5. Constant-Structure Envelope from Test No.28.

Table 7 Results from Test No. 31

TEST NO. 31		STAGE NO.		
PT NO.	STRAIN	PORE PRESS	MEAN STRESS	SHEAR STRESS
1	.00826	.00000	1.32375	.32375
2	.00832	.21090	1.05330	.26420
3	.00837	.42180	.80192	.22372
4	.00861	.63280	.53935	.17215
5	.00869	.84370	.25929	.10299
6	.00922	.98430	.04067	.02496

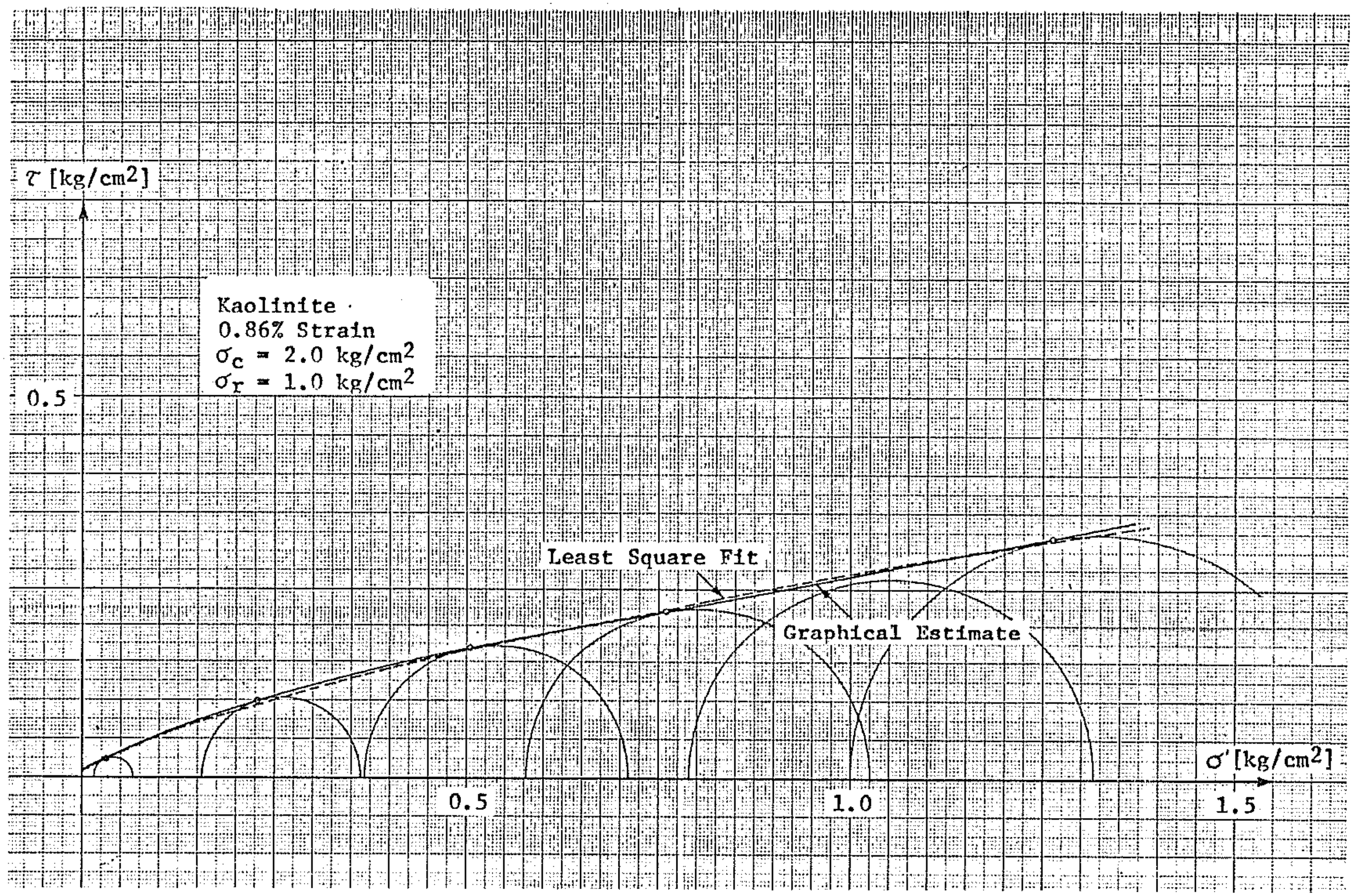


Figure VI-6. Constant-Structure Envelope from Test No.31.

CHAPTER VII

DISCUSSION AND CONCLUSION

Discussion

In this research project, an attempt was made to develop a test procedure for determining a constant-structure envelope for different soisl. Three materials were chosen, cohesionless, a cohesive clay and a material with artificial bondings.

Cohesionless Glass Beads

A successful procedure was found for the saturated glass beads material. A CSE-test with approximately 5 pore pressure increments could be performed during 4 days.

A more convenient procedure using a dry sample and with air as the pore pressure medium was not possible with techniques used. Water penetrated the sample during a long term test. A dry test may be conducted successfully using air as the confining medium.

Cohesive Soil - Kaolinite

A test procedure was introduced for determining a constant-structure envelope for a clay with a low swelling index.

Two different ways were attempted to correct for the swell of the material, but were unsucessful. The swelling of the kaolinite, which was relatively small, together with the precision of the equipment were, in the writer's opinion, the reasons for the unfortunate results. A test with a more expanding material may be of help in determining the swell correction though many investigators have pointed out that it is difficult to measure the swell effect in the triaxial cell.

One of the objections with the CSE-test for a cohesive material is the long time a test takes. A CSE-test could be performed in remolded kaolinite in 7 days. The test time perhaps can be reduced by using external filter drains but these have the disadvantage of strengthening the sample and a correction for this strength is difficult to determine.

Artificial Bondings

It was not possible for the author to develop a test procedure for determining a constant-structure envelope for the material with artificial bond during the time allowed for the test program.

In the test procedure, one of the main steps was the attaining of equilibrium conditions at a certain soil structure, effective stress and strength of the specimen. Applying these test methods to the glass beads plus hydrocal samples did not work out since a satisfactory equilibrium condition took an impractically long time.

At the present time, the author has no suggestions to solve the problem with such a material.

General

Many different test techniques were tried before the final procedures were established. This permitted the investigator to gain experience in handling the triaxial cell. This is necessary for the conduct of a sophisticated test like the CSE-test without introducing operator uncertainties.

The main purpose of this form of testing was to maintain a constant soil structure for different effective stresses and measure the strength of the sample for that particular structure. It turned out to be impossible to keep a constant strain. The absolute variation was between 0.07 and 0.15 percent. The effect of change in strain depends on how steep the stress-strain curve is at the particular strain of the test.

During the testing and curve fitting, it was established that it was of great importance to obtain the last point on the stress path as close to the origin as possible.

The main objection to the test procedures is the time problem. A CSE-test in a spherical, uniform cohesionless material will take 4 days and for a remolded kaolinite, cohesive material, 7 days, when the samples are saturated.

The bond strength, I_o determined from the least square fit appears too low compared with results from earlier investigationd done on the same materials. Perhaps a lower limit of the constant-structure envelope equation also exists. There is definitely an upper limit for the equation since at higher effective stresses it starts to give decreasing shear strength values.

Conclusions

1. The existance of Schmertmann's imaginary constant-structure envelope has been proved.

2. It was possible to determine a constant-structure envelope for a cohesionless saturated glass beads material and a cohesive saturated kaolinite clay.

3. Good correlation was found between Schmertmann's constant-structure envelope equation and the experimental results.

4. A constant-structure envelope could not be determined experimentally for the material with artificial bonding, which was made up of glass beads and hydrocal.

5. The most exciting prospect seems to be the investigation of undisturbed soil. The CSE-test developed makes such a study possible since only one sample is necessary for determining the constant-structure envelope and thereby the soil parameters, including bond strength.

CHAPTER VIII

SUGGESTIONS FOR FUTURE RESEARCH

An obvious suggestion for future research is the performance of CSE-test on other materials including undisturbed soils. Tests performed for different strain values of the same soil or even same sample would be desireable. Data sheets and annotated Wang 700 programs are included as appendices in this thesis in order to help future researchers. Such an investigation may help to explain the mechanism of shear strength of soils.

The author would also like to see additional research on swelling clays to complement this study and help develop a CSE-test for swelling clay.

APPENDIX A

DATA SHEETS FOR CSE-TEST ON GLASS BEADS

University of Florida

SOIL MECHANICS LABORATORY

Test no. 24
Test type CSE
Spec. gravity 2.43

Name H.S.
Date 2/9-71
Job M.T.

SAMPLE PROPERTIES BEFORE TEST:

Height,H 7.836 cm
Diameter,D 3.564 cm

Volume,V_i 78.17 cm^3
Area,A_i 9.98 cm^2

Weight of mold + sample ______ g
Weight of mold ______ g
Weight of sample,W_i ______ g

Water content,w_i 23.2 %
Void ratio,e_i 0.564
Saturation,S_i 100 %

CONSOLIDATION:

Isotropic 1.0 kg/cm^2
Rebounding ______ kg/cm^2

DRAINAGE:

Internal ______ Top ______
External ______ Bottom X

Height change 0 cm
Height,H_c 7.836 cm
Strain,ε_c 0 %

Volume change 0.98 cm^3
Volume,V_c 77.19 cm^3
Area,A_c 9.85 cm^2

LOADING:

Height change 0.040 cm
Height,H_1 7.796 cm
Strain,ε_1 0.51 %

Volume change -0.15 cm^3
Volume,V_1 77.34 cm^3
Area,A_1 9.92 cm^2

SAMPLE PROPERTIES AFTER TEST:

Weight of wet sample + tare ______ g
Weight of tare ______ g
Weight of wet sample,W_{fw} ______ g

Weight of dry sample + tare 625.10 g
Weight of tare Dish # 17 503.63 g
Weight of dry sample,W_{fd} 121.47 g

Water content,w_f 22.9 %
Void ratio,e_f 0.555
Saturation,S_f 100%

Average measured height,H_f 7.788 cm
Average measured diameter,D_f 3.565 cm
Average calculated volume,V_f 77.74 cm^3

REMARKS: ______

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Test No. 24
Material Glass Beads

Name H.S
Date 22/9-71

Application of Deviator Stress by Straining

Triaxial Cell No. 3
Confining Pressure 10 kg/cm²
Loading Condition ½% strain
Rate of Compression 5%/hr

Start Compress. Read. 228 div
Final Compress. Read. 382 div
Compression Reading .000254 cm/div
LMD Reading .001508 kg/div

Time	LMD Reading [div]	Volume Reading [cm³]	Vertical Deformation Reading [div]
8.18	29576	18.95	228
	29665	18.95	238
	30196	18.93	262
	30435	18.93	278
	30578	18.93	295
	30735	18.99	318
	30855	19.01	342
	30912	19.03	358
	30949	19.08	371
9.38	30985	19.10	382

Equilibrium after Straining

Time	Δ Time [min]	LMD Reading [div]	Δ LMD Reading [div]	Volume Reading [cm³]	Vertical Deformation Reading [div]
9.38		30985	0	19.10	382
	1	30950	35	-	382
9.41	3	30935	50	-	-
9.45	7	30920	65	-	-
9.58	20	30900	85	-	383
10.38	60	30888	97	-	384
11.38	120	30876	109	-	-
13.38	240	30874	111	19.11 *	-
16.38	410	30874	111	19.13 *	385
22.15	757	30830	155	19.18 *	-
23/9 8.20	1312	30815	170	19.20 *	-
11.25	1497	30842	143	19.22 *	-
15.30	1742	30850	135	19.2 *	-
19.00	1952	30846	139	19.28 *	-
22.30	2162	30837	148	19.30 *	-
24/9 8.30	2762	30804	181	19.34 *	-
13.00	3032	30833	152	19.35 *	-

Remarks: Temperature effect investigated   * Evaporation

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Increasing of Pore Pressure

Test No. 24

Material Glass Beads

Date 24/9 - 71

Name H.S

Triaxial Cell No. 3

Confining Pressure 10 kg/cm²

LMD Calibration 0015108 kg/div

Start LMD Reading 29576 div

Time	△ Time [min]	Pore Press. [kg/cm²]	LMD Reading [div]	△ LMD Reading [div]	Vertical Deformation Reading [div]
13.05		0	30833		385
		0.3516			-
	1		30560	273	-
	3		30543	290	-
	6		30535	298	-
13.20	15		30521	312	-
14.05	60		30508	325	-
15.05	120		30502	331	-
16.05	180		30499	334	-
17.05	240		30499	334	-
		0.5626			391
	1		30245	254	-
	3		30233	266	-
17.11	6		30229	270	-
17.25	20		30219	280	-
18.05	60		30213	286	-
19.10	125		30207	292	-
22.30	325		30200	299	-
22.43		0.7032			-
	1		30030	170	-
	3		30023	177	-
22.50	7		30016	184	-
23.00	17		30010	190	-
25/9 7.30	527		29991	209	395
9.00	617		29991	209	-
		0.8439			405
	1		29799	192	-
	3		29794	197	-
9.07	7		29790	201	-
9.25	25		29787	204	-
12.15	195		29784	207	-
13.00	240		29784	207	-
23.20	860		29780	211	-
		0.9845			412
	1		29596	184	-
23.23	3		29590	190	-
23.27	7		29590	190	-
26/9 7.40	500		29581	199	417
13.24	845		29581	199	-

Remarks:

University of Florida

SOIL MECHANICS LABORATORY

Test no.	29	Name	H.S
Test type	CSE	Date	21/10-71
Spec. gravity	2.43	Job	M.T

SAMPLE PROPERTIES BEFORE TEST:

Height, H	7.924 cm	Volume, V_i	78.65 cm^3
Diameter, D	3.555 cm	Area, A_i	9.93 cm^2
Weight of mold + sample	g	Water content, w_i	21.87 %
Weight of mold	g	Void ratio, e_i	0.5314
Weight of sample, W_i	g	Saturation, S_i	100 %

CONSOLIDATION:

Isotropic 1.0 kg/cm^2
Rebounding kg/cm^2

DRAINAGE:

Internal ____ Top ____
External ____ Bottom x

Height change	0.005 cm	Volume change	1.61 cm^3
Height, H_c	7.919 cm	Volume, V_c	77.04 cm^3
Strain, ε_c	0.06 %	Area, A_c	9.73 cm^2

LOADING:

Height change	0.080 cm	Volume change	-0.32 cm^3
Height, H_1	7.839 cm	Volume, V_1	77.36 cm^3
Strain, ε_1	1.01 %	Area, A_1	9.87 cm^2

SAMPLE PROPERTIES AFTER TEST:

Weight of wet sample + tare	g		
Weight of tare	g		
Weight of wet sample, W_{fw}	g		
Weight of dry sample + tare	617.15 g	Water content, w_f	23.00 %
Weight of tare	492.35 g	Void ratio, e_f	0.5589
Weight of dry sample, W_{fd}	124.80 g	Saturation, S_f	100 %

Average measured height, H_f 7.844 cm
Average measured diameter, D_f 3.605 cm
Average calculated volume, V_f 80.06 cm^3

REMARKS:

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Test No. 29
Material Glass Beads

Name H.S.
Date 3/10-71

Application of Deviator Stress by Straining

Triaxial Cell No. 3
Confining Pressure 1.0 kg/cm²
Loading Condition 1% strain
Rate of Compression 0.5%/hr

Start Compress. Read. 1047 div
Final Compress. Read. 1359 div
Compression Reading .000254 cm/div
LMD Reading 0.015108 kg/div

Time	LMD Reading [div]	Volume Reading [cm³]	Vertical Deformation Reading [div]
13.47	29595	19.30	1047
14.00	29782	19.28	1068
14.17	29980	19.28	1110
14.30	30220	19.28	1135
14.43	30385	19.28	1154
14.55	30630	19.31	1194
15.25	30915	19.41	1261
15.55	31075	19.58	1332
16.06	31120	19.62	1360

Equilibrium after Straining

Time	Δ Time [min]	LMD Reading [div]	Δ LMD Reading [div]	Volume Reading [cm³]	Vertical Deformation Reading [div]
16.06	0	31120	0	19.62	1360
	1	31082	38	-	-
	3	31055	65	-	-
16.11	5	31048	72	-	-
16.16	10	31030	90	19.60	-
16.41	35	31006	114	-	-
15.01	55	3 996	124	-	-
17.42	96	30984	136	-	-
20.06	240	30970	150	19.65 *	-
22.36	390	30960	160	19.70 *	-
23.50	464	30956	164	19.71 *	-
9.36	1050	-	-	19.82 *	-
11.46	1180	-	-	19.93 *	-

Remarks: * Evaporation

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Increasing of Pore Pressure

Test No. 29
Material Glass Beads
Date 22/10 - 71
Name H.S.

Triaxial Cell No. 3
Confining Pressure 1.0 kg/cm²
LMD Calibration 0.015108 kg/div
Start LMD Reading 29575 div

Time	△ Time [min]	Pore Press. [kg/cm²]	LMD Reading [div]	△ LMD Reading [div]	Vertical Deformation Reading [div]
11.46		0	30956		1360
		0.2109			-
	1		30760	196	-
	2		30760	196	-
11.50	4		30758	198	-
12.01	15		30748	208	-
12.16	30		30742	214	-
12.46	60		30738	218	-
13.26	100		30730	226	-
15.16	210		30725	231	-
16.46	300		30720	236	-
17.46	360		30716	240	-
21.46	600		30706	250	-
24.00	734		30703	253	-
		0.4218			-
	1		30482	221	-
	2		30475	228	-
	5		30463	240	-
	10		30457	246	-
	20		30450	253	-
23/10 1.00	60		30440	263	-
9.00	540		30423	280	-
13.22	802		30420	283	-
		0.6328			-
	1		30143	277	-
	2		30139	281	-
	4		30130	290	-
13.30	8		30124	296	1369
13.37	15		30122	298	-
16.42	200		30109	311	-
21.41	499		30109	311	-
		0.8437			-
	1		29798	311	-
	2		29795	314	1374
21.45	4		29792	317	-
21.51	10		29789	320	-
22.21	40		29787	322	-
23.41	120		29785	324	-
24/10 9.36	715		29785	324	-

Remarks:

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CSE - TEST

Increasing of Pore Pressure

Test No. 29
Material Glass Beads
Date 24/10 - 71
Name H.S.

Triaxial Cell No. 3
Confining Pressure 10 kg/cm^2
LMD Calibration 0015108 kg/div
Start LMD Reading 29515 div

Time	△ Time [min]	Pore Press. [kg/cm^2]	LMD Reading [div]	△ LMD Reading [div]	Vertical Deformation Reading [div]
936	0	09842			1374
	1		29597	188	-
	2		29595	190	-
9.40	4		29595	190	-
9.46	10		29593	192	-
10.01	25		29588	197	-
11.16	100		29588	197	-
13.16	220		29585	200	-
15.41	365		29585	200	-
2226	770		29585	200	-

Remarks:

APPENDIX B

DATA SHEETS FOR CSE-TEST ON KAOLINITE

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SOIL MECHANICS LABORATORY

Test no. 12
Test type CSE
Spec. gravity 2.609

Name H.S.
Date 9/8-71
Job M.T.

SAMPLE PROPERTIES BEFORE TEST:

Height, H 7.990 cm
Diameter, D 3.617 cm

Volume, V_i 82.10 cm^3
Area, A_i 10.28 cm^2

Weight of mold + sample 503.01 g
Weight of mold 355.10 g
Weight of sample, W_i 147.91 g

Water content, w_i 37.7 %
Void ratio, e_i 0.993
Saturation, S_i 98.9 %

CONSOLIDATION:

Isotropic 4.0 kg/cm^2
Rebounding 2.0 kg/cm^2

DRAINAGE:

Internal 5
External
Top
Bottom x

Height change 0.147 cm
Height, H_c 7.843 cm
Strain, ε_c 1.84 %

Volume change 4.31 cm^3
Volume, V_c 77.79 cm^3
Area, A_c 9.92 cm^2

LOADING:

Height change 0.304 cm
Height, H_1 7.539 cm
Strain, ε_1 3.88 %

Volume change 0.46 cm^3
Volume, V_1 77.33 cm^3
Area, A_1 10.26 cm^2

SAMPLE PROPERTIES AFTER TEST:

Weight of wet sample + tare 167.97 g
Weight of tare # 149 20.30 g
Weight of wet sample, W_{fw} 147.67 g

Weight of dry sample + tare 127.75 g
Weight of tare 20.30 g
Weight of dry sample, W_{fd} 107.45 g

Water content, w_f 37.4 %
Void ratio, e_f 0.973
Saturation, S_f 100.4 %

Average measured height, H_f 7.571 cm
Average measured diameter, D_f 3.696 cm
Average calculated volume, V_f 81.26 cm^3

REMARKS:

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Test No. 12 Name H.S.
Material Kaolinite Date 10/8-71

Application of Deviator Stress by Hanger System

Triaxial Cell No. 1 Loading Condition 80% $(\sigma_1 - \sigma_3)$
Constant Dead Load 0.492 kg Confining Pressure 2.0 kg/cm²
Uplift on Piston 1.900 kg Area of Sample 10 cm²

Time	Hanger Load [kg]	Δ Piston Load [kg]	Piston Load [kg]	$(\sigma_1 - \sigma_3)A$ [kg]	Volume Reading [cm³]	Vertical Deform. Reading [div]
10.10	0	0	0.492	0	22.00	1161
	Hanger + arm *					
	0.500	3.940	4.432	2.532		
10.20					22.00	1166
	0.500	2.500	6.932	5.032		
10.45					22.00	1166
	1.000	5.000	11.932	10.032		
10.50					21.98	1315
11.30					21.92	1379
	1.194	5.968	17.900	16.000		
11.40					21.79	2215

Equilibrium after Loading

Time	Δ Time [min]	Volume Reading [cm³]	Vertical Deform. Reading [div]	Δ Vertical Deform. Reading [div]
11.40	10	21.79	2215	836
12.45	75	21.53	2319	940
14.20	170	21.50	2334	955
16.20	290	21.49	2344	965
21.15	685	21.50 *	2354	975
11/8 7.50	1220	21.54 *	2356	977

Remarks: * Evaporation

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Transformation of Hanger Load

Test No. 12
Material Kaolinite
Name H.S
Date 11/8-71

Constant Dead Load 0.492 kg
Piston Load 17.900 kg
Start LMD Reading 29500 div
LMD Calibration .0007496 kg/div

Time	Dead Load [kg]	Indicator Load [kg]	Δ Load [kg]	Δ LMD Reading [div]	LMD Reading [div]	Vertical Deform. Reading [div]
8.15	17.900	0			29500	2356
			5.968	796		
	11.932	5.968			30296	
			5.000	667		
	6.932	10.968			30963	
			2.500	334		
	4.432	13.468			31297	
			3.940	525		
8.20	0.492	17.408			31822	2355

Remarks: No volume change during transformation. Burette closed.

Equilibrium after Transformation

Time	Δ Time [min]	Volume Reading [cm³]	Vertical Deform. Reading [div]	ΔVertical Deform. Reading [div]
8.20	0	21.54	2355	0
8.45	25	-	-	0
9.10	50	-	-	0
10.30	130	-	-	0

Remarks: Load kept constant.

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CSE - TEST

Increasing of Pore Pressure

Test No. 12
Material Kaolinite
Date 11/8 - 71
Name H.S.

Triaxial Cell No. 1
Confining Pressure 2.0 kg/cm²
LMD Calibration .0007496 kg/div
Start LMD Reading 29675 div

Time	△ Time [min]	Pore Press. [kg/cm²]	LMD Reading [div]	△ LMD Reading [div]	Vertical Deformation Reading [div]
10.30		0	31822		2355
		1.0			-
10.55	25		31496	326	2358
11.15	45		31390	432	2359
12.30	120		31305	517	-
13.35	185		31273	549	2355
15.35	305		31221	601	-
17.25	415		31192	630	-
12/8 0.05	815		31167	655	2354
		1.3			-
10.10	605		30974	193	-
13.05	780		30970	197	2357
		1.6			-
14.35	90		30644	326	2360
16.05	180		30592	378	-
19.20	375		30546	424	-
22.20	555		30544	426	-
		1.9			-
13/8 9.15	595		30075	469	-
9.30	670		30064	480	-
13.30	910		30058	486	2355
16.15	1075		30055	489	-

Remarks:

University of Florida

SOIL MECHANICS LABORATORY

Test no. 28
Test type CSE
Spec. gravity 2.609

Name H.S.
Date 11/10-71
Job M.T.

SAMPLE PROPERTIES BEFORE TEST:

Height, H 7.992 cm
Diameter, D 3.615 cm

Volume, V_i 82.03 cm^3
Area, A_i 10.26 cm^2

Weight of mold + sample ______ g
Weight of mold ______ g
Weight of sample, W_i 147.19 g

Water content, w_i 38.5 %
Void ratio, e_i 1.014
Saturation, S_i 99.0 %

CONSOLIDATION:

Isotropic 2.0 kg/cm^2
Rebounding 1.0 kg/cm^2

DRAINAGE:

Internal 5
External ______
Top ______
Bottom x

Height change 0.071 cm
Height, H_c 7.921 cm
Strain, ε_c ______ %

Volume change 2.58 cm^3
Volume, V_c 79.45 cm^3
Area, A_c ______ cm^2

LOADING:

Height change 0.172 cm
Height, H_1 7.749 cm
Strain, ε_1 ______ %

Volume change 0.32 cm^3
Volume, V_1 79.13 cm^3
Area, A_1 ______ cm^2

SAMPLE PROPERTIES AFTER TEST:

Weight of wet sample + tare 168.84 g
Weight of tare 20.30 g
Weight of wet sample, W_{fw} 148.54 g

Weight of dry sample + tare 126.58 g
Weight of tare 20.30 g
Weight of dry sample, W_{fd} 106.28 g

Water content, w_f 39.8 %
Void ratio, e_f 1.010
Saturation, S_f 102.7 %

Average measured height, H_f 7.781 cm
Average measured diameter, D_f 3.660 cm
Average calculated volume, V_f 81.86 cm^3

REMARKS: ______

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Test No. 28 Name H.S
Material Kaolinite Date 12/10-71

Application of Deviator Stress by Hanger System

Triaxial Cell No. 1
Constant Dead Load 0.492 kg
Uplift on Piston 0.950 kg
Loading Condition 80% $(\sigma_c-\sigma_r)$
Confining Pressure 6.0 kg/cm²
Area of Sample 10 cm²

Time	Hanger Load [kg]	Δ Piston Load [kg]	Piston Load [kg]	$(\sigma_1-\sigma_3)A$ [kg]	Volume Reading [cm³]	Vertical Deform. Reading [div]
8.55	0	0	0.492	0	19.42	1237
	Hanger + arm	1.440	1.932	0.982		
9.00					19.41	1240
	0.500	2.500	4.432	3.482		
9.05					19.40	1251
9.10					19.40	1251
	0.500	2.500	6.932	5.982		
9.15					19.39	1301
9.20					19.39	1320
	0.400	2.000	8.932	7.982		
9.25					19.24	1778

Equilibrium after Loading

Time	Δ Time [min]	Volume Reading [cm³]	Vertical Deform. Reading [div]	Δ Vertical Deform. Reading [div]
9.25	5	19.24	1778	458
9.30	10	19.21	1800	480
9.40	20	19.20	1814	494
9.55	35	19.14	1834	514
10.30	70	19.12	1853	533
11.10	110	19.10	1867	547
11.55	155	19.10	1873	553
12.55	215	19.10	1879	559
14.35	315	19.10	1887	567
16.15	415	19.10	1888	568
21.40	740	19.12 *	1895	575
9.35	1455	19.18 *	1902	582

Remarks: * Evaporation

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Transformation of Hanger Load

Test No. 28
Material Kaolinite
Name H.S
Date 8/11-71

Constant Dead Load 0.492 kg
Piston Load 8.932 kg
Start LMD Reading 29423 div
LMD Calibration 0.007496 kg/div

Time	Dead Load [kg]	Indicator Load [kg]	Δ Load [kg]	Δ LMD Reading [div]	LMD Reading [div]	Vertical Deform. Reading [div]
9.35	8.932	0			29423	1902
			2.000	267		
	6.932	2.000			29690	
			2.500	334		
	4.432	4.500			30024	
			2.500	333		
	1.932	7.000			30357	
			1.440	192		
9.40	0.492	8.440			30549	1910

Remarks: No volume change during transformation. Burette closed.

Equilibrium after Transformation

Time	Δ Time [min]	Volume Reading [cm³]	Vertical Deform. Reading [div]	ΔVertical Deform. Reading [div]
9.40	0	19.18	1910	0
10.40	60	19.18	1915	5
12.40	180	19.18	1915	5

Remarks: Load kept constant.

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CSE - TEST

Increasing of Pore Pressure

Test No. 28
Material Kaolinite
Date 13/10 - 71
Name H.S

Triaxial Cell No. 1
Confining Pressure 10 kg/cm²
LMD Calibration 0.007496 kg/div
Start LMD Reading 29496 div

Time	△ Time [min]	Pore Press. [kg/cm²]	LMD Reading [div]	△ LMD Reading [div]	Vertical Deformation Reading [div]
		0	30549		1915
12.50		0.2109		0	1922 *
	1		30586	-37	1923
	2		30596	-47	-
	4		30590	-41	-
13.00	10		30565	-16	-
13.20	30		30515	34	1924
13.55	65		30484	65	-
15.00	130		30463	86	-
18.45	355		30433	116	-
21.05	500		30420	129	-
14/10 8.30	1180		30392	157	-
12.50	1440		30393	158	-
		0.4218		0	-
	1		30418	-25	-
	2		30422	-29	-
	4		30427	-34	-
13.00	10		30424	-31	-
	30		30410	-17	-
13.55	65		30397	-4	-
14.50	120		30382	11	-
20.05	435		30335	58	1927
15/10 0.05	675		30325	68	1929
9.00	1210		30314	79	1934
13.00	1450		30315	80	-
		0.6328			-
	1		30350	-35	-
	3		30355	-40	-
	6		30348	-33	-
	15		30305	10	-
	30		30270	45	-
14.20	80		30210	105	-
15.15	135		30192	123	-
16.00	180		30180	135	-
20.20	440		30150	165	-
23.50	650		30140	175	-
16/10 8.40	1180		30135	180	-
12.20	1400		30135	180	-

Remarks: * Jumped when motor started

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Increasing of Pore Pressure

Test No. 28
Material Kaolinite
Date 16/10-71
Name H.S.

Triaxial Cell No. 1
Confining Pressure 1.0 kg/cm²
LMD Calibration 0.007496 kg/div
Start LMD Reading 29496 div

Time	Δ Time [min]	Pore Press. [kg/cm²]	LMD Reading [div]	Δ LMD Reading [div]	Vertical Deformation Reading [div]
12.20		0.8437			1934
	1		30175	-40	–
	2		30178	-43	–
	4		30168	-33	–
	8		30142	-7	–
	23		30070	65	–
13.00	40		30023	112	–
16.45	265		29925	210	–
21.45	565		29915	220	–
17/10 9.20	1260		29905	230	–
12.10	1430		29905	230	–
		0.9843			–
	1		29931	-26	–
	2		29930	-25	–
	4		29914	-9	–
	8		29870	35	–
12.25	15		29818	87	–
12.50	40		29710	195	1949
13.15	65		29667	238	–
15.05	175		29645	260	1951
17.05	295		29642	263	–
22.45	635		29631	274	–
18/10 8.45	1235		29621	284	1953
12.35	1465		29620	285	–

Remarks:

University of Florida

SOIL MECHANICS LABORATORY

Test no. 31
Test type CSE
Spec. gravity 2.609

Name H.S.
Date 28/10 - 71
Job H.T.

SAMPLE PROPERTIES BEFORE TEST:

Height, H 7.921 cm
Diameter, D 3.643 cm

Volume, V_i 82.56 cm^3
Area, A_i 10.42 cm^2

Weight of mold + sample ______ g
Weight of mold ______ g
Weight of sample, W_i 147.12 g

Water content, w_i 38.1 %
Void ratio, e_i 1.023
Saturation, S_i 97.3 %

CONSOLIDATION:

Isotropic 2.0 kg/cm^2
Rebounding 1.0 kg/cm^2

DRAINAGE:

Internal 5   Top ____
External ____   Bottom x

Height change 0.069 cm
Height, H_c 7.852 cm
Strain, ε_c 0.87 %

Volume change 2.25 cm^3
Volume, V_c 80.31 cm^3
Area, A_c 10.23 cm^2

LOADING:

Height change 0.068 cm
Height, H_1 7.784 cm
Strain, ε_1 -0.87 %

Volume change 0.16 cm^3
Volume, V_1 80.15 cm^3
Area, A_1 10.30 cm^2

SAMPLE PROPERTIES AFTER TEST:

Weight of wet sample + tare 181.08 g
Weight of tare 32.10 g
Weight of wet sample, W_{fw} 148.98 g

Weight of dry sample + tare 138.60 g
Weight of tare 32.10 g
Weight of dry sample, W_{fd} 106.50 g

Water content, w_f 39.9 %
Void ratio, e_f 1.033
Saturation, S_f 100.7 %

Average measured height, H_f 7.777 cm
Average measured diameter, D_f 3.686 cm
Average calculated volume, V_f 82.99 cm^3

REMARKS: ____

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Test No. 31
Material Kaolinite

Name HS
Date 29/10-71

Application of Deviator Stress by Hanger System

Triaxial Cell No. 1
Constant Dead Load 0.492 kg
Uplift on Piston 0.950 kg

Loading Condition ~ 1% strain
Confining Pressure 1.0 kg/cm²
Area of Sample 10 cm²

Time	Hanger Load [kg]	Δ Piston Load [kg]	Piston Load [kg]	$(\sigma_1 - \sigma_3)A$ [kg]	Volume Reading [cm³]	Vertical Deform. Reading [div]
9.40	0	0	0.492	0	18.23	1022
	Hanger + arm	1.440	1.932	0.982		
9.50					18.22	1022
	0.500	2.500	4.432	3.482		
10.00					18.21	1024
10.05					18.19	1028
	0.500	2.500	6.932	5.982		
10.10					18.15	1094
10.47					18.11	1152
11.20					18.10	1161
	0.100	0.500	7.432	6.482		
11.25					18.10	1174

Equilibrium after Loading

Time	Δ Time [min]	Volume Reading [cm³]	Vertical Deform. Reading [div]	ΔVertical Deform. Reading [div]
11.25	5	18.10	1174	13
12.45	85	18.08	1222	61
17.15	355	18.07	1254	93
19.20	480	18.07	1259	98
21.15	595	18.07	1259	98
22.30	670	18.08 *	1261	100
9.05	1305	18.10 *	1267	106

Remarks: * Evaporation

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Transformation of Hanger Load

Test No. 31
Material Kaolinite
Name H.S.
Date 30/10-71

Constant Dead Load 0.492 kg
Piston Load 7.432 kg
Start LMD Reading 29520 div
LMD Calibration 0.007496 kg/div

Time	Dead Load [kg]	Indicator Load [kg]	Δ Load [kg]	Δ LMD Reading [div]	LMD Reading [div]	Vertical Deform. Reading [div]
940	7.432	0			29520	1267
			0.500	67		
	6.932	0.500			29587	
			2.500	334		
	4.432	3.000			29921	
			2.500	333		
	1.932	5.500			30254	
			1.440	192		
945	0.492	6.940			30446	1274

Remarks: No volume change during transformation. Burette closed.

Equilibrium after Transformation

Time	Δ Time [min]	Volume Reading [cm³]	Vertical Deform. Reading [div]	Δ Vertical Deform. Reading [div]
945	0	18.10	1274	0
1250	185	18.10	1278	4

Remarks: Load kept constant.

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Increasing of Pore Pressure

Test No. 31
Material Kaolinite
Date 30/10-71
Name H.S.

Triaxial Cell No. 1
Confining Pressure 10 kg/cm²
LMD Calibration 0007496 kg/div
Start LMD Reading 29520 div

Time	△ Time [min]	Pore Press. [kg/cm²]	LMD Reading [div]	△ LMD Reading [div]	Vertical Deformation Reading [div]
13.00		0	30446		1288 *
		0.2109			
	1		30454	-8	-
	2		30460	-14	-
	5		30450	-4	-
	10		30436	10	-
	20		30409	37	-
	45		30372	74	-
15.20	140		30322	124	-
16.00	180		30315	131	-
17.00	240		30305	141	-
23.30	630		30260	186	-
31/10 10.25	1285		30241	205	-
13.00	1440		30240	206	-
		0.4218			
	1		30285	-45	-
	2		30293	-53	-
	5		30295	-55	-
	15		30280	-40	-
	30		30254	-14	-
15.30	150		30205	35	-
18.10	310		30180	60	-
23.00	600		30155	85	-
1/11 11.00	1320		30130	110	-
13.45 ~ 12.45	1485		30130	110	-
		0.6328			
12.45	1		30177	-47	-
	2		30185	-55	-
	4		30188	-58	-
	8		30175	-45	-
	15		30154	-24	-
13.35	50		30092	38	-
16.20	215		30040	90	-
23.45	660		30007	123	1294
2/11 8.10	1165		29996	134	-
13.00	1455		29990	140	-

Remarks: Dial jumped when motor started

UNIVERSITY OF FLORIDA - SOIL MECHANICS LABORATORY

CSE - TEST

Increasing of Pore Pressure

Test No. 31
Material Kaolinite
Date 2/11-71
Name H.S.

Triaxial Cell No. 1
Confining Pressure 1.0 kg/cm²
LMD Calibration 0.007496 kg/div
Start LMD Reading 29520 div

Time	△ Time [min]	Pore Press. [kg/cm²]	LMD Reading [div]	△ LMD Reading [div]	Vertical Deformation Reading [div]
13.00		0.8437	29990		1294
	1		30035	-45	-
	2		30041	-51	-
	5		30037	-47	-
	45		29907	83	-
15.20	140		29843	147	-
16.55	235		29825	165	-
21.00	480		29810	180	-
3/11 8.00	1140		29803	187	-
13.30	1470		29802	188	-
		0.9843			
	1		29835	-33	-
	2		29835	-33	-
	5		29815	-13	-
	12		29775	27	-
14.25	55		29670	132	-
16.45	195		29622	180	1308
20.00	390		29616	186	-
24.05	635		29610	192	-
4/11 7.55	1105		29599	204	-
9.50	1220		29591	211	-
13.00	1410		29590	212	-

Remarks:

APPENDIX C

WANG 700 PROGRAM FOR COMPUTATION OF MEAN AND SHEAR STRESSES

The following gives the operation instruction and description of the enclosed Wang 700 program. It calculates the mean and shear stresses for a triaxial test where the load, vertical deformation, volume change and pore pressure are measured.

The program will be explained in steps in the order it appears.

Step 1

Key search 1. The typewriter prints out the heading for computation.

Step 2

Enter the initial height of the sample, h_i, and key search 2.

Step 3

Enter the following initial readings and key go after each.

a. Height change due to consolidation, ΔH_c

b. Initial diameter, D_i

c. Volume change due to consolidation, ΔV_c

d. Initial dial readings, δ_o in thousandth of an inch

e. Load cell calibration, c_p

f. Initial load cell reading, R_o

g. Initial cell pressure, $(\sigma_3)_o$

h. Initial pore pressure, u_o

i. Vertical deformation correction, .0004 cm/kg

Step 4

Key point number and search 3.

Step 5

Enter load reading, R_x, vertical dial reading, δ_x, volume change, ΔV, and pore pressure, U_x. The computation begins and follows the procedure

a. Compute $(R_x - R_o)$, then ΔH correction = $.0004(R_x - R_o)$, resulting in height change.

b. Calculate cross sectional area A_x, correcting for volume change and drains.

c. Compute deviator stress $(\sigma_1 - \sigma_3) = e_p \frac{(R_x - R_o)}{A_x}$ which is uncorrected for membrane strength.

d. Compute membrane corrections for axial, $\Delta\sigma_{am}$, and radial, $\Delta\sigma_{rm}$, stresses using the following equations:

$$\Delta\sigma_{am} = -2/3\ E[1+2\ \varepsilon_{at} - \sqrt{\frac{1}{1-\varepsilon_{at}}}\ ]\ \frac{A_m}{A_s}$$

$$\Delta\sigma_{rm} = -2/3\ E]\,2+\varepsilon_{at} - 2\sqrt{\frac{1}{1-\varepsilon_{at}}}]\ \frac{2tm}{rs}$$

assuming $\varepsilon_v = 0$ and using

$E = 175$ psi $= 12.38$ kg/cm^2

$D_s = 3.60$ cm

$t_m = 0.006$ cm

e. Calculate corrected σ_1' and σ_3', mean stress $\frac{\sigma_1' + \sigma_3'}{2}$, and shear stress $\frac{\sigma_1' - \sigma_3'}{2}$

f. Print following results.

a. Point No.

b. Vertical strain

e. Pore pressure

f. Mean stress

g. Shear stress.

Step 6

Return to step 4 and continue through all data points.

700 PROGRAM TITLE: *Mean and Shear Stresses* NO. Page 1 of 9

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
0	Mark						
1	1						
2	write α						
3	1200						
4	0103						
5	T	0207					
6	E	0205					
7	S	0101					
8	T	0207					
9	0002						
10	N	0206					
1	O	0109					
2	.	0106					
3	end α						
4	write						
5	1510						
6	write α						
7	S	0101					
8	T	0207					
9	A	0112					
20	G	0015					
1	E	0205					
2	0002						
3	N	0206					
4	O	0109					
5	.	0106					
6	0108						
7	0108						
8	P	0005					
9	T	0207					
30	0002						
1	N	0206					
2	O	0109					
3	.	0106					
4	end α						
5	write						
6	1503						
7	write α						
8	S	0101					
9	T	0207					

700 PROGRAM TITLE: *Mean and Shear Stresses* NO. Page 2 of 9

Step	Key	Code	Comment	Display		Storage Register	
				X	Y	No.	Contents
40	R	0113					
1	A	0112					
2	I	0104					
3	N	0206					
4	end α						
5	write						
6	1503						
7	write α						
8	P	0005					
9	O	0109					
50	R	0113					
1	E	0205					
2	0002						
3	P	0005					
4	R	0113					
5	E	0205					
6	S	0101					
7	S	0101					
8	end α						
9	write						
60	1503						
1	write α						
2	M	0115					
3	E	0205					
4	A	0112					
5	N	0206					
6	0002						
7	S	0101					
8	T	0207					
9	R	0113					
70	E	0205					
1	S	0101					
2	S	0101					
3	0002						
4	S	0101					
5	H	0201					
6	E	0205					
7	A	0112					
8	R	0113					
9	0002						

700 PROGRAM TITLE: *Mean and Shear Stresses* NO. Page 3 of 9

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
80	S	0101					
1	T	0207					
2	R	0113					
3	E	0205					
4	S	0101					
5	S	0101					
6	0102						
7	0108						
8	0108						
9	end α						
90	stop			h_i			
1	Mark						
2	2						
3	st Dir						
4	00					00	h_i
5	stop			Δh_c			
6	st Dir						
7	01					01	Δh_c
8	Re Dir						
9	00						
100	↑						
1	Re Dir						
2	01					01	$h_i - \Delta h_c = h_c$
3	−						
4	↓↑						
5	St Dir						
6	01						
7	stop			d_i			
8	x^2						
9	↑						
110	π						
1	×						
2	4						
3	÷						
4	↓↑						
5	× Dir						
6	00					00	V_i
7	stop			ΔV_c			
8	− Dir						
9	00					00	$V_i - \Delta V_c = V_c$

700 PROGRAM TITLE: *Mean and Shear Stresses* NO. Page 4 of 9

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
120	stop		in .0001"	δ_0			
1	change sign						
2	↑						
3	.						
4	0						
5	0						
6	0						
7	2						
8	5						
9	4						
130	x						
1	↓						
2	st Dir						
3	02		in cm			02	δ_0
4	stop			C_p			
5	st Dir						
6	03					03	C_p
7	stop			R_0			
8	st Dir						
9	04					04	R_0
140	stop			$(\sigma_3)_0$			
1	st Dir						
2	05					05	$(\sigma_3)_0$
3	stop			μ_0			
4	st Dir						
5	06					06	μ_0
6	stop			0.0004			
7	st Dir						
8	07						
9	Re Dir						
150	03						
1	x Dir						
2	07						
3	Mark						
4	3						
5	subroutine	0200					
6	Re Dir						
7	08			Pt. No.			
8	write						
9	0300						

700 PROGRAM TITLE: *Mean and Shear Stresses* NO. Page 5 of 9

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
160	write						
1	1503						
2	Re Dir						
3	11			ε_1			
4	write						
5	0105						
6	write						
7	1503						
8	Re Dir						
9	23			μ_x			
170	write						
1	0105						
2	write						
3	1506						
4	Re Dir						
5	21		Mean stress	$\frac{\sigma_1'+\sigma_2'}{2}$			
6	write						
7	0105						
8	write						
9	1503						
180	Re Dir						
1	22		Shear stress	$\frac{\sigma_1-\sigma_2}{2}$			
2	write						
3	0105						
4	write α						
5	0108						
6	end α						
7	stop				Pt. No.		
8	Mark						
9	0200						
190	St Dir						
1	08					08	Pt. No.
2	stop				R_x		
3	↑						
4	Re Dir						
5	04				R_0		
6	−						
7	↓						
8	St Dir						
9	09					09	$R_x - R_0$

700 PROGRAM TITLE: *Mean and Shear Stresses* NO. Page 6 of 9

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
200	Re Dir						
1	07						
2	x						
3	↑↓						
4	St Dir						
5	10					10	$\Delta h_{corr.}$
6	stop			δ_x			
7	Change sign						
8	↑						
9	.						
210	0						
1	0						
2	0						
3	2						
4	5						
5	4						
6	x						
7	Re Dir						
8	02			δ_o			
9	↑↓						
220	−				$\delta_o - \delta_x$		
1	Re Dir						
2	10						
3	−				$\delta_o - \delta_x - \Delta h_{corr}$		
4	Re Dir						
5	01			h_c			
6	÷				ε_1		
7	↑↓						
8	St Dir						
9	11					11	ε_1
230	1						
1	↑						
2	Re Dir						
3	11						
4	−				$1 - \varepsilon_1$		
5	Re Dir						
6	01			h_c			
7	x						
8	St Y	0414					
9	12					12	h_x

700 PROGRAM TITLE: *Mean and Shear Stresses* NO. Page 7 of 9

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
240	Re Dir						
1	00			V_c			
2	↑						
3	Stop			ΔV_L			
4	−				$V_c - V_L$		
5	Re Dir						
6	12			h_x			
7	÷				A_x		
8	.						
9	1						
250	2		Only Kaolinite				
1	7						
2	−						
3	St Y	0414					
4	13					13	A_x
5	Re Dir						
6	09			$R_x - R_o$	A_x		
7	↑↓						
8	÷						
9	Re Dir						
260	03			C_p			
1	x						
2	St Y						
3	14					14	$C_p \frac{(R_x - R_o)}{A}$
4	1						
5	↑						
6	Re Dir						
7	11			ε_1			
8	−				$1 - \varepsilon_1$		
9	↑↓						
270	$\frac{1}{x}$			$\frac{1}{1-\varepsilon_1}$			
1	$\sqrt{x}$						
2	St Dir						
3	15					15	$\sqrt{\frac{1}{1-\varepsilon_1}}$
4	Re Dir						
5	11						
6	↑						
7	+				$1 + 2\varepsilon_1$		
8	1						
9	+						

700 PROGRAM TITLE: *Mean and Shear Stresses*

NO. Page 8 of 9

Step	Key	Code	Comment	Display		Storage Register	
				X	Y	No.	Contents
280	Re Dir						
1	15			$\sqrt{\frac{1}{1-\varepsilon_1}}$			
2	−						
3	.						
4	1						
5	1						
6	x				$.11(1+2\varepsilon_1-\sqrt{\frac{1}{1-\varepsilon_1}})$		
7	St Y	0414					
8	16					16	$\Delta\sigma_{am}$
9	Re Y	0415					
290	11				ε_1		
1	2						
2	+				$2+\varepsilon_1$		
3	Re Dir						
4	15			$\sqrt{\frac{1}{1-\varepsilon_1}}$			
5	−						
6	−						
7	.						
8	0		$\frac{2}{3}(12.38)\frac{0.005\times 2}{1.8}$				
9	5						
300	5						
1	x						
2	St Y	0414					
3	17					17	$\Delta\sigma_{rm}$
4	Re Y	0415					
5	05				$(\sigma_3)_0$		
6	Stop			u_x			
7	St Dir						
8	23					23	u_x
9	−				σ_3'		
310	St Y	0414					
1	18					18	σ_3'
2	Re Dir						
3	14			$(\sigma_1-\sigma_3)$			
4	+				σ_1'		
5	Re Dir						
6	16						
7	−				$\sigma_1'{}_{corr}$		
8	St Y	0414					
9	19					19	$\sigma_1'{}_{corr}$

700 PROGRAM TITLE: *Mean and Shear Stresses* NO. Page 9 of 9

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
320	Re Y	0415					
1	18				σ_3'		
2	Re Dir						
3	17						
4	−				$\sigma_{3\,corr}'$		
5	St Y						
6	20					20	$\sigma_{3\,corr}'$
7	Re Dir						
8	19						
9	+						
330	2						
1	÷						
2	St Y	0414					
3	21					21	$\left(\frac{\sigma_1'+\sigma_3'}{2}\right)_{corr}$
4	Re Y						
5	19						
6	Re Dir						
7	20						
8	−						
9	2						
340	÷						
1	St Y	0414					
2	22					22	$\left(\frac{\sigma_1'-\sigma_3'}{2}\right)_{corr}$
3	Return						
4	Mark						
5	4						
6	write α						
7	0108						
8	1201						
9	End α						
0	Stop						
1	End prog.						
2							
3							
4							
5							
6							
7							
8							
9							

APPENDIX D

WANG 700 PROGRAM FOR LEAST SQUARE FIT FOR CONSTANT-STRUCTURE ENVELOPE

The following gives the description and operation instruction for the enclosed Wang 700 program. It calculates the least square fit for the constant-structure envelope equation to the observed data points.

Equations:

The constant-structure envelope is given by

$$\tau = I_o + \alpha\sigma' - \beta\sigma' \ln\sigma' \qquad \text{(D-1)}$$

where I_o, α and β are constants.

Rewriting in linear form and letting

$$\begin{aligned} Y &= \tau \\ X_1 &= \sigma' \\ X_2 &= \sigma' \ln\sigma' \\ b &= I_o \\ b_1 &= \alpha \\ b_2 &= -\beta \end{aligned} \qquad \text{(D-2)}$$

equation (D-1) becomes

$$Y = b_0 + b_1X_1 + b_2X_2 \qquad \text{(D-3)}$$

where

$$b_2 = \frac{(\Sigma x_1 y)(\Sigma x_1 x_2) - (\Sigma x_1{}^2)(\Sigma x_2 y)}{(\Sigma x_1 x_2)^2 - (\Sigma x_1{}^2)(\Sigma x_2)^2}$$

$$b_1 = \frac{(\Sigma x_1 y) - b_2(\Sigma x_1 x_2)}{\Sigma x_1{}^2} \qquad \text{(D-4)}$$

$$b_0 = \overline{Y} - b_1\overline{x}_1 - b_2\overline{x}_2$$

in which

$$x_1 = (X_1 - \overline{X}_1)$$
$$x_2 = (X_2 - \overline{X}_2) \quad \text{(D-5)}$$
$$y = (Y - \overline{Y})$$

For ease of computations are used

$$\Sigma x_1^2 = \Sigma X_1^2 - \frac{(\Sigma X_1)^2}{N}$$

$$\Sigma x_2^2 = \Sigma X_2^2 - \frac{(\Sigma X_2)^2}{N}$$

$$\Sigma x_1 x_2 = \Sigma X_1 X_2 - \frac{(\Sigma X_1)(\Sigma X_2)}{N}$$

$$\Sigma x_1 y = \Sigma X_1 Y - \frac{(\Sigma X_1)(\Sigma Y)}{N} \quad \text{(D-6)}$$

$$\Sigma x_2 y = \Sigma X_2 Y - \frac{(\Sigma X_2)(\Sigma Y)}{N}$$

$$\Sigma y^2 = \Sigma Y^2 - \frac{(\Sigma Y)^2}{N}$$

where N = number of data points.

Programming steps:

a. The program calculates first

ΣX_1, ΣX_2, ΣX_1^2, ΣX_2^2, $\Sigma X_1 X_2$, ΣY, $\Sigma X_1 Y$, $\Sigma X_2 Y$, ΣY^2

b. It then evaluates

Σx_1^2, Σx_2^2, $\Sigma x_1 x_2$, $\Sigma x_1 y$, $\Sigma x_2 y$, Σy^2 from equations (D-6).

c. Next the parameters b_2, b_1 and b_0 are computed from equations (D-4) in that order.

d. The multiple correlation coefficient, $R_{y,12}$ is then computed from

$$R_{y,12} = \sqrt{\frac{b_1 \Sigma x_1 y + b_2 \Sigma x_2 y}{\Sigma y^2}}$$

$$= \sqrt{\frac{\text{Regression sum of squares}}{\text{Total sum of squares}}}$$

R should lie between 0 and 1.

e. Standard error of estimate. $s_{y,12}$ is now calculated by

$$s_{y,12} = \sqrt{\frac{\Sigma y^2 - \text{Regression sum of squares}}{N - 3}}$$

Operating instruction:

Step 1. Search 1

Step 2. Key σ' and go

Step 3. Key τ and go

Step 4. Repeat step 2 and 3 for all data points

Step 5. Key special function 0015 and the typewriter prints out b_0, b_1, b_2 and $R_{y,12}$ and $s_{y,12}$ written as R and s.

700 PROGRAM TITLE: *Least Square Fit for C-S Envelope* NO. Page 1 of 10

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
0	Mark						
1	1						
2	0						
3	St Ind						
4	1						
5	÷						
6	3						
7	0						
8	Skip if Y·X						
9	Search						
10	1						
1	Mark						
2	0200						
3	write α						
4	1200						
5	0108						
6	End α						
7	Stop			$X_1 = \sigma'$			
8	write						
9	0703						
20	St Dir						
1	17					17	X_1
2	+ Dir						
3	00					00	$\sum X_1$
4	x^2						
5	+ Dir						
6	01					01	$\sum X_1^2$
7	1						
8	+ Dir						
9	02					02	N
30	Re Dir						
1	17						
2	↑						
3	$\log_e x$						
4	x						
5	↓		$X_2 = X_1 \log_e X_1$				
6	write						
7	0703						
8	+ Dir						
9	03					03	$\sum X_2$

700 PROGRAM TITLE: Least Square Fit for C-S Envelope NO. Page 2 of 10

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
40	St Dir						
1	18					18	X_2
2	x^2						
3	+ Dir						
4	04					04	$\sum X_2^2$
5	Re.Y	0415					
6	17				X_1		
7	Re Dir						
8	18			X_2			
9	x				X_1X_2		
50	↓						
1	+ Dir						
2	05					05	$\sum X_1X_2$
3	Stop			Y • τ			
4	write						
5	0703						
6	St Dir						
7	19					19	Y
8	+ Dir						
9	06					06	$\sum Y$
60	x^2						
1	+ Dir						
2	20					20	$\sum Y^2$
3	Re Dir						
4	19			Y			
5	↑						
6	Re Dir						
7	17			X_1			
8	x						
9	↓						
70	+ Dir						
1	07					07	$\sum X_1Y$
2	Re Y	0415					
3	19				Y		
4	Re Dir						
5	18			X_2			
6	x						
7	↓						
8	+ Dir						
9	08					08	$\sum X_2Y$

700 PROGRAM TITLE: *Least Square Fit for C-S Envelope* NO. Page 3 of 10

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
80	Search						
1	.0200.						
2	Mark						
3	.0015						
4	Re Dir		compute Σx_1^2				
5	00			$\sum X_1$			
6	x^2						
7	↑				$(\sum X_1)^2$		
8	Re Dir						
9	02			N			
90	÷						
1	Re Dir						
2	01			$\sum X_1^2$			
3	↑↓						
4	−				Σx_1^2		
5	St Y	0414					
6	09					09	$\Sigma x_1^2 = a$
7	Re Dir		compute Σx_2^2				
8	03			$\sum X_2$			
9	x^2						
100	↑				$(\sum X_2)^2$		
1	Re Dir						
2	02			N			
3	÷						
4	Re Dir						
5	04			$\sum X_2^2$			
6	↑↓						
7	−				Σx_2^2		
8	St Y	0414					
9	10					10	$\Sigma x_2^2 = d$
110	Re Dir		compute Σy^2				
1	06			$\sum Y$			
2	x^2						
3	↑				$(\sum Y)^2$		
4	Re Dir						
5	02			N			
6	÷						
7	Re Dir						
8	20			$\sum Y^2$			
9	↑↓						

700 PROGRAM TITLE: Least Square Fit for C-S Envelope NO. Page 4 of 10

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
120	−						
1	St Y						
2	21					21	Σy^2
3	Re Y	0415	compute $\Sigma x_1 x_2$				
4	00				ΣX_1		
5	Re Dir						
6	03			ΣX_2			
7	x						
8	Re Dir						
9	02			N			
130	÷						
1	Re Dir						
2	05			$\Sigma X_1 X_2$			
3	↑↓						
4	−						
5	St Y	0414					
6	11					11	$\Sigma x_1 x_2 = c$
7	Re Y	0415	compute $\Sigma x_1 y$				
8	00				ΣX_1		
9	Re Dir						
140	06			ΣY			
1	x						
2	Re Dir						
3	02			N			
4	÷						
5	Re Dir						
6	07			$\Sigma X_1 Y$			
7	↓↑						
8	−						
9	St Y	0414					
150	12					12	$\Sigma x_1 y = e$
1	Re Y	0415	compute $\Sigma x_2 y$				
2	03				ΣX_2		
3	Re Dir						
4	06			ΣY			
5	x						
6	Re Dir						
7	02			N			
8	÷						
9	Re Dir						

700 PROGRAM TITLE: *Least Square Fit for C-S Envelope* NO. Page 5 of 10

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
160	08			$\Sigma X_2 Y$			
1	↓↑						
2	−						
3	St Y	0414					
4	13					13	$\Sigma x_2 y = f$
5	Re Y	0415	compute b_2				
6	12				Σxy		
7	Re Dir						
8	11			$\Sigma x_1 x_2$			
9	x						
170	St Y	0414				14	ec
1	14						
2	Re Y	0415					
3	09				a		
4	Re Dir						
5	13			f			
6	x						
7	↓						
8	− Dir						
9	14					14	ec − af
180	Re Y	0415					
1	09				a		
2	Re Dir						
3	10			d			
4	x				ad		
5	Re Dir						
6	11						
7	x^2						
8	↓↑						
9	−				$c^2 - ad$		
190	↓						
1	÷ Dir						
2	14					14	b_2
3	Re Y	0415					
4	14				b_2		
5	Re Dir		compute b_1				
6	11			c			
7	x				cb_2		
8	Re Dir						
9	12			e			

700 PROGRAM TITLE: *Least Square Fit for C-S Envelope* NO. Page 6 of 10

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
200	↓↑						
1	−				$e - cb_2$		
2	Re Dir						
3	09			a			
4	÷						
5	St Y	0414					
6	15					15	b_1
7	Re Y	0415	compute b_0				
8	06				ΣY		
9	Re Dir						
210	02			N			
1	÷				$\bar{y}$		
2	St Y	0414					
3	16					16	$\bar{y}$
4	Re Y	0415					
5	00				ΣX_1		
6	Re Dir						
7	02			N			
8	÷				$\bar{x}_1$		
9	Re Dir						
220	15			b_1			
1	x						
2	↓						
3	− Dir						
4	16						
5	Re Y	0415					
6	03				ΣX_2		
7	Re Dir						
8	02			N			
9	÷				$\bar{x}_2$		
230	Re Dir						
1	14						
2	x				$\bar{x}_2 b_2$		
3	↓						
4	− Dir						
5	16					16	b_0
6	write α						
7	0108						
8	End α						
9	Re Dir						

700 PROGRAM TITLE: Least Square Fit for C-S Envelope NO. Page 7 of 10

Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
240	16			b_0			
1	write						
2	1506						
3	write						
4	0909						
5	write α						
6		0002					
7	=	0006					
8	b	0200					
9		0103					
250	(	0300					
1		0102					
2	0	0301					
3		0103					
4	)	0301					
5		0102					
6	0108						
7	End α						
8	Re Dir						
9	15			b_1			
260	write						
1	1506						
2	write						
3	0909						
4	write α						
5		0002					
6	=	0006					
7	b	0200					
8		0103					
9	(	0300					
270		0102					
1	1	0209					
2		0103					
3	)	0301					
4		0102					
5	0108						
6	End α						
7	Re Dir						
8	14			b_2			
9	write						

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Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
280	1506						
1	write						
2	0909						
3	write α						
4		0002					
5	-	0006					
6	b	0200					
7		0103					
8	(	0300					
9		0102					
290	2	0306					
1		0103					
2	)	0301					
3		0102					
4	0108						
5	1201						
6	End α						
7	Re Y	0415					
8	15				b_1		
9	Re Dir						
300	12			$\Sigma x_1 y$			
1	x				$b_1 \Sigma x_1 y$		
2	St Y	0414					
3	22					22	$b_1 \Sigma x_1 y$
4	Re Y	0415					
5	14				b_2		
6	Re Dir						
7	13			$\Sigma x_2 y$			
8	x				$b_2 \Sigma x_2 y$		
9	Re Dir						
310	22			$b_1 \Sigma x_1 y$			
1	+						
2	St Y	0414					
3	23		Regression S of S			23	$b_1 \Sigma x_1 y + b_2 \Sigma x_2 y$
4	Re Dir						
5	21			Σy^2			
6	÷						
7	↓						
8	$\sqrt{x}$						
9	St Dir						

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Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
320	24					24	R
1	Re Y						
2	21				Σy^2		
3	Re Dir						
4	23			Reg. S of S			
5	-						
6	St Y	0414					
7	25					25	Σy^2 - Reg. S of S
8	Re Y	0415					
9	02				N		
330	3						
1	-				N - 3		
2	Re Dir						
3	25			Σy^2 - Reg. S of S			
4	↓↑						
5	÷						
6	↓						
7	$\sqrt{x}$						
8	St Dir						
9	26					26	s
340	write α						
1	1200						
2	0108						
3	0108						
4	End α						
5	Re Dir						
6	24						
7	write						
8	0205						
9	write α						
350		0002					
1	=	0006					
2		0103					
3	R	0113					
4		0102					
5	0108						
6	End α						
7	Re Dir						
8	26						
9	write						

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Step	Key	Code	Comment	Display X	Display Y	Storage Register No.	Storage Register Contents
360	0205						
1	write α						
2		0002					
3	=	0006					
4	S	0101					
5	0108						
6	1201						
7	End α						
8	Stop						
9	End prog.						
370							
1							
2							
3							
4							
5							
6							
7							
8							
9							
380							
1							
2							
3							
4							
5							
6							
7							
8							
9							
390							
1							
2							
3							
4							
5							
6							
7							
8							
9							

APPENDIX E

SAMPLE PROPERTIES

Table E-1

Water Contents and Void Ratios of Glass Beads

Test No.	σ_c kg/cm^2	Before Test		After Consolidation		After Compression		After Test	
		W_i	e_i	W_c	e_c	W_s	e_s	W_f	e_f
24	1	32.2	0.564	22.4	0.544	22.5	0.547	22.9	0.555
29	1	21.9	0.531	20.6	0.500	20.8	0.506	23.0	0.556

Table E-2

Water Contents, Degrees of Saturation and Void Ratios of Kaolinite

Test No.	σ_c kg/cm^2	σ_r kg/cm^2	Before Test			After Consolidation			After Compression			After Test		
			W_i	S_i	e_i	W_c	S_c	e_c	W_s	S_s	e_s	W_f	S_f	e_f
12	4	2	37.7	98.9	0.993	33.7	98.8	0.889	33.2	98.7	0.873	37.4	100.4	0.973
28	2	1	38.5	99.0	1.014	36.1	99.1	0.950	35.8	98.9	0.943	39.8	102.7	1.010
31	2	1	38.1	97.3	1.023	36.0	97.2	0.967	35.9	97.2	0.964	39.9	100.7	1.033

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Miss Helle Strømann was born on April 16, 1947, in Copenhagen, Denmark. In June, 1966, she graduated from Mathematics-Physics side at the Danish High School, Øregaard Gymnasium. From August, 1966, to June, 1970, she studied Civil Engineering at Danmarks Ingeniørakademi in Copenhagen. Following her graduation, she was employed as a teaching assistant in Soil Mechanics and Foundations at Danmarks Ingeniørakademi until September 1970. From then until the present time, December 1971, she has pursued her work at the University of Florida toward the Degree of Master of Science in Engineering. She was supported throughout her graduate study by an assistantship together with financial support from Otto Mønsted Foundation, Denmark.

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I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a thesis for the degree of Master of Science in Engineering.

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I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a thesis for the degree of Master of Science in Engineering.

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