THE EFFECTS OF COMPACTION ON SHEAR STRENGTH AND ONE-DIMENSIONAL CONSOLIDATION OF A SILTY SAND

by

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INTRODUCTION

Soil has been used as a foundation and construction material since the very beginning of civilization. Man has thus long encountered the problems in soil mechanics which include bearing capacity of soil and settlement of foundation.

Foundation must satisfy three requirements:

1. The pressure induced by external loads must be sufficiently less than the ultimate bearing capacity of the soil to assure foundation safety.
2. The total and differential settlement must be small enough to assure that the structure will not be damaged by foundation movements.
3. The effects of the structure and its necessary construction operations on adjoining buildings and facilities must be evaluated and necessary protective measures taken.

In order to meet the above three requirements the soil engineer must know cohesion, internal friction angle and the compressibility of the soil deposits on which the foundation will be placed.

Usually the soil at a site is not ideal from the viewpoint of soil engineer, because of lack of shear strength, high compressibility, etc. The soil engineer has to select one of three measures as follows.

1. Choosing another site.
2. Replacing the undesirable soil with desirable one.
3. Improving the soil in situ.

Today, the soil engineer is generally forced to take one of the last two measures, because of limitation of other conditions such as cost and availability of better sites.

Other kinds of construction, using earth as construction materials are dams, retaining walls, highways, airports, and any other types of which require earth fill. In these structures, it is necessary to compact soil materials to meet the following requirements:

1. Minimize settlements.
2. Increase in shear strength.
3. Decrease in permeability.

All earth embankment constructions need the first two of these requirements and water control structures usually require all three.

The concept of relative compaction is usually employed in designing and controlling the construction quality. The designer specifies the percentage of compaction in earth construction to obtain desirable strength and tolerable settlement. By this standard, the quality of earth construction can be effectively and economically controlled.

THE PURPOSE OF THE STUDY

The purpose of this study was to investigate

a. The relationships between maximum dry density and compactive effort of the selected soil.

b. The relationships between shear strength and relative density of compacted soil.

c. The relationships between compressibility and relative density of compacted soil.

d. The relationships between consolidation and relative density of compacted soil.

e. The relationships between permeability and relative density of compacted soil.

THE SCOPE OF THE STUDY

A single soil was used in this study. The engineering characteristics of this soil is shown in Table (1). According to the Unified Soil classification, it belongs to the group of silty sand, abbreviated as SM.

First, the standard Proctor density tests were run to obtain optimum moisture content and maximum dry density.

Second, four more compaction tests with different compactive efforts were run to obtain the relationships between maximum dry density and compactive effort.

Third, the soils were remolded into samples with different dry densities and a constant volume (2.5 in. diameter, 1 in. thick). The direct shear tests and consolidation tests were run to obtain the relationships of shear strength, compressibility, and permeability to the dry density of compacted soil.

REVIEW OF LITERATURE

In broad sense, there are two major problems in earth construction and foundations; shear strength and compressibility of the soil deposits. Engineers and scientists have tried to mathematically model and analyze through experimental investigations and practical judgment to form rational laws which permit extrapolation and understanding of the behavior of soil materials in these two areas. But the variety of soil types and conditions and diversity of states of stresses made it very difficult.

The following reviews the literature of Proctor density compaction, shear strength, compressibility, comparisons of shear strength and relative density of compacted soil, and comparisons of compressibility and relative density of compacted soil.

Proctor Density Compaction

The Proctor density compaction method was first described by R. R. Proctor (1) in a series of articles published in Engineering News Record Aug. 31, Sept. 21, and Sept. 28, 1933. It was developed for controlling the compaction of soil at that time. Under this procedure a series of simple laboratory and field tests serves as a basis for the control of construction operations, and furnishes a definete check on the effectiveness of the work as it proceeds.

Proctor (1) found that soil can be compacted to a maximum dry density with the least amount of effort at a specific moisture content which he termed the optimum moisture content. He found further that the optimum moisture content varied with the amount of compactive effort such that an increase in compactive effort would result in a higher dry density at a lower moisture content. Wishing to make the test as practical as possible Proctor adjusted the laboratory compactive effort to be equal to field compactive effort using an economical by reasonable amount of compactive effort. This test procedure is called the standard Proctor density test and is described in detail in ASTM Standard Test method D-698 (2).

Shear Strength of Soil

The shear strength of soil is generally expressed by the Coulomb's equation

$$S = C + P \tan \phi$$

where S = shear stress

C = cohesion

P = normal stress on the shear plane

ϕ = angle of internal friction

That means, soil materials when subjected to any type of compressive condition conducive to failure, appear to develop their strength as the summation of two factors, cohesion and internal friction.

The internal friction depends upon the normal stress on the plane of failure. The angle of internal friction is a constant, independent of water content and stress history.

Hvorslev (3) discovered that cohesion is also independent of stress history and is only a function of water content. It implies that cohesion originates on bonding forces, which are a function of water content, whereas internal friction depends only on normal pressure on the plane of failure.

Another idea has been advanced to discriminate between the physical components of the shear strength of soils. It was first suggested by Taylor (4), who contended that cohesion may not be physically different from internal friction, but may only be an expression of the intrinsic pressure. To him an internal stress would be a function of the pre-consolidation pressure.

Compressibility of Soil

The modern history of soil mechanics is directly traceable to development of the theory of consolidation by Terzaghi (5). This theory for the first time developed a procedure based on mathematical rigor. It enables the engineers to predict the amount and the time rate of settlement of structures constructed on fine grained soils.

The assumptions of one-dimensional consolidation theory (4) are:

1. Homogeneous soil.
2. Complete saturation.
3. Negligible compressibility of soil grains and water.
4. Action of infinitesimal masses no different from that of layer, representative masses.
5. One-dimensional compression.

6. One-dimensional flow.

7. The validity of Darcy's law.

8. Constant values for certain soil properties which actually vary somewhat with pressure.

Based on these assumptions, one-dimensional consolidation theory can be derived as follows.

The equations governing consolidation (or swell) (6) are:

1. Equation of equilibrium for an element of soil

$$\sigma_v = \gamma_t z + \text{surface stress}$$

2. Stress-strain relation for the mineral skeleton

$$\frac{\partial e}{\partial \bar{\sigma}_v} \doteq - a_v$$

3. Continuity equation for the pore fluid

$$k \frac{\partial^2 h}{\partial z^2} = \frac{1}{(1+e)} \frac{\partial e}{\partial t}$$

where

σ_v = vertical stress at any depth

γ_t = total unit weight of soil

z = coordinate in vertical direction

$\bar{\sigma}_v$ = effective vertical stress

a_v = coefficient of compressibility

k = permeability of soil in vertical direction

h = total head

t = time

The coefficient of consolidation C_v is defined as

$$C_v = \frac{k\,(1 + e)}{\gamma_w\, a_v} = \frac{k}{\gamma_w\, m_v}$$

where m_v is coefficient of volume change.

Finally, Terzaghi's consolidation equation gives

$$C_v \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t} - \frac{\partial \sigma_v}{\partial t}$$

where u_e = excess pore pressure.

This derivation marked the birth of modern soil mechanics.

The simplest case is one-dimensional consolidation which gives additional conditions:

1. The total stress is constant with time,

   $\partial \sigma_v / \partial t = 0$;

2. The initial excess pore pressure is uniform with depth;
3. There is drainage at both the top and bottom of consolidation stratum;

Under these conditions the governing equation will be simplified as

$$C_v \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t}$$

There is a solution for this partial differential equation (4).

As stated by O. Moretto (2), Terzaghi's theory of one-dimensional consolidation is based on well-known assumptions about soil parameters as mentioned previously. By changing these assumptions and/or conditions, mathematical solutions for various other situations for one-, two, and three-dimensional consolidations have been obtained to express how the pore pressure and the degree of consolidation change with time.

A paper presented by R. L. Schiffman (7) extended the theory by generalizing the conditions of derivation to include variations in the rate of loading and variations in the coefficient of permeability during the process of consolidation.

Comparison of Shear Strength and Compaction

According to L. J. Langfelder and V. R. Nivargikar (8), the shear strength of cohesionless materials is essentially controlled by five factors:

1. Mineralogical composition
2. Size and gradation of the individual particles
3. Shape of the individual particles
4. Void ratio or dry density
5. Confine pressure

The first four factors affect the angle of internal friction, whereas the fifth factor controls the normal stress.

As described in the same paper, for a given material it is only the void ratio or dry density that can be modified by the compaction process. For a given cohesionless material it appears that the shear strength is directly related to the density and it is independent of the compaction process used to obtain this dry density.

Data presented by R. E. Means and J. V. Parcher (9) indicate that for a particular granular material the angle of internal friction is inversely related to the void ratio of the soil.

The change in the angle of internal friction with a change in void ratio appears to differ somewhat depending on the soil being tested-varying from 2 degrees for silty sands to about 6 degrees for uniform gravel for a 0.1 change in void ratio.

T. H. Wu (10) investigated the effect of initial void ratio on the angle of internal friction by using uniform sands with mean diameters of 0.15, 0.44, and 1.00 mm. It shows that the angle of internal friction for a given void ratio is different for each material; however the change in the angle of internal friction increased by about 2 degrees for a 0.1 decrease in void ratio.

The shear strength of a compacted cohesive soil is primarily affected by

1. Water content
2. Gradation
3. Dry density
4. Soil structure
5. Thixotropy
6. Normal effective stress acting on the failure plane

The changes in shear strength of cohesive soil are produced as a function of changes in dry density alone can be determined by using several different compaction energies and comparing the strength at a constant value of mold water content.

H. B. Seed, J. K. Mitchell, and C. K. Chan (11) have presented data on the relationship between dry density and shear strength at different water contents which indicates that an increase in dry density will cause an increase in shear strength for a given water content. In general, the rate of increase in shear strength with an increase in dry density is largest for the lowest value of water content. As the molding water content increases, the increase in shear strength is smaller to nonexistent, depending on the soil being investigated.

Comparison of Compressibility and Compaction

As L. J. Langfelder and V. R. Nivargikar (8) described, the compressibility characteristics of compacted cohesionless materials are primarily influenced by the same factors that influence the shear strength. In general, the compressibility decreases with increasing gradation, decreasing void ratio, decreasing angularity and increasing confining pressure.

The influence of relative density on compressibility is similar to the effect of relative density on shear strength, that is, increasing relative densities for a given material will cause decreasing compressibilities.

The compressibility characteristics of compacted cohesive material are significantly influenced by soil type, molding water content, dry density, degree of saturation, and the compaction method. In general, the compressibility increase with increasing liquid limit, increasing degree of saturation, and compaction procedures that produce large shear strains during the compaction process.

Based on data presented by H. B. Seed, J. K. Mitchell, and C. K. Chan (11), the compressibility characteristics of cohesive soil that are compacted on the wet side of optimum will be greatly influenced by the compaction procedure used to compact the soil. In general, it appears that the compressibility will increase for a soil compacted by static, vibratory, impact, and kneading compaction methods in that order.

That is, the statically compacted specimens should be less compressible than the specimens that are compacted using kneading methods for the same water content and dry density on the wet side of optimum; however, on the dry side of optimum the compressibility should be approximately the same regardless of the compaction method used.

DESIGN OF EXPERIMENT

The experiment was design to find the effects of compaction on shear strength and consolidation. Therefore, the following tests were adopted to obtain the necessary data.

1. It is essential to determine some elementary and important engineering characteristics of soil, including specific gravity, Atterberg limits and indices, grain size analysis. All of them give us whole view on a soil.

2. In order to determine the maximum dry density and optimum moisture content, the standard Proctor density test was selected.

3. Four more compaction tests with different numbers of blows per layer, 10, 15, 20, and 30, respectively, were run to find the relationships between maximum dry density and different compactive energy.

4. The strain-controlled direct shear test was used to determine the shear strength of soil at different levels of dry density and with water content close to optimum moisture content obtained from the standard Proctor test.

5. One-dimensional consolidation tests were used to determine the compressibility, consolidation and permeability of soil at different levels of these dry density.

PREPARATION OF SPECIMEN

Samples used for compaction tests were the soil passing U.S. Standard sieve #4. The amount of water was added to raise soil water content by approximately 4 % increments until the maximum dry density was obtained.

Samples used for both direct shear tests and one-dimensional consolidation tests were passed through the U.S. Standard sieve #10 and water was added equal to the optimum moisture content. Samples were molded to a constant volume (2.5 in. diameter, 1 in. high) at different percents of maximum dry density as determined by the standard Proctor density test. A list of total weight, water content, and percent of dry density for each sample are shown in Table (1).

The soil samples of constant total weight were compacted statically into a specimen by a Soil Test Versatester as shown in Fig. (1) which can reproduce samples easily and quickly according to desirable densities.

Table (1) A list of total weight, water content, and percent of max. dry density for each sample.

(A). Samples*a used for direct shear tests

Total Weight, g	Water Content, %	Weight of Solid, g	Percent of Max. Dry Density*b
164.64	17.76	139.81	105.3
156.41	17.39	133.24	100.4
148.41	17.29	126.36	95.2
139.95	16.95	119.67	90.2

(B). Samples*c used for consolidation tests

Total Weight, g	Water Content, %	Weight of Solid, g	Percent of Max. Dry Density
123.48	17.24	105.32	105.8
117.31	17.24	100.06	100.5
104.96	13.83 *d	92.21	92.6
98.86	14.98 *d	86.00	86.4

*a The samples were compacted statically to be 2.5 in. diameter, 1 in. high.

*b The 100 % of maximum dry density is equal to 103 pcf which was determined by standard Proctor density test.

*c The samples were compacted statically to be 2.5 in. diameter, 1 in. high, and then trimmed to be 2.5 in. diameter, 0.75 in. high.

*d The water content is not close to 17 % optimum moisture, but it does not affect the results of consolidation test.

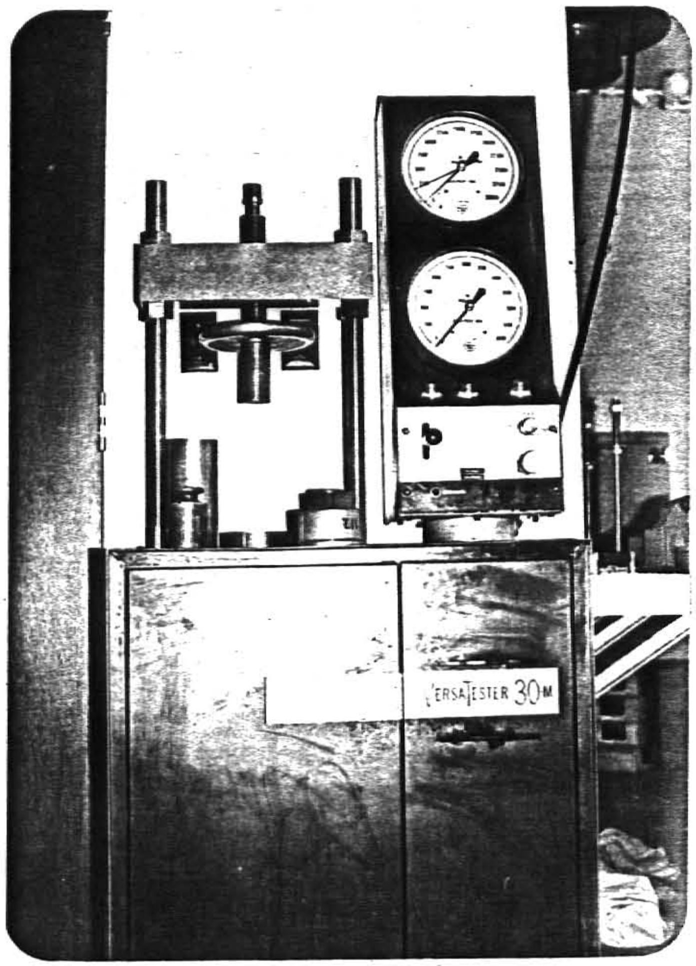

Fig. (1) Soil Test Versatester and a statically compacted sample.

APPARATUS AND TESTING OF SPECIMEN

(A). Compaction test

The soil was compacted according to recommended procedures from "Soil Testing for Engineers" by Professor T. William Lambe (12) based upon ASTM Standard Test Method D-698. The compaction tests were run with different numbers of blows per layer in order to obtain different compactive energy per volume. Table (2) shows different numbers of blows vs. different compactive energies per volume. The procedures included the preparation of specimen, as described previously, and processes of dynamic compaction. The latter using $5\frac{1}{2}$-lb. hammer with 12-in. drop compacted uniformly on the each of three layers of soil in the molds, and then trimmed and weighed soil. To obtain the soil water content, a representative sample were taken from the mold. The apparatus used for Proctor compaction test is shown in Fig. (2).

(B). Direct shear test

The preparation of specimen for direct shear tests was mentioned before. Furthermore, two water content determinations were made before and after direct shear test to assure that it was approximately 17 %. The sample was then put in sample box of direct shear test machine. The different loadings 23.24, 19.48, 15.72, 11.96, 8.20, and 4.44 kg. were applied to a sample as the normal loadings. Those caused 10.44, 8.75, 7.06, 5.37, 3.68, and 1.99 psi. normal stresses, respectively.

The direct shear machine, as shown in Fig. (3), is composed of five major parts: an electric motor with variable speed reduction, a threaded rod, a proving ring with a dial, a sample box, and a strain dial.

The electric motor generated power to drive the upper portion of the sample box. In this way, the shear force was applied to the face which was parallel with the circular faces of both ends of sample.

The readings from stress dial and strain dial gave shear stresses and shear strains respectively.

All of direct shear tests were conducted at a rate of 0.025 inches per minute.

Table (4) thru Table (7) will show the results of direct shear tests for this study.

(C). Consolidation test

The consolidation machine shown in Fig. (4) used for this study was lever type Odemeter designed by Professor A. W. Bishop. During the consolidation the specimen was confined laterally, and allowed drain vertically at both faces.

After removing specimen which was compacted from Soil Test Versatester, it was put into the locating ring. Then the soil sample was trimmed carefully to fit the ring as perfectly as possible and was saturated with water.

The adjustment of the yoke-frame, and the capstan adjusting nut were made. The purpose of it were:

(1). to keep the capstan adjusting nut in contact with the upper porous stone,

(2). to keep the loading beam level, and

(3). to set the dial gauge at the beginning of its release run.

Then the load was applied to give a total pressure intensity of 0.25 ton/sq.ft on the soil specimen. Compression readings were taken at total elapsed times of 0, 0.25, 1, 2.25, 4, 6.25, 9, 12.25, 16, 20.25, 25 minutes and after 24 hours or the increase in the dial reading was small enough to be neglected, the total pressures of 0.5, 1, 2, 4 ton/sq.ft were applied at successive times. Readings were taken as they were for the total pressure of 0.25 ton/sq.ft.

The results of consolidations and some characteristics, such as 90 % consolidation time, coefficient of consolidation, and coefficient of compressibility of soil which can obtain from consolidation tests, are shown in Table (9) thru Table (13).

All machines, apparatus and supplies mentioned previously in this study are in the soil mechanics laboratory of Kansas States University.

Table (2) Summary of compaction tests used for this study

Tests	Mold Size	Weight of Hammer, lb.	Number of Layers	Height of Hammer Drop, in.	Number of Blows per Layer	Compactive Energy per Volume ft-lb/cu.ft.
10-blow Proctor	4.6 x 4 in. dia.	$5\frac{1}{2}$	3	12	10	5,000*
15-blow Proctor	4.6 x 4 in. dia.	$5\frac{1}{2}$	3	12	15	7,400
20-blow Proctor	4.6 x 4 in. dia.	$5\frac{1}{2}$	3	12	20	9,900
Standard Proctor	4.6 x 4 in. dia.	$5\frac{1}{2}$	3	12	25	12,400
30-blow Proctor	4.6 x 4 in. dia.	$5\frac{1}{2}$	3	12	30	14,900

* The compactive energy per volume = weight of hammer x number of layers x height of hammer x number of blows per layer / total volume

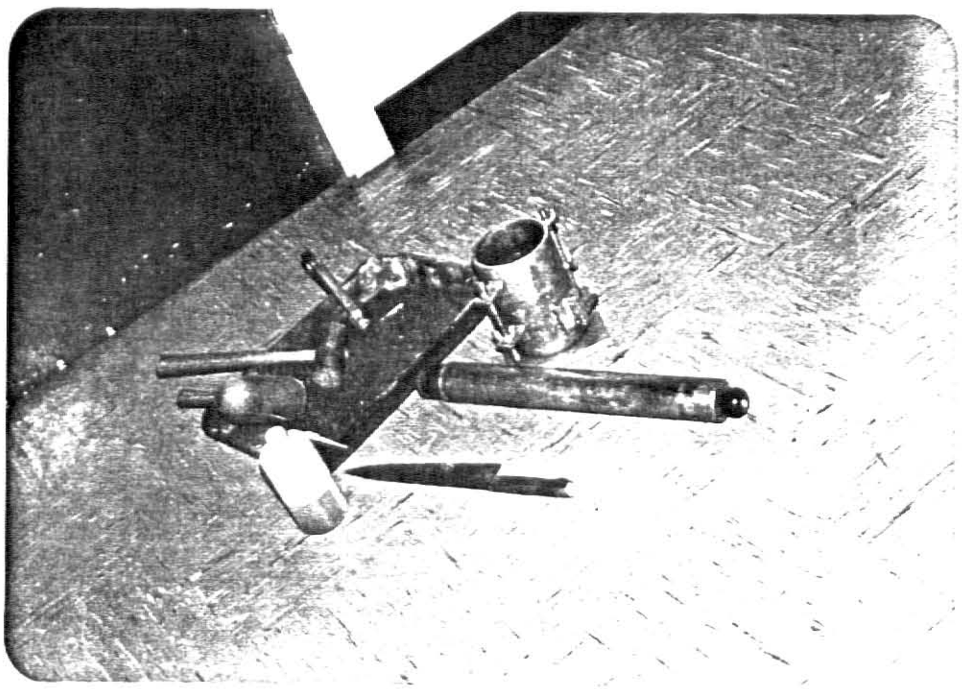

Fig. (2) Apparatus used for Proctor compaction test and soil sample.

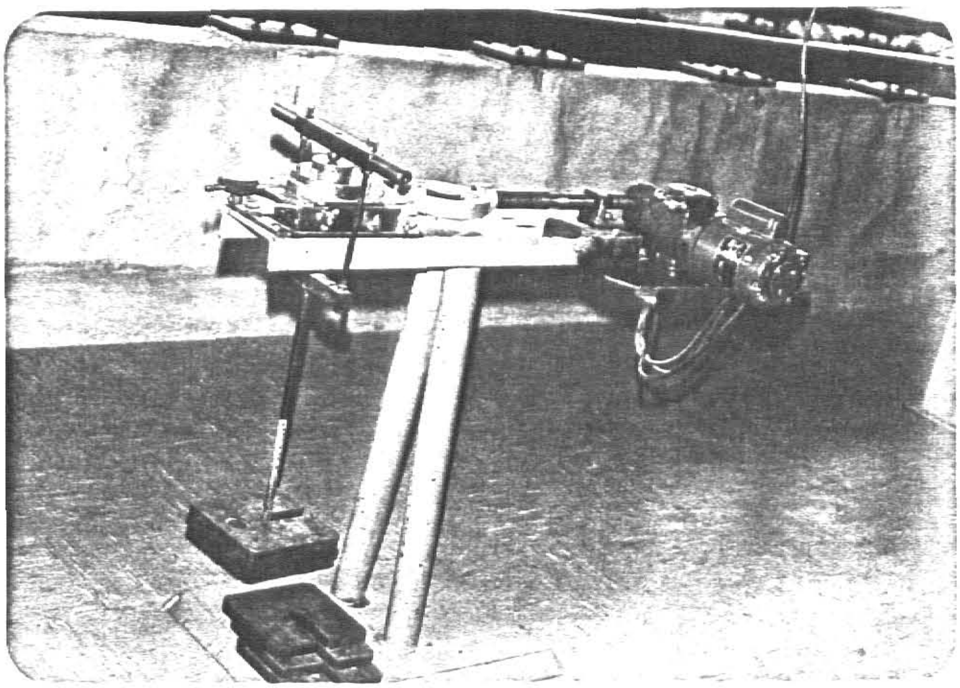

Fig. (3) Direct shear machine and sample subject to shear failure.

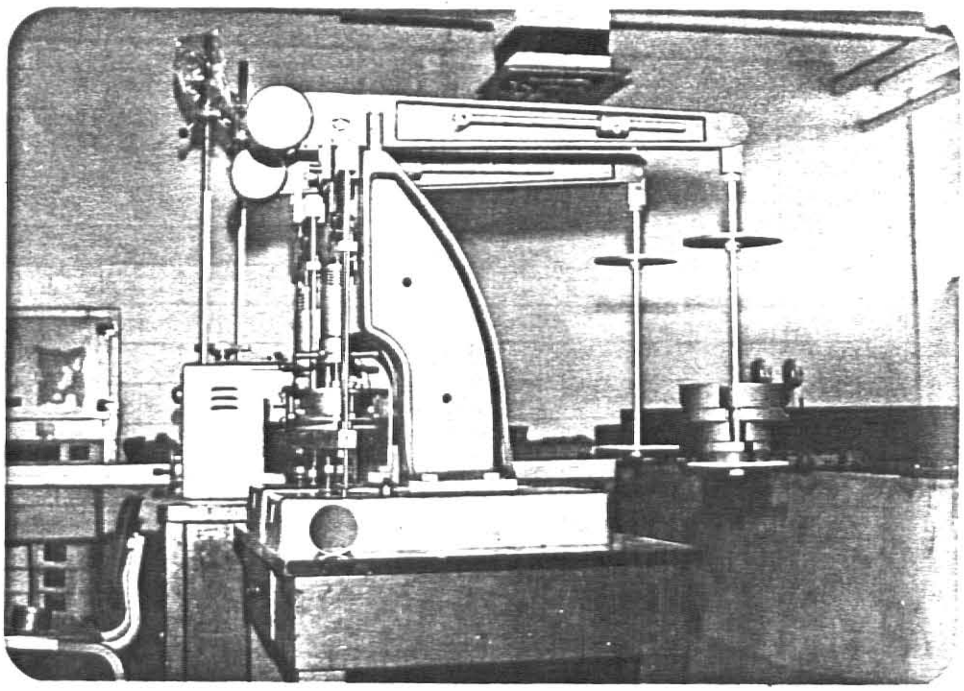

Fig. (4) One-dimensional consolidation test machine and sample after consolidation test.

PRESENTATION OF DATA

(A). General

Some kinds of soil testings were run in accordance with procedures recommended by T. William Lambe (12) in order to classify the soil and determine engineering characteristics of soil. The general physical characteristics of the soil are described in Table (3), and the grain size distribution of soil particles is graphed in Fig. (5).

(B). Compaction test

The results of compaction tests with different number of blows per layer indicated in Fig. (6) which shows a family of curves relating dry density and molding water content and describing the relationship between maximum dry density with number of blows per layer.

(C). Direct shear test

Table (4) thru Table (7) shows the results of direct shear tests for varying dry density. Each level of dry density has three samples to be tested. The mean values of shear stresses corresponding to normal stresses for each level of dry density are also shown in Table (8).

(D). Consolidation test

Table (9) thru Table (12) presents the results of one-dimensional consolidation tests for this study, which meantime presents the computed values of 90 % consolidation time T_{90}, coefficient of consolidation C_v, the final void ratio e, coefficient of compressibility a_v, and permeability k under corresponding loading increments. The e-log P curves are graphed in the Fig. (15) thru Fig. (18).

Table (3) Soil characteristics of the raw soil

1. Specific gravity of solids	2.711
2. Atterberg limits	
a. Liquid limit	30.8 %
b. Plastic limit	22.6 %
c. Plastic index	8.2 %
3. Group index	1.0
4. Grain size analysis percent	
a. Passing #200 sieve	39.48
b. Clay (smaller than 0.002 mm.)	0.00
c. Silt (0.005 to 0.074 mm.)	39.48
d. Sand (greater than 0.074mm.)	60.52
5. Uniformity coefficient	10.4
6. Coefficient of curvature	0.59
7. Standard Proctor compaction	
a. Optimum moisture content	17 %
b. Maximum dry density lb/cu.ft.	103
8. Unified Soil Classification	SM

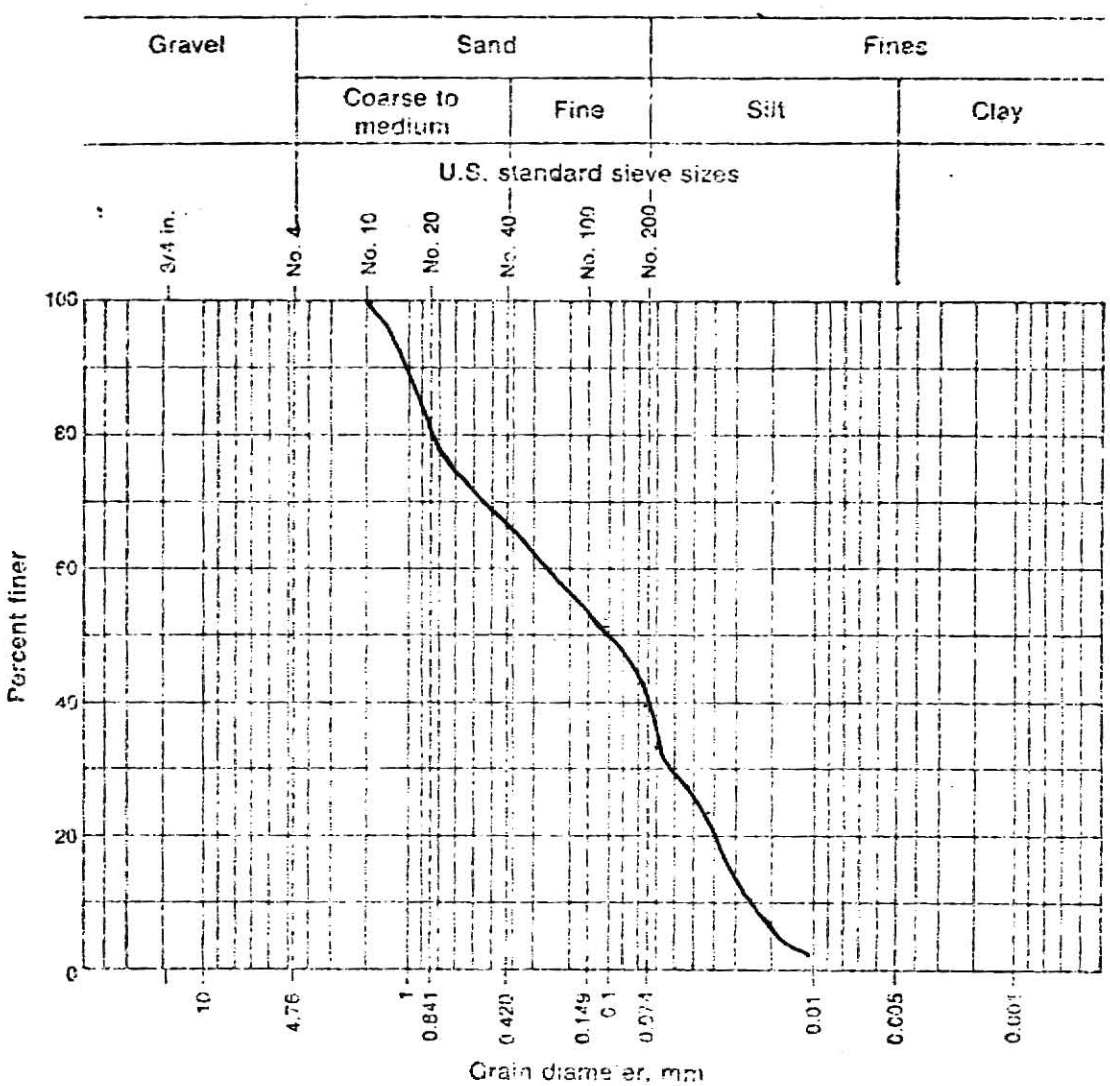

Fig. (5) Grain size distribution.

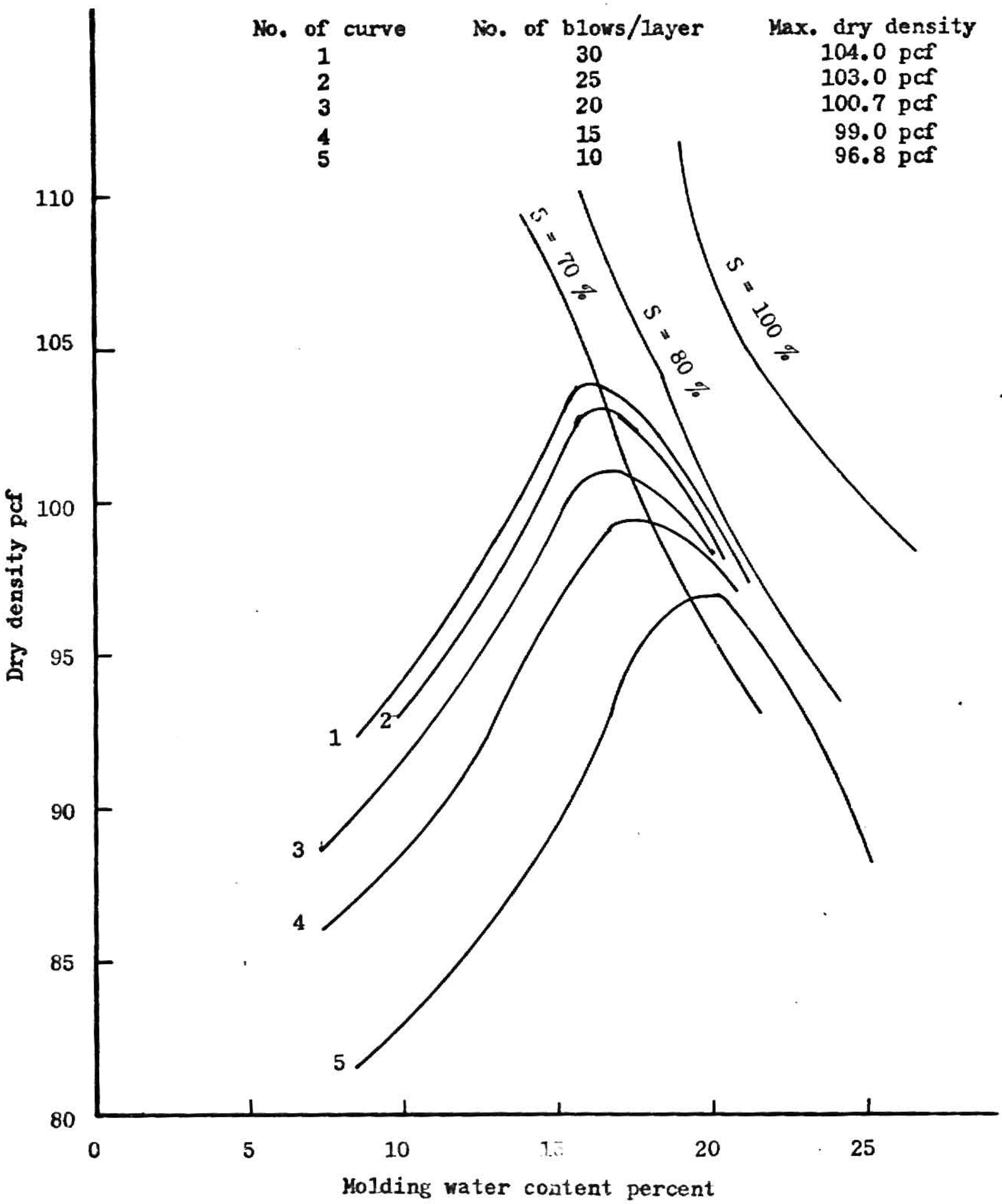

Fig. (6) Relationships of dry density and molding water content with different no. of blows per layer.

Table (4) Results of direct shear tests

Level I. Dry density = 105.3 % max. dry density by standard Proctor test

Weight of wet soil = 164.64 g

Water content = 17.76 %

Sample (1).

Normal Stresses	Shear Stresses
10.44 **	11.73 **
8.75	10.99
7.06	10.52
5.37	6.95
3.68	3.71
1.99	2.90

Sample (2).

Normal Stresses	Shear Stresses
10.44	11.86
8.75	11.19
7.06	10.65
5.37	7.82
3.68	4.65
1.99	2.70

Sample (3).

Normal Stresses	Shear Stresses
10.44	11.66
8.75	10.79
7.06	10.18
5.37	7.08
3.68	3.50
1.99	2.09

** **Normal stresses and shear stresses are in psi.**

Table (5) Results of direct shear tests (cont.)

Level II. Dry density = 100.4 % max. dry density by standard Proctor test

Weight of wet soil = 156.41 g

Water content = 17.39 %

Sample (1).

Normal Stresses	Shear Stresses
10.44	10.86
8.75	10.32
7.05	9.78
5.37	9.24
3.68	4.59
1.99	3.17

Sample (2).

Normal Stresses	Shear Stresses
10.44	10.65
8.75	9.64
7.06	9.04
5.37	8.16
3.68	6.61
1.99	4.32

Sample (3).

Normal Stresses	Shear Stresses
10.44	11.46
8.75	11.06
7.06	9.64
5.37	8.36
3.68	7.08
1.99	4.99

Table (6) Results of direct shear tests (cont.)

Level III. Dry density = 95.2 % max. dry density by standard Proctor test

Weight of wet soil = 148.18 g

Water content = 17.27 %

Sample (1).

Normal Stresses	Shear Stresses
10.44	10.65
8.75	9.78
7.06	9.23
5.37	9.10
3.68	6.74
1.99	4.72

Sample (2).

Normal Stresses	Shear Stresses
10.44	10.92
8.75	10.52
7.06	9.04
5.37	8.43
3.68	5.66
1.99	3.84

Sample (3).

Normal Stresses	Shear Stresses
10.44	10.32
8.75	9.44
7.06	9.17
5.37	7.62
3.68	6.07
1.99	3.71

Table (7) Results of direct shear tests (cont.)

Level IV. Dry density = 90.2 % max. dry density by standard Proctor test

Weight of wet soil = 139.95 g

Water content = 16.95 %

Sample (1).

Normal Stresses	Shear Stresses
10.44	10.25
8.75	9.31
7.06	8.63
5.37	7.55
3.68	6.27
1.99	3.51

Sample (2).

Normal Stresses	Shear Stresses
10.44	9.98
8.75	9.24
7.06	9.10
5.37	6.88
3.68	7.75
1.99	6.41

Sample (3).

Normal Stresses	Shear Stresses
10.44	10.86
8.75	8.63
7.06	7.75
5.37	7.48
3.68	7.08
1.99	4.86

Table (8) Mean values of shear stresses corresponding to normal stresses for each level of dry density.

Level I. Dry density = 105.3 % max. dry density by standard Proctor test

Normal Stresses	Mean Values of Shear Stresses
10.44	11.75
8.75	10.99
7.06	10.45
5.37	7.28
3.68	3.95
1.99	2.56

Level II. Dry density = 100.4 % max. dry density by standard Proctor test

Normal Stresses	Mean Values of Shear Stresses
10.44	10.99
8.75	10.34
7.06	9.49
5.37	8.59
3.68	6.09
1.99	4.16

Level III. Dry density = 95.2 % max. dry density by standard Proctor test

Normal Stresses	Mean Values of Shear Stresses
10.44	10.63
8.75	9.91
7.06	9.15
5.37	8.38
3.68	6.16
1.99	4.09

Level IV. Dry density = 90.2 % max. dry density by standard Proctor test

Normal Stresses	Mean Values of Shear Stresses
10.44	10.36
8.75	9.06
7.06	8.49
5.37	7.30
3.68	7.03
1.99	4.93

Table (9) The results of consolidation test - level I.

Level I. Dry density = 105.8 % of max. dry density by standard Proctor density test
Water content = 17.24 %
Initial void ratio = 0.5528
Total change of void ratio = 0.1253
The g.m. of k = 0.8924×10^{-6} cm/sec.

P	R_f	H	T_{90}	C_v	H_v	e	a_v	k
$\frac{1}{4}$	231	3692	23.1	3.2792	2439	0.5050	19.580	4.1349
$\frac{1}{2}$	308	3615	21.6	3.3809	2362	0.4890	6.554	1.4723
1	401	3573	21.6	3.2716	2269	0.4698	3.932	0.8640
2	496	3526	21.6	3.1911	2174	0.4510	1.925	0.4180
4	605	3475	21.6	3.1036	2065	0.4275	1.203	0.2574

where P = total pressure, ton/sq.ft

R_f = final dial reading, 10^{-4} in.

H = half thickness, 10^{-4} in.

T_{90} = 90 % consolidation time, sec.

C_v = coefficient of consolidation, 10^{-3} sq.cm/sec.

H_v = height of void ratio, 10^{-4} in.

e = void ratio

a_v = coefficient of compressibility, sq.cm/g.

k = permeability, 10^{-6} cm/sec.

g.m., the geometric mean of n observations (positive numbers) is the nth root of their product, or

$$\text{g.m. of } x = (x_1 \cdot x_2 \cdot \cdots \cdot x_n)^{1/n}$$

Table (10) The results of consolidation test - level II.

Level II. Dry density = 100.5 % of max. dry density by standard Proctor density test

Water content = 17.24 % Total change of void ratio = 0.1375

Initial void ratio = 0.6347 The g.m. of k = 0.9817×10^{-6} cm/sec.

P	R_f	H	T_{90}	C_v	H_v	e	a_v	k
$\frac{1}{4}$	211	3697	20.8	3.6467	2701	0.5887	18.843	4.2032
$\frac{1}{2}$	297	3623	21.6	3.3929	2615	0.5700	7.660	1.6359
1	399	3576	21.6	3.2817	2513	0.5477	4.567	0.9547
2	505	3524	21.6	3.1920	2407	0.5246	2.366	0.4879
4	631	3466	21.6	3.0939	2281	0.4972	1.403	0.2847

Table (11) The results of consolidation test - level III.

Level III. Dry density = 92.6 % max. dry density by standard Proctor density test

Water content = 13.83 %   Total change of void ratio = 0.2051

Initial void ratio = 0.7739   The g.m. if k = 1.2295×10^{-6} cm/sec.

P	R_f	H	T_{90}	C_v	H_v	e	a_v	k
$\frac{1}{4}$	102	3725	20.8	3.6742	3110	0.7356	15.699	3.2496
$\frac{1}{2}$	153	3686	21.6	3.4413	3059	0.7235	4.957	0.9828
1	300	3637	21.6	3.3304	2912	0.6887	7.128	1.3773
2	539	3540	21.6	3.1741	2673	0.6322	5.786	1.0876
4	807	3414	21.6	2.9521	2405	0.5688	3.246	0.5872

Table (12) The results of consolidation test - level IV.

Level IV. Dry density = 86.4 % of max. dry density by standard Proctor density test

Water content = 14.89 % Total change of void ratio = 0.2610

Initial void ratio = 0.9016 The g.m. of k = 1.5370×10^{-6} cm/sec.

P	R_f	H	T_{90}	C_v	H_v	e	a_v	k
$\frac{1}{4}$	166	3709	20.8	3.6585	3390	0.8595	17.245	3.3178
$\frac{1}{2}$	637	3549	20.8	3.4640	2919	0.8209	15.812	2.9455
1	768	3399	21.6	3.0568	2788	0.7840	7.558	1.2720
2	1036	3299	20.8	2.9501	2520	0.7087	7.711	1.2752
4	1279	3172	21.6	2.6515	2278	0.6406	3.487	0.5411

ANALYSIS OF DATA

(A). Compaction test

From the results of compaction tests, the maximum dry density vs. compactive energy per volume is shown in Fig. (7). It is noted that experimental data are almost on a straight line on that figure. That means the maximum dry density of compaction nearly linearly relates to compactive energy per volume in the ranges near 100 % standard Proctor density.

(B). Direct shear test

The shear strength of soil is generally expressed by the Coulomb's linear equation as described previously. With this knowledge, the results of experiment graphed in Fig. (8) to Fig. (11) are fitted by a straight line. The best way to find this straight line is using the least squares method (see Appendix A) which can search for a optimum relationships among variables and minimeze the error. It is then easy to calculate the values of cohesion and angle of internal friction. The computation were made by digital computer. The computer program and output are shown in Appendix B. This program provides sufficient data to find the correlations of angle of internal friction and void ratio which are shown in Fig. (12), and Fig. (13) respectively. The angle of internal friction, ϕ in terms of void ratio, e is found approximately to be the equation

$$\phi = 86.67 - 68.86\, e$$

where ϕ is in degrees.

This equation shows that the angle of internal friction increases by about 6.9 degrees for a 0.1 decrease in void ratio of the silty sand investigated.

However, the correlation of cohesion and dry density of soil becomes ambiguous, because of variation in cohesion vs. different dry density. The Fig. (14) shows that increasing in dry density will cause a decrease in the cohesion. It was not possible to explain this phenomena in a logical way.

(C). Consolidation test

The e-log P curve for each sample is shown in Fig. (15) thru Fig. (18) respectively. This shows that e-log P curve is generally a straight line. This means the structure of soil was not destroyed or reorientated during the consolidation tests. Four curves are almost parallel. With the results of computed values in Table (9) to Table (12), the total change of void ratio for each test is shown in Fig. (19). It is noted that the increase in dry density will reduce the total change of void ratio under the loading increment as expected. It is also noted that the difference of total change of void ratio of 100.5 % of maximum dry density determined by standard Proctor test and 105.8 % of Proctor density is very small. This means if the settlement is regarded as major factor for design, the 100 % maximum dry density of standard Proctor test will be economically worthwhile.

As mentioned previously, the Terzaghi's one-dimensional consolidation theory assumed constant values for certain soil properties which actually vary somewhat with pressures. From Table (9) to Table (12), the permeability, k varies with pressures contrary to the assumption of Terzaghi's one-dimensional consolidation theory. The permeability varies with different dry density as shown in Fig. (20). The permeability relates to dry density inversely, or relates to compactive energy inversely.

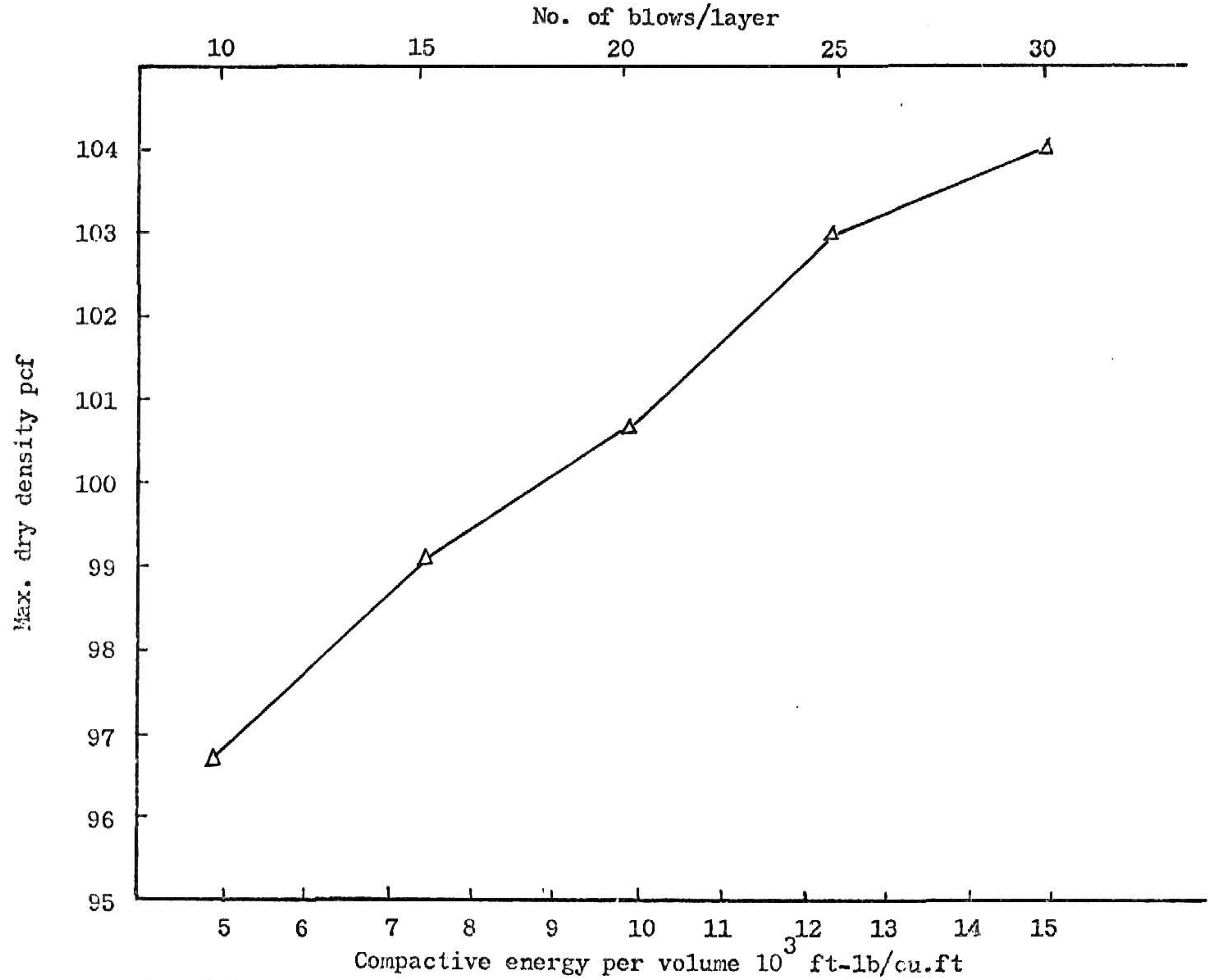

Fig. (7) Max. dry density of compaction test vs. compactive energy per volume and no. of blows per layer.

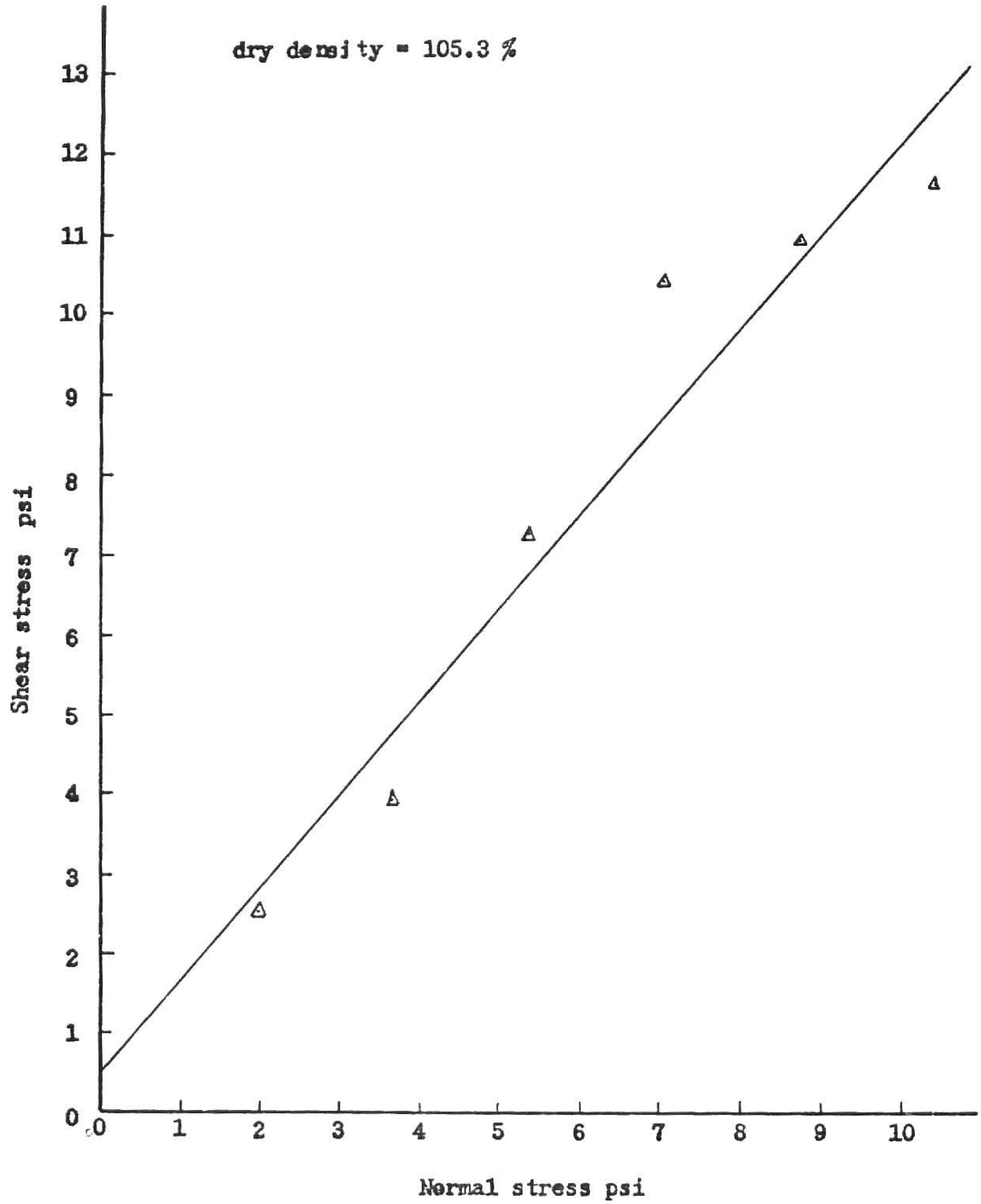

Fig. (8) Relationship of shear stress and normal stress - level I.

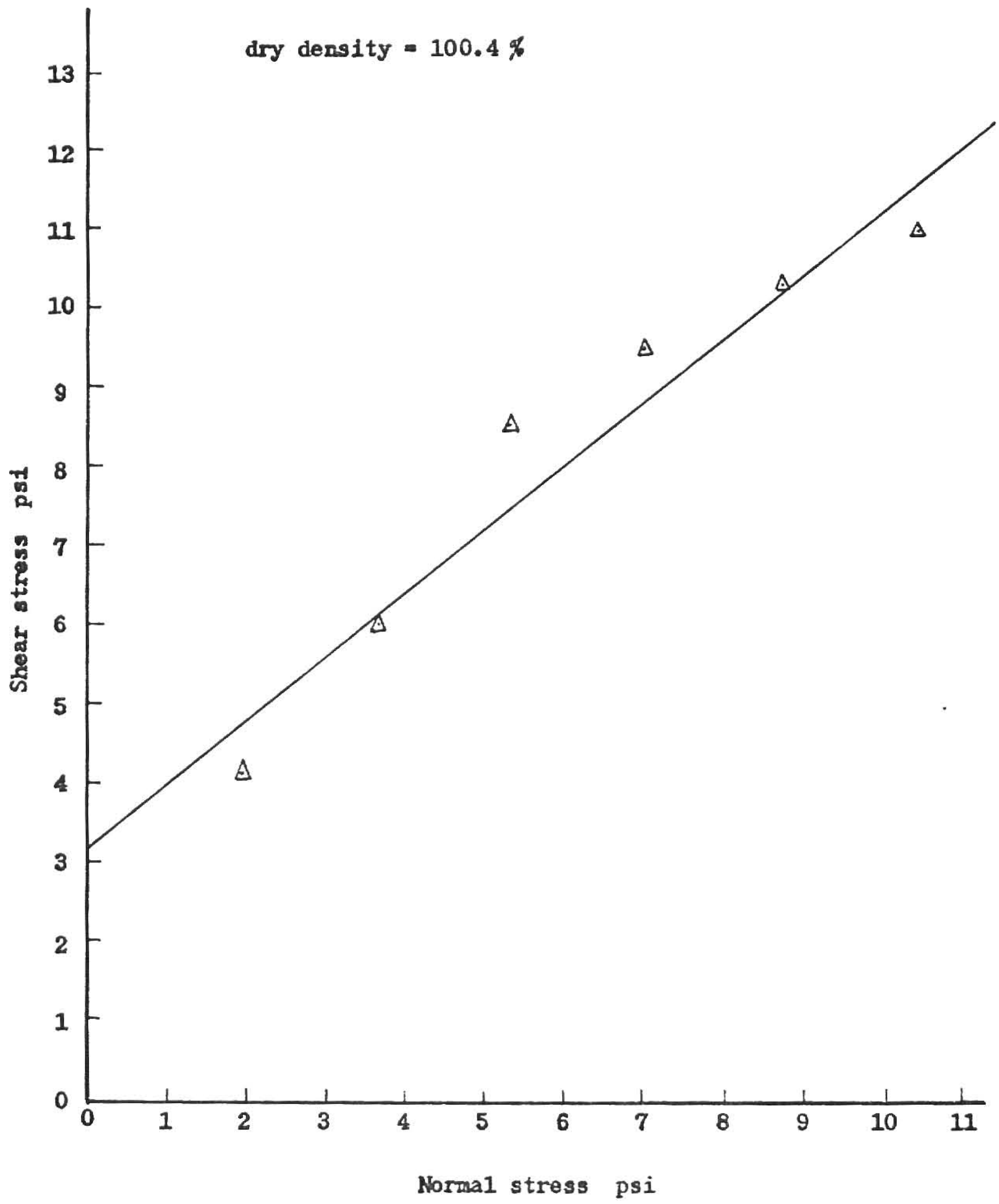

Fig. (9) Relationship of shear stress and normal - level II.

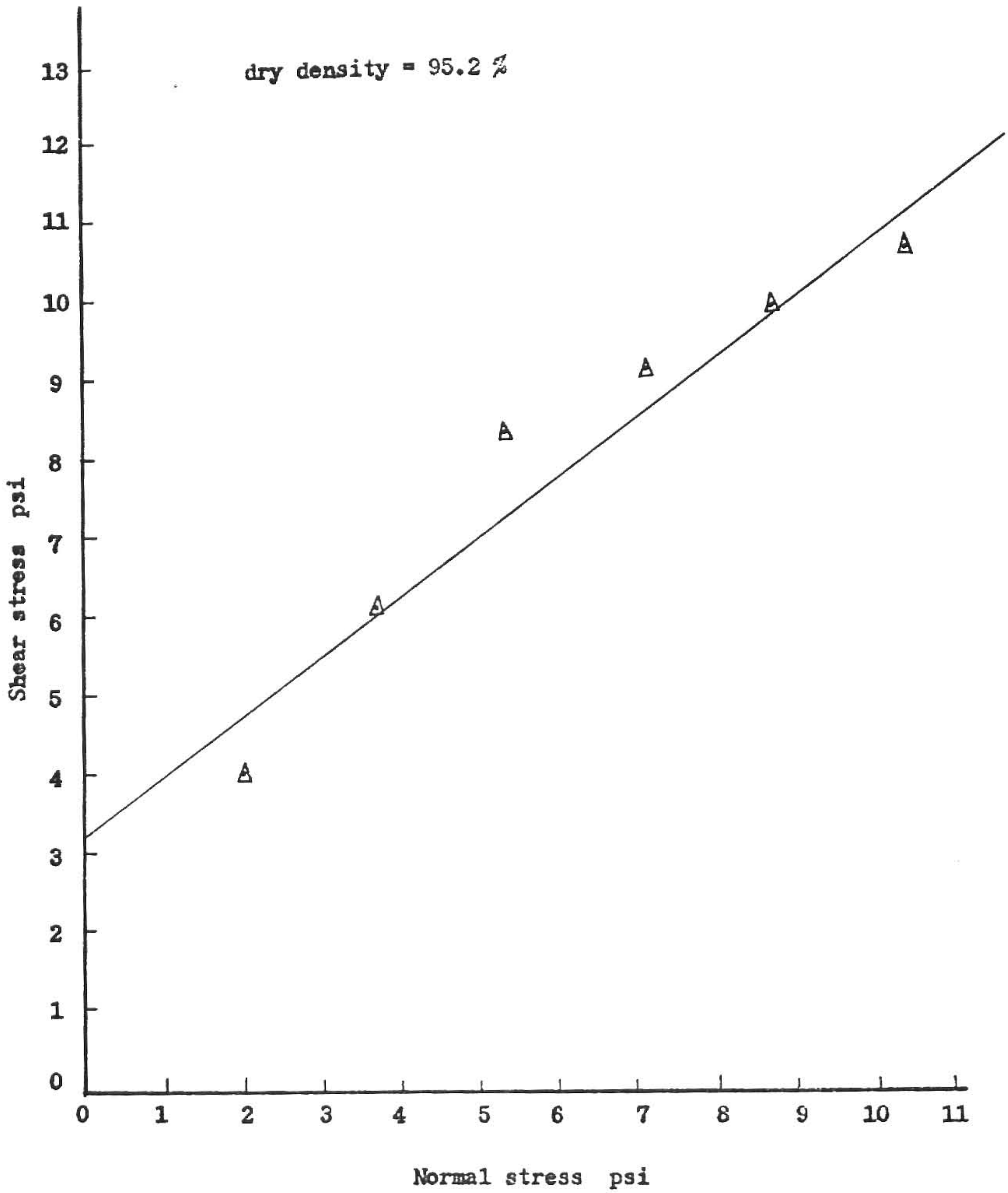

Fig. (10) Relationship of shear stress and normal stress - level III.

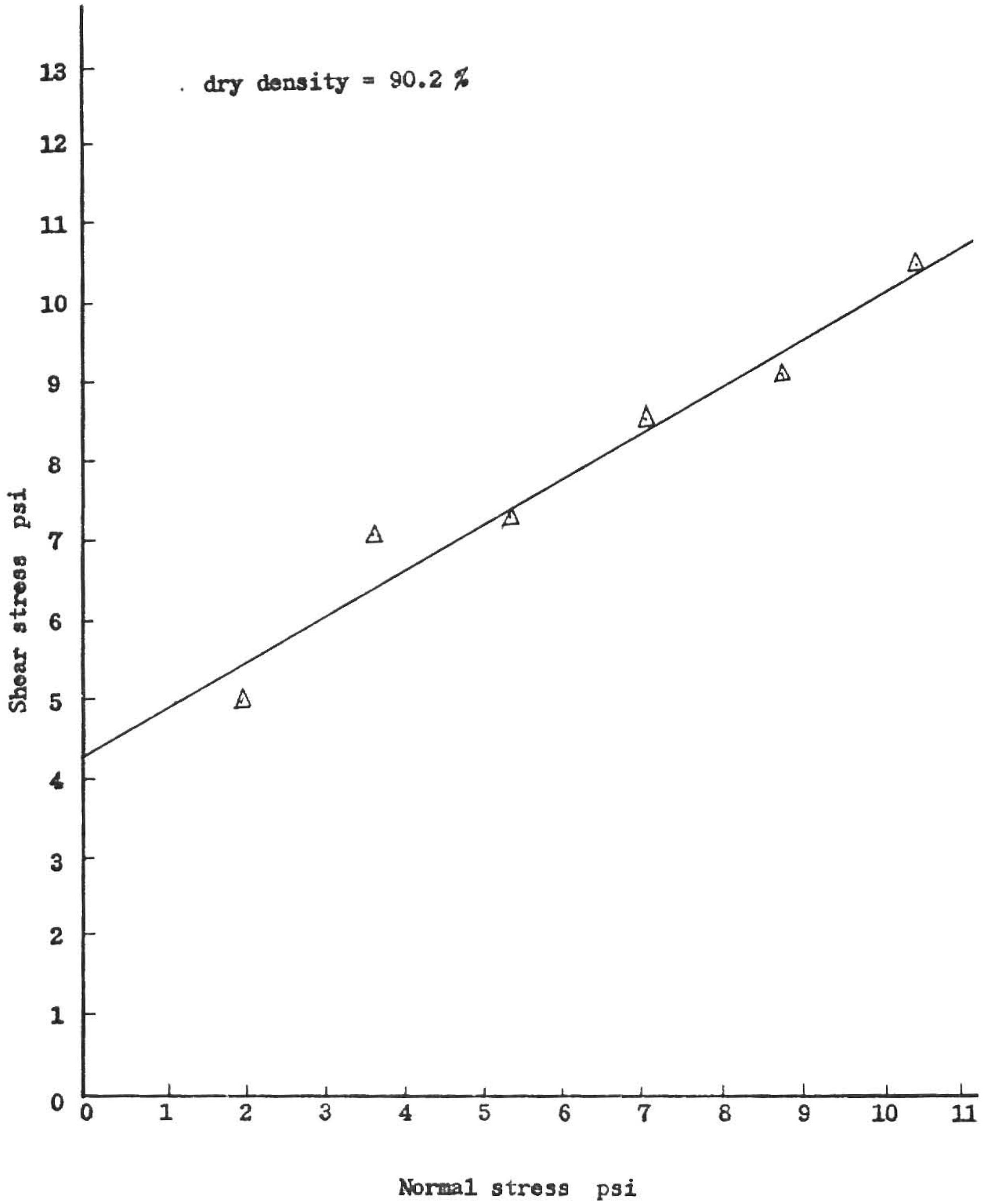

Fig. (11) Relationship of shear stress and normal stress - level IV.

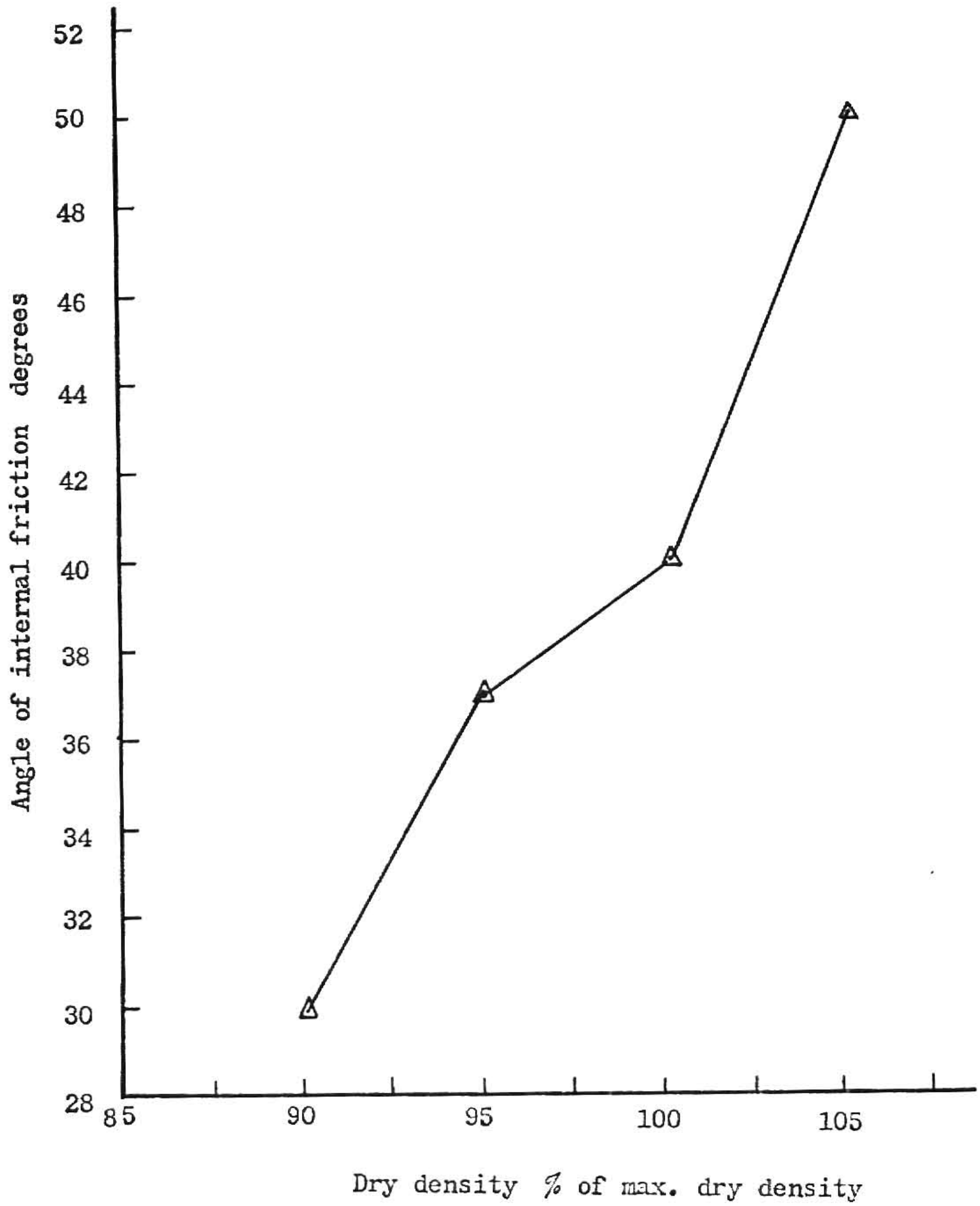

Fig. (12) Angle of internal friction vs. dry density.

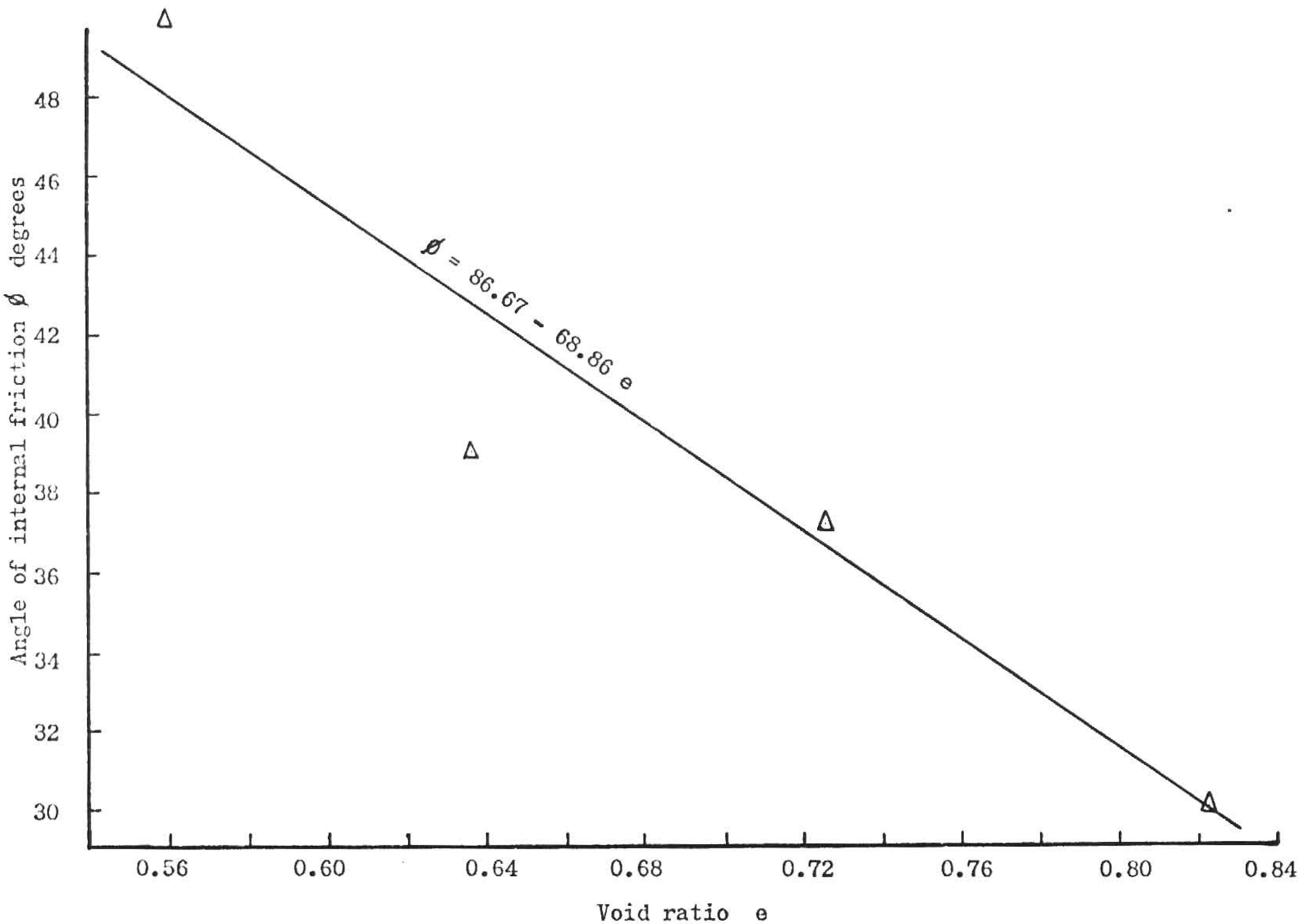

Fig. (13) Angle of internal friction vs. void ratio.

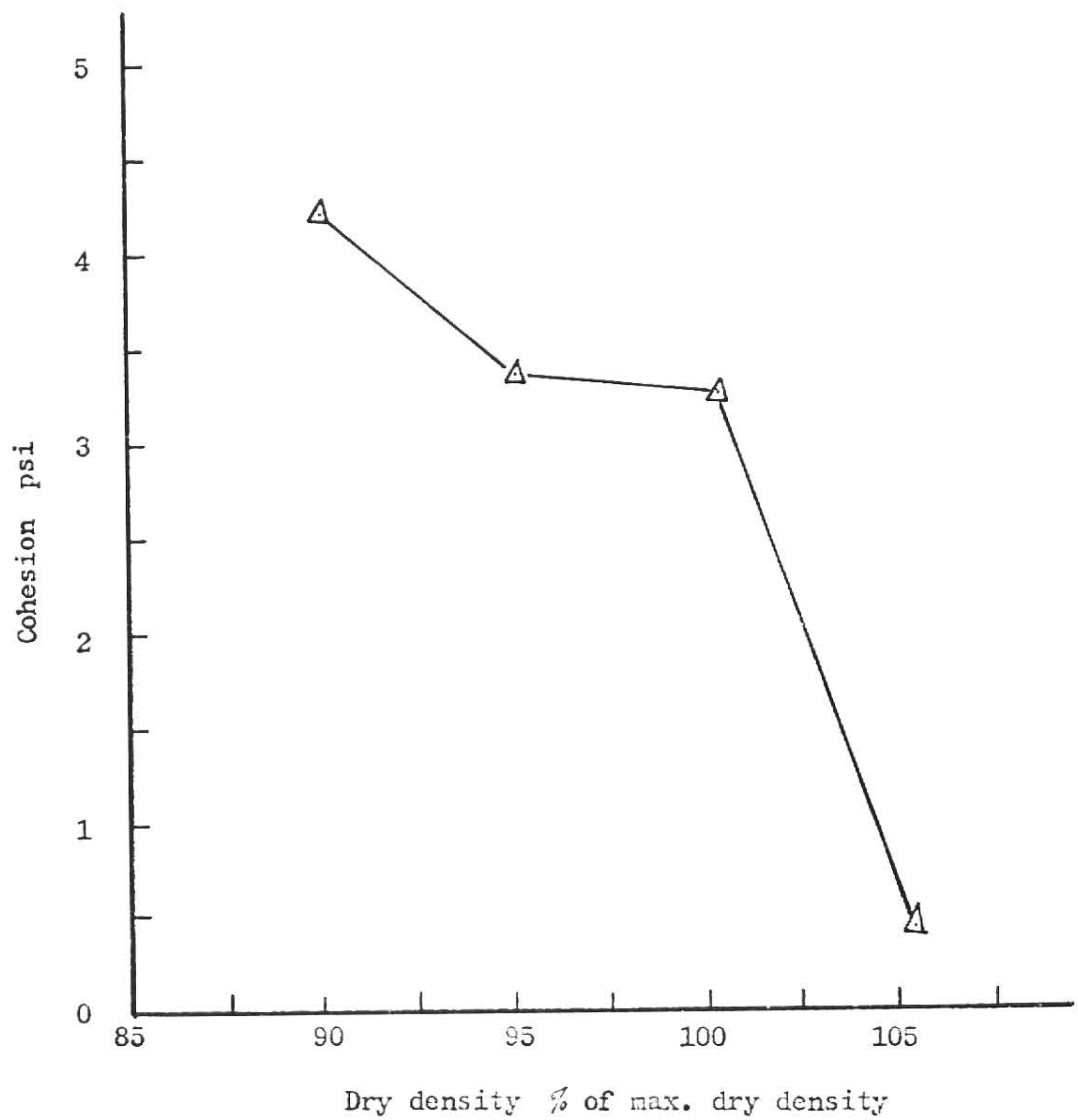

Fig. (14) Cohesion vs. dry density.

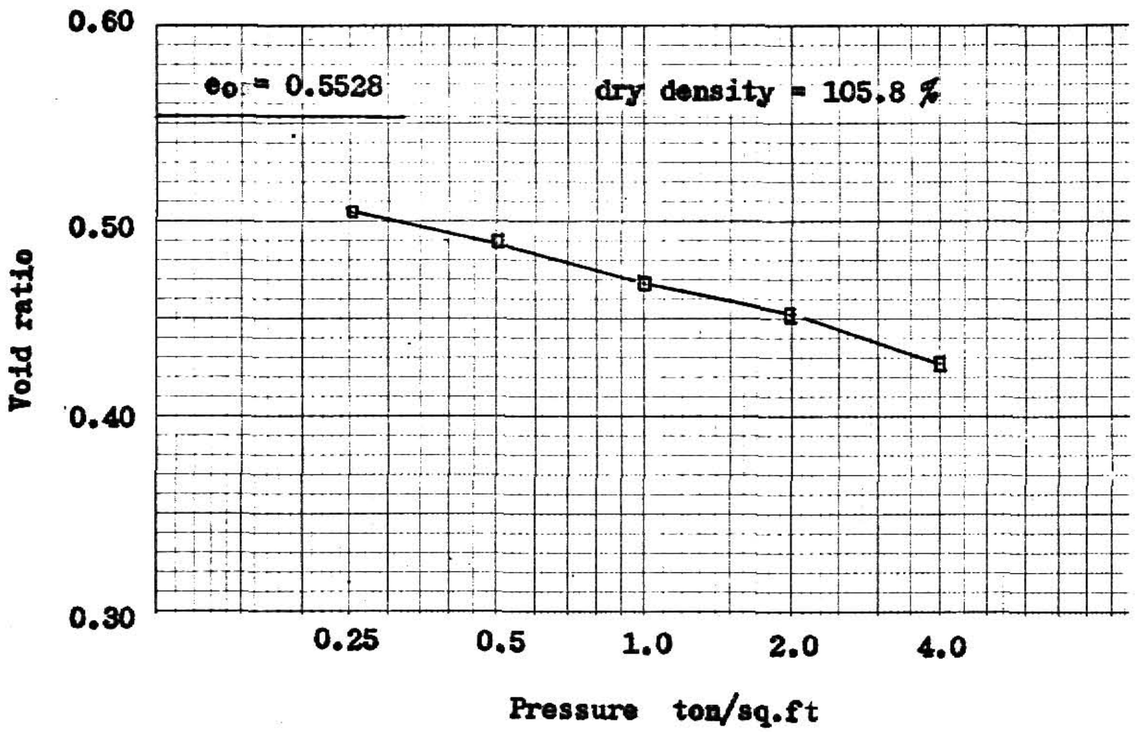

Fig. (15) The e-log P curve - level I.

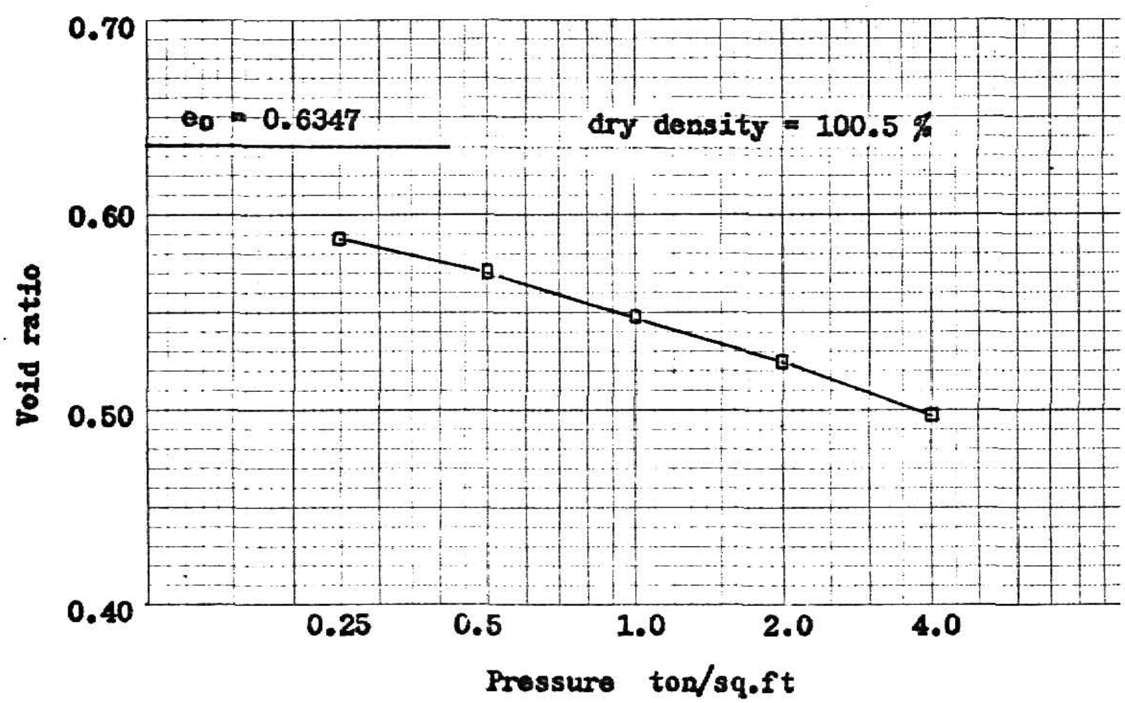

Fig. (16) The e-log P curve - level II.

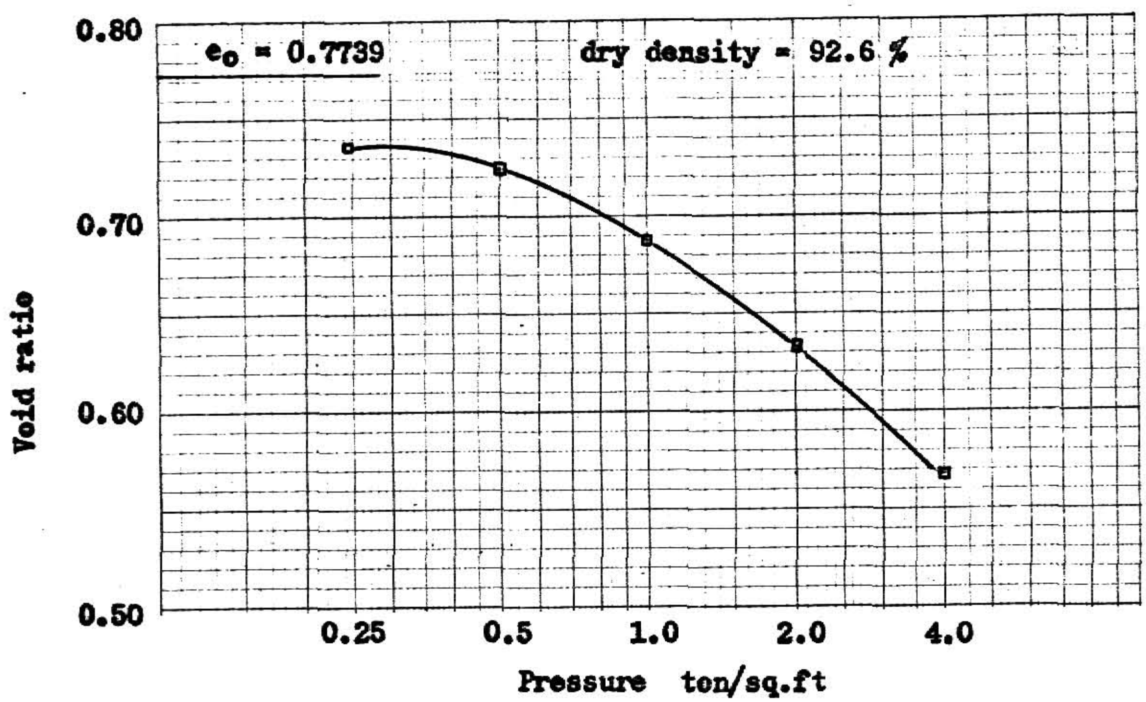

Fig. (17) The e-logP curve - level III.

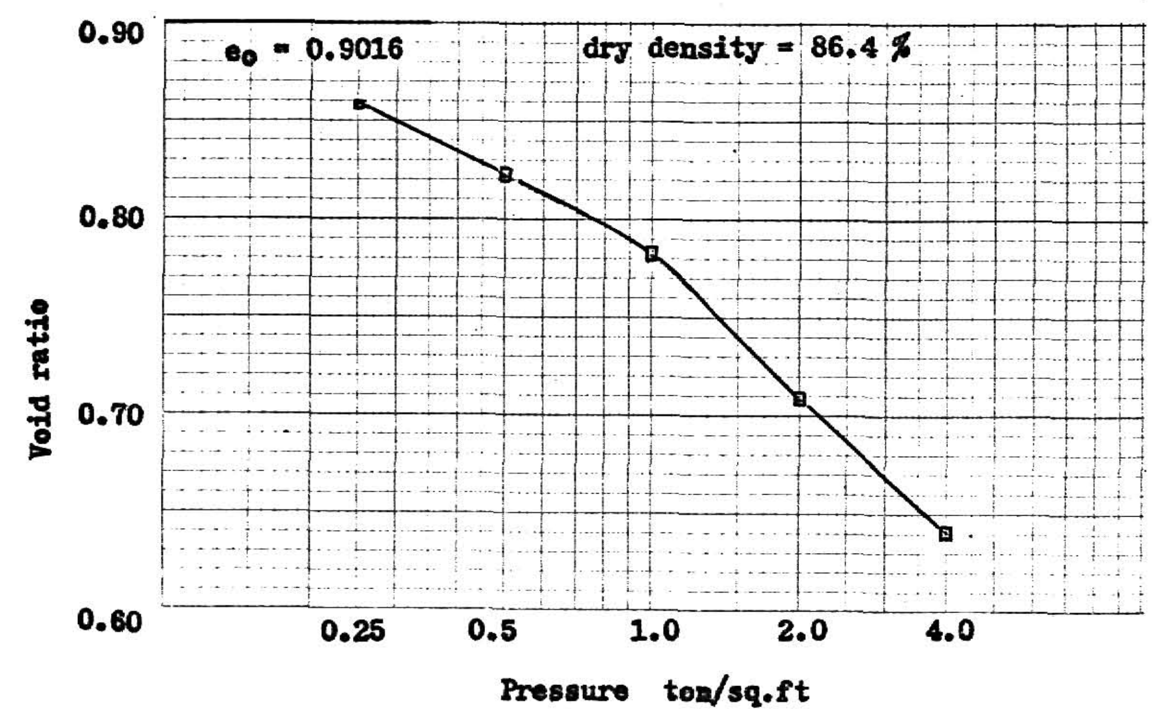

Fig. (18) The e-logP curve - level IV.

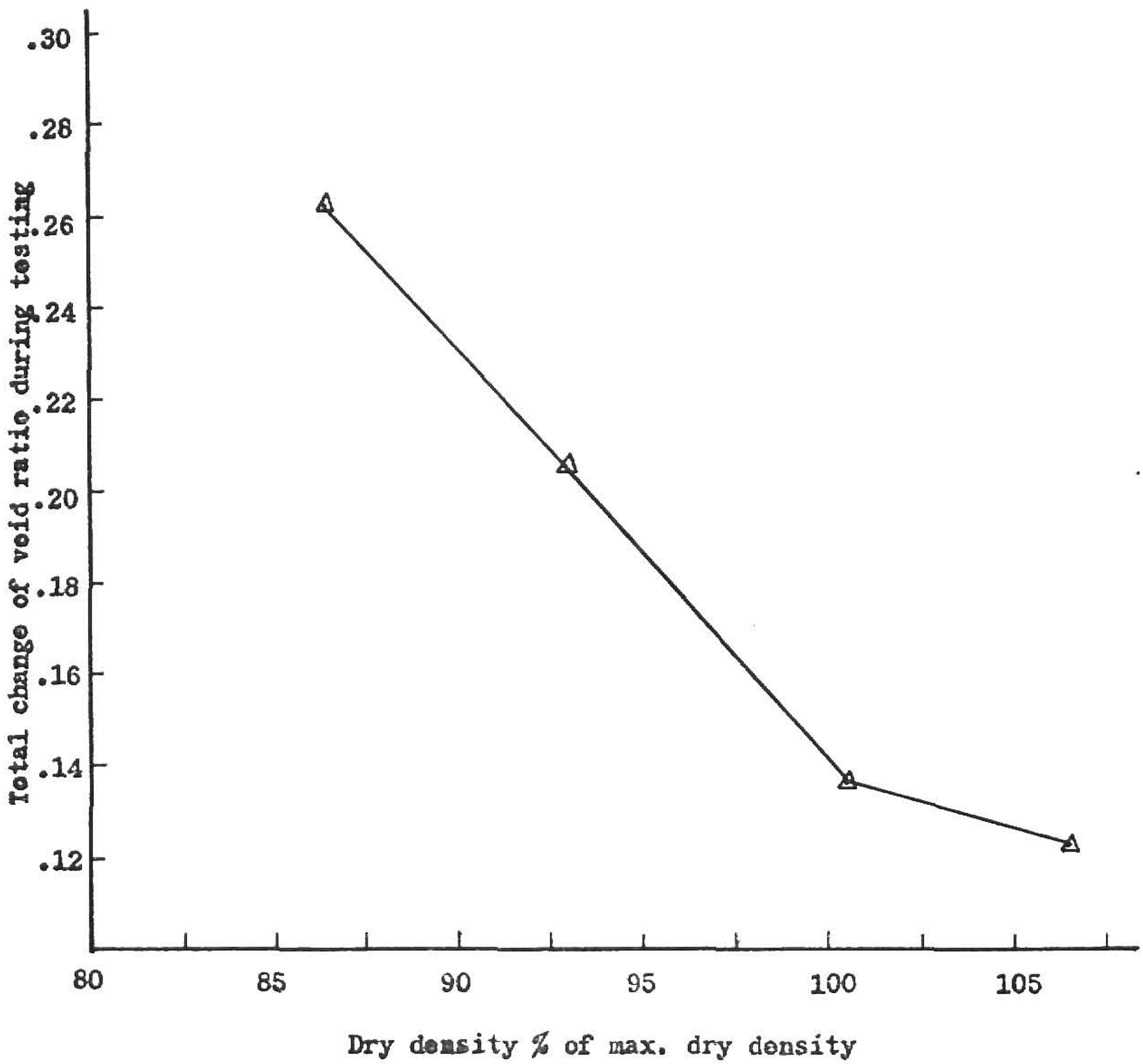

Fig. (19) Total change of void ratio during consolidation testing vs. dry density.

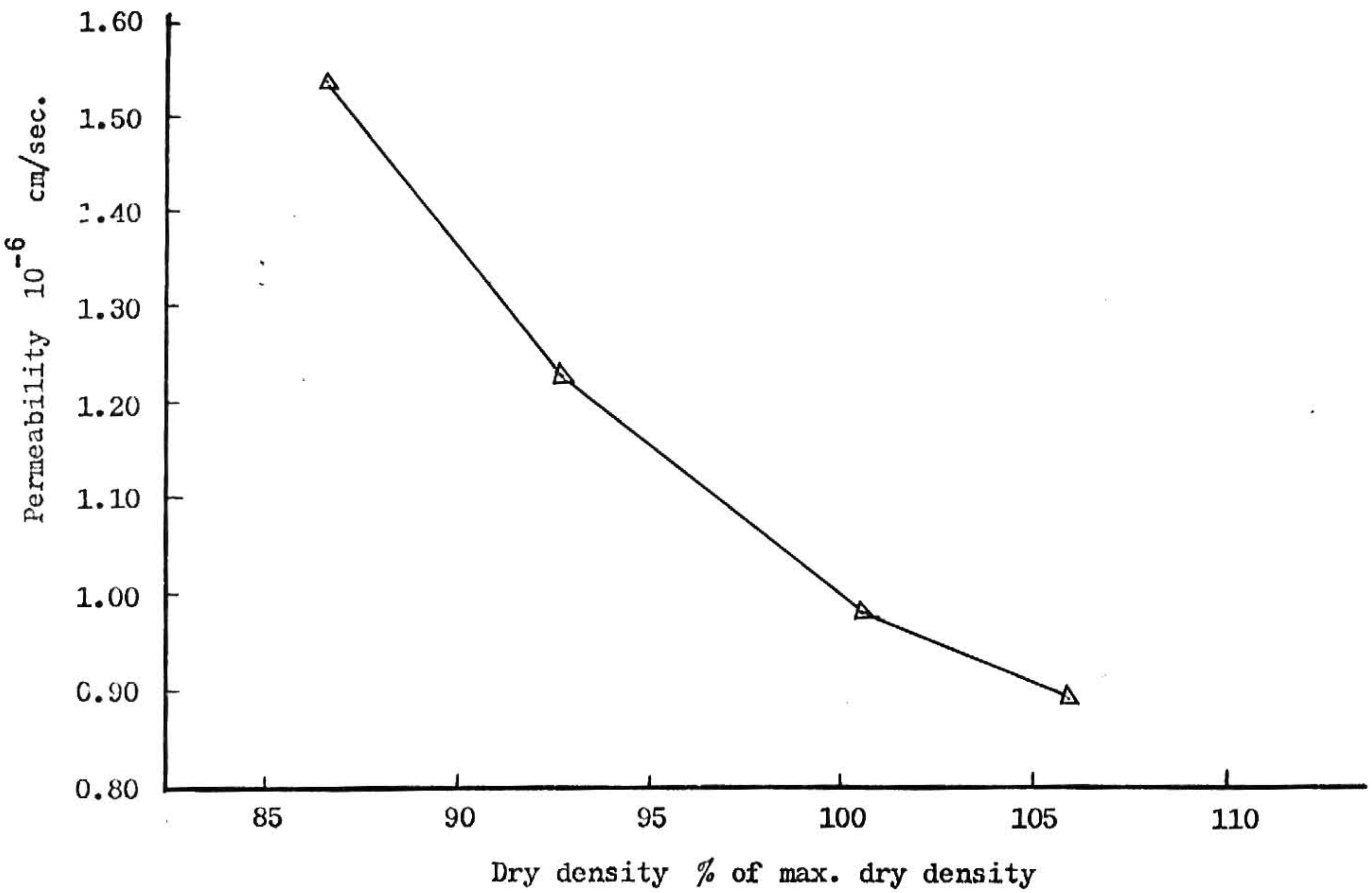

Fig. (20) Permeability (geometric mean value) vs. dry density

CONCLUSIONS

(a). The maximum dry density of compaction is nearly linearly related to compactive energy per volume for this silty sand studied between some ranges of compactive energy.

(b). The change in the angle of internal friction increases by about 6.8 degrees for a 0.1 decrease in void ratio of a silty sand investigated in the ranges which dry density of soil is around 100 % of standard Proctor density.

(c). The contribution of compactive energy to cohesion of soil becomes ambiguous during compaction, because of reduction of cohesion with greater dry density.

(d). The increase in dry density, i.e. increase in compactive energy will reduce the total change of void ratio during consolidation test.

(e). Maximum dry density of standard Proctor test is a economical criterion to design fill considering the settlement as major factor.

(f). The permeability of soil varys with different pressures during consolidation that is contrary to assumption of Terzagh's one-dimensional consolidation theory.

(g). The permeability inversely relates to dry density, or relates to compactive energy inversely.

RECOMMENDATIONS FOR FURTHER STUDY

Direct shear test is simple and easy to determine the shear strength of soil. But it has disadvantages: (a). assuming failure plane which does not occur on the actual failure plane. (b). variation in friction between the contact surfaces of upper and lower sample boxes. It causes low accuracy and even lack of reliability in results of investigations. Therefore it is further recommended that using other procedure, such as triaxial compression test, will be satisfactory. Because of versatility of triaxial compression test, it has advantages over other shear tests to duplicate field conditions in the laboratory study.

From the result of direct shear tests shows that the reduction of cohesion with greater dry density. It can not be interpreted in a logical way and should be investigated further.

It is also further recommended that a study in application of laboratory investigations to predict the shear strength and compressibility on field work which takes into consideration the soil types and conditions and states of stresses in situ. That will make this study more meaningful.

ACKNOWLEDGMENT

The author expresses most sincere appreciation and thanks to the advice, guidance, and review by the major professor of this study, Professor Wayne W. Williams, Department of Civil Engineering, Kansas State University. Without his encouragement and help, this investigation could not be completed.

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APPENDIX A

Least Squares Method

Consider the problem of fitting a given number of observation values (see Table A.1) by a straight line in the form

$$y = C + K x \qquad \text{Eq. (1)}$$

Table A.1

x	y
x_1	y_1
x_2	y_2
.	.
.	.
x_n	y_n

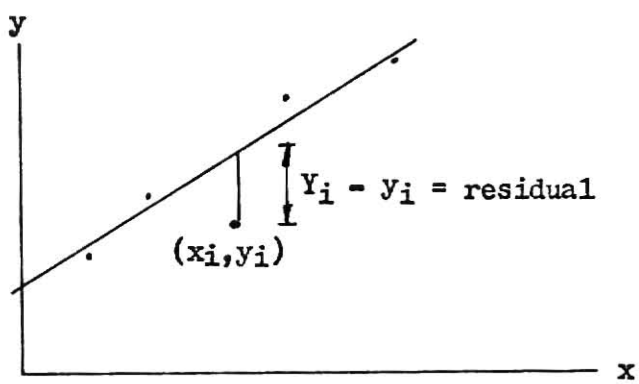

Fig. A.1

Substituting x's and y's into the Eq. (1) to determine two coefficients C and K gives the overdeterminated linear equations, if n is greater than 2. But the straight line can plotted (see Fig. A.1) which requires that $S = \sum(Y_i - y_i)^2$ be a minimum, where Y_i is evaluated from Eq. (1). The values $Y_i - y_i$ are called residuals.

To obtain the minimum value for S, which is a function of two variables C and K, it is necessary to set the following two first partial derivatives to zero:

$$\frac{\partial S}{\partial C} = 0 \quad \text{and} \quad \frac{\partial S}{\partial K} = 0$$

This yeilds the two simultaneous linear equations:

$$\frac{\partial S}{\partial C} = \sum_{i=1}^{n} \frac{\partial}{\partial C}(C + K x_i - y_i)^2 = \sum_{i=1}^{n} 2(C + K x_i - y_i) = 0$$

and

$$\frac{\partial S}{\partial K} = \sum_{i=1}^{n} \frac{\partial}{\partial K}(C + K x_i - y_i) = \sum_{i=1}^{n} 2\, x \cdot (C + K x_i - y_i) = 0$$

From above equations gives

$$\sum_{i=1}^{n} y_i = n\,C + K \sum_{i=1}^{n} x_i$$

and

$$\sum_{i=1}^{n} x_i\, y_i = C \sum_{i=1}^{n} x_i + K \sum_{i=1}^{n} x_i^2$$

These two equations represent two conditions which must be met in order to obtain the best fit of the straight line, based on the least squares criterion.

```
$JOB
C***          THIS PROGRAM, USING LEAST SQUARES METHOD,
C***          IS DESIGNED TO FIND THE COHESION AND
C***          INTERNAL FRICTION ANGLE OF SOIL
C***          WHICH RESULT FROM DIRECT SHEAR TESTS.
          DIMENSION X(30), Y(30)
          REAL K, K1
          WRITE(6,200)
      200 FORMAT(1H1,///8X,'     ***** ***** ** ******      ')
          READ(5,1)M
        1 FORMAT(I5)
          DO 100 J=1,M
          SUMX=0.00
          SUMY=0.00
          SQUX=0.00
          SXY=0.00
          READ,W
          WRITE (6,2) J, W
        2 FORMAT (///8X,'LEVEL =',I2,3X,'DRY DENSITY =',F6.1,
         1 ' PERCENT'/8X,'BY STANDARD PROCTOR DENSITY TEST')
          READ(5,1) N
          WRITE(6,3) N
        3 FORMAT(/8X,'THE NUMBER OF DATA POINTS =',I3/8X,
         - 'THE VALUES OF EXPERIMENTAL DATA'/12X,'X(I)',
         - 13X,'Y(I)')
          DO 80 I=1,N
          READ(5,4) X(I),Y(I)
        4 FORMAT(2F10.4)
          WRITE(6,5) X(I),Y(I)
        5 FORMAT(F16.2,F17.2)
          SUMX=SUMX+X(I)
          SUMY=SUMY+Y(I)
          SQUX=SQUX+X(I)*X(I)
       80 SXY=SXY+X(I)*Y(I)
          A=N
          DET=A*SQUX-SUMX*SUMX
          IF(ABS(DET)-0.0001) 7,6,7
        6 K1=(Y(N)-Y(1))/(X(N)-X(1))
          K=Y(N)-K1*X(N)
          GO TO 8
        7 K=(SUMY*SQUX-SUMX*SXY)/DET
          K1=(A*SXY-SUMX*SUMY)/DET
        8 ANG=ATAN(K1)
          ANG=ANG*57.29578
          WRITE(6,9) K,ANG
        9 FORMAT( /8X,'COHESION = ', F8.4, 2X,'PSI',/8X,
         1 'INTERNAL FRICTION ANGLE =', F8.4, 2X,'DEGREES')
      100 CONTINUE
          WRITE (6,10)
       10 FORMAT ('1')
          STOP
          END

$ENTRY
```

```
****************

LEVEL = 1    DRY DENSITY = 105.3 PERCENT
BY STANDARD PROCTOR DENSITY TEST

THE NUMBER OF DATA POINTS =   6
THE VALUES OF EXPERIMENTAL DATA
     X(I)                Y(I)
    10.44               11.75
     8.75               10.99
     7.06               10.45
     5.37                7.28
     3.68                3.95
     1.99                2.56

COHESION =     0.4498  PSI
INTERNAL FRICTION ANGLE = 49.8988   DEGREES


LEVEL = 2    DRY DENSITY = 100.4 PERCENT
BY STANDARD PROCTOR DENSITY TEST

THE NUMBER OF DATA POINTS =   6
THE VALUES OF EXPERIMENTAL DATA
     X(I)                Y(I)
    10.44               10.99
     8.75               10.34
     7.06                9.49
     5.37                8.59
     3.68                6.09
     1.99                4.16

COHESION =     3.2542  PSI
INTERNAL FRICTION ANGLE = 38.9424   DEGREES


LEVEL = 3    DRY DENSITY =  95.2 PERCENT
BY STANDARD PROCTOR DENSITY TEST

THE NUMBER OF DATA POINTS =   6
THE VALUES OF EXPERIMENTAL DATA
     X(I)                Y(I)
    10.44               10.62
     8.75                9.91
     7.06                9.15
     5.37                8.38
     3.68                6.16
     1.99                4.39

COHESION =     3.3645  PSI
INTERNAL FRICTION ANGLE = 37.0910   DEGREES
```

```
LEVEL = 4    DRY DENSITY =  90.2 PERCENT
BY STANDARD PROCTOR DENSITY TEST

THE NUMBER OF DATA POINTS =  6
THE VALUES OF EXPERIMENTAL DATA
    X(I)               Y(I)
   10.44              10.36
    8.75               9.06
    7.06               8.49
    5.37               7.30
    3.68               7.03
    1.99               4.93

COHESION =    4.2440  PSI
INTERNAL FRICTION ANGLE = 30.2030  DEGREES
```

THE EFFECTS OF COMPACTION ON SHEAR STRENGTH AND ONE-DIMENSIONAL CONSOLIDATION OF A SILTY SAND

by

CHING-KNOWN WILLIAM WANG

Diploma in Civil Engineering,

Taiwan Provincial Taipei Institute of Technology,
Taiwan, China, 1967

AN ABSTRACT OF A MASTER'S THESIS

sumitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas
1975

ABSTRACT

Shear strength and compressibility are two major topics in soil mechanics and foundation. In general, compaction densifies the earth to increase the shear strength and reduce the compressibility of soil. The purpose of this study was to investigate the effects of compaction on shear strength and one-dimensional consolidation of silty sand. Specifically the laboratory study provided a procedure to show the relationships of shear strength and compressibility to maximum dry density for a selected soil.

Five compaction test series with varying compactive energy were run to obtain maximum dry density for each series. Direct shear and one-dimensional consolidation tests were made for these different levels of dry densities and the variations of shear strength and compressibility of soil were determined.

The testing data was analyzed by the least squares method which can determine an optimum relationship among variables. The results of the study showed that increasing the dry density will cause an increase in th angle of internal friction and a decrease in compressibility of soil. The effects on the unit cohesion of the soil was ambiguous.