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RESISTANCE OF SOIL STRUCTURE TO CONSOLIDATION

by

C. B. CRAWFORD

(including discussion)

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THE RESISTANCE OF SOIL STRUCTURE TO CONSOLIDATION

C. B. CRAWFORD*

ABSTRACT

The rate at which soil specimens are compressed in the standard consolidation test is farther from reality than the rate of strain in any other test. For a short period after the application of a load increment the rate of compression may be several million times faster than that experienced in the field. This paper describes a series of tests carried out at constant rates of compression varying from 0.16 to 8 per cent per hour in which pore pressures were measured. The effective stress-compression curves are compared with an average curve obtained by incremental loading. Based on these results, it is concluded that the compressibility of this soil is dependent on the average rate of compression and that the soil structure has a substantial time-dependent resistance to compression. This time-dependent resistance may be expected to have a significant influence on computed permeability coefficients and field rates of consolidation.

SOMMAIRE

Le taux auquel les échantillons de sol sont comprimés dans l'essai standard de consolidation est moins proche de la réalité que le taux de la résistance dans aucun autre essai. Pour une courte période après l'application d'une augmentation de charge le taux de compression peut être de plusieurs millions de fois plus rapide que celui qu'on a expérimenté *in situ*. On décrit dans cette étude une série d'essais effectués à des taux de compression constants, évalués entre 0.16 et 8 pour cent à l'heure, au cours desquels les pressions des pores ont été mesurées. Les courbes contrainte-compression sont comparées à une courbe moyenne obtenue par chargement progressif. A partir de ces résultats on a conclu que la compressibilité du sol étudié dépend du taux moyen de compression et que la structure du sol a une bonne résistance dans le temps à la compression. Cette résistance dans le temps peut influer de façon marquée sur les coefficients de perméabilité calculés et sur les taux de consolidation *in situ*.

Stress release, temperature and moisture changes, and other disturbances caused by sampling and storage may have a profound influence on the measured stress-deformation properties of an undisturbed clay specimen. The rate at which the test specimen is loaded may have an equal or greater influence on these properties. From a study of available literature, Bishop and Henkel (1962) concluded that the decrease in compressive strength for a tenfold increase in time to failure is approximately 5 per cent. Tests on a sensitive clay indicated decreases in excess of 10 per cent for a similar increase in time to failure (Crawford, 1959).

The rate at which soil specimens are compressed in the standard consolidation test has received relatively little attention although this rate is farther from reality than the rate of strain in any other test. The reason for this is that in the tiny laboratory specimen the drainage path is so short that relatively large load increments are needed to produce sufficient pore pressures for the hydrodynamic lag to control the rate of compression. Consequently, for a short period after loading the laboratory rate of compression may be several million times faster than that experienced in the field.

In a previous paper (Crawford, 1964) it was shown that for a sensitive clay the interpretation of preconsolidation pressure depends in large measure on the duration of load increments, especially the load increment nearest the

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probable preconsolidation pressure. This paper describes an extension of these studies in which the resistance to deformation of the soil structure is measured at rates of strain so slow that the hydrodynamic effect is negligible.

Factors That Influence Compressibility

The various factors that influence the compressibility of a saturated soil have been reviewed by Wahls (1962). Wahls's paper, and the discussion of it, provide a penetrating assessment of the relationship between the phenomena known as "primary" and "secondary" consolidation. It is generally agreed that under appropriate loading and boundary conditions the hydrodynamic effect will control the rate of compression until the pore pressures within the soil are substantially dissipated. Less is known of the "secondary" phase of consolidation, but it is attributed to the soil's structural resistance to compression, a factor not taken into account in the original Terzaghi theory. Later, Terzaghi (1941) discussed in great detail the influence of the natural soil structure on compressibility characteristics.

Taylor's (1942) classical work was devoted principally to the "... study of the differences between the rates of compression observed in laboratory samples and the rates given by theory...." As a result of this he developed a theory to account for the plastic resistance to compression of the soil structure. Tjong-Kie (1958) emphasized the importance of viscous resistance and consolidation hardening as secondary time effects. Leonards and Girault (1961) associated large rates of secondary compression with small load-increment ratios.

The sensitive Leda clay requires special consideration. Usually an undisturbed specimen of this clay is brittle and will fail in compression at a low axial strain. Similarly, the recompression curve in an oedometer is very flat, with a sudden break into virgin compression. An irreversible breaking of bonds seems to occur when the structure is slightly deformed and the applicability of a laboratory test in which sudden deformations are caused must be questioned.

The Soil

Tests were performed on specimens cut from an undisturbed block sample of Leda clay obtained at a depth of 53 ft. from the excavation for the Ottawa sewage treatment plant. Information on the nature of the material and the strength characteristics of an adjacent block of soil have been published (Crawford, 1963). The clay has a sensitivity greater than 50, a liquid limit of 31 per cent, and a plastic limit of 23 per cent. The natural water content averages 55 per cent and it contains 65 per cent of clay size material.

Test Procedure

All specimens were fitted into 20 sq. cm. by 2 cm. high teflon-coated consolidation rings smeared with molybdenum disulfide grease to reduce side-wall friction. One set of tests was carried out in an ordinary 11 to 1 lever ratio loading frame and a second set was carried out at a variety of strain rates in a gear-driven compression testing machine. Deflection of the proving ring

TABLE I

Results of Incremental Loading Tests

Specimen	94–19–3		94–19–5		94–19–6		94–19–8		94–19–9	
Initial Void Ratio	1.599		1.582		1.582		1.548		1.532	
Pressure (kg./sq. cm.	Duration of Increment (hr.)	Compression (per cent)	Duration of Increment (hr.)	Compression (per cent)	Duration of Increment (hr.)	Compression (per cent)	Duration of Increment (hr.)	Compression (per cent)	Duration of Increment (hr.)	Compression (per cent)
0	0	0	0	0	0	0	0	0	0	0
1	16.7	0.42	2.5	0.39	16.7	0.62	2.0	0.16	1.5	0.35
2	2.0	0.54	1.1	0.50	1.9	0.74	19.3	0.39	1.5	0.39
4	2.9	0.81	18	0.93	21	1.08	4.6	0.78	19	1.03
6	—	—	—	—	—	—	24	7.8	25	7.82
8	24	14.4	24	13.3	24	14.9	19.7	14.9	24	15.1
12	—	—	—	—	—	—	72	20.5	22	20.1
16	72	23.2	24	22.2	24	23.4	22	22.8	72	23.1

reduced the rate of strain in the recompression range. Drainage was from the top face only, and pore pressures were measured through a recessed $\frac{1}{2}$-in. diameter porous stone connected to a pressure transducer. All tests were performed at a controlled temperature of 68°F ($\pm\frac{1}{4}$°).

Test Results

Incremental Loading

The percentage of compression under each load increment, computed from initial volume, is reported in Table I, together with the duration of increment. A variety of load durations were used in the recompression range with little apparent effect on the pressure-compression curve. Above the preconsolidation pressure, daily loads were applied except for the last one or two increments (see Table I). Previous tests on a similar type of soil (Crawford, 1964) revealed a much greater influence of load duration when one of the load increments happened to be just about equal to the probable preconsolidation pressure.

The pressure-compression curve for one of the specimens (94–19–8) is drawn in Figure 1. Compared with similar tests in Table I, the reproducibility of these

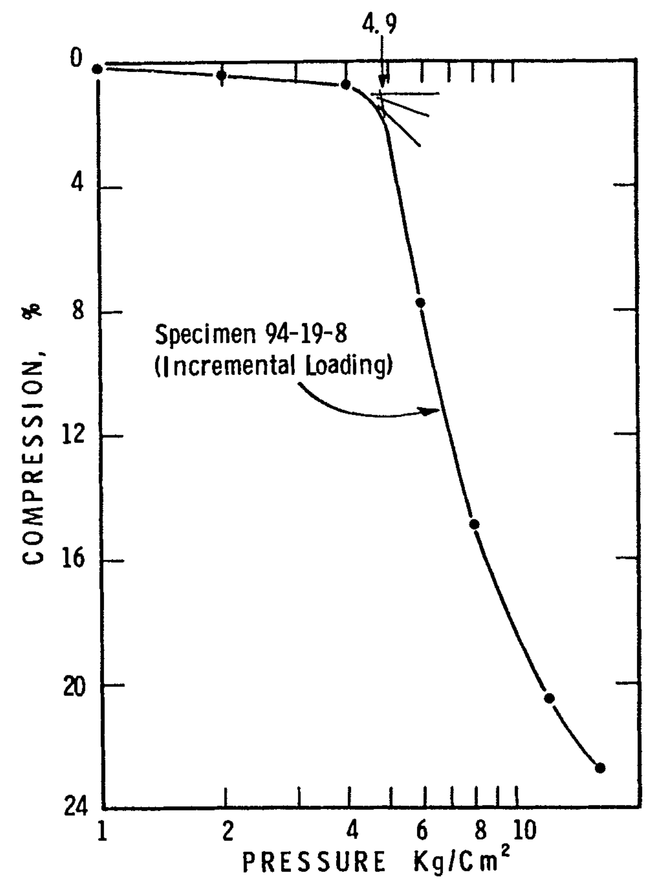

FIGURE 1. Pressure-compression curve for specimen 94-19-8

TABLE II

Results of Controlled Strain Tests

Specimen	Initial Void Ratio	Natural Water Content (per cent)	Rate of Strain (per cent per hr.)	Max. Pore Pressure (per cent of applied P)	Preconsolidation Pressure (kg./sq. cm.)
94–19–1	1.579	54.7	8	15	7.4
94–19–2	1.582	54.9	2	3	6.9
94–19–7	1.584	55.2	0.6	1	5.7
94–19–4	1.560	54.1	0.16	0.9	4.9

results is considered to be satisfactory and they indicate a preconsolidation pressure of 4.9 kg./sq. cm.

Controlled Strain Tests

Controlled strain tests were run at four rates of strain, 8, 2, 0.6, and 0.16 per cent per hour, and the results are given in Table II. Pressure-compression curves for two of the tests are given in Figures 2 and 3, with time recorded on the right-hand scale. The other tests have curves of similar shape.

Maximum measured pore pressures are given relative to total applied pressure in Table II. Although specimen 94–19–1 had a maximum pore pressure equal to 15 per cent of the applied pressure, the maximum was obtained only after a great amount of compression. The pore pressure was only 3 per cent at the preconsolidation pressure and made little difference to the interpretation. Figures 2 and 3 and all interpretations are based on average effective stresses by subtracting one half of the measured pore pressure from the total vertical stress.

Figure 4 compares the compression-time relationships for three tests beginning with the virgin consolidation. Each of the tests began virgin consolidation after about 1 per cent strain.

Discussion

For these very uniform, undisturbed specimens of sensitive clay the preconsolidation pressure, as interpreted from effective stress-compression curves, varies from 7.4 kg./sq. cm. at a rapid rate of strain to 4.9 kg./sq. cm. at a slow rate of strain. In accordance with Taylor's theory (1948) the plastic resistance is larger at fast rates of compression than at slow rates. It may be inferred that at slower rates of strain, which were not possible with available equipment, the pressure-compression curve would be displaced even farther to the left and the interpretation of preconsolidation pressure would be lower than 4.9 kg./sq. cm.

The pressure-compression relationship at the slow continuous rate of strain (94–19–4) is nearly identical to the curve for incremental loading (Figure 3). Although the incremental test compresses at an average rate not much faster than the controlled strain test (94–19–4), the actual rate varies from 20 per cent per hour during the first minute of loading application to 0.008 per cent

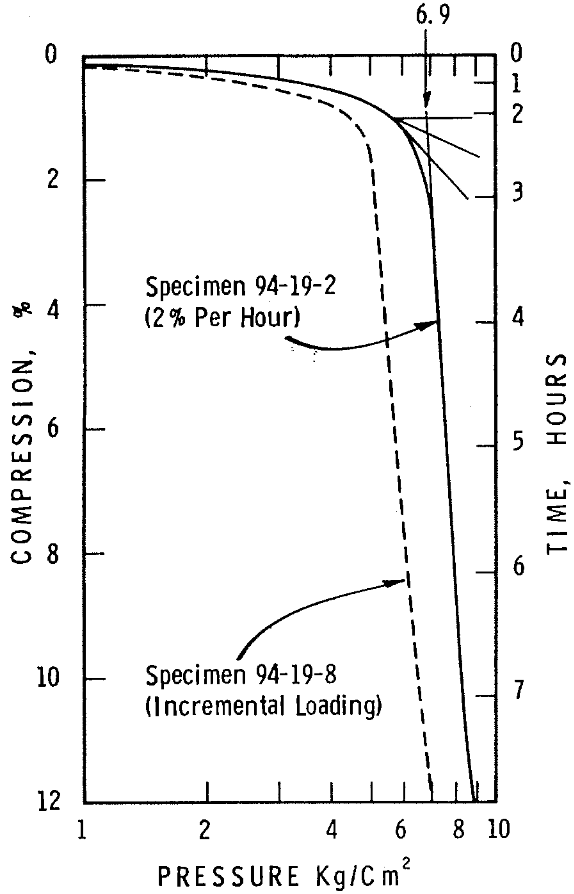

FIGURE 2. Pressure-compression curve for specimen loaded at fast constant rate

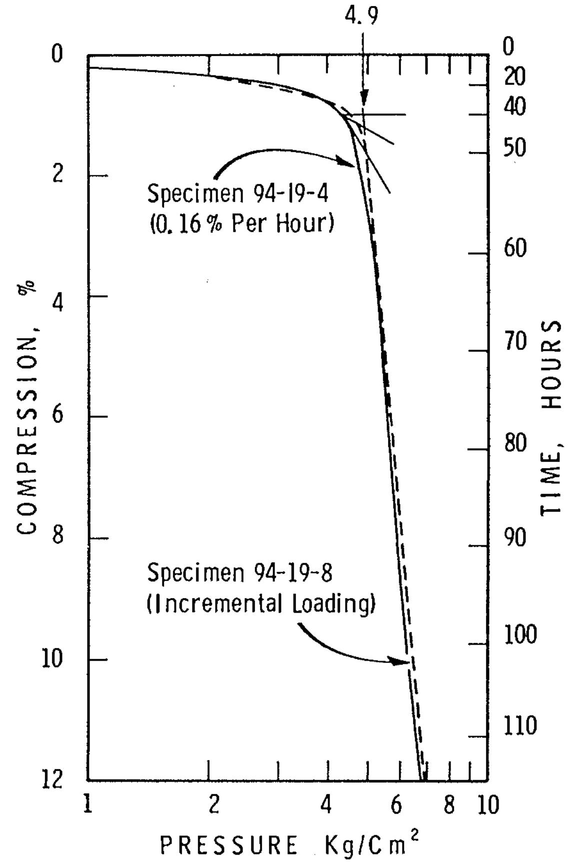

FIGURE 3. Pressure-compression curve for specimen loaded at slow constant rate

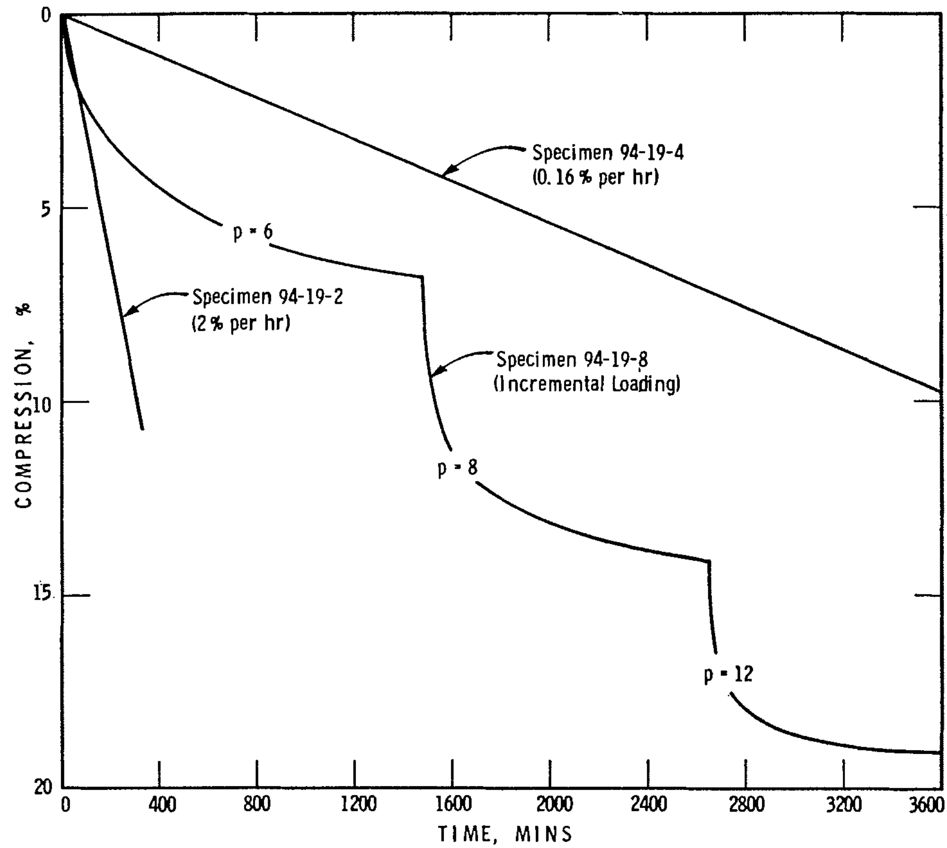

FIGURE 4. Virgin compression-time curves for incremental loading compared with controlled strain

per hour at the end of one day. This suggests that the final void ratio under a particular load depends primarily on the average rate of compression and is not greatly affected by shock loading as visualized by Langer (1936). An important question requiring further study is the maximum laboratory rate of compression that can be used to compute the correct field settlement.

The very fast rates of compression experienced by the laboratory specimens immediately after an incremental load application create an important interpretation problem. If the plastic resistance is high during this period, then the time lag is only partly due to the hydrodynamic lag and the computed permeability will be too low. The estimated time for consolidation in the field will therefore be too long.

CONCLUSIONS

By compressing almost identical specimens of undisturbed sensitive clay at various rates of strain and correcting for pore pressure influences, it has been shown that the soil structure has an important time-dependent resistance to compression. This resistance may be expected to have a substantial influence on computed permeability coefficients and on estimates of field rates of consolidation.

The pressure-compression relationship for these specimens is relatively independent of method of loading, provided that the average rate of compression is the same. Further investigation of very slow laboratory rates is required, but the greatest need is for comparisons with field compression in order to assess and evaluate laboratory results.

Acknowledgments

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DISCUSSION

THE RESISTANCE OF SOIL STRUCTURE TO CONSOLIDATION

NYAL E. WILSON*

The author, in referring to tests with incremental loading, has stated that the load duration during the recompression range had little influence on the pressure-compression curve. It was observed that the load duration had a much greater influence when the load increment happened to be just equal to the probable preconsolidation pressure.

The analysis of data (Crawford, 1964) for tests on Leda clay (a similar clay but having a lower preconsolidation pressure) indicates the influence of load duration on the rates of compression. The rates of compression in the laboratory have been examined as the relationship to field rates is not realistic and the arbitrary breakdown into primary and secondary processes is questionable.

The construction of the pressure-compression curve either by interpreting the $e - \log t$ curves or by pore pressure measurements leads to a range of values for the preconsolidation pressure. Terzaghi's theory considers that consolidation is due solely to the expulsion of pore water and neglects structural deformations; as most soils exhibit some secondary effects, the load duration is important.

For consolidation tests on Leda clay (Crawford, 1964) and for tests on organic soil (Lo, 1964) it was found that a significant change in the rates of compression occurred at some time prior to the dissipation of the pore-water pressure to zero and at some greater time prior to "end of primary" as interpreted from the $e - \log t$ curves. When the rates of compression are plotted versus time on a log–log graph, the data form two tangents joined by a short curve (see Figure 1); the significant change in the rates occurs at the intersection of the two tangents. It is at this point of significant change in the rates of compression that the primary consolidation is predominated by the secondary effects. Structural deformations exist from the initial instant of loading but do not predominate, in the case of laboratory testing, during the period of excess pore-water pressure. According to the rates of compression, the influence of secondary effects predominates earlier than that found by either pore-water pressure measurements or by extrapolation from $e - \log t$ curves. For field consolidation where the rates of compression are very small, the secondary effects may have an even greater influence.

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*Assistant Professor, McMaster University, Hamilton, Ont.

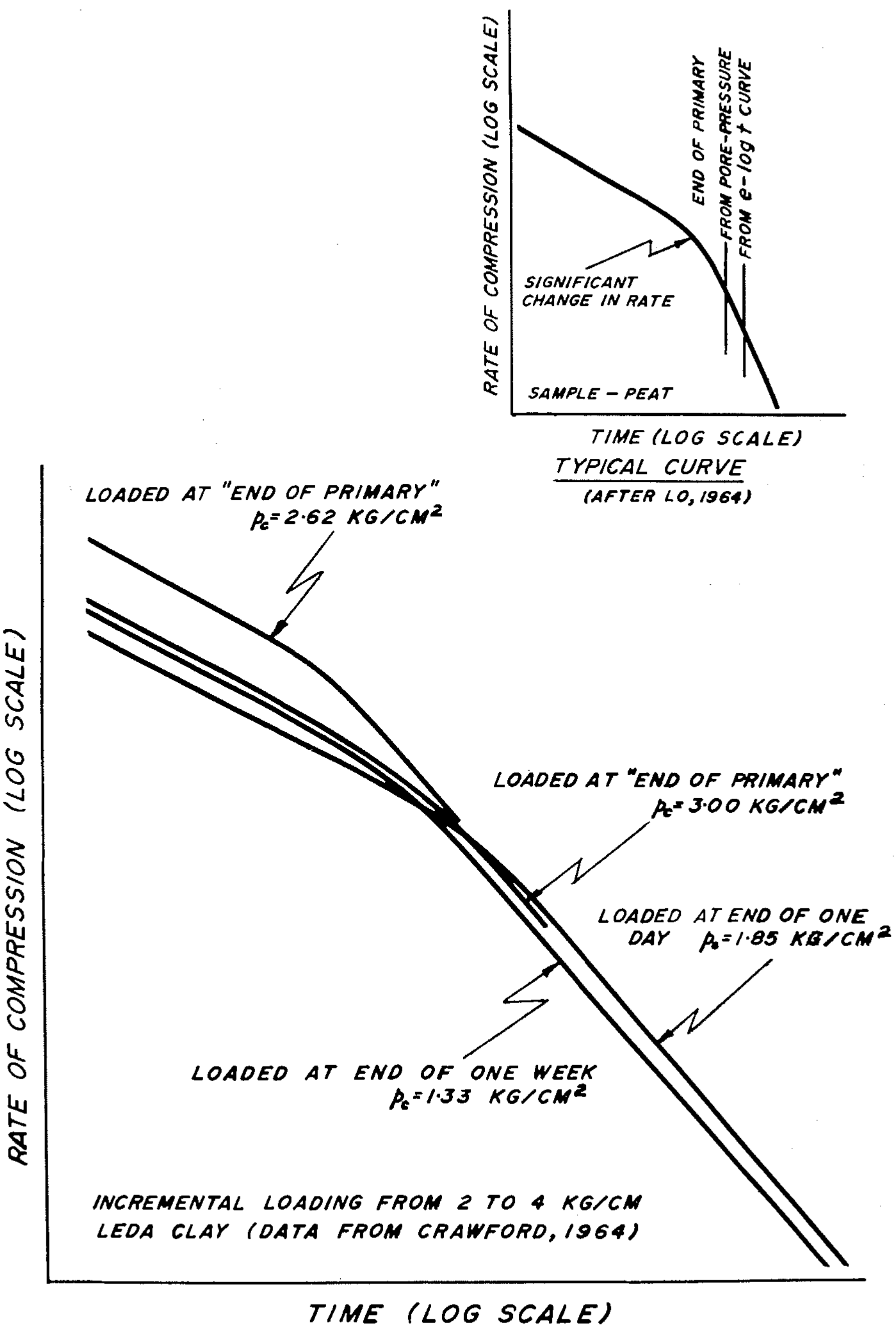
RATE OF COMPRESSION (LOG SCALE)
END OF PRIMARY
FROM PORE-PRESSURE
FROM e-log t CURVE
SIGNIFICANT
CHANGE IN RATE
SAMPLE – PEAT
TIME (LOG SCALE)
TYPICAL CURVE
(AFTER LO, 1964)
LOADED AT "END OF PRIMARY"
p_c = 2·62 KG/CM²
RATE OF COMPRESSION (LOG SCALE)
LOADED AT "END OF PRIMARY"
p_c = 3·00 KG/CM²
LOADED AT END OF ONE
DAY p_c = 1·85 KG/CM²
LOADED AT END OF ONE WEEK
p_c = 1·33 KG/CM²
INCREMENTAL LOADING FROM 2 TO 4 KG/CM
LEDA CLAY (DATA FROM CRAWFORD, 1964)
TIME (LOG SCALE)

FIGURE 1.

GERALD PATRICK RAYMOND*

It would appear that the test results presented in the paper for the one soil tested, Leda clay, are in agreement with the author's statement that "the soil structure has an important time-dependent resistance to compression." On the other hand there appears to be no data presented to support his conclusion that "this resistance to compression may be expected to have a substantial influence on computed permeability coefficients and on estimates of field rates of consolidation"; indeed the paper is void of any data on the coefficient of consolidation or the coefficient of permeability of the soil. Furthermore, no reference has been made to any results already published. Hamilton and Crawford, (1959) published some test data for Leda clay in which the coefficients of compressibility, permeability, and consolidation were tabulated. According to Hamilton and Crawford these tests were performed using increment ratios of about 1, $\frac{1}{3}$, and $\frac{1}{10}$. Owing to the difficulty in determining the end of primary consolidation Hamilton and Crawford used a modified log time curve-fitting method. The results have been plotted in Figures 1, 2, and 3.

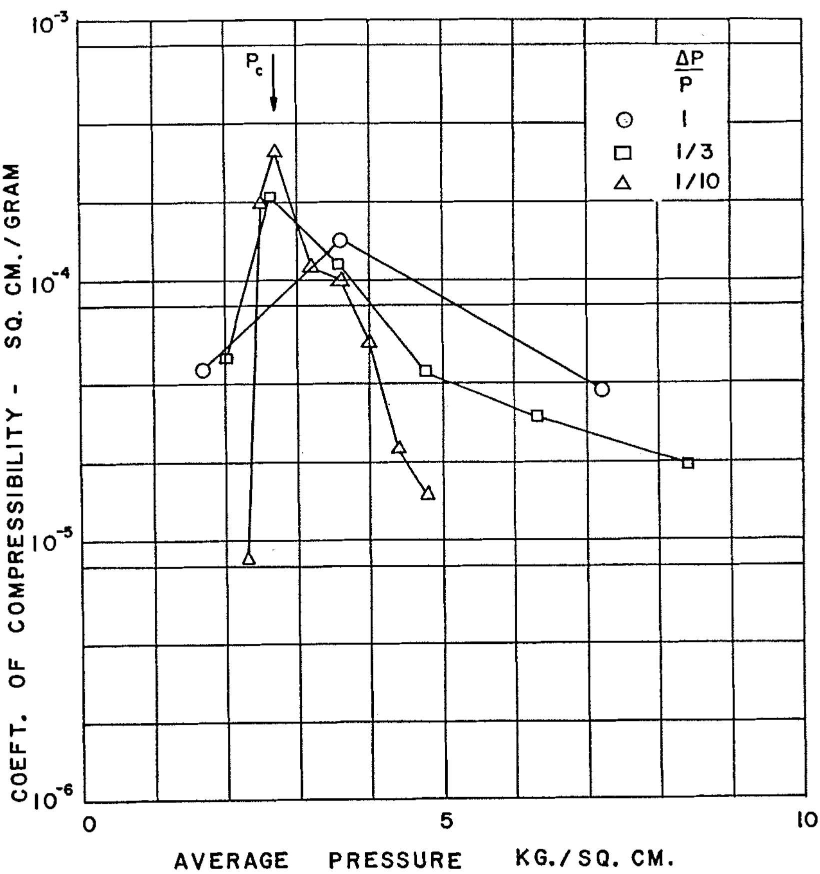

FIGURE 1. Effect of increment ratio on the coefficient of compressibility of Leda clay (after Hamilton and Crawford, 1959)

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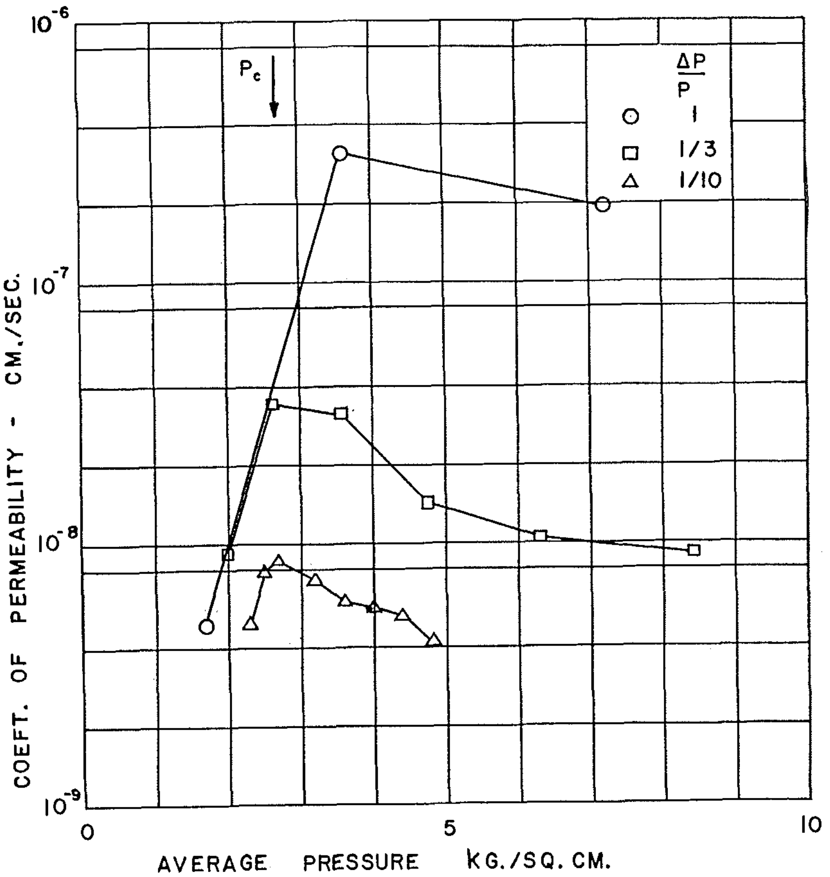


FIGURE 2. Effect of increment ratio on the coefficient of permeability of Leda clay (after Hamilton and Crawford, 1959)

The results indicate that as the increment ratio is increased both the coefficients of permeability and consolidation are increased. Taylor (1942) and Leonards and Ramiah (1959) also noted an increase in the coefficient of consolidation (calculated from conventional curve-fitting methods) as the load increment increased. The author, making reference to the theoretical work of Taylor (1942), suggests that as the rate of deformation is increased, and therefore presumably the load increment ratio, the computed coefficient of consolidation would decrease. This appears to be contradictory to already published results. Furthermore Taylor's final equation for rate of settlement is independent of load increment. This may, however, be due to some of Taylor's simplifying assumptions made while deriving the settlement relationship.

In support of the author's deduction the writer has observed that when small load increments are used during consolidation testing the rate of dissipation of pore water pressure increases. This phenomenon has also been reported by Leonards and Girault (1961). This would suggest that if the coefficient of consolidation was based on the rate of dissipation of pore water pressure its

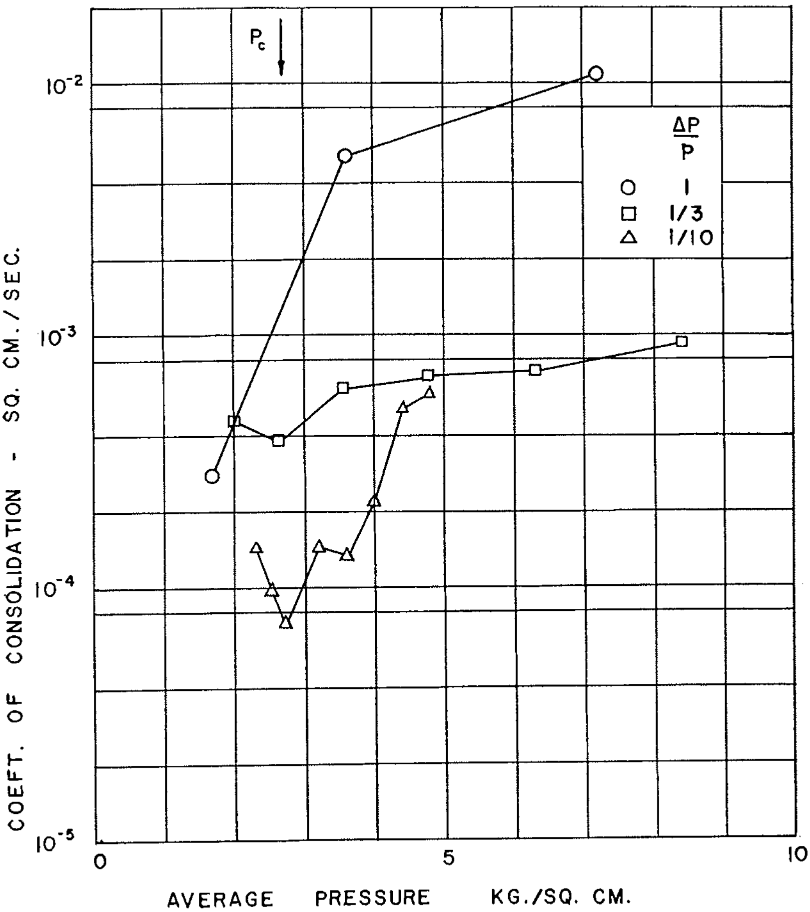


FIGURE 3. Effect of increment ratio on the coefficient of consolidation of Leda clay (after Hamilton and Crawford, 1959)

magnitude would decrease with increasing load ratio. Crawford (1964) has already suggested one possible method of curve-fitting using the observed midplane pore pressures and the settlements.

The writer would hope that the author would publish, preferably in tabular form, the coefficients of compressibility, permeability, and consolidation obtained from his latest series of tests, and also, if possible, the information relating to the results reported in an earlier paper (Crawford, 1964). It is also hoped that he will develop some new theoretical work which may explain the experimental results. This should preferably be done before attempting to relate laboratory data to field data.

Considering the main point of this paper, which may be regarded as the determination of a laboratory preconsolidation pressure which agrees with the field preconsolidation pressure, it should be pointed out that in the field the soil will, in general, be subject to different principal stress ratios, except when the consolidation is purely one dimensional.

In an attempt to investigate the effect of various stress ratios Jarrett (1964) under the supervision of the writer conducted some six series of tests on undisturbed Leda clay from Cornwall, Ontario. Each series consisted of tests on from two to four specimens from one sample tube. The soil specimens were consolidated at a constant stress ratio with the lateral pressure being increased at a constant rate of about 0.5 kg./sq. cm./day from an initially consolidated constant pressure of $\frac{1}{8}$ kg./sq. cm. The pressure was increased by means of motorizing a mercury pot constant pressure device similar to that described by Bishop and Henkel (1957). The axial load was applied by means of a hydraulic loading device which was connected to the mercury pot constant-pressure device. Each test was performed in a triaxial cell with one way drainage from the top of the specimen only. Pore water pressures were measured at the base of the specimen. The main part of the equipment is shown in Figure 4. The results from any one series of tests may be summarized as follows:

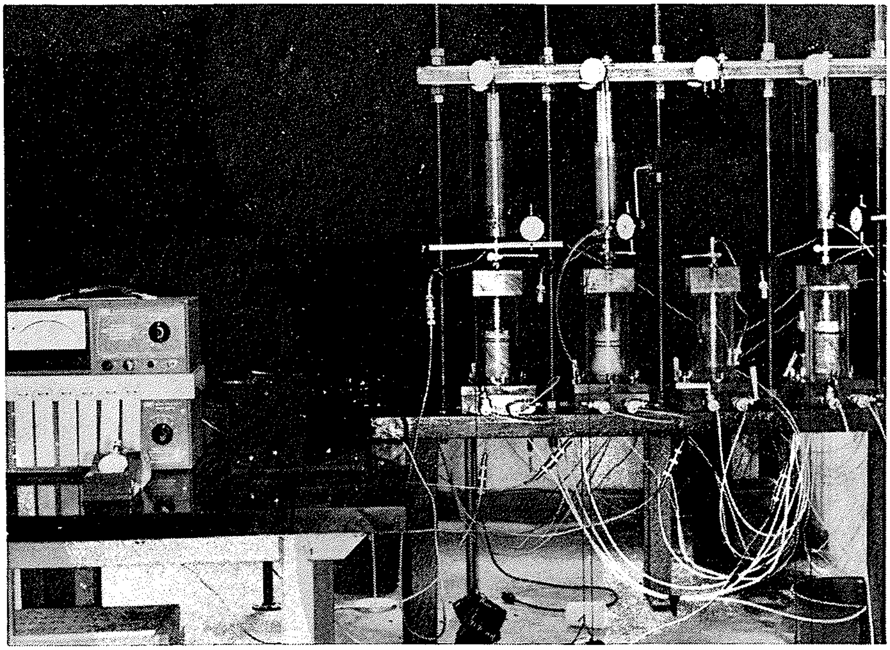


FIGURE 4. Layout of equipment for testing soil specimens at a constant principal stress ratio

(1) Pore Water Pressure *v.* Total Lateral Pressure (or Time). The higher the principal stress ratio the quicker the breakdown in grain structure and thus the quicker the rise in pore water pressure at the base (Figures 5 and 6). This observation is also obtained when the pore water pressure is plotted against the axial stress.

(2) Volume Change *v.* Total Axial Stress. The curves tend towards a unique relationship for all values of stress ratio (Figures 7 and 8).

(3) Change in Height *v.* Total Axial Stress. The higher the principal stress ratio the greater the change in height (Figures 9 and 10).

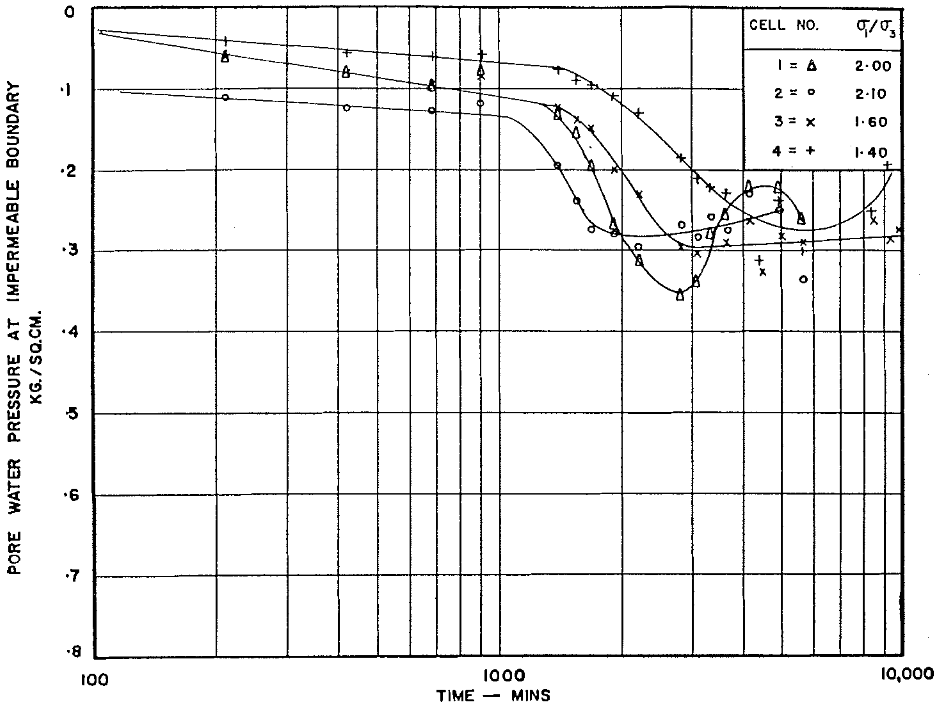


FIGURE 5. Constant rate of loading consolidation test results: pore water pressure *v.* time for sample from 31.5 ft. depth

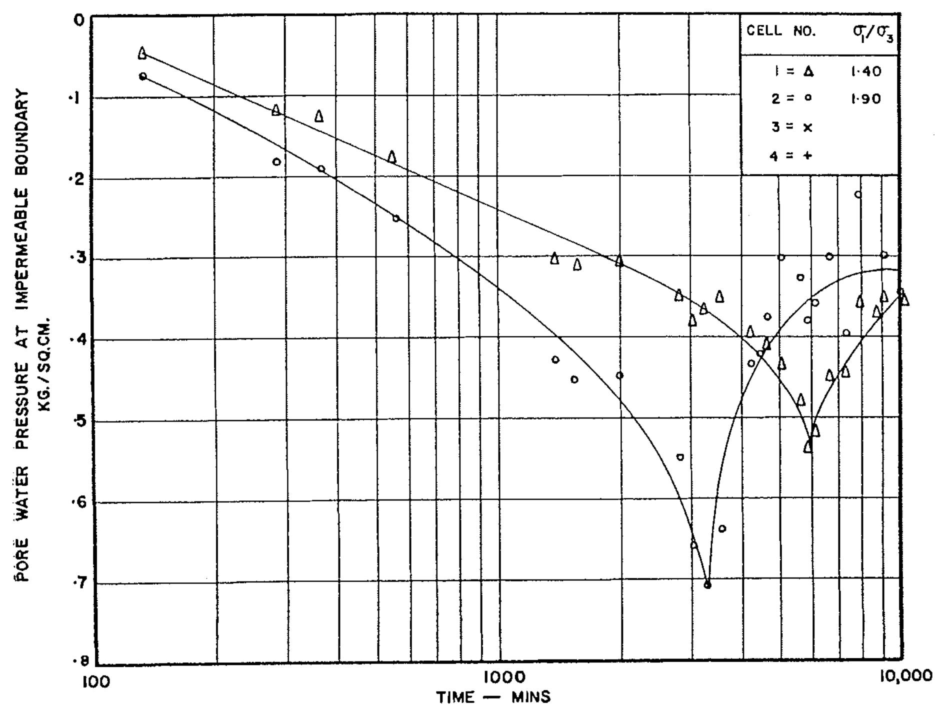


FIGURE 6. Constant rate of loading consolidation test results: pore water pressure *v.* time for sample from 42.0 ft. depth

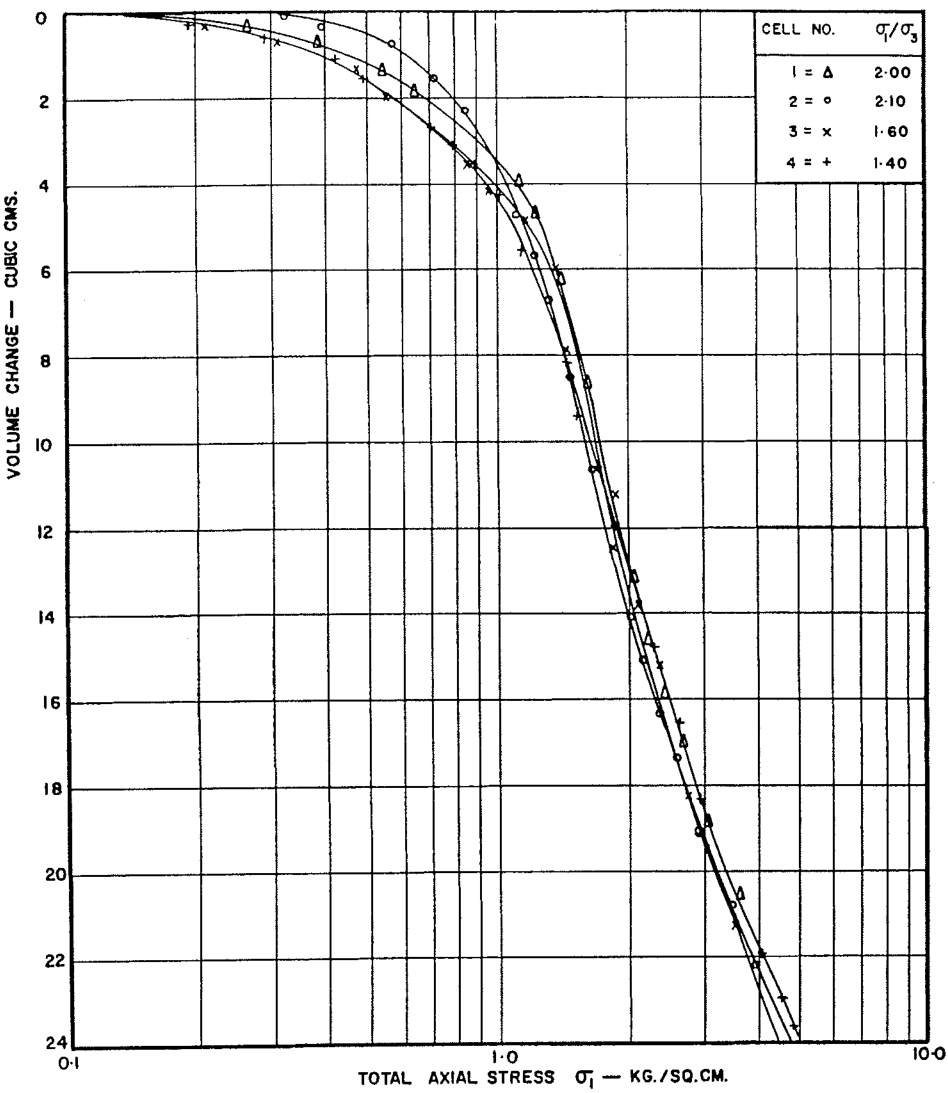


FIGURE 7. Constant rate of loading consolidation test results: volume change *v.* total axial stress for sample from 31.5 ft. depth

It is perhaps interesting to report that the above observations were observed for all six series of tests without exception. More testing is being done, and the effects in terms of effective stresses are being analysed.

The graph of change in height *v.* axial stress is presumably the graph to use when estimating the preconsolidation load. If this is an acceptable statement then it is immediately apparent that for these tests different stress ratios produce different preconsolidation pressures. Furthermore the lowest preconsolidation pressure would appear to occur where the stress ratio is largest. If the results are plotted in terms of effective stresses this difference is increased. In the field the writer would expect this to occur where the vertical stress increase

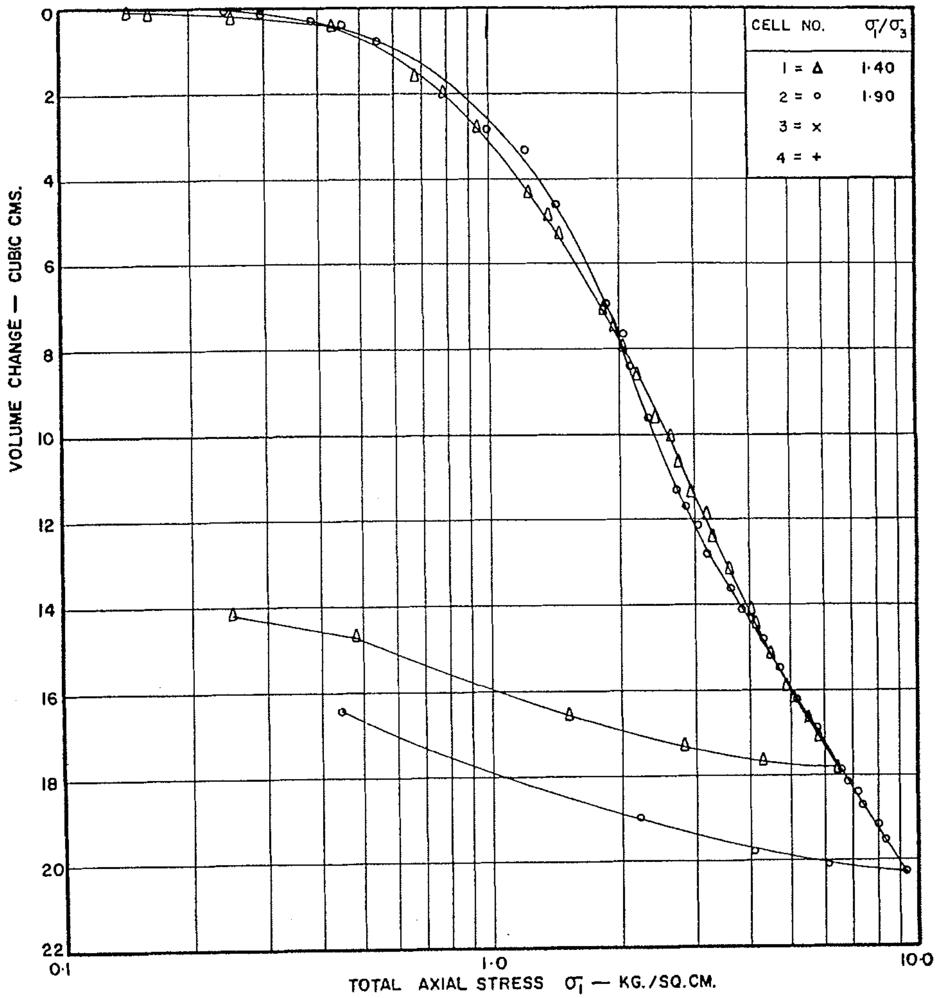


FIGURE 8. Constant rate of loading consolidation test results: volume change *v.* total axial stress for sample from 42.0 ft. depth

and thus the settlements are likely to be largest. It appears that the estimation of the preconsolidation pressure is by no means easy, even, as the author has pointed out, for the purely one-dimensional problem.

Regarding the one-dimensional test the writer would refer to the observations and conclusions reported by Leonards and Altschaeffl (1964). Leonards and Altschaeffl suggest that secondary consolidation will induce a preconsolidation pressure higher than the existing pressure. Leonards and Altschaeffl have called this induced preconsolidation pressure the quasi-preconsolidation pressure. They also suggest that all natural clays should exhibit this phenomenon. The writer suggests that some quasi-preconsolidation may occur in any sample which exhibits secondary consolidation. Furthermore since secondary consolidation is believed to occur at the same time as primary consolidation

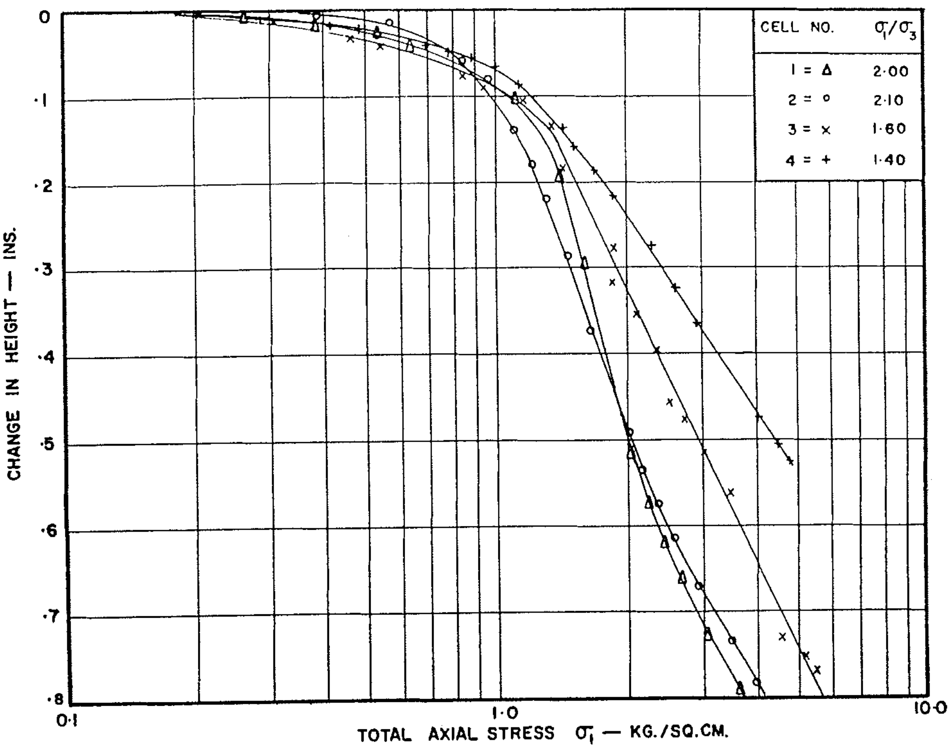


FIGURE 9. Constant rate of loading consolidation test results: change in height *v.* total axial stress for sample from 31.5 ft. depth

this phenomenon should be observed for all test increments, although the magnitude of the quasi-preconsolidation pressure may be dependent on a function of the duration of the previous load increment.

Chan (1964) under the supervision of the writer has performed one-dimensional and hydrostatic consolidation tests on two unlike clay layers where drainage was allowed at one end of the sample only. Each layer of the sample was placed in an oedometer (Figure 11) or triaxial cell, Cells were connected for drainage purposes in series (Figure 12). The settlements (or volume changes) of the individual layers and the pore water pressures on their boundaries were recorded. Typical results using artificially deposited (Townsend and Gay, 1964) New Liskeard clay and Leda clay are shown in Figure 13. It may be seen that when New Liskeard soil constitutes layer A, the layer closest to the permeable boundary, the Leda clay, layer *B*, which is the more permeable of the two soils, consolidates with almost identical pore water pressures on its two boundary faces. Figure 14 shows, for this case, the increase in effective stress on the two boundaries of Leda clay for a one-dimensional test in which the load was instantaneously increased from 1 kg./sq. cm. to 8 kg./sq. cm. The load was increased by decreasing the back pressure, thus ensuring a 100 per cent initial response of the pore water pressure gauges. No attempt was made to rest the soil at the end of the previous load increment yet a small quasi-preconsolidation pressure is apparent. Also apparent is the fact that the soil

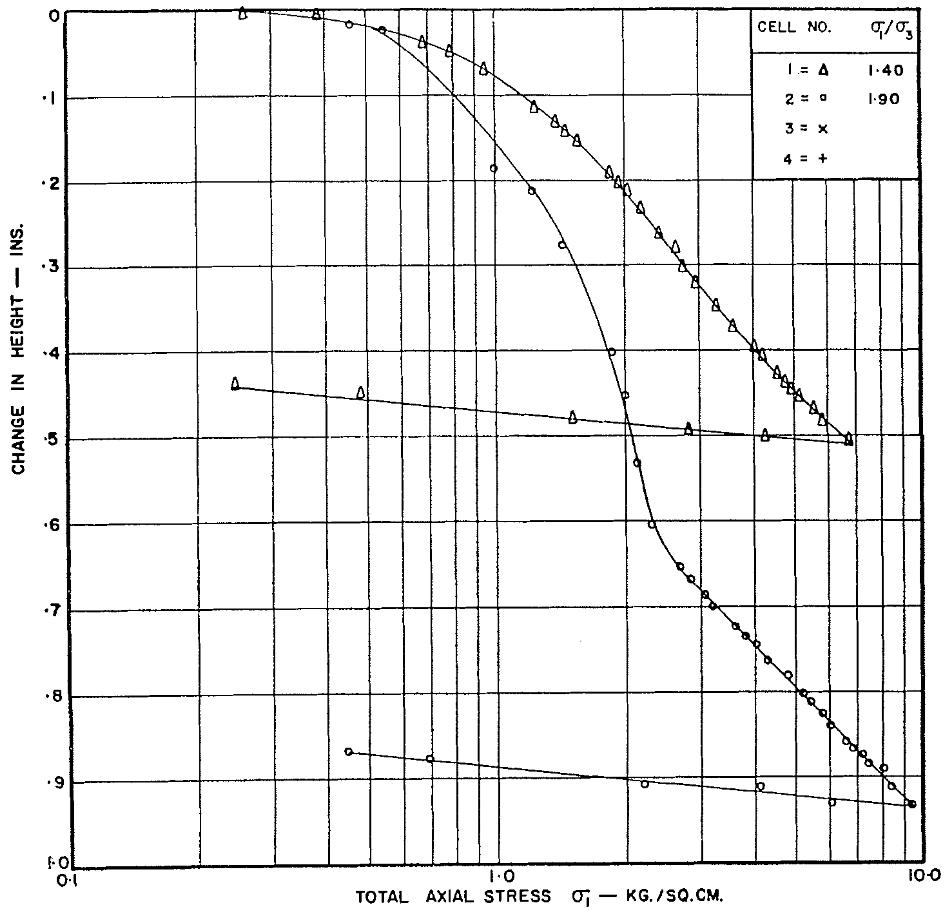


FIGURE 10. Constant rate of loading consolidation test results: change in height *v.* total axial stress for sample from 42.0 ft. depth

follows an almost linear void ratio-log pressure relationship during the consolidation process, when the quasi-preconsolidation pressure has been exceeded.

It may be seen from Figure 13 that considerable secondary consolidation has occurred within layer A prior to the end of primary consolidation based on the total settlement of both layers A and B combined. This supports the writer's belief that secondary consolidation occurs during primary consolidation and the two processes are to some extent interrelated. The end of primary consolidation of each layer was obtained using the semi log curve-fitting method.

The writer is working on a theory of slightly overconsolidated soils assuming two linear void ratio-pressure relationships meeting at a common critical value of effective stress (σ_c). Thus the soil acts as a two-layered soil with an internal moving boundary. The coefficient of permeability is assumed constant within any one layer, although it may be different in each different layer. A typical series of results is shown in Figure 15 for different values of the critical pore pressure which is expressed as:

$$u_c = \{(\sigma_f - \sigma_c)/(\sigma_f - \sigma_i)\}\ 100 \text{ per cent}$$

where σ_f = the final effective pressure,

σ_i = the initial effective pressure.

Assuming that for the overconsolidated range the coefficient of consolidation is larger than, and the modulus of volume change is smaller than, their respective values in the normally consolidated range, then, for a specified initial and

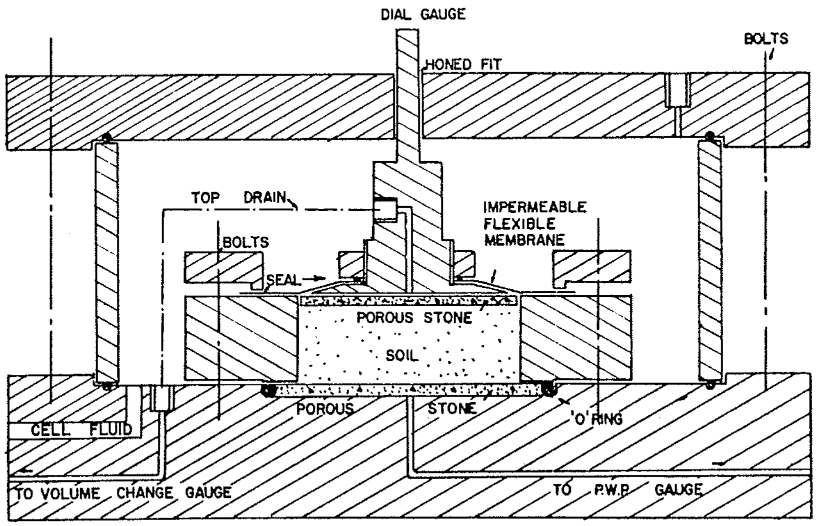


FIGURE 11. Oedometer cell suitable for use with a back pressure

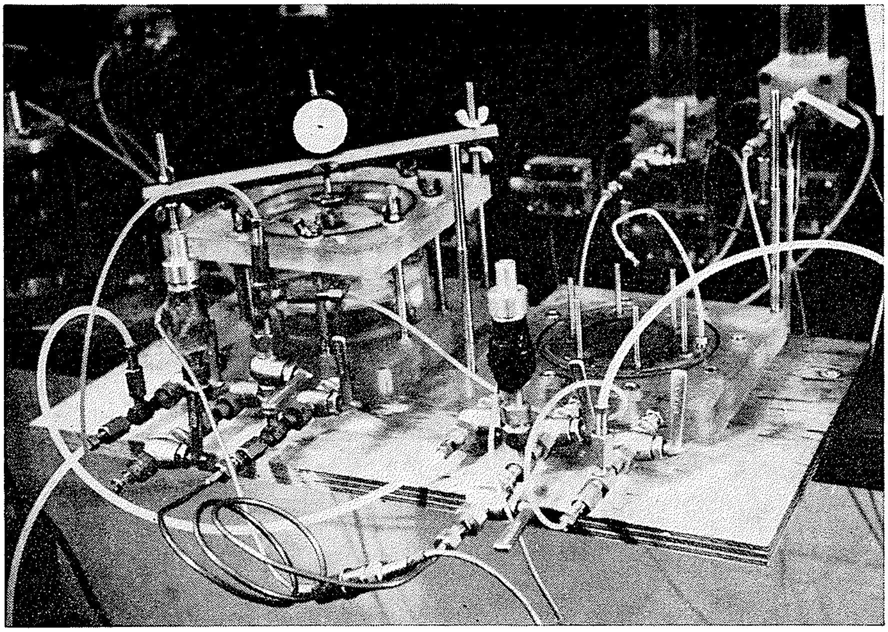
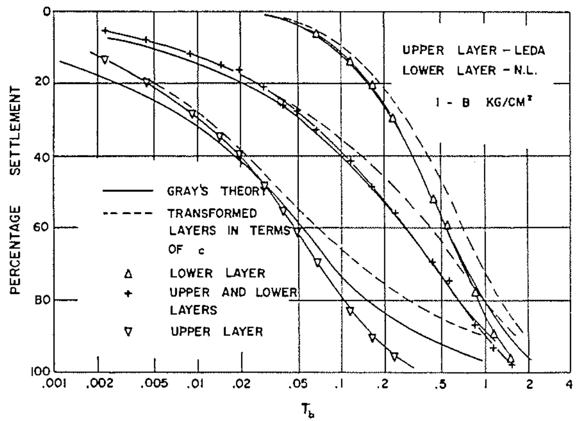
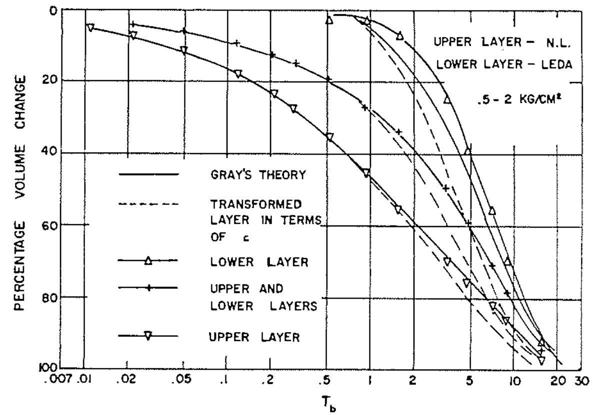


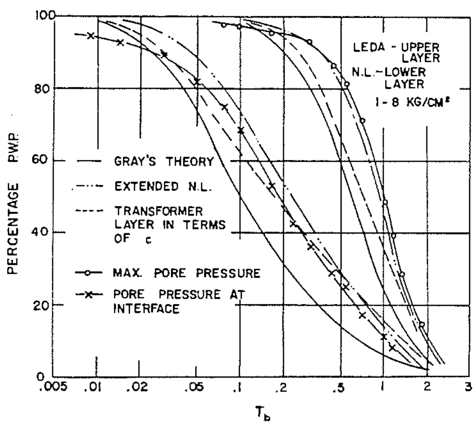
FIGURE 12. Layout of oedometer cells for simultaneous one-dimensional consolidation tests



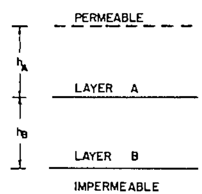
A COMPARISON OF EXPERIMENTAL AND THEORETICAL CURVES FOR A TWO-LAYERED SOIL (SIMULTANEOUS ONE-DIMENSION CONSOLIDATION)



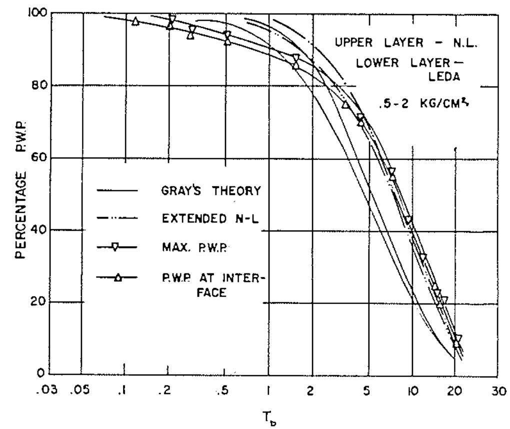
A COMPARISON OF EXPERIMENTAL AND THEORETICAL SETTLEMENT CURVES FOR A TWO-LAYERED SOIL (SIM. HYDROSTATIC CONSOLIDATION)



A COMPARISON OF EXPERIMENTAL AND THEORETICAL PORE-PRESSURE CURVES FOR A TWO-LAYERED SOIL (SIM. 1-D CONSOLIDATION)



BOUNDARY CONDITIONS



A COMPARISON OF EXPERIMENTAL AND THEORETICAL PORE PRESSURE CURVES FOR A TWO-LAYERED SOIL (SIM. HYDROSTATIC CONSOLIDATION)

FIGURE 13. Typical results of simultaneous one-dimensional consolidation tests

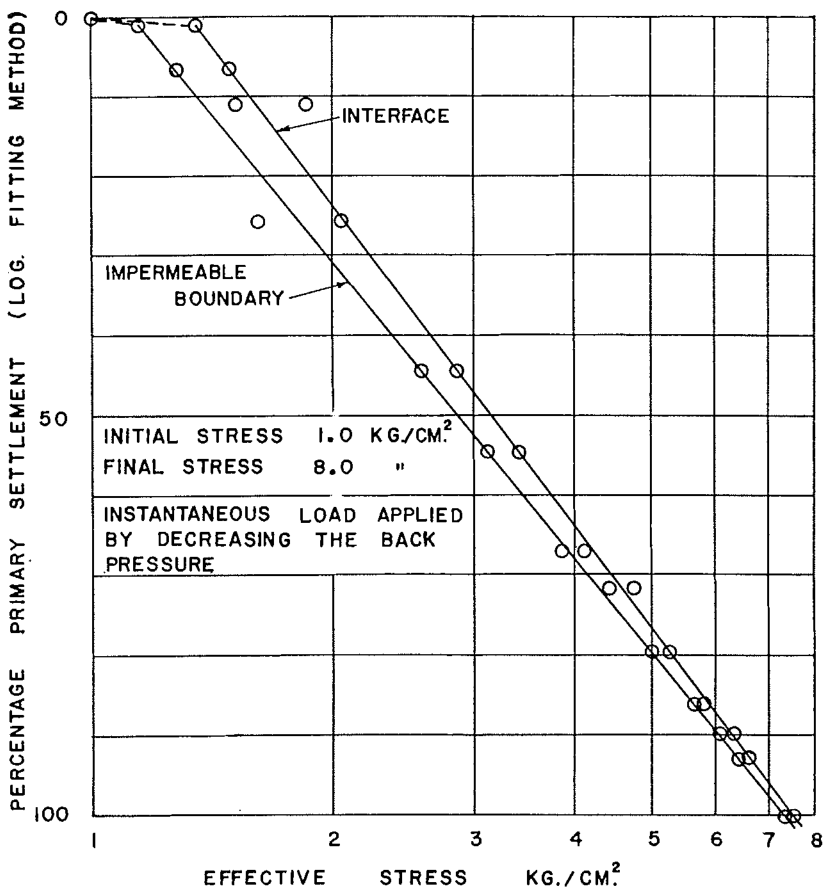


FIGURE 14. Typical observed effective stress path followed during consolidation

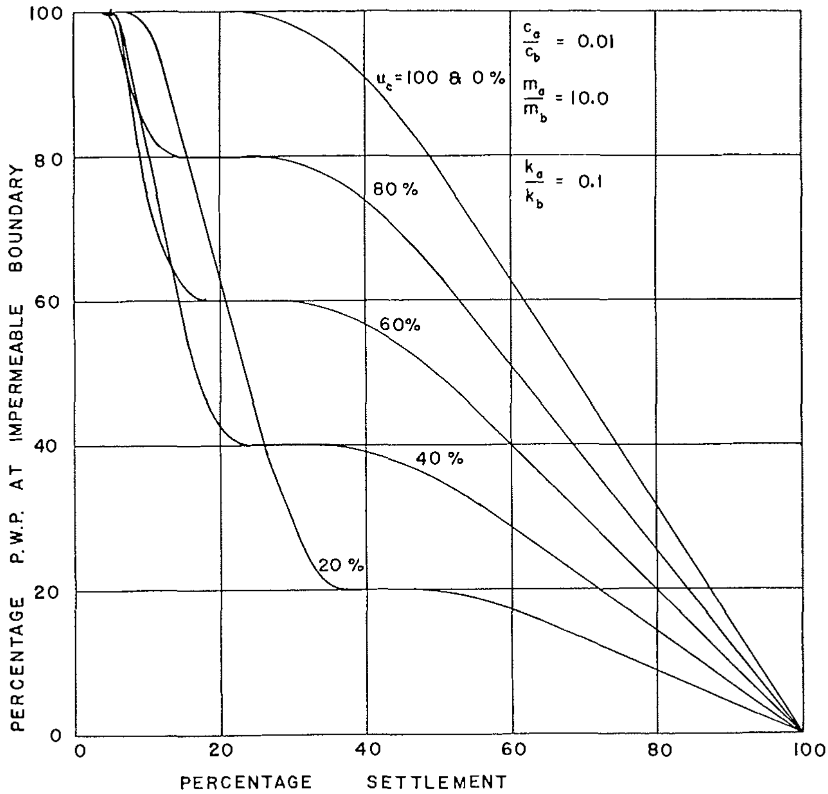


FIGURE 15. Theoretical results for the consolidation of a slightly overconsolidated soil

critical effective pressure, larger load increments would result in smaller values of the coefficient of consolidation and larger values of the modulus of volume change, provided that the resulting time-settlement or time-pore water pressure curves were analysed by means of one of the conventional curve fitting methods. The resulting value of the coefficient of permeability may increase or decrease with increasing load increment depending on the resulting values of the coefficient of consolidation or modulus of volume change. The theory, of course, does not take into account secondary consolidation.

It is the writer's opinion that the best method of obtaining the coefficient of permeability is to determine it by direct measurement.

Finally the shape of the results from the new theory may be compared with some experimental results published by Crawford (1964), Figure 16.

Acknowledgments

The experimental results performed at Queen's University by Messrs. Chan and Jarrett were sponsored by the Department of Highways of Ontario and are published with its approval.

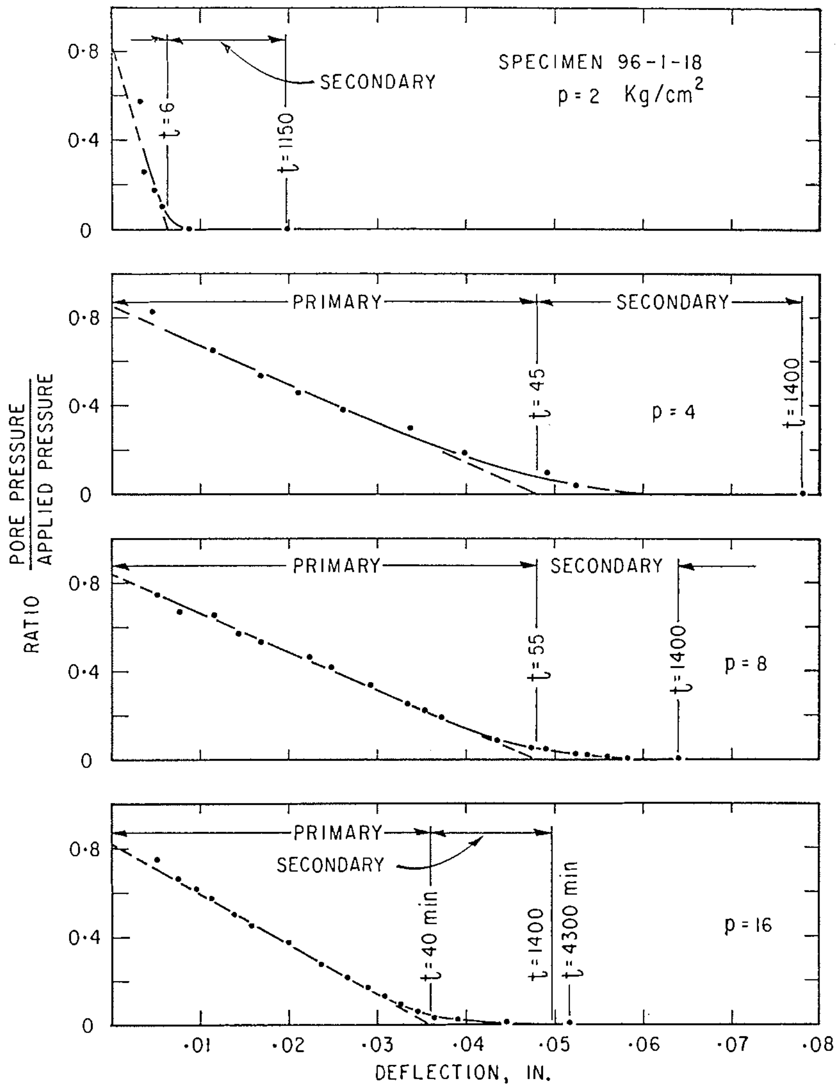


FIGURE 16. Relationship between pore water pressure and deflection under incremental loads (after Crawford, 1964)

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K. Y. Lo*

The author raised an important problem on the appropriate testing procedure for the determination of preconsolidation pressure and the compression curve for the estimation of settlement and rate of settlement. By using strain-controlled tests he has shown that both the preconsolidation load and resistance to compression are lowered with reduced rates of compression. In other words, the e log p curve is lowered and displaced to the left.

In addition to the effect of time, another factor has to be considered: viz., the effect of the pressure increment ratio or small loading increments. It has been shown by Langer (1936) that, by reducing the loading increments, the resistance to compression increases, the compression curve is displaced to the right. The effect of using a smaller pressure increment ratio is similar. As the reduction in loading increments (or the increment ratio) serves in effect to decrease the average rate of compression, these results appear to be inconsistent with the findings reported by the author. Again, in a paper by Hamilton and Crawford (1959) it was shown that for Leda clay the pressure–void ratio curves are not affected by the load increment ratio. However, the difference in the average rate found in these tests is as much as seven times. Similar results were obtained by Casagrande and Fadum (1944) who showed that the void ratio–pressure curves for two samples of Boston Blue clay were almost identical although the average rate of compression differed by fourteen times. The author's explanation of these apparently conflicting results is sought.

If the author's hypothesis as expressed in his present paper is correct, then, by complying with the extremely small field rate of strain, the preconsolidation load would be reduced to a very small value by an extrapolation of the results of his Table II. This leads to a paradoxical situation since experience shows that the settlements in sensitive normally consolidated clay are usually overestimated.

Finally, a comparison of the results from block samples and borehole samples would be of interest.

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