An investigation of consolidation on compacted partially saturated Clarksville soil

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AN INVESTIGATION OF CONSOLIDATION ON COMPACTED 
PARTIALLY SATURATED CLARKSVILLE SOIL 

BY 
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A 
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The Terzaghi theory of consolidation and the assumption of his theory, pore pressure and effective stress, secondary compression and rheology were all reviewed and discussed. Eight compacted unsaturated samples made from Clarksville silty clay (CL) under different compaction efforts and moisture contents were subjected to consolidation load increments. Each load increment was maintained for 24 hours. An extra compacted sample was loaded for one month to investigate secondary compression.

From e-p curves, five settlement vs. compacted embankment height curves were derived for different compaction effort and moisture contents. These curves may be of practical use, when checked by field measurements and observations.
ACKNOWLEDGMENT

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Special recognition is due to my fiancée Ada and my Chinese roommates. Their encouragement and understanding have made this paper possible.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>11</td>
</tr>
<tr>
<td>ACKNOWLEDGMENT</td>
<td>iii</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>v</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>vi</td>
</tr>
<tr>
<td>I. INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>II. REVIEW OF LITERATURE</td>
<td>3</td>
</tr>
<tr>
<td>III. MATERIAL, EQUIPMENT AND PROCEDURES</td>
<td>12</td>
</tr>
<tr>
<td>A. MATERIAL</td>
<td>12</td>
</tr>
<tr>
<td>B. EQUIPMENT</td>
<td>12</td>
</tr>
<tr>
<td>C. PROCEDURES</td>
<td>12</td>
</tr>
<tr>
<td>IV. EXPERIMENTAL RESULTS</td>
<td>16</td>
</tr>
<tr>
<td>V. DISCUSSION OF RESULTS</td>
<td>18</td>
</tr>
<tr>
<td>VI. CONCLUSIONS AND RECOMMENDATIONS</td>
<td>28</td>
</tr>
<tr>
<td>BIBLIOGRAPHY</td>
<td>31</td>
</tr>
<tr>
<td>APPENDIX</td>
<td>33</td>
</tr>
<tr>
<td>VITA</td>
<td>111</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>CONCEPT OF INTERGRANULAR OR EFFECTIVE STRESS ON AREA</td>
<td>34</td>
</tr>
<tr>
<td>2.</td>
<td>EFFECTS OF COMPACTION ON STRUCTURE</td>
<td>35</td>
</tr>
<tr>
<td>3.</td>
<td>EFFECT OF ONE-DIMENSIONAL COMPRESSION ON STRUCTURE</td>
<td>35</td>
</tr>
<tr>
<td>4.</td>
<td>GRAIN SIZE DISTRIBUTION CURVE</td>
<td>36</td>
</tr>
<tr>
<td>5.</td>
<td>CONSOLIDATION APPARATUS</td>
<td>37</td>
</tr>
<tr>
<td>6.</td>
<td>COMPACTION CURVES AT VARIOUS COMPACTIVE EFFORTS FOR CLARKSVILLE SILTY CLAY</td>
<td>38</td>
</tr>
<tr>
<td>7-12.</td>
<td>TIME-CONSOLIDATION CURVES FOR CS 1-1 TO CS 1-6</td>
<td>39</td>
</tr>
<tr>
<td>13-19.</td>
<td>TIME-CONSOLIDATION CURVES FOR CS 2-1 TO CS 2-7</td>
<td>45</td>
</tr>
<tr>
<td>20-25.</td>
<td>TIME-CONSOLIDATION CURVES FOR CS 3-1 TO CS 3-6</td>
<td>52</td>
</tr>
<tr>
<td>26-31.</td>
<td>TIME-CONSOLIDATION CURVES FOR CS 4-1 TO CS 4-6</td>
<td>58</td>
</tr>
<tr>
<td>32-37.</td>
<td>TIME-CONSOLIDATION CURVES FOR CS 5-1 TO CS 5-6</td>
<td>64</td>
</tr>
<tr>
<td>38-43.</td>
<td>TIME-CONSOLIDATION CURVES FOR CS 6-1 TO CS 6-6</td>
<td>70</td>
</tr>
<tr>
<td>44-49.</td>
<td>TIME-CONSOLIDATION CURVES FOR CS 7-1 TO CS 7-6</td>
<td>76</td>
</tr>
<tr>
<td>50-55.</td>
<td>TIME-CONSOLIDATION CURVES FOR CS 8-1 TO CS 8-6</td>
<td>82</td>
</tr>
<tr>
<td>56-63.</td>
<td>CONSOLIDATION VS. LOG PRESSURE CURVES</td>
<td>88</td>
</tr>
<tr>
<td>64-71.</td>
<td>PRESSURE VS. CONSOLIDATION CURVES</td>
<td>96</td>
</tr>
<tr>
<td>72.</td>
<td>LONG DURATION SETTLEMENT-TIME CURVE</td>
<td>104</td>
</tr>
<tr>
<td>73.</td>
<td>COMPACTION EFFORT AND FINAL VOID RATIO RELATIONSHIP</td>
<td>105</td>
</tr>
<tr>
<td>73a.</td>
<td>MOISTURE CONTENT AND FINAL VOID RATIO RELATIONSHIP</td>
<td>106</td>
</tr>
<tr>
<td>74.</td>
<td>COMPACTION EFFORT AND COMPRESSION INDEX RELATIONSHIP</td>
<td>107</td>
</tr>
<tr>
<td>74a.</td>
<td>MOISTURE CONTENT AND COMPRESSION INDEX RELATIONSHIP</td>
<td>108</td>
</tr>
<tr>
<td>75.</td>
<td>SETTLEMENT AND EMBANKMENT HEIGHT RELATIONSHIP</td>
<td>109</td>
</tr>
<tr>
<td>75a.</td>
<td>SETTLEMENT AND EMBANKMENT HEIGHT RELATIONSHIPS</td>
<td>110</td>
</tr>
</tbody>
</table>
LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>PHYSICAL PROPERTIES OF CLARKSVILLE SOIL</td>
<td>13</td>
</tr>
<tr>
<td>II</td>
<td>VOID RATIO VALUES FOR CS1, 2, 3, 6, 8</td>
<td>24</td>
</tr>
<tr>
<td>III</td>
<td>UNIT WEIGHT VALUES FOR CS1, 2, 3, 6, 8</td>
<td>24</td>
</tr>
</tbody>
</table>
I. INTRODUCTION

Analysis of the consolidation characteristics of a soil and the interrelation between these characteristics and the settlement of soils in the field, constitute one of the most complex problems that faces the soils engineer. In any construction project where soil is to be used as a construction material, such as in embankments, or where it is to be used to support structures such as buildings, bridges, pavements, etc., then consolidation and the accompanying settlement is a problem which must be carefully analyzed.

At the present time, the methods for analyzing consolidation test data and relating this information to field conditions is basically that as developed by Terzaghi in 1925. There are assumptions in this analysis which are more thoroughly explained later in this paper. However, one of the basic assumptions is that the soil is completely saturated and that the rate of flow of water out of the soil pores controls the rate of consolidation of the soil.

Soils which are manipulated in some manner before they are used as a construction material are very seldom placed in their new location at saturation moisture contents. The moisture content at which they are placed is usually close to a predetermined optimum moisture for a maximum density under a given compactive effort and this is always below saturation. When an embankment is made up of layers of soil compacted at a controlled moisture content below saturation, there will still be settlement under the influence of applied load. The question to be answered by the engineer is how much settlement will occur and what time elapse is necessary for this settlement to take place. Neither
of these questions can be answered with exactitude from the theory presented by Terzaghi which is used successfully for saturated soils.

This investigation consists of evaluating the consolidation characteristics of a soil compacted at different compactive energies and moisture contents below saturation to aid in formulating concepts concerning the mechanism of consolidation of such soils.
Consolidation in a general sense may be defined as any volume decrease of a soil mass. \(^{(1)}\) Scott states that "the process of transient flow of water (or other fluids) through a soil structure which compresses or expands in time is called consolidation in soil mechanics." \(^{(2)}\)

In order to predict the settlements of structures resulting from the consolidation of soils, a method of extrapolating laboratory test results and relating them to field conditions is needed. The method commonly used is one proposed by K. V. Terzaghi. In 1925, Terzaghi published his fundamental approach to settlement analysis in his book "Erbaumechanik." \(^{(3)}\) A more recent treatment of consolidation theory is given in "Theorie der Setzung von Tonschichten", by K. V. Terzaghi and O. K. Frohlich, Franz Deuticke, Leipzig and Vienna, 1936. \(^{(3)}\)

Although there are some revisions of Terzaghi's Consolidation Theory by Merchant and Taylor (1949, 1942), Tan (1957), Gibson and Lo (1961), I. F. Christie (1964, 1965), \(^{(4)}\)(\(^{(5)}\)) it is still, in its original form, the basis for conventional methods of interpretation of consolidation test data and time rate predictions in settlement analysis.

The analysis of consolidation as presented by Terzaghi contains several limiting assumptions which must be considered before the degree of the validity of the approach for a particular problem is established. The assumptions are as follows: \(^{(1)}\)(\(^{(6)}\))

1. The soil being tested is homogeneous.
2. Complete saturation of the soil exists.
3. The water and the soil solids are incompressible.
(4) The action of a differential soil mass is similar to the action of a larger soil mass.

(5) One-dimensional drainage and compression take place in a soil undergoing consolidation.

(6) A linear relationship exists between pressure and void ratio.

(7) Darcy's law of fluid flow is valid.

(8) Certain soil properties are assumed to remain constant.

(9) Strain or void ratio change in the sample will be small.

(10) Temperature is constant during the consolidation process.

The assumption that the relationship between pressure and void ratio during the consolidation process is linear infers that all compression is due to drainage caused by hydrostatic excess pressure. Usually the major cause of error in applying the Terzaghi's Theory is that this assumption is not fulfilled. The error has more effect on predictions of settlement rates than it does on predictions of amounts of settlement. For this reason, several revisions, such as the Theory of Taylor and Merchant (1940), and Gibson and Lo's Theory (1961), have been made as mentioned before. The limitation to Terzaghi's Theory of analysis has been apparent since its inception. However, it is virtually the only method universally accepted at the present time.

From laboratory results of consolidation tests, two important relationships are plotted to aid in the evaluation of the soil properties. The first plotted curves are settlement versus the log of time for each load increment, called the time curves. The second is a void ratio versus the log of pressure over the range of loading used, called the e-log p curve.

According to T. W. Lambe, there are three stages in a time curve.
They are: (1) initial compression, (2) primary compression, and (3) secondary compression. Initial compression is due to the instantaneous compression of air in the voids. Primary compression is due to the drainage of pore water, because of the hydrostatic excess pressure. The secondary compression is probably due to plastic flow or gradual structural adjustment under the imposed load.

According to R. H. Karol, the void ratio versus log pressure (e-log p) curve can generally be divided into three parts. The first portion is called the reloading curve and serves to define the pressure range in which consolidation effects are relatively small. The upper pressure boundary of the reloading range is called the preconsolidation load or the pressure corresponding to the in-place void ratio. The second portion is called the virgin line which usually is a straight line with a lower pressure boundary at the preconsolidation load, and an upper pressure boundary at the load which is high enough to modify the basic soil structure. The third portion is called the remolding range, and is most noticeable in soils of flocculent structure. Loads in this range are high enough to modify the arrangement of flocs, and must not be permitted to occur in the field.

Secondary consolidation is also called plastic flow, creep, intolerable settlement, or post settlement. R. H. Karol states: "Plastic flow is a slow process relatively independent of thickness of stratum, and mainly dependent upon intergranular pressure." In the laboratory, true consolidation is almost complete before plastic flow is of any consequence. For thick strata in the field, it is probable that consolidation and plastic flow are occurring simultaneously.

According to A. R. Jumikis, in secondary compression, the colloid-
chemical process and surface phenomena such as the stressed moisture surface tension, viscosity and density changes in the stressed moisture films, molecular attractive forces between soil particles and moisture film, induced electrokinetic potential, and other processes become active. All these processes are very slow by their very nature. Therefore, the secondary consolidation process requires a long time to consolidate the soil fully in the laboratory. Secondary consolidation of mineral soils is usually negligible, but in the case of organic soils it may be considerable because of their colloidal nature. A possible disintegration of clayey soil particles and other processes probably make some contributions to secondary consolidation. J. B. Hansen, in his paper, "A model law for simultaneous primary and secondary consolidation", stated that "secondary consolidation must start as soon as an effective stress is developed and the primary and secondary consolidation processes must therefore actually proceed simultaneously from the very beginning, although of course with different time rates, because they follow different model laws."

Secondary compression, according to Taylor, is caused by remolding or the disturbance of structure by the occurrence of the primary compression. Remolding may be defined as the subjecting of a soil to shearing strains, thus moving the individual particles relative to each other and destroying the natural bond or natural structure. Soil action is too complicated to permit complete understanding of the structural aspects and the grain rearrangement processes associated with secondary compression. R. F. Scott pointed out in his book "Principles of soil mechanics" (1962), "because of the development of knowledge concerning physiochemical properties of very small mineral particles, and improved equipment and techniques (for studying particle
interaction), most soil behavior and phenomena have better explanations at the present time."

The stresses that act within a saturated mass of soil may be divided into two kinds. The term "effective pressure" has the same meaning as intergranular pressure. It refers to pressures transmitted through grain to grain contact points through a soil mass. (See Figure 1.) Such pressure, when it exists, decreases the void ratio of a soil mass and increases its shear strength; hence, the term "effective." The term neutral pressure has the same meaning as pore water pressure. It refers to hydraulic pressure in the voids of saturated soils. Since water exerts the same pressure in all directions and has no shear strength, pore water pressures do not affect the void ratio of a soil, nor do they increase the soil shear strength. Thus the notation "neutral." (6) From Hilf's research on "Pore pressure in cohesive soils" (9), water in unsaturated soil is held in the soil by the phenomenon of surface tension which occurs at the boundaries separating the water and pore air. It is known that surface tension is virtually independent of the form of the surface contact, but is a function of temperature. The rise of liquid in fine pore tubes is a result of surface tension, and is called capillarity. Since the capillary tube is open to atmospheric air, which also acts on the free water surface, the pressure at the meniscus must be equal to $-\gamma_w h$. In general, the pressure on any liquid film in contact with the atmosphere at any point can be expressed by the equation: $u_c = -T_s \left( \frac{1}{r_1} + \frac{1}{r_2} \right)$. Where $u_c$ is capillary pressure (taking atmospheric pressure = 0), $r_1$ and $r_2$ are radii of curvature and $T_s$ is the surface tension. In an unsaturated soil, the water wets the soil grains and is held in capillary spaces.
between them. Since the pore spaces in a soil mass are interconnected, at equilibrium all menisci will have the same curvature \((\frac{1}{r} + \frac{1}{r_2})\), and all water will have the same capillary pressure \(u_c\). The pressure in the air of the voids of a soil mass, which has been compressed without permitting escape of the pore fluid, can be calculated by combining Boyle's law of compressibility of air with Henry's law of solubility of air in water. The air pressure \(u_a\) can be expressed as follows.\(^9\) (Hamilton, 1939, and Hilf, 1948).\[ u_a = \frac{Pa \Delta V}{Va + hVw - \Delta V} \] Where \(Pa\) is initial pore pressure (usually considered as atmospheric pressure—absolute pressure); \(Va\) is volume of air; \(Vw\) is volume of water; \(\Delta V\) is volume change; \(h\) is the coefficient of solubility of air in water by volume. The pressure in the pore water of an unsaturated soil, which is pressure in the fluid contact with soil skeleton, is given by \(u_w = u_a + u_c\). The pore water pressure \(u_w\) in unsaturated soils can be negative whenever the positive value of \(u_a\) is smaller than the negative value of \(u_c\). In soils unstressed externally, where \(u_a\) will be zero at equilibrium, the pore water pressure is always negative, \(u_w = u_c\). It is possible for \(u_a\) to be negative, as in a sealed specimen of unsaturated soil which, during a shear test, expands to a volume greater than its volume when it was first sealed. In this case, \(u_a\) and \(u_c\) are both negative.

In the paper, "limitations to the use of effective stress in partly saturated soils", by Jennings and Burland,\(^10\), there are the following statements: Terzaghi's effective stress principle may be stated in the form of two propositions: (1) changes in volume and shearing strength of a soil are due exclusively to changes in effective stress, (2) the effective stress \(\sigma'\) in a soil is defined as the excess of the total applied stress \(\sigma\) over the pore pressure \(u\), \[ \sigma' = \sigma - u \]
The validity of the principle of effective stress for saturated soils has been adequately verified by the work of Rendulic (1936), Bishop and Eldin (1950), Henkel (1959, 1960) and Skempton (1960). For unsaturated soils $\sigma' = \sigma + \chi (u_a - u_w) - u_a$ or $\sigma' = \sigma - [\chi u_w + (1 - \chi) u_a] = \sigma - u^*$. Where $u_a$ is gas pressure, $u_w$ is water pressure, $\chi$ is a parameter, $u^*$, the quantity $\chi u_w + (1 - \chi) u_a$ may be considered as an equivalent pore pressure. $u^*$, i.e., that portion of the effective stress in a soil resulting from fluid pressures in the pores. Bishop and Donald (1961) performed a triaxial test on partly saturated silt. They found that $(\sigma_3 - u_a)$ and $(u_a - u_w)$ remained constant throughout the test. A change in $(\sigma_3 - u_a)$ or $(u_a - u_w)$ alone had a marked effect on the shape of the stress strain curve. It was therefore concluded that the form of the equation $\sigma' = \sigma + \chi (u_a - u_w) - u_a$ is correct, and the behavior of the soil is independent of the absolute values of $\sigma$, $u_w$ and $u_a$.

The definition of effective stress by Skempton (1960) is that "The effective stress is that stress controlling changes in volume or strength of the soil." According to Jennings & Burland (10), the assumption that the effective stress under conditions of externally applied pressures are directly comparable with the effective stresses under applied suction is a tacit assumption of the validity of the principle of effective stress over the whole range of partial saturation. However, some experimental data indicate that it is not $\sigma'$. 
that controls the behavior of the majority of partly saturated soils, but rather functions of the separate values of $\sigma$ and the equivalent pore pressure $u^*$. Therefore, to say that equation $\sigma' = \sigma + \chi(u^* - u) - u_e$ defines the effective stress in a partly saturated soil according to the effective stress principle is not always correct. This equation actually defines an intergranular stress; the important difference being that, whereas an effective stress is that stress controlling soil behavior, the same is not necessarily true of intergranular stress.

Holtz (1948) and Wagener (1961)(10) have obtained compression curves showing that collapse occurs in soils compacted dry of optimum moisture content, even when the densities are high. Michaels (1959) has suggested that a clay dries, and the menisci at the surface are drawn into the soil matrix, the clay structure tends to form into "Packets" of clay particles (10). T. W. Lambe (1958)(11) proposed a mechanistic theory to account for the effects of compaction on the behavior of clay. The theory attempted to explain the known properties of clay in terms of "structure" -- the arrangements of soil particles and the electrical forces between adjacent particles. The effects of compaction on structure and one-dimensional compression on structure(12) are shown in Figure 2 and 3. An important paper, "Compression of partially saturated cohesive soils", was published by Yoshimi and Osterberg (1963)(13). They compacted samples of Vicksburg silty clay on the dry side of the optimum water content and subjected them to laterally confined static compression. It was observed that:

1. There was virtually no outflow of pore water during compression while the degree of saturation increased from 70% to 90%.

2. The samples were permeable to air under a small pressure gradient at degrees of saturation exceeding 90%.
(3) For the same stress increment ratio the time rate of compressive strain was independent of sample thickness and drainage conditions.

(4) For virgin compression, the time rate of compression increased with increasing stress increment ratio.

For cohesive soil compacted on the dry side of the optimum water content they further concluded that:

(1) At the equilibrium state, the pore water carries subatmospheric pressure and tends to fill small capillaries, leaving larger pore space for air to form interconnected channels even at a very high degree of saturation.

(2) Because of stiffness of soil structure and high compressibility of soil air, application of a stress increment may not raise the initially subatmospheric pressure in the pore water above one atmosphere. In such a case, the pore water remains in the soil because it cannot flow out into the atmosphere against a pressure gradient.

(3) If the pore water movement is negligible, the time dependency of the compression is governed by the rheological characteristics of the soil structure. Thus, time rate of compressive strain is independent of drainage conditions or thickness, but is dependent on the stress increment ratio.

The writer's work is to take a series of samples of compacted, unsaturated silty clay of Clarksville soils which are subjected to one-dimensional compression. The samples are prepared on different compaction efforts and moisture contents -- both dry and wet sides of optimum moisture content.
III. MATERIAL, EQUIPMENT AND PROCEDURES

A. MATERIAL

The sample of soil used in this investigation was obtained locally from the B Horizon of the Clarksville soils predominating in this area. The soil is a reddish yellow silty clay containing large amounts of gravel.

After air drying, crushing, and passing the Number 10 U.S. Standard sieve, the soil was mixed to obtain maximum uniformity and then stored in a galvanized bin.

The general physical properties of the Clarksville soil are listed in Table I.

The grain size distribution curves are shown in Figure 4. The soil has been classified according to the unified classification system and the AASHO classification system, the two most widely used systems of classification. The values in Table I were obtained from the Atterberg limits tests, Specific gravity test, Grain size analysis, and Standard Proctor Compaction tests.

B. EQUIPMENT

A Lancaster Grinding mixer was used to crush the air dry soil for passing the No. 10 sieve. It was also used to uniformly mix the water in the soil for the compaction tests. An automatic Proctor type compaction device was used for compaction tests. For the one-dimensional compression test, the fixed-ring type consolidometer and lever system type loading frame apparatus was used (See Figure 5).

C. PROCEDURES

A systematic procedure for the preparation, compaction and
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<thead>
<tr>
<th>CHARACTERISTIC</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Material Passing #10 Sieve)</td>
<td></td>
</tr>
<tr>
<td>Air dry M. C.</td>
<td>3.65% (2.93-4.41)</td>
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<tr>
<td>Liquid Limit</td>
<td>37.8 %</td>
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<tr>
<td>Plastic Limit</td>
<td>17.6 %</td>
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<tr>
<td>Plastic Index</td>
<td>20.2</td>
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<tr>
<td>Shrinkage Limit</td>
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<td>Shrinkage Ratio</td>
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<tr>
<td>Specific Gravity</td>
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<tr>
<td>Unified Classification</td>
<td>CL</td>
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<tr>
<td>AASHO Classification</td>
<td>A-6(13)</td>
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<tr>
<td>Passing No. 10 Sieve</td>
<td>100.0 %</td>
</tr>
<tr>
<td>Passing No. 40 Sieve</td>
<td>93.0 %</td>
</tr>
<tr>
<td>Passing No. 140 Sieve</td>
<td>82.0 %</td>
</tr>
<tr>
<td>Passing No. 200 Sieve</td>
<td>78.0 %</td>
</tr>
<tr>
<td>Optimum Moisture Content (Standard Proctor)</td>
<td>18.3 %</td>
</tr>
<tr>
<td>Maximum Dry Density (Standard Proctor)</td>
<td>105.0 pcf</td>
</tr>
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</table>
consolidation of the soil samples was established and followed. The whole process consists of four steps,

(1) obtaining the sample,

(2) general physical property tests,

(3) compaction tests,

(4) one-dimensional compression tests.

The first and second steps have been previously discussed in Section A. In the third step, compactive efforts of 20, 25, 30, 35, and 40 blows per layer with the standard proctor hammer, for three layers were used to compact the soil and optimum moisture and maximum density for each compactive effort was determined. Eight different samples were prepared for consolidation testing, five of them (CS 1, CS 2, CS 3, CS 4, CS 5) were at approximately the same moisture content, near the optimum moisture content of 18.3% which was determined for the 25 blow/layer compaction effort, but at different compactive efforts (9900, 12375, 14850, 17325, 19800 ft-lb/cu ft.) Four of the samples (CS 6, CS 2, CS 7, CS 8) were prepared at the same compactive effort (12375 ft-lb/cu. ft.) but at different moisture contents (14.2%, 17.0%, 19.3%, 21.3%). The compacted samples were carefully trimmed into fixed-ring consolidometers, then mounted on the consolidation apparatus. An annular cotton cloth, connected to a small dish with water in it, was added to each consolidometer (as shown in Figure 5) to minimize evaporation of moisture from the sample. By placing a 10 Kg load on the sample, the one-dimensional compression test was started. The sample size was 1.00 inch high and 2.50 inch in diameter. A load was applied of sufficient magnitude to give a pressure intensity of 0.322 TSF on the soil specimen, and time and vertical deflection readings
were taken at total elapsed times of 0, 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, 1440 minutes, respectively. On successive days, loads of 0.645, 1.29, 2.58, 5.16, 10.32 TSF were applied. After the loading sequence was completed, the apparatus was dismantled and the moisture content of the sample determined. With several samples, the loading periods on certain loads was prolonged to three or four days. There was an extra compacted sample (CS 5-1A), whose first load increment (0.322 TSF) was maintained for one month for the investigation of secondary compression (Figure 72).
IV. EXPERIMENTAL RESULTS

A. General Physical Property Tests:

2. Specific gravity test.
3. Grain size analysis.

The results of the above tests are listed in Table I and Figure 4.

B. Standard Proctor Compaction tests:

Five compaction curves for samples compacted under different compactive efforts are shown in Figure 6.

C. One-Dimensional Compression Tests:

For each load increment applied to the soil a dial reading vs. logarithm of time curve has been plotted and are shown in Figures 7 through 55. The void ratios at the end of each load increment period have been calculated and are shown as "void ratio vs. logarithm of pressure" curves (e-log p curves) in Figures 56 through 63. To eliminate the logarithmic distortion of the standard e-log p curves, the void ratios have also been plotted against pressure on a linear scale and are shown in Figures 64 through 71. Since the deformations at the end of normal loading time intervals were still continuing, it was decided to run one load for a much longer time period. Figure 72 represents a dial reading time curve for a sample compacted at a moisture content of 17.0% with a dry density of 111 pcf., compactive effort of 19,800 ft. lb. per cu. ft., and loaded with .322 TSF. The load was allowed to remain on the sample for 30 days.

D. Relationship between Settlement and Embankment Height:

From e-p curves, the settlement due to embankment's own weight
can be calculated. From these calculations, a series of curves for settlement vs. embankment height are plotted as shown in Figure 75 and 75a.
V. DISCUSSION OF RESULTS

A. Analysis of Results

The compaction test results for the soil used in this investigation are shown in Figure 6. These curves are as expected for a silty clay soil with the optimum moisture content at approximately 86% saturation for all compactive efforts.

Standard procedures were followed (M.I.T. Soil Testing for Engineers, by T. W. Lambe) during the consolidation testing. 24 hours is accepted as the normal loading period per load increment. Because of the observed continuing deformation at the end of the 24 hour period, a few load increments were allowed to remain on the specimen beyond the normal time limit to observe their continuing deformation characteristics. Figures 7 through 55 represent void ratio vs. log time curves for the different samples for each load increment. At the beginning of each load increment, and in too short a time to be measured accurately using normal procedure, there is an initial dial reading change that takes place. This dial reading change reflects various components of deformation which are difficult to evaluate individually. In the samples compacted at moisture contents below saturation, there is probably a deformation due to the immediate expulsion of air, there is compression of the porous stones and in the loading device, there is additional squeezing of material into the porous stones, and in some cases, an expansion of the soil into voids where the soil had been imperfectly trimmed into the ring. Finally, there is elastic deformation of the soil particles themselves. Upon checking the dial readings in the laboratory caused by compression of the porous stones and the loading plates, it was found that they were small compared to the total change in
thickness under each load. They have, however, been accounted for in
the void ratio calculations. All time curves are included in the
appendix. They are plotted in the standard semi-log form for purpose
of comparing to the more familiar log time curves for saturated soils.
For the most part, the dial reading-log time curves are very similar
for all samples. They are very flat for the first 100 minutes then
becoming increasingly steep up to the 24 hour period when the load was
changed. There is no reversal of the curve at the end of the loading
time on those samples which were below saturation at all times. Thus
A. Casagrande's procedures for determining 100% consolidation can
not be applied, nor does Taylor's plot of void ratio vs. the square root
of time give a sensible value of 90% consolidation. No reasonable
method of determining 100% consolidation presents itself from the time
data accumulated during this investigation. Figure 72 represents a
settlement-time curve for a sample loaded under one load for a period
of 30 days. The rate of settlement throughout the duration of the test
remains practically constant with the exception that the initial
consolidation taking place immediately after the load is applied is
quite large for the reasons previously explained.

Figures 44 through 55 represent the deformation time curves for
specimens CS 7 and CS 8 which are compacted at moisture contents above
optimum. Both of these specimens reached computed saturation before the
end of their loading sequence. The curves begin to look a little more
like the standard Casagrande time curve after this has happened.

It should be noted that very little loss of water took place
during the entire loading sequence in the samples tested, and it would
seem that the loss that did occur was due to evaporation and not due to
excess pore water pressure forcing the water out of the voids. The pore water pressure within the sample was probably negative during the entire test period. Since this is the case, then Terzaghi's theory is not applicable to this study. The time lag in the deformation must be due to the time necessary for the air in pores to escape, but even more important is the time necessary for the adjustment of soil structure under the influence of the load. The entire sequence of deformation seems to fulfill the definition of secondary consolidation. The influence of structure on the shape of the curves is also evident to a degree. Where the samples were compacted dry of optimum, such as CS 6 (Figure 38 through 43), the time curves are very flat for the low loads, dropping off suddenly at the end of the third load increment. Significant structural breakdown seemed to take place at the end of load 3 with 4, 5, and 6 developing time characteristics similar to those represented by CS 7 (Figure 44 through 49 which are the time curves for a soil compacted wet of optimum). Soils compacted dry of optimum are said to be more flocculated than those compacted wet of optimum\(^{(12)}\). This gives even more credence to the statement that the compacted soil below saturation is exhibiting structural reorientation consolidation with time, normally referred to as secondary consolidation. Further investigation of this soil reaction is certainly warranted for a better understanding of secondary consolidation in saturated soils.

Figures 56 through 63 represent void ratio vs. log pressure curves presented in the same manner as for standard consolidation tests. Also, the void ratio vs. pressure curves have been plotted to an arithmetic scale so that the distortion of the log scale is eliminated (Figure 64 to 71).

It was thought initially that there would be evidence of systematic
changes in the preconsolidation load values for the sample compacted at various compactive energies and at different moisture contents. It would be possible on some of the curves to find such a preconsolidation pressure, but not on all of them, and there does not seem to be any systematic value change evident.

From the tabulation of data and the pressure void ratio relationships, there are some features that are of interest. Samples compacted within ± 2% of optimum moisture content at compactive energies of 20, 25, 30, 35, and 40 blows per layer reach final void ratios as shown in Figure 73. The values shown should not be construed to be the final void ratio that would exist if the loads had been left on for a much longer period of time, but it is interesting that they should all collect about the same value as shown, even though the initial void ratios were different. Values of Cc for samples compacted at different compactive energies at moisture content within ± 2% of optimum seen to fall off exponentially with compactive effort as shown in Figure 74. The values of Cc for this soil are quite low. The highest value of .122 is about 1/2 of that which would be expected if this soil were normally consolidated, and the lower value of .044 about 1/6 of the value that would be expected. The values of Cc are lowest for moisture contents below optimum and increase with moisture contents above optimum (Figure 74a).

For samples compacted with 25 blows/layer, at moisture contents below, at, and above optimum, the final void ratios are shown in Figure 73a. The change in void ratio is the smallest for CS 6 which was compacted below optimum and the largest for CS 8 which had practically the same initial density but was compacted at well over optimum moisture. There
is almost four times as much void ratio change in CS 8 as in CS 6.

Samples CS 2 and CS 7 have almost the same change in void ratio even though CS 7 had a somewhat higher density initially. Thus again the inference is that the wetter soil will give more settlement in a given time than the dryer soil with the limitation to there being at saturation where permeability to water would control the amount and time of settlement. The final void ratio under the same final load decreases with increasing compaction moisture content. (Figure 73a). The difference is obviously a function of soil structure and the viscosity of the absorbed moisture films.

For Cc values plotted vs. moisture content for the 25 blows per layer compactive energy (Figure 74a), the increase in Cc with increase in moisture again infers that a sample would reach some limiting value of void ratio, when compacted above optimum, sooner than if compacted below optimum. The influence of structure and the rheological properties of the system seem to be quite significant. If it can be assumed that all samples of the same soil when under the influence of the same load would all reach a common final void ratio, then it appears that the sample compacted wet of optimum would reach this limit first. The structure of the soil must be significantly influencing the secondary type of consolidation taking place within the system.
B. Application of e-p Curve

A great need exists for field data and research investigations in the settlement of foundations on partially saturated soils. Extensive research on compacted unsaturated soils is developing at the present time. The main field of practical application anticipated for these studies is a better understanding of the behavior of soils as placed in large fills, especially in the construction of earth dams.

During construction of the rolled earth embankments, the objective is for each layer of soil to be identical, and to be compacted at the same water content, and to the same unit weight.

A possible method of estimating the settlement of the embankment due to consolidation of the material within the embankment is as follows: Consider an element of soil with unit volume of soil solids, then the initial void ratio \( e_0 \) will be the same as the voids volume of the element. If the element has unit cross sectional area, then the total height of the element will be \( (1 + e_0) \). Where \( H \) denotes the embankment height in feet, \( S \) denotes the settlement within embankment in feet, \( e \) denotes void ratio after load is applied, \( \Delta e \) denotes void ratio difference between initial and final \((24 \text{ hours after increment is applied})\) void ratio, \( \gamma \) denotes compacted wet unit weight in tons per cubic foot, the

\[
S = H \frac{\Delta e}{1 + e_0} \quad \text{(1)}
\]

Let \( e = f(p) \) represent e-p curve.

as \( p = \gamma h \), so \( e = f(\gamma h) \)

\[
\Delta e = e_0 - e = e_0 - f(\gamma h)
\]

Consider a differential layer thickness \( \Delta h \) (see Figure B)

\[
\Delta S = \Delta h \frac{\Delta e}{1 + e_0} = \Delta h \frac{e_0 - f(\gamma h)}{1 + e_0}
\]
Table II. Void Ratio Values for CS 1, 2, 3, 6, 8

<table>
<thead>
<tr>
<th>W%</th>
<th>N</th>
<th>$\gamma_d$ pcf</th>
<th>$\gamma_{wet}$ pcf</th>
<th>$\gamma_{wet}$ TCF</th>
<th>$e_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS 1</td>
<td>18.9</td>
<td>20</td>
<td>101.1</td>
<td>120.1</td>
<td>.0600</td>
</tr>
<tr>
<td>CS 2</td>
<td>17.0</td>
<td>25</td>
<td>104.7</td>
<td>122.4</td>
<td>.0612</td>
</tr>
<tr>
<td>CS 3</td>
<td>17.9</td>
<td>30</td>
<td>104.5</td>
<td>121.6</td>
<td>.0608</td>
</tr>
<tr>
<td>CS 6</td>
<td>14.2</td>
<td>25</td>
<td>102.0</td>
<td>116.2</td>
<td>.0581</td>
</tr>
<tr>
<td>CS 8</td>
<td>21.3</td>
<td>25</td>
<td>101.0</td>
<td>122.6</td>
<td>.0613</td>
</tr>
</tbody>
</table>

Table III. Unit Weight Values for CS 1, 2, 3, 6, 8

<table>
<thead>
<tr>
<th>W%</th>
<th>N</th>
<th>$\gamma_d$ pcf</th>
<th>$\gamma_{wet}$ pcf</th>
<th>$\gamma_{wet}$ TCF</th>
<th>$e_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS 1</td>
<td>18.9</td>
<td>20</td>
<td>101.1</td>
<td>120.1</td>
<td>.0600</td>
</tr>
<tr>
<td>CS 2</td>
<td>17.0</td>
<td>25</td>
<td>104.7</td>
<td>122.4</td>
<td>.0612</td>
</tr>
<tr>
<td>CS 3</td>
<td>17.9</td>
<td>30</td>
<td>104.5</td>
<td>121.6</td>
<td>.0608</td>
</tr>
<tr>
<td>CS 6</td>
<td>14.2</td>
<td>25</td>
<td>102.0</td>
<td>116.2</td>
<td>.0581</td>
</tr>
<tr>
<td>CS 8</td>
<td>21.3</td>
<td>25</td>
<td>101.0</td>
<td>122.6</td>
<td>.0613</td>
</tr>
</tbody>
</table>
(where \( p \) in TSF, \( \gamma \) in TCF, 
\( h \) in ft.)

\[
S = \int S = \int_0^H \frac{e_0 - f(\gamma h)}{i + e_0} \, dh \quad \quad \quad (2)
\]

From one-dimensional test data, there are seven points on the e-p curve. The numerical values are shown in Table II. By inspection, there are three likely functions which can approximately represent the e-p curve. These are \( e = \frac{A}{p+B} \), 
\( e = Ap^2 + Bp + C \), \( e = Ap^3 + Bp^2 + Cp + D \).

By trial and error, it is found that the e-p curves are very close to a cubic polynomial function.

Now let \( e = f(p) = Ap^3 + Bp^2 + Cp + D \), and the experimental points are \((e_0, p_0)\), \((e_1, p_1)\), \((e_2, p_2)\), \((e_3, p_3)\), \((e_4, p_4)\), \((e_5, p_5)\), \((e_6, p_6)\). If \( r \) denotes the residual error, then:

\[
\begin{align*}
  r_0 &= e_0 - f(p_0) = e_0 - Ap_0^3 - Bp_0^2 - Cp_0 - D \\
  r_1 &= e_1 - f(p_1) = e_1 - Ap_1^3 - Bp_1^2 - Cp_1 - D \\
  r_2 &= e_2 - f(p_2) = e_2 - Ap_2^3 - Bp_2^2 - Cp_2 - D \\
  r_3 &= e_3 - f(p_3) = e_3 - Ap_3^3 - Bp_3^2 - Cp_3 - D \\
  r_4 &= e_4 - f(p_4) = e_4 - Ap_4^3 - Bp_4^2 - Cp_4 - D \\
  r_5 &= e_5 - f(p_5) = e_5 - Ap_5^3 - Bp_5^2 - Cp_5 - D \\
  r_6 &= e_6 - f(p_6) = e_6 - Ap_6^3 - Bp_6^2 - Cp_6 - D
\end{align*}
\]

then:

\[
\begin{align*}
  r_0^2 &= (e_0 - D)^2 - 2(e_0 - D)(Ap_0^3 + Bp_0^2 + Cp_0) + (Ap_0^3 + Bp_0^2 + Cp_0)^2 \\
  r_1^2 &= (e_1 - D)^2 - 2(e_1 - D)(Ap_1^3 + Bp_1^2 + Cp_1) + (Ap_1^3 + Bp_1^2 + Cp_1)^2 \\
  r_2^2 &= (e_2 - D)^2 - 2(e_2 - D)(Ap_2^3 + Bp_2^2 + Cp_2) + (Ap_2^3 + Bp_2^2 + Cp_2)^2 \\
  r_3^2 &= (e_3 - D)^2 - 2(e_3 - D)(Ap_3^3 + Bp_3^2 + Cp_3) + (Ap_3^3 + Bp_3^2 + Cp_3)^2 \\
  r_4^2 &= (e_4 - D)^2 - 2(e_4 - D)(Ap_4^3 + Bp_4^2 + Cp_4) + (Ap_4^3 + Bp_4^2 + Cp_4)^2 \\
  r_5^2 &= (e_5 - D)^2 - 2(e_5 - D)(Ap_5^3 + Bp_5^2 + Cp_5) + (Ap_5^3 + Bp_5^2 + Cp_5)^2 \\
\end{align*}
\]
Let \( n = 6 \) and \( n = 6 \)

\[
R = \sum_{n=0}^{\infty} \left[ (e_n - D)^2 - 2(e_n - D)(A_{n} + B_{n} + C_{n}) + (A_{n} + B_{n} + C_{n})^2 \right]
\]

By the principle of least squares, the best fit curve (most likely case) must be the case for which \( R \) has a minimum value.

So:
\[
\frac{\partial R}{\partial A} = \frac{\partial R}{\partial B} = \frac{\partial R}{\partial C} = \frac{\partial R}{\partial D} = 0 \quad \ldots \ldots .(5)
\]

i.e.
\[
\begin{pmatrix}
(\Sigma p_n^5) A + (\Sigma p_n^4) B + (\Sigma p_n^3) C + (\Sigma p_n^2) D = \Sigma e_n p_n^3 \\
(\Sigma p_n^5) A + (\Sigma p_n^4) B + (\Sigma p_n^3) C + (\Sigma p_n^2) D = \Sigma e_n p_n^4 \\
(\Sigma p_n^4) A + (\Sigma p_n^3) B + (\Sigma p_n^2) C + (\Sigma p_n) D = \Sigma e_n p_n \\
(\Sigma p_n^3) A + (\Sigma p_n^2) B + (\Sigma p_n) C + (\Sigma) D = \Sigma e_n
\end{pmatrix}
\]

\[
\ldots .(6)
\]

Solving the simultaneous equations (6) by the aid of computer, the most probable values of \( A, B, C, D \), are determined. So \( f(p) \) is determined.

For CS 1,
\[
f(p) = 0.61120278 - 0.034917808 + 0.004926330p^2 - 0.0002413690p^3
\]

For CS 2,
\[
f(p) = 0.55339551 - 0.02659867p + 0.0046613305p^2 - 0.0002840095lp^3
\]

For CS 3,
\[
f(p) = 0.55606034 - 0.017723403p + 0.0028463701p^2 - 0.0001735451p^3 \quad .(7)
\]

For CS 6,
\[
f(p) = 0.596888360 - 0.008485994p + 0.0039756928p^2 - 0.0001179562p^3
\]

For CS 8,
\[
f(p) = 0.61938867 - 0.047713103p + 0.00400080780p^2 - 0.00015294534p^3
\]

Substituting these \( f(p) \) into equation (2), the relationship between \( S \) and \( H \) is determined as shown in the following equations.
For CS 1, \( S = \left( + .00139722H + .0010475342H^2 - .000005391H^3 \right) + .000000013033921H^4 \) / 1.6126

CS 2, \( S = \left( + .00300449H + .00081273193H^2 - .0000058195779H^3 \right) + .000000016274521H^4 \) / 1.5564

CS 3, \( S = \left( + .00056034H + .00053879145H^2 - .0000035073363H^3 \right) + .000000009751317H^4 \) / 1.5595

CS 6, \( S = \left( + .00131640H + .00024651828H^2 - .00000044734628H \right) + .00000000057834822H^4 \) / 1.5982

CS 8, \( S = \left( - .00548867H + .0014624966H^2 - .0000050203715H^3 \right) + .0000000088076020H^4 \) / 1.6139

Figures 75 and 75a are plots of (8) which relate the settlement to be expected within the fill to the height of embankment. The values of settlement shown include all deformation due to initial compression which makes them appear quite large.

Judgment, experience and actual field measurements would make it possible to correct the total settlement to be expected to account for this initial settlement. Probably as much as 50% would have taken place by the end of construction.
VI. CONCLUSIONS AND RECOMMENDATIONS

The objective of this research was to study the consolidation characteristics of compacted unsaturated soils. The conclusions made from this research are:

1. Compacted soils can never be completely saturated by compaction with impact type compactors.

2. The line of optimum moisture content for Clarksville silty clay (passing No. 10 sieve) approaches very close to the 86% saturation line.

3. Compression-time curves for soils below saturation deviate from the idealized theoretical time curves for saturated soils. The consolidation is still continuing at the end of the standard loading period of 24 hours, and no value of 100% consolidation can be determined. The reason for the slow continuing deformation seems to be the time necessary for structural orientation and rotation of the particles in a viscous adsorbed water medium influenced by air-water interfaces and surface tensions.

4. A long duration (30 days) one-dimensional compression test on a compacted sample indicated that the rate of settlement under the load increment was decreasing very slowly so that leaving the test load on for longer periods of time in practical applications does not offer a solution to the determination of 100% consolidation.

5. The entire load time compression curves on compacted soils indicate that the deformation characteristics are similar to what is termed secondary consolidation in testing saturated soils. In saturated soils, the pore pressure at the beginning of secondary consolidation
are said to be approaching zero, whereas in these tests below saturation, the pore water pressures were probably always negative unless saturation occurred during the test. This infers that the effective pressures influencing consolidation will be greatest when the moisture content of the sample is below optimum, yet those samples with moisture contents low of optimum were even slower to consolidate than the soils tested with moisture contents above optimum.

6. From the pressure-void ratio curves, the values of $C_c$ decrease with increasing compaction effort and increase with increasing moisture content at the same compactive effort. If there is a limiting void ratio that would be achieved under any given load increment, then it appears that the wetter the soil the earlier it would reach this limiting value.

7. For the soils compacted with different compactive efforts, the final voids ratio after 24 hours under the 10.32 ton per square foot pressure all ended up at very nearly the same void ratio.

8. No systematic changes in the so called preconsolidation pressures can be determined from the e-log p curves. It was thought that compaction effort, increased density and moisture content factors would be reflected in the preconsolidation pressure value, but such was not the case for samples tested. The effects of compaction and preconsolidation are not alike.

9. Five settlement vs. embankment height curves are obtained from e - p curves.

Further research in the area of consolidation and shear strength of compacted partially saturated fine grained soils is necessary.
Recommendations for additional study are listed as follows:

1. Additional investigation must be performed to arrive at some means of determining the final voids ratio that should be achieved under each load increment.

2. New rate of settlement theories need to be considered for soils being consolidated below saturation.

3. Tests in the laboratory should be performed on samples loaded below saturation then saturated at the end of the loading sequence of load increment to see what effect this could have on the final voids ratio achieved under that load.

4. Pore water and pore air pressure measurements should be made in conjunction with the one-dimensional compression test.

5. The role of soil structure in consolidation should be investigated more thoroughly.

6. The value or degree of accuracy of the settlement vs. embankment height curves must be evaluated by the field measurement of embankment settlements.
BIBLIOGRAPHY


FIGURE 1. CONCEPT OF INTERGRANULAR OR EFFECTIVE STRESS ON AREA
FIGURE 2. EFFECTS OF COMPACTION ON STRUCTURE

FIGURE 3. EFFECT OF ONE-DIMENSIONAL COMPRESSION ON STRUCTURE
Figure 4. Grain size distribution curve

Clarksville soil passing No. 10 sieve (CL)
FIGURE 5. CONSOLIDATION APPARATUS
SAMPLE:
PASSING NO. 10 SIEVE TESTS:
STANDARD PROCTOR SPECIFIC GRAVITY:
\[ G_s = 2.61 \]

100% SATURATION OR ZERO AIR VOIDS CURVE

FIGURE 6. COMPACTION CURVES AT VARIOUS COMPACTIVE EFFORTS FOR CLARKSVILLE SILTY CLAY
FIGURE 7. TIME-CONSOLIDATION CURVE
FIGURE 8. TIME-CONSOLIDATION CURVE
FIGURE 9. TIME-CONSOLIDATION CURVE
FIGURE 10. TIME-CONSOLIDATION CURVE

\[ D_0 = 0.0216 \]
\[ e_0 = 0.5707 \]
\[ D_f = 0.0411 \]
\[ e_f = 0.5504 \]

\[ \Delta H = 0.0136 \text{ in} \]

Time in Minutes
\[ v = 0.0315, \quad e = 0.0135 \]

CS 1-6
LOAD = 10.32 TSF
\[ \Delta H = 0.0260 \text{ in} \]

\[ d_f = 0.00015, \quad e_f = 0.4764 \]

**Figure 12. Time-Consolidation Curve**
FIGURE 13. TIME-CONSOLIDATION CURVE
FIGURE 14. TIME-CONSOLIDATION CURVE
FIGURE 15. TIME-CONSOLIDATION CURVE
FIGURE 16. TIME-CONSOLIDATION CURVE
FIGURE 17. TIME-CONSOLIDATION CURVE
CS 2-6
LOAD=10.32 T.S.F
\Delta H=.0165 IN

\( D_o = 0.0395 \)
\( e_o = 0.5005 \)

\( D_f = 0.0560 \)
\( e_f = 0.4765 \)

FIGURE 18. TIME-CONSOLIDATION CURVE
Figure 19. Time-Consolidation Curve
FIGURE 20. TIME-CONSOLIDATION CURVE
FIGURE 21. TIME-CONSOLIDATION CURVE
Figure 22. Time-Consolidation Curve

Compression dial reading in 0.001 inch

Time in minutes

CS 3-3

Load = 1.29 TSF

ΔH = 0.044 IN

Df = 0.0161

e = 0.5368

D0 = 0.0118

e0 = 0.5424
\( \mu = 0161, \ e = 5368 \)

CS 3-4

LOAD = 2.53 TSP

\( \Delta H = 0.058 \) IN

\( D_f = 0219 \)

\( e_f = 5293 \)

**FIGURE 23. TIME-CONSOLIDATION CURVE**


**FIGURE 24. TIME-CONSOLIDATION CURVE**

- **D₀ = 0.0219**
- **e₀ = 0.5293**
- **CS 3-5**
- **LOAD = 5.16 TSP**
- **ΔH = 0.0099 IN**

Time in Minutes

Compressional Dial Reading in 0.0001 inch
FIGURE 25. TIME-CONSOLIDATION CURVE
FIGURE 26. TIME-CONSOLIDATION CURVE
FIGURE 27. TIME-CONSOLIDATION CURVE
FIGURE 28. TIME-CONSOLIDATION CURVE

\[ D_0 = 0.0187 \]
\[ e_0 = 0.4935 \]

CS 4-3
LOAD = 1.29 TSF
\[ \Delta H = 0.0046 \text{ IN} \]

\[ D_1 = 0.0233 \]
\[ e_1 = 0.4875 \]
FIGURE 29. TIME-CONSOLIDATION CURVE
FIGURE 30. TIME-CONSOLIDATION CURVE
FIGURE 31. TIME-CONSOLIDATION CURVE

LOAD = 10.32 TSP
ΔH = .0116 IN

Compress. Dial Reading in .0001 inch

Time in Minutes

D₀ = 0.435
\( \bar{e}_0 = 0.4600 \)

\( D_f = 0.551 \)
\( \bar{e}_f = 0.4440 \)
FIGURE 32. TIME-CONSOLIDATION CURVE
Compression Dial Reading in 0.0001 inch

\[ D_0 = 0.0168 \]
\[ e_0 = 0.4999 \]

\[ D_f = 0.0176 \]
\[ e_f = 0.4995 \]

CS 5-2
LOAD = 645 PSF
\[ \Delta H = 0.0009 \text{ IN} \]

**Figure 33. Time-Consolidation Curve**
FIGURE 34. TIME CONSOLIDATION CURVE
FIGURE 35. TIME-CONSOLIDATION CURVE
FIGURE 36. TIME-CONSOLIDATION CURVE
Figure 37. Time-Consolidation Curve
Figure 38. Time-consolidation curve
CS 6-2
LOAD=.645 TSP
ΔH=.0018 IN

D_i = 0.0055
ε_i = .5910

FIGURE 39. TIME-CONSOLIDATION CURVE
FIGURE 40. TIME-CONSOLIDATION CURVE
FIGURE 41. TIME-CONSOLIDATION CURVE


**FIGURE 42. TIME-CONSOLIDATION CURVE**

- **CS 6-5**
- **LOAD = 5.16 TSP**
- **ΔH = 0.110 IN**

**Equations:**

- \( D = 0.155 \)
- \( e_0 = 0.5778 \)
- \( D_f = 0.2645 \)
- \( e_f = 0.5619 \)

**Axes:**
- Vertical: Compression Dial Reading in 0.001 inch
- Horizontal: Time in Minutes

Grid lines are present for reference.
FIGURE 43. TIME-CONSOLIDATION CURVE
FIGURE 44. TIME-CONSOLIDATION CURVE
FIGURE 45. TIME-CONSOLIDATION CURVE
FIGURE 46. TIME-CONSOLIDATION CURVE
FIGURE 47. TIME-CONSOLIDATION CURVE
FIGURE 48. TIME-CONSOLIDATION CURVE
FIGURE 49. TIME-CONSOLIDATION CURVE
FIGURE 50. TIME-CONSOLIDATION CURVE
FIGURE 51. TIME-CONSOLIDATION CURVE
FIGURE 52. TIME-CONSOLIDATION CURVE
FIGURE 53. TIME-CONSOLIDATION CURVE

\[ D_0 = 0.477 \]
\[ e_0 = 5.660 \]

LOAD = 2.58 TSF
\[ \Delta H = 0.0324 \text{ IN} \]

Time in Minutes
FIGURE 54. TIME-CONSOLIDATION CURVE
FIGURE 55. TIME-CONSOLIDATION CURVE
CS 1

\( w = 18.9\% \)

\( \gamma_d = 101.1 \text{ p.c.f.} \)

\( N = 20 \text{ blows/layer} \)

\( e_0 = 0.6128, \ S_0 = 80.2\% \)

\( e_f = 0.4764, \ S_f = 100\% \)

\( C_c = 0.1220 \)

**FIGURE 56. PRESSURE-CONSOLIDATION e-log p CURVE**
FIGURE 57. PRESSURE-CONSOLIDATION e-log p CURVE

CS 2

W = 17.0%
γ_d = 104.7 p.c.f.
N = 25 blows/layer
e_o = 0.5564, S_o = 79.4%
e_f = 0.4766, S_f = 92.7%
C_c = 0.0845
CS 3  
W=17.9%  
γ_d=104.5 p.c.f.  
N=30 blows/layer  
e_o=.5596, S_o=76.0%  
e_f=.4924, S_f=86.3%  
C_c=.0780

FIGURE 58. PRESSURE-CONSOLIDATION e-log p CURVE
FIGURE 59. PRESSURE-CONSOLIDATION e-log p CURVE

CS 4
W=17.9%
\( \gamma_d = 107.1 \) p.c.f.
N=35 blows/layer
e_0 = 0.5207, S_0 = 83.0%
e_f = 0.4440, S_f = 96.7%
C_c = 0.0540

Pressure p in tons/sq ft (log scale)
CS 5
w=18.2%
\( \gamma_d = 106.8 \) p.c.f.
N=40 blows/layer
e_0 = .5250, S_o = 80.0%
e_f = .4537, S_f = 92.5%
Cc = .0440

FIGURE 60. PRESSURE-CONSOLIDATION e-log p CURVE
CS 6

\[ W = 14.2\% \]
\[ \gamma_d = 101.9 \text{ p.c.f.} \]

\[ N = 25 \text{ blows/layer} \]
\[ e_o = 0.5982, S_o = 61.8\% \]
\[ e_f = 0.5408, S_f = 68.4\% \]
\[ C_c = 0.0710 \]

**FIGURE 61. PRESSURE-CONSOLIDATION \( e - \log p \) CURVE**
FIGURE 62. PRESSURE-CONSOLIDATION e-log p CURVE
FIGURE 63. PRESSURE-CONSOLIDATION $e-\log p$ CURVE

- $W=21.3\%$
- $\gamma_d=101.0$ p.c.f.
- $N=25$ blows/layer
- $e_o=.6139, S_o=90.6\%$
- $e_f=.3946, S_f=100\%$
- $C_c=.1960$

Pressure $p$ in tons/sq ft (log scale)
CS 1
W=18.9%
γ_d=101.1 p.c.f.
N=20 blows/layer
% of settlement
=8.45%

FIGURE 64. PRESSURE-CONSOLIDATION (e-p) CURVE
FIGURE 65. PRESSURE-CONSOLIDATION (e-p) CURVE
FIGURE 66. PRESSURE-CONSOLIDATION (e-p) CURVE
FIGURE 67. PRESSURE-CONSOLIDATION (e-p) CURVE
FIGURE 68. PRESSURE-CONSOLIDATION (e-p) CURVE
CS 6
W = 14.2%
\( \gamma_d = 101.9 \) p.c.f.
N = 25 blows/layer
% of settlement = 3.59%

**FIGURE 69. PRESSURE-CONSOLIDATION (e-p) CURVE**
FIGURE 70. PRESSURE-CONSOLIDATION (e-p) CURVE
FIGURE 72. LONG DURATION SETTLEMENT-TIME CURVE
FIGURE 73. COMPACTIVE EFFORT AND FINAL VOID RATIO RELATIONSHIP
FIGURE 73a. MOISTURE CONTENT AND FINAL VOID RATIO RELATIONSHIP
FIGURE 74. COMPACTION EFFORT AND COMPRESSION INDEX "Cc" RELATIONSHIP

Compaction Effort--Number of blows per layer
in standard proctor mold at water content 18.3%
Moisture content -- W% at constant compaction effort (N=25 blow/layer)

FIGURE 74a. MOISTURE CONTENT AND COMPRESSION INDEX Cc RELATIONSHIP
MOISTURE CONTENT
18.0±1% (O.M.C.±1)

CS 1
W=18.9%
N=20 blows

CS 2
W=17.0%
N=25 blows

CS 3
W=17.0%
N=30 blows

FIGURE 75a. SETTLEMENT AND EMBANKMENT HEIGHT RELATIONSHIPS
FIGURE 75. SETTLEMENT AND EMBANKMENT HEIGHT RELATIONSHIPS
VITA

Ma-Tai Matthew Chen was born on October 10, 1937, in Hofei, Anhwei, China. He received his primary education in Mainland China and his secondary education was received in Hsinchu and Taipei, Taiwan. His college education was received in National Taiwan University where he obtained a Bachelor of Science degree in Civil Engineering in 1959.

After graduation from National Taiwan University, he received ROTC training and was commissioned a Second Lieutenant in the Republic of China Air Force as an engineering officer about one and half years.

He and his brother established a small continuation school in Taipei. He was the chairman and teacher for half year.

He worked in a soil testing laboratory as an assistant engineer for three years in Hsinchu, Taiwan.

In September of 1964, he came to the United States for further study. He enrolled in the University of Missouri at Rolla as a graduate student in Civil Engineering.

Chen is a Christian. He was engaged to Miss Ada Chu in 1964.