CONSOLIDATION CHARACTERISTICS OF A VARIED CLAY

by

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CONSOLIDATION CHARACTERISTICS OF A VARVED CLAY

SYNOPSIS

This investigation consists of a laboratory study of the effect of sample diameter, sample thickness, drainage, loading rate and duration on the magnitude and rate of consolidation of a varved clay from the Connecticut River Valley.

Vertical and radial drainage consolidation tests and direct permeability tests were carried out on undisturbed samples of the varved clay trimmed from block samples dug from a clay pit. The clay and silt layers of this varved clay were too thin to be tested as individual layers and hence it was treated as a homogeneous soil during testing.

The results of the tests indicate that:

1. The void ratio-log pressure curve is not affected by the size of the sample, load increment ratio, load duration or drainage path.

2. The values of coefficient of consolidation are dependent on sample size and load increment ratio. Versaggi’s theory can predict the rate of compression of this varved clay test sample provided the length of the drainage path is at least 1”. The values of vertical coefficient of permeability from consolidation tests on thick samples agree well with those from direct permeability tests.

3. Secondary compression is linear with log time and rate of secondary compression is independent of sample thickness. However, it is dependent on load increment ratio and stress level.

4. Extremly careful sampling and testing procedures are necessary to ensure that the laboratory test results are reliable due to the great sensitivity of this clay.
INTRODUCTION

Embankments and foundations built on a compressible soil settle due to the increase of stress in the substrate and consequent readjustment of soil particles. When a saturated soil mass is loaded, the moisture in the soil moves out causing the soil particles to come to a new position of equilibrium. This process is called "Consolidation". The physical understanding of the process of consolidation has been used to develop methods for predicting settlement of structures, which are founded on compressible soil strata. To estimate the magnitude and rate of settlement of such a structure, a consolidation test is run on an undisturbed sample of the foundation soil and the results are extrapolated to the field conditions.

The results of a laboratory consolidation test may be affected by a number of factors such as sample size, load increment ratio and duration (discussed in detail under "Review of Literature"). Extensive research has been carried out by a number of investigators to study the effects of the above variables on a laboratory consolidation test. In most of these investigations the soil used has been a homogeneous soil either undisturbed, remolded or artificially sedimented. Even today, considerable controversy exists on how a laboratory consolidation test is influenced by some of the above factors. This is much more so in case of non-homogeneous soils such as layered clays or varved clays, because the permeability and compressibility of the different layers vary to a significant extent. This investigation was undertaken in an effort to learn how the above factors influence the consolidation characteristic of a
Varved clay. The clay comes from a glacial lake in the Connecticut Valley.

Varves are the layers or alternations of material in sedimentary deposits due to annual or seasonal influence. Each varve represents the deposition during a year (or a season) and consists ordinarily of a lower coarse-grained part deposited in the summer and an upper fine-grained part deposited in the winter. Varves of silt and clay size material occur abundantly in glacial lake sediments. The fineness of grain, mineralogic constitution and the unstable arrangement of individual particles sometimes lead to special problems of engineering significance. The presence of discrete layers formed during the deposition of the varved clay may invalidate the normal analyses based on homogeneous soils being used.

Because of the layered structure and gradual transition of grain size in successive layers, the horizontal permeability greatly exceeds the vertical permeability complicating the calculation of drainage rates.

This investigation is concerned with the laboratory study of the effect of sample size, sample height, drainage, load increment ratio and duration on the magnitude and rate of consolidation of a varved clay. In the series of consolidation tests conducted, the thickness and the diameters of samples were varied; the drainage was both vertical and radial and the loading sequence and load duration were varied; pore pressures were measured at the bottom of some samples during consolidation. Also some direct permeability tests were conducted to determine the vertical and horizontal permeability coefficients of the varved clay. The results are analyzed to explain how a varved clay behaves under a load.
Consolidation and Related Factors

The term consolidation refers to the compression due to change in void ratio that a soil mass undergoes when it is loaded. Any loading applied to a saturated mass of soil causes compression if the pore water is allowed to escape. The soil grains and the pore water are relatively incompressible, and as compression requires a reduction of pore space, it can occur only as fast as the pore water is able to escape. Initially, the load is carried by a pressure in the pore water called "hydrostatic excess pressure". Drainage then begins and the hydrostatic excess pressures and their gradients gradually decrease. The stresses imposed by the loading slowly pass from hydrostatic excess pressures to intergranular pressures with compression of soil occurring simultaneously. In time, the load is entirely taken by the grain structure of the soil. This gradual process involving drainage, compression and stress transfer is called "Consolidation".

The build up and the dissipation of pore water pressure are related to the magnitude and rate of loading, the distance the draining water must travel, the permeability of the soil and the compressibility of the soil structure.

Terzaghi recognized the important influence of hydrodynamic time lag in the consolidation of relatively impervious soils. Using certain simplifying assumptions, he was able to develop a mathematical expression for the rate at which consolidation (Terzaghi's theory of consolidation) could occur during the release of water under pressure. This is called
"primary consolidation". It has been observed both in laboratory consolidation tests and in the field that the compression of the soil continues even after the excess pore pressures are dissipated. This is called "Secondary consolidation". It can be shown that both primary and secondary consolidation are simple empirical divisions of a continuous compression process. There is ample evidence that the relative contribution of each during a laboratory test is in part a function of the test procedure, particularly rate of loading.

Consolidation Test

The object of running a consolidation test is to determine the relation between the void ratio of a soil and the effective pressure (called coefficient of compressibility) and the rate of consolidation. The results of consolidation tests also permit the evaluation of the pre-consolidation pressure and the permeability of the soil sample.

The consolidation test apparatus, first developed by Terzaghi, is called an oedometer. The procedure for the usual consolidation test has been developed over the years, and the general outline is as follows:

An undisturbed sample of soil is cut to fit into a metal ring of 2 1/2" diameter and 1" height. A load is applied to the upper and lower faces of this sample through two porous discs that permit flow into or out of the sample. The load increment ratio is usually one and each increment is allowed to act for one day. The change in height is measured by means of a dial gauge and this dial reading is plotted against time. The void ratio at the 24 hour reading is plotted against the logarithms of pressure.

Load increment ratio: Ratio of load increment to the preceding consolidation load.
Compression under any given load increment can be separated into (i) initial compression, which occurs instantaneously with the loading, (ii) hydrodynamic compression or primary compression and (iii) secondary compression. Techniques for determining the coefficient of compressibility, coefficient of volume decrease, coefficient of consolidation and preconsolidation pressure from the data of a laboratory consolidation test are given in any standard Soil Mechanics textbook.

The void ratio-log pressure (e-log p) curves obtained from the data of the laboratory consolidation tests are straight lines for most natural clays beyond the preconsolidation pressure. The portion of the curve prior to preconsolidation pressure usually has a much flatter slope than the portion for pressures beyond it. Evaluation of stress history of a clay from the e-log p curve is a routine procedure in the consolidation testing of clays using methods proposed by Casagrande, Burland, or Schmertmann.

The rate of consolidation which is represented by the coefficient of consolidation (C_v), in Terzaghi's theory, is determined by applying standard so-called "curve fitting methods" to the time-consolidation curves obtained during a laboratory consolidation test.

Many investigations have been carried out to study the variables involved in the consolidation test. Considerable controversy exists concerning the behavior of soils during consolidation and a review of the current opinions concerning the effects of the following variables is presented briefly.

1. Sample size, diameter and thickness,
2. Load increment ratio and duration,
3. Pore water pressure,
4. Side friction,
5. Secondary compression,
6. Non-homogeneity of soils,
7. Sample disturbance, and
8. Temperature.

Sample Size

A difficulty that arises in the settlement analysis is due to the use of thin samples in laboratory testing whose behavior may not correspond to that of thick strata in the field. Testing of samples of more than conventional thickness is prohibitive from the point of view of side friction, time limitations, and cost.

There is controversy concerning the effect of sample size on consolidation characteristics. Thickness of sample is also important from the point of view of secondary compression. From a comparison of test results on thick and thin samples, Taylor has stated "For like increment durations (greater than 100% primary compression) there is more primary compression and less secondary compression in thick samples than there is in thin samples." This is not strictly valid as the so-called secondary compression occurs during primary compression.

Another factor influenced by the thickness of the sample is the disturbance caused by the sample preparation. Van Loest has suggested that the depth of the disturbance zone caused by trimming is independent of thickness. Therefore the thinner the sample the more important is the effect of soil disturbance.

Lamb\textsuperscript{5} reports the results of a series of tests conducted on five widely different clays in which the diameter and thickness of specimens were varied. The results indicated that the $e$-log $p$ curve was essentially
independent of size; however, the rate of compression was greatly dependent on size; higher $C_v$ values were obtained for larger specimens.

Newland and Allen\textsuperscript{9} in their investigation used samples of thickness varying from 1/4" to a maximum of 2 1/2". They found, \( \varphi \)

(a) Secondary consolidation is independent of the thickness of sample;

(b) $C_v$ shows lack of dependence on thickness of sample; and

(c) $e$-$\log p$ characteristics are only affected to a small extent by changes in sample thickness.

Most of the investigators agree that the sample size has little effect on the $e$-$\log p$ curve but considerable controversy exists over its effect on the rate of consolidation.

**Load Increment Ratio and Duration**

The recommended technique in standard laboratory tests for one dimensional consolidation tests utilizes load increment ratio of one. There is much evidence to show that the results of consolidation tests depend on the load increment ratio and duration of load increments. Load increment ratio and duration affect the coefficient of consolidation, permeability, compressibility and secondary consolidation characteristics of the test sample.

The first systematic investigation of the effect of load increment ratio was carried out at MIT on Boston blue clay by Hiltnor\textsuperscript{6}. Tests were run at 8 load increment ratios, ranging from 0.2 to 2.4. The results indicated slight influence on $e$-$\log p$ curves but a pronounced influence on the coefficient of consolidation by the load increment ratio. According to Tansagi's theory no such variation should exist.

Leonards and Eamish\textsuperscript{10} conducted tests on a residual clay and a glacial clay with varying load durations. Their conclusions were:
1. Provided the load duration is sufficiently long to permit most of the primary compression to occur and provided secondary compression is not abnormally high, the duration of load increment does not have a significant effect on the e-log p curve.

2. The duration of the load increment significantly affects the magnitude of \( C_0 \).

3. However, if the load increment ratio is small and follows a previous increment having a long duration, compression will not occur along the virgin curve until a substantial load increment has been added. By analogy with the effects of precompression, the pressure at which a sharp increase in slope of the e-log p curve is first noticed was termed the "quasi-preconsolidation pressure", and it was suggested that all natural clays should exhibit this phenomenon.

Lemonds and Bisciul^2 have found that the load increment ratio has a distinct effect on the shape of the dial-reading vs log of time curve. They have divided the curves into three typical classes depending on the shape of the curve which results from a high, intermediate or low load increment ratio. Also they found that the rate of secondary compression per unit pressure increment (at a given total pressure) increases rapidly as the load increment ratio is reduced.

Nevaland and Allis^9 found that the rate of secondary compression was independent of the duration of the previous increment. They found that consolidation tests may be completed in a shorter time than is necessary for the standard test, without affecting the e-log p curve to a significant extent, by using a larger load increment ratio and applying the next load increment as soon as primary compression in the preceding increment is complete.
Crawford in his investigation of Leda clay found that preconsolidation pressure is independent of load increment although affected somewhat by slower than normal laboratory loading rates.

All the investigators have come to the conclusion that the load increment ratio and duration do not significantly affect the $e-log p$ curve; however, the load increment ratio significantly affects the rate of consolidation. At small load increment ratios most clays do not behave according to Terzaghi's theory and no method is yet available to predict field consolidation rates accurately from laboratory tests of small load increment ratios.

Pore Water Pressure

In conventional testing procedure, initial pore pressures set up in saturated soil samples are presumed equal to the applied pressure. The accuracy of the measured pore pressure depends on the compressibility of the soil skeleton and stiffness of the pore water pressure measuring system. As per Terzaghi's theory the pore pressure at the instant the load is applied is equal to the applied pressure and is equal to zero at the end of primary compression. However, several investigators have not found this to happen in their consolidation tests on soils.

Determination of 100% primary compression on the time-settlement curves using curve fitting methods is not always consistent. On the other hand, Taylor, on the basis of limited data obtained at MIT, has suggested that dial readings corresponding to 100% primary compression determined on the basis of pore water pressure curve are much more reliable than those obtained by curve fitting methods.

Pore pressures during consolidation tests have been measured by a number of investigators. Leonardis and Circuit demonstrated that, 

(a) The ratio of initial excess pore pressure to the increment
of applied pressure is essentially one for all conditions of loading;

(b) The rate of pore pressure dissipation is reliably predicted by the Terzaghi's theory when the load increment ratio is sufficiently large, and essentially the same value of coefficient of consolidation is obtained from the compression-time curve as from pore pressure dissipation curve;

(c) For small load increment ratios, the Terzaghi's theory cannot predict, even approximately, the rate of pore pressure dissipation (measured pore pressures generally dissipate more rapidly than those predicted from the Terzaghi model using data from curve fitting methods). Consequently, it is no longer meaningful to calculate the coefficient of consolidation from the compression-time curve using curve fitting procedure based on the Terzaghi's model, for low load increment ratios.

Northey and Thomas\textsuperscript{12} found that the duration of load increment on a soil in a consolidation test can affect the maximum pore water pressures developed. If appreciable secondary consolidation is permitted to take place in a soil of low compressibility, the maximum pore water pressure developed in the following increment will be less if secondary consolidation had occurred.

Crawford\textsuperscript{11} showed results suggesting that pore water pressure at the base dissipated to zero after about 1500 minutes for specimens which reached 100% consolidation (as determined from Casagrande's construction) in 100 to 200 minutes.

Parloff, Johnson, and Degroof\textsuperscript{13} found that the pore water pressure at the base of the specimen does not decrease to zero within a day even though the theoretical 100% consolidation points occur at approximately 10 minutes. Increments have been carried out for as long as 72 hours without observation of zero pore pressure.
Christie found that for a given degree of consolidation the pore pressures at the base of the sample are always smaller than those predicted by Terzaghi's theory.

Raymond and Chan found that for a non-homogeneous specimen the coefficient of consolidation based on time to reach 50% settlement and the time to reach 50% dissipation of pore water pressure may be expected to be different.

In conclusion, for higher load increment ratios, the pore pressures can be predicted by the Terzaghi's theory. However, the pore pressures cannot be predicted, even approximately, by the Terzaghi's theory at smaller load increment ratios and the values of coefficient of consolidation computed from time-compression and time-pore pressure curves may differ greatly. This is the area where much research has to be done and more so in the case of non-homogeneous soils where the impermeable layers drain into the more permeable layers.

Side Friction

In a consolidation test sample, the lateral pressure acting on the side walls of the sample container produces frictional resistance to compression of the sample. Due to the existence of this side friction in the consolidation test apparatus, the vertical pressure on any horizontal plane will decrease with depth of sample. As side friction is a function of applied pressure, it can be expected that the e-log p curve becomes flatter with the increase of side friction, thereby giving a smaller value for the compression index than otherwise. In routine testing, the measurement of side friction is generally neglected.

Taylor was the first to carry out tests to find the effect of side friction. He reported that magnitude of frictional force varies from 12 to 22% of the applied pressure for remolded clays and 10 to 15% for
undisturbed clays.

The thinner the specimen for a given diameter, the less side friction because of the smaller lateral surface area in contact with the wall of the container. Leopold suggests a ratio of specimen diameter to thickness of about 3 to 4, to minimize side friction. Diameters greater than 2 1/2 to 2 3/4" are recommended.

Leomards and Girault found that in cases of steel or brass consolidometers, side friction can alter significantly the interpretation made from the results of one dimensional consolidation tests, particularly in the case of pore pressure dissipation rates. Greased teflon liners virtually eliminate side friction if the diameter to height ratio of the consolidometer exceeds six and if the total effective pressure exceeds a critical value.

Northey and Thomas found that, "there is little evidence that soft, low friction coatings, such as teflon, on the walls of an oedometer are more effective in reducing friction than the use of hard, inert, smooth surfaces such as polished chromium plate."

Although side friction measurements are not made in routine consolidation tests, it cannot be altogether ignored. By using a silicone grease or similar lubricant on the inner surfaces of the sample container, side friction can be virtually eliminated in a consolidation test.

Secondary Compression

The delay in settlement which occurs in a stressed soil that cannot be attributed to the flow of water from the voids has been known as secondary compression or secondary consolidation.

The cause of secondary compression has also been a point of controversy for many years. It has been attributed to plastic lag or plastic resistance to settlement, plastic adjustment of the clay structure.
combined with the resistance offered by viscosity of the adsorbed layers to slippage between grains, inability of the mineral skeleton to establish elastic equilibrium with the applied load, and a delayed progressive slippage of grain analogous to creep in other materials.

Since Terzaghi set forth his classical theory of consolidation, secondary compression has been a subject of debate and discussion. In 1940 Taylor and Merchant attempted to derive a relationship for consolidation which included secondary compression in addition to primary compression. They reasoned that the speed at which the void ratio changed in a sample was equal to the speed at which it would change if there were no variation of the intergranular pressure (this speed being the rate of secondary compression) plus the product of the void ratio change with respect to intergranular pressure and the speed with which the intergranular pressure undergoes change (this product being primary compression).

In recent years others have approached the problem through the use of other rheological models. While related the soil skeleton to a model which agrees with Terzaghi's theory when secondary compression is small and also agrees with laboratory test data when secondary effects are large as compared to primary. Tan and Gibson and Lo put forward theories of consolidation accounting for secondary consolidation using different rheological models.

Thompson and Palmer have found the rate of secondary compression to be proportional to the length of drainage path, while Harrower obtained proportionality with the square of the length of drainage path. These conflicting results illustrate the need for further research on this problem.

Leonard and Girault found that the ratio of secondary compression
per log cycle, \( z_{100} \) increases in a consistent pattern with decreasing load increment ratio, irrespective of total pressure. While it is established that this ratio 
\[ \frac{z_{100}}{z_i} \]
increases with decreasing load increment ratio, Newland and Alley found that the slope of secondary consolidation line is independent of thickness of sample, pressure increment ratio, consolidation pressure and duration of previous increment.

While concluded that \( z_{100} \) is a function of void ratio, but is independent of pressure increment or load increment ratio. Leonardo and Altshon found that:

1. To predict the rate of secondary compression from laboratory test data, laboratory consolidation tests must be performed at ground temperatures.

2. It is not certain that \( C_{og} \) for thin samples is valid for a thick layer, nor is it always true that secondary compressions are linear with log time.

3. The scaling law for the effect of drainage path length on secondary compression needs to be clarified.

As can be seen from the review presented above, the cause, magnitude and rate of secondary compression are still controversial points. Much research has to be done to understand this basic behavior of a soil under load.

**Non-homogeneity of Soils**

The Terzaghi's theory of consolidation is based on the assumption that the soil is homogeneous. In nature, a soil deposit is seldom homogeneous throughout its depth; but for purposes of analysis it is assumed to be so. In addition, layered soil deposits are found in nature and the same theory which is used for homogeneous soils may not necessarily
work for layered soils. When soil layers are thick enough tests may be performed on the individual layers and the results combined to suit the field conditions. The rate of consolidation of layered clay deposits may be expected to be dependent on the properties of the individual layers of soil. The properties, according to Terraghi, are the coefficients of permeability and compressibility.

Varved clays are a particular type of layered soil, and although it may be satisfactory to treat a soil deposit of varved clay as a homogeneous soil mass whose properties are the average properties of the individual layers, such an assumption may not be valid when thin specimens of soils are tested in the laboratory. Many varved clays consist of thin layers of different soils which are too thin to test as separate layers but are thick enough that normal test specimens are composed of a small number of layers. The results, therefore, cannot be regarded as average results, but must be considered as results of tests on multilayered soils.

Several theories have been suggested for the solution of consolidation test data on samples of multilayered soils; but little experimental evidence has been presented to support any of the theories. Most of the theories apply to a soil deposit consisting of two homogeneous layers whereas a varved clay sample of 1" thickness may have as many as 8 to 10 layers of clay and silt.

Sample Disturbance

Disturbance of a soil sample amounts in effect to partial remolding of the soil. The disturbance of a sample is caused by field operations, laboratory trimming of specimens and laboratory testing procedures. Both Casagrande and Terraghi have emphasized the fact that rapid consolidation under high load increment ratios causes breakdown of natural
In some cases, there is a cause to remold the soil, which leads to a decrease in its compressibility. Van Zele\(^7\) has presented data which suggest that the thickness of the disturbed zone caused by the remolding is essentially independent of the specimen thickness. This disturbance tended to lower the resulting \(e\)-\(log p\) curve. Since the zone of disturbance appears to be independent of thickness, the thinner the specimen the more important the effect of disturbance.

On the basis of laboratory test results, Rutledge\(^2\) has summarized the general effects of sample disturbance on the laboratory consolidation test as follows:

1. It decreases the void ratio at which the soil will carry any given vertical stress.

2. It obscures the previous stress history of the soil and its preconsolidation load; and

3. The straight line of the remolded compression curve is displaced downward from the laboratory virgin compression curve and its slope is less.

Hamilton and Crawford\(^2\) write that, "in nature, oaks usually exist in equilibrium under unequal principal stress. The stress reduction and changes in prestress ratio that occur due to soil sampling and specimen preparation are probably responsible for most of the disagreement between laboratory test results and field observations."

A knowledge of the effects of soil disturbance would certainly aid in the interpretation of test results and in the settlement analysis.

**Temperature**

Laboratory consolidation tests are generally performed at the room temperature. Pinn\(^2\) found that (1) the coefficient of consolidation varies considerably with temperature in the range of \(\theta^0\) to \(70^0\) F and
reasonably correct values can be obtained by applying a correction, and
(2) $e - \log p$ curve is not affected by the changes in temperature between
the range of $20^\circ C$ to $40^\circ C$. However, Le\textsuperscript{26} found that an increase of
approximately $1^\circ C$ temperature alters the shape and trend of the sec-
ondary consolidation curves.

It is evident that, if the temperature is lowered, higher void
ratios would be obtained for the same consolidation pressures. Hence
the laboratory consolidations tests should be run at field temperatures
to minimize the effects of temperature fluctuations.
Soil Description

The clay that was tested comes from a glacial lake in East Windsor, Conn. The samples were dug from a clay pit located off Route 5 and owned by Kelsey Ferguson Brick Co. The clay is greyish in color and a varve shows a gradual transition of grain size and properties from top to bottom, but the clay and silt layers can be easily distinguished by their dark and light colors respectively. The thickness of these layers varies from 1/16" to 3/8", but layers as thick as 1/2" have been observed in the field.

General Properties

The results of identification tests are summarized in Table 1.

According to Casagrande's plasticity chart, the clay may be classified as a highly plastic clay and the silt as an inorganic silt of low plasticity. The grain size distribution curves are shown in Fig. 1.

The relative percentages of silt and clay fractions in the layers according to KIT classification are also shown in Fig. 1.

The clay mineral was found to be illite as per x-ray diffraction patterns. It has been generally observed by Foreman that illite is the predominant clay mineral in varved clays found east of Lake Superior.

A sample of this varved clay is moderately stiff (undrained shear strength 800-1000 p.s.f.) in its natural undisturbed state. Upon remolding the sample becomes soft and cannot stand under its own weight. The sensitivity of the varved clay is high (70) and instances have been observed where the silt liquified in a bulk sample due to vibrations in
transit. Sampling and handling have to be done with extreme care.
(See Ref. #28)

**Soil Sampling**

The sample sizes used in the tests varied from 2.5" to 5.75" and large blocks of undisturbed clay were needed. As the soil was very sensitive, the sampling method had to be different from the standard methods. Such a method was devised by the authors and successfully used in the field.²⁸

The clay has an overburden of medium sand for a depth of 4 feet. This sand is stripped by the brick company for a length of about 1000 feet and a width of about 20 feet to enable them to excavate the clay for the manufacture of bricks. This permitted the cutting of large undisturbed samples.

**Testing Equipment and Test Procedure**

All the consolidation tests were run in a constant temperature room at temperatures 74³ ± 2°C.

**Consolidation Tests—Vertical Drainage**

The first series of consolidation tests were run on samples of 2 1/4" diameter and 1" height in standard fixed ring brass consolidometers. The drainage was at both top and bottom and the consolidation machine was of a standard platform type. The samples were trimmed using a soil knife and consolidation rings were polished and coated with a silicone grease on the inside to eliminate side friction. The load increment ratio was varied from 0.15 to 1.0 and the load duration was varied from one hour to six weeks. The details of the testing program are given in Table 2. In case of standard and Arteme consolidometers, there was no provision to test samples of different sizes and hence a new consolidometer had to be built.

- 20 -
In the second series of tests, the size of the samples was varied. The truing of a soil sample using a soil lathe was found unsatisfactory (the blade of the soil disturbed the silt layers while truing, unless done with extreme care) and hence the method was modified. A new method of truing a soil sample into a stainless steel consolidation ring was used to get an undisturbed soil sample. Pore pressures were measured at the bottom of the samples during consolidation. For this series, a new consolidometer was designed and built in the machine shop.

The new consolidometer consisted of a brass base plate in which provision was made to seat stainless steel consolidation rings of different diameters and a pressure transducer during testing (Fig. 2). The tests were carried out in a Karel-Warnar pneumatic frame. The load could be applied in 0.1 second and maintained indefinitely, regardless of sample compression. The pore pressure at the base of the sample was measured by a 'Dynisco' pressure transducer and a "Hewlett Packard" X-Y recorder. The transducer was excited by a 6 volt storage battery.

In the second series of tests samples of diameters 2.76", 3.76", 4.76" and 5.76" were tested. The load increment ratio was 1.0 and the load duration was 24 hours. Two samples of 4.76" and 5.76" diameter were tested using a load increment ratio of 0.5. The details are given in Table 2.

Consolidation Tests—Radial Drainage

In the third series of tests, several consolidation tests with radial drainage were conducted. The diameters of the test samples were 3.82", 4.76" and 5.76" and the load increment ratio was one. Pore pressure measurement at the base of the samples was attempted without success.

In this case, the brass base plate had a central hole of 1/2" diameter and 1/2" depth to accommodate a porous stone. This hole was connected to
the side of the base plate where provision for a stopcock was made. The base plate had concentric grooves to accommodate the three sizes of steel consolidation rings.

During the consolidation test the sample was allowed to drain radially toward a central 0.306" drain. A central hole for the drain was cut in the samples by an ordinary cork borer which was centered by means of a template fitting over the consolidation ring. Visual inspection of the hole showed the clay and silt layers undistorted and unaltered.

Direct Permeability Tests

Direct vertical permeability tests were run on samples of \( \frac{1}{2} \) diameter and different heights using the principle of a variable head permeameter. The permeability of the sample was determined at the end of 24 hours after a load increment was put on. Readings were taken at different time intervals until consistent values of coefficient of permeability were obtained.

Two other permeability tests were conducted to determine the horizontal permeability of the varved clay. In one test, the sample was trimmed such that the varves were vertical in the consolidation ring and the permeability test was carried out as explained earlier.

In the other test, a sample of 4.25" diameter and 1.46" height was trimmed and a central hole of 0.306" was bored in it. A loading plate of diameter slightly less than 4.76" diameter was placed on the top of the sample; this loading plate had circumferential slots close to the outer edge such that when positioned on the sample, they would be directly above the outer sand drain. The setup is shown in Fig. 1.

After 24 hours the burette was filled with water and the water entered the central drain and traveled radially toward the outer sand drain through the sample. The fall in head in the burette was measured
at different time intervals and the coefficient of horizontal permeability was calculated. The sample was loaded using a load increment ratio of one. The permeability test was run after each load increment was on the sample for 24 hours until consistent values were obtained. After the test, the sample was removed and the heights of clay and silt layers were noted. The details of the permeability tests are given in Table 2.

After each test (both consolidation and permeability) the sample was cut by a wire saw and the thicknesses of the clay and silt layers were measured using a measuring scale and recorded. The percent clay layers in each test sample are shown in Table 2.
TEST RESULTS

CONSOLIDATION TESTS: VERTICAL DRAINAGE

Compression Behavior

Compression-pressure relationships: Void ratio-log pressure (e-log p) curves for all the tests were drawn in order to compare the magnitude of compression under different load increments for different samples. A typical e-log p curve is shown in Fig. 4.

Different samples had varying thicknesses of silt and clay layers and as such the initial average void ratio was not the same for all the samples. For comparison of tests, the percentage of compression (rather than the void ratio) was plotted against the log of pressure in Fig. 5. It can be seen that all the curves for the samples tested under different test conditions fall into a narrow band.

The magnitude of compression under each load increment gradually increased as the load on the sample increased, attained a maximum under the first or second load increment after the preconsolidation pressure and then decreased as the load on the sample increased. The total compression under a load of 15 T.S.F. was of the order of 25-27% of the initial thickness of the sample.

Preconsolidation pressure: Using Casagrande's method, the preconsolidation pressure for the varved clay was found to be about 2 T.S.F.

*Void ratio is that attained by the soil sample at the end of the load increment and pressure is the effective pressure at the end of the same load increment.
This is shown in Fig. 4. In the first series of tests, the samples tested were from depths varying from 2' to 20'. From the $e$-$log$ $p$ curves of all the tests, the preconsolidation pressure was determined and found to vary between 1.9 and 2.13 T.S.F., though the present overburden is 4' of sand. This may be due to lowering of water table, desiccation or removal of overburden.

Quasi-preconsolidation pressure: This is defined under 'Load increment and duration' in "Review of Literature". In test No. 5 the sample was allowed to rest for 4 1/2 weeks at a load of 1 T.S.F., and then loaded up to 2.144 T.S.F. at a load increment ratio of 0.1; again it was allowed to rest for 6 1/2 weeks at 2.144 T.S.F. and then loaded further on up to 15.4 T.S.F. at a load increment ratio of 0.1.

In test No. 6 the sample was allowed to rest for 5 weeks at a load of 1.88 T.S.F. and then loaded further at a load increment ratio of 0.15.

These rest periods were allowed to find the magnitude of the quasi-preconsolidation pressure effect. The $e$-$log$ $p$ curves for these tests are plotted in Fig. 6. In test No. 5 the $e$-$log$ $p$ curve after the first rest period does not join the conventional $e$-$log$ $p$ curve until a load of 1.6 T.S.F. was put on. This is in the recompression range and hence the effect of quasi-preconsolidation pressure seems to be high, but beyond the preconsolidation pressure, the $e$-$log$ $p$ curve after the rest period joins the conventional $e$-$log$ $p$ curve right after the first load increment. Even in test No. 6, where the first load increment after the rest period straddles the preconsolidation pressure, the same behavior occurs. It can be seen that the effect of the quasi-preconsolidation pressure is almost negligible except in the recompression range.
Rate of Consolidation

Coefficient of Consolidation \( C_v \): To determine \( C_v \) during any load increment, Casagrande's log time fitting method was used. In tests where pore pressures were measured, \( C_v \) was also calculated from the time required for 50% dissipation of pore pressure at the bottom. Typical dial reading-log time and percent pore pressure at bottom-log time curves are shown in Figs. 7 and 8.

In tests of load increment ratio equal to and greater than 0.45, the dial reading-log time curves obtained, with exception of secondary compression, were characterized by Terzaghi theory (called Type I curves by Leomars and Sirwal). In tests of load increment ratio smaller than 0.45, the dial reading-log time curves were such that the secondary compression masked the primary compression (called Type II and III curves by Leomars and Sirwal) and these cannot be characterized by any theory put forward hitherto. In case of Type I curves, Casagrande’s curve fitting method delineates the dial reading corresponding to 100% primary consolidation to a good approximation. In case of Type II and III curves, it was not possible to calculate the value of \( C_v \) using any curve fitting method. Test results typifying Type I and II curves are shown in Figs. 7 and 9 respectively.

Pore water pressure behavior: For a proper understanding of the consolidation process, it is essential to study the pore water pressure behavior in relation to compression. The pore water pressure at the bottom of the samples during consolidation was measured in tests of Series 2 using a pressure transducer. The stiffness of the measuring system to that of the soil skeleton was of the order of \( 5 \times 10^5 \). The response was as rapid as the load application most of the time, but a
time lag of 15-30 seconds was observed during some load increments. The initial pore water pressure recorded was of the order of 92-100% of the applied load increment; sometimes the response was a little greater than 100%, probably due to errors in measurements.

A typical pore water pressure-log time curve is shown in Fig. 3. It was generally observed that the pore water pressure at the base remained above zero long after the specimen reached 100% consolidation as per Casagrande's curve fitting method (at this time the pore water pressure at the base was of the order of 5-6% of the applied load increment).

Similar observations have been noticed by Crawford\textsuperscript{11}, Perloff et al.\textsuperscript{13} and Harden.\textsuperscript{29}

Comparison of theoretical and laboratory curves: Theoretical and laboratory compression-log time curves are shown in Fig. 10 for a load increment in the normally consolidated range of a test of load increment ratio 1.0. The theoretical curve was obtained using the \( G_v \) value computed from the laboratory dial reading-log time curve using Casagrande's curve fitting method. It can be seen that the fit between the theoretical and laboratory compression curves is very good. This was also the case in tests of load increment ratio of 0.5. The agreement between the theoretical and laboratory compression curves was good only in cases of thick samples. In the case of thin samples, the laboratory and theoretical curves did not agree well.

Though the agreement between the theoretical and laboratory compression curves is good for thick samples, the agreement between the theoretical and measured rates of pore pressure dissipation is poor. This can be seen in Figs. 11 and 12. The theoretical rate of pore pressure dissipation was computed from the \( G_v \) values obtained from the
laboratory dial reading-log time curves. In Figs. 11 and 12, the curves are of tests of load increment ratio 1.0 and 0.5, respectively. It can be seen that the pore water pressure dissipates faster than predicted by the Terzaghi theory. However, the agreement between the theoretical and measured rates of pore pressure dissipation is better in tests of load increment ratio 1.0 than in tests of load increment ratio 0.5 as can be seen from Figs. 11 and 12.

$C_v$-log $p$ curves: A plot of $C_v$ vs log $p$ for the tests of Series 2 is shown in Fig. 13. The values of $C_v$ were calculated from dial reading-log time curves using Casagrande's curve fitting method. It can be seen that there is quite a scatter in the recompression range, but the $C_v$ values fall in a narrow band in the normally consolidated range. This variation is due to the difference in sample thickness, percent clay layers and load increment ratio of the different test samples. The effect of these on the value of $C_v$ is discussed later.

Another plot of $C_v$ (calculated from the time required for 50% dissipation of pore pressure at the base of the sample) vs log $p$ is shown in Fig. 14 for four tests of Series 2 in which the load increment ratio was 1.0. It can be seen that, except for test PP3, the curves of $C_v$ vs log $p$ fall in a narrow band for the entire pressure range. In case of test PP3, the $C_v$ values beyond the preconsolidation pressure are quite consistent with those from other tests.

The values of $C_v$ calculated using Casagrande's method and pore pressure data are tabulated in Table 3 for four tests PP1, PP2, PP3 and PP4. It can be seen from Table 3 that, except for two load increments in the test PP2, there is very good agreement between the $C_v$ values obtained by both methods for all load increments except the one which
straddles the preconsolidation pressure (i.e. 1.6 to 3.2 T.S.F.). When the load increment straddles the preconsolidation pressure, after a certain time interval, the consolidation which is taking place near the drainage face (where the effective stress has exceeded the preconsolidation pressure) is governed by a coefficient of compressibility which is greater than that of the soil at the bottom where the effective stress has not exceeded the preconsolidation pressure. In computing $C_v$ using Casagrande's method, average compression values of the sample are made use of. In computing $C_v$ from pore pressure data, the time required for 50% dissipation of pore pressure was made use of and this was measured at the bottom of the sample where the soil is still in the recompression range. Hence there is no good agreement between the $C_v$ values obtained by the two methods.

In case of tests of load increment ratio of 0.5, the agreement between the $C_v$ values obtained by the two methods is poor. This can be seen in Table 4 in which the results of tests PP$_1$ and PP$_6$ are tabulated. The $C_v$ values obtained by the time required for 50% dissipation of pore pressure are consistently higher than those obtained by the Casagrande's method indicating that the pore pressure dissipates faster than predicted by the theory. In case of tests of low load increment ratio, the mid-plane pore pressure dissipates faster than Terzaghi's theory would indicate while the settlements are retarded. Hence there will be discrepancy between the $C_v$ values calculated by the two methods.

General inferences are: (1) The values of $C_v$ computed by the two methods for any particular test of load increment ratio of 1.0 agree well except for the load increment which straddles the preconsolidation pressure, and (2) in tests of load increment ratio of 0.5 and probably
between 0.5 and 1.0, the \( C_v \) values computed by the two methods do not seem to agree for any particular test.

**Variation of \( C_v \):** The value of \( C_v \) calculated from the dial reading-log time curves using Casagrande's curve fitting method depends on:

(a) Percent clay layers in the test sample,
(b) Drainage length of the sample,
(c) Load increment ratio, and
(d) Load duration.

In order to compare the \( C_v \) values of any group of tests, all the variables involved are kept the same except the factor under discussion. Because of the many variables involved, only 3 to 4 tests under any group are available.

(a) Effect of percent clay layers: This can be seen in Fig. 15 in which a plot of \( C_v \) vs percent clay layers is shown. In general, it can be seen that as the percent clay layers in a sample increases, the \( C_v \) value decreases (all the other factors remaining the same). This is reasonable as the permeability of a test sample goes down with the increase in percent clay layers and thus the value of \( C_v \) also decreases. In the recompression range the \( C_v \) value decreases by about 42% as the clay content increases from 36% to 60%. However, in the normally consolidated range the change in \( C_v \) value is between 10 and 30% for the same change in percent clay layers. The higher the consolidation load, the smaller the variation in \( C_v \).

(b) Drainage length of the sample: The effect of the drainage length on the value of \( C_v \) can be seen in Fig. 16. If a test sample is behaving as per Terzaghi's theory, \( C_v \) should be independent of the length of the drainage path. However, it can be seen from Fig. 16
that $C_v$ increases as the length of the drainage path increases. The increase in $C_v$ in the recompression range is much more than that in the normally consolidated range for the same increase in the length of the drainage path.

Further, $C_{v50}$ is found to be greater than $C_{v90}$ in the case of thin samples where the length of the drainage path was about $1/2"$ initially. In the case of thick samples, the $C_{v50}$ and $C_{v90}$ values are almost the same and the samples were behaving according to Terzaghi's theory. This can be seen in Fig. 10 in which the laboratory and theoretical compression curves are compared for a thick sample. Fig. 12 shows the same comparison for a thin sample. It can be seen that neither of the theoretical curves obtained using $C_{v50}$ and $C_{v90}$ fits the laboratory compression curve. However, the theoretical curve obtained using $C_{v50}$ fits the laboratory curve better than the one obtained using $C_{v90}$.

(c) Effect of load increment ratio: Fig. 18 shows the effect of load increment ratio on the value of $C_v$. As the load increment ratio decreases, the $C_v$ values decrease for the same pressure in the normally consolidated range. This is also observed in tests of Series 2 where the load increment ratio was decreased from 1.0 to 0.5 for two thick samples. However, the same trend is not seen in the recompression range.

(d) Effect of load duration: Fig. 19 shows the effect of load duration on the value of $C_v$. In general, the values of $C_v$ of tests of one-hour duration are higher than those of tests 1, 3 or 7 day duration. No definite trend can be seen in the values of $C_v$ because of load duration. Similar observations have been noticed by Leonards and Radial and they $C_{v50}$ and $C_{v90}$ are coefficients of consolidation calculated from the time required for 50% and 90% consolidation under a load increment using Casagrande's curve fitting method on dial reading-log time curves.
have indicated that this variation of $C_v$ depends on the soil type and consolidation pressure.

Permeability Tests (Vertical Drainage)

The results of permeability tests $P_1$ and $P_2$ are shown in Fig. 20. In test $P_2$, the total thickness of clay layers in the test sample was equal to that of the silt layers, whereas in case of test $P_1$, the thickness of clay layers was more (56%) than the thickness of silt layers (44%). Hence, the log vertical coefficient of permeability ($K_v$) vs log p curve for the test $P_1$ is lower than that of test $P_2$.

The vertical coefficient of permeability was also calculated from the values of $C_v$ (from pore pressure data) and consolidation test data in case of tests of load increment ratio of 1.0 of Series 2. These values are also plotted in Fig. 20. The percentages of clay and silt layers in the consolidation test samples varied from 40:60 to 68:32 and these percentages for the different test samples are shown in Fig. 20. Because of the difference in the percentages of clay and silt layers between the permeability and consolidation test samples, the values of $K_v$ calculated from the consolidation test data scatter in the plot. Depending on the percentages of clay and silt layers, the values of $K_v$ computed from $C_v$ tend to fall below or above the curves of the permeability tests.

In general, the values of $K_v$ measured directly and calculated from the consolidation test data seem to agree very well.

Secondary Compression

To study the rates of secondary compression, the load duration was increased from 1 day to 3 days and 7 days (see Table 2) and some samples were allowed to rest under certain loads for longer periods of time.
Figs. 21(a), (b) and (c) show the typical results of these tests. The ordinate is in terms of change in void ratio (Δε) during consolidation. It can be seen that secondary compression is linear with log time. Rates of secondary compression ($R_2 = \Delta\epsilon/10\text{g cycle of time}$) are marked for each load increment in Figs. 21(a), (b) and (c). Fig. 22 gives a plot of $R_2$ vs log p for tests 11 and 12 of Series 1 (see Table 2). It can be seen from Fig. 22 that $R_2$ gradually increases in the reconsolidation range and for one load increment beyond preconsolidation pressure and then decreases as the pressure on the sample increases. This was also noted in other tests in which samples were thicker.

The ratio of secondary compression to primary compression varied from 0.06 to a maximum of 0.25 in a daily load increment test of load increment ratio of 1.0. In the normally consolidated range, this ratio was about 0.12. As the load increment ratio decreased, this ratio increased and this is explained further.

Fig. 23 shows the effect of load increment ratio on rate of secondary compression. The abscissa is total pressure at the end of an increment and the ordinate is secondary compression in inches per log cycle of time ($R'$), per unit pressure increment ($\Delta p$), per unit height of the sample ($H'$), i.e., $R'/\Delta p.H'$ in.²/1b. The curves are for pressures beyond the preconsolidation pressure of 250.S.F. It can be seen that as the load increment ratio decreases, the ratio $R'/\Delta p.H'$ increases for any value of total load on the sample. For example, at a total pressure of 450.S.F., $R'/\Delta p.H'$ increases 3.5 times as load increment ratio decreases from 1.0 to 0.15. This indicates that large rates of secondary compression per unit height, per unit pressure increment (at a given total pressure) are associated with small load increment ratios. The same trend was
observed in the recompression range also.

**Effect of Thickness of Sample**

The main effect of the thickness of a sample is to retard primary compression. Fig. 24 shows a plot of percent compression vs log time for a load increment beyond the preconsolidation pressure. At the end of the load increment, the percent strain in the different samples shown is not the same because of the difference in the thickness of silt and clay layers in the samples.

It can be seen from Fig. 24 that secondary compression lines for all the samples are essentially parallel, though the length of drainage path \( (h) \) varied from 0.442" to 1.4388". This is also true in the recompression range as can be seen from Fig. 25. This indicates that the rate of secondary compression is independent of the length of the drainage path for any load increment.

**CONSOLIDATION TESTS: PARTIAL DRAINAGE**

In these tests the drain diameter was kept constant and the diameter of the sample was varied from 3.82" to 5.79". Shields and Rowe\(^3\) recommended that accurate measurements of the horizontal coefficient of consolidation and settlement of a clay sample during radial drainage may be made with the aid of a model sand drain where the ratio of odometer to drain diameter is 20. In the tests of Series 3 (see Table 2) this ratio varied from 12.5 to 18.5.

Compressibility was not influenced by the direction of drainage, i.e., the percent strain-log \( p \) curves for the radial drainage tests fall into the narrow band of the percent strain-log \( p \) curves of other tests shown in Fig. 5.

To determine the horizontal coefficient of consolidation \( C_v \),
methods similar to Taylor's square root of time fitting and Cassagrande's log time fitting for one dimensional consolidation were devised by the Junior Author for the two dimensional case. The equation describing consolidation under the test conditions, with radial flow to a nonmeasured central drain having infinite permeability, was presented by Barron.  

\[ U_r = 1 - e^{-2\frac{n^2}{a^2}} \]  
(for equal strain consolidation)  \hspace{1cm} (1)  

in which \( n = \frac{n^2}{a^2} - 1 \) \hspace{1cm} (2)  

\[ n = \frac{a^2}{u^2} \]  
\hspace{1cm} (3)  

and \[ T_r = C_{fr} t / a^2 \]  
\hspace{1cm} (4)  

in which \( U_r \) = degree of consolidation due to radial drainage,  

\( T_r \) = time factor for radial drainage,  

\( d_n \) = diameter of sample drained,  

\( d_u \) = diameter of the central drain,  

\( t \) = consolidation time,  

and \( C_{fr} \) = horizontal coefficient of consolidation.  

Equation (1) was solved for the dimensions of the test apparatus and theoretical plots of \( U_r \) vs \( T_r \) and \( U_r \) vs \( \log T_r \) were drawn. In cases of \( U_r \) vs \( \sqrt{T_r} \), the theoretical curve was a straight line between 20% and 60% consolidation and the abscissa of the curve at 90% consolidation was approximately 1.15 times the abscissa of the straight line. This relationship was used to determine the point of 90% consolidation on the laboratory curve. This is shown in Fig. 26. \( C_{fr} \) was calculated from the time required for 90% consolidation.

*This value varied from 1.145 to 1.153 for the three tests and as per Michells and Herbert, it depends on the value of \( n \) and can be expected to vary from below 1.10 for very small values of \( n \) to the value of 1.22 observed by Nescio and Uriel for radial outward drainage with no central drain, or for very small central drains for which \( n \) approaches infinity.  

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In case of \( U_p \) vs log \( T_p \), the theoretical curve was a straight line between 40% and 80% consolidation and the intersection of this tangent and the asymptote to the theoretical consolidation curve was at the ordinate of 100% consolidation. The use of intersection of the two corresponding tangents to the laboratory curve was made use of in determining 100% consolidation. This is shown in Fig. 27. \( C_{VT} \) was calculated from time required for 50% consolidation.

The values of \( C_{VT} \) calculated by the two curve fitting methods agreed well for the entire pressure range. A plot of \( C_{VT} \) vs log \( p \) is shown in Fig. 28 for the three radial drainage consolidation tests (\( C_{VT} \) used is the average of the values obtained by the two methods). It can be seen that the values of \( C_{VT} \) fall within a narrow band for the entire pressure range; i.e., \( C_{VT} \) is independent of sample size and ratio of sample diameter to drain diameter provided it is at least 12.

It was also noticed that the slope of the secondary compression line was linear with log time. The rate of secondary compression under any load increment was the same as in the vertical drainage consolidation tests and this is shown in Fig. 24.

**Permeability Test (Radial Drains)**

A direct horizontal permeability test (\( P_h \) -- see Table 2) was run on a sample of varved clay to determine the horizontal coefficient of permeability (\( K_h \)). The results are shown in Fig. 29. The values of \( X_2 \) calculated from the \( C_{VT} \) values of the radial drainage consolidation tests \( R_1 \), \( R_2 \) and \( R_3 \) are also plotted in Fig. 29 for comparison. In the sample of test \( P_4' \) the percentages of silt and clay layers were 60 and 40 respectively. In case of test samples of \( R_1 \), \( R_2 \), and \( R_3 \), the same percentages were 50:50, 40:60 and 47:53 respectively. Even though the percentages
of silt layers in samples of tests R₂, R₃, and R₄, were less than that in sample of test P₄, the \( K_h \) values calculated from the data of the radial drainage consolidation tests were higher than those obtained in the horizontal permeability test for the same effective pressures in the recompression range. This can be seen in Fig. 29 and can be explained as follows. The assumption in the theory of radial flow was that the pore water flowed entirely radially into the central drain. In case of a varved clay sample, a combination of radial flow along the silt layers and axial flow from the clay layers into the silt layers could be expected in the test. This is shown in Fig. 30(a). The time required for any degree of consolidation would depend on the relative permeabilities and thicknesses of clay and silt layers. Consolidation under the conditions described above could be faster giving a higher value of \( C_{nt} \) than otherwise. This high value of \( C_{nt} \) would give an apparent value of \( K_h \) which is higher than the real \( K_h \). The authors feel that this phenomenon would only be significant when the ratio of drainage path length to varve thickness is less than 30 and so would not generally occur in the field where the above ratio is much greater than 30.

In case of a direct horizontal permeability test, the boundary conditions were such that the flow of water was essentially parallel to the varves as shown in Fig. 30(b). A true value of \( K_h \) would be obtained in such a test.

It can also be seen from Fig. 29 that the \( K_h \) values from the radial drainage consolidation tests, in the normally consolidated range, fall below those obtained from the horizontal permeability test. The percentage of clay layers in the consolidation test samples is more than that in permeability test samples; the clay layers being more compressible than
the silt layers, the permeability of the consolidation test samples was reduced to a greater extent than that of the permeability test sample for the same pressure increment. Hence the $K_h$ values from the consolidation tests fall below those obtained from the permeability test. If the values of $K_h$, calculated from the $G_{yy}$ values, had been the true values, they would have been much lower than those from the horizontal permeability test for the same pressure increment in the normally consolidated range.

Table 5 gives the results of vertical and horizontal permeability tests on two samples at different load increments.

For comparison, the ratio of $K_h$ and $K_v$ is calculated at the end of every load increment. The samples in the two tests have different percentages of silt and clay layers. It can be seen that the ratio $K_h/K_v$ is more or less constant in the recompression range and increases 3-4 times in the normally consolidated range. In the normally consolidated range, the compression is much greater than in the recompression range and hence the permeability reduces to a considerable extent. As the direction of loading is the same as that of deposition of clay in nature (i.e., vertical), the vertical permeability is affected more than the horizontal permeability due to the horizontal orientation of the clay particles.

Consolidation Tests (Varves Vertical in Cylindrically)

Tests 14, 15, and 16 (see Table 2) were conducted on samples cut with their axes horizontal so that when positioned in the oedometer cell, the varves were in the vertical plane. In this case, the axis of loading and drainage were both parallel to the varves.
The percent strain-log p curves for these tests also fall into the narrow band of percent strain-log p curves of other tests shown in Fig. 5. The preconsolidation pressure, estimated from the e-log p curves, was of the order of 1.9 to 2.15 T.S.P. These results indicate that the compressibility of the varved clay is the same in both horizontal and vertical directions and imply that the clay has been subjected to precompression in the horizontal direction to the same magnitude as in the vertical direction.

Although the compressibility is the same in both horizontal and vertical directions, the rate of compression in these tests is different from that in the radial drainage consolidation tests where the drainage is also along the varves. In case of a radial drainage consolidation test, the loading is in the same direction as the deposition, whereas in case of a test with varves vertical in the oedometer the loading is perpendicular to direction of deposition. The structure of the clay behaves differently under the two different directions of loading and hence the coefficients of consolidation (in the normally consolidated range) derived from the tests in which the varves were vertical in the oedometer were intermediate in value between those from the vertical and radial drainage consolidation tests. The values are shown in Table 6. Similar findings were reported by McKinlay.

The values of horizontal coefficient of permeability obtained from a direct permeability test (P_3) — see Table 2) in which the varves were vertical in the oedometer are smaller than those obtained from a direct horizontal permeability test. This is shown in Fig. 29. This is due to the difference in the percentages of silt and clay layers in the samples and also due to the difference in the directions of loading to the structure of clay which affects permeability.
DISCUSSION OF TEST RESULTS

The consolidation tests on this varved clay have indicated that it can be treated as a homogeneous soil for carrying out tests and consistent results have been obtained from consolidation tests. In spite of the difference in the compressibility characteristics of clay and silt layers, the percent strain-log pressure curves for different samples fall within a narrow band regardless of sample size, load increment ratio, load duration and drainage. The width of the band was equivalent to a compression of 1-2% in the recompression range and 3-4% in the normally consolidated range.

Terzaghi's theory can reliably predict the rate of compression of a thick sample in the laboratory when applying a load increment ratio of 0.45 or greater. Barcelo has discussed a nonlinear viscosity mechanism to explain the consolidation behavior of a clay and with experimental evidence has shown that behavior according to Terzaghi's theory is approached when the load increment ratio is large or when the drainage length of the sample is large. This type of behavior was noticed in case of this varved clay, i.e., the consolidation behavior was greatly influenced by the load increment ratio and the length of the drainage path. This can be seen in Figs. 18 and 19. The value of $G_\phi$ decreases as the load increment ratio decreases and increases as the length of the drainage path increases. The physical reason for the important effect of the load increment ratio is the high non-linearity of the viscosity law which governs the consolidation process. A large load increment in
causing a high rate of strain will develop only a relatively small vis-
ous resistance, which will be masked by the large increment in pore
pressure leading to Tensagi behavior. A small load increment in causing
a low rate of strain will develop viscous resistance which may be compa-
rable with the load increment, and hence viscous effects rather than pore
Pressure will dominate.

When the drainage length of a sample is small, the primary stage is
rapid and since viscous effects are dependent on rate of strain, secondary
effects will dominate. Put in another way, if the primary stage is
rapid the creep is left undeveloped, whereas if it is slow there will be
little undeveloped creep to follow. This explains the dependence of \( C_v \)
on the length of the drainage path.

In the case of thick samples tested at a load increment ratio of
1.0, the \( C_{yv} \) (coefficient of consolidation calculated from the dial
reading-log time curve) was equivalent to \( C_y \) (coefficient of consoli-
dation calculated from pore pressure data), but as the load increment
ratio was decreased to 0.5 (keeping the length of the drainage path the
same in both the cases), the viscous effects increased, and \( C_{yv} \) was less
than \( C_y \). This has also been observed by Harder.29

In addition to the load increment ratio and length of the drainage
path, the other variables involved are the percent clay layers in the
test sample and load increment duration. To eliminate the effect of the
difference of the percent clay layers in test samples, the tests should
be run on samples with 50% (+5%) clay layers. As seen from Fig. 19
the variation between the \( C_y \) values of 3 day and 7 day load duration
tests is small and hence the one day load incremental load tests seem to
be reasonable in predicting the consolidation characteristics of this
Varved clay.

In the field the load increment ratio may be smaller than in the laboratory test, but the thickness of the deposit is such that the drainage length in the field is much greater than that in the laboratory. Because of this the primary compression requires considerable time and hence the field deposit may behave according to Terzaghi's theory. Field data should be sought to check this point.

In case of radial drainage consolidation tests, \( C_{vr} \) calculated from the compression-time curves gave consistent values for tests of load increment ratio of 1.0. However, the values of \( C_{vr} \) would be dependent on load increment ratio. The values of \( C_{vr} \) obtained from laboratory radial drainage tests would be higher than those that can be expected in the field because in the laboratory tests the sample is small compared to varve thickness and the boundary conditions are such that the pore water from the clay layers flows into more permeable silt layers as shown in Fig. 30(a), i.e., to a certain extent drainage is both vertical and radial thus giving a higher rate of consolidation than if the flow had been entirely radial. In the field, the distance between the sand drains will be many times greater than the thickness of clay varve and hence the flow of the pore water will be essentially radial except around the drain. Hence the laboratory \( C_{vr} \) values will have to be modified before applying them to the field conditions.

The slope of secondary compression line was found to be linear with log time. For any particular load increment, the rate of secondary compression was the same regardless of length of the drainage path. However, as the load increment ratio decreased from 1 to 0.15, the magnitude of secondary compression per log cycle of time, per unit thickness
of sample, per unit load increment increased by a factor of 5.5 at a pressure of 4 T.S.F.

**Field Application**

To estimate the behavior of the field deposit under a certain load, laboratory consolidation tests should be run on samples (with drainage lengths of at least 1 ft) from different depths at load increment ratios comparable to those in the field. The magnitude of compression can be calculated from the percent strain-log p relationship. To estimate the rate of consolidation in the field due to excess pore pressure dissipation, the laboratory values of coefficient of consolidation might have to be modified depending on the type of load in the field. Two cases are considered here:

1. Uniform and extensive load over a large area of the clay deposit; in such a case, the load increment ratio varies with depth. Assuming that no sand drains are installed in the clay deposit, all the excess pore pressures will have to dissipate in the vertical direction. One-dimensional vertical drainage consolidation tests should be run on samples from different depths at corresponding load increment ratios to determine the values of coefficient of consolidation. These $C_v$ values will vary from one sample to another as the load increment ratio is different. To estimate the rate of consolidation in the field, (i) an average of the values of $C_v$ obtained from tests on samples from different depths can be used; or (ii) to get a more exact solution, the theory proposed by Schifman and Gibson $^{35}$, which takes into account the variation of $C_v$ through the depth of the clay stratum, can be used.

2. Load same as in (1) but sand drains are installed in the clay deposit; in such a case, the horizontal coefficient of consolidation
\(Q_{vp}\) has to be determined to estimate the rate of consolidation in the field. A radial drainage consolidation test is a difficult one to be run and the \(Q_{vp}\) obtained is questionable. Hence an alternative method to calculate the actual \(Q_{vp}\) values from laboratory permeability tests is proposed.

Horizontal and vertical permeability tests (with varying horizontal in both tests) should be run on samples next to each other to establish a relationship between \(K_h\) and \(K_v\) (similar to one in Table 5). This relationship will be unique for any load increment in the field. Once this relationship is established, one-dimensional vertical drainage consolidation tests should be run on undisturbed samples from different depths at corresponding load increment ratios and for any sample \(Q_{vp}\) can be calculated from the relationship \(Q_{vp} = Q_v \times K_h/K_v\). This \(Q_{vp}\) should be used to estimate the rate of consolidation in the field due to radial drainage towards the sand drains.

To estimate the rate of secondary compression in the field, the laboratory rates of secondary compression of different samples loaded at the corresponding field load increment ratios can be used.
CONCLUSIONS

The following conclusions are intended to reflect the major findings of this investigation on varved clay from the Connecticut Valley.

1. Sampling Procedure

The field and laboratory sampling techniques used were successful in getting good undisturbed soil samples. Frequent checks on the trimmed soil samples indicated that they were undistorted and undisturbed. The laboratory samples were found to be far better than the standard Shelby tube samples.

2. Test Results

(a) The clay has been subjected to a precompression pressure of 2 T.S.F. both in the vertical and horizontal directions. The effect of quasi-preconsolidation pressure is negligible except in the recompression range.

(b) Provided the load increment duration is sufficiently long to permit the completion of primary consolidation, the diameter of the sample, the height of the sample, load increment ratio, load duration and drainage do not significantly affect the void ratio-log pressure curve.
(c) Though a sample of varved clay is non-homogeneous, Terzaghi's theory can reliably predict the rate of compression of a sample in the laboratory provided the load increment ratio is greater than 0.45 and the length of the drainage path is at least 1'.

The value of vertical coefficient of consolidation is independent of sample diameter but is dependent on sample height and load increment ratio. The load duration does not have a significant effect on the values of vertical coefficient of consolidation provided each load is left for at least 24 hours on the sample.

(d) Radial drainage consolidation tests give higher values of horizontal coefficient of consolidation than the values that can be expected in the field. The value of the horizontal coefficient of consolidation is independent of sample size.

(e) The values of vertical coefficient of permeability calculated from consolidation test data agree well with the results of direct permeability tests for comparative samples.

(f) The slope of secondary compression line is linear with log time. The rate of secondary compression, under any particular load increment, is independent of sample thickness (or drainage path). However, as the load increment ratio decreases, the magnitude of secondary compression per log cycle of time,
per unit thickness of the sample, per unit pressure increment (at a given total pressure) increases considerably.

(g) The magnitude and rate of consolidation of the varved clay deposit in the field under a load can be as accurately estimated from the data of laboratory consolidation tests as for a homogeneous deposit. The values of coefficient of consolidation obtained from laboratory tests might have to be modified depending on the type of load and how well the drainage paths can be defined in the field.
RECOMMENDATIONS

To eliminate the effect of many variables involved in estimating the coefficient of consolidation from the laboratory consolidation tests, the following procedures are recommended:

1. The test sample should be at least 3" in diameter and thickness of the sample should be at least 1" and contain at least 3-4 varves. The ratio of diameter to height of the sample should be 3 in any case.

2. Consolidation tests performed in the conventional apparatus should have single drainage.

3. The percent clay and silt layers should be equal within 2%.

4. The laboratory consolidation tests should be run at load increment ratios corresponding to those in the field.

5. Each load increment should be left on the sample for at least 24 hours.

6. The consolidation rings should be coated with silicone grease to minimize friction.

7. Laboratory tests should be run as close to field temperatures as possible to eliminate the effect of temperature on the rate of compression.

8. Extreme care should be taken in handling and trimming the varved clay samples.
ACKNOWLEDGEMENTS

The authors wish to express their gratitude to the Joint Highway Research Advisory Council which sponsored this research.

The clay samples used in this research came from Koehy Ferguson Brick Co. Their cooperation is gratefully acknowledged.
BIBLIOGRAPHY


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<th>Bulk Sample</th>
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<td>70-74%</td>
<td>44-48%</td>
</tr>
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<td>Liquid limit</td>
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<td>72</td>
<td>50</td>
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<td>Plastic limit</td>
<td>-</td>
<td>33</td>
<td>31</td>
</tr>
<tr>
<td>Plasticity index</td>
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<td>37</td>
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<tr>
<td>Specific gravity</td>
<td>2.6</td>
<td>2.8</td>
<td>2.79</td>
</tr>
<tr>
<td>Clay fraction (&lt;0.002mm)</td>
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<td>66.0</td>
<td>18.0</td>
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<tr>
<td>Degree of saturation</td>
<td>95-97%</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Total density p.o.f.</td>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sl. No.</td>
<td>Test No.</td>
<td>Sample</td>
<td>Sample</td>
</tr>
<tr>
<td>--------</td>
<td>----------</td>
<td>--------</td>
<td>--------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>dist. (ins.)</td>
<td>height (ins.)</td>
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<td>3</td>
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**Series No. 1 — Standard Drains Consolidations with Double Drains**

| 1 | 1 | 2.5 | 1.0 | 1.0 | 24 hours | 1/4-8 | 58 | 3 |
| 2 | 2 | 2.5 | 1.0 | 1.0 | 24 hours | 1/4-8 | -  | - |
| 3 | 3 | 2.5 | 1.0 | 1.0 | 24 hours | 1/4-8 | 57 | 3 |
| 4 | 4 | 2.5 | 1.0 | 1.0 | 24 hours | 1/16-16 | 30 | 3 |
| 5 | 5 | 2.5 | 1.0 | 1.0 | 24 hours | 0.125 | 37 | 3 |

6

7

8

9

6**

7

8

9

The thicknesses of clay and silt layers in a test sample were measured at the end of the test using a measuring scale.

** Unloaded from 2 TSP to 1/8 TSP and then reloaded to 16 TSP.

*** 1 TSP maintained for 4 1/2 weeks and 2.144 TSP maintained for 6 1/2 weeks.

**** 1.82 TSP maintained for 5 weeks.
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<td>11</td>
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<td>1/16-8</td>
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<td>12</td>
<td>2.5</td>
<td>0.92</td>
<td>1.0</td>
<td>7 days</td>
<td>1/8-8</td>
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<td>0.1-12.8</td>
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<td>24 hours</td>
<td>0.1-18</td>
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<td>1.0</td>
<td>1.0</td>
<td>24 hours</td>
<td>1/8-8</td>
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<td>24 hours</td>
<td>0.1-6.6</td>
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<td>17*</td>
<td>17</td>
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<td>1.0</td>
<td>1.0</td>
<td>24 hours</td>
<td>0.1-12.8</td>
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<td>18**</td>
<td>18</td>
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<td>1.0</td>
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<td>24 hours</td>
<td>0.1-12.8</td>
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<td>0.6</td>
<td>24 hours</td>
<td>0.1-13.52</td>
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<tr>
<td>20**</td>
<td>20</td>
<td>4.0</td>
<td>0.750</td>
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<td>0.1-6.4</td>
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<td>21</td>
<td>21</td>
<td>3.76</td>
<td>0.942</td>
<td>1.0</td>
<td>24 hours</td>
<td>0.1-6.4</td>
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** SERIES NO. 2 -- SPECIAL CONSOLIDOMETER WITH SINGLE PRELOAD **

(pore pressure at bottom of samples was measured)

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<th>2.76</th>
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<td>0.7935</td>
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<td>24 hours</td>
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<td>44</td>
<td></td>
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* Varves vertical in the sample.
** Antreas consolidation testing machine.
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<td></td>
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<td>1.5670</td>
<td>1.0</td>
<td>24 hours</td>
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<tr>
<td>25</td>
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<td>5.76</td>
<td>1.8400</td>
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<td>24 hours</td>
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<td>26</td>
<td>PP₃</td>
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**SERIES NO. 3 - SPECIAL CONSOLIDATION WITH RADIAL DRAINAGE**

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<td>R₁</td>
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<tr>
<td>29</td>
<td>R₂</td>
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<td>1.5900</td>
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<td>30</td>
<td>R₃</td>
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<td>1.9600</td>
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</table>

**SERIES NO. 4 - PERMEABILITY TESTS**

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<td>P₁</td>
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<td>1.4065</td>
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<td>P₂</td>
<td>4.23</td>
<td>1.6500</td>
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<tr>
<td>33</td>
<td>P₃</td>
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<td>1.5275</td>
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<tr>
<td>34</td>
<td>P₄</td>
<td>4.25</td>
<td>1.4600</td>
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</table>

- Permeability in the vertical direction determined 24 hours after each load increment.
- Varies vertical in the sample.
- Permeability in the horizontal direction determined 24 hours after each load increment.
<table>
<thead>
<tr>
<th>Load (H = 0.0980m)</th>
<th>( C_{v_s} )</th>
<th>( C_{v_p} )</th>
<th>( C_{v_s} )</th>
<th>( C_{v_p} )</th>
<th>( C_{v_s} )</th>
<th>( C_{v_p} )</th>
<th>( C_{v_s} )</th>
<th>( C_{v_p} )</th>
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<tbody>
<tr>
<td>0.2</td>
<td>-</td>
<td>1.615</td>
<td>3.04</td>
<td>-</td>
<td>4.84</td>
<td>5.16</td>
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<td>5.16</td>
</tr>
<tr>
<td>0.4</td>
<td>6.52</td>
<td>5.7</td>
<td>4.335</td>
<td>4.41</td>
<td>12.50</td>
<td>11.45</td>
<td>5.16</td>
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<td>0.8</td>
<td>6.225</td>
<td>8.3</td>
<td>1.63</td>
<td>5.565</td>
<td>14.45</td>
<td>12.86</td>
<td>5.30</td>
<td>5.84</td>
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<tr>
<td>1.6</td>
<td>5.53</td>
<td>5.25</td>
<td>2.39</td>
<td>4.81</td>
<td>10.00</td>
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<td>5.44</td>
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<td>2.95</td>
<td>0.995</td>
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<td>4.95</td>
<td>1.677</td>
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<td>6.4</td>
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<td>1.085</td>
<td>1.29</td>
<td>1.72</td>
<td>1.70</td>
<td>1.50</td>
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<td>12.8</td>
<td>1.195</td>
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<td>-</td>
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<td>1.865</td>
<td>1.63</td>
<td>1.605</td>
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</tr>
</tbody>
</table>

**TABLE 3**

**COMPARISON OF \( C_v \) VALUES (LOAD INCREMENT RATIO = 1.0)**

- \( H = 0.0980 \text{m} \)
- \( H = 0.7935 \text{m} \)
- \( H = 1.3670 \text{m} \)
- \( H = 1.640 \text{m} \)

**Legend:**
- \( C_{v_s} \): Vertical coefficient of consolidation in \( 10^{-2} \) in.²/min., computed from dial reading-log time curves using Casagrande's curve fitting method.
- \( C_{v_p} \): Vertical coefficient of consolidation in \( 10^{-2} \) in.²/min., computed from the time required for 50% dissipation of pore pressure at the base of the sample.

**H =** Length of the drainage path at the start of the test.
## TABLE 4

**Comparison of \( C_v \) Values**

(LOAD INCREMENT RATIO = 0.5)

<table>
<thead>
<tr>
<th>LOAD</th>
<th>( PF_5 )</th>
<th>( PF_6 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( H = 1.260&quot; )</td>
<td>( H = 1.890&quot; )</td>
</tr>
<tr>
<td>TSF</td>
<td>( C_{vs} )</td>
<td>( C_{vp} )</td>
</tr>
<tr>
<td>0.27</td>
<td>8.325</td>
<td>9.65</td>
</tr>
<tr>
<td>0.405</td>
<td>7.47</td>
<td>25.7</td>
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<td>0.607</td>
<td>8.25</td>
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<td>0.911</td>
<td>9.0</td>
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<td>1.367</td>
<td>6.25</td>
<td>15.1</td>
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<td>2.051</td>
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<td>3.077</td>
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<td>4.615</td>
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<td>10.383</td>
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<td>15.574</td>
<td>1.235</td>
<td>1.905</td>
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</table>

\( H \) = Length of the drainage path at the start of the test.

\( C_{vs} \) = Coefficient of consolidation in \( 10^{-2} \text{ in.}^2/\text{min.} \), computed from skinreading-log time curves using Oesumdrive's curve fitting method.

\( C_{vp} \) = Coefficient of consolidation in \( 10^{-2} \text{ in.}^2/\text{min.} \), computed from the time required for 90% dissipation of pore pressure at the base of the sample.
### Table 5

**Comparison of Horizontal and Vertical Coefficients of Permeability**

<table>
<thead>
<tr>
<th>LOAD</th>
<th>$K_h \times 10^{-7}$ cm/sec.</th>
<th>$K_v \times 10^{-7}$ cm/sec.</th>
<th>$K_h / K_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TNSF (40% clay &amp; 60% silt)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0</td>
<td>11.95</td>
<td>2.03</td>
<td>6.88</td>
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<td>9.20</td>
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<td>6.4</td>
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<td>12.8</td>
<td>3.81</td>
<td>0.13</td>
<td>29.5</td>
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</table>

### Table 6

**Comparison of Coefficients of Consolidation**

<table>
<thead>
<tr>
<th>Load Increment Ratio</th>
<th>$q_v$ in^2/min.</th>
<th>$C_v$ in^2/min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>TNSF (40% clay &amp; 60% silt)</td>
<td>(1.0)</td>
<td>(0.5)</td>
</tr>
<tr>
<td>Vertical drainage tests (varves horizontal)</td>
<td>0.01-0.0186</td>
<td>0.0083-0.019</td>
</tr>
<tr>
<td>Vertical drainage tests (varves vertical)</td>
<td>0.04-0.046</td>
<td>0.0229-0.0275</td>
</tr>
<tr>
<td>Radial drainage tests</td>
<td>0.14-0.30</td>
<td></td>
</tr>
</tbody>
</table>
GRAIN SIZE DISTRIBUTION CURVES

Clay fraction: silt fraction
- Clay layers: 66% 26%
- Silt layers: 18% 74%
SCHEMATIC OF SPECIAL CONSOLIDOMETER

--- FIGURE 2 ---
SCHEMATIC OF HORIZONTAL PERMEABILITY TEST

--- FIGURE 3 ---
Figure 10.2

Time in Minutes

Dial Reading (1 x 10^-4 lbs)

Laboratory compression curves

Comparison of theoretical and

Initial load = 104 lbs

Test No. 1

4.5 x 10^-4 lbs

Old

New

Experimental
FIGURE 11

- Time in Minutes

% Percent Pure Pressure at Base of Sample

- Theoretical
- Measured

Test no. PP 6.4 - 12.8 TSP

Dissipation

Measured Rates of Pore Pressure Comparison of Theoretical and