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THE EFFECT OF LABORATORY
COMPACTION ON THE UNCONSOLIDATED—
UNDRAINED STRENGTH BEHAVIOR
OF A HIGHLY PLASTIC CLAY

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INDIANA STATE HIGHWAY COMMISSION

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Interim Report

THE EFFECT OF LABORATORY COMPACTION ON THE UNCONSOLIDATED-UNDRAINED
STRENGTH BEHAVIOR OF A HIGHLY PLASTIC CLAY

TO: H. L. Michael, Director
Joint Highway Research Project

FROM: C. W. Lovell, Research Engineer
Joint Highway Research Project

Sept. 5, 1979

Project: C-36-5M

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Attached is an Interim Report on the HPR Part II Study titled "Improving Embankment Design and Performance". This is Interim Report No. 7 and is titled "The Effect of Laboratory Compaction on the Unconsolidated-Undrained Strength Behavior of a Highly Plastic Clay". It is authored by D. W. Weitzel and C. W. Lovell of our staff.

The report describes the experimental program on the as-compacted undrained strength behavior of laboratory compacted St. Croix clay. Test samples were formed by kneading compaction at moisture-density combinations defined by three impact-type compaction control curves. The samples were confined at total pressures representing a range of embankment confinements and then shear tested in the undrained state. Prediction equations were formed for the compacted dry density and the confined undrained strength in terms of the compaction variables. Research on these relationships for field compacted soil continues.

The Report is presented for acceptance as partial fulfillment of the objectives of the HPR Project of which it is a part. Reviews by all sponsors will result in a final copy which will be published.

Respectfully submitted,

C. W. Lovell
Research Engineer

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Interim Report

THE EFFECT OF LABORATORY COMPACTION ON THE UNCONSOLIDATED-UNDRAINED
STRENGTH BEHAVIOR OF A HIGHLY PLASTIC CLAY

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

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16. Abstract Samples of the St. Croix clay were compacted by the kneading device to fit moisture-density values on impact compaction control curves. The compacted combinations were for low, medium and high energies at moisture contents dry, near, and wet of optimum. The samples were confined at levels representing various positions within an embankment. Both confinement and shear to failure were without drainage. Regression techniques were used to form statistically valid equations for the dependent variables of dry density and compressive strength. Dry of optimum density was a function of water content and compactive work ratio, while wet of optimum density depended upon water content only. Dry of optimum compressive strength depended upon water content, dry density, degree of saturation and confining pressure. Wet of optimum strength decreased with increasing water content. These equations will be helpful in writing compaction specifications to control as compacted stability of clay embankments.			
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LIST OF ABBREVIATIONS AND SYMBOLS

MIT	Massachusetts Institute of Technology
MSE	mean square error
SPSS	Statistical Package for the Social Sciences
°C	degrees Centigrade
cm	centimeter
°	degrees
e	void ratio
e_f	void ratio at failure
ft	feet
G	specific gravity
γ	dry unit weight
$\hat{\gamma}$	estimated dry unit weight
$\bar{\gamma}$	mean dry unit weight within statistical subset
gm	gram
in	inch
kg	kilogram
kg/m^3	kilogram per cubic meter
kJ/m^3	kiloJoule per cubic meter
kN/m^2	kiloNewton per square meter
lb	pound
m	meter

mm	millimeter
N	Newton
P	one half the sum of the major and minor principal stress
P_c	kneading compaction pressure
$\hat{P}_s$	estimated prestress
psi	pounds per square inch
q	one half the difference of the major and minor principal stress (one half the deviator stress)
q_c	compressive strength (stress difference at failure)
$\hat{q}_c$	estimated compressive strength
q_{c_m}	minimum estimated compressive strength
q_u	unconfined compressive strength
$\hat{q}_u$	estimated unconfined compressive strength
$\bar{q}_u$	mean compressive strength within statistical subset
R^2	coefficient of multiple determination
R_a^2	adjusted R^2
ρ_d	dry mass density
$\hat{\rho}_d$	estimated dry mass density
ρ_w	mass density of water, 1000 kg/m ³
S	degree of saturation
S_f	degree of saturation at failure
$S(\gamma_d)$	variation of dry unit weight
$S(q_u)$	variation of unconfined compressive strength

$s(\hat{q}_c)$	estimated standard deviation of compressive strength
σ_1	major principle stress
σ_3	minor principle stress
$(\sigma_1 + \sigma_3)/2 = p$	
$(\sigma_1 - \sigma_3)/2 = q$	
$(\sigma_1 + \sigma_3)_f/2 = p$	at failure
$(\sigma_1 - \sigma_3)_f/2 = q$	at failure
$v(\hat{q}_c)$	estimated variation of compressive strength
w	water content
$\bar{w}$	mean water content within statistical subset
$\bar{w}_R$	average work ratio
χ	effective stress parameter, varies between 0 and 1 and depends upon saturation

HIGHLIGHT SUMMARY

Compaction of soils for earth structures is a common and usually economical means of improving their engineering properties. This study investigated the effects of compaction variables on the strength of a compacted highly plastic clay. The strength tests were unconsolidated-undrained triaxials at various confining pressures, to simulate the end of construction conditions at several embankment depths. Samples for the triaxial tests were prepared in the laboratory by a kneading compactor. Measurements were made of work done during the compaction process.

Three Proctor (impact) compaction curves were developed for this soil at low, Standard and Modified energy levels. Kneading compaction results were fitted to these curves. The results indicated that to fit the kneading compaction results to the Proctor curves, the nominal energy of the kneading mode decreased as the water content increased. It therefore appears that kneading compaction is a more efficient method than impact compaction. Furthermore, the measured work done on the soil generally

decreased as water content increased. The ratio of the compactive work between the different energy levels, when compared at the same saturation, tends toward a constant value of the same magnitude as the nominal impact energy ratio.

Strength behavior was consistent with that previously reported in the literature, viz., an increase in strength results from an increase in density or a decrease in water content. Strength also increased with confining pressure until a near-saturation condition was reached in the sample. The strength varied from 103 to 556 kN/m² for the low energy level, from 145 to 844 kN/m² for the Standard Proctor energy level and from 833 to 2087 kN/m² for the Modified Proctor energy level.

Statistical prediction models were formulated for density and strength using the test data. Dry-of-optimum density was a function of water content and compactive work ratio, while wet-of-optimum density depended upon water content only. The strength prediction model dry-of-optimum includes variables of water content, dry density, degree of saturation and confining pressure. Wet-of-optimum, the logarithm of strength decreases linearly with initial void ratio. In addition, the expected variation in strength from its mean value was found to be directly proportional to that strength value.

Comparison of the prediction model for density and for the strength model at zero confining pressure for this

highly plastic clay was made with prediction models for density and unconfined strength that were developed for a silty clay in another study. Differences were observed, apparently caused by differences in soil minerals and compacted fabric.

INTRODUCTION

Compaction is one of the chief methods for improving the engineering performance of earth structures. Costs of an earth structure can probably be lowered if it is built to the minimum requirements for stability. However, if a large number of tests are needed to define the engineering properties, the cost effectiveness of the engineered design is reduced. It has been the goal of the "Improvement of Embankment Design" project to reduce the number of such tests required through the development of techniques to predict the performance of the compacted soil from compaction variables. The approach has been to investigate the engineering properties of both laboratory and field compacted soils and to correlate these results with the compaction variables. In this manner, tests in the laboratory can be interpreted to predict field behavior, or to evaluate the behavior of field compacted soil from field compaction tests.

This particular study investigated the "short-term" or as-compacted laboratory strength of a highly plastic clay. The "short term" refers to the fill material immediately after compaction and before environmental factors have an opportunity to alter the as-compacted condition of the soil. The as-compacted strength was measured in

unconsolidated-undrained triaxial tests. Samples prepared by kneading compaction to densities that fit on three impact energy curves, with four water contents on each, were sheared at four levels of confining pressure to simulate a variety of compaction conditions and embankment depths. In addition, the work expended to compact the soil was estimated by measurements of force on, and displacements of, the compactor foot.

The results of these tests are evaluated and discussed in light of previous work. These data were then used in statistical regression analysis to develop prediction equations for the density, as-compacted strength and the variability of these, in terms of the compaction variables. Correlation of these results to field data will allow the engineer to better design earth embankments for short-term shear behavior.

LITERATURE REVIEW

Strength is a very important engineering property of soil. To understand this property, a knowledge of the major factors affecting it, including soil fabric and applied and internal stresses, is required. Accordingly, a review of the fabric of and effective stress in partially saturated soils was undertaken.

The Fabric of Compacted Fine-Grained Soil

As defined by Mitchell (1976), "fabric" of a soil is the arrangement of its particles, particle groups and pore space. The "structure" includes the interacting effects of composition and interparticle forces along with the fabric. There are generally two levels of investigation of fabric. Microfabric is a level where study involves the use of at least an optical microscope. Macrofabric is that which can be seen with a hand lens or the naked eye. Some macrofabric features are: large voids, fissures, large soil aggregates and stratifications.

One of the earliest theories of compacted soil fabric was presented by Lambe (1958). His theory was at the micro-structure scale, that is, the various orientations of individual clay particles. The Gouy-Chapman double layer

theory was used to explain the different arrangements of the clay particles. Soil compacted at dry-of-optimum water contents had a high ion concentration in the double layer, causing low repulsive forces, and hence flocculation of the particles. Flocculation produces a low degree of particle orientation; dispersion produces a higher degree, to a limit of parallel orientation. As moisture content approached optimum, the double layers expanded (decreased ion concentration), thereby increasing repulsive forces and decreasing flocculation. Wet-of-optimum, a sufficient quantity of water was available to develop double layers with repulsive forces great enough to result in a dispersed structure.

Seed and Chan (1959) expanded Lambe's hypothesis to other clays and to compaction methods which involved varying effects of shearing strains during compaction. The flocculated structure of samples at water contents less than optimum was found to be strong enough to minimize shear during compaction. Therefore, dry-of-optimum samples (at similar water content and density for a given soil) tended to have similar open fabrics, regardless of compaction method. Wet-of-optimum, the higher repulsive forces caused lower shear strengths and allowed greater shearing strains. The methods which generated higher shearing strains during compaction produced greater dispersion and orientation of particles.

Seed and Chan's evidence was qualitative, based on volume changes and on stresses at different strain levels in

triaxial tests. For all compaction methods (kneading, impact, vibratory and static), compacted clay behavior dry-of-optimum was similar. Samples compacted wet-of-optimum at similar water content and density has greater shrinkage, less swelling and lower strengths at low strains for the compaction method that produced the greatest shear strain. A sample wet-of-optimum had these same traits when compared to a sample dry-of-optimum at equal dry density.

Olson (1963) reported that calcium illite (Unified Classification, CL) did not follow the double-layer theory, although it had a typical compaction curve. He concluded that physiochemical theory was not generally applicable to the compacted fabric of all fine grained soils. Olson agreed with the flocculated particle fabric dry-of-optimum, dispersed fabric wet-of-optimum idea. However, he considered that the effective stress during compaction had a greater effect in producing a particular fabric. He believed the negative pore water pressures in samples dry-of-optimum caused large effective stresses that could resist penetration of the (kneading) compaction foot. Wet-of-optimum, the greater amount of water increased pore pressures and lowered the effective stress. This left the soil unable to resist foot penetration. The shearing that occurred with penetration resulted in a more dispersed structure.

With increasing use of X-ray diffraction and electron microscopes, investigators began to get a more accurate

picture of the fabric of compacted clay. Sloane and Kell (1966) in a scanning electron microscope (SEM) study on compacted kaolin found little or no individual particle contact. Instead, the particles were arranged into packets or aggregates of particles. These aggregates were oriented in a random manner in samples compacted dry-of-optimum by kneading, impact and static compaction methods. Strong orientation of the packets was found wet-of-optimum for all the same compaction methods. Smalley and Cabrera's (1969) SEM micrographs brought them to the same conclusion, viz., that clay packets and not individual particles, were the effective unit of the fabric.

Diamond (1970) also used x-ray diffraction and electron microscopy to examine the fabric of impact compacted kaolin and illite clays. He found randomly oriented packets or domains for clay compacted dry of optimum. These packets touched at only their peripheral points, leaving large inter-domain spaces. Wet-of-optimum, domains were indistinct and had few interstitial voids. However, unlike Sloane and Kell, he did not find significant preferred orientation of the domain normal to the compaction axis for either wet or dry-of-optimum samples.

An electron microscope study on kaolin (CH) and a low plasticity clay (CL) was part of an investigation on compacted clay structure by Barden and Sides (1970). It showed little individual particle fabric at high magnification, whether wet or dry-of-optimum for either clay. However,

for both clays, macroped (or packet) fabric was very visible dry-of-optimum at low magnification; the fabric was homogeneous for both soils compacted wet-of-optimum. They suggested that the difference in macrostructure was due to the strength of the peds. Dry-of-optimum, the peds had high strength and could not be deformed to fill the macropores. The peds at moistures wet-of-optimum had lower strength and could be deformed to fill the pores. They concluded that the differences in engineering behavior, as they showed in compressibility and permeability tests, were due to the macrostructure variation.

Compacted clay macrostructure was also the subject of a study by Hodek (1972) in which he proposed his "deformable aggregate" theory. The deformable aggregate was an agglomeration of clay particles, or macropeds. His X-ray diffraction test results on compacted kaolin aggregates showed them to have increased orientation with higher water contents and compactive efforts. Also, the work to compact the aggregates to a certain unit weight was measured and found to decrease with increasing water content. These results showed that as water content increased, the aggregates became weaker, more deformable, and squeezed into each other to form a more oriented fabric normal to the direction of the compacting force.

The size and distribution of the pore space of compacted clay fabric has also been studied considerably.

Bhasin (1975) found pore size distributions for several different clays at various Proctor energy levels. The distributions of the pore sizes for soil at equal porosities wet and dry-of-optimum were very different, the dry sample having larger pores than the wet one. Also, as compactive effort increased at a constant water content dry-of-optimum, the quantity of larger pores was vastly reduced, until a point was reached where further changes in the pore size distribution would not occur. (At this point, the sample would actually be wet-of-optimum on a compaction curve of higher effort level.) For the samples wet-of-optimum, increases in compactive effort would neither decrease total porosity or change the distribution of pore sizes.

The results above corroborated with those of Sridharan, et al. (1971) and Ahmed, et al. (1974). Ahmed also showed that compaction to a constant water content and density by different methods had little effect on the pore size distribution. Garcia-Bengochea (1978) also showed results similar to previous investigators for a 50-50 silt-kaolin and mixture. However, for his soils that contained 70 and 90% silt (only 30 or 10% kaolin), a significant number of large pores were retained despite higher compaction energy and molding water content.

These studies have brought the model of compacted clay fabric to basically one of macrostructure. Instead of treating individual clay particles, collections of

particles into groups called, by various authors, domains, packets, macropeds or aggregates, are considered the more important part of the fabric of compacted fine-grained soils.

The arrangement of these aggregates vary wet and dry-of-optimum. Dry-of-optimum, the aggregates are distinct. The void space is principally between aggregates and a considerable quantity of it in larger pores. As optimum water content is approached, the soil gets closer to its plastic limit; the aggregates become weaker and more deformable. Hence, the aggregates distort and squeeze closer together, reducing the number of large pores. Past the optimum water content, the aggregates become much less distinct, forming a homogeneous mass. The pore space around the aggregates approaches the size of the intraaggregate pores, a much finer range of pores sizes.

At constant water content, increases in energy also change the arrangement of aggregates. As energy increases, aggregates again become more broken or deformed and the quantity of large pores is reduced. A point will be reached where further energy application can cause no further change in the pore frequency or distribution.

The degree to which a compacted fine-grained soil will follow the above trends depends on the type and amount of clay present, the type of compaction, electrolyte and temperature.

The arrangements of the aggregates, size and distribution of pores and the amount of water in those pores will affect the way that stress develops in the soil. Hence, the above discussion will be useful in understanding and explaining the strength behavior trends in the data to be presented.

Effective Stress in Partially Saturated Soils

Effective stress in saturated soil has been found to follow the Terzaghi equation:

$$\sigma' = \sigma - u \quad (1.1)$$

where σ' = effective stress

σ = total stress

u = pore pressure

However, effective stress in partially saturated soils can not be represented by this equation. This is because of the existence of three phases, soil, water and air. Accordingly, the effective stress is affected by pressures in both air and water, pressures which are not usually equal. This pressure difference arises due to the presence of capillary meniscii in the water. These meniscii cause tension in the water that results in negative pore water pressure. The water pressure will be less than that of the pore air pressure unless the air is occluded.

A revised effective stress principle was sought to account for these pressure differences in the pore fluids. Some of the first revisions suggested to Equation (1.1) were proposed by Croney, et al. (1958), Jennings (1960) and Aitchison (1960), shown in Equations 1.2a, 1.2b and 1.2c respectively.

$$\sigma' = p - \beta'u \quad (1.2a)$$

where: σ' = effective stress

p = total stress

u = pore water pressure

β = holding or bonding factor related to number of bonds of water under tension, effective in contributing to shear strength of the soil.

$$\sigma' = \sigma + \beta p'' \quad (1.2b)$$

where: σ' = effective stress

σ = total stress

p'' = soil moisture suction

β = statistical factor of the same type as contact area of water per total unit area in the soil.

$$\sigma' = \sigma + \psi p'' \quad (1.2c)$$

where: σ' = effective stress

σ = total stress

p'' = soil moisture suction

ψ = ratio of component of effective stress due to p'' ; has values between 0 and 1.

Bishop (1960) proposed a more general form of Equations (1.2) in which pore air pressure was not assumed as zero, and so included the term u_a :

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \quad (1.3)$$

where: u_a = pore air pressure

u_w = pore water pressure, and

χ = a parameter between 0 and 1 that varies with saturation.

Bishop and Aitchison (1960) agreed on the general equivalence of Equations (1.2) and (1.3). The parameters β , β' , ψ and χ were essentially the same, with values that varied between 0 and 1 as the degree of saturation varied from 0 to 100%.

Bishop and Donald (1961) examined Equation (1.3) in light of isotropic consolidation and triaxial shear tests on partially saturated silt. They varied the stress components σ , u_a and u_w , keeping $(\sigma - u_a)$ and $(u_a - u_w)$ constant, and found shear strength unaffected. They also calculated values of χ by using the results from tests on both saturated and partly saturated silt in Equation (1.3). The values of χ calculated in this way from results on shear tests were essentially the same as the χ values calculated from consolidation tests. This led Bishop and Donald to conclude Equation (1.3) was valid.

Jennings and Burland (1962) however contended that the validity of Equation (1.3) is proven only if the behavior of soil is unchanged when changes occur in $(\sigma - u_a)$ and $\chi(u_a - u_w)$, so that their sum is constant (constant effective stress). This could not be accomplished since there was no complete knowledge of χ . Therefore they sought qualitative assessment of the effective stress equation by comparing the volume change behavior of a saturated sample with that of an unsaturated one under similar load. For saturations as high as 90%, a partly saturated silty clay soil tended to collapse upon wetting. With wetting, the negative pore pressures increased, thereby decreasing effective stress. If partially saturated soil followed the effective stress principle, a volume increase would have been expected as a result of the decreased effective stress. Hence, Jennings and Burland concluded that for unsaturated soils, separate functions of applied stress, σ , and equivalent pore pressure, u^* or $(\chi u_w - (1 - \chi)u_a)$ ¹, controlled the behavior.

¹ The equivalent pore pressure is just a rearrangement of Equation (1.3) by collecting the pore pressure terms

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \quad (1.3)$$

$$= \sigma - (\chi u_w - (1 - \chi)u_a) \quad (1.3b)$$

$$\text{or} \quad = \sigma - u^*$$

Bishop and Blight (1963) also later recognized the inadequacy of Equation (1.3). As a consequence, they stressed the importance of stress path and the paths of $(\sigma - u_a)$ and $(u_a - u_w)$, as it was evident that χ could change significantly as these components varied. However, because intergranular stress had more control in shear failure than in volume change, where stress path was more important, they found that the effective stress equation had better application to shear strength than to volume change.

Assuming that Equation (1.3) was still valid for shear strength, Bishop and Blight performed triaxial shear tests on different soils and calculated χ parameters by the same method as Bishop and Donald (1961). The χ parameters showed similar trends for the different soils, but they also found (as did Jennings and Burland (1962)) that the effective stress model was unsatisfactory below certain degrees of saturation.

Burland (1964) agreed with Bishop and Blight that the effective stress equation could be more readily applied to shear strength than volume change, especially at the maximum deviator stress. However, he showed that the application of the effective stress equation to volume change could result in negative values of χ .

Triaxial tests performed at MIT (1963) on silty clay at constant void ratio and structure, but different water contents, showed poor applicability of Equation (1.3) to

shear strength. Their values of χ , calculated by the same procedure as Bishop and Donald (1961), were all greater than one. They concluded Equation (1.3) was inadequate. These inadequacies were due to the effects of the two stress components, $(\sigma - u_a)$ and $(u_a - u_w)$. Their explanation was that the applied stress created both normal and shear stresses at particle contacts, whereas the capillary stresses $(u_a - u_w)$ could only produce normal forces at the particle contacts. Hence, the initial values of those components and the paths they took would result in different behavior. Soil with large amounts of existing shear stresses (due to high applied stress) would have these stresses acting in many directions at particle contacts. Large strains would be required to reverse and mobilize these shear stresses in the same direction. This would in essence be a "progressive failure". Soil with mostly capillary forces had little existing shear stress, and so little strain was needed to mobilize shear all in one direction.

The MIT report still considered the principle of effective stress to be valuable to shear strength, but the internal and external stresses would have to be correlated to an intergranular stress that would contribute to the value of χ .

Lambe (1960) considered internal stresses in an equation for intergranular stress:

$$\sigma = \bar{\sigma} a_m + p_a a_a + u_a u_w + R - A \quad (1.4)$$

where: σ = total stress

$\bar{\sigma}$ = intergranular stress of mineral-mineral contact

a_m = percent of total area in mineral-mineral contact

p_a = pressure of air in air-mineral contact

a_a = percent of total area in air-mineral contact

u = pressure in the water in water-mineral or water-water contact

a_w = percent of total area in water-mineral contact

R = pressure due to electrical repulsion between particles

A = pressure due to electrical attraction between particles

As Mitchell (1976) noted, this expression differs from the conventional effective stress expression by the forces of attraction and repulsion. However, there are considerable limitations in the use of Equation (1.4) due to the difficulties in evaluation of its terms.

Sridharan (1968) used a similar equation for intergranular stress, $\bar{C}$,

$$\bar{C} = \sigma - (\bar{u}_a + \bar{u}_w) - R + A \quad (1.5)$$

where: σ = total stress

$\bar{u}_a$ = effective pore air pressure

$\bar{u}_w$ = effective pore water pressure

R = net repulsive pressure due to particle electrical forces

A = net attractive pressure due to particle
electrical forces

He related the effective intergranular stress to shear strength, S, by

$$S = f(\bar{C}) + c \quad (1.6)$$

where c was a constant. Using theoretical considerations of attractive and repulsive forces, he calculated these electrical forces and found that they controlled the strength (and effective stress) up to saturations of about 35% for the compacted soils he tested in unconfined compression. For saturations larger than this, the negative pore water pressures (which were calculated by knowledge of pore sizes) controlled the strength (effective stress). These negative pore water pressures increased up to a particular degree of saturation and then decreased as the saturation continued to increase.

The difference between repulsive and attractive forces (R-A) is also known as the solute or osmotic suction. Its importance was pointed out by Aitchison (1965). Although solute suction is difficult to obtain, it can be found by subtracting matrix suction ($u_a - u_w$), from total suction, both of which can be measured. Aitchison noted that not all clays react to changes in solute suction, e.g., calcium illite does not, but some react strongly to such changes.

Aitchison (1973) related matrix and solute suctions to the effective stress by:

$$\sigma' = \sigma + \chi_m \rho_m'' + \chi_s \rho_s'' \quad (1.7)$$

where: σ = total applied stress

ρ_m'' = matrix suction = $u_a - u_w$

ρ_s'' = solute suction = $\rho_t'' - \rho_m''$, ρ_t'' = total suction

and χ_m and χ_s were parameters, as before, with value from 0 to 1 and dependent on stress path among other factors. Because of the stress path dependency of χ_m and χ_s , and their interdependency, they cannot be independently determined. Therefore, although Aitchison and Martin (1973) showed compression curves due to independent loading by applied pressure, or matrix suction, or solute suction, they could not be related to an effective stress because of a lack of χ_m and χ_s values. Thus, Equation (1.7) has not been verified in its relation to volume change, and has not been investigated for shear strength.

Fredlund and Morgenstern (1977) defined the air-water interface or "contractile skin" as a fourth phase in the soil-air-water system. Using multiphase continuum mechanics, they derived two stress matrices from the equilibrium equations on the soil particles and contractile skin. These matrices were:

$$\begin{bmatrix} \sigma_x - u_w & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_y - u_w & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_z - u_w \end{bmatrix} \text{ and } \begin{bmatrix} u_a - u_w & 0 & 0 \\ 0 & u_a - u_w & 0 \\ 0 & 0 & u_a - u_w \end{bmatrix} \quad (1.8)$$

The authors verified this model with volume change tests on silt, 80% silt-20% kaolin, and pure kaolin soils. They changed total stress and pore water pressure, but kept $(\sigma - u_w)$ and $(u_a - u_w)$ constant. In all tests they found essentially no volume change.

A summary of the effective stress equations for partially saturated soils as discussed above is shown in Table 1. None of these equations has resulted in a simple and complete explanation of the behavior of partially saturated soils. However, the components of equations such as Aitchison's Equation (1.7) have been shown to be important factors in unsaturated soil behavior, and are measurable quantities. The main obstacle to the approaches discussed above seems to be the particulate nature of clays, especially as was seen in the fabric of dry-of-optimum compacted clay.

Although none of the above models of effective stress in partially saturated soils has been found to be fully applicable, the effects that the stress components have on strength behavior are illustrated by them. A knowledge of the kinds of stresses to be expected in a compacted soil

Table 1.1 Effective Stress Equations for Partially Saturated Soils

Reference	Equation*	
Croney, et al. (1958)	$\sigma' = \rho - \beta'u$	(1.2a)
Jennings (1960)	$\sigma' = \sigma + \beta\rho''$	(1.2b)
Aitchison (1960)	$\sigma' = \sigma + \psi\rho''$	(1.2c)
Bishop (1960)	$\sigma' = \sigma - u_a + \chi(u_a - u_w)$	(1.3)
Lambe (1960)	$\sigma = \bar{\sigma}_a + p_a a_a + u_a w + R - A$	(1.4)
Sridharan (1968)	$\bar{C} = \sigma - (\bar{u}_a + \bar{u}_w) - R + A$	(1.5)
Aitchison (1973)	$\sigma' = \sigma + \chi_m \rho_m'' + \chi_s \rho_s''$	(1.7)
Fredlund and Morgenstern (1977)	$\begin{bmatrix} \sigma_x - u_x & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_y - u_w & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_z - u_w \end{bmatrix} \begin{bmatrix} u_a - u_w & 0 & 0 \\ 0 & u_a - u_w & 0 \\ 0 & 0 & u_a - u_w \end{bmatrix}$	(1.8)

* See text for explanation of symbols

will be useful in understanding and explaining the strength behavior of such a soil.

Unconsolidated-Undrained Compressive Strength
of Partly Saturated Fine-Grained Soil

A review by Rutledge (1947) of the Cooperative Triaxial Research Program, presented some of the first comprehensive results on unconsolidated-undrained (U-U) strength of compacted clays. He found that the major variables were: minor principal stress, the dry density and water content and degree of saturation. Additionally, the results indicated that U-U strength:

- (1) increased as the minor principal stress increased
- (2) increased as the dry density increased
- (3) decreased as the water content and saturation increased.

The U-U strength increases with the minor principal stress only until the confining pressure becomes high enough for a sample to become fully saturated. This happens when the air in the sample voids is dissolved in the water due to increased pressure. This behavior has been shown by Casagrande and Hirschfeld (1960, 1962). A generalized plot of their typical failure envelopes is shown in Figure 1.1. They tested a silty clay soil compacted by the kneading method to a constant density. Each curve is for samples at constant water content and the water contents increased from curve 1 to curve 5.

FIGURE 1.1 SCHEMATIC FAILURE ENVELOPES FOR UNCONSOLIDATED - UNDRAINED TRIAXIAL TESTS

The water content of curve 5 in Figure 1.1 was high enough so that the samples were very nearly saturated and little pressure was needed to dissolve the air in the pores. The envelope quickly became horizontal, and the $\phi = 0$ concept applies (Lambe and Whitman (1969)). Under these conditions, further increases in confining pressure are taken up by the pore water and not the soil fabric. Therefore, the effective stress and strength stay constant, which results in the horizontal envelope. The samples for curves, 4, 3, 2 and 1 had successively lower initial saturation. Some of these samples never reached 100% saturation, and the envelopes continued to slope upward. Higher pressure would have been required to dissolve the larger volumes of air that were present in samples of low initial saturation.

Lee and Haley (1968) showed however, that under very high confining pressures, such as would be encountered in a high earth dam, soils compacted at low water contents would eventually become saturated.

Results similar to Casagrande and Hirschfeld were presented by Conlin (1972) for samples compacted at constant nominal energy and varying water content (varying density). Water content increased in the same manner, but density did not follow the same trend. If samples for curve 3 (Figure 1.1) were at maximum dry density, then density decreased in either direction, i.e., envelopes above and below were for samples at lower densities.

Lambe (1961) and Olson and Langfelder (1965) showed the existence of highly negative pore water pressures in compacted soils dry-of-optimum. These pore pressures result in greater effective stress and hence greater strength. This was the reason Conlin's dry-of-optimum samples were stronger, even though their densities were lower.

Strength may also decrease with increasing density at constant water content, depending on the strength criterion that is selected. Seed and Monismith (1959) showed from triaxial compression tests on silty clay, that strength determined at 20% strain increased with increasing density. But strength determined at 10% strain increased with increasing density up to a point, and then decreased as density continued to increase. Seed and Monismith's small strain results are similar to the CBR test results reported by Foster (1955), Turnbull and Foster (1958) and others. The CBR test of course involves small amounts of strain.

Seed and Chan (1959) provided additional evidence on the importance of strain criterion. They explained the strength-density relationship in terms of fabric. As density at a particular water content increases, the fabric becomes more dispersed. (The higher energy required to raise density at constant water content, results in a sample at a higher $\gamma_d - w$ curve where it would be closer to or above optimum water content.) This more dispersed fabric is more plastic than the stiffer random fabric of a sample at lower

density at the same water content. Hence, at lower strains, the stiffer sample would have picked up more load than the more plastic one at a higher density. At higher strains, all samples at all water contents and densities have had their fabrics reduced to a more dispersed one due to the shear strains. When all samples at a constant moisture content had their fabrics modified by high strains, the sample with the highest density had the greatest strength. Seed and Chan point out however, that these results do not apply to all soils and compaction methods. Soils such as the sandy clay and highly plastic clay tested showed lesser effects of fabric and more of inter-particle forces. Compaction methods that induce fewer shear strains, e.g., static, may result in samples with a relatively undispersed fabric at all levels of water content and density.

Samples of similar fabric and initial water content show that strength will increase with an increase in density. In this connection, Leonards (1955) found a unique relationship between void ratio at failure and compressive strength that was independent of confining pressure, the drainage condition, water content and degree of saturation.

At different initial water contents, different compacted fabrics result and each initial condition would have a different void ratio-strength relation. DaCruz (1963) found the differences could be accounted for by including saturation and void ratio, i.e., $e\sqrt{S}$. These values at failure plotted against the $\log (1/2(\sigma_1 - \sigma_3)_f)$ showed a linear

relationship for all compacted samples of his residual clay.

The importance of the third strength factor enumerated by Rutledge, viz., water content and saturation, has been verified by many of the studies cited above. Seed and Monismith (1954) noted that stability changes often occurred near optimum water content, as seen in Figures 1.2a and b. This indicated to them that saturation level, not merely water content, exercised the primary control over stability. Therefore, they plotted strength² vs. density at equal saturation levels, as shown in Figures 1.3a and b. At each level of saturation, strength continued to increase with density at both high and low strain levels. Figures 1.2a and b and 1.3a and b also show that, for a constant density, strength decreased as water content or saturation increased. The relative magnitude of the strength decrease depended on the relative differences in fabric.

Stress-Strain Characteristics

Typical stress-strain curves from Seed and Chan (1959) for a kneading compacted silty clay are shown in Figure 1.4. Samples compacted on the wet side of optimum have curves typically like 4, 5 and 6 in the figure. Samples with the higher water content, lower density and oriented fabric have this plastic type behavior and reach ultimate strength at very high strains. As water content

²The writer assumes Seed and Monismith refer to strength as the deviator stress at failure.

FIGURE 1.2a RELATIONSHIP BETWEEN DRY DENSITY AND STRESS AT 10% STRAIN FOR CONSTANT WATER CONTENT (from SEED & MONISMITH, 1954)

FIGURE 1.2b RELATIONSHIP BETWEEN DRY DENSITY AND STRESS AT 20% STRAIN FOR CONSTANT WATER CONTENT (from SEED & MONISMITH, 1954)

FIGURE 1.3a RELATIONSHIP BETWEEN DRY DENSITY AND STRESS AT 10% STRAIN FOR CONSTANT DEGREES OF SATURATION (from SEED & MONISMITH, 1954)

FIGURE 1.3b RELATIONSHIP BETWEEN DRY DENSITY AND STRESS AT 20% STRAIN FOR CONSTANT DEGREES OF SATURATION (from SEED & MONISMITH, 1954)

FIGURE 1.4 INFLUENCE OF MOLDING WATER CONTENT ON STRESS-STRAIN RELATIONSHIP FOR COMPACTED KAOLINITE (from Seed and Chan, 1959)

decreases to optimum and below, the fabric, as earlier discussed, becomes more random and rigid. This results in a faster and greater increase in load. Axial straining may continue past ultimate strength at a value of residual strength, as for curve 3. At very low water contents, the randomly oriented fabric, in conjunction with highly negative pore water pressures, results in a steep stress-strain curve with very brittle characteristics, as in curves 1 and 2. Similar results are shown in the many stress-strain curves of Casagrande and Hirschfeld (1960, 1962), Conlin (1972) and others. This behavior is of course, not the same for all compacted clay soils. Variations will depend on amount and type of clay fraction, density, compaction method, water content and saturation, and minor principal stress.

Withiam and Kulhawy (1976) measured volumetric strains during U-U triaxial tests on a sandy clayey silt. They found the trend of volumetric strains at failure with dry density and water content were similar regardless of compaction method and confining pressure. At higher compactive efforts, the volume changes were more dilatent, but became more compressive with increasing confining pressure. Since these data showed a good range of dilatent and compressive behavior, they looked at the applicability of the critical void ratio-confining pressure relationship. They found it to be well defined and independent of compaction energy and method, density, and water content.

Certain fabric and effective stress effects in a partially saturated clay are well-illustrated in Conlin's (1972) volumetric strain data. He measured volume change both during application of confining pressure and throughout shear in his U-U triaxial tests on a highly plastic clay. The general trends are shown in Figure 1.5. The volumetric strain due to confining pressure increased with water content, up to optimum. The addition of water reduced the negative pore water pressure; this decreased the normal stress between soil aggregates and allowed more deformation with confining pressure application. This volumetric strain also increased with increasing confining pressure at constant water content. However, total volumetric strain (the sum of volumetric strains due to confining pressure and shearing) decreased as water content increased because of the increased saturation. Water, being many times less compressible than air, left less space that was susceptible to volume change, as water content increased. Hence, the total volumetric strain approached that due to confining pressure as the saturation became high.

Statistical Prediction of Laboratory Compacted Soil Properties and Variability

It was seen in an earlier section of this review that although some indications of behavior may be obtained from knowledge of fabric and changes in applied stress and suctions, no valid effective stress equation has been

FIGURE 1.5 VOLUMETRIC STRAIN BEHAVIOR DURING AN UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST (After Conlin, 1972)

derived for partly saturated soil. Hence, some investigators have turned to parameters of the compacted mass, e.g., water content, dry density and energy, to predict shear strength behavior. This is often done with statistical techniques.

Peterson (1975) performed regression analysis on the data from unconfined compression tests. The predictive model for the dry density ($\hat{\gamma}_d$) of a laboratory kneading compacted silty clay included water content (w) and water content squared (w^2), that is,

$$\hat{\gamma}_d = -47.32 + 26.14 w - 1.016 w^2 \quad (1.9)$$

His unconfined compressive strength ($\hat{q}_u$) prediction equation included density and water content cubed:

$$\hat{q}_u = -91.16 + 1.56 \gamma_d - .0187 w^3 \quad (1.10)$$

Essigmann (1976) also performed unconfined compression tests on a laboratory impact compacted silty clay. From a list of water content, dry density, energy, strength and combinations thereof, significant variables were selected by an all possible regressions analysis. These variables were further combined and analyzed in separate complete regression runs to obtain valid prediction models. The equations Essigmann selected are shown in Table 1.2.

Similar regression techniques were used successfully by Scott (1977) to formulate prediction equations for dry

Table 1.2 Regression Results for a Silty Clay (from Essignmann, 1976)

γ_d = dry unit weight, pcf

w = water content, %

E = impact energy ratio

q_u = unconfined compressive strength, psi

$S(q_u)$ = variation in unconfined compressive strength, psi

$S(\gamma_d)$ = variation in dry unit weight, pcf

$\bar{\gamma}_d$ = mean dry unit weight of statistical subset, pcf

$\bar{w}$ = mean water content of statistical subset, %

$\bar{q}_u$ = mean unconfined compressive strength of statistical subset, psi

Table 1.2

DEPENDENT VARIABLE	MOISTURE RANGE	REGRESSION RELATIONSHIP	R ²
Dry Density	All Moistures	No Models Significant	
	Dry of Optimum	$\gamma_d = 0.375w + 2.12v + 90.6$	0.905
	Wet of Optimum	$\gamma_d = -1.90v + 145.7$	0.919
Variation in Dry Density	All Moistures	No Models Significant	
	Dry of Optimum	$S(\gamma_d) = 3.02w - 1.06 \times 10^{-5}w^2 + 0.142\gamma_d$	0.789
	Wet of Optimum	$S(\gamma_d) = 0.00916w + 0.55$	0.645
Unconfined Strength	All Moistures	$q_u = 4.12\gamma_d - 0.049\gamma_d w - 393.3$	0.889
	Dry of Optimum	$q_u = 4.14\gamma_d - 0.047\gamma_d w - 396.2$	0.888
	Wet of Optimum	$q_u = 3.10\gamma_d - 0.073\gamma_d w - 231.8$	0.943
Variation in Unconfined Strength	All Moistures	$S(q_u) = 0.128\bar{q}_u + 2.38s(\gamma_d) - 2.6$	0.866
	Dry of Optimum	$S(q_u) = 0.124\bar{q}_u + 2.88s(\gamma_d) - 3.4$	0.814
	Wet of Optimum	$S(q_u) = 0.129\bar{q}_u + 1.88s(\gamma_d) - 2.18$	0.891

density and strength. Price (1978) also developed prediction equations similar to Scott and Essigmann but by a slightly different approach. He selected independent variables by plotting them in scattergrams against the dependent variable. The variables that were found significant were run in all possible regressions program to select significant models for further investigation. Separate complete computer regression analysis was then performed on significant models. If criteria were met at this stage, the final step of model analysis was another regression program that tested the variables for sensitivity to the data. Tests run by Scott were on laboratory compacted and saturated samples, and those tested by Price were on field compacted samples, both as-compacted and saturated. Therefore, further discussion of their prediction equations will not be undertaken here.

Variability in compacted soil is inherent. Duplication of compaction variables cannot be achieved, hence variability in density and strength is expected. As it would be useful to know the variation expected, the above investigators also sought prediction equations for the variability. This would allow, in addition to a prediction of the expected average density and strength, a means to assure a minimum density or strength.

Peterson was unable to define a variability relationship. However, Essigmann found the variability relationships as shown in Table 1.2. These were developed by

dividing his data into 1% water content intervals relative to the optimum water content. The mean and variance of each of these subsets were used to perform an all possible regressions. Scott used an identical procedure in his variability analysis.

It appeared that all of Price's data were on the wet side of optimum, so that 1% water content ranges with respect to the optimum could not be used. In addition, the range of water content and dry density variation in his field compacted samples was much greater. Hence, investigation of variability at specified values of water content and density, as Essigmann and Scott performed, was not possible.

To find the variability, Price used the equation:

$$V(\hat{q}_u) = \lambda_s \sqrt{X_p' [X'X]^{-1} X_p} \quad (1.11)$$

where: $V(\hat{q}_u)$ = the variability of the estimated strength

λ_s = appropriate statistic of the t-distribution

$[X'X]^{-1}$ = a matrix dependent on the variance/co-

variance matrix of the prediction model, and

on the means of the independent variables

X_p = a column matrix of the values of independent variables at which the variability is to be evaluated

X_p' = transpose of X_p .

For each level of variables, there were four X_p values, because each level of variables had associated with

it a $\pm$ range of water content and a $\pm$ range of density. Hence, there were also four values of $V(\hat{q}_u)$ determined, the largest being the significant one at each level of variables. His results showed the minimum density or strength (the expected strength minus the expected variability) to be significantly less than the mean strength.

Jeng and Strohm (1976) also used statistical regression techniques to correlate strength data from several different soil types to compaction variables and index properties. The equation they used for prediction of major principal stress was:

$$\sigma_1 = A + B \sigma_3 + C \alpha_3 \quad (1.12)$$

where A, B and C were dependent variables that were studied in correlation with Atterberg limits, initial water content, initial degree of saturation, initial void ratio and dry density. The relation of A, B and C to these variables was isolated by use of a stepwise regression procedure. They found the important variables to be liquid limit, percent of maximum dry density, and initial saturation. However, the variables B and C were dependent on A and B respectively. Hence, they used A as a variable in the correlation of B, and B in the correlation of C. This violated an assumption of the regression model, where each variable is assumed to be independent of the others. The normal stress, σ , and the shear stress, s , on the failure plane at failure were calculated by:

$$\sigma = 0.25 (\sigma_1 + 3\sigma_3) \quad (1.13)$$

$$s = 0.43(\sigma_1 - \sigma_3) \quad (1.14)$$

where σ_1 was from Equation (1.12) and σ_3 is a given confining stress. Equation (1.14) was obtained assuming the failure plane to be at $(45 + \phi/2)$, with an assumed value of $\phi = 30^\circ$.

EXPERIMENTAL PROCEDURE AND APPARATUS

Soil Studied

The soil used for this study was a highly plastic clay, given the name St. Croix clay. It was a residual soil of sandstone and shale origin, taken from a cut area in the realignment project of State Road 37, about four miles south of St. Croix, Indiana. It was a tan color with red and gray mottling. Numerous friable sandstone rocks were present throughout the mass and some of the gray mottled areas of the soil had a shaly structure. Its gradation curve is shown in Figure 2.1 and Atterburg limits and classification values are given in Table 2.1.

Table 2.1 Index Properties and Classification of St. Croix Clay

Atterberg Limits, %	$w_L = 52$	$w_p = 23$	$I_p = 29$
Clay Fraction ($< 2\mu\text{m}$)	40%		
Specific Gravity	$G = 2.79$		
Unified Soil Classification		CH	
AASHTO Classification		A-7-6 (27)

FIGURE 2.1 GRAIN SIZE DISTRIBUTION

Soil Preparation and Mixing

A large quantity of soil was brought to the laboratory ($\sim 2 \text{ m}^3$) and was kept from drying as well as the large natural quantity and storage facilities allowed. The soil was processed by forcing it through a No. 4 sieve with a metal blade. This removed large stones and resulted in a more uniform material. The sieved soil was stored in garbage cans lined with plastic bags to retain the moisture.

A quantity of processed soil required for one compaction test, about 2.6 kg (5.7 lbs.), was sampled for moisture content, placed inside two polyethylene bags, and stored in a large sealed plastic barrel while the water content samples dried for 24 hours in 105°C ovens. The water content information was used to calculate the amount of water to be added or removed to give the soil the desired compaction water content.

If a water content lower than the storage value was needed, the soil was air dried. The drying soil was stirred and weighed until the weight of water remaining corresponded to the desired moisture content. If water was to be added, deionized water was sprayed in with a hand operated atomizer while constantly stirring the soil. Again, weighing indicated when the correct amount of water had been added to reach the specified water content.

After mixing the soil, it was rebagged, replaced in the plastic barrel and cured for 72 hours. Since the soil

was quite plastic, a fairly long cure time was needed for the water to equally distribute. Numerous preliminary compaction tests and other experiences with the soil led to the conclusion that the 3 days was sufficient.

Compaction

Equipment

Kneading compaction was used to prepare samples for triaxial testing. It was felt that this method, among the common laboratory types, was the most like field compaction, i.e., the shearing strains and loading patterns that occur while compacting a sample are more similar to those in the field.

The kneading compactor was manufactured by the August Manufacturing Company of Oakland, California and is shown in Figure 2.2. It is an automatic device, with mold rotation and foot tamping actions being driven by an electric motor. The foot pressure is supplied by a pneumatic-hydraulic system. Details of the compactor construction and operation are given by Gaudette (1960).

Full face coverage of a layer of soil in the compaction mold is obtained by six tamps of the compactor foot, with the table, to which the compaction mold is bolted, rotating 60° between each tamp. The amount of pressure exerted by each tamp is controlled by valves to the air pressure source. The air pressure gage can be calibrated with

a) Kneading compactor	d) Linear potentiometer
b) Compaction mold	e) Strip chart recorder
c) Load cell	

FIGURE 2.2 KNEADING COMPACTOR AND ENERGY MEASUREMENT EQUIPMENT

the actual pressure exerted by the foot. However, as noted by Garcia-Bengochea (1978), this calibration seems to vary seasonally, although the precise reason is not known.

Instead of using calibration curves, in this study work applied to the soil was measured. This was accomplished by measuring the load and displacement of the foot during compaction, for every tamp applied to the soil. The load was measured by a strain gaged aluminum sleeve, positioned as shown in Figure 2.2. The output of the load cell was recorded on one channel of an Oscillo/Writer, two channel, strip chart recorder made by Texas Instruments. On the other channel of the recorder, simultaneous foot displacements were recorded by the use of a Servonic Rectilinear Potentiometer. Its position is also seen in Figure 2.2. The body of the potentiometer moved along with the foot, while the shaft tip remained fixed to a special bracket that was bolted to the frame of the compactor. The potentiometer was calibrated to read 2.54 cm (1 in) of foot movement for every 10 divisions (10 mm) on the recording chart.

Procedure

To compact a sample, a Standard Proctor mold of 944 cm^3 ($1/30 \text{ ft}^3$) and a collar were bolted down on the rotating table of the compactor. Enough soil for one of five layers of approximately equal thickness was placed in the mold. Before compaction was begun, the foot was positioned so that it would be at the bottom of its stroke when the

machine was turned on. (If this was not done the foot would develop high pressure and plunge into the soil.) The foot was then lowered until the selected pressure had developed on the soil and further downward movement of the foot had stopped. The shaft of the potentiometer was extended 12.7 cm (5 in) to allow room for the stroke of the foot, and the end was bolted to the slotted bracket on the compactor frame. The slot would allow the potentiometer shaft to be fixed at successively higher positions for the next four layers. At this point, the compactor was turned on and allowed to run at its fixed rate of 30 tamps/min for one minute. After the machine stopped, the potentiometer was unbolted and the foot raised so that soil could be added for the next layer. This procedure was followed for each of the five layers. Scarifying of the top of the compacted layer was done only for very dry samples compacted at high energy levels. The fifth and last layer brought the soil 3 to 6 mm ($1/8$ to $1/4$ in) above the mold. The mold was unbolted from the machine, the collar removed, and the excess soil was trimmed away. The mold was then weighed for density determination.

Sampling and Trimming

Equipment

The hydraulic jack that was used for extrusion of the compacted soil is shown in Figure 2.3. Also shown in the figure is the split mold and seamless, stainless steel

FIGURE 2.3 EXTRUSION AND SAMPLING JACK

tubes used to sample the compacted soil. The tubes were 38.10 mm outside diameter, 35.61 mm inside diameter and about 85 mm long (1.500 in O.D., 1.402 in I.D. and 3.25 in long). The piston and circular plate, shown with a tube in Figure 2.4, were used to extrude the sample from the tube.

Procedure

The frame to which the hydraulic jack was fixed is designed for Proctor molds, so the compacted sample was easily jacked out. This sample was then placed in the split mold as shown in Figure 2.3. Two lubricated tubes were positioned on top of the compacted soil and the same jack was used to push the tubes into the compacted soil, using a board as a reaction for the tubes. Although only one sample was to be tested, two were taken as a precaution. As the tubes were jacked into the soil, the split mold was slowly expanded by loosening the bolts on the front of the mold. This allowed room for the volume of the tubes, but still provided confinement to the soil so that it would not crack and fall apart as the tubes were pushed into it. The jack was used once again to extrude the sampled soil from the split mold.

Next, the soil around the tubes was cut away and used for water content samples, with the soil at the ends of the tubes being trimmed flush so that the piston (shown in Figure 2.4) could be used to extrude the sample from the tube. This was usually done by pushing the tube down over the piston by hand. However, for some samples it was necessary to jack them out using the special plate shown in

a) Sampling tube, b) Extrusion piston, c) Reaction plate

FIGURE 2.4 SAMPLING EQUIPMENT

Figure 2.4 as a reaction for the tube. This plate was designed to fit the top plate of the hydraulic jack apparatus, and had a recessed cut to hold the tube.

The extruded samples were then laid in a metal cradle of the same diameter as the samples. The ends of the cradle were perpendicular to the axis of the sample and served as guides for the fine trimming of the sample ends. A sharp stiff bladed knife was the cutting tool. After taking the dimensions with a vernier caliper, with accuracy of 0.001 in (0.0254 mm), and weighing the sample on a balance of 0.01 gm sensitivity the sample was put directly into the triaxial cell.

The Triaxial Test

Equipment

The triaxial cell was manufactured by Geotest of Wheeling, Illinois and is shown in Figure 2.5. The bearings and the seal around the piston are designed such that there is no leakage of the confining fluid (water). The cell was supplied with drainage type bases. These tests were undrained without pore pressure measurements, and all possible air traps and leakage sources had to be eliminated. Therefore, a smooth base without drains was machined and fitted to the base of the cell. Wherever pipe or valve fittings were used, male ends were chamfered 45° on the inside to eliminate bubble traps.

FIGURE 2.5 TRIAXIAL CELL AND LOADING APPARATUS

As seen in the literature review, the compressive strength of partially saturated soil is highly dependent on void ratio and saturation. Because of the volume change of unsaturated samples that occurs in the triaxial test, these variables also change. Therefore, it is desirable to measure the changes in volume.

Many different methods have been used to measure volume changes in the triaxial test. Some of these methods, their advantages and disadvantages, and the choice of the type selected for this study are discussed in Appendix A. The volume change measurement apparatus used was like that designed by Chan and Duncan (1967). It is basically a burette type of instrument that measures the quantity of water that flows into and out of the triaxial cell chamber due to changes in the sample volume. For accurate measurement in a burette, the internal diameter of the tube must be small. However, the use of a small internal diameter tube may result in a large change in head and hence change the cell pressure. This apparatus overcomes this problem by the use of a fine tube that terminates in a large diameter reservoir, so that head changes are negligible. The volume change in the tube is read by an oil-water interface. When the interface nears the end of the tube, the flow can be reversed by the turn of a single valve. This apparatus is shown in Figure 2.6. Volume changes of 0.01 cm^3 can be read. The details of the apparatus and a description of problems encountered with it are included in Appendix A.

a) Fine volume measuring tube	e) Pressure transducer
b) Large volume measuring burette	f) Reservoir
c) Oil interface	g) To triaxial cell
d) Flow reversal valve	

FIGURE 2.6 VOLUME CHANGE MEASURING DEVICE

The confining fluid was deaired, deionized water, supplied by gravity feed from an air evacuated reservoir. The cell pressures were applied by regulated air over water in the reservoir on the volume change board. The cell pressures were measured to within 0.5 kN/m^2 (0.1 psi) by the use of a strain-gage type electrical transducer model AB 100 psi (690 kN/m^2) made by Tyco.

Load was measured with a 2225N (500 lb) capacity, model U3G1, BLH load cell. The calibrated voltage output was read on a Hewlett Packard digital multimeter to an accuracy of 0.5 kN/m^2 (0.1 psi).

Axial deformation was applied by a constant strain rate loading frame manufactured by Wykenham-Farrance. This is shown in Figure 2.5 with the triaxial cell in place. The sample deformation was measured with a dial gage of 0.001 in. sensitivity.

Procedure

Because the pedestal for the sample was not high enough to allow sufficient piston travel, a lucite spacer was used. The sample was placed on this spacer on top of the pedestal. Two thin rubber membranes (prophylactics) were attached to a lucite cap by a rubber "O" ring. This smooth bottomed cap, which had a recess on top to accommodate a ball bearing was placed on top of the sample. One membrane was carefully rolled down over the sample far enough to cover the recesses in the pedestal that were the seats

for the "O" rings. The first membrane was covered with a thin coating of silicon oil. This reduced air leakage and also caused the second membrane to stick to the first, eliminating trapped air between them. The second membrane was then rolled down over the first and secured with "O" rings; two on the bottom and one more on top. The top of the cell was secured to the base, and the cell was filled with water. The above procedure was followed even when no confining pressure was to be applied, so that volume changes could be measured. If no confinement was used, axial loading was started immediately.

The triaxial cell was unfortunately quite flexible. When confining pressure was applied, a considerable expansion of the cell would occur, causing water to flow in to fill this space. The small diameter tube on the right of the meter stick in Figure 2.6 could not be used to measure this volume, because when the oil moved too quickly it would stick and separate. Therefore the larger diameter burette on the left of the meter stick in Figure 2.6 was used to measure volume change during application of the confining pressure. However, even the large burette did not have enough capacity for the higher confining pressures, so when the water level reached the bottom of the burette, it had to be refilled. This was done by closing off the valve to the cell and opening a valve to a hand pump. After refilling, the valve to the pump was closed and the valve to the cell

reopened as the pressure was increased to that desired.

The calibration curve for the cell expansion vs. confining pressure is shown in Appendix A, Figure A2. The cell expansion over most of the range of pressure was quite constant. However, for the first 69 kN/m^2 (10 psi) of confining pressure applied there was variation from linearity. This was probably due to unavoidable trapping of air bubbles, and seating effects. Many trial applications of the first 69 kN/m^2 of pressure were run and an average of the volume change occurring during this pressure change was 1.1 cm^3 more than the volume change expected according to the constant expansion calibration rate found for pressures greater than 69 kN/m^2 .

The triaxial cell also underwent creep. At the highest level of confining pressure used (414 kN/m^2 (60 psi)), a change in volume of 2 cm^3 would occur due to creep over a period of five hours. (Five hours was the length of the longest test.) The calibration curve for the creep volume change vs. time is shown in Appendix A, Figure A3. Since most of the creep occurred in a short time, the large burette remained open to the cell for 15 minutes. At the end of this time, the burette was closed off and the valve to the fine volume measuring tube was carefully opened (the tube on the right of the meter stick in Figure 2.6). The axial loading was started one minute later (16 minutes after application of confining pressure).

The cell was also calibrated for volume change due to changes in temperature. This calibration curve is shown in Appendix A, Figure A4. Although the testing was done in a small room of fairly constant temperature (23°C), the temperature could rise 0.5°C over a period of five hours due to equipment and body heat. This would result in no more than 0.13 cm^3 of volume change.

Application of the load was at a constant rate of strain of 0.058 mm/minute (0.0023 in/min.). This rate was chosen to be close to the same rate used in the consolidated-undrained triaxial testing phase of this project. This rate was shown to be acceptable by Johnson (1979) for this soil in a completely saturated condition. The samples compacted wet-of-optimum in this study are fairly close to full saturation, so this rate was an acceptable approximation.

However, because of the presence of both air and water in partially saturated soil, the permeabilities may be quite different than that for water only in saturated soil. Hence, pore pressures may not equilibrate, causing pore water migration. To accurately determine the best strain rate at each desired water content and density would involve a large testing program in itself. Therefore, a single strain rate was used for all water contents.

During loading, readings of load, axial deformation, volume change and temperature were recorded. Volume change readings were corrected using the calibrations for temperature, piston displacement, and cell expansion and creep.

Stress and axial and volumetric strains were computed from these data by a computer program written for this purpose. The cross-sectional area, A , used for the stress computations was calculated by:

$$A = A_c \frac{(1-v)}{(1-\epsilon)} \quad (2.1)$$

where: A_c = the cross-sectional area after application of confining pressure
 v = the volumetric strain occurring during shear
 ϵ = the axial strain occurring during shear

The triaxial test was run until the sample reached its peak compressive strength or 20% axial strain, whichever came first. At the completion of the test, the cell was drained and the sample removed.

As a check on the sample volume and saturation at test completion, a procedure used by Scott (1977) was followed. The sample was weighed in air, sealed in a coating of paraffin, and again weighed in air. The waxed sample was also weighed in water by suspending it in a basket attached to a balance. The specific gravity of the wax and the Archimedes Principle were used to compute the volume of the sample. The wax was then cut away and the sample was put into a 105°C oven for water content determination.

RESULTS AND DISCUSSION OF RESULTS

Testing Program

The results of impact compaction tests on St. Croix clay are shown in Figure 3.1. The three compaction curves are the result of three different energy levels that are described in Table 3.1. For samples that were tested in triaxial compression, there were four water content levels at each of the energy levels. These water content levels were chosen at equal saturation levels, viz., by the intersections of equal saturation lines of 70.0, 77.5, 85.0 and 92.5%, with the compaction curves. Dry density levels for sample preparation were also defined by those intersections. For example, the constant saturation line of 70.0% in Figure 3.1, intersects the Low Energy Proctor curve at $\rho_d = 1528 \text{ kg/m}^3$ and $w = 20.75\%$. Each of the twelve points (three energy levels each with four water contents) was determined in a similar way.

For each level of density and water content, samples were tested at confining pressures of 0, 138, 276 and 414 kN/m^2 (0, 20, 40 and 60 psi). The confining pressure of 414 kN/m^2 approximately corresponds to the vertical pressure a sample would experience at an embankment depth of 20m (70 ft). To investigate the behavior between 0 and 414 kN/m^2

FIGURE 3.1 IMPACT COMPACTION RESULTS

Table 3.1 Description of Impact Compaction Energy Levels

Compaction Level	Number of Blows/Layer	Number of Layers	wt., kg(lbs)/Drop, m(in)	Hammer	Volume of Mold, cm ³ (ft ³)
Low Energy Proctor	15	3	2.5(5.5)/0.305(12)		944 (1/30)
Standard Proctor	25	3	2.5(5.5)/0.305(12)		944 (1/30)
Modified Proctor	25	5	4.5(10)/0.457(18)		944 (1/30)

the two other confining pressures were chosen. In addition, complete replication of the tests at all these levels was planned, although not all were carried out because the soil supply was exhausted.

A nomenclature system for these tests was defined as follows. The confining pressure was denoted by the letter C and a number of 0, 1, 2 or 3: C0 for zero confining pressure, C1 = 138 kN/m^2 , C2 = 276 kN/m^2 and C3 = 414 kN/m^2 . The energy level was represented by the letters L, S, or M, for Low Energy Proctor, Standard Proctor and Modified Proctor, respectively. A number 1 to 4 accompanies this letter to define the saturation level, 1 = 70.0%, 2 = 77.5%, 3 = 85.0% and 4 = 92.5%. To designate whether it was the first test or a replication, the number 1 or 2 follows. For example, the test number C1-L1-1 would indicate:

- (C1) the confining pressure in the triaxial test was 138 kN/m^2 ;
- (L1) the soil was compacted at Low Energy Proctor level and saturation level 1 (70.0%);
- (1) the first test at these levels.

A complete guide to the numbering system is given in Table 3.2.

The samples for triaxial testing were actually formed by kneading compaction with selected foot pressures. The compaction curves differed for the kneading and impact modes as seen in Figure 3.2. For a single kneading compaction foot pressure, dry-of-optimum, the density increased

Table 3.2 Test Numbering Guide

Test Number	Confining Pressure kN/m ²	Energy Level	Water Content %	Dry Density kg/m ³
C0-L1*	0	Low Energy	20.75	1528.1
C1-L1	138	Low Energy	20.75	1528.1
C2-L1	276	Low Energy	20.75	1528.1
C3-L1	414	Low Energy	20.75	1528.1
C0-L2	0	Low Energy	22.20	1548.9
C1-L2	138	Low Energy	22.20	1548.9
C2-L2	276	Low Energy	22.20	1548.9
C3-L3	414	Low Energy	22.20	1548.9
C0-L3	0	Low Energy	24.00	1560.1
C1-L3	138	Low Energy	24.00	1560.1
C2-L3	276	Low Energy	24.00	1560.1
C3-L3	414	Low Energy	24.00	1560.1
C0-L4	0	Low Energy	27.00	1537.7
C1-L4	138	Low Energy	27.00	1537.7
C2-L4	276	Low Energy	27.00	1537.7
C3-L4	414	Low Energy	27.00	1537.7
C0-S1	0	Standard Proctor	19.00	1587.4
C1-S1	138	Standard Proctor	19.00	1587.4
C2-S1	276	Standard Proctor	19.00	1587.4
C3-S1	414	Standard Proctor	19.00	1587.4
C0-S2	0	Standard Proctor	20.10	1616.2
C1-S2	138	Standard Proctor	20.10	1616.2
C2-S2	276	Standard Proctor	20.10	1616.2
C3-S2	414	Standard Proctor	20.10	1616.2
C0-S3	0	Standard Proctor	21.60	1632.2
C1-S3	138	Standard Proctor	21.60	1632.2
C2-S3	276	Standard Proctor	21.60	1632.2
C3-S3	414	Standard Proctor	21.60	1632.2
C0-S4	0	Standard Proctor	25.00	1590.6
C1-S4	138	Standard Proctor	25.00	1590.6
C2-S4	276	Standard Proctor	25.00	1590.6
C3-S4	414	Standard Proctor	25.00	1590.6

Table 3.2 (continued)

Test Number	Confining Pressure kN/m ²	Energy Level	Water Content %	Dry Density kg/m ³
C0-M1	0	Modified Proctor	13.80	1800.4
C1-M1	138	Modified Proctor	13.80	1800.4
C2-M1	276	Modified Proctor	13.80	1800.4
C3-M1	414	Modified Proctor	13.80	1800.4
C0-M2	0	Modified Proctor	15.00	1811.6
C1-M2	138	Modified Proctor	15.00	1811.6
C2-M2	276	Modified Proctor	15.00	1811.6
C3-M2	414	Modified Proctor	15.00	1811.6
C0-M3	0	Modified Proctor	16.30	1814.6
C1-M3	138	Modified Proctor	16.30	1814.6
C2-M3	276	Modified Proctor	16.30	1814.6
C3-M3	414	Modified Proctor	16.30	1814.6
C0-M4	0	Modified Proctor	19.00	1784.6
C1-M4	138	Modified Proctor	19.00	1784.6
C2-M4	276	Modified Proctor	19.00	1784.6
C3-M4	414	Modified Proctor	19.00	1784.6

*The number 1 or 2 placed here indicates 1st or 2nd test.

FIGURE 3.2 CORRESPONDENCE BETWEEN KNEADING AND IMPACT METHODS
(After Di Bernardo, 1979)

faster with increase in water content than was the case for a constant impact energy level. Hence, the designation of an energy level as Modified, Standard or Low Energy Proctor means that the sample was compacted at the kneading compaction pressure required to obtain a sample of selected water content and density on one of the impact curves.

Compaction Results

As previously noted, soil was first compacted by the kneading compactor in a Proctor mold and sampled with small tubes for the triaxial test. The soil compacted into the Proctor mold will hereafter be referred to as the "as-compacted" soil, and the small samples for triaxial testing as "triaxial samples" or simply "samples".

The compaction results of the as-compacted soil are shown in Figure 3.3 and tabulated in Table 3.3. For comparison of densities obtained, to the desired densities, the curves from impact compaction are superimposed on Figure 3.3.

Also shown in Table 3.3 are the dry densities and degrees of saturation of the triaxial samples prior to testing. The water contents were taken to be the same as for the as-compacted soil, since measured differences averaged only 0.2%.

The density-water content relationship for triaxial samples before testing is shown in Figure 3.4. There is much more scatter than in Figure 3.3 for the as-compacted soil, which is due in part to the sampling process.

FIGURE 3.3 COMPACTION RESULTS OF AS-COMPACTED SAMPLES PREPARED BY KNEADING COMPACTION TO EQUIVALENT PROCTOR DENSITIES

Table 3.3 Compaction Results of As-Compacted Soil and
Triaxial Samples

Test Number	Water Content %	As-Compacted		Triaxial Samples	
		Density kg/m ³	Saturation %	Density kg/m ³	Saturation %
C0-L1-1	20.58	1537.6	70.36	1560.3	73.02
C0-L1-2	21.24	1523.1	71.32	1541.7	73.33
C1-L1-1	20.70	1523.5	69.56	1506.8	67.97
C2-L1-1	20.81	1524.9	70.05	1535.0	71.16
C2-L1-2	20.67	1540.3	71.16	1539.5	71.15
C3-L1-1	20.42	1525.1	68.77	1514.5	67.80
C3-L1-2	20.95	1519.9	70.02	1464.2	64.68
C0-L2-1	21.94	1559.8	77.70	1551.0	76.80
C0-L2-2	22.65	1539.3	77.88	1540.1	78.04
C1-L2-1	21.98	1550.4	76.76	1560.9	78.03
C1-L2-2	21.85	1559.9	77.30	1601.5	82.33
C2-L2-1	22.46	1543.5	77.70	1575.2	81.43
C3-L2-1	22.00	1553.6	77.22	1577.4	80.04
C0-L3-1	23.72	1579.8	86.50	1582.9	86.98
C0-L3-2	23.91	1573.4	86.38	1595.7	89.48
C1-L3-1	23.84	1554.9	83.82	1582.9	87.41
C1-L3-2	23.76	1560.3	84.22	1584.5	87.33
C2-L3-1	23.99	1560.8	85.09	1586.7	88.46
C3-L3-1	23.81	1577.1	86.49	1573.6	86.12
C3-L3-2	23.52	1565.8	84.03	1578.9	85.74
C0-L4-1	26.60	1545.4	92.25	1577.3	93.95
C0-L4-2	26.76	1539.3	92.01	1548.8	93.37
C1-L4-1	26.41	1549.3	92.11	1550.0	92.06
C1-L4-2	26.62	1540.3	91.64	1550.1	93.06
C2-L4-1	26.76	1541.6	92.30	1560.6	94.98
C2-L4-2	26.68	1542.4	92.12	1551.3	93.43
C3-L4-1	26.94	1539.8	92.67	1535.6	92.21
C3-L4-2	26.89	1536.3	92.04	1542.5	92.95
C0-S1-1	19.24	1585.8	70.78	1607.6	73.15
C0-S1-2	19.06	1578.7	69.40	1553.9	67.01
C1-S1-1	18.93	1576.2	68.65	1545.9	65.75
C1-S1-2	18.62	1593.0	69.20	1590.0	68.95
C2-S1-1	19.68	1587.7	72.59	1600.7	74.07
C3-S1-1	19.46	1586.6	71.66	1590.0	72.07

Table 3.3 (continued)

Test Number	Water Content %	As-Compacted		Triaxial Samples	
		Density kg/m ³	Saturation %	Density kg/m ³	Saturation %
C0-S2-1	20.04	1610.8	76.46	1616.2	77.15
C0-S2-2	19.86	1613.0	76.02	1648.4	80.28
C1-S2-1	20.07	1602.1	75.59	1624.7	78.23
C2-S3-1	19.93	1619.9	77.07	1587.9	73.60
C2-S2-2	20.04	1611.6	76.55	1637.2	79.59
C3-S2-1	20.10	1607.9	76.36	1598.4	75.40
C0-S3-1	21.96	1632.4	86.48	1652.1	89.15
C0-S3-2	21.57	1632.7	85.01	1630.5	84.82
C1-S3-1	21.59	1639.1	85.90	1643.0	86.49
C2-S3-1	21.89	1617.5	84.34	1609.2	83.41
C2-S3-2	21.39	1630.1	83.99	1620.5	82.90
C3-S3-1	21.40	1643.1	85.62	1649.0	86.47
C3-S3-2	21.29	1631.9	83.80	1629.7	83.62
C0-S4-1	24.77	1593.5	92.14	1597.6	92.80
C0-S4-2	24.57	1598.6	92.09	1594.3	91.60
C1-S4-1	24.90	1595.4	93.03	1597.5	93.28
C1-S4-2	25.10	1580.2	92.21	1582.3	91.94
C2-S4-1	24.78	1591.5	91.91	1598.3	92.92
C2-S4-2	24.66	1598.3	92.37	1605.2	93.39
C3-S4-1	25.09	1587.7	92.56	1586.1	92.94
C3-S4-2	24.97	1587.1	92.03	1604.2	94.47
C1-M1-1	13.80	1800.7	70.18	1835.5	74.23
C1-M1-1	13.65	1807.3	70.13	1764.7	65.70
C2-M1-1	13.75	1800.4	69.89	1782.0	68.00
C2-M1-2	13.86	1796.3	69.98	1775.7	67.87
C3-M1-1	13.66	1803.6	69.56	1780.6	67.37
C0-M2-1	14.92	1812.3	77.29	1821.4	78.57
C0-M2-2	15.06	1815.3	78.36	1802.7	76.92
C1-M2-1	15.09	1807.8	77.62	1777.8	74.12
C2-M2-1	15.05	1815.5	78.35	1789.8	75.34
C2-M2-2	14.86	1815.5	77.83	1814.5	77.32
C3-M2-1	14.77	1814.5	76.75	1796.9	74.75
C3-M2-2	15.00	1819.8	78.63	1785.7	74.61

Table 3.3 (continued)

Test Number	Water Content %	As-Compacted		Triaxial Samples	
		Density kg/m ³	Saturation %	Density kg/m ³	Saturation %
C0-M3-1	16.56	1811.0	85.57	1792.4	83.20
C0-M3-2	16.34	1820.1	85.70	1800.9	83.25
C1-M3-1	16.53	1831.5	88.24	1794.2	83.30
C1-M3-2	16.14	1816.9	84.19	1798.7	81.91
C2-M3-1	16.02	1820.1	83.98	1807.6	82.45
C2-M3-2	16.60	1819.0	86.85	1817.6	86.78
C3-M3-1	16.07	1829.9	85.55	1820.4	84.39
C0-M4-1	18.89	1776.4	92.49	1769.5	91.51
C1-M4-1	19.00	1774.2	93.15	1776.1	93.09
C1-M4-2	19.00	1784.6	94.20	1756.4	90.29
C2-M4-1	19.20	1779.8	94.49	1779.6	94.59
C2-M4-2	19.00	1780.6	95.12	1781.4	95.37
C3-M4-1	18.95	1781.4	93.51	1753.6	89.69

FIGURE 3.4 DRY DENSITY - WATER CONTENT RELATIONSHIP FOR TRIAXIAL TEST SAMPLES BEFORE TESTING

Differences between as-compacted and triaxial samples are also due to the smaller sample size. As Gau and Olson (1971) demonstrated for dry and near optimum moisture contents, density variations occur throughout a mass of soil compacted in a Proctor mold. These are averaged out for the entire as-compacted soil. However, the small triaxial sample may have been taken from one of the areas of local variation.

Measurement of Work in Compaction

It has usually been the practice to take the work done in compaction as the nominal value, viz., the total work done in operation of the compaction machine. These values do not describe the actual work done in compacting the soil since they include energy lost in the machine operation, and the work done on the soil which does not produce residual densification.

The work that produces permanent densification is the force times the residual densification deformation, viz., the displacement that occurs due to the densification of the soil only. This is the most descriptive measure of work since it excludes all energy losses. However, the residual densification is difficult to determine with the short time duration between tamps with automatic compaction machines. Measures of foot pressure and deformation under loading with time are reasonably convenient, and were used to represent compactive work in this study. This quantity is taken to

exclude the energy losses in the compactor and in the transfer of energy from the compactor into the soil. It does not exclude the energy expended in a non useful fashion to produce elastic deformation of the soil. Also, this work measurement may include non useful energy for samples that are compacted wet-of-optimum. When wet samples approach full saturation, vertical foot displacements may result in shear displacement in the soil beneath it, which in turn causes heaving. The rebound and heaving losses are most significant after a number of tamps have acted to densify the layer of soil.

Calculation of Work Done

Schematic load and displacement curves for a single tamp of the kneading compactor are shown in Figure 3.5. The upper curve of the plot is the foot displacement with time; the lower curve shows foot loading with time.

To determine the work done on the soil, point (a) was interpreted as the start of loading and therefore point (b) was taken as the position of the foot as it contacted the soil. Compression of the soil continued to point (c) in Figure 3.5, where no further downward movement of the foot occurred. Force application within the time period (a-d) was considered to do compactive work. Loading beyond point (d) caused no displacement and hence no further work was done on the soil.

FIGURE 3.5 TYPICAL LOAD AND DISPLACEMENT CURVES FROM RECORDER OUTPUT FOR WORK MEASUREMENT

The work done on the soil for this tamp can be expressed in terms of definite increments as:

$$\left(\frac{\Sigma \Delta F \Delta t}{\Sigma \Delta t} \right) \left(\frac{\Sigma \Delta d \Delta t}{\Sigma \Delta t} \right) = \Sigma \Delta F \Delta d \quad {}^3 \quad (3.1)$$

The $\Sigma \Delta F \Delta t$ is the area under the load time curve between lines (a-b) and (c-d). The area under the displacement curve between these same lines is $\Sigma \Delta d \Delta t$, and $\Sigma \Delta t$ is the time interval. The area under the curves was determined by counting the squares of the 1mm grid on the recorder paper. The work calculated in this way for each tamp was summed to obtain the work done on the as-compacted soil.

While this measure of work is superior to the nominal values ordinarily employed, it is still in error as it does not adjust for the elastic rebound (energy loss) which occurs upon removal of the foot, or for the shearing displacements that do not cause desiccation but heave in wet-of-optimum samples.

Work Measurement Data

Sample recorder output from a single layer of a Low Energy Proctor, saturation level 1 (L1) test is shown in Figure 3.6. The number associated with each set of displacement and load curves indicates the i^{th} tamp of the 30 tamps on that layer. It is seen in Figure 3.6 that in tamps 1, 2, and 3, the soil displacements are considerably larger (e.g., 2.3 cm (0.9 in)) than those of tamps 14, 15 and 16

³This expression is a product of the time averaged force and displacement.

FIGURE 3.6 TYPICAL RECORDER OUTPUT FOR WORK MEASUREMENT AT AN L1 LEVEL

(e.g., 0.38 cm (0.15in). Also, the time over which they occur is longer (0.36 sec. vs. 0.1 sec.) This is even more evident in the comparison of tamps 1, 2 and 3 to tamps 28, 29 and 30. Note also that the slopes of the load-time curves get continually steeper from tamp 1 to tamp 30.

The relationship between the end of downward foot movement and the peak of foot load, changes with the tamp sequence. For the earlier tamps, the peak load occurs first; for the intermediate tamps, the two events occur at about the same time. For later tamps, the peak occurs after the end of downward foot movement.

Figures for S1 and M1 levels are shown in Appendix B, Figures B1 and B2 respectively. The trends in Figure B1 and Figure 3.6 are generally applicable for all energy levels and water contents investigated. Levels M1 and M2 also follow the general trend except that, after the first few tamps, peak load occurs considerably later after downward foot movement ends. Figure B2 shows typical behavior of M1 and M2 levels during compaction.

The values of work applied to the soil, computed from output like that shown above, are given in Table 3.4. At each level of water content and nominal energy, the work measurements show considerable scatter. Part of this may be due to error caused by occasional malfunction of the recording equipment. Other factors contributing to the variation in work measurements were:

Table 3.4 Results of Work Measurements During Compaction

Test Number	Work kJ/m ³	Test Number	Work kJ/m ³	Test Number	Work kJ/m ³
C0-L1-1	243	C0-S1-1	389	C1-M1-1	1650
C0-L1-2	197	C0-S1-2	246	C1-M1-2	1440
C1-L1-1	228	C1-S1-1	325	C2-M1-1	1540
C2-L1-1	207	C1-S1-2	251	C2-M1-2	1760
C2-L1-2	251	C2-S1-1	518	C3-M1-1	1600
C3-L1-1	250	C3-S1-1	472		
C3-L1-2	198			C0-M2-1	1710
		C0-S2-1	435	C0-M2-2	1200
C0-L2-1	76	C0-S2-2	322	C1-M2-1	1680
C0-L2-2	198	C1-S2-1	428	C2-M2-1	1670
C1-L2-1	132	C2-S2-1	350	C2-M2-2	1480
C1-L2-2	166	C2-S2-2	380	C3-M2-1	1690
C2-L2-1	136	C3-S2-1	313	C3-M2-2	1700
C3-L2-1	172				
		C0-S3-1	331	C0-M3-1	1950
C0-L3-1	189	C0-S3-2	267	C0-M3-2	1640
C0-L3-2	93	C1-S3-1	316	C1-M3-1	2070
C1-L3-1	108	C2-S3-1	398	C1-M3-2	1510
C1-L3-2	122	C2-S3-2	282	C2-M3-1	1820
C2-L3-1	146	C3-S3-1	273	C2-M3-2	1690
C3-L3-1	224	C3-S3-2	375	C3-M3-1	1370
C3-L3-2	183				
		C0-S4-1	330	C0-M4-1	1590
C0-L4-1	285	C0-S4-2	244	C1-M4-1	1390
C0-L4-2	138	C1-S4-1	289	C1-M4-2	1320
C1-L4-1	214	C1-S4-2	320	C2-M4-1	1490
C1-L4-2	143	C2-S4-1	216	C2-M4-2	1550
C2-L4-1	149	C2-S4-2	321	C3-M4-1	1660
C2-L4-2	137	C3-S4-1	263		
C3-L4-1	210	C3-S4-2	321		
C3-L4-2	254				

1) Water content variation,

2) Soil aggregate size variation. Aggregate sizes were not measured; their size and distribution was that obtained naturally from the mixing process. Since aggregates of different sizes have different contact stresses at the same compactive load, variation in measured work will result.

3) Variation in amount of trimmed soil. The work measurements in Table 3.4 are normalized with respect to the volume of the Proctor mold. However, the work measurements also included the work done on the soil that was subsequently trimmed away from the top of the mold after compaction was complete. The amount of soil trimmed away of course varied between tests.

The averages of the amount of work in all the compaction tests at a particular level of saturation and nominal energy are given in Table 3.5. Also given is the average work as a percent of the average work at optimum water content (which is approximately 85.0% saturation for all nominal energy levels).

Table 3.6 gives the differences between the average work done at two nominal energy levels at the same saturation level. For example, the difference between the average work of L1 and S1 is 142 kJ/m^3 . The average of these differences are shown in part (b), and the averages were used to compute the average work ratio, W_R , in part (c) of Table 3.6.

Table 3.5 Average Foot Pressure, Work, Displacements, and Load During Compaction

Water Content- Energy Level	Average Foot Pressure, kN/m^2	Average Work Done, kJ/m^3	Percent of Average Work at Optimum Water Content	Average Displacement per Tamp, cm	Average Load per Tamp, N
L1	538	225	148.0	0.30	454
L2	460	147	96.7	0.30	276
L3	394	152	100.0	0.30	285
L4	394	191	125.7	0.38	298
S1	827	367	114.7	0.28	859
S2	827	371	115.9	0.30	819
S3	683	320	100.0	0.30	650
S4	525	288	90.0	0.41	467
M1	3191	1597	92.8	0.36	2706
M2	3114	1590	92.4	0.38	2577
M3	3054	1721	100.0	0.38	2986
M4	2466	1500	87.2	0.36	2826

Table 3.6 Differences Between Average Work at Different Nominal Energy-Saturation Levels and Average Work Ratios

(a) <u>Water Content-Energy Level</u>	<u>Differences Between Average Work, kJ/m^3</u>
L1 , S1	142
L1 , M1	1372
L2 , S2	224
L2 , M2	1443
L3 , S3	168
L3 , M3	1569
L4 , S4	97
L4 , M4	1403

(b) <u>Energy Level</u>	<u>Average of Differences, kJ/m^3</u>
L , S	158
L , M	1447

(c) <u>Energy Level</u>	<u>Average Work Ratio</u> (average of work for all L tests as denominator)
L	1.00
S	1.88
M	9.09

Though these differences are obviously not constant, they are of the same order of magnitude, and resulted in an average work ratio whose significance will be shown in a later section.

Discussion of Work Measurements

For the first six tamps which provide one full face coverage, the soil is very loose and easily undergoes large compressions as air is squeezed out. These first few tamps, for which large displacements were evident in Figure 3.6, B1 and B2, account for 40 to 50% of the work done on the layer of soil. This is shown in Figure 3.7, where percent of the work done on the layer is plotted versus the number of tamps, for a test at L1, for example. As the number of tamps increases, the soil becomes denser and aggregate contact area increases so that load can be supported with less compression. The increased stiffness also accounts for the more rapid increase in load to its peak (a steeper load-time curve). Some samples such as M1 and M2, become so stiff that displacement stops well before peak load is reached, as seen in Figure B2.

Figure 3.6 also showed that the peak load for each tamp is not constant. The irregularities of the curve in Figure 3.7 are also the effect of the load variation. This non-constancy is due to the small variations in the compacted soil mass. At the same gage pressure, the kneading compactor will develop lower force on a soft elastic material than on

FIGURE 3.7 THE PERCENT OF WORK DONE ON A SOIL LAYER AS A FUNCTION OF THE NUMBER OF TAMPS

a stiff one. Hence, small local variations in the soil density will cause some variation in the load developed.

The data in Table 3.5 show that generally, for kneading compaction to densities of a constant Proctor energy level, the work done on the soil, decreases as water content increases from dry to wet-of-optimum. Hodek (1972) also found that work of compaction decreased as water content increased.

The exceptions to the above trend are due mostly to soil aggregate effects. The larger amount of work, with respect to optimum, of level L4 is due to the existence of very large aggregates, or actually balls of clay. The water content of level L4 was 27.0%, which was well above the plastic limit. Hence, when mixing the soil, the smaller aggregates stuck together and formed the large plastic balls. The force initially required to deform these big balls into a closer packing was large. After the deformation of the balls by the first few tamps of the compactor, little densification occurred. Additional tamps continued downward movement of the soil, but only displaced it into heave effects rather than densification.

The levels of M1 and M2 were also exceptions to the general trend of less work done on the soil as water content increased. As shown in Table 3.5, the work at these levels is slightly lower than that at optimum (M3). This table also shows that the average force per tamp for the M1 and

M2 level is also lower than that at optimum, even though the foot pressures were higher. The reason for this is found in Figure B2 of Appendix B. After the first few tamps, the densification stops well before peak load was reached. Hence, the total load causing densification is smaller, and as a result, so is the amount of work done on the soil.

The measured work that was done on the soil decreased as water content increased along a curve of constant nominal impact energy. It is not known what compactive work is done in the impact mode. However, as shown in Figure 3.2, the nominal value of foot pressure in kneading compaction follows the same trend as the measured work, that is, it decreases, while nominal energy of impact compaction to the same densities remains constant. Hence, it is apparent that the impact method is a less efficient compaction method for this plastic soil.

The nature of the dynamic loading is one reason for the less efficient nature of the impact method. Because of a coefficient of restitution between impact hammer and soil, the hammer retains energy after the collision and bounces off the soil. In the kneading method however, the foot pressure is applied more slowly and is not allowed to bounce, but dwells on the surface momentarily.

The impact method may also lose more energy in behavior like that seen for kneading of M1 levels in Figure B2, where the densification ends well before peak available

load is reached. The more rapid rise time of the load in impact does not allow the soil skeleton to respond and more load may be taken up by the pore pressure.

The effect of pore pressures may also be the reason for the trend toward the constant average work ratio shown in Table 3.6. Although the work measured in kneading compaction along a constant impact energy curve was not constant, the ratio of the work between impact curves, compared at constant saturation was more nearly a constant (for example, the ratio of the work between L1 and S1, compared to the work ratio between L2 and S2). As discussed in Chapter 1, saturation is a major factor in the value of the parameter χ . Hence, along a constant saturation line, the pore pressures will have a more similar behavior in response to the compaction load.

The average work ratio by kneading compaction is compared to the average ratio of kneading foot pressure, and to the ratio of nominal impact energy in Table 3.7. As shown, the ratios are very similar. The magnitude of the energy losses in impact compaction are unknown. However, the data of Table 3.7 indicate that they are about the same proportion of the nominal energy as for kneading compaction. However, these ratios do not indicate the magnitude of the work to accomplish a certain amount of compaction, or the numerical efficiencies of the compactive modes.

Table 3.7 Comparison of Work and Nominal Energy Ratios

Nominal Energy Level	Ratio of Average Work by Kneading Compaction	Ratio of Average Kneading Foot Pressure	Ratio of Nominal Impact Energy
L	1.00	1.00	1.00
S	1.88	1.60	1.67
M	9.09	6.62	7.61

Unconsolidated-Undrained Shear Strength

The results of the unconsolidated-undrained triaxial tests are given in Table 3.8. As previously mentioned, the strength was defined at peak stress or the stress at 20% axial strain. For the failure condition, the points $p_f = (\sigma_1 + \sigma_3)_f / 2$ are plotted vs. $q_f = (\sigma_1 - \sigma_3)_f / 2$ in Figures 3.8, 3.9 and 3.10. Failure lines are drawn through these points and represent the relationship between one half the compressive strength and a value of one half the compressive strength plus the confining pressure.

The failure lines for the samples compacted at the Low Energy Proctor level are presented in Figure 3.8. As can be seen, the strength decreases with increasing water content or saturation. At lower water contents, the pore water pressures are more negative and hence the effective stresses and shear strength are higher.

Table 3.8 Compressive Strength Values

Test Number	Compressive Strength kN/m ²	Test Number	Compressive Strength kN/m ²	Test Number	Compressive Strength kN/m ²
C0-L1-1	264	C0-S1-1	328	C1-M1-1	1737
C0-L1-2	230	C0-S1-2	291	C1-M1-2	1655
C1-L1-1	416	C1-S1-1	524	C2-M1-1	2014
C2-L1-1	457	C1-S1-2	588	C2-M1-2	1883
C2-L1-2	566	C2-S1-1	761	C3-M1-1	2087
C3-L1-1	497	C3-S1-1	844		
C3-L1-2	643			C0-M2-1	1321
		C0-S2-1	423	C0-M2-2	1431
C0-L2-1	182	C0-S2-2	381	C1-M2-1	1472
C0-L2-2	207	C1-S2-1	616	C2-M2-1	1769
C1-L2-1	358	C2-S2-1	640	C2-M2-2	1954
C1-L2-2	394	C2-S2-2	704	C3-M2-1	2066
C2-L2-1	383	C3-S2-1	743	C3-M2-2	1829
C3-L2-1	451				
		C0-S3-1	400	C0-M3-1	1216
C0-L3-1	194	C0-S3-2	315	C0-M3-2	1333
C0-L3-2	157	C1-S3-1	376	C1-M3-1	1439
C1-L3-1	235	C2-S3-1	473	C1-M3-2	1487
C1-L3-2	278	C2-S3-2	499	C2-M3-1	1731
C2-L3-1	295	C3-S3-1	486	C2-M3-2	1548
C3-L3-1	269	C3-S3-2	479	C3-M3-1	1494
C3-L3-2	265				
		C0-S4-1	147	C0-M4-1	897
C0-L4-1	112	C0-S4-2	145	C1-M4-1	993
C0-L4-2	103	C1-S4-1	229	C1-M4-2	833
C1-L4-1	146	C1-S4-2	174	C2-M4-1	920
C1-L4-2	117	C2-S4-1	178	C2-M4-2	909
C2-L4-1	145	C2-S4-2	213	C3-M4-1	958
C2-L4-2	165	C3-S4-1	300		
C3-L4-1	131	C3-S4-2	198		
C3-L4-2	133

FIGURE 3.8 q_f vs. p_f FAILURE PLOTS WITH FAILURE LINES FOR LOW ENERGY LEVEL

FIGURE 3.9 q_f vs. p_f FAILURE PLOTS WITH FAILURE LINES FOR STANDARD PROCTOR LEVEL

FIGURE 3.10 q_f vs. p_f FAILURE PLOTS WITH FAILURE LINES FOR MODIFIED PROCTOR LEVEL

The samples for the Standard and Modified energy levels in Figures 3.9 and 3.10, respectively, show much the same behavior as those of the Low Energy Proctor level, i.e., as water content increases, strength decreases. However, exceptions are seen in the levels S1 and M1 where, at low confining pressures, the failure lines for low degrees of saturation lie below those for higher degrees. Conlin (1972) found similar behavior. The samples at low water content and saturation have more negative pore water pressures, but the capillary menisci cover little area on the soil aggregates and particles, and the result is lower average effective stress. Application of higher confining pressures compresses the fabric and allows the water to cover more area of the soil. This is in effect an increase in the parameter χ of Equation 1.3. The result is an increase in effective stress and hence the shear strength of the soil, as the envelopes of S1 and M1 show at greater confining pressures.

If the confining pressure is high enough, volume decreases during the test may be great enough to cause almost complete saturation. Hence, any further increase in confining pressure merely increases the pressures in the pore water but neither the effective stresses nor the shear strength of the soil are changed. This behavior is evident in saturation levels 1, 2 and 3 of the Low Energy and Standard Proctor failure lines in Figures 3.8 and 3.9. The failure lines for those saturation levels at the Modified

Proctor energy level in Figure 3.10 do not reach a horizontal position. The soil aggregates at these low water contents are stiffer and stronger (Hodek, 1972) and require higher confining pressures to cause significant volume change.

The failure lines of saturation level 4 of all energy levels are quite close to horizontal over the entire range of confining pressures, especially at the Low Energy level. These samples are nearly saturated, so confining pressure has little effect on the strength.

The effects of confining pressure and water content, and their relation to optimum water content is shown more clearly in Figures 3.11 to 3.13. Each figure is for a separate nominal energy level. The change in strength behavior at or about optimum water content is quite apparent. On the wet side of optimum the slopes of the strength-water content curves are much the same and nearly coincide, regardless of the confining pressure. This is true at each nominal energy level and is the consequence of a near-saturation condition. As water content decreases from optimum, the effect of confining pressure increases. Since the air voids become more continuous, the water less continuous, and the sample is less saturated, more of the confining pressure is taken up by the soil fabric rather than the pore water. This increases the effective stress and shear strength of the soil.

FIGURE 3.11 WATER CONTENT VS. DRY DENSITY AND COMPRESSIVE STRENGTH FOR SAMPLES COMPACTED AT LOW ENERGY PROCTOR LEVEL

FIGURE 3.12 WATER CONTENT VS. DRY DENSITY AND COMPRESSIVE STRENGTH FOR SAMPLES COMPACTED AT STANDARD PROCTOR ENERGY LEVEL

FIGURE 3.13 WATER CONTENT VS. DRY DENSITY AND COMPRESSIVE STRENGTH
FOR SAMPLES COMPACTED AT MODIFIED PROCTOR ENERGY LEVEL

Stress-Strain Behavior

Some difficulties were encountered in the measurement of volume changes during application of confining pressure. Further discussion of these problems and the data appear in Appendix D. The volume changes that are discussed in this section, occurred after confinement and throughout shear, and were more easily and reliably measured by the volume change device described earlier in Chapter 2.

In Figure 3.14 are stress-strain curves from tests on four Low Energy Proctor samples, sheared undrained without confining pressure. These results are quite similar to those of Seed and Chan (1959) shown in Figure 1.4. The dry-of-optimum samples are stiffer and the sample at the lowest water content, L1, is quite brittle. The more highly negative pore water pressure of samples at lower water content cause higher effective stresses that result in more normal stress and less shear stress between soil aggregates. Hence, less strain is required to mobilize the total shearing resistance.

This strain behavior is also dependent on the volume changes that occur in the sample. Without confining pressure, samples of low water content, dry-of-optimum, like L1 in Figure 3.14, quickly reach the maximum amount of densification under shear and begin to dilate. Peak load is reached at or shortly after this point. Table 3.9 which gives the volumetric strain values at failure and at the end

FIGURE 3.14 TYPICAL STRESS-STRAIN CURVES FOR TESTS ON ST. CROIX CLAY WITHOUT CONFINEMENT

Table 3.9 Volumetric and Axial Strain at Failure and at End of Test

Test Number	Volumetric Strain at Failure, %	Volumetric Strain at Test End, %	Axial Strain at Failure, %	Axial Strain at Test End, %
C0-L1-1	0.38	-0.01	3.00	5.18
C0-L1-2	0.39	-0.20	2.43	4.86
C1-L1-1	5.61	5.72	19.35	5.72
C2-L1-1	6.30	6.64	16.47	20.10
C2-L1-2	7.08	7.25	20.00	21.03
C3-L1-1	6.99	7.47	16.36	20.28
C3-L1-2	9.11	9.19	20.00	20.44
C0-L2-1	0.48	0.34	5.73	9.99
C0-L2-2	0.24	-0.01	6.90	8.54
C1-L2-1	4.80	4.68	20.00	20.26
C1-L2-2	4.03	4.08	20.00	20.63
C2-L2-1	5.11	5.14	20.00	20.33
C3-L2-1	4.30	4.34	19.93	20.74
C0-L3-1	1.13	1.13	6.83	12.10
C0-L3-2	0.52	0.63	10.21	12.76
C1-L3-1	1.94	1.99	17.52	20.76
C1-L3-2	3.51	3.56	19.63	20.83
C2-L3-1	2.66	2.74	18.72	20.74
C3-L3-1	1.73	1.91	11.62	16.10
C3-L3-2	1.69	1.93	8.41	14.54
C0-L4-2	0.88	0.72	11.06	15.80
C0-L4-2	0.65	0.59	13.20	16.84
C1-L4-1	1.15	0.88	13.99	21.20
C1-L4-2	0.52	0.50	18.59	20.69
C2-L4-1	1.04	1.06	18.77	20.00
C2-L4-2	0.83	0.89	17.04	20.72
C3-L4-1	0.72	0.84	15.73	20.95
C3-L4-2	0.83	0.83	13.92	20.11
C0-S1-1	0.42	0.20	2.78	3.37
C0-S1-2	-0.11	-0.91	2.01	3.22
C1-S1-1	3.39	3.59	12.01	16.59
C1-S1-2	4.18	4.38	15.80	20.49
C2-S1-1	3.07	3.36	9.61	12.18
C3-S1-1	5.35	5.78	14.07	17.44

Table 3.9 (continued)

Test Number	Volumetric Strain at Failure, %	Volumetric Strain at Test End, %	Axial Strain at Failure, %	Axial Strain at Test End, %
C0-S2-1	0.50	0.00	3.32	4.38
C0-S2-2	0.72	0.66	3.36	4.92
C1-S2-1	2.80	2.88	8.59	11.75
C2-S2-1	5.45	5.52	20.00	21.00
C2-S2-2	3.75	3.96	15.61	19.76
C3-S2-1	5.94	6.49	20.00	21.27
C0-S3-1	0.54	0.43	6.13	7.21
C0-S3-2	0.56	0.50	6.08	7.53
C1-S3-1	1.64	1.82	7.13	9.14
C2-S3-1	2.70	2.98	10.52	13.97
C2-S3-2	3.82	3.92	20.00	21.09
C3-S3-1	2.75	2.75	19.63	20.43
C3-S3-2	2.98	3.42	10.66	18.07
C0-S4-1	0.85	0.71	8.54	13.85
C0-S4-2	1.01	1.24	11.56	14.40
C1-S4-1	1.12	1.08	10.99	18.24
C1-S4-2	1.12	1.09	12.31	16.11
C2-S4-1	1.04	1.06	20.00	20.67
C2-S4-2	0.73	0.75	14.83	20.17
C3-S4-1	0.87	0.92	11.39	19.95
C3-S4-2	0.53	0.47	15.18	20.17
C1-M1-1	0.73	0.53	2.14	2.40
C1-M1-2	0.87	0.78	2.79	3.04
C2-M1-1	0.70	0.60	2.85	3.30
C2-M1-2	0.76	0.67	3.03	3.51
C3-M1-1	1.18	0.99	4.06	5.48
C0-M2-1	0.39	0.00	2.25	2.75
C0-M2-2	0.64	0.42	1.89	2.21
C1-M2-1	0.58	0.53	2.63	2.99
C2-M2-1	0.60	0.46	3.61	4.42
C2-M2-2	0.56	0.40	3.89	5.06
C3-M2-1	1.03	1.00	4.40	4.75
C3-M2-2	1.17	1.27	4.91	5.87

Table 3.9 (continued)

Test Number	Volumetric Strain	Volumetric Strain	Axial Strain	Axial Strain
	at Failure,	at Test End,	at Failure,	at Test End,
		%	%	%
C0-M3-1	0.86	0.07	3.54	4.56
C0-M3-2	0.68	0.55	3.12	3.39
C1-M3-1	0.66	0.49	4.65	5.33
C1-M3-2	0.47	0.38	3.88	4.23
C2-M3-1	0.67	0.65	4.68	5.42
C2-M3-2	0.60	0.54	5.32	6.46
C3-M3-1	0.74	1.14	3.06	3.84
C0-M4-1	0.63	0.61	6.15	6.97
C1-M4-1	0.63	0.61	6.15	6.97
C1-M4-2	0.36	0.32	7.11	7.74
C2-M4-1	0.28	0.25	6.46	8.69
C2-M4-2	0.17	0.10	7.19	8.97
C3-M4-1	0.76	0.73	7.01	10.41

of the test, shows this dilatant behavior for other dry-of-optimum tests without confinement.

As water content increases from dry-of-optimum the soil aggregates become weaker and more plastic, so they yield more before the shear strength between them is exceeded. Also, since wetter samples undergo more shearing under the compaction foot, the residual shear stresses are greater and more varied in direction. Greater strain is required to reverse and mobilize shear stresses all in one direction. In addition, because the aggregates become more plastic they will squeeze into a more dense configuration before dilation occurs. These trends described above generally apply to the results of all tests performed without confining pressure, as seen in Figures C1 to C24 in Appendix C.

Typical results of Low Energy Proctor tests at confining level 1 (138 kN/m^2 (20 psi)) are shown in Figure 3.15. All the curves show a more plastic behavior, regardless of water content. All the water content levels at the Low Energy and Standard Proctor levels are relatively high so that the soil aggregates are not very strong or stiff, and the densities at these levels are low. Hence, application of confining pressure causes considerable volumetric strain and accompanying shear stresses between aggregates. Again, these shear stresses require greater strains to mobilize them all in one direction to failure. (This "progressive

FIGURE 3.15 TYPICAL STRESS-STRAIN CURVES FOR TESTS ON ST. CROIX CLAY AT CONFINEMENT LEVEL 1

failure" concept described by the MIT (1963) report was briefly discussed in the literature review.)

Also, with application of confining pressure, the greater lateral effective stresses cause more densification to occur in dry-of-optimum samples, as evident in the comparison of Figures 3.14 and 3.15. This is more clearly shown in Figure 3.16, where volumetric strain that occurred during shear is plotted versus water content. As these volume changes continue to densify the sample, it is able to pick up more load and hence axial strain will also increase. Figure 3.17 shows how axial strain increases with the volumetric strain. This figure also shows that at one level of axial strain, the amount of volumetric strain that occurs decreases as the saturation increases. This occurred because an increasing amount of pore space was filled with incompressible water rather than air. This trend of decreasing volumetric strain with increasing water content is also evident in Figure 3.16.

Large confining pressures may result in enough compression to cause the sample to approach complete saturation, leaving less volume change to occur during undrained shear. This may be the explanation for the curves of 414 kN/m^2 (60 psi) in Figure 3.16(a) and 3.16(b) crossing those of lower confining pressure. Samples wet-of-optimum in all cases were already close enough to saturation, so that neither large nor small confining pressures have an effect in the amount of volumetric strain that occurs.

FIGURE 3J6 WATER CONTENT VS. VOLUMETRIC STRAIN AT FAILURE

FIGURE 3.17 THE INCREASE IN VOLUMETRIC STRAIN WITH INCREASING AXIAL STRAIN TO FAILURE FOR DRY-OF-OPTIMUM SAMPLES

Also immediately noticeable in Figure 3.16(c) was the low volumetric strain of all samples of the Modified Proctor level, regardless of water content and confining pressure. The stiff brittle behavior of all these samples and their dilatent tendency, is shown in Appendix C, Figures C17 to C24. The soil aggregates are strong and stiff and packed into a very dense configuration. Hence, very high confining pressures would be required to prevent dilation and cause behavior more like that at the lower energy levels, although the magnitude of volumetric strain would still not be as great.

The $e_f \sqrt{S_f}$ - $\log (q_c/2)$ Relationship

DaCruz (1963) found a linear relationship between the product of void ratio and square root of degree of saturation at failure ($e_f \sqrt{S_f}$) and the logarithm of one half the stress difference at failure ($q_c/2$). A similar plot of the compacted St. Croix clay is shown in Figure 3.18 except that the values of e and S are initial compacted values. (Because of problems encountered with the volume change measurement during application of confining pressure, e_f and S_f could not be determined). The linear relationship of Figure 3.18 is statistically represented by the equation ($R^2 = 0.89$),

$$\log(q_c/2) = 4.67 - 0.36 e_f \sqrt{S_i} \quad (3.2)$$

The scatter is due in part to the fact that some samples, particularly those dry-of-optimum and at optimum at Low

FIGURE 3.18 VOID RATIO TIMES SQUARE ROOT OF SATURATION VS. LOGARITHM OF ONE HALF THE COMPRESSIVE STRENGTH

Energy and Standard Proctor energy levels, underwent considerable changes in e and S during shear. As previously noted, decreases in volume increased with confining pressure.

Samples that are little affected by confining pressure and volume change, such as those of L4, S4, and M4, are much more tightly grouped than the other data in Figure 3.18. All of the samples of the Modified Proctor energy level follow the linear relationship quite well. As was seen in Figure 3.16, the Modified samples underwent little volume change, so initial values of e and S for these samples were close to those at failure.

Prestress and Unconsolidated-Undrained

Shear Strength

A soil will achieve a specific void ratio and fabric that is dependent upon the compactive process. However, the applied stresses are of short duration and produce compression at constant water content with only air squeezed out. To the extent that the soil skeleton reacts to the compaction pressure, the soil is prestressed by the compactive loading.

According to Olson (1963), the compacting loads are resisted by the development of effective stresses in the soil. The capillary tensions that remain after the load removal retain some of these effective stresses and further compression cannot occur until subsequent load exceeds these effective stresses. The value of the load that will just

overcome these stresses is the prestress. DiBernardo (1979) determined total stress values for prestress by one-dimensional compression tests and developed the statistical relationship:

$$\hat{P}_s = -343.14 - 0.0020w^2 P_c + 48.91 P_c^{1/2} \quad (3.3)$$

where: $\hat{P}_s$ = the estimated compactive prestress value
(kN/m²) as a total stress

w = water content, %

P_c = the kneading compactive pressure, kN/m²

Using the compaction variables of this study, values of predicted prestress were calculated by Equation 3.3. Those values were used to develop Figure 3.19.

The heavy lines in Figure 3.19 outline the surface of the undrained shear strength ($q_c/2$) at all water contents and confining pressures for one energy level. Therefore, a line such as (a-b) on the Modified Proctor surface represents one failure line of level M1 shown in Figure 3.10. Since this represents a failure line, the strength increase along it is due to the increase in confining pressure.

For any given set of compaction conditions (w, P_c) the predicted prestress value will be constant. Hence, the prestress along any such line as (a-b) is a constant. This is shown by the shaded surface in Figure 3.19, which is a surface of constant prestress, and intersects the shear strength planes along constant (w, P_c) conditions. These

FIGURE 3.19 RELATIONSHIP OF PREDICTED PRESTRESS TO UNDRAINED SHEAR STRENGTH

constant prestress surfaces intersect different shear strength planes at different energy levels. However, for the prestress to remain constant while moving to a higher energy level, the water content must decrease. In addition, as shown earlier, the range of strengths is smaller as water content increases, because of the decreasing effect of confining pressure on strength. Hence, as shown by the decreasing height of the constant prestress plane in Figure 3.19, the range of strengths for a constant prestress also decreases.

Figure 3.19 could be useful in identifying responses associated with certain values of prestress. For instance, if it was desired to design an embankment to achieve a particular value of prestress so that settlement could be controlled, the range of energy levels, water contents and associated strengths can be identified.

Statistical Correlation

The variable under investigation in a laboratory study (the dependent variable) can be statistically related to the controlled factors in the study (independent variables) by linear regression analysis. The result is an equation that can be used to predict the dependent variable by the known value(s) of the independent variable(s). Unlike a functional relationship, which is a fit by an analytical mathematical formula, a regression equation is the best fit through a set of data with scatter. The form

of this regression model is:

$$Y_i = \beta_0 + \beta_1 X_{i1} + \beta_2 X_{i2} + \dots + \beta_p X_{ip} + \epsilon_i \quad (3.4)$$

where: Y_i = the value of the dependent variable at the i^{th} trial

β_i = regression parameters that factor the independent variables

X_i = value of the independent variable of the i^{th} trial

ϵ_i = random error term with expected value $E(\epsilon_i) = 0$

$i = 1, n$

This is a multiple linear regression model, multiple in that there are two or more independent variables, which can be linear or non-linear, and linear in the regression parameters.

The expected value of Y_i when the values of X are X_i is $E(Y_i)$, with the assumptions that: $E(Y_i)$ is the mean of a probability distribution of Y_i at each X_i , the variance, σ^2 , for each distribution of Y_i at each X_i is a constant, and any two values of Y_i are uncorrelated. Therefore:

$$E(Y_i) = \beta_0 + \beta_1 X_{i1} + \beta_2 X_{i2} + \dots + \beta_p X_{ip} \quad (3.5)$$

as the expected value of the error term is zero. However, the model is based on population data. Hence, the β_i 's are unknown and must be estimated from sample data. This is done by the method of least squares, i.e., the minimum sum

of the squared deviations of the observed values of Y_i from its expected value, $E(Y_i) = \beta_0 + \beta_1 X_{i1} + \dots + \beta_p X_{ip-1}$ is,

$$Q = \sum_{i=1}^n (Y_i - (\beta_0 + \beta_1 X_{i1} + \beta_2 X_{i2} + \dots + \beta_p X_{ip-1}))^2 \quad (3.6)$$

The data are used to compute values of estimators for β_0 to β_p that minimize Q in Equation 3.6. The equations used to compute these estimators of β are derived and explained in greater detail in texts on regression analysis, such as Neter and Wasserman (1974) and Draper and Smith (1966). With the estimators b_0 to b_p , Equation 3.5 becomes,

$$\hat{Y}_i = b_0 + b_1 X_{i1} + b_2 X_{i2} + \dots + b_p X_{ip-1} \quad (3.7)$$

The number of independent variables can be from 1 to $p-1$. If the independent variables are not linearly related to the dependent variables, they can be correlated by the use of transformation by logarithm or reciprocal, polynomials, or interaction with other variables as products or quotients. A major problem is to decide which of these variables, or combinations thereof should be used in the study. There are a number of criteria and methods which can be used to resolve this problem. In this study the following procedure was used. It is similar to a method of analysis used by Price (1978) except that RIDGE REGRESSION (which tests the model for sensitivity to data changes) was not used on the models in this study.

1) The desired independent variables, whether they were simple linear, polynomial, logarithms, reciprocals, products or quotients, were plotted using the SPSS (Statistical Package for Social Sciences, Nie, et al., 1975) library of the Purdue University computing center, in scattergrams versus the dependent variable. Variables that showed poor correlation to the dependent variable were immediately eliminated if their $R^2 < 0.7$ (arbitarily chosen). Further variables of higher order were eliminated when terms of lower order were equally or better correlated to the dependent variable.

2) An all possible regressions program, DRRSQU, (also a PUCC library program) was executed with the variables selected by the above step. From the DRRSQU, regression models were selected for further investigation on the basis of:

a) a high value of R^2 and, b) a suitable value of C_p .

C_p is a measure of the total squared error in the observations fitted to any specific regression model. The total squared error has a random and a bias component. If the value of C_p is equal to p , when the number of independent variables in the equation is $p-1$, then there is no bias component. The closer C_p is to p , the less is the bias in the regression equation.

In addition, the correlation matrix for all the variables from an SPSS regression run was determined. When independent variables in a regression equation are highly correlated, the regression coefficients and a unique sum of

squares cannot be attributed to a particular variable, but must be viewed in connection with the other variables in the model. Thus, highly correlated variables are best avoided. The correlation matrix gives the correlation coefficients ($\sqrt{R^2}$) of every pair of independent and dependent variables.

It was found that most of the variables were highly correlated. Therefore, models were selected with as few variables as possible, viz., rarely more than two.

Models chosen by the above procedure were further analyzed. For models with very similar values of C_p and R^2 , the simpler model was chosen and run in an SPSS REGRESSION program with residual plots. Criteria in the REGRESSION run that were important in establishing the validity of the model were:

- 1) Low mean square error (MSE). This is an estimator of the variance (σ^2) for the regression model and is a measure of the mean of the squares of the residuals.

- 2) The overall F-test for linearity determines if all the independent variables form a model that is significant in accounting for the variation in the dependent variable. If the F value determined from the REGRESSION run is greater than F^* , the statistic of the F- distribution at a level of significance $\alpha = 0.05$, the model is accepted as significant.

- 3) Confidence intervals for each b_i coefficient. Tested at the $\alpha = 0.05$ level, the interval should be small and not include zero. If the interval crossed zero, it was

concluded that the variable in that particular model is insignificant in explaining variation in the dependent variable.

4) Trends of residuals. A residual is the difference of the observed value of a dependent variable and the expected value of the dependent variable calculated by the regression equation.

a) Scatter plots of residuals versus the estimated dependent variable and also versus the independent variables should be randomly distributed about zero.

b) The residuals normalized with respect to MSE should plot as a straight line on a normal probability axis. Also the Shapiro-Wilk test for normality (Hahn and Shapiro, 1967) was used.

Density Prediction Models

The variables used to generate a prediction model for density included: water content (w), compactive work (W), and the square and square root of both of these. The combination of all of these in products and quotients were investigated in scattergrams. Also used was the average work ratio (W_R) as previously given in Table 3.6 , and the deviation of water content from optimum.

It was found that although overall prediction models for density both wet and dry-of-optimum were acceptable in most of the statistical criteria, the residuals showed trends that indicated a poor fit to the observed data. This

was due in part to the differences in the effect of compaction energy wet and dry-of-optimum, as earlier described. Hence, separate models were developed for each side of optimum water content.

The procedure described above resulted in the selection of the dry-of-optimum model for dry density as:

$$\hat{\rho}_d = 1338.3 + 1284.0 \sqrt{W_R} / w + 0.32 w^2 \sqrt{W_R} \quad (3.8)$$

where: $\hat{\rho}_d$ = the estimated dry density, kg/m³

W_R = average work ratio

w = water content, %

At any particular average work ratio value the equation describes a curve, as shown in Figure 3.20. However, only the solid line portion of the curve is applicable at $W_R = 1$. This is because the data used to develop the equation covered only the water content range from 20 to 24% at $W_R = 1$. The lower part of the curve describes the density-water content relationship that will be shifted up the density axis by a greater work ratio (W_R) in Equation 3.8. Hence, it is important that this equation not be extrapolated beyond the range for which data were available to

FIGURE 3.20 DRY-OF-OPTIMUM DRY DENSITY MODEL AND THE EFFECT OF EXTRAPOLATION

develop it. The range of this model is shown by the joint region of observations in Figure 3.21. Before a set of independent variables is used in Equation 3.8, it should be plotted in Figure 3.21 to see that it lies in the range of the original data.

The prediction model developed for dry density wet-of-optimum is:

$$\hat{\rho}_d = 961.8 + 15564.6/w \quad (3.9)$$

where:

$$\hat{\rho}_d = \text{predicted dry density, kg/m}^3$$

$$w = \text{water content, \%}$$

This equation essentially describes a line of constant saturation like those seen in the dry density-water content plot in Figure 3.1. By definition, dry density is related to water content and saturation by:

$$\rho_d = \frac{SG\rho_w}{Gw + S} \quad (3.10)$$

where at constant saturation (S), all the variables are constant except water content (w), which is inversely proportional to the density. The limits of application of Equation 3.9 are between the water contents 27.0 and 19.0% because these were the limits of the data.

FIGURE 3.21 JOINT REGION OF OBSERVATIONS FOR DRY-OF-OPTIMUM DRY DENSITY PREDICTION MODEL

The statistical criteria for selection of the density models are presented in Table 3.10. As shown by the $R^2 = 0.99$, nearly all of the variation in ρ_d is explained by the independent variables in the models. The overall F statistic for both models is far in excess of that required. MSE and C_p are about the lowest obtained in any model investigated. Confidence intervals are small and do not cross zero.

The residual plots for the dry-of-optimum model are shown in Figure 3.22. The residuals are distributed fairly randomly although it can be seen there is some tendency to underestimate the density at intermediate energy levels and to overestimate it at high energy levels. This is more easily seen in Figure 3.23 where the regression relationships are plotted with the original data.

The model for wet-of-optimum dry density model also shows some trends in the residuals in Figure 3.24. The density at the Low Energy level is slightly overestimated, and underestimated at the Standard Proctor level. This is also shown in Figure 3.23. However, the error is very small and is not significant; the greatest residual is only 8 kg/m^3 (0.15 pcf).

The plots of normalized residuals showed them to be normally distributed for both models. Both are accepted at the $\alpha = 0.05$ level by the Shapiro-Wilk test for normality.

Table 3.10 Statistical Data for Density Prediction Equations

Statistical Criteria	Dry-of-Optimum		Wet-of-Optimum	
	$\hat{\rho}_d + 1338.3 + 1284.0 \sqrt{w_R} / w + 0.32w^2 \sqrt{w_R}$		$\hat{\rho}_d = 961.8 + 15564.6/w$	
C_P	30.41		36.48	
Coefficient of Multiple Determination, R^2	0.99		0.99	
Adjusted Coefficient of Multiple Determination, R_a^2	0.99		0.99	
Mean Square Error, MSE	118.93		21.32	
Overall F-test				
F	3159.48		9779.02	
F*	3.18		4.35	
95% Confidence Intervals for Regression Coefficients				
$\sqrt{w_R}/w$	1228.67 , 1339.33		-----	
$w^2 \sqrt{w_R}$	0.28 , 0.35		-----	
1/w	-----		15236.23 , 15892.87

FIGURE 3.22

RESIDUAL ANALYSIS FOR DRY DENSITY PREDICTION
MODEL DRY-OF-OPTIMUM

FIGURE 3.23 PREDICTED DENSITY VS. WATER CONTENT

FIGURE 3.24 RESIDUAL ANALYSIS FOR DRY DENSITY PREDICTION MODEL WET-OF-OPTIMUM

Other significant models identified but not selected are given in Appendix E. Usually the other models were rejected in favor of a simpler one, or for a model of somewhat better random distribution of residuals in scatterplots.

Strength Prediction Equations

The variables used in prediction models for strength were: water content, work, dry density, degree of saturation, void ratio and confining pressure, as well as the square and square roots of these.

The nearly saturated wet-of-optimum samples increased little in strength with confining pressure, whereas, for reasons already discussed, the dry-of-optimum samples were strongly affected by confining pressure. This made a good prediction model for strength for both wet and dry-of-optimum unobtainable, as was evidenced in the residual plots for these models. Hence, separate models for wet and dry-of-optimum were developed. The model for strength prediction dry-of-optimum is:

$$\hat{q}_c = -1784.8 + 3.1 \rho_d \sqrt{S_i}/w + 84.0 (1-S_i/100) \sqrt{\sigma_3} \quad (3.11)$$

where: $\hat{q}_c$ = estimated compressive strength, kN/m^2

ρ_d = dry density, kg/m^3

S_i = initial degree of saturation, %

w = water content, %

σ_3 = confining pressure, kN/m^2

Since initial void ratio is related to water content by:

$$S_i e_i = wG \quad (3.12)$$

Equation 3.11 can be written as:

$$\hat{q}_c = -1784.8 + 8.5 \rho_d / e_i \sqrt{S_i} + 84.0 (1 - S_i / 100) \sqrt{\sigma_3} \quad (3.13)$$

This equation incorporates variables that were earlier shown to be very important in the determination of strength. The variable $e_i \sqrt{S_i}$ showed a linear relationship when plotted against $\log(\sigma_1 - \sigma_3)_f / 2$. In this case the term was linearized by the use of the variable ρ_d . Since,

$$\rho_d = \frac{G \rho_w}{1 + e_i} \quad (3.14)$$

the variable $\rho_d / e_i \sqrt{S_i}$ becomes a higher order term that better linearizes the data than the logarithm. Also as noted earlier, the values of e and S at failure could not be accurately determined, so the effect of confining pressure had to be accounted for. The best variable found to do this was the third term in Equation 3.11, $(1 - S_i / 100) \sqrt{\sigma_3}$. As the confining pressure increases, its effect on strength decreases. (This behavior was evident in the $q_f - p_f$ plots of Figures 3.8 to 3.10) The factor $(1 - S_i / 100)$ reduces the influence of the confining pressure as the saturation increases.

The joint region of observations for use with this dry-of-optimum strength prediction model is shown in Figure

3.25. Values of the independent variables of water content and density to be used in Equation 3.11 should lie in this region. In addition, the range of confining pressures applicable is 0 to 414 kN/m^2 (0 to 60 psi).

The predicted compressive strength versus dry density at constant water content and confining pressure is plotted in Figure 3.26. This figure shows that the model follows previously discussed behavior. As density increases, and/or dry side water content decreases, so does the strength. However, as confining pressure increases, the volume changes increase so that initial density has less effect on strength. The trend of the curves at zero confining pressure is very similar to the data of Seed and Monismith (1954) shown in Figure 1.2(b).

The prediction equation found for compressive strength of samples wet-of-optimum is:

$$\log(\hat{q}_c) = 1.70/e_i \quad (3.15)$$

where; $\hat{q}_c$ = the estimated compressive strength, kN/m^2

e_i = initial void ratio

(The regression for this equation showed the constant, b_o , to be insignificant. Therefore another regression was run which set $b_o = 0$, giving the resulting Equation 3.15.) The relation in Equation 3.15 is well established in the literature, i.e., as the void ratio decreases the strength increases and is a linear relationship when e at failure is plotted versus the logarithm of the compressive strength.

FIGURE 3.25 JOINT REGION OF OBSERVATIONS FOR DRY-OF-OPTIMUM STRENGTH PREDICTION MODEL

FIGURE 3.26 PREDICTED COMPRESSIVE STRENGTH-DRY DENSITY RELATIONSHIP AT CONSTANT WATER CONTENT DRY-OF-OPTIMUM

The statistical criteria concerning these equations for strength are given in Table 3.11. Both models are good in the statistical criteria. R^2 is high and C_p and MSE are low, although comparison of MSE of the wet-of-optimum model would be meaningful only with respect to other models incorporating logarithms. The magnitudes of the logarithms are of course much smaller than the arithmetic numbers, so MSE will be much lower. The F statistic is far in excess of that required. Confidence intervals for all coefficients in both models are small and do not cross zero.

The plots of residuals for the dry-of-optimum strength model are given in Figure 3.27. These plots show random distribution of residuals with only a slight trend toward under-estimation for samples of low strength.

The residuals for the wet-of-optimum strength model are also randomly distributed as shown in Figure 3.28. A small trend is evident in residuals at high strengths (Modified Proctor energy level), that may slightly underestimate strength at higher void ratios and overestimate strength at lower void ratios at this energy level.

The residuals of both models showed normal distribution on a normal probability plot and were accepted as such at the $\alpha = 0.05$ level by the Shapiro-Wilk normality test.

Again, some other significant models for strength are given in Appendix E.

Table 3.11 Statistical Data for Strength Prediction Equations

Statistical Criteria	Dry-of-Optimum	Wet-of-Optimum
	$\hat{q}_c = -1784.8 + 3.1 \rho_d \sqrt{S_i/w} + 84.0 (1-S_i/100) \sqrt{\sigma_3}$	
		$\log(\hat{q}_c) = 1.70/e_i$
C_p	24.54	----
Coefficient of Multiple Determination, R^2	0.98	0.99
Adjusted Coefficient of Multiple Determination, R_a^2	0.98	0.99
Mean Square Error, MSE	8301.52	0.0069
Overall F-test		
F	1283.47	18904.56
F*	3.18	4.35
95% Confidence Intervals for Regression Coefficients		
$\rho_d S_i/w$	2.91 , 3.18	----
$(1-S_i/100) \sqrt{\sigma_3}$	72.10 , 95.88	----
$1/e_i$	----	1.67 , 1.72

FIGURE 3.27 RESIDUAL ANALYSIS FOR STRENGTH PREDICTION MODEL
DRY-OF-OPTIMUM

FIGURE 3.28

RESIDUAL ANALYSIS FOR STRENGTH PREDICTION MODEL
WET-OF-OPTIMUM

Variability Analysis

The variability analysis was performed as suggested by Kuczek (1979), by plotting the mean value of the response variable at each level versus the variance or standard deviation of the response variable at that same level.

The plot of the mean value of observed densities at each energy-water content level against the variance is shown in Figure 3.29. There is a definite trend toward reduction of variance as density increases. This is to be expected since the less dense samples have much more pore space, are at higher water content than those at high energy levels, and therefore are much more compressible, as seen in the volumetric strains (Figure 3.16). Hence, the density obtained is much more sensitive to variations in energy and other factors in the compaction process. The variance does appear to drop very quickly in the low range of density (1500 to 1600 kg/m³). However, there is not enough data in Figure 3.29 to develop a predictive relationship.

The plot of the mean of the compressive strengths at each energy-water content-confining pressure level (for example, C0-L1) versus the standard deviation of the strength is shown in Figure 3.30. The trend of the data shows that as the magnitude of the strength increases, so does the magnitude of the variation. This indicates there was a constant amount of random error in the strengths measured. This constant was numerically defined by an SPSS REGRESSION

FIGURE 3.29 PLOT FOR VARIABILITY ANALYSIS OF DRY DENSITY

FIGURE 3.30 PLOT FOR VARIABILITY ANALYSIS OF COMPRESSIVE STRENGTH

run on the data. The regression showed that the 95% confidence interval for the constant, b_0 , crossed zero. Therefore another regression program was run that forced the equation through the origin. This equation is:

$$s(\hat{q}_c) = 0.063 q_c \quad (3.16)$$

where: $s(\hat{q}_c)$ = estimated standard deviation of compressive strength, kN/m^2

q_c = compressive strength, kN/m^2

The equation fits with an $R^2 = 0.80$ and the overall $F = 126.73$. This is much greater than the action limit ($F^* = 4.16$, at $\alpha = 0.05$) to reject the equation as not significant. The confidence interval for the coefficient, 0.051 to 0.074, is small and does not cross zero.

According to statistical theory, 95% of the observations will fall within two standard deviations of the mean. Therefore, with a 95% level of confidence, the maximum variation in strength $V(\hat{q}_c)$, is plus or minus twice the standard deviation, or:

$$V(\hat{q}_c) = 2s(\hat{q}_c) = 0.126q_c \quad (3.17)$$

The minimum expected strength, $\hat{q}_{c_m}$, will then be the expected mean strength minus the expected variation:

$$\hat{q}_{c_m} = q_c - V(\hat{q}_c) = q_c - 0.126\hat{q}_c \quad (3.18)$$

Nomographs and Charts for Solution of Prediction Equations

Nomographs provide a convenient solution to equations without the necessity of a slide rule or calculator; all that is needed is a straight edge. Hence, the prediction equations developed in the previous section can be quickly solved for a particular set of compaction variables by the nomographs presented here. Charts are presented for solution of the simpler prediction equations.

The nomograph for prediction of dry density, dry-of-optimum is given in Figure 3.31. To use the nomograph enter the figure with a water content and work ratio. On the water content line, w , are two separate scales, w_{X_1} on the left, w_{X_2} on the right. A straight line is drawn from the water content on scale w_{X_1} to the value of work ratio on scale W_R , and another line is drawn from the water content on scale w_{X_2} to the same point on the W_R scale. These lines intersect the scales X_1 and X_2 , respectively. These values of X_1 and X_2 are entered in scales $X_1(b)$ and $X_2(b)$ at the right of the figure and a straight line drawn between them. This line will intersect scale $\hat{\rho}_d$ at the predicted value of dry density, dry-of-optimum.

As an example in the use of this nomograph, it is desired to predict the dry density when the work ratio is 2.0 and the water content is 20.0%. A line (shown dotted on Figure 3.31) drawn from 20.0 on the w_{X_1} scale to 2.0 on

FIGURE 3.31 NOMOGRAPH FOR DRY DENSITY PREDICTION DRY-OF-OPTIMUM

1. See Figure 3.21 for acceptable values of w_R and w .
2. w_R for kneading compaction.

the W_R scale intersects scale X_1 at 91, and a line drawn from 20.0 on the w_{X_2} scale to 2.0 on the W_R scale intersects the X_2 scale at 178. Entering the scales on the right of the figure, a line drawn between 91 on scale $X_1(b)$, and 178 on scale $X_2(b)$, intersects the $\hat{\rho}_d$ scale at 1507 kg/m^3 , the predicted dry density.

The solution of the prediction equation for dry density wet-of-optimum is given by the chart in Figure 3.32. An example of the relationship is shown dotted in the figure, i.e., at a water content of 22.0% the predicted density is 1670 kg/m^3 .

The nomograph that can be used to find the predicted compressive strength dry-of-optimum is presented in Figure 3.33. There are two sets of scales for water content, w , and dry density, ρ_d , in the figure. A line is drawn between the dry density and water content for each pair of these scales. The line drawn between the ρ_d and w scales on the left side of the figure will intersect a value of S on the left hand side of the scale S . This value of S is entered on the right hand side of the scale, and a line is drawn between this point and the desired confining pressure on scale, σ_3 . This line will intersect a value of X_2 on the left hand side of scale X_2 .

The line drawn between the ρ_d and w scales on the right side of Figure 3.33 intersects a value of X_1 on the right hand side of scale X_1 . This value is entered on the

FIGURE 3.32 CHART FOR PREDICTION OF DRY DENSITY WET-OF-OPTIMUM

FIGURE 3.33 NOMOGRAPH FOR UNCONSOLIDATED-UNDRAINED STRENGTH PREDICTION FOR MOLDING WATER CONTENTS DRY-OF-OPTIMUM

See Figure 3.25 for acceptable values of ρ_d and w .

left hand side of scale X_1 and a line is drawn between this point and the value of X_2 entered on the right hand side of scale X_2 . This final line intersects scale $\hat{q}_C$ at the predicted value of the compressive strength for a sample dry-of-optimum. An example can be followed by the dotted lines in Figure 3.33. The desired values of the independent variables are, $\rho_d = 1650 \text{ kg/m}^3$, $w = 21.0\%$, and $\sigma_3 = 100.0 \text{ kN/m}^2$. A line drawn between these values on scale ρ_d and w on the left side of Figure 3.33 intersects scale S on the left hand side at 84. The right hand side of scale S is entered with 84, and a line is drawn between it and 100 on the σ_3 scale, intersecting the left hand side of scale X_2 at 130.

The same values of water content and density ($\rho_d = 1650$, $w = 21.0$) are used in the w and ρ_d scales on the right side of Figure 3.33. A line drawn between the scales at these values intersects scale X_1 on the right hand side at 2200. A line drawn between 2200 on the left hand side of scale X_1 and 130 on the right hand side of scale of X_2 , intersects scale $\hat{q}_C$ at 550 kN/m^2 , the predicted compressive strength, dry-of-optimum.

Note that if confining pressure is zero, all operations on the scales of the left side of Figure 3.33 can be ignored, and a line is simply drawn from 0 on the right hand side of scale X_2 to the necessary value on scale X_1 to find the compressive strength.

The chart in Figure 3.34 can be used to find the predicted compressive strength wet-of-optimum. Equation 3.14 was substituted into Equation 3.15 to put it into terms of a compaction variable. Hence, Figure 3.34 can be entered on the abscissa with the desired dry density and the predicted compressive strength read off the ordinate, e.g., at a density of 1650 kg/m^3 , the predicted compressive strength would be about 290 kN/m^2 .

Figure 3.35 can be used to determine the estimated variability of the compressive strength. The dashed lines provide an example in its use, i.e., at a compressive strength of 1200 kN/m^2 , the predicted variability is about 150 kN/m^2 .

These nomographs and charts could aid in the design of an embankment. For a particular water content and energy level, the density that could be obtained in compaction may be predicted. Then with the use of this predicted density, or an observed value, the strength for the soil compacted at these conditions could be predicted and the embankment designed accordingly or the as-compacted stability could be predicted.

However, the limitations to the use of laboratory compacted samples to predict field compacted conditions should be recognized. Work is currently underway to correlate the results of laboratory compacted samples to those compacted in the field.

FIGURE 3.34 CHART FOR PREDICTION OF UNCONSOLIDATED - UNDRAINED STRENGTH FOR MOLDING WATER CONTENTS WET-OF-OPTIMUM.

FIGURE 3.35 CHART FOR PREDICTION OF COMPRESSIVE STRENGTH VARIABILITY

Comparison With Statistical Regression

Results for a Silty Clay

With the data from unconfined compression tests on a laboratory compacted glacial silty clay, Essigmann (1976) performed regression analysis with the resulting equations as were shown in Table 1.2. The variables in his dry density models were similar to those developed in this study. Essigmann's model for dry density dry-of-optimum included energy ratio and water content. His energy ratios were of nominal impact energy, whereas in this study the ratio used was of the compactive work done on the soil. As previously noted, there was little difference between nominal energy ratios and work ratios (Table 3.7).

The comparison of Essigmann's predicted density curves and those of this study is shown in Figure 3.36. The prediction curves for the silty clay dry-of-optimum are considerably steeper than the dry-of-optimum prediction curves for the St. Croix clay and are at much higher densities. This is typical in the comparison of silty soils to clayey ones.

The wet-of-optimum density model for the silty clay included only water content as an independent variable as did the model for the St. Croix clay. However, the model developed in this study used the reciprocal of water content. Therefore, the coefficient is larger and positive. The slopes of the two wet-of-optimum density models are

FIGURE 3.36 COMPARISON OF DENSITY PREDICTION MODELS FOR SILTY CLAY AND ST. CROIX CLAY

quite similar, except the curve for the St. Croix clay steepens as density increases. This is because the data used to develop the equation was at a constant saturation, where Essigmann's wet-of-optimum data were at more variable saturations.

The unconfined compressive strength models reported for the silty clay included variables of dry density and water content, as did the equation for the dry-of-optimum strength in this report, although degree of saturation was also used. The equation for St. Croix clay also included a term for confining pressure. For prediction of unconfined strength, $\sigma_3 = 0$ and Equation 3.11 becomes,

$$\hat{q}_c = -1784.8 + 3.1 \rho_d \sqrt{S_i}/w \quad (3.19)$$

For comparison, curves developed from Equation 3.19 and from Essigmann's equation for the unconfined compressive strength dry-of-optimum for silty clay are plotted in Figure 3.37. The curves for the silty clay have considerably smaller slope than those for the St. Croix clay. Also note that for the St. Croix clay, the water content is higher and the dry density is lower than for the silty clay at equal compressive strength. This is a result of the larger quantity of clay minerals in the St. Croix clay.

The comparison of compressive strength for wet-of-optimum samples for the two soils is shown in Figure 3.38. The upper part of the St. Croix curve again shows a greater slope than that for the silty clay, but also shows a less

FIGURE 3.37 COMPARISON OF DRY-OF-OPTIMUM UNCONFINED COMPRESSIVE STRENGTH PREDICTION MODELS FOR SILTY CLAY AND ST. CROIX CLAY

FIGURE 3.38 COMPARISON OF WET-OF-OPTIMUM COMPRESSIVE STRENGTH PREDICTION MODELS FOR SILTY CLAY AND ST. CROIX CLAY

linear relationship, as the slope of the curve decreases considerably toward its lower portion.

The equation Essigmann reported for strength variability, given in Table 1.2, is a function of the strength magnitude and the dry density variation. The equation found for the St. Croix clay is only a function of the strength magnitude.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

As a result of the unconsolidated-undrained triaxial tests performed on samples of laboratory compacted, highly plastic St. Croix clay, the following conclusions are drawn.

- 1) To fit kneading compaction results to compaction curves of constant Proctor (impact) energy required less nominal energy in the kneading mode as water content increased. This indicated that the kneading method is more efficient than the impact method. The results of the work measurements in kneading compaction showed the same trend as the nominal kneading values, although exceptions to this trend were caused by variations in the size and strength of the soil aggregates.

When the measured values of work were compared between their fitted Proctor energy levels at constant saturation, the result was constant work ratios. These work ratios are of the same magnitude as the ratios of nominal impact energy.

- 2) The behavior of St. Croix clay in unconsolidated-undrained triaxial tests is like that well established in the literature (Rutledge (1947),

Casagrande and Hirschfeld (1960, 1962), Conlin (1972)). The values of strength, determined at maximum stress difference or 20% axial strain, decreased with water content, increased with density and increased with confining pressure until the sample reached near-saturation.

- 3) The volume changes occurring during shear are significant, especially for samples of low water content and density. On the dry side of optimum, volumetric strains during shear increased with confining pressure and decreased as water content increased. Volumetric strains during shear for samples wet-of-optimum were independent of confining pressure.
- 4) The prediction equations found for dry density, strength and variability for the range of variables investigated are summarized below.

a) Dry density, dry-of-optimum:

$$\hat{\rho}_d = 1338.3 + 1284.0 \sqrt{w_R}/w + 0.32 w^2 \sqrt{w_R}$$

(See Figure 3.21 and Table 3.10). (3.8)

b) Dry density, wet-of-optimum:

$$\hat{\rho}_d = 961.8 + 15564.6/w$$

(See Table 3.10). (3.9)

c) Compressive strength, dry-of-optimum:

$$\hat{q}_c = -1784.8 + 3.1 \rho_d \sqrt{S_i}/w + 84.0(1-S_i/100)\sqrt{\sigma_3}$$

(3.11)

(See Figure 3.25 and Table 3.11).

d) Compressive strength, wet-of-optimum:

$$\log(\hat{q}_c) = 1.70/e_i \quad (3.15)$$

or,

$$\log(\hat{q}_c) = 1.70 \rho_d / (G\rho_w - \rho_d) \quad (3.20)$$

(See Table 3.11).

e) Variability of compressive strength:

$$V(\hat{q}_c) = 0.126 q_c \quad (3.17)$$

(See page 137).

where: $\hat{\rho}_d$ = estimated dry density, kg/m^3
 $\hat{q}_c$ = estimated compressive strength,
 kN/m^2

$V(\hat{q}_c)$ = variability of compressive strength,
 kN/m^2

W_R = average work ratio

w = water content, %

S_i = initial degree of saturation, %

ρ_d = dry density, kg/m^3

σ_3 = confining pressure, kN/m^2

e_i = initial void ratio

ρ_w = mass density of water

G = specific gravity of solids

5) All models were shown to be statistically valid.

The trends of the experimental data are well modeled by the density equations. A plot of the predicted compressive strength (dry-of-optimum) versus dry density at constant water content, is very similar to that shown in the literature (Seed

and Monismith, (1954)). The wet-of-optimum strength model is also a well established (in principle) relationship in the literature (Leonards, (1955)). Solution to all equations are presented in nomographic or chart form.

- 6) The prediction equations developed herein were compared to those of an earlier study on silty clay. The equations differ in a logical manner, considering the differences in soil type.

Recommendations

- 1) If sample volume changes in the triaxial test are to be measured by the volume of the displaced cell water, improvements may be needed in the triaxial cell to decrease the expansion and seating effects, e.g., the use of stiffer cell wall material.
- 2) The compaction conditions in the field are very different from those of the laboratory. Therefore, if the results of this study are to be of the most value for prediction of field behavior, these results should be correlated with similar tests on field compacted soil.
- 3) Improvements to the method of compactive work measurement used in this study are needed. More sophisticated sensors and transducers, and electronic integration techniques would result in a simpler, less time consuming and more accurate determination

of work. Also, the measurement of residual changes in soil elevation over the entire compaction surface in the mold may allow determination of the true work that produces residual densification. Such measurements might involve electronic distance measuring techniques.

- 4) For a complete concept of the work input by kneading compaction, a wider range of conditions need to be examined, such as:
 - a) the compactive work when a constant foot pressure is used at varying water contents.
 - b) the compactive work when varying foot pressures are used at a constant water content.
- 5) DiBernardo (1979) found that the ratio of compactive prestress to nominal foot pressure was always less than one. This occurs because the nominal value does not account for losses in the compactive process which are not important to the development of the prestress. A measure that excludes these losses, such as compactive work or compactive force applied to the soil fabric, may show essentially a 1 to 1 relationship. Such comparisons should be undertaken.
- 6) Possibly instrumenting the impact hammer with accelerometers would show the actual force being developed on the soil. This would allow determination of the distribution of energy during impact

compaction and would result in a much better understanding of that process.

Implementation of recommendations 3, 4, 5 and 6 would produce a superior definition of how nominal compaction energy is translated into useful work and losses. Such understanding could lead to the development of more efficient field compaction methods and equipment.

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APPENDICES

Appendix A

Selection and Assembly of Volume

Change Apparatus

Selection of Volume Change Apparatus

There are generally three basic techniques to measure volume changes of a partially saturated sample in the triaxial test:

- (1) Indirect sample measurement (no contact between measuring instrument and sample)
- (2) Direct sample measurement (measuring instrument has contact with sample)
- (3) Measurement of confining fluid displaced due to volume change.

The most common indirect techniques involve "optical tooling" or photography. The optical methods make use of an optical instrument to view through the cell. Dudley (1971) used a theodolite to make measurements of the change in sample dimensions. Bishop and Donald (1961) used a cathetometer to record the movement of a float sitting on a volume of mercury that surrounded the sample. Dudley also used photographs from a camera located outside the cell. Measurements were made on the prints. Marsal and Resines (1960) also used photographs in the same way, except their film was located inside the cell.

Photographs have the advantage of being a permanent record of the sample. Optical methods have the complementary advantage of instantaneous results. However, unless several set-ups are used, the sample is only viewed from one direction with the theodolite and photographic methods.

There are a number of methods that have been used in direct measurement of the sample. Movement of "feelers" that touched the sample and actuated a mercury diaphragm was used by Bishop and Henkel (1957). Ahmed (1971) also used feeler movement that was read out by a LVDT to a recorder. Holubec and Finn (1969) wrapped a piece of strain gaged beryllium copper around the sample to measure lateral strain.

The disadvantage of the above direct measurement methods is that they provide lateral restraint to the sample. This problem was overcome by Altschseffl and Mishu (1970), by using a metal ring around, but not touching the sample. The lateral strain was indicated by the change in capacitance between the ring and the sample.

The major problems of the electrical transducer types discussed above are those of wiring and calibration. Also, all of these direct measurement techniques measure the lateral strain at only one cross-section.

Of the methods measuring flow to or from a cell with sample volume change, the burette is the simplest. To attain much accuracy, the inside diameter of the burette must be quite small. However, if the diameter is very small, small changes in volume may result in large changes in head, changing the applied confining pressure. This change in head problem can be alleviated by use of a compensating mercury pot [Bishop and Henkel (1957)]. Bishop and Donald (1961) later eliminated the use of the compensating pot by using a displacing fluid with a specific gravity close to one, i.e., paraffin ($G = 0.80$).

A small bore tube in conjunction with an air-water interface inside the triaxial cell was used by O'Connor and Mitchell (1978). The air-water interface in the cell was made quite small by gluing a lucite spacer in the top of the cell. This left a small annulus between the spacer and the piston. The tube was tilted slightly from horizontal so that a small change in height of the interface in the annulus resulted in a large change in length in the tilted tube. This allowed better reading precision.

Chan and Duncan (1967) also made use of a small inside diameter tube that, except for a length of kerosene, had water continuous throughout its length. The tube's outlet was in a large diameter reservoir so that head changes were negligible. The length of kerosene provided a reference for reading the volume change. When the kerosene reached the end of the tube, the flow could be reversed by switching one valve. A diagram of this apparatus is shown in Figure A1.

Lewin (1971) made use of a null indicator to measure volume. Servo-devices automatically controlled the null indicator which in turn actuated a piston connected to a dial gage or LVDT. Flow of water also provided movement for: a brass float hooked to a Bellofram [Menzies (1975)], a glass float [Mitchell and Burns (1971)], and self compensating mercury pots [Rowlands (1972) and Darley (1973)]; all were connected to LVDT's for automatic readout.

FIGURE A1 DIAGRAM OF DEVICE FOR VOLUME CHANGE MEASUREMENT
(from Duncan and Chan, 1967)

Another method developed by Mitchell and Burns used a transducer to weigh the fluid that was displaced by the change in volume.

The advantage of volume change type meters is their instant and direct readout of volume change for the entire sample. All are simple to operate and relatively cheap, especially those without automatic readout capability. Precision is also very good. Calibration is easy and constant. Problems with the volume measurement technique include: possibility of trapped air under membranes and in the cell, and required calibrations for temperature, creep and cell expansion. However, as seen by calibration Figures A2, A3, and A4 for the apparatus used in this study, these can be quite well determined. The problems and corrections can be reduced by:

- (a) careful placement of sample membranes;
- (b) cell top machined such that water easily displaces all air from the cell as it is filled;
- (c) stiff cell wall material that would have little expansion and creep.

Also very important in volume measuring techniques is a triaxial cell that will not leak, especially around the piston.

The volumeter of Chan and Duncan (1967), as seen in Figure A1, was selected for use in this study. It was chosen over the many other types because of its simpler

FIGURE A2 CALIBRATION OF CELL EXPANSION DUE TO APPLICATION OF CONFINING PRESSURE

FIGURE A3 CALIBRATION OF TRIAXIAL CELL EXPANSION UNDER CREEP

FIGURE A4 CALIBRATION FOR VOLUME CHANGE DUE TO TEMPERATURE CHANGE

operation and readout, low cost, accuracy, and the unavailability of automated techniques.

Assembly of the Volume Change Apparatus

Problems were encountered in the assembly of the device developed by Chan and Duncan (1967). Suggestions by Anderson (1978) helped get the apparatus in working condition. Several brands of nylon and acrylic tubing were tried by the writer, without success. Problems with the kerosene (and also with the oil that was later used) sticking to the tube were encountered. It is uncertain whether surface properties of the various tubes were incompatible with the kerosene (and oil), or if the inside of the tubes were damaged or incapable of being properly cleaned. The tubing Anderson (1978) used was Imperial-Eastman, Nyla-Seal, 44-SN-1/4. It was used with success by the writer.

The tubing must be very clean. A wad of clean lintless cloth soaked with detergent water was forced back and forth through the tube to clean it. The cloth should be pushed with a nylon tube or other material that will not scratch the inside of the tube. Wire or other abrasive material may damage the tube surface which may later be the seat of problems, i.e., the sticking of the oil.

The oil used was 100 Red Unity Oil made by Meriam Instrument. It was a colored oil with a specific gravity of 1.00. The use of kerosene was unsatisfactory.

After scrubbing the inside of the tube with detergent it should be thoroughly rinsed, along with the use of a clean wad of cloth. Then a similar scrubbing is done with acetone, after which the tube was allowed to dry thoroughly. This was facilitated by a slow flow of clean air through the tube.

Next, the tube was coated with a silicon oil called Dessicote, from Beckman Instruments. The manufacturer instructed that the best results occur if the surface to be coated is not completely dessicated. Therefore, after drying the tube, it was allowed to remain at ambient room temperature and humidity for several days before being coated with the silicon oil. An even coating of silicon oil is best obtained if the tube is completely filled with the Dessicote. The excess is allowed to freely drain back out. The coated tube was allowed to dry several days before installation into the apparatus.

The rest of the procedure was as described by Chan and Duncan (1967) except that only a small length, about 3 cm, of oil was used. Care should be taken to insert a clean tube to inject the oil. If any oil runs down the injecting tube and this oil does not merge with the main body of oil, this part of the tube should be avoided for use. This small amount of oil that is separated may later cause the rest of the oil to flow non-uniformly and split it up.

Also the syringe used to inject the oil may suck a small bubble of air into the end of the injecting tube. A small amount of oil should be ejected with the top of the injecting tube below water, to be sure no air is in the tip. Release of the syringe plunger may suck a small amount of water back into the injecting tube; this is acceptable whereas air entry was not. Without removing the injecting tube from the water, it can be pushed up into the volume change apparatus filling tap to inject the oil.

Finally, the water cannot enter the volume change tube too fast. If this happens, the oil cannot keep up with the water and it will stick and separate, with no possibility of recollecting it all.

Appendix B

Additional Load - Displacement

Output From Work Measurements

FIGURE B1 RECORDER OUTPUT FOR WORK MEASUREMENT AT AN SI LEVEL

FIGURE B2 RECORDER OUTPUT FOR WORK MEASUREMENT AT AN MI LEVEL

Appendix C

Stress - Strain Curves

FIGURE C1 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-LI-1, C1-LI-1, C2-LI-1, C3-LI-1

FIGURE C2 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-L1-2, C2-L1-2, C3-L1-2

FIGURE C3 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-L2-1, C1-L2-1, C2-L2-1, C3-L2-1

FIGURE C4 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-L2-2, C1-L2-2

FIGURE C5 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-L3-1, C1-L3-1, C2-L3-1, C3-L3-1

FIGURE C6 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-L3-2, C1-L3-2, C3-L3-2

FIGURE C7 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-L4-1, C1-L4-1, C2-L4-1, C3-L4-1

FIGURE C8 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-L4-2, C1-L4-2, C2-L4-2, C3-L4-2

FIGURE C9 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS CO-SI-1, C1-SI-1, C2-SI-1, C3-SI-1

FIGURE C10 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS CO-SI-2, CI-SI-2

FIGURE CII AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-S2-1, C1-S2-1, C2-S2-1, C3-S2-1

FIGURE C 12 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-S2-2, C2-S2-2

FIGURE C13 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-S3-1, C1-S3-1, C2-S3-1, C3-S3-1

FIGURE C14 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS CO-S3-2, C2-S3-2, C3-S3-2

FIGURE C15 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-S4-1, C1-S4-1, C2-S4-1, C3-S4-1

FIGURE C16 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-S4-2, C1-S4-2, C2-S4-2, C3-S4-2

FIGURE C17 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS CO-MI-1, CI-MI-1, C2-MI-1, C3-MI-1

FIGURE C 18 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C1-M1-2, C2-M1-2

FIGURE C19 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS CO-M2-1, C1-M2-1, C2-M2-1, C3-M2-1

FIGURE C20 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-M2-2, C2-M2-2, C3-M2-2

FIGURE C2I AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-M3-1, C1-M3-1, C2-M3-1, C3-M3-1

FIGURE C22 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C0-M3-2, C1-M3-2, C2-M3-2

FIGURE C23 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN FOR TESTS C1-M4-1, C2-M4-1, C3-M4-1

FIGURE C 24 AXIAL STRESS AND VOLUMETRIC STRAIN VS. AXIAL STRAIN
FOR TESTS C1-M4-2, C2-M4-2

Appendix D

Volume Change Data

Inaccuracy of Volume Change Measurements

Problems were encountered in the measurement of volume changes during the application of confining pressure. These were mainly due to:

1) Seating of membrane. This was especially a problem for samples dry-of-optimum where sample sides were not smooth. The void spaces on the sample surface allowed the membrane to penetrate when pressure was applied and hence these volume changes were measured as sample volume changes.

2) Triaxial cell seating effects. As was previously pointed out in Chapter 2, there was difficulty in calibration of cell expansion under the first 69 kN/m^2 (10 psi) of pressure applied. This non-constancy problem showed up in the volume changes.

3) Possible unseen trapped air bubbles. In some tests the water dropped very quickly under small increases in pressure, much more than was normally observed.

The result of these problems is evident in the volume change data in Table D1. It can be seen that the change in volume due to application of confining pressure, ΔV_c , is, in some tests, greater than the volume of air in the sample, V_a . This is of course not possible since the water and soil are essentially incompressible. In other cases it is not evident that the ΔV_c is overestimated until it is summed with the changes in volume that occurred during shear, ΔV_{sh} , in which case the total volume change, ΔV_T , is again greater than the volume of air.

In addition, for reasons not completely understood, the waxing technique to find final saturations, S_w , gave poor results as also evident in Table D1. In some cases the problem may have been due to incomplete submergence of the basket that held the waxed sample below water.

Table D1 Volume Change Data

Symbols in Table D1

V_o = Original sample volume prior to test, cm^3

V_{v_o} = Original sample volume of voids prior to test, cm^3

V_a = Original sample volume of air prior to test, cm^3

ΔV_c = Change in volume occurring during application of confining pressure, cm^3

ΔV_{sh} = Change in volume occurring during shear up to failure, cm^3

ΔV_T = Total change in volume at failure, cm^3

S_w = Saturation after triaxial test determined by waxing method, %

Test Number	V_o	V_{v_o}	V_a	ΔV_c	ΔV_{sh}	ΔV_T	S_w
C0-L1-1	65.83	28.98	7.82	0.00	0.25	0.25	74.51
C0-L1-2	69.76	31.18	8.32	0.00	0.27	0.27	77.09
C1-L1-1	69.55	31.95	10.23	2.36	3.67	6.03	80.86
C2-L1-1	69.04	31.02	8.95	4.10	3.84	7.94	79.37
C2-L1-2	68.61	31.72	9.86	5.04	4.15	9.19	86.56
C3-L1-1	69.40	31.68	10.22	5.34	4.24	9.58	81.73
C3-L1-2	69.30	32.90	11.62	6.66	4.11	10.77	91.75
C0-L2-1	68.61	30.43	7.06	0.00	0.33	0.33	79.77
C0-L2-2	69.15	30.94	6.79	0.00	0.16	0.16	82.45
C1-L2-1	69.46	30.57	6.72	2.88	3.06	5.94	93.65
C1-L2-2	68.65	29.21	5.17	2.37	2.58	4.95	94.23
C2-L2-1	68.15	29.64	5.51	4.11	3.07	7.18	103.10
C3-L2-1	69.09	29.99	5.99	5.56	2.50	8.06	87.86
C0-L3-1	69.69	30.11	3.91	0.00	0.79	0.79	86.46
C0-L3-2	66.61	28.45	2.99	0.00	0.35	0.35	96.49
C1-L3-1	70.23	30.34	3.81	2.51	1.31	3.82	99.68
C1-L3-2	69.48	29.98	3.80	2.85	2.24	5.09	98.53
C2-L3-1	68.73	29.61	3.42	3.82	1.62	5.44	103.04
C3-L3-1	70.05	30.50	4.23	2.61	1.12	3.73	95.50
C3-L3-2	69.53	30.14	4.30	3.84	1.05	4.89	88.14

Table D1. (continued)

Test Number	V_o	V_{v_o}	V_a	ΔV_c	ΔV_{sh}	ΔV_T	S_w
C0-L4-1	69.24	30.56	1.85	0.00	0.61	0.61	---
C0-L4-2	68.81	30.58	2.03	0.00	0.45	0.45	93.50
C1-L4-1	68.97	30.49	2.14	3.01	0.73	3.74	94.15
C1-L4-2	67.97	30.17	2.09	2.65	0.33	2.98	101.51
C2-L4-1	69.02	30.38	1.53	5.35	0.61	5.96	101.74
C2-L4-2	69.55	30.84	2.03	3.55	0.51	4.06	100.05
C3-L4-1	66.74	29.97	2.33	3.14	0.44	3.58	93.57
C3-L4-2	69.06	30.85	2.19	3.84	0.54	4.38	93.51
C0-S1-1	69.01	29.21	7.84	0.00	0.29	0.29	70.04
C0-S1-2	69.43	30.72	10.13	0.00	-0.07	-0.07	70.93
C1-S1-1	68.83	30.66	10.51	2.30	2.13	4.48	69.70
C1-S1-2	66.94	28.76	8.93	4.31	2.49	6.80	75.42
C2-S1-1	70.73	30.11	7.81	4.50	1.88	6.38	---
C3-S1-1	67.65	29.07	8.13	4.20	3.18	7.38	---
C0-S2-1	71.32	29.97	6.85	0.00	0.36	0.36	---
C0-S2-2	65.27	26.65	5.25	0.00	0.47	0.47	82.43
C1-S2-1	70.68	29.49	6.42	3.65	1.78	5.43	---
C2-S2-1	67.73	29.15	7.70	4.20	3.24	7.44	88.08
C2-S2-2	68.55	28.29	5.78	2.01	2.42	4.43	90.64
C3-S2-1	64.73	27.61	6.79	3.36	3.45	6.81	86.43
C0-S3-1	70.14	28.57	3.10	0.00	0.38	0.38	---
C0-S3-2	68.09	28.26	4.29	0.00	0.38	0.38	82.68
C1-S3-1	70.33	28.87	3.89	2.22	1.08	3.30	92.15
C2-S3-1	70.61	29.84	4.95	5.18	1.63	6.81	---
C2-S3-2	65.99	27.62	4.72	3.15	2.28	5.43	91.17
C3-S3-1	69.37	28.33	3.83	2.91	1.49	4.64	95.30
C3-S3-2	66.07	27.44	4.49	4.96	3.49	6.64	95.02
C0-S4-1	69.89	29.83	2.15	0.00	0.58	0.58	95.74
C0-S4-2	69.38	29.69	2.48	0.00	0.70	0.70	98.52
C1-S4-1	70.55	30.12	2.03	4.50	0.69	5.19	---
C1-S4-2	67.71	29.27	2.35	2.66	0.70	3.36	88.83
C2-S4-1	69.79	29.77	2.10	3.70	0.65	4.35	99.18
C2-S4-2	68.89	29.22	1.93	2.95	0.46	3.41	98.29
C3-S4-1	70.82	30.17	2.28	3.92	0.54	4.46	---
C3-S4-2	68.47	29.07	1.61	2.96	0.31	3.27	100.83

Table D1 (continued)

Test Number	V_o	V_{V_o}	V_a	ΔV_c	ΔV_{sh}	ΔV_T	S_w
C1-M1-1	67.78	23.15	5.97	1.75	0.47	2.22	72.08
C1-M1-2	69.79	25.60	8.77	4.07	0.55	4.62	67.02
C2-M1-1	64.47	23.26	6.43	3.31	0.40	3.71	70.49
C2-M1-2	68.48	24.85	7.97	2.32	0.49	2.81	68.49
C3-M1-1	68.78	24.85	8.11	2.44	0.76	3.20	77.03
C0-M2-1	70.45	24.40	5.23	0.00	0.28	0.28	---
C0-M2-2	70.86	25.03	5.78	0.00	0.45	0.45	80.50
C1-M2-1	69.42	25.15	6.52	1.44	0.39	1.83	73.66
C2-M2-1	71.05	25.43	6.27	2.84	0.39	3.23	72.78
C2-M2-3	70.09	24.46	5.54	4.35	0.34	4.69	73.32
C3-M2-1	68.17	24.23	6.11	3.55	0.63	4.18	79.13
C3-M2-2	72.27	25.97	6.60	1.09	0.81	1.91	80.71
C0-M3-1	65.88	23.52	3.95	0.00	0.57	0.57	---
C0-M3-2	65.81	23.29	3.90	0.00	0.45	0.45	84.06
C1-M3-1	66.74	23.78	3.97	2.05	0.42	2.47	87.27
C1-M3-2	68.87	24.43	4.42	1.59	0.31	1.90	79.17
C2-M3-1	63.68	22.38	3.93	6.71	0.34	7.05	77.37
C2-M3-2	70.28	24.45	3.23	1.71	0.40	2.11	---
C3-M3-1	61.68	21.40	3.34	2.75	0.42	3.17	84.05
C0-M4-1	72.09	26.33	2.21	0.00	0.87	0.87	---
C1-M4-1	70.94	25.74	1.78	2.32	0.42	2.74	---
C1-M4-2	66.74	24.68	2.39	3.94	0.21	4.15	74.50
C2-M4-1	70.58	25.52	1.38	3.47	0.18	3.65	89.40
C2-M4-2	70.73	25.53	1.19	2.77	0.11	2.88	88.93
C3-M4-1	71.46	26.50	2.73	2.07	0.51	2.58	---

Appendix E

Additional Significant Prediction Models

Test C0-M1-1 was eliminated from the data set for these additional models and from the models in Chapter 3. This was done because the strength of this sample was more than two standard deviations from the mean and when removed from the data, a large improvement in the model was effected. Therefore, it was considered an outlier and removed from further statistical consideration.

Many models were found significant in the criteria of R^2 , C_p , MSE and overall F test. However, many of these models showed strong trends in their residual plots.

The models shown in Table E1 to E4 were also significant in these criteria, and in fact were very similar and in some of the criteria, are better than the models discussed in Chapter 3. In addition these models had residuals plots that were nearly as good as those reported earlier.

Table E1 Additional Significant Prediction Models for Dry Density, Dry-of-Optimum

Variables in Model	C_p	MSE	R^2	R_a^2	Overall F-test (F^*)	
a) $\sqrt{W_R}$	53.42	153	0.99	0.99	2447	(3.18)
$w\sqrt{W_R}$						
b) $w\sqrt{W_R}$	54.03	148	0.99	0.99	2520	(3.18)
$\sqrt{W_R}/w^2$						
c) $w W_R$	59.98	162	0.99	0.99	2303	(3.18)
$\sqrt{wW_R}$						
d) $(w_O - w)^{2^+}$	28.95	120	0.99	0.99	3127	(3.18)
$\sqrt{W_R}$						
e) $(w_O - w)$	43.22	141	0.99	0.99	2655	(3.18)
$\sqrt{W_R}$						
f) $\sqrt{w_O - w}$	52.85	155	0.99	0.99	2408	(3.18)
$\sqrt{W_R}$						

^+w_O is the optimum water content. $(w_O - w)$ = the deviation of water content of interest from optimum.

Table E2 Additional Significant Prediction Models for Dry Density, Wet-of-Optimum

Variables in Model	C_p	MSE	R^2	R_a^2	Overall F-test (F*)
a) $\sqrt{w}$	14.96	15.45	0.998	0.998	13499 (4.35)

Table E3 Additional Significant Prediction Models for Strength, Dry-of-Optimum

Variables in Model	C_p	MSE	R^2	R_a^2	Overall F-test (F*)
a) $\rho_d^2 / \sqrt{S_1}$ σ_3	52.40	13295	0.97	0.97	791 (3.18)
b) $\rho_d^2 / \sqrt{S_1}$ $\sqrt{\sigma_3}$	59.31	13038	0.97	0.97	807 (3.18)
c) $\sqrt{S_1}/w$ $\sqrt{\sigma_3}$	56.33	13411	0.97	0.96	783 (3.18)
d) $\sqrt{S_1}/w$ $(1-S_1/100)\sqrt{\sigma_3}$	27.46	8935	0.98	0.98	1190 (3.18)

Table E4 Additional Significant Prediction Models for
Strength, Wet-of-Optimum

Variables in Model	C_p	MSE	R^2	R_a^2	Overall F-test (F*)
a) W/e_i	11.45	2968	0.98	0.98	833 (4.35)
b) W/w	12.14	3037	0.98	0.97	813 (4.35)
c) W	15.51	3377	0.97	0.97	729 (4.35)
d) W/S_i^2	23.50	4219	0.97	0.97	580 (4.35)
e) $\sqrt{W}$	27.81	4617	0.96	0.96	528 (4.35)
f) $\sqrt{S_i}/w$	----	.00647	0.95	0.95	407 (4.35)
(log q_c)					
g) w	21.26	2267	0.98	0.98	548 (4.35)
w^2					

Appendix FPurdue University Negative Numbers for Photographs

Figure NumberNegative Number

2.2

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