Improved determination of preconsolidation pressure of a sensitive clay

Hamilton, J. J.; Crawford, C. B.

Publisher's version / Version de l'éditeur:

ASTM Special Technical Publication, 1960-12-01

NATIONAL RESEARCH COUNCIL
CANADA

DIVISION OF BUILDING RESEARCH

IMPROVED DETERMINATION OF PRECONSOLIDATION PRESSURE OF A SENSITIVE CLAY

BY

J. J. HAMILTON AND C. B. CRAWFORD

REPRINTED FROM

AMERICAN SOCIETY FOR TESTING MATERIALS
SPECIAL TECHNICAL PUBLICATION NO. 254, 1959
P. 254 - 271

RESEARCH PAPER NO. 115

OF THE

DIVISION OF BUILDING RESEARCH

OTTAWA

DECEMBER 1960

PRICE 25 CENTS

NRC 5528

This publication is being distributed by the Division of Building Research of the National Research Council as a contribution towards better building in Canada. It should not be reproduced in whole or in part, without permission of the original publisher. The Division would be glad to be of assistance in obtaining such permission.

Publications of the Division of Building Research may be obtained by mailing the appropriate remittance, (a Bank, Express, or Post Office Money Order or a cheque made payable at par in Ottawa, to the Receiver General of Canada, credit National Research Council) to the National Research Council, Ottawa. Stamps are not acceptable.

A coupon system has been introduced to make payments for publications relatively simple. Coupons are available in denominations of 5, 25, and 50 cents, and may be obtained by making a remittance as indicated above. These coupons may be used for the purchase of all National Research Council publications including specifications of the Canadian Government Specifications Board.

Authorized Reprint from the Copyrighted
Soils—1959 Meetings
Special Technical Publication No. 254
Published by the
American Society for Testing Materials
1959

IMPROVED DETERMINATION OF PRECONSOLIDATION PRESSURE OF A SENSITIVE CLAY

By J. J. Hamilton[1] and C. B. Crawford[2]

Synopsis

Poor agreement between calculated and measured consolidation settlement in the Leda (Laurentian) clay of eastern Canada suggested that either the classical consolidation theory might not apply to this soil or that present testing techniques were not yielding the correct consolidation characteristics of the soil. An underestimation of the preconsolidation load was found to result from tests using normal procedures. An investigation of various techniques to improve the determination of the preconsolidation pressure are outlined in this paper.

Increment loading tests with various increment sizes and increment durations were investigated as well as continuous loading tests under various controlled rates of stress application and various controlled rates of strain. It was found that small increment tests best defined the pressure - void-ratio curve, but required much time. A method of determining the point of 100 per cent primary consolidation on the log time - deflection plot of small increment tests was developed. Constant rate of strain tests show promise as possible rapid testing techniques.

The marine clay of eastern Canada, called Leda clay, covers the broad, highly developed regions of the Ottawa and St. Lawrence River valleys. It is therefore of considerable interest in foundation engineering. One of the notable features of this clay is the occurrence of an unusually sharp change in its compressibility at the preconsolidation load. This suggests a complete failure of the soil skeleton and results in extremely high compression indices.

An extensive investigation of the settlement of the National Museum Building in Ottawa by Crawford (**1**)[3] showed an unsatisfactory correlation between computed and observed settlements. It was suggested that the standard consolidation test may yield incorrect values for preconsolidation pressure and compression index or that the standard consolidation theory may not apply at all to this particular soil. A subsequent "field check" of the settlement of a large fill on a deep deposit of the marine clay confirmed the possible error in determining the preconsolidation pressure by the standard laboratory test.

In view of the serious discrepancies between the theoretical evaluation and field performance of Leda clay, a detailed laboratory investigation of its compres-

[1] Research Officer, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Ont. (Canada).

[2] Head, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Ont. (Canada).

[3] The boldface numbers in parentheses refer to the list of references appended to this paper.

sion properties was undertaken by the Division of Building Research of the National Research Council.

This paper discusses the factors that are known to affect test results and describes the laboratory investigation.

Factors That Influence Test Results

Sample Disturbance:

In nature, clays usually exist in equilibrium under unequal principal stresses. The stress reduction and changes in principal stress ratio that occur due to soil sampling and specimen preparation are probably responsible for most of the disagreement between laboratory results and field observations.

Based on laboratory observations of compression, rebound, and recompression characteristics of soil specimens, Casagrande (2) developed a graphical method for adjusting laboratory pressure - void-ratio curves to field conditions. Terzaghi (3) questions the merits of this procedure and cites cases of serious disagreement. It is defended by Casagrande and Fadum (4) as the only means of estimating the preconsolidation history of a natural clay deposit. Rutledge (5) quotes evidence in support of the procedure, and Schmertmann (6) has further developed the interpretation originated by Casagrande. It is now a generally accepted treatment, although there is no doubt that large discrepancies may occur with certain types of soil especially if sampled at great depths.

The implications of stress changes due to sampling are discussed at length by Rutledge (7) and Terzaghi (8) in an attempt to visualize the pressure - void-ratio curve in nature and its relationship to the laboratory curve. Rutledge (7) concluded that sample disturbance tended to obscure the stress history and preconsolidation pressure of a soil and decreased the equilibrium void-ratio under any particular stress. Van Zelst (9) reported that most disturbance was due to the remolding caused by trimming the flat faces of test specimens.

Increment Ratio and Duration:

From a considerable amount of research at the Massachusetts Institute of Technology, Taylor (10) concluded that consolidation proceeds more slowly for small load increments than for large and that predictions of rate of settlement may be seriously in error unless load increments in the test are essentially the same as in the field. Irregular increment durations were found to introduce irregularities in the test result. Most authorities agree that this should be standardized for any particular soil. Small load increments have been suggested for soft soils by Burmister (11). The nature of load increments appears to have less effect on the amount of settlement. Tests on adjacent specimens of Boston Blue Clay reported by Casagrande and Fadum (4) showed almost identical pressure - void-ratio curves, although one was consolidated in seven load increments in one week while the second was subjected to nearly 50 increments during an 85-day period.

The variation between rate of loading of an element of soil during deposition and during testing in the laboratory is invariably great. The fact that deep, rather uniform deposits of clay are found to exist in nature at almost constant void-ratio has led to the deduction that the difference between geological and laboratory loading rates has a great influence on the compression index of a soil.

Terzaghi (12) postulated a "rigid bond" between the adsorbed water films surrounding the particles of undisturbed clay which is unbroken by slow loading during sedimentation. Stresses are therefore able to increase along a flat pressure - void-ratio curve. Disturbance and excessive stresses during a laboratory test however are thought to destroy the "rigid bond" and transform the clay into a "lubricated" state. In this condition the void-ratio change under pressure increments is much greater than in nature and the pressure - void-ratio curve may be very steep.

Schmertmann (6) on the other hand concludes from a detailed study of a normally consolidated organic silty-clay plus some supplementary evidence that "... many natural clays consolidate in the field along virgin slopes similar, if not identical, to those obtained from a completely undisturbed laboratory consolidation test." From a study of water content decrease with depth in some borings in marine clay and from consolidation tests on undisturbed and artificially sedimented specimens of the clay, Casagrande and Fadum (4) drew similar conclusions.

Two opposite interpretations have been made on the basis of field evidence; either interpretation may be correct depending on the nature of the soil. In view of the wide possibilities of interpretation between geological and laboratory test properties, the field consolidation of some natural soil deposits is an open question.

Gas Content:

Taylor (10), Hvorslev (13), and Terzaghi (14) have suggested that the initial, almost instantaneous, compression which occurs under load increments may be largely due to gas bubbles in the specimen. These gas bubbles may be found in organic soils or in soils having pore water which has percolated through organic soils.

Presumably, gases such as hydrogen sulfide, derived from the decomposition of organic matter, and dissolved air which has been carried down into the pore water by percolation or seepage, will remain in solution until a change in pressure or temperature alters the state of equilibrium. Gases will continue to be generated in organic soils until the pore water is saturated under the existing conditions of temperature and pressure. The saturation weight of gas dissolved in the pore water is directly proportional to the pressure (Henry's law) and inversely proportional to temperature of the pore water.

Temperature and Storage:

The temperature at which laboratory tests are made usually differs appreciably from temperatures in the ground. Studies of the effect of this difference by Gray (15) and by Finn (16) show temperature to have an influence on both rate and amount of compression under load.

Undoubtedly, in some soils, the increase in temperature after sampling together with the reduction in stresses results in a slight volume increase due to expansion of gas bubbles and a release of gas from solution. Hvorslev (13) considers this problem in some detail. He discusses sampling and storage techniques to minimize the effect and suggests methods of correcting test results. Dawson (17) has considered the special case of storing and testing expansive soils.

Apparatus:

The apparatus and testing techniques are factors which influence test results. Taylor (10) investigated the effect of friction between the consolidation ring and the specimen and found some effect on pressure - void-ratio curves but little effect on rate of compression. Apparatus compressibility and seating of porous stones on the soil specimen are factors found to be worth taking into account.

Laboratory Studies

Soil Material:

The tests reported in this paper were made on specimens trimmed from an undisturbed block sample of silty-clay of marine origin. The sample was carefully cut from a trench wall at a depth of 23 ft. It is composed of more than one half clay-size particles, has a liquid limit of 48 per cent, a plastic limit of 25 per cent, and a water content range from 53 to 62 per cent. The pore water salt content is about 0.5 g per liter. The undisturbed shear strength by field vane, laboratory vane, and by unconfined compression is about 0.7 kg per sq cm, and remolded it is about 0.05 kg per sq cm. The sensitivity is therefore about 14. A complete series of triaxial compression tests on specimens from a block sample obtained just above this sample are reported separately by Crawford (18).

Sample Preparation:

Past experience with the extremely sensitive Leda clay has shown that the best consolidation samples can be obtained by a method involving partial pretrimming followed by cutting of the sample with a sharp-edged consolidation ring. The pretrimming is done by hand using a wire saw and knife slightly ahead of the advance of the consolidation ring. The ring is held by a special device while it is pushed into the pretrimmed specimen. Visual observation and comparison of test results with samples trimmed by other techniques have proved the superiority of this method. The two end surfaces of the sample are cut to the same height as the consolidation ring by a fine wire saw with a minimum of planing with a steel straight-edge to assure a horizontal surface. The porous stones are then carefully aligned on the sample surface, positioned with a slight thumb pressure, and the enclosed specimen carefully installed in the consolidation chamber.

Loading is carried out through a loading cap and ball bearing arrangement allowing complete freedom of rotation of the porous stones. The first increment of pressure, usually 0.2 to 0.4 kg per sq cm is applied to the sample before the chamber is filled with water, thus eliminating any tendency of the sample to swell. Normally consolidated or lightly overconsolidated Leda clay at its natural water content shows little or no tendency to swell when soaked in the consolidometer under these small loads. To assure greatest similarity in pretest conditions and in the quality of samples, this preparation procedure was followed in every test by the same person.

Types of Tests:

Incremental Loading.—In normal routine consolidation testing an increment ratio of one is used, that is,

$$\frac{\Delta p}{p} = 1,$$

where Δp is the load increment, and p is the previous total load. A typical load schedule of this type might be 0.2, 0.4, 0.8, 1.6, 3.2, 6.4, 12.8 kg per sq cm. In addition to these normal tests, several tests were made using increment ratios of $\frac{1}{2}$, $\frac{1}{3}$, and $\frac{1}{10}$.

Constant Rate of Stress.—Several tests were made to study load-deflection characteristics using a uniform constant rate of stress increase. Loading rates ranging from nearly 3 kg per sq cm per hr to 0.1 kg per sq cm per hr were used.

Constant Rate of Strain.—Although requiring more elaborate apparatus, a compression test at a constant rate of strain is more appropriate than a test at constant rate of stress increase because at a controlled rate of strain the recompression load can be applied quickly while the virgin compression load is applied more slowly, allowing time for strain; above the preconsolidation pressure the rate of strain is essentially constant and equal to the applied rate.

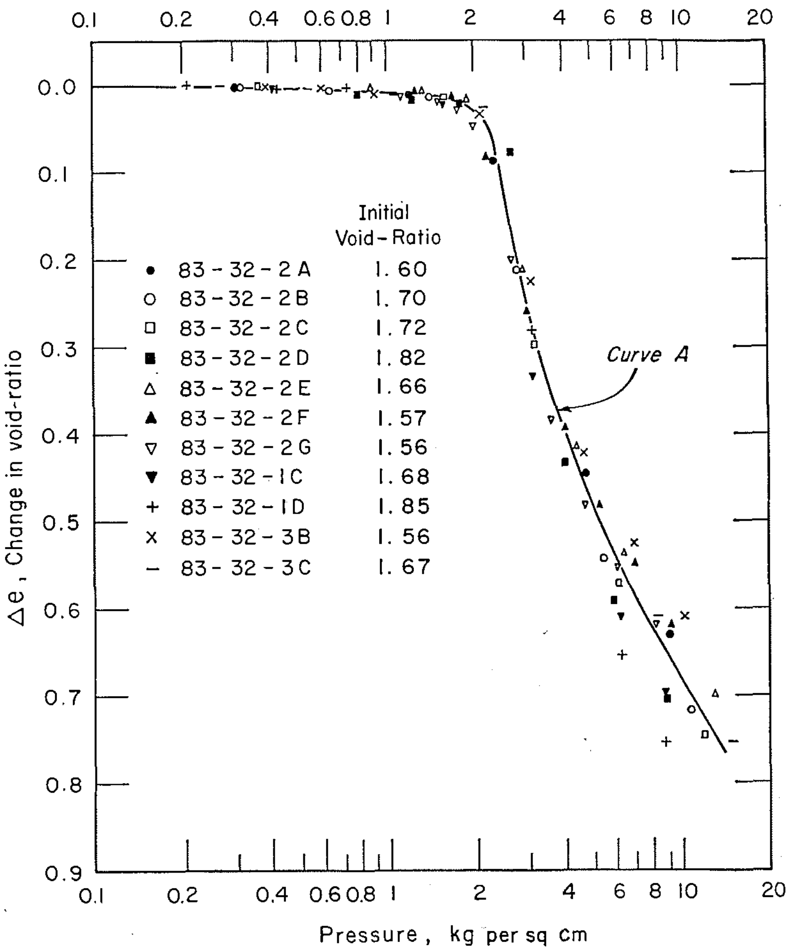


FIG. 1.—Change in Void-Ratio - Log Pressure Relationship for All Incremental Loading Tests.

drainage of pore water. Several constant rate of strain tests were made at rates of virgin compression ranging from 0.3 per cent per hr to 9.0 per cent per hr.

In these tests there are really two rates of strain application: during recompression the proving ring deflects considerably and the strain in the sample is therefore only a fraction of the applied

Test Apparatus:

In all tests, the same type of consolidation ring and porous stones were used. The consolidation ring is of high quality stainless steel, especially chosen to resist corrosion by the soil and water of Leda clay samples. It is 2 cm high with a cross-sectional area of 20.0 sq cm. The ring is patterned after one developed by the

Norwegian Geotechnical Inst. and features a sharpened cutting edge to facilitate sample preparation. For this investigation the ring was used as a floating ring with the two porous stones at the top and the bottom having a radial clearance of 0.003 in. to allow free movement but a minimum of sample loss around the sides. The porous stones were $\frac{1}{8}$-in. thick porous Alundum[4] disks of medium hardness and grain size 150, with tapered edges to minimize the possibility of binding.

The incremental loading of samples was carried out in several lever type consolidation frames. The constant rate of strain tests were made in direct-drive variable-speed triaxial compression presses. Tests at constant rate of stress increase were run either by manually adjusting hydraulic bellows-type consolidometer or by loading with a constant rate of water flow into a tank hung on the loading arm of a lever-type consolidometer.

TEST RESULTS

Incremental Loading:

The relation between change in void ratio, that is, change in void ratio from beginning of test, and pressure for 11 increment loading tests is shown in Fig. 1. Load increment ratios were 1, $\frac{1}{2}$, and $\frac{1}{3}$, with all increments being terminated after the rate of compression became less than 0.0006 in. per hr.

This plot, although somewhat unusual since change in void ratio rather than actual void ratio has been plotted, was found to provide the best means of comparing test results. The study of the eleven individual pressure - void-ratio curves, using the Casagrande graphical solution (2), resulted in a variation of most probable preconsolidation pressure from 1.97 kg per sq cm to 2.78 kg per sq cm. By plotting the change in void ratio under each load increment for all tests and drawing an average curve through the points (curve A, Fig. 1) a preconsolidation pressure of 2.40 kg per sq cm is obtained. From other tests it has been found that the range of indicated preconsolidation values due to interpretation becomes larger as the preconsolidation pressure increases, due to the nature of the log plot. The initial water content of the samples tested varied from 62.2 per cent to 52.8 per cent with the average being 57.0 per cent. In spite of the rather large variation in the initial water contents of these samples, it was found that the curve from the small increment tests could be fitted fairly closely to the curves for large increment tests. Exceptions to this may be described as follows:

(*a*) For specimens at higher than average initial water content the laboratory recompression curve was steeper and the compression index was higher than average.

(*b*) For specimens at lower than average initial water content the laboratory recompression curve was flatter and the compression index was lower than average.

A comparison of all the pressure - void-ratio curves from the increment tests showed that the size of increment has practically no effect on the shape of the curve. It appears that there is a unique relationship between pressure and void ratio and that points on the curve representing this relationship for any particular soil can be determined by a variety of load increment sizes.

The averaged curve shown in Fig. 1 agrees very favorably with the pressure - void-ratio curve from the small increment test $\left(\frac{\Delta p}{p} = \frac{1}{10}\right)$ as can be seen in Fig. 2.

[4] Alundum is fused alumina made by melting the mineral bauxite in an electric furnace.

The importance of establishing the point of 100 per cent primary consolidation was demonstrated in this investigation. The void-ratio calculated to represent 100 per cent primary consolidation varied considerably depending on the technique on the log of time-deflection plot has proved satisfactory for Leda clay test results.

Figure 3 shows curves of square root of time-deflection and log time-deflection for several load increments in each

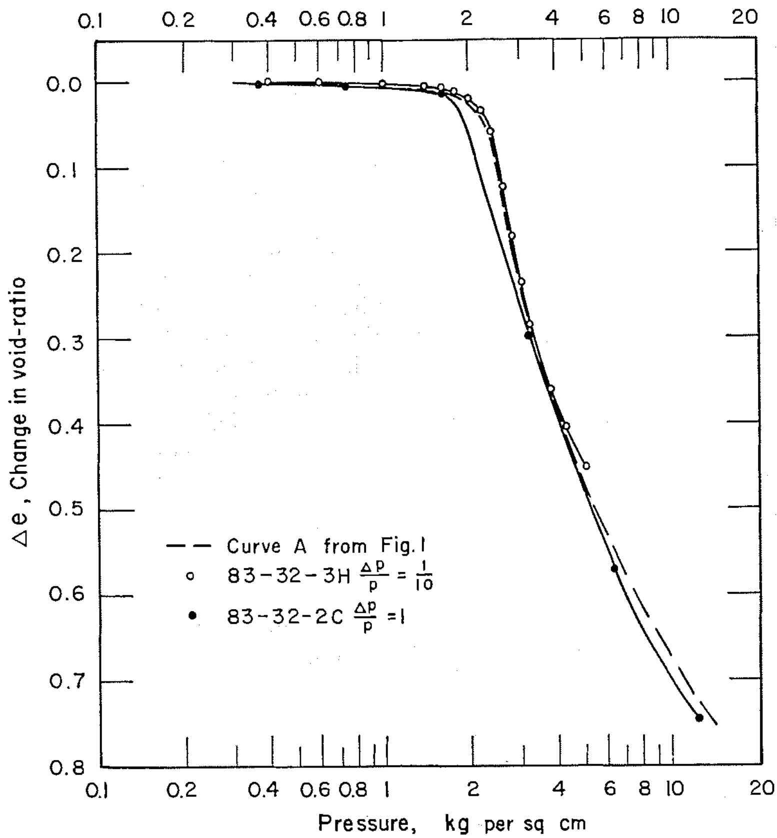


FIG. 2.—The Shapes of Pressure - Void-Ratio Curves as Obtained by Large Increment and by Small Increment Testing.

method chosen to establish this point on the time-deflection curves. In small increment tests and in large increment tests near the preconsolidation pressure, structural breakdown rather than permeability seems to be the important factor controlling the rate of compression. In these cases neither Taylor's fitting technique on the square root of the time-deflection plot nor Casagrande's fitting

of three typical tests having load increment ratios of 1, $\frac{1}{3}$, and $\frac{1}{10}$. Taylor's square root of time-fitting technique has been carried out on each increment where possible. (Only a single square root of time scale has been plotted, although for more accurate interpretation of results other scales were used.) It was found impossible to apply Taylor's technique to the results from sample 83-32-

3H. The application of the method to some of the other increments may also be seriously questioned because a straight-line portion was difficult to interpret.

flection at the end of primary consolidation was observed for load increments just beyond the preconsolidation load although an arithmetic plot of the same data would indicate a substantial reduc-

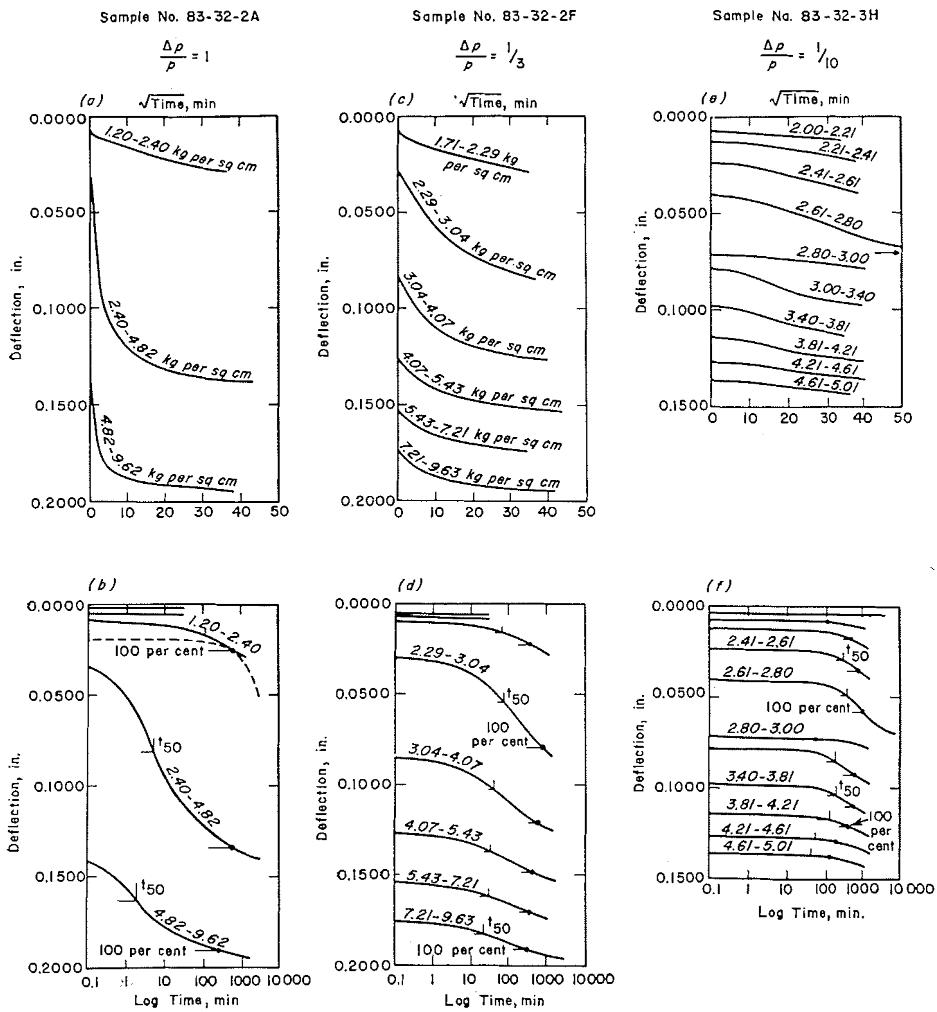


FIG. 3.—Curves of Square Root of Time-Deflection and Log-Time-Deflection for Various Load Increments.

It was necessary to develop a modification of Casagrande's fitting method to establish the point of 100 per cent primary consolidation on the time-deflection plot. Because of the nature of the semilogarithmic plot, no definite point of in-

tion in the rate of compression with time. By examining many arithmetic plots of the time-deflection relationship and by calculating the rates of compression at 100 per cent primary consolidation on those increments that showed a point of

TABLE I.—COEFFICIENTS CALCULATED BY TAYLOR'S SQUARE ROOT OF TIME METHOD AND BY THE MODIFIED CASAGRANDE METHOD.

Test	Increment, kg per sq cm	C_v, sq cm per sec		A_v, sq cm per g		K, cm per sec	
		$\sqrt{T}$ Method	Modified Log Time Method	$\sqrt{T}$ Method	Modified Log Time Method	$\sqrt{T}$ Method	Modified Log Time Method
83-32-2A	1.20 to 2.40	1.2×10^{-5}	2.8×10^{-4}	6.7×10^{-5}	4.5×10^{-5}	3.15×10^{-10}	4.9×10^{-9}
$\frac{\Delta p}{p} = 1$	2.40 to 4.82	9.69×10^{-3}	5.16×10^{-3}	1.08×10^{-4}	1.41×10^{-4}	4.35×10^{-7}	3.10×10^{-7}
	4.82 to 9.62	2.51×10^{-2}	1.09×10^{-2}	5.11×10^{-5}	3.72×10^{-5}	5.96×10^{-7}	1.96×10^{-7}
83-32-2F	1.72 to 2.29	4.48×10^{-5}	4.66×10^{-4}	7.07×10^{-5}	5.0×10^{-5}	1.25×10^{-9}	9.25×10^{-9}
$\frac{\Delta p}{p} = 1/3$	2.29 to 3.04	5.56×10^{-4}	3.85×10^{-4}	1.72×10^{-4}	2.13×10^{-4}	3.96×10^{-8}	3.45×10^{-8}
	3.04 to 4.07	2.11×10^{-3}	6.23×10^{-4}	7.4×10^{-5}	1.15×10^{-4}	6.91×10^{-8}	3.21×10^{-8}
	4.07 to 5.43	6.06×10^{-3}	6.92×10^{-4}	2.28×10^{-6}	4.41×10^{-5}	6.42×10^{-9}	1.43×10^{-8}
	5.43 to 7.21	2.20×10^{-2}	7.20×10^{-4}	8.43×10^{-6}	2.98×10^{-5}	8.98×10^{-8}	1.05×10^{-8}
	7.21 to 9.63	4.55×10^{-3}	9.32×10^{-4}	1.24×10^{-6}	1.94×10^{-5}	2.83×10^{-9}	9.12×10^{-9}
83-32-3H	2.21 to 2.41		1.45×10^{-4}		8.57×10^{-6}		4.93×10^{-9}
$\frac{\Delta p}{p} = 1/10$	2.41 to 2.61		9.87×10^{-5}		2.00×10^{-4}		7.97×10^{-9}
	2.61 to 2.80		7.35×10^{-5}		3.11×10^{-4}		9.35×10^{-9}
	3.00 to 3.40		1.46×10^{-4}		1.15×10^{-4}		7.31×10^{-9}
	3.40 to 3.81		1.35×10^{-4}		1.00×10^{-4}		6.04×10^{-9}
	3.81 to 4.21		2.21×10^{-4}		5.75×10^{-5}		5.80×10^{-9}
	4.21 to 4.61		5.16×10^{-4}		2.25×10^{-5}		5.37×10^{-9}
	4.61 to 5.01		5.96×10^{-4}		1.50×10^{-5}		4.20×10^{-9}

inflection on the log time-deflection plot (thus allowing the use of Casagrande's technique) a rate of 0.0008 in. per hr per in. height of specimen was found to represent the end of primary consolidation. For a specimen 2 cm high, a rate of 0.0006 in. per hr was therefore chosen as the end of primary consolidation. This method proved to have three distinct advantages:

1. It clearly defined the point of 100 per cent primary consolidation in all increments regardless of increment size or position with respect to the preconsolidation pressure.

Once the rate representing the end of primary consolidation had been established (that is, 0.0006 in. per hr) a curve was plotted on a sheet of the same transparent semi-logarithmic paper used to plot the log time - deflection curve, so that a tangent plotted at any point on the curve would have a slope equal to 0.0006 in. per hr. The procedure in determining 100 per cent primary consolidation was then simply to place the log time - deflection plot for any increment over this constant-rate curve and to determine the point of tangency of the two curves (see dashed curve for increment 1.20 kg per sq cm to 2.40 kg per sq cm in test No. 83-32-2A in Fig. 3).

2. It allowed immediate application of the next increment of load as soon as the point of 100 per cent primary consolidation had been reached and thus saved the considerable time that would normally be required to establish the straight-line portion of the secondary compression branch on the log time - deflection plot.

3. It allowed a simple standard test procedure.

Using the above modification to Casagrande's technique, values for void ratio at 100 per cent primary consolidation were calculated. In all the increments shown in Fig. 3, however, the duration of each increment was approximately 24 hr in order to compare this technique with the commonly used 24-hr increment.

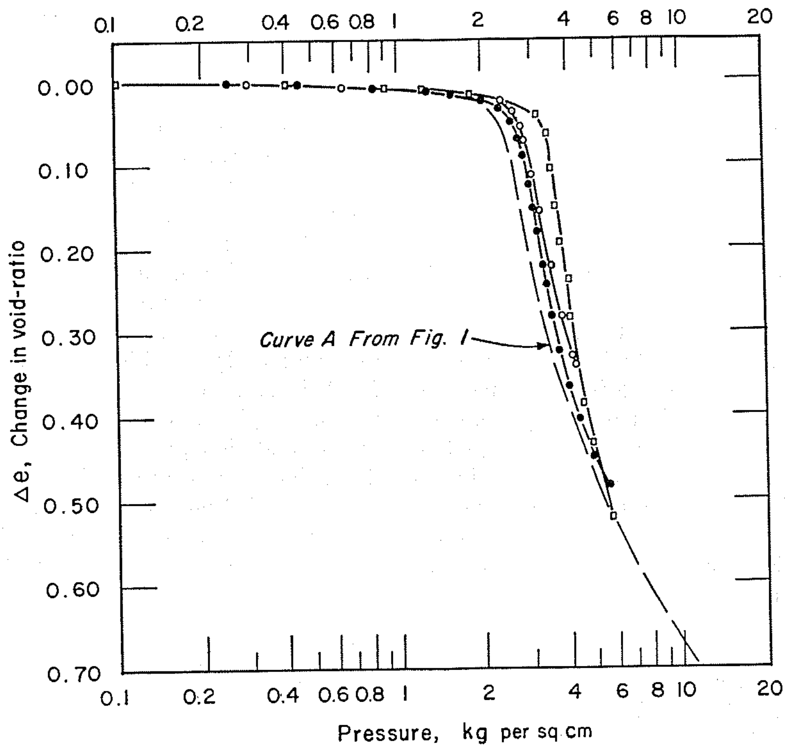
Values of the coefficient of consolidation, coefficient of compressibility, and the coefficient of permeability were calculated for each increment by Taylor's method and by the modified Casagrande method (Table I).

Tests on Constant Rate of Stress:

To investigate possibilities for the "rapid" determination of the preconsolidation pressure, a series of continuous loading tests was made using a constant-rate-of-stress-increase type of loading with rates of loading ranging from 2.7 kg per sq cm per hr to 0.14 kg per sq cm per hr. The most rapid test lasted approximately $2\frac{1}{2}$ hr while the slowest ran for 2 days. To be of value, this type of test must give a reliable value for the preconsolidation pressure in less than 3 days, since a conventional increment test can be carried out in this time.

Tests run at a constant rate of stress application throughout their loading schedule do not take advantage of the almost elastic time-deflection relationship at loads below the preconsolidation pressure, and unless the loading equipment is capable of varying the rate of loading during the test, the procedure is not particularly efficient.

It was found in the constant-rate-of-stress tests that as the rate of loading decreased the break in the pressure - void-ratio curve took place at lower values of pressure than before. At the lowest rate of loading the preconsolidation pressure determined from the log pressure - void-ratio curve was 2.91 kg per sq cm. This value is considerably higher than that indicated by any of the increment tests. It was thought that although the rate of loading was very slow, excess pore pressure must have been built up. At pressures greater than preconsolidation pressure the applied load did not accurately represent effective stresses. Since pore-pressure measurements were not made, the true effective stresses could not be calculated, and for the time being at least the technique has been abandoned.



Sample	Rate of Strain, in. per hr	Rate of Strain, per cent per hr	P_n, kg per sq cm
● 83-32-4C	0.0024	0.3	2.7
○ 83-32-1E	0.0360	4.5	2.7
□ 83-32-1F	0.0720	9.0	3.4

FIG. 4.—Effects of Loading Rates on Pressure - Void-Ratio Curves.

Constant Rate of Strain Tests:

The investigation of "rapid" test methods included several tests run under controlled rates of strain. The advantage of a constant-rate-of-strain test lies in the fact that little time is required to reach the preconsolidation load, and the rate of strain during virgin compression can be selected. Tests were run at rates varying from 0.072 in. per hr to 0.0024 in. per hr. Figure 4 shows the relationship between change in void-ratio and pressure for three tests run at various rates. The test run at the highest strain rate gave the highest "indicated" preconsolidation pressure, and it is believed that some excessive pore pressure existed in this test specimen. In spite of great differences in rates, the two curves for the lower rates of strain give comparable values for the preconsolidation pressure of 2.7 kg per sq cm. The slight difference in their shapes could be caused by differences in initial water content and grain size. Even at the slowest rate of testing, the indicated preconsolidation pressure was definitely higher than the highest value obtained by incremental loading. For comparison the "average" curve from the incremental tests (curve A, Fig. 1) is also plotted on Fig. 4.

ment tests. Neither color differences nor differential shrinkage were observed in the tests at controlled rate of strain or stress.

It is thought that the observed discontinuity in some specimens is due to differential shrinkage during oven drying and

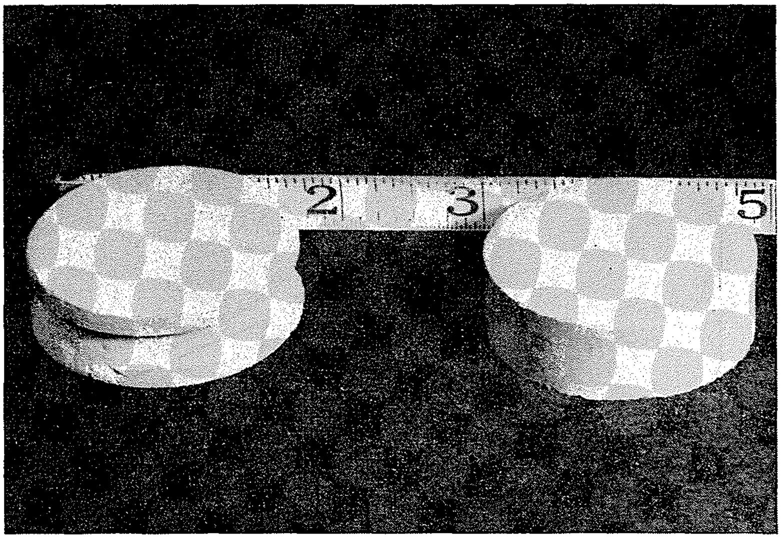


FIG. 5.—Differential Shrinkage of Oven-Dried Sample.

Specimen on left subjected to incremental loading $\left(\frac{\Delta p}{p} = 1\right)$, specimen on right tested by a continuous load increase.

DISCUSSION OF RESULTS

Stresses in the Test Specimen:

During this study it was noticed that the shape and appearance of the oven-dried test specimens depended on the method of test. Specimens that had been subjected to incremental loading, when oven dried, revealed a dark ring of soil at mid-height around the perimeter. This ring of soil appeared to have shrunk more than the rest of the specimen after oven drying and in many cases it fell away leaving a spool-shaped specimen as shown in Fig. 5. This phenomenon was observed more clearly in the larger increment tests than in the small increment tests.

that this resulted from unequal consolidation in the specimen. It is presumed that the unequal consolidation was due to unequal stress distribution in the specimen during testing and that this resulted from the boundary conditions introduced by the consolidation ring.

As a possible explanation of the above phenomenon the following stages of movement are visualized after each load increment. First the entire load increment is carried by the pore water. An instant later the pore water pressure decreases substantially at the drainage faces and effective stresses at the faces are increased suddenly. This results in an increased effective stress normal to the consolidation ring near to the drainage faces. No change has yet occurred at the mid-plane. The increasing effective stress introduces an increment of friction which will transfer some of the normal load from the specimen to the consolidation ring. It is this manner of increasing friction that is thought to cause unequal internal stress distribution and resulting unequal consolidation in the specimen. The effect is cumulative throughout the loading. Since the base plate and loading cap are relatively rigid a certain amount of lateral adjustment within the sample must take place.

In tests at constant rate of stress or strain the effective stresses within the specimen increase uniformly with applied stresses (pore pressures assumed to be zero). While friction will still exist during continuous loading the discontinuity in effective stresses due to pore pressure dissipation will not occur and a major cause of unequal consolidation is removed.

Gas in the Test Specimen:

Examination of many consolidation tests on Leda clay led to the belief that gas bubbles were playing an important role in the shape of the recompression curve. An examination of the square root of time *versus* compression and log of time *versus* compression curves for these tests revealed a high "initial compression." The loading apparatus and porous stones used in the standard floating-ring consolidation test were calibrated for compressibility, and it was found that, although significant, this correction could only partially explain the initial compression. The possibility of soil particles being squeezed into the porous stones was next investigated to find out whether this action might result in a significant "seating correction." The use of fine-grained porous disks resulted in a "smooth" contact between sample and stone, and an investigation of the dry weight of the porous stones before and after consolidation tests revealed no sig-

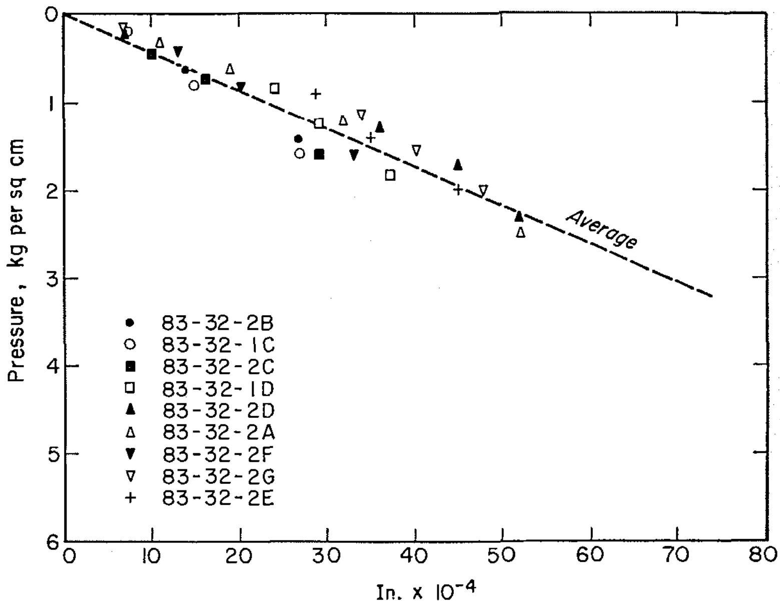


FIG. 6.—"Initial" Gas Compression *versus* Pressure at Pressures Below the Preconsolidation Pressure

nificant loss of soil solids to the porous stones. It was concluded that the compression of gas bubbles must be of considerable importance, making up as much as 80 per cent of the "initial" compression in the recompression load increments.

An investigation of all the incremental type tests on the block sample was carried out in an attempt to evaluate the amount and significance of the compression of gas bubbles. The cumulative initial compression for any load increment was calculated by summing the amount of compression taking place in the first $\frac{1}{10}$ min of load application for each load increment up to and including the applied pressure in question. A curve expressing the total correction for compressibility of apparatus was found by calibration. Figure 6 is a plot of the difference between the cumulative initial compression values and the cumulative apparatus compressibility and is therefore a presentation of the cumulative "initial gas compression" *versus* applied pressure. At pressures above the preconsolidation pressure, the amount of initial compression was about equal to the difference between the zero point (computed by assuming a parabolic curve) and the $\frac{1}{10}$ min reading.

The slope of the average curve through the data on Fig. 6 is expressed by the equation:

$$c_G = 0.0023P$$

where:

c_G = total initial gas compression, in., and

P = applied pressure, kg per sq cm.

The average sample height during these tests was 0.790 in. Therefore in terms of unit values:

$$C_G = 0.0029P$$

where:

C_G = unit initial gas compression due to unidirectional compression, and

P = applied pressure, kg per sq cm.

This formula, when used to calculate "initial" compression of a 100-ft-deep deposit of Leda clay subjected to recompression by the construction of a 20-ft-high embankment gave encouragingly close agreement with the measured compression. The calculated compression was $2\frac{1}{2}$ in. and a settlement of 2 in. was measured by accurate field instrumentation.

Effects of Leaching on Compression Indices:

To determine whether the geological pressure - void-ratio relationship was the same as that found for virgin compression in the consolidometer, the relationship between effective stress and void ratio was calculated for several natural deposits of Leda clay in the Ottawa area. It was found that there was a great difference in the compression indices obtained by these two methods; the laboratory indices were many times higher than those implied by field conditions. For this deposit of clay these observations tend to confirm Terzaghi's hypothesis (12) and refute Schmertmann's theory (6) on the relationship between geological and laboratory virgin compression, and the following explanation is offered.

Geological opinion is that Leda clay was deposited in brackish water. At depth some deposits of the clay in the Ottawa area yield pore-water salt concentrations as high as 15 g per liter while the upper layers contain less than 1 g of dissolved salt per liter of pore water. This indicates that the clay has undergone considerable leaching subsequent to deposition. Skempton and Northey (19) have shown that the action of leaching is that of developing a meta-stable structure which is extremely sensitive to rearrangement of particle orientation caused by stress charges or remolding.

It is suggested that under loads causing virgin compression of a clay which has been leached subsequent to deposition, significant changes take place in the microstructure from the flocculated to a more dispersed state as visualized by Lambe (20). This collapse of structure will result in large changes in void ratio under relatively small increases in load. The result of leaching following geologic consolidation is, therefore, to give very high virgin compression indices at loads slightly greater than the maximum geological loading.

Field void ratios are the result of consolidation under geological rates of sedimentation at the depositional pore-water salt concentration. Laboratory virgin compression void ratios are the result of consolidation of the leached clay under very much higher rates of compression. The consolidation indices determined in the laboratory may or may not be representative of the *in situ* characteristics, depending on how closely laboratory rates agree with field stress conditions. Incremental loading of even very small increment ratios may have serious effects on their meta-stable structure with the high pore pressures created by loss of strength of the structure contributing to the rearrangement of particle orientation.

Field Tests

A study of laboratory methods of determining the preconsolidation pressure of a natural clay is incomplete without some full-scale field investigations to provide absolute values on which to evaluate the laboratory results. One high earth embankment on a deep deposit of Leda clay has already yielded valuable information regarding the preconsolidation pressure and the reconsolidation characteristics of this clay. Briefly this investigation has shown that previous laboratory techniques and interpretation of laboratory data have tended to result in:

(*a*) An underestimation of the preconsolidation pressure owing to difficulties involved in determining 100 per cent primary consolidation at loads near the preconsolidation pressure and to errors in interpreting the pressure - void-ratio curve when large increments of load are involved.

(*b*) An overestimation of reconsolidation settlements owing to test errors resulting from compressibility of apparatus and compression of gas bubbles in the specimen.

The construction and instrumentation of another high embankment fill which will undergo considerable virgin compression is planned. This will provide another field check with which to evaluate laboratory testing techniques used to determine the preconsolidation pressure and virgin consolidation indices.

Conclusions

1. Experience with a sensitive marine clay shows the need for extreme care in sampling, handling, and trimming test specimens to obtain good test results for the preconsolidation pressure of the undisturbed clay.

2. The size of load increments in incremental loading tests does not affect the pressure - void-ratio curve, but the interpretation of most probable preconsolidation pressure becomes more difficult as load increments become larger. In general when load increments are large, estimates of the most probable preconsolidation pressure are too low.

3. Initial water content of specimens from the same block sample affects the slope of the recompression and virgin compression curves.

4. A systematic method of determining the end of primary consolidation under a load increment is essential and such a method has been developed.

5. Rapid methods for obtaining a satisfactory value for preconsolidation pressure of this clay were investigated and found to show promise. The best rapid method seems to be a controlled-strain type of test.

6. There are indications of more serious stress variations through a test specimen during incremental loading than during continuous loading and this may result in an underestimation of preconsolidation pressure. Further work must be done to verify this.

7. Test results indicated that gas bubbles in test specimens can account for most of the initial compression observed at loads below the preconsolidation load.

8. There are indications that the leaching of salt from the pore water after deposition has led to the development of a meta-stable structure in the clay and that this change prevents the determination of the geological pressure - void-ratio curve by laboratory testing and is responsible for the extremely high compression indices found for virgin compression.

Acknowledgments:

The authors are indebted to their colleagues in the Soil Mechanics Section, Division of Building Research, for their assistance in obtaining samples and in the laboratory testing associated with the study. They wish to express their appreciation to B. J. Bordeleau for his assistance in the laboratory testing program. Grateful acknowledgment is also made to R. F. Legget, the Director of the Division, for continued encouragement in this study and with whose approval this paper is published as a contribution from the Division of Building Research, National Research Council of Canada.

REFERENCES

(1) C. B. Crawford, "Settlement Studies on the National Museum Building, Ottawa, Canada," *Proceedings*, Third International Conference on Soil Mechanics, Zurich (Switzerland), Vol. 1, pp. 338–345 (1953).

(2) A. Casagrande, "The Determination of the Preconsolidation Load and its Practical Significance," *Proceedings*, First International Conference on Soil Mechanics, Cambridge, Mass., Vol. 3, pp. 60–64 (1936).

(3) K. Terzaghi, discussion of A. Casagrande and R. E. Fadum, "Application of Soil Mechanics in Designing Building Foundations," *Transactions* Am. Soc. Civil Engrs., Vol. 109, pp. 427–435 (1944).

(4) A. Casagrande and R. E. Fadum, "Application of Soil Mechanics in Designing Building Foundations," author's closure, *Transactions*, Am. Soc. Civil Engrs., Vol. 109, pp. 463–490 (1944).

(5) P. C. Rutledge, "Relation of Undisturbed Sampling to Laboratory Testing," author's closure, *Transactions*, Am. Soc. Civil Engrs., Vol. 109, pp. 1208–1216 (1944).

(6) J. H. Schmertmann, "The Undisturbed Consolidation Behavior of Clay," *Transactions*, Am. Soc. Civil Engrs., Vol. 120, pp. 1201–1233 (1955).

(7) P. C. Rutledge, "Relation of Undisturbed Sampling to Laboratory Testing," *Transactions*, Am. Soc. Civil Engrs., Vol. 109, pp. 1155–1183 (1944).

(8) K. Terzaghi, discussion of P. C. Rutledge, "Relation of Undisturbed Sampling to Laboratory Testing," *Transactions*, Am. Soc. Civil Engrs., Vol. 109, pp. 1197–1201 (1944).

(9) T. W. Van Zelst, "An Investigation of the Factors Affecting Laboratory Consolidation of Clay," *Proceedings*, Second International Conference on Soil Mechanics, Rotterdam (The Netherlands), Vol. 7, pp. 52–61 (1948).

(10) D. W. Taylor, "Research on Consolidation of Clays," Mass. Inst. Technology, Serial 82 (1942).

(11) D. M. Burmister, "Suggested Methods of Test for Consolidation of Soils," Procedures for Testing Soils, Am. Soc. Testing Mats., Compiled by ASTM Committee D-18, pp. 287–295 (1958).

(12) K. Terzaghi, "Undisturbed Clay Samples and Undisturbed Clays," *Journal*, Boston Soc. Civil Engrs., Vol. 28, No. 3 (1944).

(13) M. J. Hvorslev, "Sub-surface Exploration and Sampling of Soils for Civil Engineering Purposes," Waterways Experiment Station, Vicksburg, Miss. (1949).

(14) K. Terzaghi, "Theoretical Soil Mechanics," John Wiley and Sons, Inc., New York, N. Y. (1942).
(15) H. Gray, "Progress Report on Research on the Consolidation on Fine-Grained Soils," *Proceedings*, First International Conference on Soil Mechanics, Cambridge, Mass., Vol. 2, pp. 138–141 (1936).
(16) F. N. Finn, "The Effect of Temperature on the Consolidation Characteristics of Remolded Clay," Symposium on Consolidation Testing of Soils, Am. Soc. Testing Mats., pp. 65–71 (1951). (Issued as separate publication *ASTM STP No. 126*.)
(17) R. F. Dawson, discussion of P. C. Rutledge, "Relation of Undisturbed Sampling to Laboratory Testing," *Transactions*, Am. Soc. Civil Engrs., Vol. 109, pp. 1190–1193 (1944).
(18) C. B. Crawford, "The Influence of Rate of Strain on Effective Stresses in Sensitive Clay," see p. 36, this publication.
(19) A. W. Skempton and R. D. Northey, "The Sensitivity of Clays," *Géotechnique*, Vol. 3, No. 1, pp. 30–53 (1952).
(20) T. W. Lambe, "The Structure of Inorganic Soil," *Proceedings*, Am. Soc. Civil Engrs., Vol. 79, No. 315 (1953).

Authorized Reprint from the Copyrighted
Soils—1959 Meetings
Special Technical Publication No. 254
Published by the
AMERICAN SOCIETY FOR TESTING MATERIALS
1959

DISCUSSION

MR. M. D. MORRIS.[1]—I still do not quite understand and I wish the authors would explain in a little more detail, just what it is that is causing that hour-glass effect in the specimen.

MR. CARL B. CRAWFORD (*author*).—I am sorry that I have not made clear the reasons suggested in the paper for the development of the "spool" or "hour glass" shape by some specimens after oven drying. This is believed to be due to boundary conditions introduced by the consolidation ring which result in unequal consolidation of the specimen. When the specimen is dried the part next to the ring shrinks more than the part near the center of the specimen. This differential shrinkage causes a flaking-off of a portion of the specimen leaving the "spool" shape.

MR. CARL A. CARPENTER.[2]—You used one word just now that was not in your presentation. You mentioned "flaking off." If the hour-glass shape resulted from flaking off, I think this clarifies the whole thing.

MR. CRAWFORD.—Perhaps "flaking off" does not give the exact interpretation. It did not come off in very thin flakes, it more generally came off in small chunks.

MR. CARPENTER.—That is the point. Since there was quite an appreciable loss of material, it appears that the hour-glass shape resulted from this loss rather than from some less readily explainable cause.

MR. RONALD C. HIRSCHFELD[3] (*by letter*).—The comments which the authors have made concerning the effect of gas in the test specimen and the effects of leaching on the compression index are especially valuable. It is hoped that they will undertake more comprehensive studies of these factors.

The new definition for 100 per cent consolidation which the authors propose does not appear to take into account fully the basic assumptions of the theory of consolidation. In the theory of consolidation it is assumed that the time lag is solely that associated with the squeezing of the water out of the voids of the soil. Secondary time lag is ignored. The common definitions of 100 per cent consolidation for laboratory consolidation tests are somewhat arbitrarily chosen so that we can apply the theory of consolidation to that part of the laboratory test in which the primary hydrodynamic time

[1] Materials Equipment Consultant, New York, N. Y.

[2] Bureau of Public Roads, Division of Physical Research, Washington, D. C.

[3] Instructor in Soil Mechanics, Harvard University, Division of Engineering and Applied Physics, Cambridge, Mass.

lag clearly overshadows the secondary time lag. If the primary time lag is very small compared to the secondary time lag, the theory of consolidation can no longer be applied to the analysis of the test results and any definition of 100 per cent consolidation would seem to have little significance.

MR. CRAWFORD (*author's closure*).—The authors appreciate Mr. Hirschfeld's questioning of the "new definition for 100 per cent consolidation" but must point out that an effort has been made to retain the original concept of 100 per cent primary consolidation. It was because of the impossibility of determining 100 per cent primary consolidation for small increment tests and at pressures near the preconsolidation pressure by the usual method that an empirical relation between rate of deformation and 100 per cent primary consolidation (in tests where the usual method was satisfactory) was sought. This rate proved to be consistently of the order of 0.0008 in. per hr per in. height of sample for the soil under study. The same value should not be expected to apply to all soils.

It must be admitted that the reasons for the obscure "end of primary consolidation" in certain tests are not clear. It is for this reason that efforts are being made to check laboratory values by full scale field measurements.

A list of all publications of the Division of Building Research is available and may be obtained from the Publications Section, Division of Building Research, National Research Council, Ottawa, Canada.