JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP-81/14

COMPRESSIBILITY OF FIELD
COMPACTED CLAY

P.S. Lin

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Interim Report

COMPRESSIBILITY OF FIELD COMPACTED CLAY

To: H. L. Michael, Director
Joint Highway Research Project
August 13, 1981
Project: C-36-5M
From: C. W. Lovell, Research Engineer
Joint Highway Research Project
File: 6-6-13

Attached is an Interim Report on the HPR Part II Study titled "Improving Embankment Design and Performance". This is Interim Report No. 9 and is titled "Compressibility of Field Compacted Clay". It is authored by P. S. Lin and C. W. Lovell of our staff.

The report describes the compression testing of the field compacted St. Croix clay. Compactive prestress was found to be a function of roller pressure and compaction water content. Volumetric strain of the compacted soil upon saturation varied with the compaction water content, the compacted void ratio, and the confinement. The field compacted equations involve the same variables as the laboratory compacted ones, which means it is relatively easy to relate one to another.

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

Respectfully submitted,

C. W. Lovell

C. W. Lovell
Research Engineer

CWL:cm

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Interim Report
COMPRESSIBILITY OF FIELD COMPACTED CLAY

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Project No.: C-36-5M

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and the

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

Purdue University
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16. Abstract The compressibility of a plastic Indiana clay (St. Croix) is studied for both laboratory and field compaction. This interim report deals with the field compacted phase. Conventional oedometer tests were performed on a field compacted plastic clay to determine the effects of water content, dry density and compaction pressure on the as-compacted and soaked compressibility behavior. As-compacted samples were incrementally loaded; of particular interest was the value of compactive prestress induced during the compaction process. Environmental changes due to increased saturation were simulated using a back pressure technique. Of interest was the one-dimensional percent volume change on wetting under load. Statistical analyses were performed to establish the best predictive models for prestress and one-dimensional percent volume change on wetting, in terms of the important compaction variables. The field compacted relationships are similar to the laboratory compacted relationships developed by DiBernardo (1979), involving the same variables, exponents and signs. Predictions of field compacted relationships from laboratory tests can be accomplished for soils which somewhat different by using plasticity index ratio.			
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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	- American Association of State Highway and Transportation Officials
TRB	- Transportation Research Board
ISHC	- Indiana State Highway Commission
LIR	- load increment ratio
MSE	- error mean square
SPSS	- Statistical Package for the Social Science
e	- void ratio
e_o	- initial void ratio
f	- coefficient of compaction
P_{cp}	- compaction pressure (kPa)
P_o	- confining pressure (kPa)
P_s	- compactive prestress (kPa)
P_s/P_{cp}	- prestress ratio
pcf	- pounds per cubic foot
$\hat{p}_s$	- estimated response value of compactive prestress (kPa)
R^2	- coefficient of multiple determination
R_a^2	- adjusted coefficient of multiple determination
R_{100}	- compression value of 100% primary consolidation
S_r	- initial degree of saturation
u_w	- pore water pressure
u_a	- pore air pressure

- Δu_w - excess pore water pressure
- Δu_a - excess pore air pressure
- $\Delta u/\Delta \sigma$ - excess pore water pressure by applied pressure increment
- w - water content (%)
- ρ_d - dry density (kg/m^3)
- $\Delta V/V_o$ - one-dimensional percent volume change on wetting
- $\Delta \hat{V}/V_o$ - estimated one-dimensional percent volume change on wetting

CONVERSIONS

U.S. Customary

SI Equivalent

in

0.0254 m

ft

0.3048 m

lb f

4.45 N

ton f

8.90 kN

kg f

9.807 N

lb f/in²

6.895 kPa

lb f/ft²47.88 N/m²ton f/ft²

95.76 kPa

kg f/cm²

98.07 kPa

lb f/ft³0.1572 kN/m³

HIGHLIGHT SUMMARY

This study investigates the compressibility of a plastic Indiana clay (St. Croix) compacted in the field. Correlation among compaction variables and compacted properties was a prime objective. The clay was compacted to three levels of effort and at five levels of water content by two kinds of rollers. As-compacted compressibilities were assessed in the laboratory oedometer, and compaction prestress values were interpreted from the e -log p curves. These values were always less than the nominal roller pressures previously applied to the soil. A regression model for prestress was written in terms of the compaction pressure and an interaction between pressure and compaction water content.

Other compacted samples were saturated under three levels of confinement, with the aid of vacuum and back pressure. The subsequent volume changes depended upon the compaction variables, as well as the confinement during saturation. It was possible to develop a correlation among the volumetric strain, the initial void ratio, the compaction water content and the confinement during saturation. Soaked compressibilities were also measured.

A similar study on the compressibility characteristics of laboratory compacted samples of the same area soil had been conducted by DiBernardo (1979), and the coupling of the relations for field compaction with those previously established for laboratory compaction is reported here.

INTRODUCTION

Compacted soils are used in large quantities in the construction of roadway embankments and other fills. The stability of these structures against a slope failure is always of major concern, and in most cases it is the only criterion taken into consideration in actual design. However, with the construction of higher embankments becoming more common, it is increasingly more important to specify compaction procedures such that embankment settlement can be predicted and controlled for both the short and long term conditions.

During an embankment design, it is the desire of the geotechnical engineer to quantitatively predict and control the overall performance of field compacted soil. One method is to construct a special fill section using a range of compaction processes and then test samples from the soil mass after each process. Such a test pad with the associated costs of field sampling, laboratory testing and analysis is not economically feasible for most projects. Therefore, the design engineer must infer the compressibility behavior of field compacted soils from laboratory developed relations. As this inference process may not be the most desirable, this research has the ultimate objective of developing a rational method of predicting the field compaction response from laboratory tests.

This study investigates the compressibility behavior of a field compacted soil in the as-compacted and soaked conditions. The type of soil used was a plastic residual clay, and field compaction was achieved using a Caterpillar Model 825 roller and a RayGo Rascal Model 420C vibratory roller. Three energy levels and five molding water contents were used for each roller type to study their effects on compressibility behavior. To examine the as-compacted compressibility characteristics, the compacted samples were trimmed to appropriate size and incrementally loaded in the oedometer. Of particular interest was the value of compactive prestress. During the service life of an earthen embankment, environmental changes may effect an increase or decrease in the volume of the mass. In order to simulate the changes in the mass that may occur in-service, compacted samples were saturated by a back pressure technique in the oedometer under an equivalent embankment load. Of major interest was the percent volume change on wetting under loading. Saturated compressibility was also measured.

A similar study of the compressibility characteristics of laboratory compacted samples from the same area soil had been made by DiBernardo (1979). Combined, the results of both studies will allow the engineer to predict the field response from laboratory compacted samples.

REVIEW OF THE LITERATURE

Characteristics of Compacted Fine-Grained Soils

Theories of Compaction and Soil Structure

Compaction is the oldest method of improving soils for foundations and earthwork, and even today, it remains, in most instances, the most practical and economical method. The most obvious effect of compaction of soil is an increase in the density. While the actual increase in density may be only five or ten percent, the improvement in engineering properties may amount to several hundred percent. In general, most properties of cohesionless soils such as compressibility, coefficient of permeability and the strength angle depend only on the degree of compaction (or relative density). In contrast to this, it has been found for cohesive soils that the influence of the type of compaction, compactive effort and molding water content can be of greater significance than relatively small changes in the degree of compaction. The type of externally applied compaction energy, as well as the soil properties, will determine the structure of the compacted mass.

Proctor (1933) gave one of the first descriptions of the compaction process, the variables which affect this process, and the changes in soil structure that result from it.

Densification, or a reduction in the void ratio, occurs in a number of ways: reorientation of the particles; fracture of the

grains or the bonds between them, followed by reorientation; and bending or distortion of the particles and their adsorbed layers. Energy consumed in this process is supplied by the compactive effort of the compaction device. The effectiveness of the energy depends on the type of particles of which the fill is composed and on the way in which the effort is applied. In a cohesive soil the densification is primarily accomplished by distortion and reorientation, both of which are resisted by the interparticle attractive forces of "cohesion". As the water content of the soil is increased, the cohesion is decreased, the resistance becomes less, and the effort becomes more effective. If the moisture content is very high, however, the densification of cohesive soils leads to a high degree of saturation. The buildup of neutral stress prevents further reduction in void ratio, so that additional effort is wasted. Saturation, therefore, is the theoretical limit for compaction at any water content.

Hilf (1956) provided an explanation of the compaction process based on his pore water pressure theory for unsaturated soils. Using the capillary pressure model, he hypothesized that dry soils are difficult to compact because of the high frictional forces caused by high-curvature menisci. As the water content increases, the capillary forces decrease; consequently, the density increases. Past the optimum moisture content, the density decreases due to the buildup of transient air pressure which reduces compaction effectiveness.

In more recent years, however, the compaction process has proven to be governed by a number of complex phenomena, namely, capillary pressure, effective stress, osmotic pressure, pore air pressure and pore water pressure.

Lambe (1958a) introduced physico-chemical concepts to explain the structure and engineering behavior of compacted clays. Clay particles are dominated by electrical forces rather than mass forces because of their small size. The negatively charged clay particles attract cations and polar water molecules to them. The swarm of counterions and water is called the double layer, and the clay particles plus the double layer is termed the micelle. In his theory of compaction, Lambe (1958a) introduced the concept of water deficiency. This concept recognized that any given soil particle under any state of stress requires a certain amount of water to fully develop its double layer. The difference between this needed water and the existing water is deficient water which the particle will try to imbibe. A soil compacted dry of optimum is water deficient; as a result, the high electrolytic concentration depresses the double layer. Moreover, interparticle repulsion decreases, causing flocculation, which results in a low compacted density and a random particle arrangement. Increasing water content expands the double layer (reduction in electrolytic concentration), which, in turn, reduces the degree of flocculation. This permits a more orderly arrangement of particles, as well as a higher compacted density. The lubrication of particles allows for greater slippage which establishes a denser state. A further increase in water content

causes an expansion of the double layer and a continued reduction in the net attractive forces between particles. The compacted density decreases, even though the particle arrangement is more orderly, because the added water has diluted the electrolytic concentration of soil particles per volume. An increase in compactive effort results in higher densities by causing a more parallel arrangement and closer spacing of the clay particles. At high molding water contents, however, the effect may be to merely align the particles without significantly altering the particle spacing.

Seed, Mitchell and Chan (1960) and Seed and Chan (1961) attempted to explain and demonstrate some of the factors affecting the structure and strength of compacted clays, and the relationships between composition and strength in the as-compacted condition. The structure developed in a compacted soil is greatly influenced by the shear strains induced in the soil during the compaction process. Such strains apparently tend to produce a oriented arrangement of soil particles. Thus, for soils in which the interparticle forces are not so great that flocculation will occur under all compaction conditions, methods of compaction inducing shear strains produce a greater degree of particle orientation, greater shrinkage and less swelling, than methods of compaction inducing little shear strain. As a consequence of this effect, different methods of compaction tend to produce similar characteristics in samples compacted dry of optimum to any given density and water content, but produce different characteristics in samples compacted wet of optimum. For samples compacted wet of optimum to

any given density and water content, particle orientation and shrinkage tend to decrease in the following order of compaction methods: kneading, impact, vibratory and static.

A qualitative theory was developed in terms of effective stresses for kneading compaction by Olson (1963). This theory is predicated on the belief that the resistance of soil to compaction is the integrated sum of the shearing stresses developed between the particles at their points of contact, and that the macroscopic manifestation of these stresses can be examined in terms of the principle of effective stress.

Molding water content has been shown to have a dominant effect on a wide range of engineering properties of a compacted clay soil. Microscopic examination of the soil structure by Barden and Sides (1970) has revealed that major differences exist in macrostructure, and these have been shown to account for the main variations in engineering behavior. Dry of optimum, with continuous air voids, there is no consolidation problem in the classical sense and the major problems are associated with swelling or collapse on wetting. Wet of optimum, with occluded air voids, consolidation is a major problem and the Terzaghi theory is generally as relevant as any of the more complex theories. Unfortunately, the relevance of the Terzaghi theory must decrease as the transition is approached in the region of optimum moisture content.

Hodek (1972) explained the behavior of kaolinite compacted in the laboratory by static pressures under conditions of no lateral strain in terms of a deformable aggregate model. According to

Garcia-Bengochea (1978), pore size distribution measurements for compacted clays have also provided strong evidence for a deformable aggregate model. He cited Bhasin (1975) who found that as the compactive effort increases, the fraction of pores on the dry side of optimum decreases, whereas, on the wet side, the distribution remains the same.

Hodek and Lovell (1978 and 1979) developed a model or mechanism to explain the achievement of the compacted unit weight. The mechanism took into account the pre-compaction soil preparation and conditioning, as well as the soil interactions which occur during compaction. Figure 1 illustrates the effect of moisture content on compaction of a clayey soil. The "aggregation" is a fabric unit within which individual clay particles of various sizes interact. On the dry side, the clay aggregates are shrunken, hard and brittle. The compaction forces move these pieces around and perhaps even break some of them, but the result is a system with a minimum volume of small pores and a maximum volume of large pores. In contrast, on the wet side, the clay aggregates are swollen and plastic. The compaction forces can not only move the pieces closer together, but also deform them to minimize the interaggregate space. The system now has few large pores and many small ones.

White (1980) studied the prediction problem by investigating a possible source of the difference between the laboratory and field compacted earth - the soil fabric. The fabric is characterized by descriptors obtained from measurements of the distribution of the sizes of the voids in the fabric. At same moisture content, density,

Dry Side

Grains and clay
aggregations are
hard and brittle

The compacted result is a system with
a minimum volume of small pores and
a maximum volume of large pores

Wet Side

Clay aggregations are
plastic and swollen

The compacted result is a system with
few large pores and many small ones

FIGURE 1 THE EFFECT OF WATER CONTENT ON COMPACTION OF
A CLAYEY SOIL (AFTER HODEK AND LOVELL, 1979)

and soil, fabric differences should come from compactive action and compactive energy. The soil was St. Croix clay that had been compacted in a test embankment by two different pieces of compaction equipment, i.e., a Caterpillar Model 825 and a RayGo Rascal Model 420C. In the laboratory, the Proctor impact procedure and the California kneading procedure were used for compaction. White (1980) found that laboratory compaction by impact and kneading methods at about the same moisture content and density produced the same fabric. This supported the findings of Ahmed (1971) in his test on Grundite.

Terdich (1981) investigated the swelling pressure response of St. Croix clay compacted both in the field and laboratory. He found regression models for laboratory and field results which are different in variables and coefficients.

From the information presented in the literature, a comprehensive study of field compressibility behavior should contain several features. The range in water content should produce sufficient data for wet-side and dry-side relationships to be investigated. As noted by Price (1978) variations in compaction variables and behavior response should be measured and accounted for in the analysis. The effect of different types of field compaction equipment should be investigated. Field relationships should be compared with those of laboratory samples for the same soils at the same moisture content and dry density to determine if a large difference in their fabrics exists. The ultimate objective is to relate field and laboratory compactive effects on properties, and to determine how to predict field compacted properties from measurements on laboratory compacted soils.

Compactive Prestress

The concept of compactive prestress is generally defined as analogous to the preconsolidation pressure, with the pressure effect being caused by the compaction process (Woodsum, 1951). Lambe (1958b) agreed with this definition, but added that the value of the compactive prestress was sensitive to chemical and physical changes in the soil with time.

The process of compaction of a soil involves transmitting the external compaction energy to the soil skeleton and pore fluids. Upon completion of the process, there is an induced prestress in the soil which may or may not be equal to the compaction pressure. Abeyesekera (1978) observed that this value of compactive prestress is important with respect to compacted shale behavior.

In order to determine compactive prestress, Woodsum (1951) statically compacted soil directly in oedometer rings, allowed the soil to come to equilibrium with water under an applied load, and performed the consolidation test. He found that as the confining pressure remained constant and as the compaction pressure (P_{cp}) increased or void ratio decreased, the value of prestress (P_s) increased. Moreover, he found that although the value of prestress increased with increasing compaction pressure, the prestress ratio (P_s/P_{cp}), which was similar to the overconsolidation ratio for saturated soils, varied but slightly. This prestress ratio reflects the effective transmitting of external compaction energy to the soil skeleton. Upon completion of the compaction process, the induced prestress in the soil may or may not be equal to the compaction

pressure. In practical applications, this may suggest that for a given compaction process and type of soil, the efficiency of the process does not increase with increasing effort.

Table 1 is the summary of the results of the few prior studies concerning total compaction pressure and the induced prestress for compacted materials. In impact or kneading compaction the external pressure is maintained for a very short time before being removed. If the molding water content is greater than optimum during static compaction, and the load is released before the excess pore pressure can be fully dissipated, the end result may be similar to that of impact or kneading compaction.

Woodsum (1951) reached some conclusions regarding the prestress induced in statically compacted specimens of a highly plastic clay from the Fort Union formation and of a modified loess: (1) subsequent volume changes (swell) induced by the adsorption of water reduce the prestress and increase the compressibility, and (2) the values of prestress for both soils decrease with increasing initial void ratio. Leonards (1952), using Woodsum's data, showed that the confining pressure applied prior to sample soaking affected the level of prestress defined from e -log p curves. The higher the confining pressure before soaking, the greater the value of prestress for samples compacted to the same initial conditions.

Campbell (1952) studied the compressibility of bituminous concrete specimens formed under a static pressure of 2500 psi (17.2 MPa) applied for a dwell time of one minute. The specimens were loaded in a consolidation ring using a load increment ratio

TABLE 1. COMPACTION PRESSURE AND PRESTRESS (AFTER ABEYESEKERA, 1978)

Reference	Material	Compaction Method	Compaction Pressure P_{cp} (kPa)	Dwell Time (seconds)	Prestress Values P_s (kPa)	Prestress Ratio P/P_s cp
Campbell (1952)	Bituminous Concrete	Static	17238	60	1050 to 1450	0.06 to 0.08
Yoshimi and Osterberg (1963)	Silty Clay	Static 138 kPa/min	648	280	648	1.00
Mishu (1963)	Residual Clay	Kneading 30 blow/min	655 862 1172	1 1 1	249 249 383	0.38 0.29 0.33
Abeyesekera (1978)	Shale Aggregate	Kneading 30 blow/min	345 690 1379	60 60 60	345 469 to 510 676 to 869	1.00 0.68 to 0.74 0.49 to 0.63
DiBernardo (1979)	Residual Clay	Kneading 30 blow/min	460 to 657 657 to 814 2466 to 3919	1 1 1	72 to 315 55 to 505 85 to 1120	0.11 to 0.63 0.08 to 0.64 0.03 to 0.40
Witsman (1979)	Compacted Shale	Kneading 30 blow/min	394 657 to 788 2906 to 2990	1 1 1	195 to 394 280 to 788 630 to 1550	0.50 to 1.00 0.43 to 1.00 0.22 to 0.52

of unity. Each increment was applied for 24 hours. Abeyesekera (1978) estimated the induced prestress from the e-log p curves presented by Campbell, using the Casagrande construction, and found that it varied between 152 and 210 psi (1050 and 1450 kPa). The ratio of the prestress to the compaction pressure was only 0.06 to 0.08, and this was due to the dwell time of one minute being inadequate for the induced pore pressure to dissipate.

Yoshimi and Osterberg (1963) compressed a silty clay in a consolidation ring at constant loading rate of approximately 20 psi (138 kPa) per minute, using a hydraulic compression machine. When the specimen was reloaded in one-dimensional compression, the e-log p curve showed a distinct break near 94 psi (648 kPa), corresponding to the stress to which the soil was loaded previously. The ratio of the prestress to the compaction pressure in this case was close to unity and this was due to the slow rate of loading applied in forming the specimen.

Mishu (1963) compacted a clay by a California kneading compactor with foot pressures of 95, 125, and 170 psi (655, 862, and 1172 kPa) at a tamping rate of 30 blows per minute. For this tamping rate the dwell time is approximately one second. Abeyesekera (1978) estimated the induced prestress from the e-log curves and calculated that the ratio of the prestress to the compaction pressure varied between 0.29 and 0.38. These low values were due to the buildup of pore pressure in the specimens during compaction and inadequate dwell time for these pore pressures to dissipate fully.

Abeyesekera (1978) performed one-dimensional compression tests and determined the prestress induced in specimens of compacted shale. California kneading compaction pressures of 200, 100 and 50 psi (1379, 690 and 345 kPa) were applied to 500 gm of shale aggregate at various moisture contents over a period of one minute, and at a tamping rate of 30 blows per minute. The ratio of the prestress to the compaction pressure was found to vary between 0.49 and 1.0. Initial conditions of the test specimens affected the ratio. Either increases in compaction pressure or initial water content will reduce the ratio of induced prestress to the compaction pressure.

DiBernardo (1979) compacted a highly plastic residual clay by the California kneading compactor. He found that for samples compacted dry of optimum moisture content for any given level of compaction, the values of compactive prestress decreased with increasing water content. Wet of optimum, the value of compactive prestress was not influenced appreciably by varying the moisture content or level of compaction effort.

Witsman (1979) compacted New Providence shale in the California kneading compactor to simulate low, standard and modified impact energy levels. He found that the value of compactive prestress increased with increased compaction pressure and decreased with increasing moisture content at a given compaction pressure. The ratio of the prestress to the compaction pressure is near unity for samples compacted to the dry-of-optimum and optimum moisture conditions at the equivalent low and standard compactive efforts.

The equivalent modified compactive effort probably caused a reduction in the large pore mode of shale samples as compared to the lower effort levels. However, the energy required to obtain this reduction is not imparted as prestress in the soil skeleton. Therefore, the prestress ratio became substantially less than unity. Increasing moisture content also apparently reduced the large pore mode in the compacted shale. This occurrence, in conjunction with the excess moisture, probably reduced air permeability to zero and permitted the buildup of excess pore air and pore water pressures during compaction. The soil skeleton did not resist all of the compactive effort and therefore the prestress ratio is less than unity.

The behavior of a compacted soil will depend to a large extent on the induced prestress. More research should be carried out in evaluating this prestress and relating it to the compaction variables. Partially saturated soil compressibility and strength depend on the void ratio and the degree of saturation. However, soils having the same void ratio can have very different fabrics. Abeyesekera (1978) proposed that it would be appropriate to define the state of a compacted soil in terms of the prestress induced during compaction, in addition to the void ratio and degree of saturation.

Compressibility

DiBernardo (1979) states that the compressibility of compacted clay has not been dealt with extensively in the engineering literature.

This may be explained by two reasons: (1) the solution to the problem is not a straight forward adaptation of the Terzaghi consolidation theory for saturated soils, and (2) it is difficult to define the fundamental relationships which govern the compressibility of unsaturated soil under load.

Wilson (1952) studied the effect of water content on the compressibility of a compacted clayey sand. From the void ratio-pressure curves of the consolidation tests he found that the specimen compacted on the wet side of optimum was about 30% more compressible than that of the drier specimen. Since the wet specimen had a higher degree of saturation and was much less pervious than the dry specimen, it followed that the loading of a large mass of soil compacted on the wet side might produce high pore pressures and consequent reduction in effective stresses. Wilson suggested that highway embankments consisting of cohesive soils would best be compacted somewhat on the dry side of optimum in order to achieve high strength and low volume compressibility.

Leonards (1952) showed that the compressibility of the clay is affected by the confining pressure during contact with water, but this effect is minimized when high compaction is achieved. The data indicated that for a given load increment, the total settlement of fill constructed of highly plastic clay at a given initial water content and void ratio will be much greater if the fill comes in contact with water before the load increment is applied. The data also suggested that a compacted sample wetted in the oedometer at a low confining pressure will compress more

than a sample of equal void ratio wetted and confined at a higher pressure. Based on this, Leonards concluded that saturation by submergence would result in greater settlement than saturation through capillary suction since confining pressures are lower for the former case.

Lambe (1958b) explained the compressibility characteristics of compacted clay mainly in terms of particle rearrangement and colloidal chemistry; however, he does agree that other components, viz., gas change in volume or amount, particle deformation, and change in size of micelle, do come into play.

Yoshimi (1958) studied whether the classical consolidation process strongly influenced the overall compression behavior of an unsaturated silty clay. He showed that the pore water in a compacted silty clay was immobile during compression and that the time rate of compressive strain was independent of the soil thickness and drainage conditions.

Danielson (1963) studied three other compacted clays and tried to define some fundamental relationships for the compression of an unsaturated soil under load. He concluded that at saturations of 94 percent or less, no outflow of pore water could be expected in any of the soils. The compression of air upon loading accounted for between 35 to 65 percent of the total compression measured for the clays investigated. It was found that the amount of soil compression increased with increasing moisture content. The excess pore air pressure developed in an unsaturated soil was dissipated much more rapidly than the pore water pressure in a saturated soil.

The shape of the void ratio versus logarithm of time curves was the basis for Danielson's conclusions. During the first minute of a load increment, the curves had a decreasing slope that would change to a slope of constant value. He explained that the shape of the curve resulted from the dissipation of the excess pore air pressure due to initial compression. The rate of sample deformation after the first minute was constant. This was due to dissipation of residual pore pressures, overcoming viscous resistance, and particle rearrangement. The curves for the various soils flattened out after 400 to 600 minutes.

The compressibility characteristics of compacted soils were reviewed by Wahls, Fisher and Langfelder (1966) in great detail. A compacted soil was considered to be a three phase system composed of solids, air and water. The volume decrease which is caused by an applied load is generally the result of: (1) the compression of the solids, (2) compression of the water, (3) compression of the air, and (4) escape of water and/or air from the pores. From a practical standpoint the change in volume is principally the result of (3) and (4). They also concluded that the factors primarily influencing compacted compressibility were soil type, degree of saturation, molding water content and method of compaction.

DiBernardo (1979) examined the effect of laboratory kneading compaction on the as-compacted and soaked compressibility behavior of St. Croix clay. He found that a large percentage of as-compacted compression occurred within the first minute of loading. The compression vs. time relations indicated similar fluid continuity

conditions, i.e., continuous air voids, for dry and optimum conditions. He also summarized soaked compressibility behavior as follows: (1) for a given initial compaction condition, the soaked compressibility is greater than the as-compacted compressibility for the corresponding pressure ranges, (2) for a given compaction pressure and level of confinement on soaking, as the water content increases, the virgin compressibility (C_c) decreases, and (3) for a given degree of saturation and confinement level on soaking, as the compaction pressure increases, C_c decreases.

As a consequence of the laboratory compressibility studies on compacted New Providence shale, Witsman (1979) found that a large percentage of the as-compacted compression occurred within the first minute of loading. Continuous air voids and large pores permit a sample to equilibrate quickly with a change in stress. Samples compacted to the dry-of-optimum moisture condition for any compactive effort have the lowest compressibility at low applied loads, but compressibility increases greatly beyond the prestress level. This results in the overall compressibility being greatest for dry-side samples. Soaking and saturating compacted samples under vertical confining stress led to the following conclusions: (1) Under a low confining stress, at optimum and dry-of-optimum samples will swell. If the sample density is great enough, wet-of-optimum samples may also swell, (2) in general, wet-of-optimum samples show a tendency for a slight volume reduction upon saturation for the range of confining stresses investigated, and (3) the virgin compressibility of samples compacted dry-of-optimum and then saturated is less than

that of the as-compacted samples. Saturation does not seem to affect the virgin compressibility of the other saturated samples in relation to the as-compacted samples.

Compactive Effort of Roller

In the past, little progress has been made in the formulation of equations which explain and predict compactor performance. Even the determination of the principal performance parameters for each type is not complete. There is also an uncertainty about the influence of some of the parameters, such as the foot pressure for tamping rollers, and frequency for vibratory rollers. The designer today must primarily rely upon "cut and try" methods guided by past experience; the equipment user can determine capability only after actual field trials. Such an approach is limiting enough for the design process, but it is entirely unsatisfactory for selecting equipment from the wide variety available.

Johnson and Sallberg (1960) gave data that illustrated the compaction and operation characteristics of the several types of compactors on different types of soils. This was followed by brief statements on methods used as aids in the control of moisture content and density in construction. They also provided tabulated data on current state highway department practice, as indicated by specifications governing compaction requirements and compaction equipment. They also included tabulated data on manufacturers' specifications for compaction equipment, and, for the first time, provided data on permissible loads and inflation pressures for tires

used on pneumatic-tired rollers. Thus, the summary of the results of research with full-scale equipment and other data are very useful for the construction engineer.

Full-scale field tests dealing with soil compaction for highway construction were undertaken by Hampton and Selig (1967) to determine: (a) the desired characteristics of compacted soil, (b) how best to measure and specify the proper compaction, and (c) the effectiveness of various methods of achieving compaction. The test variables included were type of soil, moisture content, lift thickness, type of compactor, compactive effort, and number of roller coverages. The experiments were divided into 6 sets, 3 for subgrade soils and 3 for base course materials, each incorporating some of the independent variables for different purposes. Statistical techniques were used for planning the experiments and analyzing the data. They found that even with the aid of statistical methods and computer techniques, a considerable amount of analysis was required for proper interpretation. In addition, it was recognized that in spite of the great mass of data collected, more research was required both in the field and in the laboratory. It was necessary to broaden understanding of all aspects of the compaction process and the properties of compacted soils.

Selig (1971) developed a set of equations for defining compactive efforts of different classes of roller. Expressions were then derived for calculating production rate and power requirements. Although the compactor models used were elementary, they did provide a first order approximation for these three system characteristics

(compactive effort, production rate and power) in terms of the basic controlling parameters, including compactor weight, rolling width, lift thickness and travel speed. The resulting expressions have been checked using available data on compactor performance. Unfortunately, a direct measure of compactive effort has almost never been made in all of the field testing reported in the literature. This and other basic omissions make it impossible to obtain an accurate check on the equations and a precise estimate of the coefficient required, i.e., the coefficient of compaction(f). Nevertheless, the approach taken is suitable for a preliminary compactor analysis, and it is believed to be the best approach available at present.

Tamping Roller

The variety in tamping rollers can be quite broad, ranging from the sheepfoot type, with a contact area ratio of 7% or less, to the segmented pad type with ratios of 50% and higher. The performance characteristics vary considerably over this range, with the segmented pad type approaching the behavior of smooth wheel rollers, at least in some soil conditions. The emphasis here will be on the rollers at the lower end of the contact area ratio range. Tamping rollers of this type are most effective in high plasticity soils, such as heavy clays. The layer thickness must be properly controlled, which means keeping the loose lift thickness no more than about 2 in. greater than the foot length. Moisture greatly affects the roller behavior, including productivity, perhaps more so than with any other roller type. If the soil is too wet, the foot pressures will not be

supported, hence, the roller will continue to sink in. This is not an adverse feature, but rather a desirable one in many respects, because the surface breakup by the feet will aid in drying the soil until the moisture diminishes to the level where it is suitable for compaction.

One difficulty, particularly in wet cohesive soil, is keeping the feet clean. This problem, and tearing up of the soil, is the primary reason for limiting the forward speed. As the foot contact area ratio decreases and the foot length increases, the maximum speed should be decreased. Tamping rollers are less versatile, in general, because they cannot be used on cohesionless soils, base course materials and pavements, and they are not suitable on low plasticity soils.

The tamping roller represents a considerably more complex situation than other types of rollers. The surface area of soil contacted each pass is equal to the total area of the compaction feet times the ratio of distance traveled to rolling circumference. Thus,

$$A_L = NA \frac{L}{\pi(D + 2\ell)} \quad (1)$$

where:

A_L = Area contacted per pass, sq m

A = Contact area of tamping foot, sq m

N = Total number of feet

L = Length of pass, m

D = Drum diameter, m

ℓ = Length of compactor foot on drum, m

The volume of soil compacted is thus

$$V_c = c k_o P t A_L \quad (2)$$

where:

V_c = Volume of soil compacted, cu m

c = Foot area correction factor, ≥ 1.0

k_o = Overlap correction factor, ≤ 1.0

P = Number of passes

t = Compacted lift thickness, m

The foot area correction factor takes into account the fact that the volume of soil compacted by each individual foot is greater in horizontal extent than the foot area itself. The overlap correction factor is based on the fact that a portion of the roller feet contact an area of the soil surface already covered in previous passes. The overlap increases as the percentage of drum surface covered by feet increases.

The compactive effort, e (m-kN), is defined as

$$e = F L P \quad (3)$$

where:

F = Average towing force, kN

L = Distance towed, m

P = Number of passes over distance L

This equation implies that the compactive effort is all provided by the towing unit through the draw bar.

The towing force is assumed to be related to roller weight by

$$F = f w \quad (4)$$

where:

w = Total roller weight, kg

f = Coefficient of rolling resistance

Inserting Eq. (4) into Eq. (3) gives

$$e = f w P L \quad (5)$$

The compactive effort per unit volume is given by

$$E = \frac{e}{V_c} \quad (6)$$

which, after substituting Eqs. (2) and (5), becomes

$$E = \frac{f w \pi (D + 2\ell)}{k_o t c N A} \quad (7)$$

If desired, the drum weight, w , may be expressed in terms of foot pressure by

$$w = n A p \quad (8)$$

where:

n = Average number of feet in contact at one time

A = Foot area, sq m

p = Foot pressure, kg/sq m

With this substitution, Eq (7) would become

$$E = \frac{f n p \pi (D + 2\ell)}{k_o t c N} \quad (9)$$

The production rate, R (cu m/hr), is defined as

$$R = \frac{V_c}{T} \quad (10)$$

where:

T = Total time involved in compaction of volume V_c , hr

The time, T, may be expressed as

$$T = \frac{PL}{1000 S} \quad (11)$$

where:

S = Forward speed in km/hr

1000 = Conversion of km to m

Substitution of Eqs. (2) and (11) into Eq. (10) gives for production rate (cu m/hr)

$$R = \frac{1000 k_o t c N A S}{\pi(D + 2\ell)} \quad (12)$$

If Eq. (7) is solved for $k_o t c N A$ and substituted into Eq. (12), the resulting equation for R is

$$R = 1000 \frac{f w S}{E} \quad (13)$$

The power requirements may be computed by recognizing that RE is the work done per unit time (m-kN/hr). Thus, the power, H, is given by

$$H = \frac{RE}{86.4} = \frac{1000 f w S}{8816} \quad (14)$$

where:

86.4 = m-kN/hr/watt

8816 = m-kg/hr/watt

1000 = km/m

or

$$H = 0.114 f w S \quad (15)$$

The area covered in P passes is

$$A_1 = A_L P k_o c = \frac{N A P L k_o c}{\pi(D + 2\ell)} \quad (16)$$

The total area being compacted is BL, thus the percent area, A_p , covered in P passes is

$$A_p = \frac{A_1}{BL} = \frac{N A P k_o c}{\pi(D + 2\ell)B} \quad (17)$$

The number of passes for 100% coverage is then obtained by rearranging Eq. (17) and setting $A_p = 1$. Hence,

$$P_{100} = \frac{\pi(D + 2\ell)B}{N A k_o c} \quad (18)$$

The approach taken with the tamping roller indicates that the volume of soil beneath area BL which has been compacted decreases with each pass rather than remaining a constant as for the smooth roller. However, if it is the average state of compaction over area BL which is of interest, then $V_c = BL t$, and the expression for compaction effort per unit volume is

$$E = \frac{f w P}{Bt} \quad (19)$$

Vibratory Roller

Vibratory rollers are suitable for cohesive as well as cohesionless soils. However, efficiency decreases as plasticity increases, with the exception that saturated sands and wet silts are subject to strength loss during vibration. In clays, heavier rollers are generally required and lower frequencies are more effective. In fact, for the same vibration energy, as plasticity increases, frequency should be decreased and amplitude increased. The vibration of the soil particles is an aid to compaction only

in cohesionless materials. In cohesive soils it is primarily impact of the roller mass which produces results. The vibratory roller is claimed to compact to the greatest depth of any roller type. This probably is only true in cohesionless materials.

In the case of vibratory rollers, most of the compactive effort is usually applied by an engine on the roller drum. Additional effort may be applied by the towing unit through the drawbar. Neglecting the latter component, the applied compactive effort, e (m-kN) is (Selig, 1971),

$$e = z w T_L \quad (20)$$

where:

z = Net work per cycle, m-kN/cycle

w = Frequency of vibration, cycle/min

T_L = Time to travel distance L , min

Assuming that the roller configuration is the equivalent of a smooth steel drum, the volume compacted, V_c , is

$$V_c = BL t \quad (21)$$

where:

B = Roller width, m

L = Length of pass, m

t = Compacted thickness of layer, m

From Eqs. (20) and (21)

$$E = \frac{e}{V_c} = \frac{z w T_L}{BL t} \quad (22)$$

But the roller forward speed, S (km/hr), is equal to $17 \frac{L}{T_L}$, where 17 is the conversion factor from km/hr to m/min. Thus

$$E = \frac{z w}{17 S Bt} \quad (23)$$

The production rate is given by Eq. (10). Thus, from Eqs. (10), (11), and (21)

$$R = 1000 \frac{Bt S}{P} \quad (24)$$

Upon substitution for SBt from Eq. (23), R becomes

$$R = 58.8 \frac{z w}{P E} \quad (25)$$

Recalling from the derivation of Eq. (15) that

$$H = \frac{RE}{86.4}$$

when Eq. (25) is substituted for RE

$$H = 0.681 \frac{z w}{P} \quad (26)$$

The difficulty with this approach is that the net work per cycle is not easily determined. Instead, let the vibrator engine power being absorbed be designated H_v . Then, from Eq. (26)

$$z = 1.469 \frac{PH_v}{w} \quad (27)$$

Using this definition for z , Eqs. (23) and (25) become

$$E = \frac{0.086 H_v P}{SBt} \quad (28)$$

and

$$R = \frac{86.4 H_v}{E} \quad (29)$$

The power equation is, of course, simply

$$H = H_v \quad (30)$$

Define an equivalent, f , which will provide the same E through the drawbar as is being provided by the vibrator engine. That is, equate Eq. (28) to Eq. (19) and,

$$f = \frac{0.086 H_v}{w S} \quad (31)$$

If Eq. (31) is used to replace H_v in Eqs. (28) to (30),

$$E = \frac{f w P}{B t} \quad (32)$$

$$R = \frac{1000 f w s}{E} \quad (33)$$

and

$$H = 0.114 f w S \quad (34)$$

These last equations for R and H are identical to those for the tamping roller.

A summary of the equations derived in the preceding section is given in Table 2. In essence they relate total compactive effort (E), production rate (R), and power (H) to the roller parameters expressed in terms of weight (w) and width (B), the operational parameters of speed (S) and number of passes (P), and the soil parameters defined by compacted layer thickness (t) and coefficient of compaction (f), which is also a function of the roller characteristics and number of passes. By means of these equations it is possible, for example, to assign a numerical value to total compaction effort. It is also possible to evaluate trade-offs in power and production rate between the various controlling parameters for a given compaction effort.

TABLE 2. SUMMARY OF EQUATIONS BY SELIG (1971)

Compactive Effort, E

<u>Roller</u>	<u>Expression</u>
Tamping	$E = \frac{f w \pi (D + 2\ell)}{k_o t c N A}, \text{ or}$ $= \frac{f w P}{Bt} \text{ (if average compaction is specified)}$
Vibratory	$E = \frac{0.086 H_v P}{S Bt}, \text{ or}$ $= \frac{f w P}{Bt}$ $\text{with } f = \frac{0.086 H_v}{w S}$

Production Rate, R

All Types	$R = 1000 \frac{f w S}{E}, \text{ or}$ $= 1000 \frac{Bt S}{P}$
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Power, H

All Types	$H = 0.114 f w S, \text{ or}$ $= 1.15 \times 10^{-2} RE$
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Symbols and Dimensions

A = Contact area of tamping foot, sq m

B = Roller width, m

c = Foot area correction factor, ≥ 1.0

D = Roller drum diameter, m

E = Compactive effort per unit volume m-kN/cu m

TABLE 2. (CONTINUED)

f = Coefficient of compaction

H = Power, watt

H_v = Power of vibrator engine, watt

k_o = Overlap correction factor ≤ 1.0

l = Tamping foot length, m

N = Number of tamping feet

P = Number of passes

R = Production rate, cu m/hr

S = Forward speed, km/hr

t = Compacted lift thickness, m

w = Total weight, kg

EXPERIMENTAL APPARATUS AND PROCEDURE

Soils Studied

The soils used for both the field and laboratory compaction in this investigation were obtained from the State Road 37 realignment project in Perry County, approximately 4 miles south of St. Croix, Indiana and just north of Bandon. The soil stratigraphy at this location is shown in Figure 2 and was obtained from boring logs issued by the Indiana State Highway Commission (ISHC), Division of Materials and Tests. The soil will be called St. Croix clay from now on. Pertinent data are given in Table 3. The difference between the soils used for the field and laboratory compaction is apparently caused by the soil used for field compaction being taken from shallower depths (0 to 5.5 ft), while the samples taken for laboratory compaction were from deeper depths (5.5 to 13.0 ft). Laboratory variability is low due to the soil being taken from several locations and thoroughly mixed before compaction and testing.

Field Compacted Samples

Test Pad Construction

Ten test pads were constructed of the test soils in the area of the previously mentioned relocation project of State Road 37 in

Boring No. 20

State Road 37

1005+50,35 ft Left

FIGURE 2 BORING PROFILE (ISHC, DIVISION OF MATERIALS AND TESTS)

TABLE. 3 ST. CROIX CLAY CLASSIFICATION

	Field	Laboratory
Atterberg Limits	mean (low, high)	
Liquid Limit (w_l , %)	40.0(30.0,53.2)	53.0
Plastic Limit (w_p , %)	18.4(16.7,21.3)	21.0
Plasticity Index (I_p , %)	21.9(16.4,29.0)	32.0
Specific Gravity (G_s)	2.78	2.80
Soil Classification		
Unified	CL	CH
AASHTO	A-6	A-7-6

FIGURE 3 TEST PAD LAYOUT SHOWING PAD NUMBER, COMPACTOR TYPE, WATER CONTENT LEVEL, AVERAGE LIQUID LIMIT AND AVERAGE PLASTICITY INDEX

June of 1978. The test pads were constructed in order to create field-compacted soil for subsequent investigation of mechanical properties and fabric descriptors. Each pad was 14 feet (4.3 m) wide and 116 feet (35.4 m) long. Figure 3 shows the layout of the pads. Also shown are the average values for the Atterberg Limits. The test pads were placed on a leveled test site in 8 inch (20 cm) loose lifts. Prior to rolling of the test pads, water was sprinkled into the soil and the area was disked to aid mixing.

Five test pads were rolled by a Caterpillar Model 825 segmented pad tamping roller (C) and five pads were rolled with a RayGo Rascal Model 420C segmented pad vibratory roller (R). Samples taken from test pads compacted with each of these rollers were designated "C" and "R", respectively. Specifications for the equipment may be found in Table 4. These rollers were selected by the contractor because of their capability to perform satisfactorily with the soils at the site. Tamping rollers are most effective in high plasticity soils, such as heavy clays. Moisture greatly affects the roller behavior. If the soil is too wet, the general bearing capacity of soil is exceeded, the foot pressures will not be supported, hence, the roller will continue to sink in. Selig (1971) found that vibratory rollers are suitable for cohesive as well as cohesionless soils. However, efficiency decreases as plasticity increases.

The segmented pad rollers which were on both pieces of compacting equipment preclude applying a completely uniform compactive effort to each test pad. During a single pass of the roller, only

TABLE 4. FIELD COMPACTION EQUIPMENT SPECIFICATIONS

Caterpillar Model 825

Dimensions

Length, with dozer	23ft 4in.
Width, w/o clearers	11ft 11in.
w/o dozer	12ft 6in.
w/o dozer	13ft 7½in.
Wheelbase	140in.

Weight, shipping

w/o dozer	59,000 lb
with dozer	63,000 lb

No. of Drums 4

No. of Pads/Drums 65

Each drum width 44½in.

Max. ballast

per wheel 244 U.S. Gal

Bulldozer dimensions

Length 14ft

Height 40½in.

Maximum Speeds

	Gear	MPH
Forward	1	3.1
	2	7.0
	3	17.0

RayGo Rascal Model 420C

Dimensions

Length, with blade	18ft 9in.
Width	9ft
Wheelbase	9ft

Weight 25,160 lb

Vibration Drive Hydraulic, Direct Drive

Frequency 1100 to 1500 vpm

Dynamic Force 32,000 lb

No. of Drums 1

No. of Pads/Drums 140

Each drum width 84in.

Maximum Speeds

	Gear	MPH
Forward	1	4
	2	6
	3	8

the soil under the raised pads received the maximum applied energy; areas surrounding the pads have experienced no compaction at all. On subsequent passes the compactor pads could run over either previously compacted soil or relatively uncompactive areas. As the number of passes increased, the process should provide increasing uniformity in compactive effort applied across the test pad. The variation in applied energy was believed to be one cause of the variability in field dry density measurements at various places within a given test pad, as shown in Table 5.

Additional discussion of field compaction variability can be found in Price (1978). The compactors themselves could have provided a source of variation; because their configurations were different, the ways in which they applied energy to the soil were also different. In a single pass, the Rascal roller applied compactive effort through one wide drum followed by two rear rubber tires. Higher confinement, and thus higher compaction, might have been achieved near the center of this drum, with more "squeezing-out" having occurred near its end. The Caterpillar roller used four tamping drums to apply compactive effort. For each pass of this equipment, two drums traveled over a given area of soil which increased the probability of pad-to-soil contact for a given soil location. The effect of confinement was probably not as large for the narrower Caterpillar drums, and neglecting the effect of travel speed, all compaction energy was supplied by static weight.

It was desired to obtain a set of five test pads at five different water contents for each method of compaction, with water contents on both the dry side and wet side of field optimum water contents.

TABLE 5. DENSITY AND MOISTURE CONTENT VALUES OBTAINED FROM FIELD
NUCLEAR GAGE MEASUREMENTS

No. of Passes			4		8		16	
Pad No.	Compactor Type	Moisture Level	γ_d (pcf)	w (%)	γ_d (pcf)	w (%)	γ_d (pcf)	w (%)
1	R	5	108.2	22.5	106.9	23.6	111.9	17.0
			95.0	26.0	106.3	19.4	102.6	28.1
			101.7	23.1	103.4	19.2	105.7	21.8
			R5A		R5B		R5C	
2	C	5	97.2	26.1	101.2	25.1	106.0	22.6
			106.9	23.8	97.7	27.4	105.5	21.8
			100.0	20.0	103.8	21.1	106.3	20.7
			C5A		C5B		C5C	
3	C	4	103.1	21.2	106.6	20.5	111.6	17.0
			108.0	19.2	100.0	22.7	104.0	21.2
			100.0	22.1	105.5	19.4	99.4	21.4
			C4A		C4B		C4C	
4	C	3	105.0	10.3	109.1	14.7	107.4	15.4
			104.9	16.3	110.3	12.4	105.9	15.6
			107.4	12.7	103.8	14.0	107.5	14.0
			C3A		C3B		C3C	
5	R	4	99.5	20.6	101.2	17.9	103.3	19.0
			99.4	21.1	102.5	25.7	95.6	22.8
			96.9	19.2	101.2	17.9	105.9	17.0
			R4A		R4B		R4C	
6	C	2	103.2	17.1	108.4	13.7	102.1	15.2
			105.3	15.7	111.5	16.1	108.4	17.2
			105.8	13.9	114.2	14.7	103.9	15.5
			C2A		C2B		C2C	
7	C	1	98.0	14.8	113.2	15.4	110.2	15.6
			104.0	14.9	109.3	14.5	112.5	16.0
			96.0	16.9	111.1	16.5	112.8	16.8
			C1A		C1B		C1C	
8	R	2	104.3	17.7	111.9	13.8	105.9	15.1
			98.0	16.6	106.8	12.5	110.7	14.2
			101.8	14.2	102.3	14.9	108.7	15.6
			R2A		R2B		R2C	

TABLE 5. (Continued)

No. of Passes			4		8		16	
Pad No.	Compactor Type	Moisture Level	γ_d (pcf)	w (%)	γ_d (pcf)	w (%)	γ_d (pcf)	w (%)
9	R	1	98.9	15.7	103.2	16.8	114.5	13.6
			101.7	16.4	107.6	12.9	112.5	14.7
			104.2	14.6	107.4	14.4	106.3	14.5
			R1A		R1B		R1C	
10	R	3	103.5	14.0	107.6	17.0	111.9	16.5
			98.3	13.0	104.8	15.7	105.8	15.0
			95.9	14.6	106.4	12.9	108.6	15.9
			R3A		R3B		R3C	

For identification these were designated "1" to "5", from the lowest to the highest moisture level. It was hoped to maintain a uniform water content within each pad. Although special precautions were taken in the field, it can be seen from Table 5 that some variation did exist. Variation appears to be inherent in compaction projects, and control of water content variation is essential to predictable performance of the embankment (Price, 1978).

The desired water content for each test pad was achieved in the field in different ways. First, moisture content samples were taken at four different locations in each pad using the Speedy Moisture Tester apparatus, and the average was tabulated. If the pad was initially too wet, a tractor-drawn disk was employed to facilitate drying and to help produce moisture uniformity. If the pad was initially too dry, water was added by passage of a calibrated water truck; the soil was then mixed again by disking. Those pads which were at the desired water content were briefly disked for mixing purposes and then compacted.

Each test pad was sampled following the completion of 4, 8 and 16 passes of the compaction equipment over the pad. These samples were labeled "A", "B", and "C", respectively, for identification of energy levels. Several steps were taken during the compaction process to reduce variability in applied energy. Equipment was operated at top speed in first gear (about 3 mph or 4.8 kmph) for all passes. Enough room was allowed for the roller to reach this top speed before entering the actual test pad area, and equipment

was carefully guided to prevent overlap of adjacent coverages. The same equipment operator was used for all compaction work. Figures 4 and 5 show the moisture-density-energy curves from field nuclear gage values for the Caterpillar roller and Rascal roller, respectively. These plots indicate large variability within the soil mass.

Field Sampling

Following completion of the required number of passes by the compaction equipment, field sampling of the test pad was started. The test pad was laid out in a 2 ft by 2 ft (0.6 m by 0.6 m) grid pattern and marked around the perimeter with consecutively numbered stakes. Sampling locations had been determined by a random number process in an earlier phase of this project. Figure 6 shows a typical field sample label and its explanation. For this study, six samples were taken for each combination of roller type, water content level and energy level. A total of 180 were taken for this compressibility study.

Sampling tubes were driven into the ground with a drop hammer at the specified grid location at the bottom of the taper-foot depression made by the roller. Figure 7 shows the driving assembly with sampling tube. The sampling tubes were machined in the Central Machine Shop at Purdue University. The consolidation-sample tubes were made from steel tubing with an internal diameter of 2.51 in. (6.38 cm), an external diameter of 2.75 in. (6.99 cm), and were 5.0 in. (12.7 cm) long. One end was machined to form a cutting edge angle. The assembly used to drive the sampling tubes consisted of: (1) a driving head to which the sampling tube was attached with

FIGURE 4 CATERPILLAR COMPACTION CURVES FOR FIELD NUCLEAR GAGE VALUES (AFTER TERDICH, 1981)

FIGURE 5 RASCAL COMPACTION CURVES FOR FIELD NUCLEAR GAGE VALUES (AFTER TERDICH, 1981)

CONSOLIDATION
C2A5 (4/42)

CONSOLIDATION – Behavior property to be studied

C	– Compactor type
2	– Water content level
A	– Energy level
5	– Sample number for the compaction
conditions indicated above
4	– Grid location in the East–West direction
42	– Grid location in the North–South direction

FIGURE 6 TYPICAL FIELD SAMPLE LABEL

FIGURE 7 DRIVING ASSEMBLY WITH SAMPLING TUBE

four set screws, (2) a pipe threaded into the driving head to guide the falling weight, (3) the falling weight itself, (4) a disk on the end of the pipe to use as a handle and to control the height of drop, and (5) two wood cushions, one placed between the sampling tube and driving head and the other placed between the driving head and the falling weight. The 16.7 lb (7.6 kg) weight fell 28 inches (71 cm) producing about 40 ft-lb (54 N·m) of energy per blow, with 10 to 25 blows being required to drive each tube. The total weight is 27.5 lb (12.5 kg), light enough to be operated by one person.

Shortly after the sampling tubes were driven, they were removed from the ground; samples were then extruded from the tubes using either an hydraulic jack (see Figure 8) or an electrically driven loading press. Samples were then wrapped in plastic, covered with cheesecloth, and finally coated with paraffin. They were then labeled and transported to Purdue University for testing. During extrusion of tube samples, several observations were made. Many dry-side samples were quite brittle, broke easily, and their heights were generally shorter than the average height. Samples in the low and intermediate water content ranges sometimes showed distinct zones of different water contents. This was due to the incomplete mixing of soil lumps which were at different initial water contents. Wet-side samples produced the longest and most uniform samples. Rock fragments were found in all samples, and some samples were destroyed because large rocks were present. Some samples showed horizontal cracks, while others showed distinct interfaces between individual lumps of soil which had not been intermixed by the compaction process.

FIGURE 8 HYDRAULIC JACK USED FOR EXTRUSION OF TUBE SAMPLES

As-Compacted Compressibility

Apparatus

A Karol-Warner fixed ring consolidation cylinder was used in this investigation; it will henceforth be referred to as an "oedometer". It is made of brass, except for the loading piston, which is stainless steel. Its major feature is the capability of safely withstanding large backpressures, i.e., 150 psi (1035 kPa). The oedometer ring is 2.50 in. (63.5 mm) in inside diameter, 4.0 in. (101.6 mm) in outside diameter, and 1.0 in. (25.4 mm) in height. It is not designed to eliminate or reduce wall friction; therefore a silicone lubricant (high vacuum grease) was applied to the wall of the ring before each test. Figure 9 shows the ring as well as the complete assembly.

Two bronze medium-porous stones of 0.25 in. (6.35 mm) thickness were placed at the top and bottom of the sample and allowed pore fluid to drain during compression. The top stone diameter was slightly less than the ring inside diameter; this reduced the risk of the stone hanging up on the side of the ring during compression. A single sheet of medium porous filter paper separated the stones from the sample; this prevented the soil particles from intruding the porous stones.

Figure 10 shows the loading frame used to compress the sample. It is a lever arm-weight type loading system capable of performing six oedometer tests at the same time. The mechanical advantage of the system is a 10:1 load ratio for the back hanger and a 40:1 load ratio for the front hanger. The maximum design capacity of the

FIGURE 9 KAROL-WARNER CONSOLIDATION CYLINDER

FIGURE 10 LOADING FRAME

frame is 25.1 kN (2560 kg), which provided a maximum vertical stress of about 8000 kN/m^2 (81.6 kg/cm^2).

The sample compression was measured by a Federal dial gage, which measures deflection to the nearest 0.0001 in. (0.00254 mm). The gage can measure a maximum deflection of 0.4 in. (10.16 mm). During compression the gage stem is in contact with the loading yoke and measures the movement of the piston, which, in turn, measures sample compression.

Procedure

The first step in the trimming process for the field samples was to remove the wax and cheese cloth covering. This was accomplished by using a sharp knife to cut through the covering, and care was taken to prevent damage of soil sample inside.

The next step was to put the soil sample directly into the oedometer ring. The upper and lower faces of the sample were trimmed with a steel straight edge using the top and bottom of the ring as guides. The trimmings were collected for moisture content determinations.

The soil sample was weighed to the nearest 0.01 g. Four separate heights were measured 90° apart by a venier caliper and the average height was calculated. The filter paper was placed above the bottom porous stone, and the soil sample was seated against the filter paper. Then the filter paper and the other porous stone were placed on the top of the specimen. The loading platten, as well as a saturated strip of cotton (used to reduce moisture loss by evaporation), were placed on the top stone. The oedometer was assembled,

and the loading piston was seated firmly in its position on the platten. The oedometer was placed on the loading frame, and the yoke was adjusted to be horizontal before the dial gage was mounted on the yoke. During the compression the gage stem was in contact with the loading yoke and measured the movement of the piston, which, in turn measured sample compression. If the yoke was not horizontal, distortion would cause reading error after the load was applied. After the yoke had been adjusted to be horizontal, the dial gage was mounted, and a seating load of 9.81 kN/m^2 (0.1 kg/cm^2) was placed. All dial gage readings were ignored up to this point.

Following the seating load adjustment period (typically 10 minutes), the applied pressure was increased, using a load increment ratio (LIR) of 0.5, to: 14.9, 22.3, 34.7, 49.5, 76.8 kN/m^2 , etc., before the prestress value for the particular sample was reached. The applied pressure was decreased and then increased again until the prestress value for the particular sample could be well defined. Since the loading frame used in this study is a level arm-weight type loading system, the arm should be leveled to the horizontal after each load. The duration of each load and unload was 10 minutes, during which, dial readings were typically recorded at 0, 0.1, 0.5, 1, 2, 4, 8, and 10 minutes. At the end of the last load, the weights were removed from the loading frame, the oedometer assembly was dismantled and a final water content was measured.

Discussion

DiBernardo (1979) studied the effect of laboratory compaction on the compressibility of a compacted highly plastic clay. He used

St. Croix clay and found that it was difficult to trim the tube sample to the top and bottom of the ring. Dry of optimum samples tended to crumble very easily when trimmed, which usually resulted in larger voids at the top and bottom of the sample. Wet of optimum samples remained intact, however, they tended to pull from the side of the ring in the direction of trimming motion.

Except reducing evaporation from the sample during compression by a saturated strip of cotton, little was done to maintain isothermal conditions in the testing room. Plum and Esrig (1969) found that changing soil specimen temperature could produce effects similar to changes in stress history, as well as changes in pore water pressure. However, throughout the four months as-compacted testing program, the room temperature varied by only $22^{\circ} \pm 3^{\circ}\text{C}$.

The 10 minutes duration for load and unload was based principally on the findings of Yoshimi (1958) and DiBernardo (1979). Yoshimi (1958) performed one-dimensional compression tests on statically compacted Vicksburg silty clay and observed that a relatively large amount of compression took place within the first 4 minutes after load application, following which, a relatively small amount occurred. DiBernardo (1979) studied the effect of laboratory compaction on the compressibility of a compacted highly plastic clay (St. Croix) and found that a large percentage of as-compacted compression occurred within the first minute of loading.

The load increment ratio used for the as-compacted compression test should meet the following requirements: (1) a large amount of the compression must occur within 10 minutes, (2) as few loads as

possible should be used, to reduce moisture loss by evaporation, and (3) the prestress value could be accurately defined by the Casagrande construction. DiBernardo (1979) studied the results of Leonards and Ramiah (1959) for determining the preconsolidation value for saturated soils and the results of Yoshimi (1958). He found that if an LIR = 1 were used, requirements (1) and (2) would be satisfied, however, requirement (3) may not; if an LIR = 0.1 or 0.2 were used, requirement (3) would be satisfied, but not requirements (1) and (2). He concluded that a load increment ratio ideally equal to actual field loading conditions should be used, however, that depends on a given application and may be difficult to determine. Therefore, an LIR = 0.5 was chosen, since it most suitably matched the requirements. An LIR = 0.5 was also chosen for the field compaction study in order to make a comparison with laboratory results.

Leonards (1976) showed a typical curve of e versus σ' for 3 cycles of loading and unloading in Figure 11 and stated that the recompression index C_r depends on: (1) the magnitude of σ' at which unloading is begun, especially whether σ' is less than or exceeds the preconsolidation pressure P_s ; (2) the overconsolidation ratio OCR to which rebound (and reloading) is allowed (e.g., σ'/σ'_r in Figure 11); and (3) the degree to which gas bubbles are present in the sample because of the reduction in pore water pressure that resulted from sampling. In this study, C_r was evaluated over the range $P'_0 + \Delta P$ (ΔP is the external load applied to the soil sample as shown in Figure 11).

FIGURE 11 TYPICAL CURVE OF VOID RATIO VERSUS EFFECTIVE VERTICAL STRESS (AFTER LEONARDS, 1976)

Soaked Compressibility

Apparatus

The oedometer and loading frame used to determine the soaked compressibility characteristics are the same as those used in the as-compacted compressibility test.

In order to measure pore pressures during consolidation and back pressures during saturation, a Tyco pore pressure transducer (Model AB) was used. This transducer can safely measure up to 500 psi (3450 kPa), which is well beyond the maximum required. The device is made of a series of strain gages which measure the deflection of a diaphragm. Pressure exerted on the diaphragm causes it to deflect, thus changing the electrical resistance of the gages. The accuracy of the transducer is rated at 1 percent over the full pressure range at normal temperature; it was found to be quite accurate and reliable. The pressures were recorded by a Hewlett-Packard Digital Multimeter (Model 3476A).

A de-airing system was used to remove air from the oedometer. It consisted of a vacuum regulator, a vacuum pump, an aspirator and a de-aired water reservoir. The use of this apparatus was not essential; however, it expedited the saturation process.

Procedure

The field sample was assembled in the oedometer as described in the as-compacted compressibility test. The sample was compressed using a $LIR = 0.5$ and load duration of 10 minutes in the as-compacted condition until a vertical pressure of either 160, 320, or 480 kN/m^2

was reached, whereupon, it was allowed to compress for 30 minutes. At the end of this sustained loading period, the de-airing apparatus was connected to the oedometer at its top and bottom. 35 kN/m^2 (5 psi) of vacuum was applied and maintained for approximately 1 hour. Then the vacuum was disconnected from the bottom of the cell. An overhead reservoir filled with de-aired water was connected to the bottom of the cell. The top vacuum pressure sucked the water through the cell, therefore filling it with water, without air entering the cell. Once filled, the de-airing apparatus and the overhead reservoir were disconnected from the oedometer. The sample, now in contact with water, was allowed to come to equilibrium. During this period the pore pressure transducer was connected to the bottom of the cell.

After equilibrium was reached, the back pressure lines were connected to the top and bottom of the cell. The back pressure was increased at 70 kN/m^2 (10 psi) every hour until a maximum pressure of 965 kN/m^2 (140 psi) was reached. Then the sample was permitted to adjust under full back pressure for a period of 48 to 72 hours. At the end of the adjustment period, the back pressure line was disconnected from the bottom of the cell; however, the back pressure was still maintained at the top of the cell.

The sample was unloaded from the original sustained load to 89.2 kN/m^2 using a LIR = 1.0. During unloading, dial readings were recorded at the conventional times. The duration of each unload was based on t_{100} values determined from pore pressure measurements. Then the back pressure line was again connected to the bottom of

the cell. The sample was reloaded, using a LIR = 0.5, to 113.9, 173.4, 260.0, 383.9, 582.0, 866.8, 1362.1, 1981.3, 2971.9, and 3962.6 kN/m². During loading, dial readings were recorded at the conventional times. The duration of each load was based on t_{100} values determined from the Casagrande construction. At the end of the last load, the back pressure lines were disconnected, the weights were removed from the loading frame, the oedometer assembly was dismantled, and a final water content was measured.

Discussion

In order to determine the compressibility characteristics of a saturated embankment subjected to further load, soaked compressibility tests were performed on the field compacted St. Croix clay. The sample was initially compressed in the as-compacted condition to a sustained load to simulate field loading. This load--either, 160, 320, or 480 kN/m²--represents the pressure exerted by an equivalent embankment height of 7.6 m (25 ft), 15.2 m (50 ft), and 22.9 m (75 ft), respectively. A wet unit weight of 20.9 kN/m³ (133 pcf) was assumed.

The back pressure was used to saturate the sample. As stated previously, the back pressure was incremented at 70 kN/m² every hour until the maximum pressure 965 kN/m² was reached. Lowe, Zaccheo, and Feldman (1964) suggest incrementing every 30 minutes, therefore twice this time is considered reasonable.

After 36 hours of sustained loading and maximum back pressure, the following approach was taken to determine saturation. The drainage lines from the oedometer were closed, and a 0.5 kg weight

was added. Two to five minutes after application, the pore water pressure response was recorded and the value of $\Delta u/\Delta \sigma$ was determined. The above approach was performed every five hours; if three successive attempts gave the same $\Delta u/\Delta \sigma$ ratio, the back pressure was increased to 1035 kN/m^2 (150 psi). If not, the forementioned procedure was repeated until the requirement was met.

Lowe and Johnson (1960) and Black and Lee (1973) presented theoretical curves which show the back pressure required to bring an unsaturated sample from an initial degree of saturation to a final degree of saturation. DiBernardo (1979) found that for the range of saturation greater than approximately 80 percent, the maximum back pressure used in this procedure was sufficient for achieving full saturation; however, for lower degrees of saturation, greater amounts of back pressure would theoretically be required. Greater amounts of back pressure was not applied in this study, since, if the time was long enough, the saturation check was satisfied.

During this study, the air compressor failed and could not be economically repaired. A cylinder of compressed nitrogen was substituted, which could supply the back pressure to 3500 kN/m^2 (500 psi).

During loading, dial readings were recorded at the conventional times. Since the soaked compression versus log time curves have a characteristic type I shape, as proposed by Leonards and Girault (1961), the division of consolidation into primary and secondary components can be determined by the Casagrande construction.

ANALYSIS OF DATA AND DISCUSSION

As-Compacted Compressibility

Compaction

Figure 12 shows the moisture-density-energy relations of the Rascal vibratory roller for three energy levels A (4 passes), B (8 passes) and C (16 passes). The water contents and dry densities are the average values of the field samples compacted at the same water content and energy levels by the same roller type. Also shown in Figure 12 is the moisture-density-energy relations of St. Croix clay for the Low Energy, Standard and Modified Proctor impact levels from Terdich (1981). In this figure, observe that the energy level A (4 passes) of the Rascal vibratory roller corresponds to the Low Energy level of impact method. Also observe that the energy levels B (8 passes) and C (16 passes) of the Rascal vibratory roller are close to and above the Standard Proctor impact level, respectively.

A similar correspondence between the Caterpillar tamping roller and the impact method is shown in Figure 13. In this figure, observe that the energy level A (4 passes) of Caterpillar tamping roller is above the Low Energy impact level but below the Standard Proctor impact level. Also observe that the energy levels B (8 passes) and C (16 passes) of the Caterpillar tamping roller are both above the Standard Proctor impact level.

FIGURE 12 CORRESPONDENCE BETWEEN RASCAL ROLLER AND IMPACT METHOD (AFTER TERDICH, 1981)

FIGURE 13 CORRESPONDENCE BETWEEN CATERPILLAR ROLLER AND IMPACT METHOD (AFTER TERDICH, 1981)

In view of the above, it is suggested that the Caterpillar tamping roller would produce more efficient compaction with this soil than the Rascal vibratory roller.

The compaction variables used to characterize each as-compacted compressibility test sample are defined as follows:

- (1) Water content (w , %) - field tube sample moisture determinations.
- (2) Dry density (ρ_d , kg/m^3) - the dry density of the oedometer sample computed from its oven-dry weight and volume.
- (3) Compaction pressure (P_{cp} , kPa) - the average compactive force per unit area applied during compaction.
In the case of vibratory roller, usually most of the compactive force is applied by the vibration energy produced with an engine on the roller drum. Additional effort may be applied by the towing unit through the drawbar, but the latter component is neglected in this study.
- (4) Plasticity index (I_p , %) - the liquid limit minus the plastic limit is the plasticity index. Changes in I_p imply changes in soil composition.

The other computed variables were initial degree of saturation (S_r , %) and initial void ratio (e_o).

The compaction water contents were selected to lie between four percent wet and four percent dry of field optimum water contents. Their specific values within this range were selected arbitrarily to account for the expected variation in water contents among field-

compacted samples.

The dry density variable used in this study refers to the oedometer sample density rather than the field nuclear gage value. This density was used due to the trimming and sampling effects. That is, trimming often caused voids in the brittle dry-of-optimum samples, and the release of tube confinement prior to testing caused swelling in the wet-of-optimum samples.

Twenty-four as-compacted compressibility tests were performed; the values of the previously defined compaction variables are listed in Table 6. A sample designated by R5B2 would indicate the sample was compacted by the Rascal equipment (R) at the fifth water content level (5) and was the second sample (2) collected after the equipment made 8 passes (B).

Compression vs. Time Relations

A series of typical compression vs. time curves for samples compressed from initially dry of optimum are shown in Figure 14. For each load increment shown, a large amount of compression occurs within the first minute of loading, with little compression occurring thereafter. Yoshimi (1958) and DiBernardo (1979) observed similar behavior for laboratory compacted silty clay and highly plastic clay (St. Croix), respectively. They described the two parts as follows: (1) initially rapid compression - the extremely rapid dissipation of excess pore air pressure ($\Delta u_a > 0$, $\Delta u_w < 0$), as well as the initial compression of the pore air and soil skeleton; (2) subsequent negligible compression - the dissipation of residual pore air pressure and soil particle rearrangement.

TABLE 6 INITIAL COMPACTION VARIABLES FOR AS-COMPACTED COMPRESSIBILITY SAMPLES

Sample No.	Water Content (%)	Dry Density ρ_d (kg/m ³)	Compaction Pressure P_{cp} (kPa)	Plasticity Index I_p (%)	Degree of Saturation S_r (%)	Initial Void Ratio e_o
C1A2	12.64	1654.5	797	21.0	51.12	0.692
C2A1	18.37	1618.8	797	23.3	70.49	0.730
C3A2	13.23	1645.2	797	21.1	52.78	0.702
C4A1	17.36	1723.3	797	22.3	77.80	0.625
C5A1	17.26	1707.1	797	24.8	75.48	0.640
C1B1	12.47	1799.1	1204	21.0	62.78	0.556
C3B1	12.63	1855.3	1204	21.1	69.45	0.509
C4B1	17.27	1738.3	1204	22.3	79.17	0.611
C5B1	16.44	1804.0	1204	24.8	83.37	0.552
C1C1	10.65	1910.0	1771	21.0	63.98	0.466
C2C1	11.96	1892.4	1771	23.3	69.80	0.480
C4C1	15.79	1804.1	1771	22.3	80.09	0.552
C5C1	12.98	1837.9	1771	24.3	69.42	0.523
R1A1	12.61	1795.0	780	18.1	63.04	0.560
R2A1	12.59	1819.7	780	16.9	65.42	0.604
R4A1	14.06	1827.7	780	26.5	73.98	0.532
R1B1	15.60	1779.5	1038	24.8	76.16	0.573
R2B1	13.66	1888.2	1038	18.1	79.19	0.483
R3B1	14.03	1824.3	1038	20.4	73.46	0.535
R4B1	17.17	1821.2	1038	26.5	89.47	0.537
R5B2	15.89	1808.3	1038	24.8	81.11	0.548
R1C1	12.61	1856.0	1525	24.8	69.44	0.509
R2C1	13.16	1817.4	1525	18.1	68.13	0.541
R3C1	12.18	1847.3	1525	20.4	66.11	0.516

FIGURE 14 COMPRESSION VS. TIME (DRY OF OPTIMUM)

A series of typical compression vs. time curves for samples compressed from initially near optimum conditions are shown in Figure 15. Of interest is that these relations are quite similar to the ones shown in Figure 14. That is, both sets of curves show initially large compression within the first minute, followed by very little compression in subsequent minutes. DiBernardo (1979) also observed similar behavior for a laboratory compacted highly plastic clay and suggested that samples compacted dry and at optimum exhibit similar fluid continuity conditions, i.e., continuity of air voids.

The relative compression (compression at time = t divided by the total compression at time = 10 minutes) vs. time curves for dry, wet and at optimum samples are shown in Figure 16. For the two samples just discussed (C1A2, C2A3), the magnitudes of relative compression are virtually the same at each successive time plotted. It is believed that the compression of both samples was dominated by the outflow of pore air; specifically, the air voids were interconnected (Yoshimi, 1958). The magnitude of relative compression with time for an initially wet of optimum sample, C4A3, is also shown in Figure 16. In comparison, the wet-side sample exhibits the least amount of relative compression within the 10 minutes period.

DiBernardo (1979), reinforced by the previous findings of Yoshimi (1958) and Danielson (1963), reported that there was no outflow of pore water in compacted samples compressed to final saturations of 94 to 97 percent. So one might conclude that the compression of sample C4A3 did not involve an outflow of pore water and dissolved air. However, if the degree of saturation had become sufficiently high during compression, i.e., $S_r > 97$ percent, volume

FIGURE 15 COMPRESSION VS. TIME (NEAR OPTIMUM)

FIGURE 16 COMPARISON OF RELATIVE COMPRESSION FOR DRY, WET AND AT OPTIMUM SAMPLES

change may have occurred via a process involving rearrangement of soil particles at constant water content or a process governed by the outflow of air.

Compressibility Characteristics

The effects of increasing water content and degree of saturation on compressibility behavior of samples compacted to energy level A (4 passes) are shown in Figure 17. In this figure, there is a marked difference in the compressibility behavior for dry and wet-side samples, depending on the range of consolidation pressure considered. That is, in the low pressure range (20 to 300 kPa), the wet-side sample is more compressible than the dry-side sample. However, in the high pressure range (> 300 kPa), the dry-side sample is more compressible than the wet-side sample. Similar effects of increasing water content and degree of saturation on compressibility behavior for samples compacted to energy levels B (8 passes) and C (16 passes) are shown in Figures 18 and 19, respectively.

DiBernardo (1979) observed similar behavior when he studied the effect of laboratory compaction on the compressibility of a compacted highly plastic clay. Lambe (1958a, b) also observed similar behavior and explained this behavior by compacted clay structure, colloidal chemistry, and soil particle rearrangement. Hodek and Lovell (1979) explained such observations by pore size distribution and deformable aggregate theory, as follows. On the dry side, the clay aggregates are shrunken, hard and brittle. The compaction forces move these pieces around and perhaps even break

FIGURE 17 EFFECT OF MOISTURE CONTENT ON COMPRESSIBILITY (4 PASSES)

FIGURE 18 EFFECT OF MOISTURE CONTENT ON COMPRESSIBILITY (8 PASSES)

FIGURE 19 EFFECT OF MOISTURE CONTENT ON COMPRESSIBILITY (16 PASSES)

some of them, but the result is a system with a minimum volume of small pores and a maximum of large pores. In contrast, on the wet side, the clay aggregates are swollen and plastic. The compaction forces not only move the pieces close together, but also deform them to minimize the interaggregate space. The system now has few large pores and many small ones. Thus, a dry-side sample would compress less in the low pressure range due to the large intergranular forces resulting from the many well-developed capillary menisci. However, when the loading is increased, these forces are overcome, and the brittle aggregates displace into adjacent pores. The large amount of available interaggregate pore space causes a large amount of compression. On the other hand, a wet-side sample compresses more in the low pressure range due to the smaller number of menisci developed (high degree of saturation) and less in the higher pressure range due to the relatively small number of large interaggregate pores.

The effect of increasing compactive effort on the compressibility behavior of initially dry of optimum samples is shown in Figure 20. From this figure, the slope of the curves within the respective high pressure ranges becomes steeper, i.e., increasingly more negative, with decreasing compactive effort. This suggests that for dry-side samples, when the compactive effort is increased, the magnitude and frequency of the large pore mode are decreased.

The effect of increasing compactive effort on the compressibility behavior of initially wet of optimum samples is shown in Figure 21. It may be observed that when the compactive effort is increased, the compressibility behavior is not changed to as great a degree as

FIGURE 20 EFFECT OF COMPACTIVE EFFORT ON COMPRESSIBILITY (DRY OF OPTIMUM)

FIGURE 2.1 EFFECT OF COMPACTIVE EFFORT ON COMPRESSIBILITY (WET OF OPTIMUM)

the dry-side samples of Figure 20. This suggests that for wet-side sample, when the compactive effort is increased, the magnitude and frequency of the large pore mode is little affected.

Garcia-Bengochea (1978), Bhasin (1975), and DiBernardo (1979) found that the fraction of large pores for dry-side compacted clays decreases with increasing effort; however, for wet-side compacted clays, this fraction remains unchanged.

Compactive Prestress

The value of compactive prestress can be very useful in design, since the compressibility behavior of the mass will be different at embankment confining pressures above and below this value.

Table 7 lists the values of compactive prestress (P_s) and prestress ratio (P_s/P_{cp}) for each of the 24 as-compacted compressibility test samples. Each value of compactive prestress was determined from its e -log p curve with the Casagrande approximation commonly employed for determining the most probable preconsolidation pressure for saturated soils. Since the current state of art is such that very little is known of the fundamental relationships governing unsaturated compression, the use of this method is justified in a practical sense.

The values of compactive prestress are plotted at their respective values of water content and dry density in Figure 22. Also shown is the equivalent impact effort level. The 82 percent saturation curve is selected and plotted in this figure to separate wet and dry-side samples. From Figure 22, the following conclusions may be drawn with respect to the capacity for prestressing. (a) For

TABLE 7 VALUES OF COMPACTIVE PRESTRESS AND PRESTRESS RATIO

Sample No.	Compactive Prestress P_s (kPa)	Compaction Pressure P_{cp} (kPa)	Prestress Ratio P_s/P_{cp}	Degree of Saturation S_r (%)
C1A2	530	797	0.665	51.12
C2A1	440	797	0.552	70.49
C3A2	520	797	0.652	52.78
C4A1	450	797	0.565	77.80
C5A1	400	797	0.502	75.48
C1B1	750	1204	0.620	62.78
C3B1	640	1204	0.530	69.45
C4B1	610	1204	0.507	79.17
C5B1	610	1204	0.507	83.37
C1C1	800	1771	0.452	63.98
C2C1	750	1771	0.423	69.80
C4C1	650	1771	0.367	80.09
C5C1	740	1771	0.418	69.42
R1A1	510	780	0.654	63.04
R2A1	500	780	0.641	65.42
R4A1	490	780	0.628	73.98
R1B1	500	1038	0.482	76.16
R2B1	530	1038	0.511	79.19
R3B1	580	1038	0.559	73.46
R4B1	520	1038	0.501	89.47
R5B2	560	1038	0.539	81.11
R1C1	820	1525	0.538	69.44
R2C1	800	1525	0.525	68.13
R3C1	820	1525	0.538	66.11

FIGURE 22 PRESTRESS VALUES (kPa) FOR AS-COMPACTED COMPRESSIBILITY TESTS

dry of optimum conditions, $S < 82\%$: (1) At a given compactive effort level, when the water content is increased, the value of compactive prestress is decreased. (2) As the compactive effort is increased, the value of compactive prestress is also increased.

(b) For wet of optimum conditions, $S_r > 82\%$: (1) At a given compactive effort level, when the water content is increased, the value of compactive prestress remains virtually unchanged. (2) As the compactive effort is increased, the capacity for prestressing is virtually unchanged.

Prestress ratio vs. degree of saturation is plotted in Figure 23. From this figure, it is observed that the prestress ratio decreases with increasing saturation. For the partial range of saturation considered in this study, the prestress ratio is always less than one, which may indicate that not all of the energy delivered is achieving densification. In the case of the Rascal vibratory roller, most of the compactive effort is usually supplied by the vibration energy produced with an engine on the roller drum. Additional effort may be applied by the towing unit through the drawbar, but in this study the latter component is neglected. From Figure 23, the prestress ratio of the Rascal vibratory roller is higher than that of the Caterpillar tamping roller at the same degree of saturation. The compactive efforts reported herein are only nominal pressures; i.e., they are the pressures applied by the compactor during the compaction process. Therefore, the actual prestress ratios, i.e., compactive prestress/effective compactive effort, are more than likely greater than the values listed in Table 7.

FIGURE 23 PRESTRESS RATIO VS. DEGREE OF SATURATION

Soaked Compressibility

Compaction

Table 8 lists the initial compaction variables for the 47 soaked compressibility samples. The definition of each variable, e.g., water content, dry density, compaction pressure, initial degree of saturation, and initial void ratio is the same as that for the as-compacted compressibility samples. Figure 24 shows the location of each sample with respect to its initial water and dry density. Also shown in Figure 24 is the moisture-density-energy relations of St. Croix clay for the Low Engery, Standard and Modified Proctor impact levels from Terdich (1981). This figure may be useful as a quick reference for subsequent discussions.

A soaked compressibility sample designated by C2A2a would indicate: the sample was compacted by the Caterpillar equipment (C) at the second water content level (2), was the second sample (2) collected after the equipment made 4 passes (A), and was incrementally loaded and soaked at an applied pressure corresponding to an embankment height of 7.8m (a).

Two-Way Drainage — Without Pore Pressure Measurement

DiBernardo (1979) showed typical soaked compression versus log time curves as in Figure 25. Each curve has a characteristic Type I shape, as proposed by Leonards and Girault (1961). The division of consolidation into primary and secondary components was illustrated by two techniques, i.e., the usual Casagrande approximation and pore water pressure measurements. As shown in Figure 25, the amount of compression at the end of primary consolidation (R_{100}), as determined

TABLE 8 INITIAL COMPACTION VARIABLES FOR SOAKED COMPRESSIBILITY SAMPLES

Sample No.	Water Content (%)	Dry Density ρ_d (kg/m ³)	Compaction Pressure P_{cp} (kPa)	Degree of Saturation S_r (%)	Initial Void Ratio e_o
C1A1a	13.00	1918.9	797	79.26	0.459
C2A2a	13.25	1857.3	797	73.09	0.508
C5A2a	16.60	1808.2	797	84.76	0.549
C4B2a	17.04	1750.9	1204	76.46	0.599
C2C2a	12.75	1875.8	1771	72.47	0.493
C3C2a	11.22	1745.9	1771	52.05	0.604
C4C2a	17.82	1782.0	1771	87.32	0.571
R2A2a	13.96	1786.6	780	68.89	0.567
R3A2a	13.89	1776.4	780	67.50	0.576
R2B2a	13.27	1845.4	1038	71.85	0.517
R3B2a	14.54	1835.9	1038	77.52	0.525
R4B2a	16.94	1796.6	1038	85.00	0.558
R2C2a	14.26	1795.6	1525	71.36	0.559
R3C3a	14.24	1796.8	1525	71.45	0.558
C3A6b	14.43	1785.8	797	71.14	0.568
C4A3b	20.35	1704.8	797	88.72	0.642
C2B3b	16.51	1833.6	1204	87.70	0.527
C3B4b	19.08	1718.1	1204	84.82	0.630
C4B3b	15.33	1863.4	1204	85.43	0.503
C2C3b	14.26	1858.6	1771	78.81	0.507
C3C3b	11.62	1920.7	1771	71.02	0.458
C4C3b	16.78	1797.3	1771	84.20	0.558
R2A3b	12.86	1817.9	780	66.66	0.540
R3A3b	13.69	1852.1	780	74.90	0.512
R4A3b	20.07	1739.1	780	90.58	0.610
R2B6b	14.37	1864.8	1038	80.25	0.502
R3B3b	14.11	1911.6	1038	84.99	0.465
R2C3b	13.17	1894.4	1525	77.14	0.478
R4C3b	14.69	1845.5	1525	79.50	0.517
C2A5c	13.51	1869.2	797	75.98	0.498
C4A4c	16.28	1822.0	797	84.86	0.537
C5A4c	21.32	1708.4	797	93.40	0.639
C2B5c	16.47	1836.3	1204	87.85	0.525
C3B4c	18.09	1773.0	1204	87.46	0.579
C4B4c	16.90	1802.5	1204	85.50	0.553
C2C6c	14.16	1719.4	1771	63.04	0.629
C4C5c	14.76	1867.4	1771	82.82	0.499
C4C6c	16.80	1808.7	1771	85.85	0.548
R2A4c	15.71	1824.7	780	82.32	0.534
R3A5c	15.20	1836.2	780	81.06	0.525
R4A4c	15.05	1866.9	780	84.31	0.500
R2B4c	14.99	1856.9	1038	82.61	0.508

TABLE 8 (Continued)

Sample No.	Water Content (%)	Dry Density ρ_d (kg/m ³)	Compaction Pressure P_{cp} (kPa)	Degree of Saturation S_r (%)	Initial Void Ratio e_o
R3B4c	15.54	1859.5	1038	85.99	0.506
R4B4c	17.15	1809.1	1038	87.60	0.548
R2C6c	14.91	1885.1	1525	86.07	0.485
R4C1c	17.18	1800.6	1525	86.70	0.555
R5C4c	17.97	1773.2	1525	86.92	0.579

FIGURE 24 INITIAL MOISTURE-DENSITY-ENERGY CURVES FOR SOAKED COMPRESSIBILITY TESTS

FIGURE 25 COMPARISON OF METHODS FOR OBTAINING R_{100} AND t_{100} (AFTER DIBERNARDO, 1979)

by two methods is nearly identical. Additionally, the time for full dissipation of pore pressure to occur (t_{100} value at $\Delta u = 0$) corresponds well with the Casagrande t_{100} value; however, typically the former time was 20 percent less. Because the Casagrande construction provides a very good approximation for the end of primary consolidation, DiBernardo (1979) suggested doubly-drained oedometer tests be used without measuring pore water pressure for routine testing. Such a procedure will considerably reduce the duration of each test; however, a certain amount of time is still needed before each R_{100} can be determined by the Casagrande construction.

Volume Change on Saturation

Testing the compacted material in the as-compacted condition gives an indication of the expected compressibility behavior of that soil prior to modification by the environment. In order to get some measure of the effects of a change in the service environment, as-compacted samples were loaded to simulate different levels of embankment confining stresses and saturated by first soaking the samples and then applying back pressure saturation. The one-dimensional percent volume changes ($\Delta V/V_o$) which occurred on incremental loading and wetting will be examined hereafter.

($\Delta V/V_o = \Delta e/(1 + e_o)$, where Δe = change in void ratio on saturation, and e_o = initial void ratio).

The relationship between percent volume change on wetting ($\Delta V/V_o$), initial void ratio (e_o), and the sustained load on saturation (P_o) is shown in Figure 26. From this figure, it is observed that for a given sustained load, the percent increase in volume

FIGURE 26 PERCENT VOLUME CHANGE UPON SATURATION VS. INITIAL VOID RATIO

increases with decreasing initial void ratio, and the percent decrease in volume increases with increasing initial void ratio. Abeyesekera (1978) and DiBernardo (1979) obtained similar results for a compacted shale and a compacted highly plastic clay, respectively.

The percent volume change upon saturation versus confining stress is shown in Figure 27. From this figure, it is observed that most samples increase in volume when saturated under a confining stress of 161 kPa. This stress is approximately equivalent to a depth of cover of 7.6 m in a compacted earthen embankment. The higher confining stresses of 322 and 483 kPa corresponded to depths of cover 15.2 and 22.9 m, respectively. At these stresses, most samples displayed a tendency for slight volume reduction, except for the near optimum samples compacted with the energy level C (16 passes). These showed a significant decrease in volume, which might be termed a "collapse". Such large decreases occurred quickly upon exposure to water. The volume changes for all the samples were essentially completed while the samples were soaking. There was little additional volume change (0.05% to 0.10%) during back pressure saturation.

The introduction of water to an as-compacted sample affects the clay on both the micro and macro-scale. On the micro-scale, water coming into contact with the clay minerals can result in significant swelling of the clay. The amount of swell depends on the degree of hydration of the clay minerals present and the initial moisture condition of the clay. At the macro-scale, the expanding

FIGURE 27 PERCENT VOLUME CHANGE UPON SATURATION VS. CONFINING STRESS

clay minerals result in a softening of the intact clay aggregates, which may result in greater deformations and consequently an increased compressibility under load. The addition of water to a partially saturated clay will decrease the negative pore water pressure, and the subsequent decrease in effective stress should permit swelling. The volume change resulting from any addition of water to a compacted clay can be seen to be the combination of several processes. Whether or not there is a net volume increase or decrease depends on the initial moisture content, the dry density and the applied confining stresses.

At the low confining pressure of 161 kPa , most samples compacted near optimum or dry of optimum swell when saturated (Figure 27). Apparently the swelling pressure from the hydrating clay minerals, in conjunction with the reduced effective stress due to saturation, exceeded the confining pressure and resulted in a volume increase. The confining pressures of 322 and 482 kPa were sufficient to overcome the swelling tendency for all samples.

The sample compacted with the energy level C (16 passes) to near optimum condition collapsed when saturated under a confining stress of 483 kPa. The collapse behavior seems to be related to the compactive prestress which has been induced in the sample. As previously discussed, near the optimum the compacted structure is open, with large pores between the intact clay aggregates. The introduction of water to the compacted samples reduces the effective stress and softens the clay aggregates. The shear resistance at an aggregate contact will be reduced as the aggregate softens. If the

confining stress is low, the samples may swell, as was observed for most dry-of-optimum samples with a confining stress of 161 kPa. The tendency for swell is counteracted as the confining stress is increased. There is a critical confining stress for each initial sample condition, at which no volume change occurs during saturation. At confining stresses greater than the critical stress, volume reduction occurred upon saturation. Mishu (1963) observed that the amount of collapse of a compacted soil reaches a maximum at some level of confining stress and then decreases at higher confining stresses. The maximum collapse, and the confining stress required for the collapse to occur, both increase with increasing compactive effort. This behavior was also observed by DiBernardo (1979) for a highly plastic clay.

The collapse behavior may be related to the compactive prestress induced in a sample. The induced prestress is the level of stress below which a sample compresses little when loaded. Introducing water will soften aggregate contacts and reduce resistance. Consequently, clay aggregates tend to rearrange, resulting in a volume reduction. When the confining stress is near the prestress, the soil skeleton will not have compressed much. However, any additional stress will exceed the prestress level, and result in a large volume reduction.

Compressibility Characteristics

The effect of confinement on soaked compressibility behavior for dry-of-optimum samples is shown in Figure 28. For each sample, the stress-deformation behavior is described as follows: a dashed

FIGURE 28 EFFECT OF CONFINEMENT ON SOAKED COMPRESSIBILITY BEHAVIOR (DRY OF OPTIMUM)

line for the as-compacted loading; a solid vertical line for the volume change on wetting; a solid line for the soaked unloading and reloading. From these curves, inferences can be drawn for soaked compressibility characteristics, as-compacted compressibility characteristics, and volume change on wetting characteristics. From Figure 28, at the low confining pressure of 161 kPa, the sample R3C3a swelled on wetting. At the high confining pressures of 322 and 483 the samples R4C3b and R2C6c compressed on wetting. This is consistent with the previous discussion, i.e., at low confinement the swell pressure from the hydrating clay minerals, in conjunction with the reduced effective stress due to saturation, exceeded the confinement and resulted in a volume increase. At high confinements the confining pressures were sufficient to overcome the swelling tendency and produced a volume decrease.

The comparison between samples compacted to different initial conditions dry and wet of optimum, but loaded and saturated at the same confining pressures, is shown in Figure 29. The values of C_c (320 to 1300 kPa pressure range) for samples C3A6b and C4A3b are 0.12 and 0.13, respectively. This indicates that in the virgin compression range, the dry-side sample C3A6b is slightly less compressible than the wet-side sample C4A3b. However, these values are not very different, and may indicate similar macrostructures due to loading, wetting, and subsequent collapse.

The effect of increasing confinement on soaked compressibility for initially wet-side samples is shown in Figure 30. From this figure, for confinements of 866.8 to 1981.3 kPa, the values of C_c range between 0.12 and 0.16. This may indicate the independence of

FIGURE 29 COMPARISON OF SOAKED COMPRESSIBILITY BEHAVIOR FOR ENERGY LEVEL A (4 PASSES) SAMPLES

FIGURE 30 EFFECT OF CONFINEMENT ON SOAKED COMPRESSIBILITY BEHAVIOR (WET OF OPTIMUM)

soaked virgin compressibility on confinement, for these conditions.

The effect of increased compaction pressure on soaked compressibility behavior for initially dry of optimum samples is shown in Figure 31. Within the virgin compression region (582.0 to 2971.9 kPa), the values of C_c are 0.156 (R3A2a) and 0.140 (R3B2a), respectively. Clearly, the energy level B (8 passes) sample is less compressible in this range, which may be due to the higher rigidity of the initial soil fabric.

Statistical Correlations

A statistical technique useful in studying the relationship between a dependent variable and a set of independent variables is multiple regression. It is used to develop a linear model of independent variables that can predict, control or describe a dependent response. Using the data collected from the experimental tests, regression models were developed which could describe the response of either the as-compacted prestress or the percent volume change on saturation.

The general linear regression model is given as follows:

$$Y_i = \beta_0 + \beta_1 X_{i1} + \beta_2 X_{i2} + \dots + \beta_{p-1} X_{i, p-1} + \epsilon_i \quad (35)$$

where Y_i represents the dependent variable, the X values represent the independent variables, the β coefficients are regression parameters, and ϵ_i is the error term. The statistical form of this model is linear with $(p-1)$ independent variables.

The error term (ϵ_i) cannot actually be determined for the model, since some of the factors influencing the dependent variable are unknown or unquantified. However, an estimate of the general regress-

FIGURE 31 EFFECT OF COMPACTION PRESSURE ON SOAKED COMPRESSIBILITY BEHAVIOR (DRY OF OPTIMUM)

ion model can be developed if certain basic assumptions are made with respect to the error term, and if the regression coefficients are selected such that the sum of squared residuals ($\sum(Y_i - \hat{Y}_i)^2$) is minimized. The resulting estimated response function can be written as:

$$\hat{Y}_i = b_0 + b_1 X_1 + b_2 X_2 + \dots + b_{p-1} X_{p-1} \quad (36)$$

where $\hat{Y}_i$ is the estimated mean response of Y_i and $b_0, b_1, \dots, b_{p-1}$ are the estimated regression parameters. This model was used for the statistical analysis of the data in this study. Further detail on multiple regression, its discussions and limitations, can be found in Neter and Wasserman (1974).

The two dependent variables were the compactive prestress (P_s) for the as-compacted samples, and one-dimensional percent volume change on saturation ($\Delta V/V_o$) for the soaked samples. The independent variables were water content, dry density, compaction pressure, initial degree of saturation, initial void ratio, and equivalent embankment pressure on wetting (for $\Delta V/V_o$ only). The values of the dependent variable and the linear form of the independent variables used are listed in Tables 9 and 10. The independent variables investigated for inclusion into the multiple regression models included the above mentioned initial conditions, and a large number of combinations, such as the squares, square roots, cross products, inverses and base 10 logarithms of the initial conditions.

Each of the independent variables was first plotted against the appropriate dependent variable. This was done by using the SPSS procedure SCATTERGRAM. If the scatter plots showed a linear trend,

TABEL 9 COMPACTION VARIABLES FOR COMPACTIVE PRESTRESS STATISTICAL ANALYSIS

Sample No.	Compactive Prestress P_s (kPa)	Water Content $w(\%)$	Dry Density ρ_d (kg/m ³)	Compaction Pressure P_{cp} (kPa)	Plasticity Index I_p (%)	Degree of Saturation S_r (%)	Initial Void Ratio e_o
C1A2	530	12.64	1654.5	797	21.0	51.12	0.692
C2A1	440	18.37	1618.8	797	23.3	70.49	0.730
C3A2	520	13.23	1645.2	797	21.1	52.78	0.702
C4A1	450	17.36	1723.3	797	22.3	77.80	0.625
C5A1	400	17.26	1707.1	797	24.8	75.48	0.640
C1B1	750	12.47	1799.1	1204	21.0	62.78	0.556
C3B1	640	12.63	1855.3	1204	21.1	69.45	0.509
C4B1	610	17.27	1738.3	1204	22.3	79.17	0.611
C5B1	610	16.44	1804.0	1204	24.8	83.37	0.552
C1C1	800	10.65	1910.0	1771	21.0	63.98	0.466
C2C1	750	11.96	1892.4	1771	23.3	69.80	0.480
C4C1	650	15.79	1804.1	1771	22.3	80.09	0.552
C5C1	740	12.98	1837.9	1771	24.8	69.42	0.523
R1A1	510	12.61	1795.0	780	18.1	63.04	0.560
R2A1	500	12.59	1819.7	780	16.9	65.42	0.604
R4A1	490	14.06	1827.7	780	26.5	73.98	0.532
R1B1	500	15.60	1779.5	1038	24.8	76.16	0.573
R2B1	530	13.66	1888.2	1038	18.1	79.19	0.483
R3B1	580	14.03	1824.3	1038	20.4	73.46	0.535
R4B1	520	17.17	1821.2	1038	26.5	89.47	0.537
R5B2	560	15.89	1808.3	1038	24.8	81.11	0.548
R1C1	820	12.61	1856.0	1525	24.8	69.44	0.509
R2C1	800	13.16	1817.4	1525	18.1	68.13	0.541
R3C1	820	12.18	1847.3	1525	20.4	66.11	0.516

TABLE 10 COMPACTION VARIABLES FOR ONE-DIMENSIONAL PERCENT VOLUME CHANGE ON SATURATION STATISTICAL ANALYSIS

Sample No.	Percent Volume Change $\Delta V/V_o$ (%)	Water Content w(%)	Dry Density ρ_d (kg/m ³)	Compaction Pressure P_{cp} (kPa)	Degree of Saturation S_r (%)	Initial Void Ratio e_o	Embankment Pressure P_o (kPa)
C1A1a	-0.39	13.00	1918.9	797	79.26	0.459	161
C2A2a	-0.04	13.25	1857.3	797	73.09	0.508	161
C5A2a	-0.10	16.60	1808.2	797	84.76	0.549	161
C4B2a	+0.14	17.04	1750.9	1204	76.46	0.599	161
C2C2a	-0.62	12.75	1875.8	1771	72.47	0.493	161
C3C2a	0.28	11.22	1745.9	1771	52.05	0.604	161
C4C2a	0.11	17.82	1782.0	1771	87.32	0.571	161
R2A2a	0.40	13.96	1786.6	780	68.89	0.567	161
R3A2a	0.17	13.89	1776.4	780	67.50	0.576	161
R2B2a	-0.22	13.27	1845.4	1038	71.85	0.517	161
R3B2a	-0.59	14.54	1835.9	1038	77.52	0.525	161
R4B2a	-0.26	16.94	1796.6	1038	85.00	0.558	161
R2C2a	-0.10	14.26	1795.6	1525	71.36	0.559	161
R3C3a	-0.41	14.24	1796.8	1525	71.45	0.558	161
C3A6b	0.77	14.43	1785.8	797	71.14	0.568	322
C4A3b	0.79	20.35	1704.8	797	88.72	0.642	322
C2B3b	0.27	16.51	1833.6	1204	87.70	0.527	322
C3B4b	0.60	19.08	1718.1	1204	84.82	0.630	322
C4B3b	0.19	15.33	1863.4	1204	85.43	0.503	322
C2C3b	0.01	14.26	1858.6	1771	78.81	0.507	322
C3C3b	0.24	11.62	1920.7	1771	71.02	0.458	322
C4C3b	0.46	16.78	1797.3	1771	84.20	0.558	322
R2A3b	0.17	12.86	1817.9	780	66.66	0.540	322
R3A3b	0.33	13.69	1852.1	780	74.90	0.512	322
R4A3b	0.56	20.07	1739.1	780	90.58	0.610	322
R2B6b	0.14	14.37	1864.8	1038	80.25	0.502	322
R3B3b	0.14	14.11	1911.6	1038	84.99	0.465	322

TABLE 10 (continued)

Sample No.	Percent Volume Change $\Delta V/V_o$ (%)	Water Content w(%)	Dry Density ρ_d (kg/m ³)	Compaction Pressure P_{cp} (kPa)	Degree of Saturation S_r (%)	Initial Void Ratio e_o	Embankment Pressure P_o (kPa)
R2C3b	0.12	13.17	1894.4	1525	77.14	0.478	322
R4C3b	0.49	14.69	1845.5	1525	79.50	0.517	322
C2A5c	0.29	13.51	1869.2	797	75.98	0.498	483
C4A4c	0.39	16.28	1822.0	797	84.86	0.537	483
C5A4c	0.55	21.32	1708.4	797	93.40	0.639	483
C2B5c	0.26	16.47	1836.3	1204	87.85	0.525	483
C3B4c	0.65	18.09	1773.0	1204	87.46	0.579	483
C4B4c	0.45	16.90	1802.5	1204	85.50	0.553	483
C2C6c	1.55	14.16	1719.4	1771	63.04	0.629	483
C4C5c	0.12	14.76	1867.4	1771	82.82	0.499	483
C4C6c	0.35	16.80	1808.7	1771	85.85	0.548	483
R2A4c	0.26	15.71	1824.7	780	82.32	0.534	483
R3A5c	0.28	15.20	1836.2	780	81.06	0.525	483
R4A4c	0.21	15.05	1866.9	780	84.31	0.500	483
R2B4c	0.16	14.99	1856.9	1038	82.61	0.508	483
R3B4c	0.25	15.54	1859.5	1038	85.99	0.506	483
R4B4c	0.51	17.15	1809.1	1038	87.60	0.548	483
R2C6c	0.23	14.91	1885.1	1525	86.07	0.485	483
R4C1c	0.50	17.18	1800.6	1525	86.70	0.555	483
R5C4c	0.89	17.97	1773.2	1525	86.92	0.579	483

*Values were computed from $\Delta V/V_o = \Delta H/H_o = \Delta e/(1+e_o)$, where Δe = change in void ratio on saturation, e_o = initial void ratio.

(-) swell

(+) settlement

the independent variable was considered to be correlated with the dependent variable, and was selected for further investigation. Most of the computer procedures, including SCATTERGRAM, which were used for the regression analysis are included in the SPSS (Statistical Package for the Social Science) procedural programs at Purdue University.

The next step was to isolate a subset of the independent variables which were found to be correlated with the dependent variable, so that an optimal expression using as few variables as possible could be obtained. STEPWISE is a SPSS search procedure which progressively adds independent variables to the model, and may delete variables already in the model after each step, if they no longer add significantly to the model's descriptive capabilities. The program is also capable of testing for high correlation among the independent variables. Such correlated variables can then be prevented from entering the model. This option was suppressed during this analysis due to the fact that the set of independent variables was derived from combinations of the initial sample conditions.

After the subset of variables had been isolated by the STEPWISE procedure, a number of regression equations were obtained by using those variables singly and in combination. Criteria were set by which the models could be compared and the "best" model could be selected. Requirements for the overall multiple regression equation include: (1) a high coefficient of multiple determination (R^2), which indicates the amount of variation "explained" by the variables

included in the model; (2) an increase in the adjusted coefficient of multiple determination (R_a^2) with each additional independent variable entered into the model; and (3) the overall F-test at the $\alpha = 0.05$ significance level must be met.

In addition, the regression parameter (b_i) must have a relatively small confidence limit that does not cross zero. The scatter plots of the residuals vs. the independent variables must show normal constancy of variance trends. The residuals should be normally distributed random variables; that is, the residuals divided by the square root of error mean square ($e_i/\sqrt{MSE}$) must be within the range of ± 3 .

If all the above criteria were suitably met by more than one model, the final selection was based on the value of R_a^2 and the simplicity of the regression equation.

Results of Statistical Analysis

Two dependent variables, i.e., compactive prestress (P_g) and one-dimensional percent volume change on saturation ($\Delta V/V_o$), were used in regression analysis. A wide selection of possible independent variables based on the initial sample conditions was considered in the selection procedure. A number of regression equations met the criteria previously discussed for the compactive prestress. Before selecting one of these regression equations, the variables chosen by DiBernardo (1979) to predict the prestress for the laboratory compacted St. Croix clay were forced into a regression equation for the field compacted St. Croix clay. The adjusted coefficient of multiple determination (R_a^2) was 0.86.

It should be recognized that any "best" model is highly dependent on the procedure adopted, and an equivalent model may not be established given another set of procedures. In addition, regression analysis is a numerical technique that may be highly sensitive to the data to which it is applied. That is why the final selection criteria allow the program operator to apply personal judgment as to the significance of the chosen variables and the simplicity of the final regression equation. In this case, the regression equation using the same variables as those chosen for the laboratory compacted St. Croix clay was deemed to be the most straightforward.

The variables used by DiBernardo (1979) for predicting percent volume change on saturation (water content and compaction pressure) were forced into a regression equation for the field compacted St. Croix clay, with very poor results. It was found that the equivalent embankment pressure on wetting had to be included in any regression relationship in order to obtain an acceptable value of R_a^2 . This is also rational for the laboratory compacted data, and this prediction equation was rewritten to include the effect of confinement.

The final regression models selected for the field compacted soil, as well as prediction models for laboratory compacted soil based on the DiBernardo (1979) data, are given below:

(a) For Compactive Prestress, ($\hat{P}_s$):

$$\text{Laboratory: } \hat{P}_s = -343.52 - 0.00200 w^2 \cdot P_{cp} + 48.91 P_{cp}^{\frac{1}{2}} \quad (37)$$

$$\text{Field: } \hat{P}_s = -160.99 - 0.00063 w^2 \cdot P_{cp} + 27.04 P_{cp}^{\frac{1}{2}} \quad (38)$$

where $\hat{P}_s$ = estimated value of compactive prestress (kPa)

w = water content (%)

P_{cp} = compaction pressure (kPa)

$w^2 \cdot P_{cp}$ = interaction term between water content (%)
squared and compaction pressure (kPa)

$P_{cp}^{1/2}$ = the square root of compaction pressure (kPa) $^{1/2}$

(b) For 1-D Percent Volume Change on Saturation ($\Delta\hat{V}/V_o$):

$$\text{Laboratory: } \frac{\Delta\hat{V}}{V_o} = -6.50 + 1.102 e_o \cdot P_o^{1/2} - 0.00102 w \cdot P_o \quad (39)$$

$$\text{Field: } \frac{\Delta\hat{V}}{V_o} = -2.26 + 0.400 e_o \cdot P_o^{1/2} - 0.00026 w \cdot P_o \quad (40)$$

where $\Delta\hat{V}/V_o$ = estimated value of 1-D percent volume change on
wetting (%)

$e_o \cdot P_o^{1/2}$ = interaction term between initial void ratio and
the square root of equivalent embankment pressure
on wetting (kPa) $^{1/2}$

$w \cdot P_o$ = interaction term between initial water content (%)
and equivalent embankment pressure on wetting (kPa)

The pertinent statistical data for the prediction models are presented in Table 11. In compactive prestress models, the R^2 values are 0.88 and 0.87 for the laboratory and field compacted soils, respectively. In percent volume change models, the R^2 values are 0.74 and 0.79 for the laboratory and field compacted soils, respectively. These R^2 values give an indication of the amount of variation which can be explained by the models. R^2 values of 0 and 1.0 indicate that none and all of the variation is explained by the independent variables in the

TABLE 11 STATISTICAL DATA FOR PREDICTION MODELS

Compactive Prestress Models:

Laboratory	Field
$\hat{P}_s = -343.52 - 0.0020 w^2 \cdot P_{cp} + 48.91 P_{cp}^{1/2}$	$\hat{P}_s = -160.99 - 0.00063 w^2 \cdot P_{cp} + 27.04 P_{cp}^{1/2}$
$R^2 = 0.88$	$R^2 = 0.87$
$R_a^2 = 0.87$	$R_a^2 = 0.86$
Overall F-Test = 103.24	Overall F-Test = 72.3
95% Confidence Limits	95% Confidence Limits
$w^2 \cdot P_{cp} : -0.0024 \text{ to } -0.0016$	$w^2 \cdot P_{cp} : -0.00095 \text{ to } -0.00030$
$P_{cp}^{1/2} : 41.82 \text{ to } 55.99$	$P_{cp}^{1/2} : 22.25 \text{ to } 31.83$
constant: -466.80 to -219.40	constant: -299.90 to -22.07

Percent Volume Change on Saturation Models:

Laboratory	Field
$\hat{\Delta V/V}_o = -6.50 + 1.102 e_o \cdot P_o^{1/2} - 0.00102 w \cdot P_o$	$\hat{\Delta V/V}_o = -2.26 + 0.400 e_o \cdot P_o^{1/2} - 0.00026 w \cdot P_o$
$R^2 = 0.74$	$R^2 = 0.79$
$R_a^2 = 0.69$	$R_a^2 = 0.78$
Overall F-Test = 16.79	Overall F-Test = 82.13
95% Confidence Limits	95% Confidence Limits
$e_o \cdot P_o^{1/2} : 0.654 \text{ to } 1.550$	$e_o \cdot P_o^{1/2} : 0.313 \text{ to } 0.488$
$w \cdot P_o : -0.00165 \text{ to } 0.000388$	$w \cdot P_o : -0.000340 \text{ to } -0.000178$
Constant: -9.916 to -3.084	Constant: -2.716 to -1.808

model, respectively. Additionally, it can be seen from Table 11 that each model meets all of the other statistical criteria. That is, for each model, the overall F-test value is significant and the 95 percent confidence limits for each regression coefficient are small and do not cross zero. An assumption in regression analysis is that the error component or residuals are normally distributed random variables with zero mean and constant variance. Plots of the residuals versus the variables in the models can indicate whether or not this assumption is valid for the chosen models. None of the plots show a linear, conical or quadratic trend, which indicates that the constancy of variance criteria is met. Normality is also suggested since the standardized residuals lie within a range of ± 3 .

Comparison of Laboratory and Field Models

For compactive prestress, the variables used for the laboratory and field models are the same, but the coefficients for the variables are different from each other. The soils are also different, i.e., the soil used for the laboratory compaction is a highly plastic clay (CH); the soil used for the field compaction is a low to medium plastic clay (CL). See Table 3. Whether or not the difference between the laboratory and field models is statistically significant can be tested as follows.

Assume two models:

$$\begin{aligned} \text{Model 1: } \hat{P}_s = & \beta_0 + \beta_1 (w^2 \cdot P_{cp}) + \beta_2 (P_{cp}^{\frac{1}{2}}) + \alpha_0 Z + \alpha_1 Z (w^2 \cdot P_{cp}) \\ & + \alpha_2 Z (P_{cp}^{\frac{1}{2}}) + \epsilon \end{aligned}$$

$$\text{Model 2: } \hat{P}_s = \beta_0 + \beta_1 (w^2 \cdot P_{cp}) + \beta_2 (P_{cp}^{\frac{1}{2}}) + \epsilon$$

$$\text{where: } Z = \begin{cases} 1 & \text{if field} \\ 0 & \text{if laboratory} \end{cases}$$

$\beta_0, \beta_1, \beta_2, \alpha_0, \alpha_1, \alpha_2$ are parameters

ϵ is error term

w is water content (%)

P_{cp} is compaction pressure (kPa)

Using the SPSS procedure--REGRESSION, the computer results are summarized as follows:

$$\begin{aligned} \text{Model 1: } \hat{P}_s = & -343.52 - 0.0020 (w^2 \cdot P_{cp}) + 48.92 (P_{cp}^{\frac{1}{2}}) \\ & + 182.53 + 0.0014 (w^2 \cdot P_{cp}) - 21.88 (P_{cp}^{\frac{1}{2}}) \end{aligned}$$

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)
Regression	4,189,429 (SSR ₁)	5
Error	418,059 (SSE ₁)	50
Total	4,607,488 (SST)	

$$\text{Model 2: } \hat{P}_s = -291.45 - 0.0013 (w^2 \cdot P_{cp}) + 36.33 (P_{cp}^{\frac{1}{2}})$$

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)
Regression	3,903,133 (SSR ₂)	2
Error	704,355 (SSE ₂)	53
Total	4,607,488 (SST)	

Test $H_0: \alpha_0 = \alpha_1 = \alpha_2 = 0$

$H_1: \alpha_0, \alpha_1, \alpha_2$ are not all equal to zero

Alternative H_0 implies that there is no difference between the laboratory and field models; alternative H_1 implies that the laboratory model is statistically different from the field model.

The test statistic is easily constructed from the computer results.

$$F^* = \frac{(SSR_1 - SSR_2)/5-2}{SSE_1/n-6} = \frac{(4,189,429 - 3,903,133)/3}{418,059/50} = 11.41$$

Controlling the level of significance at 0.05, we require $F(1 - \alpha; 3, n - 6) = F(0.95; 3, 50) = 2.80$. Since F^* exceeds this action limit, we conclude H_1 , that there is difference between the laboratory and field models. (See Neter and Wasserman, 1974).

For one-dimensional percent volume change on wetting, the variables used for the modified laboratory and the field models are also the same, but the coefficients for the variables are different from each other. The soils are different, i.e., the soil used for the laboratory compaction is a highly plastic clay (CH); the soil used for the field compaction is a low to medium plastic clay (CL). Whether or not the difference between the modified laboratory and field models is statistically significant can be tested as follows.

Assume two models:

$$\text{Model 1: } \hat{\Delta V}/V_o = \beta_o + \beta_1 (e_o \cdot P_o^{1/2}) + \beta_2 (w \cdot P_o) + \alpha_o Z + \alpha_1 Z (e_o \cdot P_o^{1/2}) + \alpha_2 Z (w \cdot P_o) + \epsilon$$

$$\text{Model 2: } \hat{\Delta V}/V_o = \beta_o + \beta_1 (e_o \cdot P_o^{1/2}) + \beta_2 (w \cdot P_o) + \epsilon$$

$$\text{where: } Z = \begin{cases} 1 & \text{if field} \\ 0 & \text{if laboratory} \end{cases}$$

$\beta_o, \beta_1, \beta_2, \alpha_o, \alpha_1, \alpha_2$ are parameters

ϵ is error term

e_o is initial void ratio

P_o is equivalent embankment pressure on wetting (kPa)

Using the SPSS procedure - REGRESSION, the computer results are summarized as follows:

$$\begin{aligned} \text{Model 1: } \hat{\Delta V}/V_o &= -6.50 + 1.102 (e_o \cdot P_o^{\frac{1}{2}}) - 0.00102 (w \cdot P_o) \\ &+ 4.24 - 0.702 (e_o \cdot P_o^{\frac{1}{2}}) + 0.00076 (w \cdot P_o) \end{aligned}$$

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)
Regression	142.83251 (SSR_1)	5
Error	41.47688 (SSE_1)	56
Total	184.30939 (SST)	

$$\text{Model 2: } \hat{\Delta V}/V_o = -4.04 + 0.76 (e_o \cdot P_o^{\frac{1}{2}}) + 0.00062 (w \cdot P_o)$$

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)
Regression	125.94705 (SSR_2)	2
Error	58.36233 (SSE_2)	59
Total	184.30938 (SST)	

$$\text{Test } H_o: \alpha_o = \alpha_1 = \alpha_2 = 0$$

$$H_i: \alpha_o, \alpha_1, \alpha_2 \text{ are not all equal to zero}$$

Alternative H_o implies that there is no difference between the modified laboratory and the field models; alternative H_i implies that the modified laboratory model is statistically different from the field model.

The test statistic is easily constructed from the computer results.

$$F^* = \frac{(SSR_1 - SSR_2)/5-2}{SSE_1/n-6} = \frac{142.83251 - 125.94705}{41.47688/56} = 22.80$$

Controlling the level of significance at 0.05, we require $F(1-\alpha; 3,$

$n-6) = F(0.95; 3, 56) = 2.78$. Since F^* exceeds this action limit, we conclude H_1 , that there is difference between the modified laboratory and the field models.

Discussion of Prediction Models

The prediction model to estimate the compactive prestress ($\hat{P}_s$) induced by the compaction process is described in terms of compaction water content and compaction pressure. Figure 32 shows the compactive prestress regression models for the laboratory as-compacted soil and the field as-compacted soil, superimposed. From this figure, it is observed that for a given water content, the estimated compactive prestress increases with increasing compaction pressure. Similar results had been obtained by Woodsum (1951), Abeyesekera (1978) and DiBernardo (1979). If the compaction pressure is held constant, an increase in the water content will result in a lower value of compactive prestress.

Olson and Langfelder (1965) found that an increase in water content, for a given compaction pressure, increases the pore water pressure to a less negative value, and therefore reduces the capacity for prestressing. The regression coefficient for the term containing water content is a negative value. This indicates that as the water content increases, the estimated compactive prestress will decrease in accordance with the behavior observed by others (Lambe, 1961; Olson and Langfelder, 1965; and DiBernardo, 1979).

The effect of increasing compaction pressure on the compactive prestress has been discussed and is suitably illustrated in Figure 32. The effect of compaction pressure is to increase the soil skeleton rigidity. Consequently, in one-dimensional compression much of the load is supported by the skeleton, and the net result is less compression and a higher prestress value. The regression

FIGURE 32 COMPACTIVE PRESTRESS MODELS

model shows that increasing compaction pressure (P_{cp}) increases the estimated prestress value ($\hat{P}_s$). Thus, the effects of water content and compaction pressure interact to determine the estimated compressive prestress. The two variables act on a sample in different ways and must be considered in combination to determine the end result.

The percent volume change on wetting model is described in terms of the initial void ratio (e_o) and the water content (w), each in combination with the equivalent embankment pressure (P_o). In these models, for laboratory compacted soil,

$$\Delta\hat{V}/V_o = -6.50 + 1.102 e_o \cdot P_o^{1/2} - 0.00102 w \cdot P_o \quad (39)$$

and for field compacted soil,

$$\Delta\hat{V}/V_o = -2.26 + 0.400 e_o \cdot P_o^{1/2} - 0.00026 w \cdot P_o \quad (40)$$

a positive value of percent volume change indicates settlement and a negative value indicates swell when water is introduced under load. Because of the negative value of the constant term, any sample would swell if there is no embankment pressure ($P_o = 0$). The reduction in effective stress on saturation and the volume increase of the clay minerals during hydration are the causes for swelling tendency.

The effect of void ratio on estimated percent volume change with constant water content is shown in Figure 33. The variable term in the model involving the initial void ratio (e_o) and the square root of the equivalent embankment pressure ($P_o^{1/2}$) is preceded by a positive sign. This indicates that this term overcomes the swelling tendency. From Figure 33, increasing the initial void ratio reduces the swelling tendency. As the equivalent embankment

FIGURE 33 EFFECT OF VOID RATIO ON PERCENT VOLUME CHANGE MODEL (CONSTANT WATER CONTENT)

pressure increases, the percent volume change eventually becomes positive. Figure 27 shows all field compacted samples undergo volume reduction at the two higher vertical confining stresses (322 and 483 kPa).

The variable term in the model involving the water content (w) partially offsets the effect of the positive term. Figure 34 shows that increasing water content reduces the expected volume decrease on saturation. The coefficients of the variables in the models are such that the negative term never exceeds the positive term within the range of values investigated. This insures that under no conditions can sample swell exceed that of the free swell case.

The relationships shown in Figures 32, 33, and 34 are applicable for only the range of variables investigated. Table 12 gives the range in variables for application of models for field compacted soil.

Use of Prediction Models

Although the shapes of the field and laboratory compactive prestress and percent volume change on wetting curves are similar, they are not identical. However, the simple procedure of superimposing equational models, as has been done in Figures 32, 33, and 34, may prove to be a useful practical technique for predicting the field response from laboratory tests.

Showing how the laboratory-field correlation can be developed and used is an important objective of this report. These same procedures can be employed to cover additional typical soils and compaction rollers. Where the correlation between the laboratory

FIGURE 34 EFFECT OF WATER CONTENT ON PERCENT VOLUME CHANGE MODEL (CONSTANT VOID RATIO)

Table 12. Allowable Range in Variables for Application of Models

Prestress Model		Percent Volume Change on Wetting Model:	
$\hat{P}_s = 160.99 - 0.00063 w_c^2 \cdot P_c + 27.04 P_c^{1/2}$		$\Delta \hat{V}/V_o = -2.26 + 0.400 e_o \cdot P_o^{1/2} - 0.00026 w \cdot P_o$	
Compaction Pressure P_{cp} (kPa)	Initial Water Content $w(\%)$	Initial Void Ratio e_o	Confining Stress During Saturation P_o (kPa)
780 to 797	12.59 to 21.32	0.459 to 0.730	161 to 483
1038 to 1204	12.47 to 19.08	0.465 to 0.630	161 to 483
1525 to 1771	10.65 to 17.97	0.458 to 0.629	161 to 483

and field compacted soils has been accomplished, the engineer can simply take bag samples from the excavation, run laboratory compaction and compressibility tests, and make good predictions of embankment compressibility. Thus, there is an alternate to the present procedures of (1) simply assuming that the field compacted values are the same as the laboratory compacted ones, or (2) generating the field compacted parameters from test pad samples.

The experimental studies of DiBernardo (1979) and of this report clearly indicate the independent variables of compaction which control embankment compressibility. For example, settlement under self weight occurs as rapidly as the embankment is built. If there is reason to control this settlement, the prestress value should be made as large as practicable. Figure 32 shows how combinations of compaction water content and compaction pressure operate to control the prestress.

The volumetric strain upon saturation may produce important post-constructional settlements. To control these, plots like Figure 33 and Figure 34 are examined for the effects of moisture content and density (void ratio) at various embankment confinements. The predictions so generated are unlikely to be precise, but they are likely the best available at this time.

If only laboratory data are available, consider how field-compacted prestress and volumetric strain on saturation in service may be estimated. Two methods of prediction are presented. Both apply for cases where the soils and compactors are similar to those studied in this report.

Method A applies intuitive judgements to the data previously described. The Atterberg limits are helpful in defining the similarity of soils, as are typical compaction curves. Compaction pressures can be estimated from manufacturer's data for the rollers to be employed. Estimates of compaction water content and density (void ratio) are used to enter Figures 32, 33 and 34. Predictions of prestress and volumetric strain are extracted and subjected to a final adjustment to account for soil differences.

Method B uses a statistical method to adjust for soil differences between the case in question and the soils for which the experimental relations (Table 11) were generated.

Recall that the two prediction models of compactive prestress are:

for laboratory compacted samples

$$\hat{P}_s (\text{Lab}) = -343.52 - 0.0020 w^2 \cdot P_{cp} + 48.91 P_{cp}^{1/2} \quad (37)$$

$$\text{and } I_p (\text{Lab}) = 32.0;$$

for field compacted samples

$$\hat{P}_s (\text{Field}) = -160.99 - 0.00063 w^2 \cdot P_{cp} + 27.04 P_{cp}^{1/2} \quad (38)$$

$$\text{and } I_p (\text{Field}) = 16.4 \text{ to } 29.0.$$

Let IR be the ratio of field soil plasticity index to laboratory soil plasticity index, i.e., $IR = I_p (\text{Field}) / I_p (\text{Lab})$. Assume the correlation between the field response $\hat{P}_s (\text{Field})$ and the laboratory response $\hat{P}_s (\text{Lab})$ is:

$$\begin{aligned} \hat{P}_s (\text{Field}) &= f(\text{soil type}, \hat{P}_s (\text{Lab})) \text{ or} \\ &= f(IR, \hat{P}_s (\text{Lab})) \end{aligned}$$

Twenty-four combinations of water content, compaction pressure and plasticity index were selected from within the allowable range of values, and substituted into equation (37) to generate 24 values of $\hat{P}_s$ (Lab). Twenty-four values of $\hat{P}_s$ (Field) had been measured in the as-compacted compressibility tests. Accordingly, a correlation between the field response and the laboratory response was generated by the SPSS procedure (REGRESSION),

$$\begin{aligned}\hat{P}_s \text{ (Field)} &= 95.42 + 75.60 \text{ IR} + 0.5470 \hat{P}_s \text{ (Lab)} \\ R^2 &= 0.80\end{aligned}\quad (41)$$

Thus, with known ratios of field soil plasticity index to laboratory soil plasticity index (IR), the field response $\hat{P}_s$ (Field) can be predicted.

Also recall that the two prediction models of percent volume change on wetting are:

- (1) for laboratory compacted samples

$$\Delta\hat{V}/V_0 \text{ (Lab)} = -6.50 + 1.102 e_o \cdot P_o^{1/2} - 0.00102 w \cdot P_o \quad (39)$$

$$\text{and } I_p \text{ (Lab)} = 32.0;$$

- (2) for field compacted samples

$$\Delta\hat{V}/V_0 \text{ (Field)} = -2.26 + 0.400 e_o \cdot P_o^{1/2} - 0.00026 w \cdot P_o \quad (40)$$

$$\text{and } I_p \text{ (Field)} = 16.4 \text{ to } 29.0$$

Thirty-seven combinations of plasticity index, water content, initial void ratio and vertical confining stress (within the allowable range of values) were substituted into equation (39), generating

thirty-seven values of $\Delta\hat{V}/V_o$ (Lab). The same number of values of $\Delta\hat{V}/V_o$ (Field) had been measured in the saturated compressibility tests. The correlation between the field response $\Delta\hat{V}/V_o$ (Field) and the laboratory response $\Delta\hat{V}/V_o$ (Lab) can be found by the SPSS procedure (REGRESSION) as follows:

$$\Delta\hat{V}/V_o \text{ (Field)} = 1.0927 - 2.75 \text{ IR} + 3.20 \text{ IR}^2 + 0.3463 \Delta\hat{V}/V_o \text{ (Lab)}$$

$$R^2 = 0.71 \quad (42)$$

Thus, with known ratios of field soil plasticity index to laboratory soil plasticity index (IR), the field response $\Delta\hat{V}/V_o$ (Field) can be predicted.

Consider an example problem, where I_p (field soil) is 22.4. Given the compaction pressure $P_{cp} = 800$ kPa and water content $w = 18\%$, the laboratory compactive prestress can be determined from the laboratory prediction model, equation (37), as $\hat{P}_s$ (Lab) = 521.46 kPa. The ratio of field soil plasticity index to laboratory soil plasticity index $IR = 22.4/32.0 = 0.70$. Thus, the field compactive prestress from equation (41), is $\hat{P}_s$ (Field) = 433 kPa.

Given the water content $w = 13\%$, initial void ratio $e_o = 0.65$ and vertical confining stress $P_o = 483$ kPa, and applying equation (40), $\Delta\hat{V}/V_o$ (Lab) = 0.362%. With a known ratio of field soil plasticity index to laboratory soil plasticity index $IR = 0.70$, and using the correlation equation (42), $\Delta\hat{V}/V_o$ (Field) = 0.86%.

The solution of equation (41) is shown in graphical form in Figure 35; equation 42 is solved in Figure 36. Dotted lines on these figures are for the example cited above.

FIGURE 35 PREDICTION OF FIELD COMPACTIVE PRESTRESS FROM LABORATORY COMPACTED SAMPLES

FIGURE 36 PREDICTION OF FIELD PERCENT VOLUME CHANGE ON WETTING FROM LABORATORY COMPACTED SAMPLES

FIGURE 36 (Continued)

FIGURE 36 (Continued)

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

This study has examined the effect of field compaction on the as-compacted compressibility, volume change on soaking, and soaked compressibility behavior of a plastic compacted clay (St. Croix). A similar study on the compressibility behavior of laboratory compacted samples of the same area soil has been made by DiBernardo (1979). Field compaction was achieved using a Caterpillar Model 825 roller and a RayGo Rascal Model 420C vibratory roller; a California kneading procedure was employed in the laboratory. The coupling of the relations for field compaction with those previously established for laboratory compaction is reported, and methods for predicting the field compactive prestress and volumetric strain on saturation from laboratory compaction data are also presented.

The experimental and statistical results of the field compaction study lead to the following conclusions.

1. The compression vs. time relations show that a large percentage of the as-compacted compression occurs within the first minute of loading.
2. Samples compacted on the dry side of the line of moisture optimums for any compactive effort are less compressible than those compacted on the wet side of the line of optimums

for loads less than the prestress level. However, the dry-side samples are more compressible than the wet-side samples when the applied load is greater than the prestress level.

3. The value of compactive prestress induced in a sample increases with increasing compaction pressure and decreases with increasing moisture content at a given compaction pressure.
4. The ratio of compactive prestress to nominal compaction pressure was less than unity for the range of partial saturation considered. This ratio tended to decrease with increasing initial degree of saturation. The ratio of the Rascal vibratory roller is higher than that of the Caterpillar tamping roller at the same initial degree of saturation.
5. Saturated compressibility may be evaluated in a double-drained oedometer without pore pressure measurements, since the Casagrande construction provides a very good approximation for the end of primary consolidation.
6. Saturating compacted samples under confinement leads to the following conclusions:
 - (a) At low confining stresses the swell pressure from the hydrating clay minerals, in conjunction with the reduced effective stress due to saturation, exceeded the effect of confinement and resulted in a volume increase. At high confinements the confining pressures were

sufficient to overcome the swelling tendency and resulted in a volume decrease.

- (b) For a given degree of saturation and vertical confining stress level on soaking, as the compaction pressure increases, the virgin compressibility, as measured by the compression index (C_c), decreases.
 - (c) Saturation of samples compacted near optimum resulted in large volume reductions at the higher confining stresses. The largest settlement upon saturation occurs when the confining stress is approximately equal to or greater than the prestress value.
7. From the study of the effect of field compaction on the as-compacted and soaked compressibility behavior, the following predictive models for compactive prestress and one-dimensional percent volume change on wetting were developed:

- (a) For Compactive Prestress, P_s (Field):

$$\hat{P}_s \text{ (Field)} = -160.99 - 0.00063 w^2 \cdot P_{cp} + 27.04 P_{cp}^{1/2};$$

$$R^2 = 0.87$$

- (b) For 1-D Percent Volume Change on Wetting, $\Delta V/V_o$ (Field):

$$\Delta \hat{V}/V_o \text{ (Field)} = -2.26 + 0.400 e_o \cdot P_o^{1/2} - 0.00026 w \cdot P_o;$$

$$R^2 = 0.79$$

where w = water content in percent, P_{cp} = nominal compaction pressure in kPa, e_o = initial void ratio, and P_o = confining stress in kPa.

8. The field compacted relationships are similar to those developed by DiBernardo (1979), involving the same variables, exponents and signs. Regression constants are different.
9. The similarity of laboratory and field compacted relationships allows the two predictions to be simply related in plots (Figures, 32, 33, and 34).
10. Predictions of field compacted relationships from laboratory tests can be accomplished for soils which are somewhat different by using $IR = I_p \text{ (Field)} / I_p \text{ (Lab)}$. I_p is plasticity index. The relevant equations are 41 and 42.

Recommendations for Future Research

1. Simple models of compacted clay behavior should be developed from the data reported herein and similar data. This will allow the practicing engineer to make reasonable predictions without extensive testing and statistical manipulation.
2. Measurements of compactive prestress under a greater range of laboratory compaction, field rolling, and environmental changes are needed to implement recommendation (1).
3. Studies are needed which produce a better estimate of the compactive work imparted by common field rollers to typical Midwestern soils. This is needed to better relate laboratory and field compaction.
4. Lateral soil stresses which remain after laboratory compaction and/or field rolling must be predicted more accurately if stability and settlement estimates of compacted fills are to be improved.

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APPENDICES

APPENDIX A
COMPRESSIBILITY DATA

TABLE A-1 AS-COMPACTED COMPRESSIBILITY TEST DATA

Applied Load (kPa)	Void Ratio				
	<u>Sample Designation</u>				
	C1 A2	C2 A1	C3 A2	C4 A1	C5 A1
14.86	0.692	0.730	0.702	0.625	0.640
22.29	0.692	0.728	0.701	0.623	0.639
34.67	0.691	0.725	0.699	0.620	0.637
49.53	0.690	0.721	0.695	0.617	0.634
76.77	0.686	0.714	0.688	0.611	0.627
113.92	0.682	0.703	0.679	0.605	0.614
173.36	0.673	0.687	0.654	0.597	0.596
260.04	0.665	0.669	0.641	0.585	0.577
383.87	0.653	0.650	0.620	0.573	0.557
582.00	0.627	0.627	0.590	0.555	0.530
866.81	0.598	0.600	0.558	0.531	0.500
1362.12	0.555	0.561	0.516	0.503	0.471
1981.28	0.512	0.521	0.480	0.474	0.447
2971.92	0.471	0.474	0.429	0.444	0.420
3962.56	0.439	0.423	0.389	0.416	0.391
5696.17	0.407	0.387	0.348	0.391	0.367

TABLE A-1 (Continued)

Applied Load (kPa)	Void Ratio <u>Sample Designation</u>				
	R1 A1	R2 A1	R4 A1	C1 B1	C3 B1
14.86	0.559	0.539	0.532	0.555	0.509
22.29	0.559	0.538	0.531	0.555	0.508
34.67	0.558	0.537	0.530	0.552	0.507
49.53	0.555	0.536	0.529	0.548	0.506
76.77	0.552	0.533	0.526	0.544	0.502
113.92	0.546	0.529	0.522	0.538	0.497
173.36	0.539	0.521	0.516	0.532	0.490
260.04	0.532	0.512	0.509	0.525	0.480
383.87	0.522	0.502	0.500	0.517	0.469
582.00	0.508	0.489	0.486	0.506	0.453
866.81	0.489	0.472	0.467	0.491	0.436
1362.12	0.453	0.447	0.441	0.470	0.413
1981.28	0.417	0.425	0.421	0.451	0.394
2971.92	0.380	0.399	0.396	0.423	0.371
3962.56	0.353	0.379	0.369	0.399	0.349
5696.17	0.329	0.357	0.343	0.367	0.327

TABLE A-1 (Continued)

Applied Load (kPa)	Void Ratio <u>Sample Designation</u>				
	C4 B1	C5 B1	R1 B1	R2 B1	R3 B1
14.86	0.611	0.551	0.573	0.483	0.534
22.29	0.609	0.551	0.573	0.482	0.534
34.67	0.606	0.549	0.572	0.481	0.533
49.53	0.602	0.547	0.571	0.480	0.532
76.77	0.588	0.544	0.569	0.479	0.530
113.92	0.576	0.541	0.566	0.477	0.527
173.36	0.567	0.536	0.561	0.473	0.521
260.14	0.558	0.531	0.553	0.468	0.515
383.87	0.548	0.524	0.543	0.460	0.507
582.00	0.534	0.513	0.528	0.449	0.496
866.81	0.515	0.502	0.510	0.439	0.431
1362.12	0.494	0.486	0.486	0.421	0.456
1981.28	0.473	0.467	0.466	0.406	0.431
2971.92	0.446	0.445	0.444	0.386	0.403
3962.56	0.414	0.425	0.422	0.368	0.382
5696.17	0.391	0.403	0.400	0.346	0.362

TABLE A-1 (Continued)

Applied Load (kPa)	Void Ratio				
	<u>Sample Designation</u>				
	R4 B1	R5 B2	C1 C1	C2 C1	C4 C1
14.86	0.537	0.548	0.466	0.479	0.552
22.29	0.536	0.546	0.465	0.479	0.551
34.67	0.533	0.544	0.464	0.479	0.551
49.53	0.531	0.541	0.462	0.478	0.550
76.77	0.529	0.537	0.458	0.477	0.547
113.92	0.525	0.532	0.452	0.475	0.543
173.36	0.520	0.524	0.444	0.471	0.537
260.04	0.514	0.516	0.431	0.466	0.529
383.87	0.505	0.504	0.422	0.459	0.519
582.00	0.491	0.487	0.410	0.447	0.504
866.81	0.476	0.469	0.397	0.436	0.483
1362.12	0.457	0.448	0.378	0.418	0.462
1981.28	0.439	0.421	0.361	0.401	0.442
2971.92	0.414	0.391	0.338	0.381	0.417
3962.56	0.380	0.372	0.317	0.361	0.389
5696.17	0.354	-	0.297	0.343	0.365

TABLE A-1 (Continued)

Applied Load (kPa)	Void Ratio <u>Sample Designation</u>			
	C5 C1	R1 C1	R2 C1	R3 C1
14.86	0.523	0.508	0.540	0.515
22.29	0.523	0.508	0.540	0.514
34.67	0.522	0.507	0.539	0.512
49.53	0.520	0.506	0.537	0.511
76.77	0.518	0.504	0.534	0.508
113.92	0.514	0.502	0.531	0.505
173.36	0.508	0.499	0.527	0.501
260.04	0.500	0.496	0.523	0.497
383.87	0.491	0.492	0.517	0.491
582.00	0.480	0.487	0.507	0.482
866.81	0.466	0.478	0.493	0.466
1362.12	0.446	0.462	0.472	0.438
1981.28	0.428	0.442	0.450	0.408
2971.92	0.404	0.415	0.424	0.374
3962.56	0.387	0.396	0.404	0.350
5696.17	0.367	0.376	0.382	0.330

TABLE A-2 SOAKED COMPRESSIBILITY TEST DATA

Applied Load (kPa)	Void Ratio <u>Sample Designation</u>				
	C1 A1a	C2 A2a	C5 A2a	C4 B2a	C2 C2a
9.80	0.459	0.508	0.549	0.599	0.493
14.86	0.457	0.508	0.547	0.585	0.492
22.29	0.457	0.508	0.545	0.584	0.490
34.67	0.457	0.507	0.543	0.582	0.488
49.53	0.456	0.506	0.540	0.580	0.485
76.77	0.454	0.504	0.537	0.576	0.482
113.92	0.452	0.501	0.533	0.571	0.478
161.00	0.451	0.496	0.529	0.564	0.474
161.00	0.456	0.497	0.530	0.562	0.483
89.20	0.467	0.529	0.534	0.565	0.489
113.92	0.466	0.523	0.533	0.564	0.486
173.36	0.461	0.510	0.530	0.560	0.483
260.04	0.456	0.497	0.525	0.555	0.478
383.87	0.450	0.479	0.512	0.541	0.470
582.00	0.440	0.456	0.491	0.518	0.457
866.81	0.429	0.433	0.469	0.487	0.440
1362.12	0.414	0.400	0.440	0.455	0.419
1981.28	0.400	0.379	0.417	0.427	0.400
2971.92	-	0.355	0.391	0.398	0.374

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio <u>Sample Designation</u>				
	C3 C2a	C4 C2a	R2 A2a	R3 A2a	R2 B2a
9.80	0.604	0.571	0.567	0.576	0.517
14.86	0.609	0.570	0.566	0.573	0.516
22.29	0.606	0.568	0.566	0.573	0.515
34.67	0.605	0.566	0.564	0.570	0.513
49.53	0.604	0.563	0.563	0.568	0.511
76.77	0.602	0.558	0.560	0.564	0.508
113.92	0.598	0.553	0.557	0.561	0.505
161.00	0.592	0.546	0.553	0.557	0.502
161.00	0.587	0.548	0.547	0.555	0.505
89.20	0.615	0.554	0.559	0.569	0.517
113.92	0.612	0.553	0.556	0.567	0.514
173.36	0.598	0.550	0.551	0.560	0.509
260.04	0.583	0.544	0.543	0.549	0.504
383.87	0.555	0.532	0.529	0.523	0.496
582.00	0.518	0.511	0.507	0.495	0.482
866.81	0.485	0.487	0.480	0.469	0.466
1362.12	0.449	0.459	0.452	0.438	0.437
1981.28	0.421	0.435	0.428	0.414	0.416
2971.92	0.387	0.403	0.401	0.384	0.391
3962.56	0.368	0.383	-	-	-

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio			
	<u>Sample Designation</u>			
	R3 B2a	R4 B2a	R2 C2a	R3 C3a
9.80	0.525	0.558	0.559	0.558
14.86	0.523	0.558	0.560	0.555
22.29	0.523	0.557	0.560	0.555
34.67	0.522	0.556	0.559	0.553
49.53	0.520	0.555	0.558	0.552
76.77	0.518	0.554	0.556	0.550
113.92	0.515	0.552	0.554	0.547
161.00	0.512	0.549	0.552	0.544
161.00	0.521	0.554	0.554	0.551
89.20	0.532	0.564	0.563	0.558
113.92	0.531	0.562	0.561	0.557
173.36	0.526	0.559	0.557	0.554
260.04	0.520	0.553	0.551	0.547
383.87	0.509	0.541	0.542	0.530
582.00	0.489	0.521	0.524	0.505
866.81	0.469	0.499	0.502	0.485
1362.12	0.443	0.472	0.476	0.458
1981.28	0.419	0.446	0.454	0.432
2971.92	0.390	0.419	0.427	0.403
3962.56	0.370	0.397	-	-

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio				
	<u>Sample Designation</u>				
	C3 A6b	C4 A3b	C2 B3b	C3 B4b	C4 B3b
9.80	0.568	0.642	0.527	0.630	0.503
14.86	0.567	0.640	0.537	0.634	0.503
22.29	0.566	0.639	0.536	0.633	0.503
34.67	0.565	0.637	0.535	0.631	0.502
49.53	0.563	0.635	0.534	0.629	0.501
76.77	0.561	0.630	0.531	0.623	0.498
113.92	0.557	0.622	0.527	0.615	0.495
173.36	0.553	0.613	0.521	0.603	0.490
260.04	0.548	0.602	0.514	0.591	0.482
322.00	0.545	0.595	0.510	0.584	0.478
322.00	0.533	0.583	0.506	0.575	0.475
173.36	0.539	0.586	0.512	0.584	0.480
89.20	0.557	0.595	0.538	0.616	0.500
113.92	0.558	0.594	0.534	0.612	0.498
173.36	0.553	0.593	0.526	0.602	0.491
260.04	0.547	0.589	0.518	0.588	0.484
383.87	0.537	0.582	0.508	0.575	0.476
582.00	0.517	0.562	0.496	0.554	0.463
866.81	0.496	0.535	0.478	0.519	0.445
1362.12	0.470	0.500	0.455	0.482	0.420
1981.28	0.448	0.470	0.434	0.453	0.398
2971.92	-	-	0.409	0.417	0.371

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio				
	<u>Sample Designation</u>				
	C2 C3b	C3 C3b	C4 C3b	R2 A3b	R3 A3b
9.80	0.507	0.458	0.558	0.540	0.512
14.86	0.512	0.458	0.560	0.545	0.521
22.29	0.512	0.457	0.559	0.544	0.520
34.67	0.512	0.455	0.558	0.544	0.519
49.53	0.511	0.453	0.556	0.542	0.517
76.77	0.509	0.448	0.553	0.540	0.514
113.92	0.505	0.442	0.548	0.536	0.511
173.36	0.500	0.432	0.540	0.529	0.503
260.04	0.493	0.421	0.530	0.520	0.494
322.00	0.489	0.415	0.525	0.515	0.486
322.00	0.489	0.412	0.518	0.512	0.483
173.36	0.497	0.418	0.522	0.517	0.491
89.20	0.532	0.445	0.543	0.534	0.522
113.92	0.530	0.441	0.540	0.531	0.518
173.36	0.521	0.432	0.533	0.525	0.507
260.04	0.510	0.423	0.527	0.519	0.500
383.87	0.496	0.413	0.519	0.513	0.484
582.00	0.480	0.400	0.505	0.500	0.465
866.81	0.463	0.383	0.486	0.481	0.444
1362.12	0.440	0.361	0.463	0.456	0.419
1981.28	0.418	0.341	0.442	0.437	0.395
2971.92	0.395	0.318	0.417	0.417	0.369

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio <u>Sample Designation</u>				
	R4 A3b	R2 B6b	R3 B3b	R2 C3b	R4 C3b
9.80	0.610	0.502	0.465	0.478	0.517
14.86	0.622	0.505	0.475	0.485	0.528
22.29	0.621	0.504	0.475	0.484	0.528
34.67	0.619	0.502	0.473	0.483	0.526
49.53	0.616	0.500	0.472	0.481	0.523
76.77	0.612	0.495	0.470	0.478	0.519
113.92	0.605	0.491	0.467	0.475	0.513
173.36	0.597	0.486	0.461	0.469	0.504
260.04	0.587	0.480	0.455	0.463	0.493
322.00	0.581	0.476	0.452	0.459	0.487
322.00	0.572	0.474	0.449	0.458	0.479
173.36	0.581	0.478	0.456	0.461	0.485
89.20	0.611	0.490	0.476	0.475	0.527
113.92	0.608	0.488	0.473	0.471	0.506
173.36	0.599	0.484	0.467	0.466	0.498
260.04	0.585	0.480	0.460	0.462	0.490
383.87	0.570	0.474	0.452	0.457	0.480
582.00	0.550	0.466	0.442	0.449	0.467
866.81	0.525	0.452	0.429	0.438	0.449
1362.12	0.492	0.431	0.412	0.422	0.423
1981.28	0.460	0.413	0.395	0.407	0.400
2971.92	0.423	0.397	0.374	0.389	0.372
3962.56	-	-	0.358	0.375	0.351

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio				
	<u>Sample Designation</u>				
	C2 A5c	C4 A4c	C5 A4c	C2 B5c	C3 B4c
9.80	0.498	0.537	0.639	0.525	0.579
14.86	0.513	0.546	0.644	0.532	0.584
22.29	0.513	0.545	0.643	0.532	0.584
34.67	0.512	0.545	0.642	0.531	0.583
49.53	0.510	0.544	0.639	0.530	0.582
76.77	0.507	0.542	0.636	0.527	0.579
113.92	0.502	0.539	0.631	0.524	0.575
173.36	0.494	0.534	0.624	0.518	0.569
260.04	0.484	0.527	0.615	0.509	0.560
383.87	0.474	0.519	0.603	0.498	0.550
482.90	0.466	0.513	0.595	0.492	0.543
482.90	0.462	0.507	0.587	0.488	0.533
260.04	0.467	0.511	0.596	0.494	0.536
92.90	0.497	0.533	0.636	0.519	0.565
113.92	0.495	0.532	0.629	0.518	0.564
173.36	0.489	0.529	0.626	0.514	0.559
260.04	0.481	0.523	0.615	0.504	0.551
383.87	0.471	0.515	0.602	0.497	0.541
582.00	0.460	0.506	0.585	0.487	0.530
866.81	0.435	0.490	0.560	0.469	0.510
1362.12	0.414	0.468	0.524	0.442	0.483
1981.28	0.394	0.448	0.490	0.416	0.457
2971.92	0.365	0.427	0.451	0.388	0.431
3962.56	0.346	0.409	0.421	0.365	0.411
1981.28	0.354	0.448	0.490	0.416	0.457
2971.92	0.365	0.427	0.451	0.388	0.431
3962.56	0.346	0.409	0.421	0.365	0.411
1981.28	0.354	0.411	0.430	0.376	0.416
866.81	0.367	0.420	0.456	0.395	0.429
383.87	0.383	0.431	0.491	0.415	0.446
173.36	0.404	0.455	0.524	0.440	0.471
123.80	0.416	0.468	0.548	0.454	0.488

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio				
	<u>Sample Designation</u>				
	C4 B4c	C2 C6c	C4 C5c	C4 C6c	R2 A4c
9.80	0.553	0.629	0.499	0.548	0.534
14.86	0.562	0.646	0.505	0.553	0.535
22.29	0.561	0.646	0.505	0.553	0.534
34.67	0.560	0.645	0.505	0.552	0.533
49.53	0.559	0.643	0.505	0.550	0.531
76.77	0.554	0.639	0.504	0.548	0.528
113.92	0.550	0.633	0.503	0.544	0.525
173.36	0.539	0.622	0.500	0.538	0.519
260.04	0.530	0.606	0.496	0.529	0.512
383.87	0.518	0.585	0.489	0.517	0.504
482.90	0.510	0.570	0.484	0.509	0.498
482.90	0.503	0.546	0.482	0.503	0.495
260.04	0.510	0.556	0.487	0.508	0.498
92.90	0.538	0.618	0.514	0.533	0.518
113.92	0.537	0.616	0.513	0.532	0.515
173.36	0.528	0.604	0.508	0.527	0.510
260.04	0.524	0.586	0.502	0.520	0.505
383.87	0.513	0.566	0.493	0.510	0.500
582.00	0.500	0.544	0.482	0.499	0.493
866.81	0.478	0.512	0.468	0.478	0.478
1362.12	0.451	0.473	0.449	0.450	0.457
1981.28	0.427	0.440	0.431	0.426	0.436
2971.92	0.399	0.408	0.411	0.399	0.415
3962.56	0.377	0.386	0.395	0.379	0.397
1981.28	0.383	0.394	0.396	0.384	0.400
866.81	0.399	0.415	0.407	0.395	0.410
383.87	0.423	0.441	0.421	0.411	0.419
173.36	0.446	0.481	0.445	0.435	0.433
123.80	0.463	0.500	0.455	0.445	0.442

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio <u>Sample Designation</u>			
	R3 A5c	R4 A4c	R2 B4c	R3 B4c
9.80	0.525	0.500	0.508	0.506
14.86	0.535	0.520	0.511	0.512
22.29	0.535	0.519	0.510	0.512
34.67	0.533	0.518	0.509	0.511
49.53	0.531	0.516	0.508	0.510
76.11	0.526	0.513	0.506	0.507
113.92	0.520	0.509	0.503	0.503
173.36	0.510	0.503	0.498	0.498
260.04	0.500	0.495	0.492	0.490
383.87	0.489	0.484	0.485	0.481
482.90	0.482	0.477	0.481	0.475
482.90	0.478	0.474	0.473	0.471
260.04	0.483	0.480	0.482	0.477
92.90	0.508	0.516	0.498	0.501
113.92	0.506	0.515	0.497	0.499
173.36	0.500	0.509	0.493	0.495
260.04	0.493	0.500	0.489	0.489
383.87	0.485	0.488	0.483	0.480
522.00	0.476	0.474	0.476	0.469
866.81	0.460	0.457	0.464	0.454
1362.12	0.435	0.434	0.444	0.432
1981.28	0.413	0.410	0.424	0.409
2971.92	0.387	0.383	0.401	0.384
3962.56	0.367	0.362	0.382	0.364
1981.28	0.371	0.369	0.388	0.370
866.81	0.382	0.389	0.398	0.385
383.87	0.395	0.411	0.409	0.406
173.36	0.416	0.438	0.423	0.427
123.80	0.429	0.455	0.432	0.441

TABLE A-2 (Continued)

Applied Load (kPa)	Void Ratio <u>Sample Designation</u>			
	R4 B4c	R2 C6c	R4 C1c	R5 C4c
9.80	0.548	0.485	0.555	0.579
14.86	0.550	0.488	0.569	0.586
22.29	0.550	0.488	0.568	0.585
34.67	0.549	0.487	0.566	0.584
49.53	0.547	0.486	0.564	0.582
76.77	0.545	0.485	0.561	0.579
113.92	0.537	0.482	0.557	0.574
173.36	0.531	0.478	0.551	0.568
260.04	0.523	0.472	0.543	0.560
383.87	0.512	0.466	0.533	0.548
482.90	0.504	0.461	0.526	0.541
482.90	0.496	0.458	0.518	0.527
260.04	0.502	0.461	0.527	0.534
92.90	0.530	0.477	0.559	0.561
113.92	0.529	0.476	0.559	0.561
173.36	0.524	0.473	0.554	0.556
260.04	0.515	0.468	0.544	0.547
383.87	0.505	0.463	0.531	0.536
582.00	0.492	0.456	0.516	0.523
866.81	0.472	0.445	0.496	0.503
1362.12	0.443	0.425	0.467	0.476
1981.28	0.516	0.408	0.439	0.450
2971.92	0.386	0.387	0.409	0.423
3962.56	0.363	0.371	0.385	0.403
1981.28	0.371	0.377	-	-
866.81	0.386	0.387	-	-
383.87	0.409	0.399	-	-
173.36	0.434	0.415	-	-
123.80	0.453	0.422	-	-

APPENDIX B

LIST OF NEGATIVE NUMBERS FOR PHOTOGRAPHS

Figure No.	Negative No.
7	81189
8	81190
9	77634-14
10	77634-5

COVER DESIGN BY ALDO GIORGINI