AN ABSTRACT OF THE THESIS OF

DONALD GORDON ANDERSON for the MASTER OF SCIENCE
(Name) (Degree)

in CIVIL ENGINEERING presented on May 5, 1968
(Major) (Date)

Title: CONSOLIDATION CHARACTERISTICS OF SAND-CLAY MIXTURES

Redacted for Privacy

Abstract approved: Dr. J. R. Bell

Combinations of cohesive and cohesionless soils are often encountered when evaluating the engineering properties of soils. The stress-volume characteristics of the mixed soil are found to vary according to the proportion of each component. If the cohesive soil composes more than a certain fraction of the material, the magnitude of volume change for a stress increment will be a function of the compressibility of the cohesive soil. When the amount of cohesive material is less than this fraction, volume change is controlled by the behavior of the cohesionless material.

Two methods have gained wide acceptance for predicting the behavior of soil mixtures. If more than 50 percent of the material is fine grained, the Unified Classification System defines the behavior of the system by the characteristics of the fine grained material. The American Association for State Highway Officials requires 35 percent

or more fine grained material to define fine grained behavior.

The purpose of this investigation was to establish a method for predicting the compressional characteristics of a mixture controlled by cohesive materials and to determine the percentage at which the cohesionless soil began to control the behavior.

The two sized system was first investigated theoretically. Relationships were established for predicting the behavior of the mixture in one dimensional consolidation and for determining the point of control by the cohesionless component. The validity of the theoretical approach was established by performing a laboratory investigation. The program included nine consolidation tests on specimens with varying proportions of sand and clay.

The results of the laboratory investigation indicated that the theoretical relationship for calculating the compressional characteristics was valid if the percentage of sand was less than 65 percent of the total weight of solids. Above 65 percent sand the cohesionless material dominated the behavior of the system making the theoretical relationship invalid.

Consolidation Characteristics of
Sand-Clay Mixtures

by

Donald Gordon Anderson

A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Master of Science

June 1969

APPROVED:

Redacted for Privacy

Professor of Civil Engineering

in charge of major

Redacted for Privacy

Head of Department of Civil Engineering

Redacted for Privacy

Dean of Graduate School

Date thesis is presented May 5, 1968

Typed by Clover Redfern for Donald Gordon Anderson

ACKNOWLEDGMENT

The author wishes to express gratitude to Dr. J. R. Bell for his patience, advice, suggestions, and assistance in directing the research and writing of this thesis. Thanks are also extended to Dr. W. L. Schroeder for his advice and to fellow graduated students for their encouragement.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF TERMS

Term	Description	Dimensions
C_c	Compression index.	dimensionless
C_{cm}	Compression index for mixture.	dimensionless
C_{cp}	Compression index for clay.	dimensionless
e	Void ratio.	dimensionless
e_o	Initial void ratio.	dimensionless
e_t	Void ratio at contact.	dimensionless
f	Percentage of clay.	dimensionless
ΔH	Change in height of the system.	inch
H_o	Initial height of the system.	inch
$H_{T_{1,2}}$	Total height at load "1, 2".	inch
H_c	Height of clay.	inch
H_{nc}	Height of nonclay.	inch
H_v	Height of voids.	inch
P_c	Apparent preconsolidation pressure.	kg/cm^2
P_o	Actual preconsolidation pressure.	kg/cm^2
V_c	Volume of clay.	ft^3
V_{nc}	Volume of nonclay.	ft^3
$V_{void\ nc}$	Volume of voids for nonclay.	ft^3
W_c	Weight of clay solids.	lb.
W_{nc}	Weight of nonclay solids.	lb.

Term	Description	Dimensions
$W_{\ell\ell}$	Liquid limit.	percent
$W_{c\ell\ell}$	Liquid limit for clay.	percent
X	Ratio of weight of sand to total weight	percent
σ_o'	Initial effective stress.	kg/cm^2
$\Delta\sigma'$	Change in effective stress.	kg/cm^2

CONSOLIDATION CHARACTERISTICS OF SAND-CLAY MIXTURES

INTRODUCTION

A soils problem of particular interest to the engineer is the displacement induced at the boundary of a soil deposit by application of building loads. The magnitude of this deformation often controls the foundation design of the structure.

Mathematical models based on stress-volume relationships have been developed for predicting settlements of structures founded on cohesive soils. Boundary displacements for cohesionless soils result more from deformations at constant volume than from densification. Since these conditions do not lend themselves to this mathematical treatment, foundations for structures on cohesionless soils are usually designed by more empirical methods. Difficulties may develop when dealing with mixtures of the two soils.

Rather than consider soils as mixtures, engineers usually classify soils as cohesive or cohesionless and design accordingly. The proportion of the fine to the coarse solid particles is used to classify the soil and define the mode of expected behavior. There is not, however, universal agreement on the relative proportion of materials which should be used to distinguish between the two types of soil.

The purpose of this investigation was to study the compression characteristics of sand-clay mixtures. Specifically, an investigation was performed to determine the relationship between the compression index and the sand content for a cohesive soil and to determine the sand content at which the soil properties were dominated by the cohesionless fraction.

A theoretical investigation defined a mathematical relationship for predicting the compression index. A similar theoretical approach was used to determine the percentage of cohesionless material necessary to control the settlement of the system. The validity of the mathematical relationships was determined by performing consolidation tests on eight mixtures containing different proportions of a uniform sand and a kaolinite clay.

The results of the laboratory study verified the theory for less than 65 percent sand. Above this percentage interaction of the sand particles altered the system's behavior significantly.

The investigation was concluded by comparing the results from the consolidation test to the relationships between the liquid limit and the compression index proposed by Skempton (1944).

THEORY

General

A decrease in volume usually occurs when the stresses acting on a soil system are increased. The decrease in volume causes displacements at the boundary of the soil mass which control, to a degree, the adequacy of structures founded on or composed of the soil.

A mathematical method for estimating the magnitude of these displacements was proposed by Karl Terzaghi in the early 1920's (Leonards, 1962). In his solution Terzaghi made four limiting assumptions: 1) the consolidation of the mass was unidimensional, 2) the solid particles were incompressible, 3) the volume of the solid particles remained constant, and 4) a functional relationship between the effective stress and the void ratio existed. If these four conditions were met, the change in height for a stress increase could be calculated from the following equation.

$$\Delta H = \frac{H_o}{1+e_o} C_c \log \left(\frac{\sigma_o' + \Delta\sigma'}{\sigma_o'}\right)$$

The solution is applicable for all soils whether cohesive or cohesionless, saturated or unsaturated.

All the components of Terzaghi's equation are readily obtainable except C_c, the compression index. The compression index is numerically equal to the rate of change of void ratio with respect to the common logarithm of the effective stress. A laboratory investigation is usually required for the determination of C_c.

If the material is cohesionless, the particles are naturally arranged in stable positions. By assuming unidimensional consolidation any volume decrease caused by the lateral movement of particles is prevented. Since particles are assumed incompressible, deflection in the vertical direction is zero. Therefore, when stresses are applied to the theoretical system, no volume change occurs. However, in reality a very small decrease in volume results from solid particles deforming elastically (Schultz and Moussa, 1961). The elastic deformation causes a slight change in void ratio which, in turn, defines a compressive index. But the magnitude of the volume change for the cohesionless material is so small that its determination is unwarranted.

The solid particles in a cohesive soil are arranged in unstable positions. To maintain the orientation relatively strong interparticle bonds must exist in the soil-water system. The bonded soil particles react to stress changes by slippage of particles into a denser packing. Rearrangement continues until the bonds of the structure are able to maintain the stress level. The size of the stress increase, the initial structure of the system, and the strength of the bonds determine the amount of rearrangement which, in turn, controls the magnitude of vertical deflection. To predict the deflection for the system, the compression index must be established. A laboratory investigation is usually necessary to define C_c for the cohesive soil.

Sand-Clay Mixture

Most soils in nature are combinations of cohesive and cohesionless materials. Depending upon the proportion of one to the other, deformation may or may not be an engineering problem. If the soil acts as a cohesionless material a laboratory investigation is unnecessary. But if the material is a cohesive soil, the compression index would be essential for predicting any volume change.

The proportion of clay minerals required to control the behavior is not well defined. Various organizations have used different percentages for predicting the expected performance of the soil.

In 1890, the U.S. Bureau of Soils introduced the Textural Classification System (Burmister, 1950). This system suggested that 30 percent or more clay caused the "larger granules to float in the clay mass." At lower percentages of clay, the soil was said to have a skeleton which tended to give it strength and stability. The Public Roads Administration presented another classification in 1928 (Oglesby and Hewes, 1963). The minimum percentage of coarse material required for subgrade stability was 55 percent. After several years of use the American Association of State Highway Officials (AASHO) modified the old Public Roads' classification to 65 percent granular material (Mitchell, 1956). The U.S. Army, in 1942, adopted a system of soil designation developed by Arthur Casagrande (1947). The system was originally referred to as the Airfield Classification and later renamed the Unified Soil Classification System. A coarse grained soil was defined as one having more than 50 percent of its particles retained on the Number 200 U.S. Standard sieve.[1] If more than 50 percent of the particles passed the Number 200 U.S. Standard sieve, the soil was a fine grained material.

The behavior of coarse grained soils is characteristic of cohesionless materials while fine grained soils with plasticity are considered cohesive. The variation in percentage for distinguishing cohesive from cohesionless material is obviously large. Since the cost of a structure could be significantly altered by an improper settlement estimate, selecting the correct percentage is essential.

Glennon Gilboy (1927) made perhaps the first study of the compressibility of a mixture of soils while at the Massachusetts Institute of Technology. Rather than using a clay, Gilboy used fine mica flakes. The mica did not exhibit surface charges, but its flat, elastic, platey structure produced a response somewhat similar to a clay. A sand of uniform composition and grain size was used as the granular

---

[1] 0.074 mm.

material. Both the compressibility and the void ratio increased with increasing mica content.

A significant correlation between the compression index and the percentage of clay size material was discovered by Alec Skempton (1944). The plot of this relationship between the clay content and the compression index is shown in Figure 1.

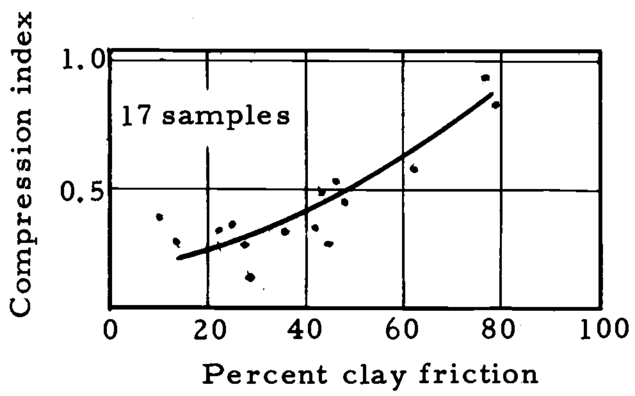

Figure 1. Correlation of the compression index with the percentage clay fraction (after Skempton).

The consolidation of clay samples containing stones was the subject of an investigation conducted by L. Murdock (1948). Tests were performed on samples containing clay and 0, 20, 40, 60 percent gravel by weight of the total mix. The compression index varied inversely with the percentage of gravel by weight. At 71 percent gravel the compression index became zero. The slope for the compression index was less between 20 and 40 percent than was expected from a straight line relationship. Murdock concluded that the consolidation characteristics of a soil are affected by the presence of stones, and that computations of settlements cannot be made accurately by testing samples of the soil fines and assuming that the compression was reduced in proportion to the volume of stones present.

The properties of morainal soils were studied by Bernell (1957). By performing consolidation tests on glacial soils of varying clay content, a linear relationship between the compression index and the clay fraction was established. The compression index was defined by

$$C_{cm} = 0.0044f + 0.003 \tag{1}$$

where "f" is the percentage of clay by volume. The consolidation was controlled by the amount of clay above 2.5 percent. Below this percentage the value of the compression index was constant.

Seed, Woodward, and Lundgren (1964) and Novais-Ferreira (1967) studied the effect of sand on the plasticity of soils. The results of a theoretical approach by Seed established the following relationship between the liquid limit and the percentage of clay by weight.

$$W_{\ell\ell} = \frac{f}{100} W_{c\ell\ell} \tag{2}$$

This condition would exist only as long as the nonclay particles were not in contact. Excellent agreement was found when the actual liquid limit test results were compared to the theoretical answers.

The significance of the liquid limit studies becomes apparent when the relationship between liquid limit and compression index is introduced. Skempton (1944) related the compression index and the liquid limit for remolded soils by the following equation.

$$C_c = 0.007(W_{\ell\ell} - 10) \tag{3}$$

If the materials were ordinary, undisturbed clays of medium to low sensitivity the relationship would be defined by

$$C_c = 0.009(W_{\ell\ell} - 10) \tag{4}$$

In the following investigation, rather than determining the compression index indirectly from a liquid limit relationship, an attempt was made to predict the amount of consolidation from the original Terzaghi assumptions. The constituents in the mixture were assumed to be composed of a clay fraction and a nonclay fraction. The two components were assumed not to interact.

If the mixture contains a high proportion of clay particles and a small proportion of nonclay particles, the system may be visualized as being composed of a series of individual granular particles floating in a clay-water matrix (Trollope and Chan, 1961) (Figure 2). The distribution of coarse grains is assumed to be random throughout. As stresses are increased, a uniform compression is assumed to occur within the matrix. The behavior of the soil is dominated by the clay-water mixture when the nonclay particles are not in contact (Seed, 1964).

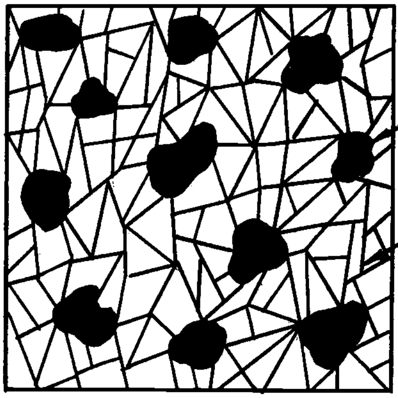

Figure 2. The nonclay-clay system.

Since the clay-water phase dominates the behavior of the system, a linear relationship would be expected on the void ratio versus the log of the effective stress plot. For one dimensional consolidation, the void ratio is directly related to the height of the system (Figure 3). The change in height in Figure 3 for the stress increment[2] may be expressed mathematically as

$$\Delta H = H_{T_1} - H_{T_2}$$

$$\Delta H = (H_v + H_{nc} + H_c)_1 - (H_v + H_{nc} + H_c)_2$$

---

[2] σ_1' to σ_2'

Since the volumes of clay and nonclay remain constant, this equation may be simplified to the following.

$$\Delta H = H_{v_1} - H_{v_2}$$

By definition,

$$e = \frac{V_v}{V_s},$$

which for one dimensional consolidation may be expressed as

$$e = \frac{H_v}{H_s}$$

Therefore,

$$\Delta H = (H_c + H_{nc})e_1 - (H_c + H_{nc})e_2$$

$$\Delta H = (H_c + H_{nc})\Delta e$$

If $$\Delta e = C_{cm} d \log (\sigma') \qquad \text{(Leonards, 1962)}$$

then $$\Delta H = (H_c + H_{nc}) C_{cm} d \log (\sigma') \tag{5}$$

If the nonclay is incompressible, it effectively reduces the volume of compressible structure in the soil system. The random positions of the particles in the unit volume (Figure 2) could be replaced by an incompressible layer at the bottom of the system (Figure 4). The proportion by volume of nonclay, clay, and voids would remain the same.

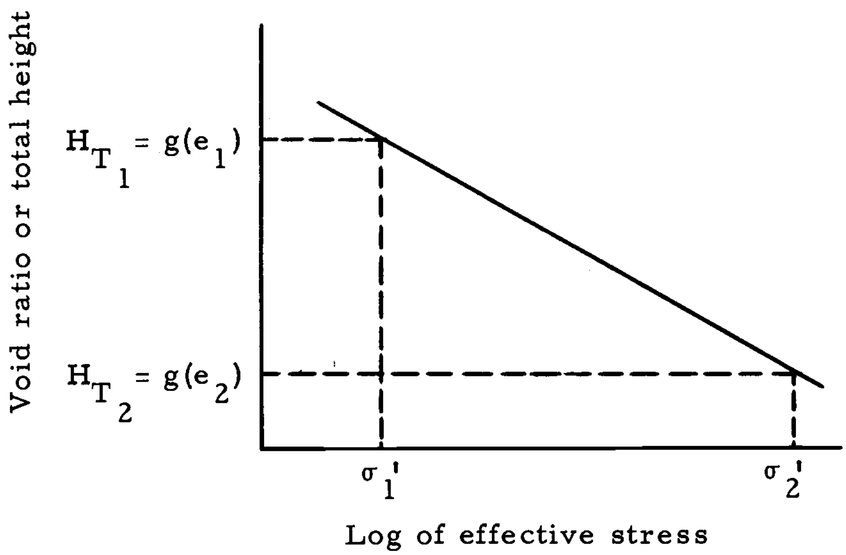

Figure 3. Height or void ratio versus the log of the effective stress.

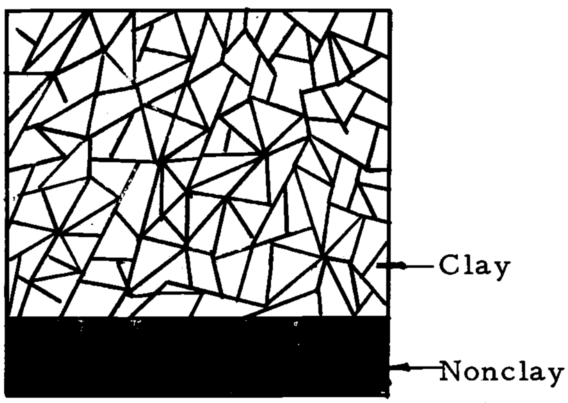

Figure 4. Nonclay as an incompressible layer in the nonclay-clay system.

A consolidation analysis on the layered system defines the same change in total height. However, the void ratio is now a function of the volume of the voids and clay solids. Mathematically this relationship is represented by the following set of equations.

$$\Delta H = H_{T_1} - H_{T_2}$$

$$\Delta H = (H_v + H_c)_1 - (H_v + H_c)_2$$

Since the volume of clay remains constant,

$$\Delta H = H_{v_1} - H_{v_2}$$

$$\Delta H = H_c \Delta e_c \qquad (e_c = \text{void ratio } {}^{w}/\text{o sand})$$

If, $$\Delta e_c = C_{cp} \, d \log (\sigma')$$

then, $$\Delta H = (H_c) C_{cp} \, d \log (\sigma') \qquad (6)$$

The change in heights for both systems are equal; therefore, combining Equations (5) and (6) and simplifying, the following relationship is obtained.

$$C_{cm} = \frac{H_c + H_{nc}}{H_c} C_{cp}$$

If,

$$X = \frac{W_{nc}}{W_{nc} + W_c} \qquad (7)$$

and the specific gravity of the nonclay is equal to the specific gravity of the clay, an equation relating the compression index of the mixture to the compression index of clay is defined.

$$C_{cm} = (1-X)C_{cp} \tag{8}$$

When this equation is compared to relationships established by Skempton (1944), Murdock (1948), and Bernell (1957), differences are noticeable. Skempton related the compression index to a curved line. Murdock could produce no single linear relationship. Bernell determined a straight line equation but its use was restricted to the soils he tested.

The equation for the compression index of a mixture (Equation 8) is linear and can be represented as shown in Figure 5. The linear relationship between the compression for the clay-nonclay mixture would exist as long as the nonclay granules floated in the clay matrix.

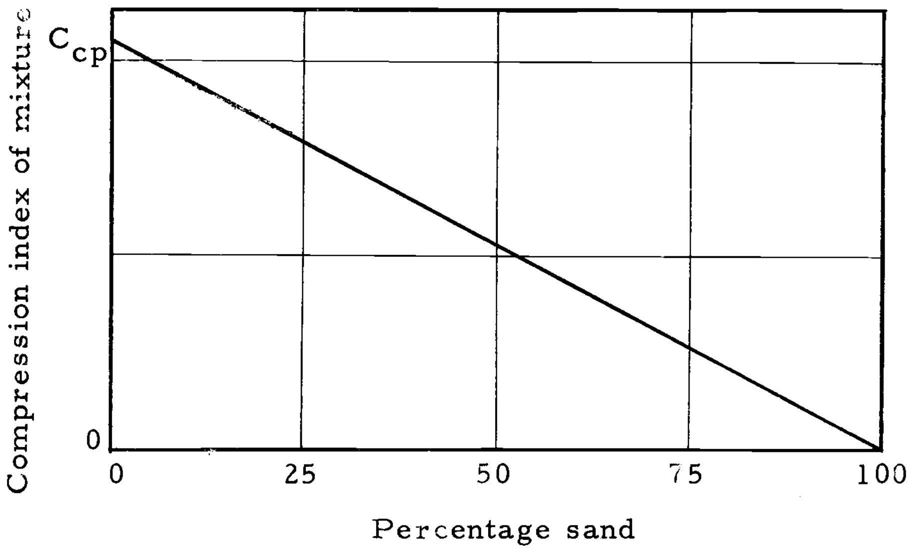

Figure 5. The general theoretical correlation between the compression index of a mixture and the percentage of nonclay material.

As compression continues two cases appear theoretically possible. The nonclay particles may come into contact at some pressure or the nonclay grains may always be separated by the clay solids. If the nonclay particles come into contact before zero void ratio is reached and there is no interference by the clay material, the deformation of the system would be as indicated by the solid line in Figure 6. If interference occurs before contact a gradual transition would be expected.

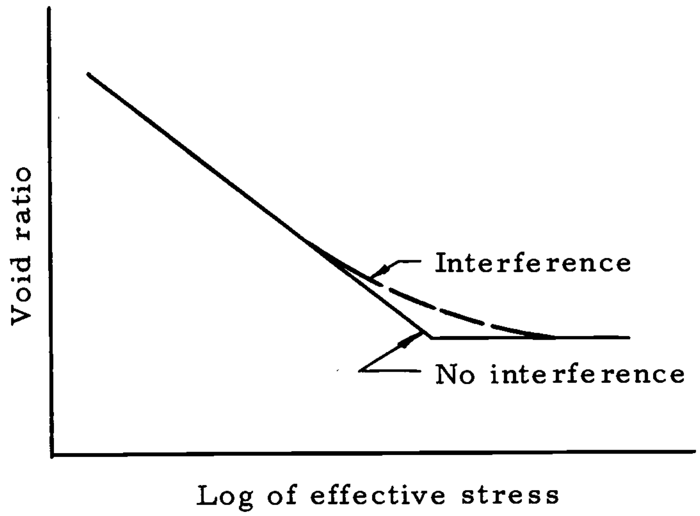

Figure 6. The general "e - log (σ')" curve for particle contact.

To determine the void ratio at which the nonclay particles come into contact, a cubic array is assumed. The void ratio for the array is 0.91 (Farouki and Winterkorn, 1964). Whenever the volume of voids of the array is exceeded by the volume of the clay solids, no contact is possible. Represented mathematically, preventing contact requires,

$$V_c > V_{voids\ nc}$$

If the specific gravity of the nonclay and the clay are the same, 52.5 percent sand is theoretically the minimum required for contact. At 52.5 percent sand, the void ratio at contact is zero.

For percentages of sand greater than 52.5 percent the void ratio at contact is defined by the equation,

$$e_t = \frac{V_{voids\ nc} - V_c}{V_{nc} + V_c}$$

The equation assumes the specific gravities are equal. Since the volume of voids for the nonclay material is represented by

$$V_{voids\ nc} = 0.91\ V_{nc}$$

the equation for the void ratio at contact could be rewritten as

$$e_t = \frac{0.91\ V_{nc} - V_c}{V_{nc} + V_c}$$

or on a weight basis

$$e_t = \frac{0.91\ W_{nc} - W_c}{W_{nc} + W_c}$$

and substituting,

$$X = \frac{W_{nc}}{W_{nc}+W_c}$$

$$e = 1.91X - 1.00 \tag{9}$$

A plot of the theoretical void ratio at contact for cubic packing is shown in Figure 7.

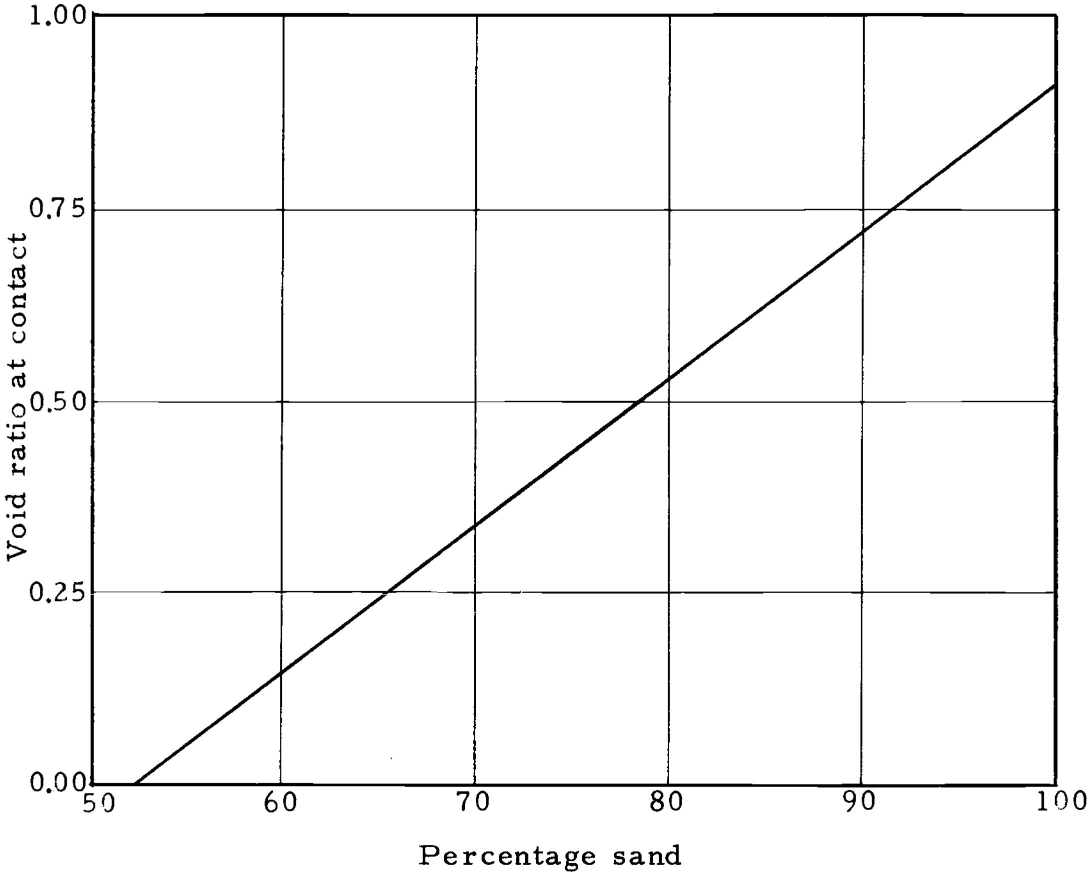

Figure 7. Correlation between void ratio at contact and percentage of nonclay material.

The void ratio at contact increases linearly with the percentage of sand; therefore, the percentage distinguishing the cohesive from the cohesionless material is a function of the mixture's void ratio. For the Unified Classification System, the void ratio is theoretically equal to zero before nonclay particles come into contact. For the percentage recommended by the American Association of State Highway Officials, contact occurs at a void ratio of 0.24. Murdock concluded that the change from cohesive to cohesionless behavior was at 71 percent sand. The corresponding void ratio is 0.36. Since void ratios encountered can hardly be expected to be less than 0.24, 65 percent sand or more is expected to determine the probable lower limit of coarse material for cohesionless behavior. However, the possibility of C_c becoming so small at lower percentages of cohesionless material that it is negligible, has not been considered.

LABORATORY STUDIES

General

The theoretical investigation introduced two concepts which were used in describing consolidation characteristics of clay-nonclay mixtures. If the nonclay material floated in the clay matrix, the compression index of the mixture was a linear function of the compression index for the pure clay. When the nonclay particles were in contact, the compression index of the mixture approached zero. To verify this bimodal relationship between the mixture and the amount of consolidation, a laboratory study was conducted.

The compression indices for clay-nonclay mixtures were established by performing a series of consolidation tests in the Department of Civil Engineering Soils Laboratory, Oregon State University, during the first four months of 1968.

The clay used in the mixture was a powdered HYDRITE PX kaolinite produced by the Georgia Kaolin Company. A particle size distribution curve supplied by the Company indicated 90 percent of the particles had an equivalent diameter of less than two microns. The specific gravity of the clay was 2.61. At the liquid and plastic limits the water contents were 56.9 and 31.8 percent, respectively. On Casagrande's Plasticity Chart (Means and Parcher, 1963), this material was classified as an inorganic clay with high plasticity.

The nonclay portion of the mixture was composed of a subangular Willamette River sand. The individual grains of sand passed the Number 30 U.S. Standard sieve (0.59 mm) and were retained on the Number 40 U.S. Standard sieve (0.42 mm). The specific gravity of the sand was 2.64.

Nine consolidation tests (Lambe, 1951) were performed on the sand-clay mixtures. The percentages of sand weight to total solids weight investigated were 0, 23, 30, 40, 55, 65, 75, and 100 percent.

Test Procedure

The test samples composed of sands and clays were prepared by mixing predetermined amounts of sand and clay with distilled water. Since a random distribution of sand within the clay matrix was desired, the mixture was prepared as fluid as possible while avoiding a consistency which would permit gravitational segregation. To achieve this consistency the amount of water added was 65 percent of the dry weight of clay plus 3 percent of the dry weight of sand. The selected water contents provided the semi-viscous state in the clay and saturated the surface of the sand grains. The ingredients were rapidly combined in an electric planetary mixer and then stored overnight in a closed container in a humidity room to improve the uniformity of the water content for the mix.

After one night of storage, the mixture was placed in a

4 1/2" x 12 1/2" x 3 1/2" form for preconsolidation. The consolidation pressure of 1.0 kg/cm^2 was applied in four increments during a four day period. The pressure was maintained at 1.0 kg/cm^2 for an additional four days before removing the load and storing the block at 100 percent relative humidity for seven days. The preconsolidation of the sample gave the mixture enough stiffness to trim and established a pseudo stress history.

The test specimen was obtained from the consolidated block by forcing a sharpened ring (1.0" x 2.5" dia.) down over a soil sample cut slightly larger than the inside diameter of the ring. The cutting ring also served as the containing ring in the ensuing floating ring consolidation test. To minimize evaporation loss, cutting was done in a humid room. From the trimmings, representative samples were selected for liquid and plastic limits and water content determinations. The initial weights of the ring and specimen were recorded.

Saturated porous stones were fitted to the top and bottom of the specimen before placing the assemblage in a lucite dish. The loading head was adjusted above the sample, the deflection dial set, and the sample flooded. A load increment ratio of two was used in these tests and each load was left on for a 24 hour duration.

The 100 percent sand specimen was prepared by loosely placing a predetermined amount of oven dry Willamette River sand in a fixed ring consolidometer. After adjusting the porous stone, loading head, and the deflection dial, loads were applied at a load increment ratio of two. The void ratio at the end of 100 percent primary consolidation was determined from deflection dial readings taken at various time intervals for each load increment.

To apply the load to the specimen a lever arm consolidometer was employed (Figure 8). Loading was started at a pressure of 0.316 kg/cm^2 and terminated when a pressure of 161 kg/cm^2 was reached. Casagrande's logarithm of time method was used to interpret test results (Taylor, 1948).

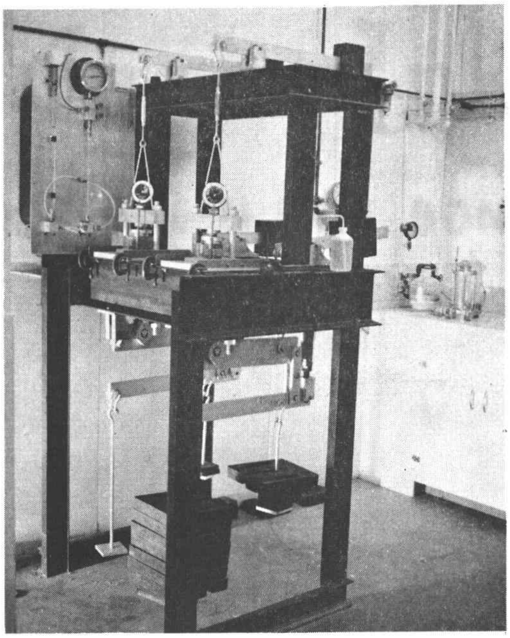

Figure 8. Lever arm consolidometer.

At the completion of each test on the sand-clay mixture, the sample was removed from the loading head and weighed. The final water content was established by oven drying and reweighing. The dry weight of any material extruded from the ring during the test was recorded. For each pressure increment the void ratio at the end of primary consolidation was calculated. This void ratio was then corrected for machine and porous stone deflections and for the material forced out during the loading sequence.

The compression index for each mixture was determined by plotting a corrected value of void ratio against the logarithm of the pressure (Appendix). A procedure proposed by Casagrande (Leonards, 1962), was used to establish the apparent preconsolidation pressure for the mixture. The virgin curve whose slope represented the compression of the mixture before disturbance by trimming and handling (Schmertman, 1953) was constructed by drawing a line from the point at which the initial void ratio and the apparent preconsolidation pressure intersected to a point established by the intersection of the laboratory plot and 0.4 times the initial void ratio (Appendix).

Skempton's equation (Equation 4) relating the compression index to the liquid limit for a normally consolidated clay was used to determine a second set of compression index values.

TEST RESULTS

The results of laboratory tests performed on the sand-clay mixtures are summarized in Table 1. The values for compression index, apparent preconsolidation pressure, and initial void ratio were established from the void ratio - log effective stress plots in the Appendix. The liquid limits were obtained by averaging the results from three Atterberg limit tests on each mixture.

Table 1. Test results.

Percentage sand $(W_s/W_s+W_c) \times 100$	Compression index	Apparent preconsolidation pressure (kg/cm^2)	Liquid limit (%)
0	0.44	1.3	56.90
23	0.33	2.1	38.56
30	0.31	1.4	36.17
40	0.25	1.7	-----
55	0.19	1.8	24.00
65	0.16	5.2	20.87
75	0.16	9.0	18.66
100	0.02 - 0.30	---	-----

The theoretical and laboratory relationships between the compression index and the percentage of sand are plotted in Figure 9. The theoretical curve is obtained from Equation (8) using a value of 0.44 for C_{cp}.

For samples containing less than 65 percent sand, the agreement between the theoretical and the laboratory plots was exceptionally good. At approximately 65 percent sand a distinct flattening in the

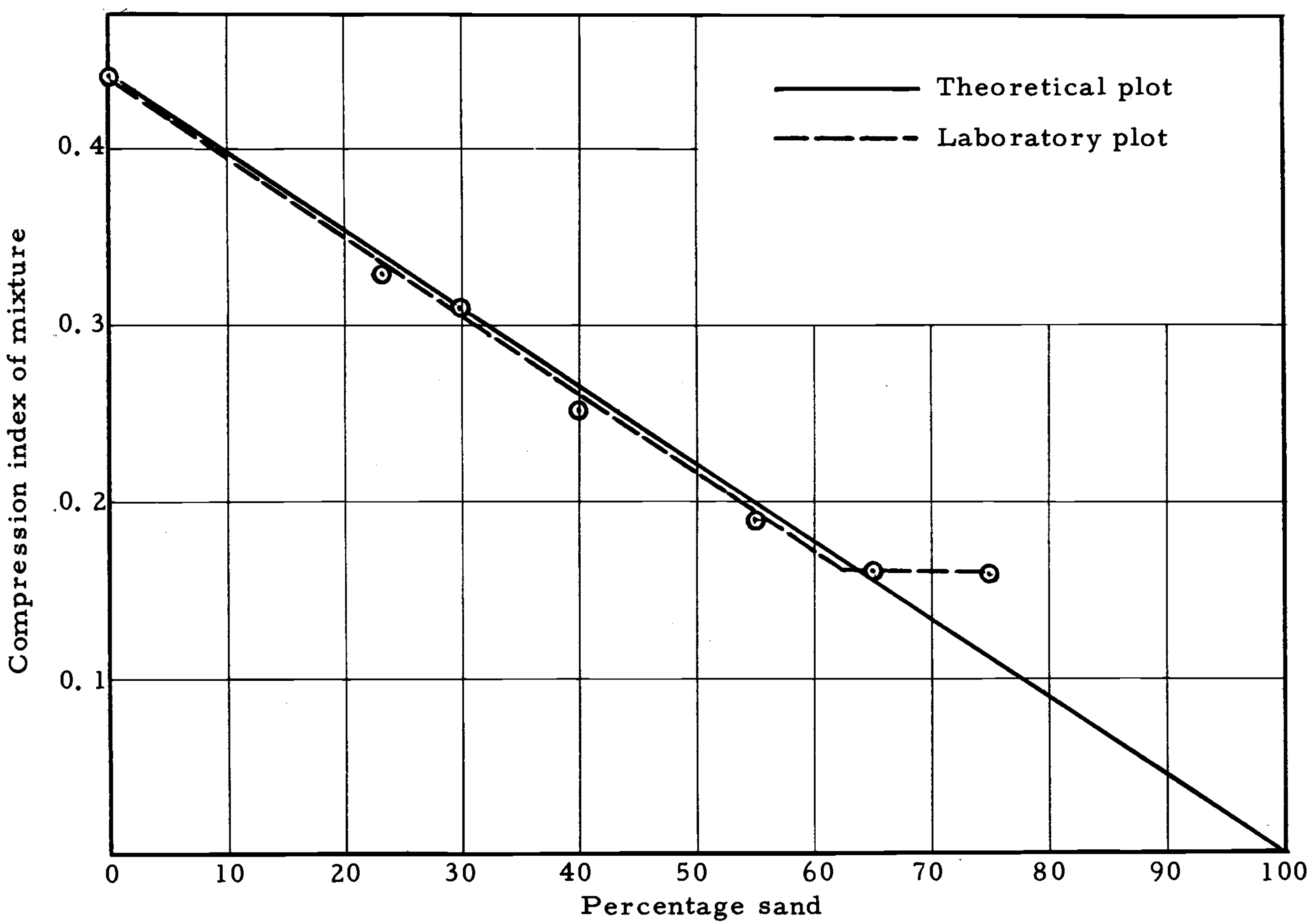

Figure 9. A comparison of laboratory and theoretical values for compression index.

slope of the laboratory plot occurs. No results were obtained to establish the nature of the transition between 75 and 100 percent sand.

In Figure 10 the compression index computed from the liquid limit data was compared to the consolidation test results and the theoretical results determined from a combination of the relationships developed by Seed (Equation 2) and Skempton (Equation 4). The liquid limits compared favorably to results established from theoretical relationships presented by Seed. However, the compression indices were significantly less than the results of the laboratory tests.

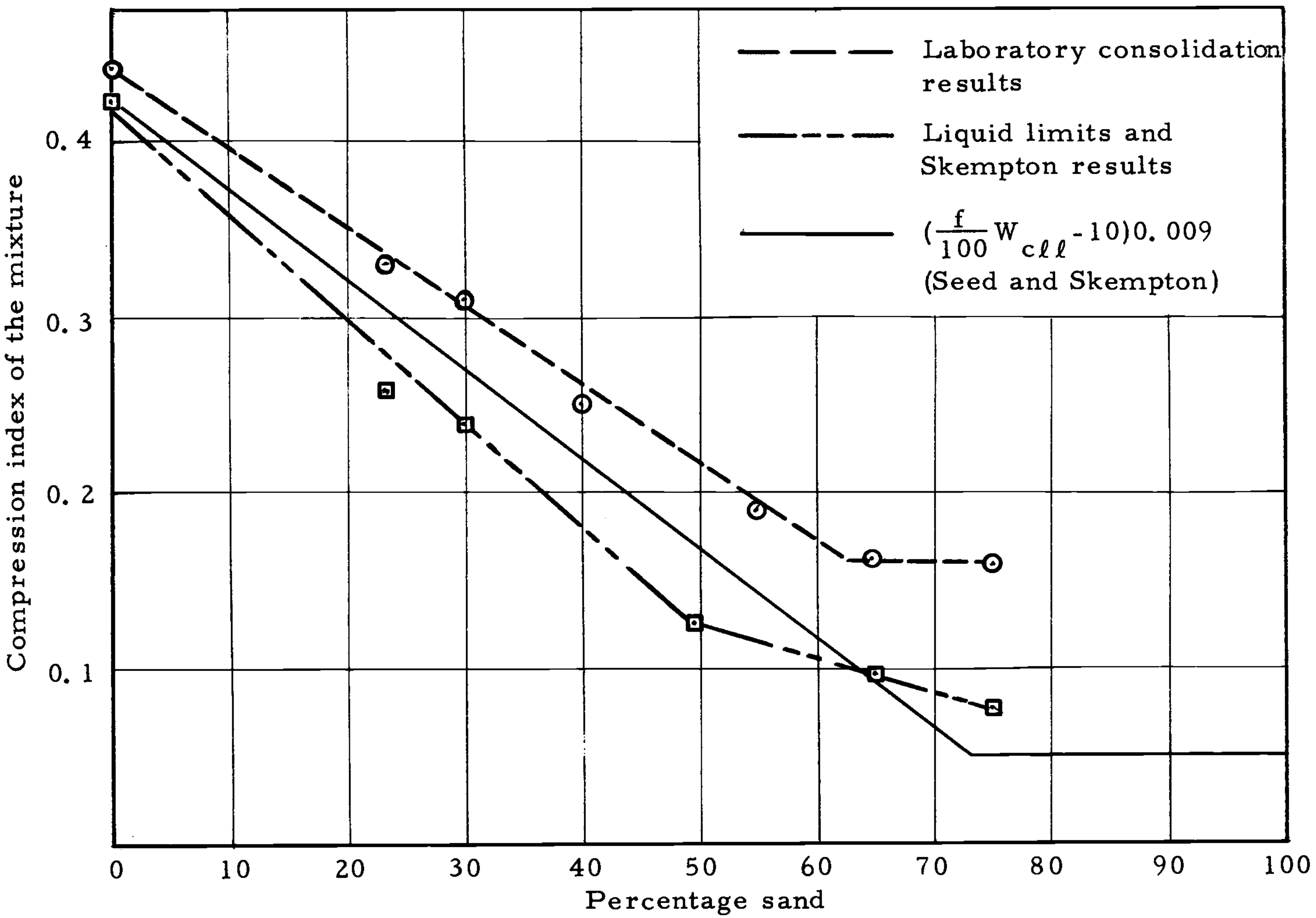

Figure 10. A comparison of the compression indices.

DISCUSSION OF RESULTS

Compression Index

When the compression indices from the laboratory test results were plotted with the theoretical values computed from Equation (8) and the compression index for pure clay, three distinct regions were noted (Figure 9). In the range of 0 to 65 percent sand, the lines for the two plots nearly coincided. Between 65 and 75 percent sand, the compression indices for the laboratory test became constant while the theoretical plot continued to decrease linearly. Above 75 percent the value of the compression indices decreased with increasing sand content. The exact form of the relationship in this range was not indicated by the data.

The results of the laboratory test verified that the theoretical method for calculating the compression index of a mixture was valid if the percentage of sand was less than 65 percent. The validity of the theoretical approach also implied essential compliance to three of the original assumptions: 1) the particles were randomly distributed, 2) the compression was uniform, and 3) there was no interaction between clay and sand material.

Contact of sand particles in a theoretical mixture was possible when the mixture contained more than 50 percent sand. For 65 percent sand, the void ratio at contact was 0.24. The minimum void ratio obtained during laboratory testing for the 65 percent mixture was 0.35. The corresponding pressure was 161 kg/cm^2. Since the laboratory value of void ratio was much higher than the void ratio at contact, interaction between sand particles was not expected. The laboratory investigation was conducted for pressures beyond the range of pressure normally encountered in settlement estimates; therefore, contact between particles was not a practical consideration for less than 65 percent sand. This suggested that the American Association of State Highway Official's method was more realistic to soil classification than the Unified Soil Classification System.

The compression index remained constant when the percentage was increased from 65 to 75 percent. The behavior in this range involved stress redistribution, shear deformation in the clay, and particle contact. For 75 percent sand, the theoretical void ratio at contact was 0.43. The corresponding pressure on the laboratory "e - log (σ')" curve was 20 kg/cm^2. If the pressure was below 20 kg/cm^2, the compression was uniform until differential stress concentrations caused redistribution of the clay material (Figure 11). When sand particles were forced closer together, the clay material between points of contact was compressed to a higher degree than the remaining clay. According to Casagrande (1932), at some critical point, an additional force caused local shear failures in the clay. Material in the highly stressed zones flowed laterally to zones of lower stress. Extrusion of clay material continued until an equilibrium void ratio was reached in the surrounding system. This resulting void ratio was lower than the void ratio of an equivalent layered system at the same pressure and the compression index was larger than predicted by Equation (8).

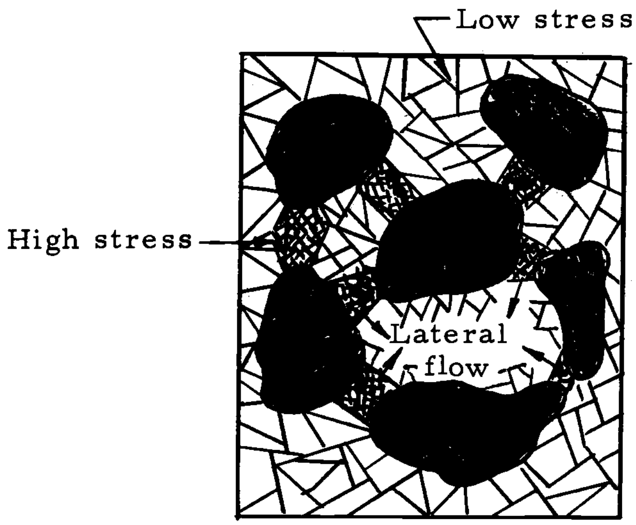

Figure 11. Stress distribution in a sand-clay mixture.

At 20 kg/cm^2 the particles theoretically came into contact. Once in contact the sand reacted to further stress increases by deforming elastically or crushing at the points of contact (Roberts and de Souza, 1958). The resulting compression index was very small for elastic deformation; whereas, a significant compression index existed for particle crushing.

To determine the characteristics of the Willamette River sand in compression a special consolidation test was performed on the sand alone. For the loosely placed sand, crushing became pronounced at 20 kg/cm^2. This value was similar to the value predicted by Roberts and de Souza. At pressures below the crushing stress the compression index was 0.02, and above this pressure the index varied from 0.08 at a stress of 25 kg/cm^2 to 0.30 at 120 kg/cm^2.

The compression index for 75 percent sand was defined by the slope of the "$e - \log(\sigma')$" plot beyond 20 kg/cm^2. Since the sand grains were theoretically in contact, crushing of particles was thought to define the compression index. No laboratory data were available between 65 and 75 percent sand; however, clay redistribution was expected to dominate the behavior of the compression index for the range below 72 percent. Above that percentage, the compression index appeared to be a function of particle crushing.

The reaction to stress in the 75 to 100 percent sand range was expected to follow a sequence similar to that for 75 percent sand. Clay would compress and redistribute by shear until particles were in contact. Depending upon the stresses at contact, the particles either crushed or deflected elastically. As the percentage of sand was increased, the void ratio at the point of contact increased. Higher void ratios are attained at lower pressures; therefore, the significance of elastic compression increased with increasing percentages of sand. The magnitude of the elastic deformation began to dominate the compression index for the system.

The compression indices established by the laboratory tests are compared to relationships defined by Skempton and Bernell in Figure 12.

Skempton's curve for compression index is significantly higher than the points established from the laboratory test results. The difference in the plots is a function of the material involved in the testing programs. Skempton tested remolded marine and lacustrine samples. A test performed on a kaolinite clay plots considerably below his other test results (Figure 12). However, the point for the kaolinite sample corresponds closely to the value established by this study. Similarities between the two curves are also noted at low percentages of clay. Both plots tend to flatten at less than 35 percent clay. This similarity implies that the change in slope for the laboratory curve (Figure 9) is not a unique feature of this testing program.

Bernell's plot closely approximates the results from the laboratory test and the test results by Skempton for a boulder clay. The Bernell curve and the laboratory curve for the compression index nearly coincide. The similarity of the plots is attributed to the materials tested. A deviation between the two plots is expected if the laboratory material is varied. The point at which the compression index becomes constant is much lower than the point of transition determined by Skempton and in this study.

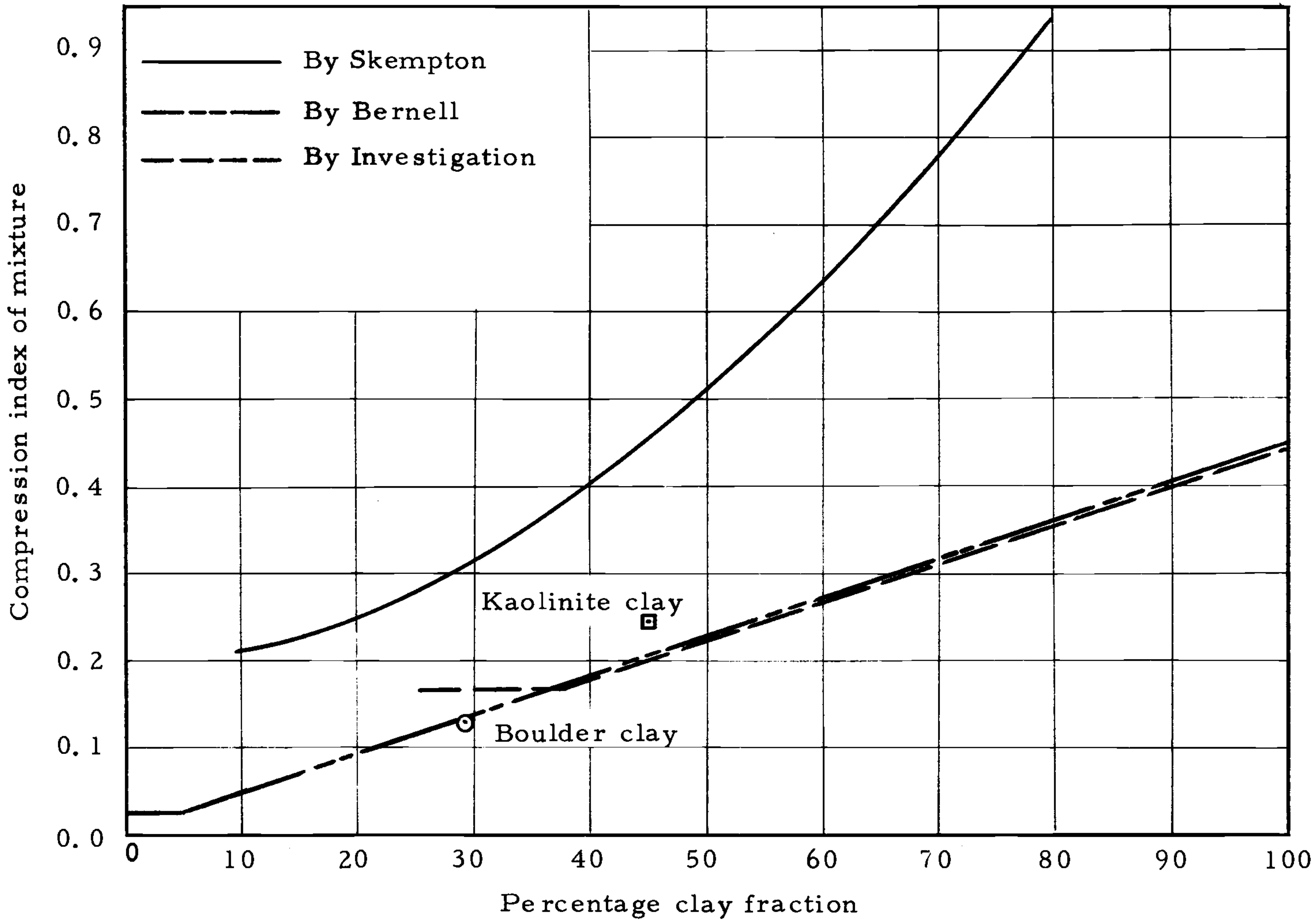

Figure 12. Correlation between compression indices and percentage of clay (Bernell, Skempton and this laboratory investigation).

Apparent Preconsolidation Pressure

The apparent preconsolidation pressure was determined by Casagrande's procedure (Leonards, 1962). In all cases the samples were actually preconsolidated to a pressure of 1.0 kg/cm^2. As shown in Figure 13, the apparent preconsolidation pressure was larger than the actual preconsolidation pressure for each mixture.

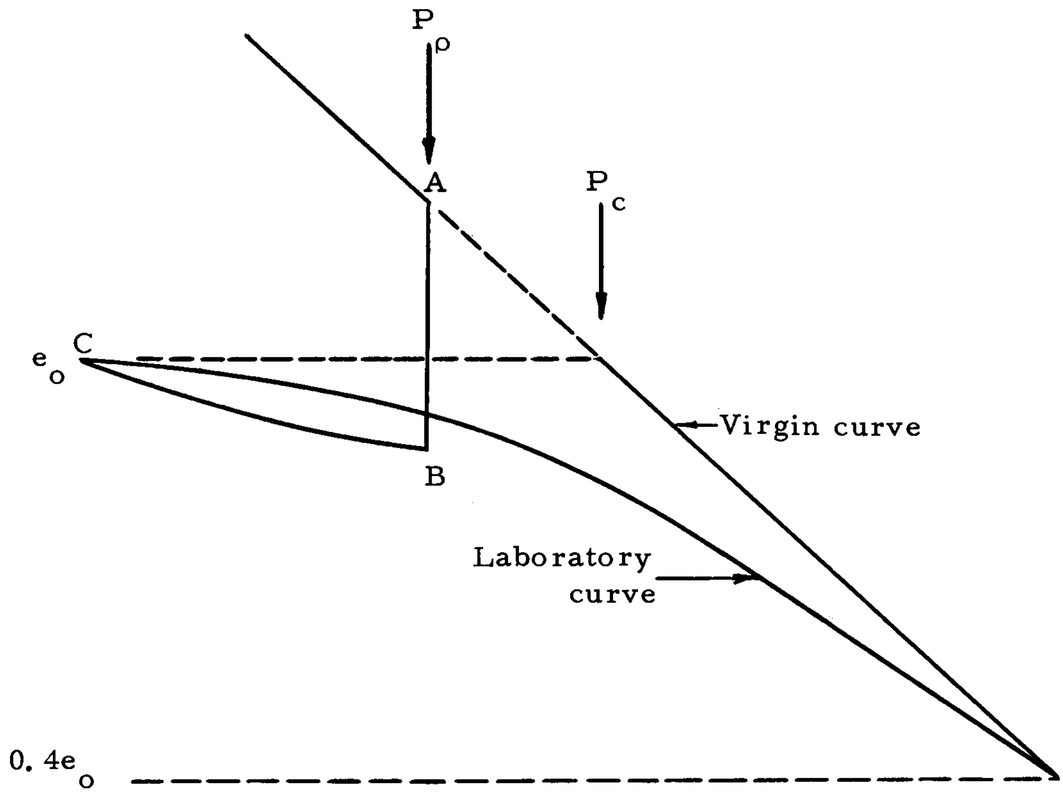

Figure 13. Relationship between virgin and laboratory consolidation curve.

The void ratio at the end of primary consolidation for samples preconsolidated to 1.0 kg/cm^2 is indicated by Point A on Figure 13. Maintaining that pressure for seven days permitted secondary compression to occur which caused reduction of the void ratio at a constant stress to Point B on Figure 13. The initial void ratio for the laboratory test specimen (Point C) was considered to be the void ratio after removing the load and allowing sem-elastic rebound to occur.

When a sample with an initial void ratio at Point C is reloaded, the laboratory test results do not coincide with the so-called virgin curve until a void ratio of approximately 0.4 times the initial void ratio is achieved. The plastic readjustment of the soil structure during secondary consolidation (Christie, 1965) provides the soil with greater resistance to volume change than exhibited by the virgin curve for primary consolidation at the same pressure increments. As the stress on the system increases the two curves approach each other and intersect at 0.4 times the initial void ratio. Utilizing this relationship enables construction of the virgin curve by connecting the 0.4 e_o point to a point on the virgin curve defined by the intersection of the initial void ratio and the apparent preconsolidation pressure with a straight line.

For 100 percent clay, the difference between the actual and the apparent preconsolidation pressure was small. From 0 to 60 percent sand, a very slight increase in preconsolidation pressure occurred. Above 60 percent sand the magnitude of the apparent preconsolidation pressure increased greatly (Figure 14). The difference between the two pressures for less than 60 percent sand was caused by secondary

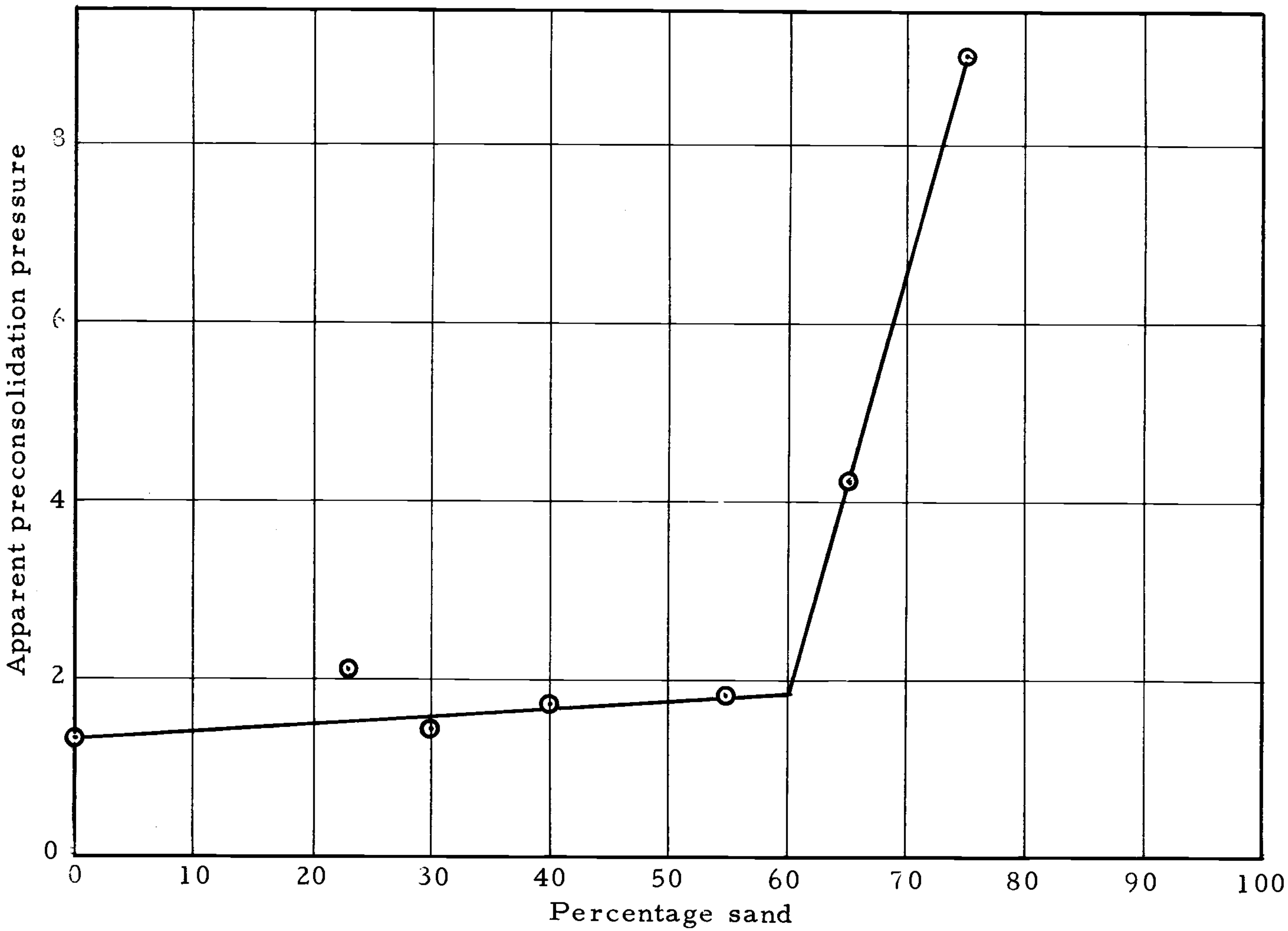

Figure 14. Relationship between apparent preconsolidation pressure and percentage of sand.

compression and semi-elastic rebound of the specimen. Increasing the sand content resulted in a linear decrease in the slope of the virgin consolidation curve. Secondary compression and semi-elastic rebound decreased similarly. The results of the three effects was not expected to change the magnitude of the apparent preconsolidation pressure. Above 60 percent sand an entirely different reaction seemed to dominate the system. The large increase in the apparent preconsolidation pressure was attributed to unequal consolidation of the clay in the sand-clay system.

An inconsistency was noted at 23 percent sand during the comparison of the values of apparent preconsolidation pressure. The effect of secondary consolidation and semi-elastic rebound was more than twice the amount expected. Since the sample had been prepared in the same manner as other samples, this discrepancy could not be explained.

Difficulties were also encountered when determining the position of the apparent preconsolidation pressure at high percentages of sand. The point of maximum curvature needed for the Casagrande construction was difficult to define. Varying the value of apparent preconsolidation caused a slight variation in the slope of the virgin curve.

Liquid Limits

The liquid limit for each sample compared favorably to the limit established from the relationship proposed by Seed (Equation 2). The magnitude of the deviation was insignificant for less than 60 percent sand (Figure 15). At 60 percent sand a transition occurred as sand particles began interacting. Seed predicted this transition at approximately 75 percent sand.

The linear decrease in liquid limit represented the net reduction in the percentage of clay in the mixture. Since the volume of clay was reduced linearly, the net attractive force and, therefore, the corresponding liquid limit were lowered.

When the liquid limit was correlated to the compression index by Skempton's relationship (Equation 4) and then compared to the laboratory values of compression index (Figure 10), deviations were noticeable. The values of compression index were significantly lower than the laboratory tests. Since the correlation between liquid limit and the percentage of sand was relatively good, the departure from actual laboratory values was attributed to the Skempton relationship.

The equation proposed by Skempton (Equation 4) relating the compression index to the liquid limit was determined by performing numerous consolidation and liquid limit tests on various marine and lacustrine soils. The statistical average of the test results defined the straight line relationship. Since this equation represented the statistical average of numerous soil types, a deviation for any given soil was expected. By adjusting the slope and the liquid limit

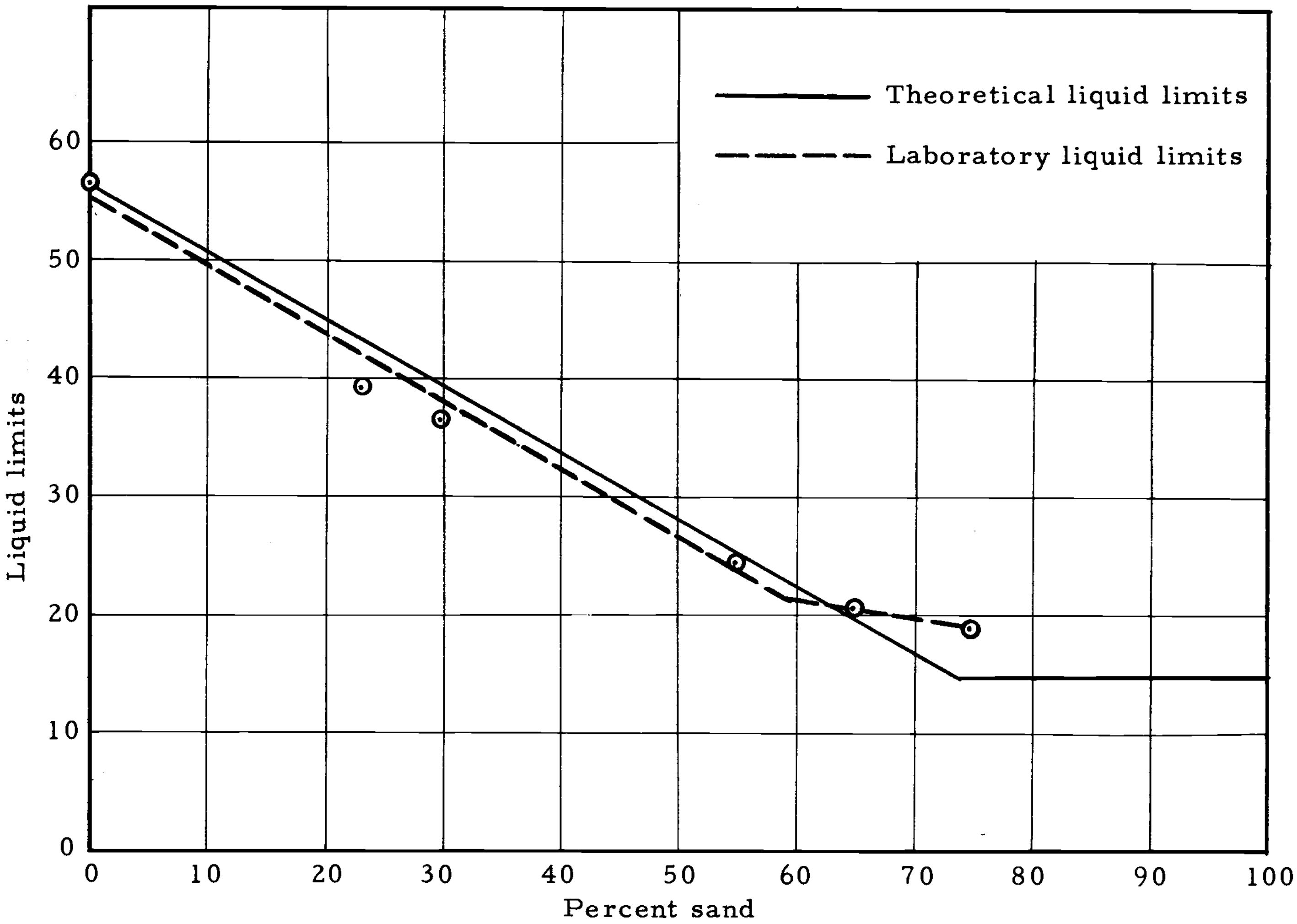

Figure 15. A comparison of the laboratory and theoretical liquid limits.

intercept for Skempton's plot, the approximation of the compression index by using the liquid limit of the mixture was expected to be much closer to the laboratory test results.

General Application

The compression index for a mixture of less than 65 percent sand may be predicted for normally consolidated soils by using the theoretical relationship (Equation 8) established in this investigation. Although some stress redistribution occurs in the upper portion of the range, the magnitude is not enough to affect the test results. The solution requires the compression index for 100 percent clay and the percentage of sand in the mixture.

The compression index for 100 percent clay may be determined by using the Skempton relationship (Equation 4) between the compression index and the liquid limit. The percentage of sand in the system may be determined by a sieve analysis. The liquid limit of the mixture should not be used to predict the compression index of the mixture.

If the soil is overconsolidated, consolidation tests must be performed on the mixture. The theory could be used to extrapolate from the results of one laboratory test in a given deposit where the mineralogy and stress history are the same.

Above approximately 65 percent sand the compression index would be controlled by the sand. Except for the rate of consolidation, the material would behave as a granular soil and should be treated as such.

CONCLUSIONS

1. The theoretical relationship (Equation 8) defining the compression index of a mixture is valid if the system has the following characteristics.

   a. The mixture is composed of a uniform cohesive soil and a uniform cohesionless soil.

   b. The proportion of coarse soil is less than 65 percent.

   c. The maximum pressure on the soil system does not exceed 161 kg/cm^2.

2. For soils containing more than 65 percent coarse soil, the behavior of the system is determined by the percentage of coarse materials and the stress level. Before the contact of the coarse particles, uniform compression or stress redistribution controls the compression characteristics of the mixture. Once the particles are in contact, the compression is controlled by the elastic deformation or crushing of particles at points of high stresses.

3. The method recommended by the American Association of State Highway Officials for distinguishing the cohesionless from the cohesive soil on the basis of 65% coarse material should be used when classifying a uniform sand clay mixture with respect to consolidation characteristics.

RECOMMENDATIONS FOR FUTURE STUDY

The scope of the relationship established during this study may be enlarged by considering several factors which influence the results (Van Zelst, 1948) or increase the possibility of practical application.

The laboratory investigation was performed on a uniform sand. Since many mixtures are composed of nonuniform coarse materials, the effect of gradation on the test results would be of great interest. If the materials were well or poorly graded (Wise, 1952), the influence of the coarse soil would be expected to occur at a lower percentage of sand.

All tests in this investigation were performed on a kaolinite clay. Although the relationship for compression indices compared favorably with Bernell's results for glacial soils, the relationship varied from the Skempton's test results on marine and lacustrine mixtures. The investigation should be extended to include other varieties of clays.

The specific behavior of mixtures containing more than 75 percent cohesionless material was not investigated. A laboratory study is necessary to define the compression index in this range. An investigation of this percentage range might also determine redistribution and particle contact characteristics for mixtures of high percentages of cohesionless particles.

A better method is necessary for defining the compression index from Skempton's liquid limit equation. The relationship is suitable for a small group of soils, but significant variations occur when other soils are considered. If the technique could be refined, the liquid limit would serve as a reliable indication of the compression index for a normally consolidated mixture.

BIBLIOGRAPHY

Bernel, L. 1957. The properties of moraines. In: Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957. Vol. 2. London. p. 286-290.

Burmister, Donald M. 1950. Identification and classification of soils--an appraisal and statement of principles. In: A symposium on the identification and classification of soils, Atlantic City, N. J., 1950. Philadelphia, American Society for Testing Materials. p. 3-24. (Special Technical Publication no. 193)

__________ 1958. Suggested methods for consolidation testing of soils. In: Procedures for testing soils. Philadelphia, American Society for Testing Materials. p. 287-295.

Casagrande, Arthur. 1932. The structure of clay and its importance in foundation engineering. In: Contributions to soil mechanics, 1925-1940. Boston, Boston Society of Civil Engineers. p. 72-125.

__________ 1947. Classification and identification of soils. Proceedings of the American Society of Civil Engineers 73:783-810.

Christie, I. F. 1965. Secondary compression effects during one dimensional consolidation tests. In: Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, Toronto, 1965. Vol. 1. Toronto. p. 198-202.

Farouki, Omar T. and Hans F. Winterkorn. 1964. Mechanical properties of granular systems. Highway Research Record 52:10-42.

Gilboy, Glennon. 1928. The compressibility of sand-mica mixtures. Proceedings of the American Society of Civil Engineers 54:555-573.

Lambe, T. William. 1951. Soil testing for engineers. New York, Wiley. 165 p.

Leonards, G. A. (comp.). 1962. Foundation engineering. New York, McGraw-Hill. 1136 p.

Leonards, G. A. and A. G. Altschaeffl. 1964. Compressibility of clays. Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers 90: 133-155.

Means, R. E. and J. V. Parcher. 1963. Physical properties of soils. Columbus, Charles E. Merril. 464 p.

Mitchell, James K. 1956. The fabric of natural clays and its relations to engineering properties. Proceedings of the Highway Research Board 35:693-715.

Murdock, L. J. 1948. Consolidation tests on soils containing stones. In: Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948. Vol. 1. Rotterdam. p. 169-173.

Novais-Ferreira, H. 1967. Influence of the clay content on the plasticity of soils. In: Proceedings of the Fourth Regional Conference for Africa Soil Mechanics and Foundation Engineering, Cape Town, 1967. Vol. 1. Cape Town. p. 261-265.

Oglesby, Clarkson H. and Laurence I. Hewes. 1963. Highway engineering. 2d ed. New York, Wiley. 783 p.

Roberts, J. E. and J. M. de Souza. 1958. The compressibility of sands. Cambridge, Massachusetts Institute of Technology. 10 p.

Schmertman, John H. 1953. Estimating the true consolidation behavior of clay from laboratory test results. Proceedings of the American Society of Civil Engineers 79 (Separate no. 311):1-26.

Schultze, Edgar and Abdelmonem Moussa. 1961. Factors affecting the compressibility of sand. In: Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris, 1961. Vol. 1. Paris. p. 335-340.

Seed, H. Bolton, Richard J. Woodward and Raymond Lundgren. 1964. Fundamental aspects of the atterberg limits. Journal of the Soil Mechanics and Foundation Engineering Division, American Society of Civil Engineers 90: 75-105.

Skempton, Alec W. 1944. Notes on the compressibility of clays. Quarterly Journal of the Geological Society of London 100:119-135.

Supnick, Fred. 1940. On the dense packing of spheres. Transactions of the American Mathematical Society 65:14-26.

Taylor, Donald W. 1948. Fundamentals of soil mechanics. New York, Wiley. 700 p.

Trollope, D.H. and C.K. Chan. 1961. Soil structure and the step strain phenomenon. Transactions of the American Society of Civil Engineers 126:1650-1698.

Van Zelst, Theodore W. 1948. An investigation of factors affecting laboratory consolidation of clay. In: Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948. Vol. 7. Rotterdam. p. 52-61.

Wise, M.E. 1952. Dense random packing of unequal spheres. Philips Research Reports 7:321-343.

APPENDIX

Additional Reading

Although the following articles were not specifically referred to in the text, their information influenced the theory and procedure of the investigation.

Bolt, G. H. 1956. Physico-chemical analysis of the compressibility of pure clays. Geotechnique 6:86-93.

Fraser, H. J. 1935. Experimental study of the porosity and permeability of clastic sediments. Journal of Geology 43:910-1010.

Gibbs, Harold J. 1950. The effect of rock content and placement density on consolidation and related pore pressures in embankment construction. Proceedings of the American Society for Testing Materials 50:1343-1363.

Grim, Ralph E. 1961. Clay minerals. Transactions of the American Society of Civil Engineers 126:699-713.

Janbu, N. and E. I. Hjeldnes. 1965. Principal stress ratios and their influence on the compressibility of soils. In: Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, Toronto, 1965. Vol. 1. Toronto. p. 249-253.

Rosenqvist, I. Th. 1961. Mechanical properties of soil-water systems. Transactions of the American Society of Civil Engineers 126:745-767.

Salas, J. A. Jimenez and J. M. Serratosa. 1953. Compressibility of clays. In: Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, Switzerland, 1953. Vol. 1. Switzerland. p. 192-198.

Terzaghi, Karl and Ralph B. Peck. 1967. Soil mechanics in engineering practice. 2d ed. New York, Wiley. 729 p.

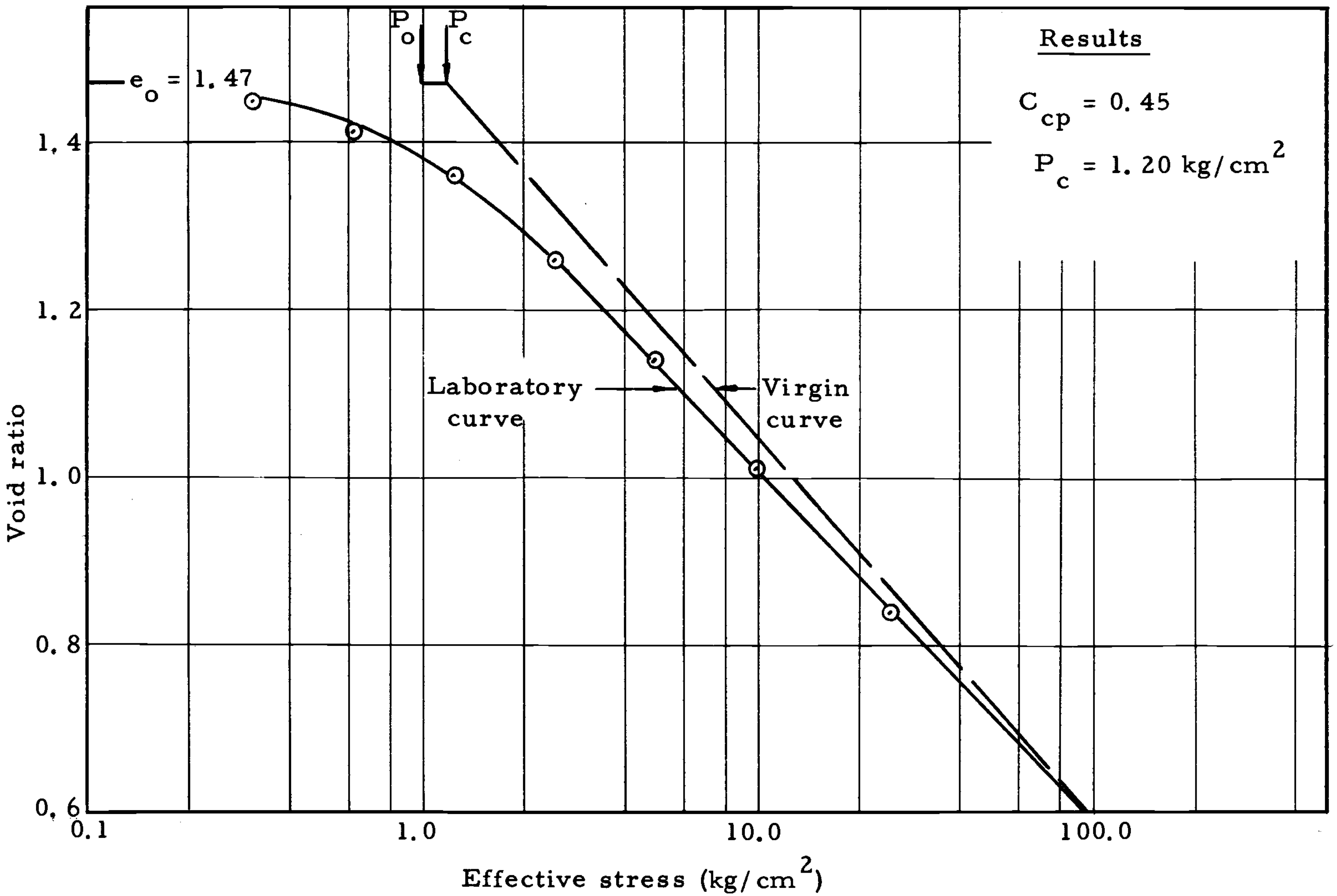

Figure 16. The "e - log (σ')" curve for 0 percent sand (test 1).

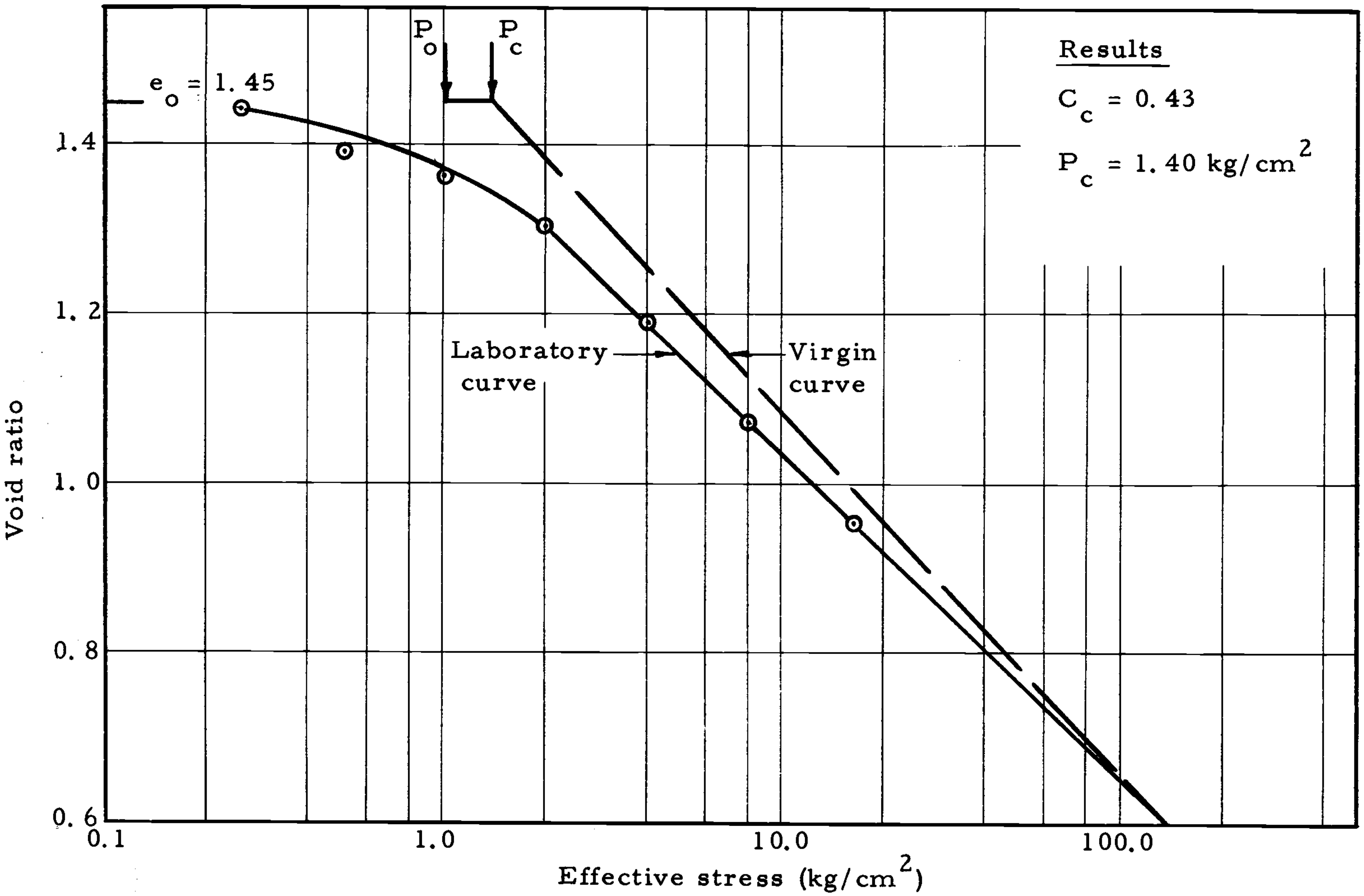

Figure 17. The "e - log (σ')" curve for 0 percent sand (test 2).

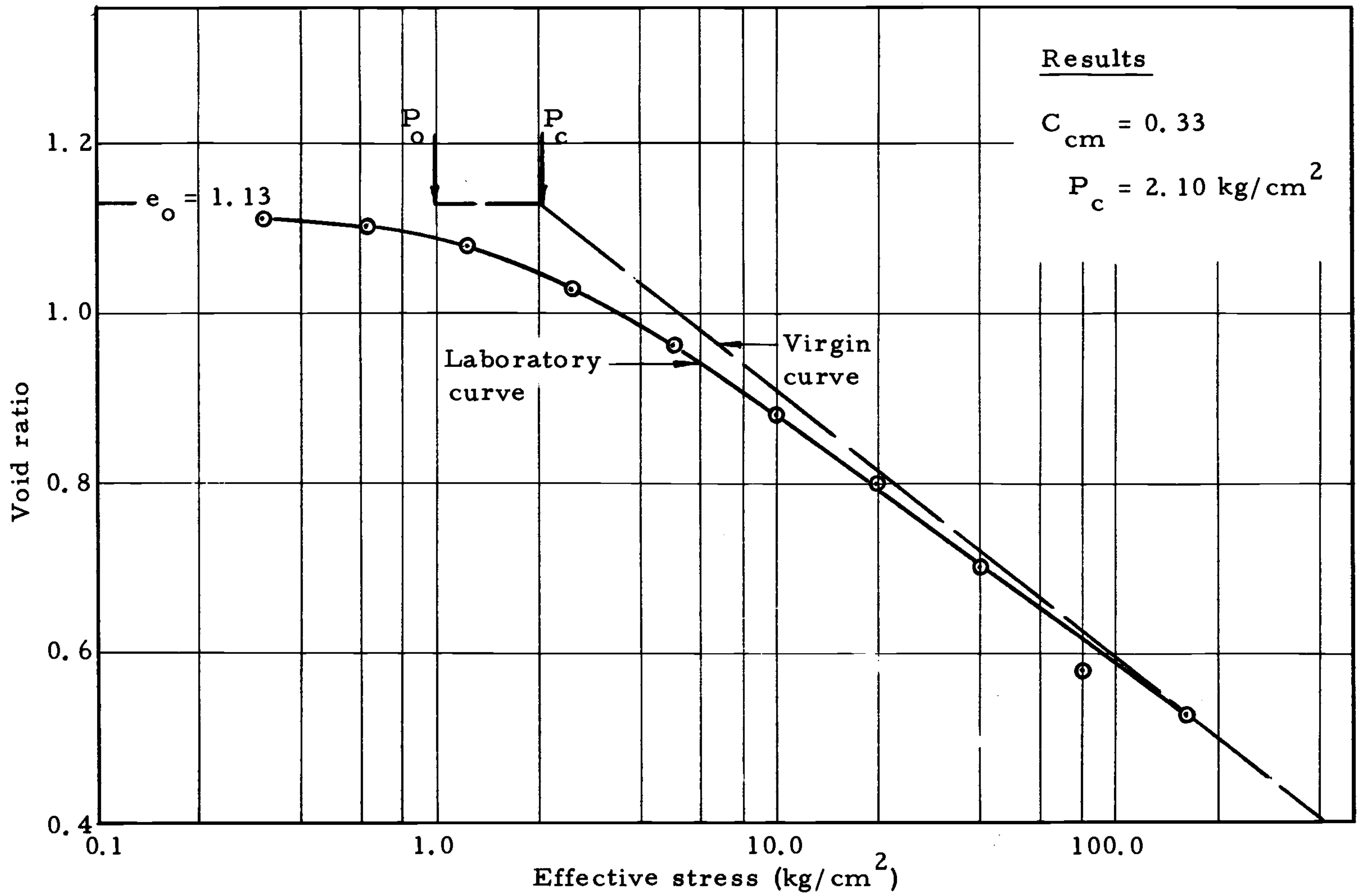

Figure 18. The "e - log (σ')" curve for 23 percent sand.

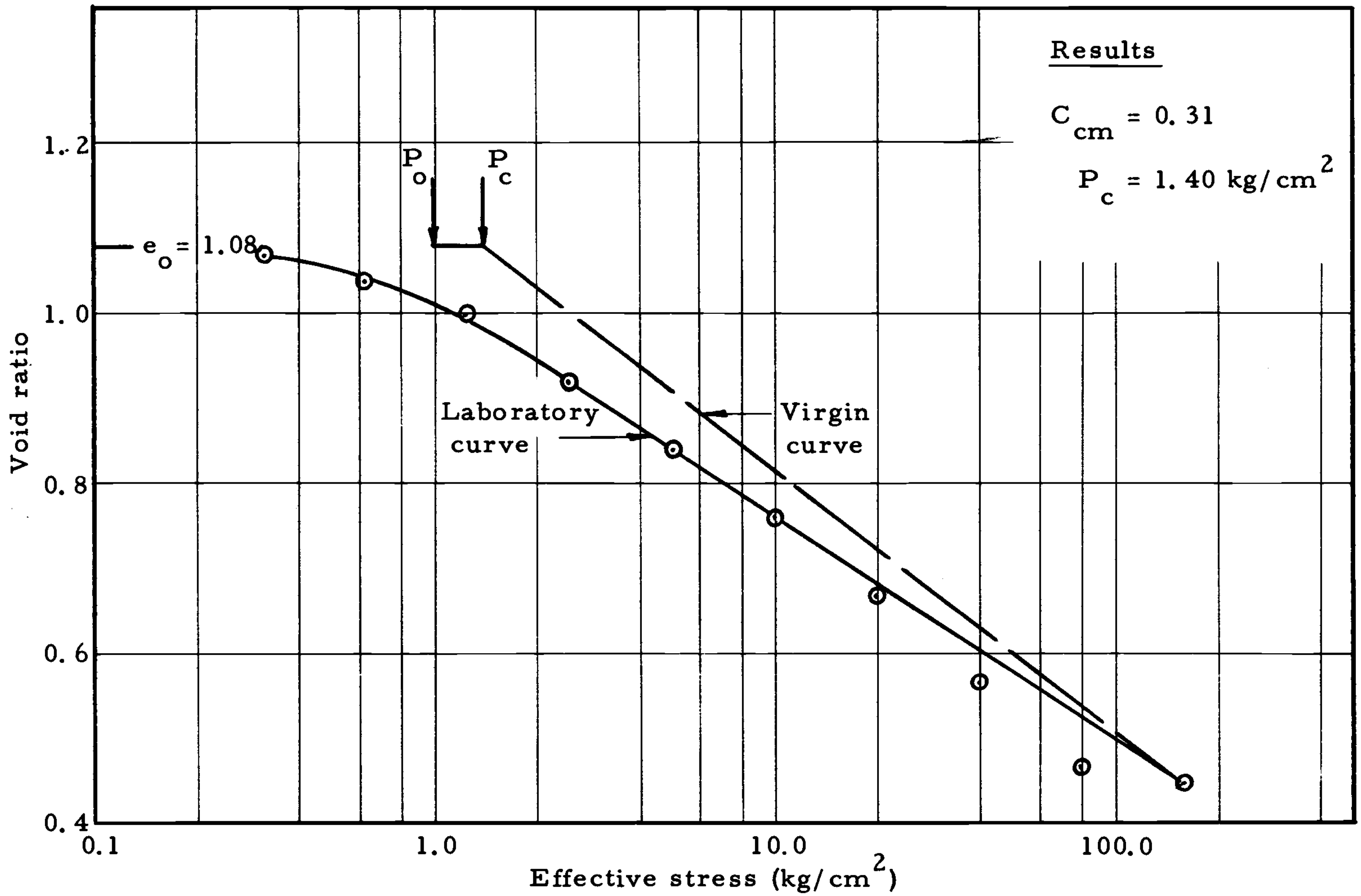

Figure 19. The "e - log (σ')" curve for 30 percent sand.

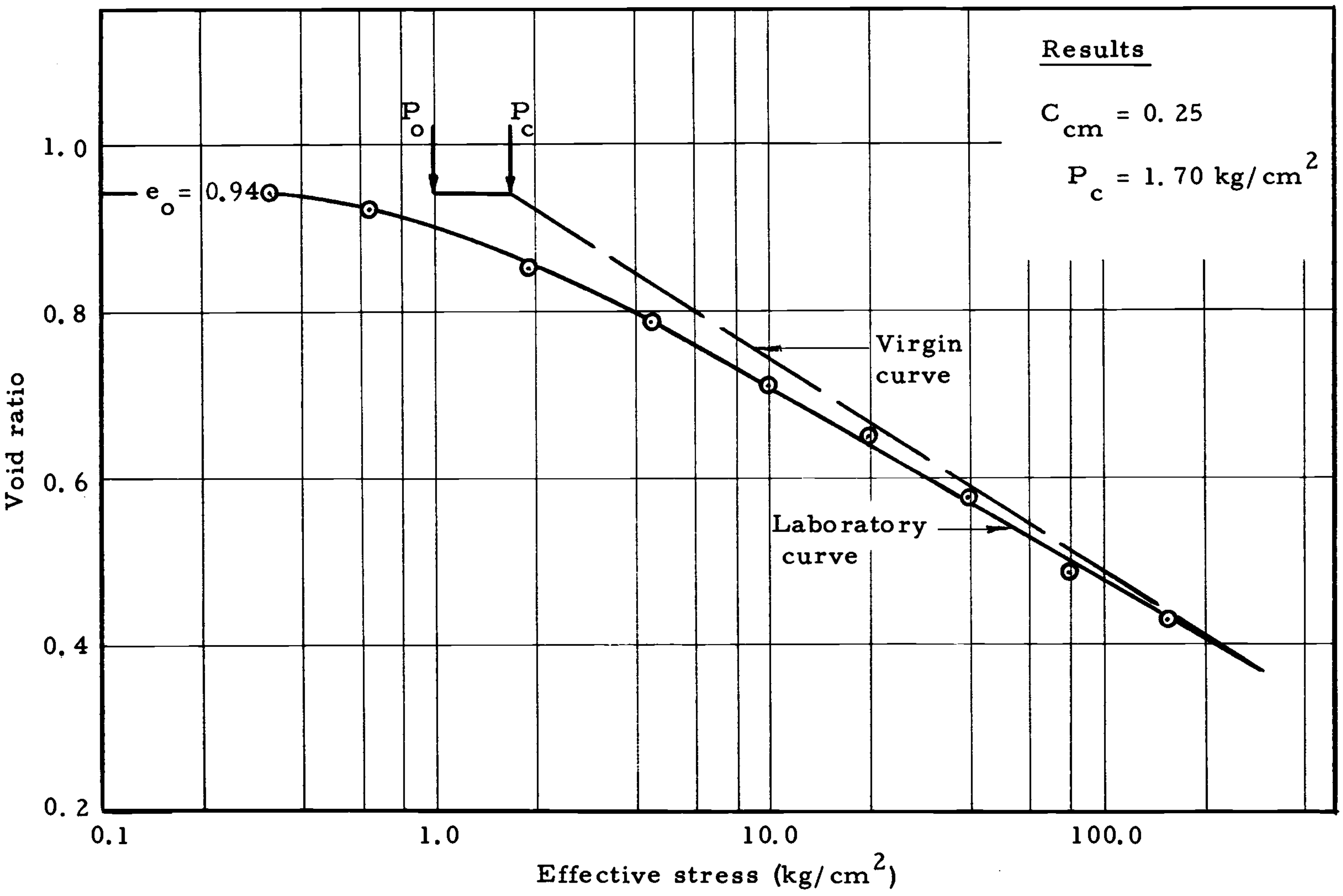

Figure 20. The "e - log (σ')" curve for 40 percent sand.

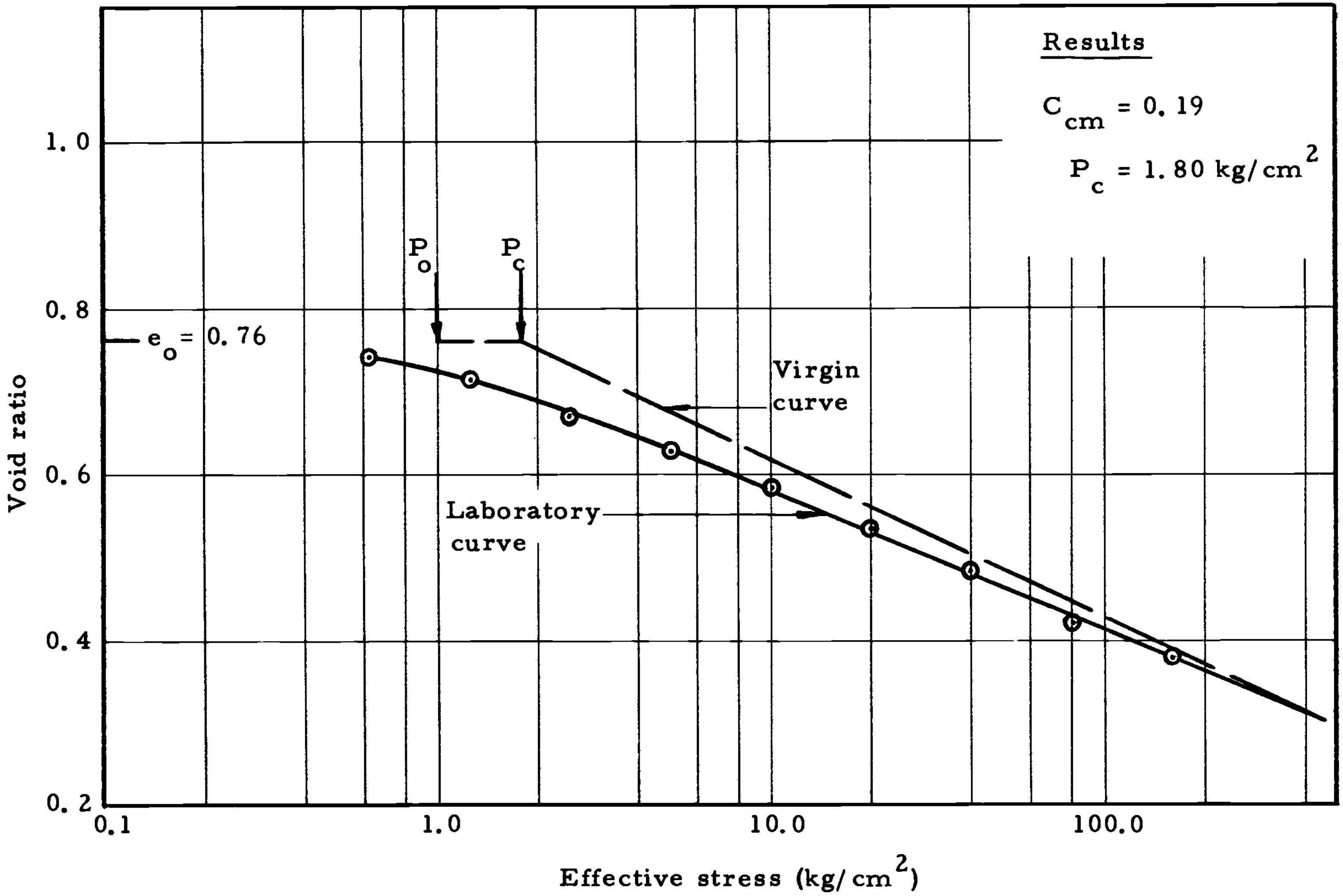

Figure 21. The "e - log (σ')" curve for 55 percent sand.

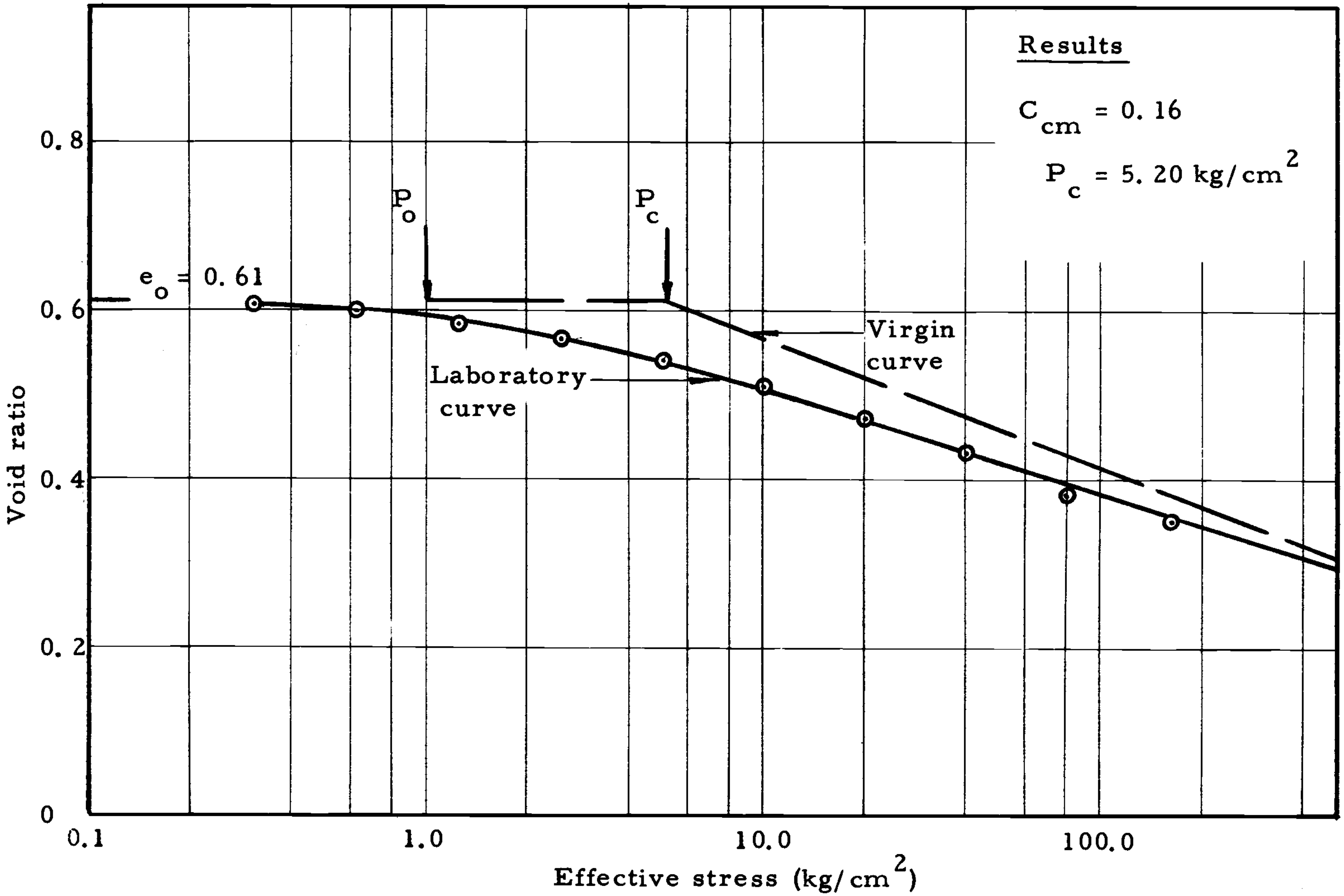

Figure 22. The "e - log (σ')" curve for 65 percent sand.

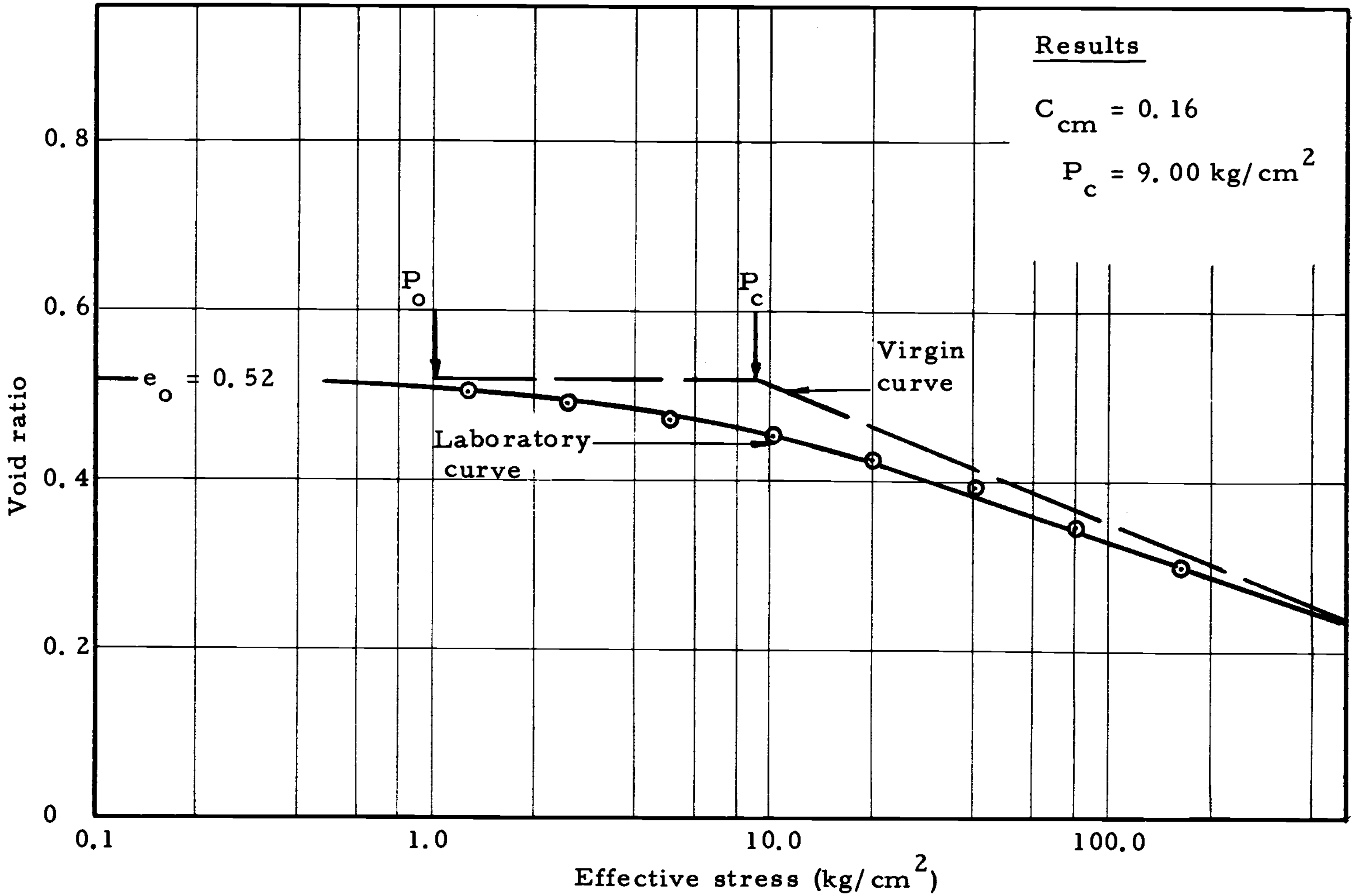

Figure 23. The "e - log (σ')" curve for 75 percent sand.

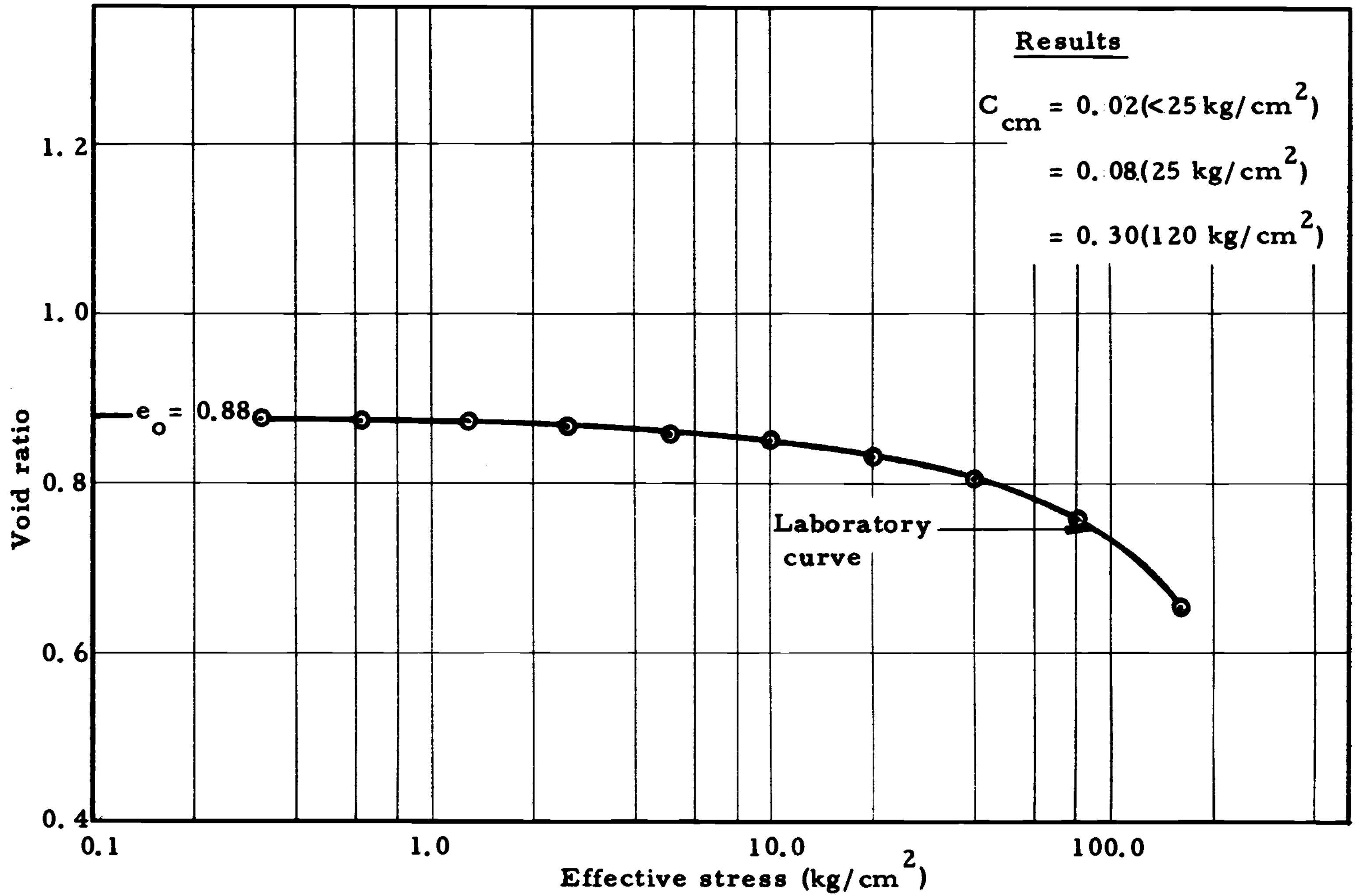

Figure 24. The "e - log (σ')" curve for 100 percent sand.