The uniqueness of the end-of-primary (EOP) void ratio-effective stress relationship

L'unicité de la courbe indice des vides-contrainte effective à la fin de la consolidation primaire

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SYNOPSIS The conventional approach of dividing consolidation into the primary and the secondary stages is a practical approach to settlement analysis because it is not yet possible to evaluate, by means of the existing testing methods, the fundamental compressibility parameters $(\partial e/\partial\sigma')_t$ and $(\partial e/\partial t)_{\sigma'}$ during the increase in effective vertical stress. The concept of primary settlement followed by the secondary settlement would be useful if for any clay a unique EOP void ratio-effective stress relationship existed which was independent of the duration of the primary consolidation stage. The uniqueness question was investigated by determining the EOP e-log σ' curves of thin and thick specimens of three natural soft clays. The results of this laboratory investigation and the existing reliable data in the literature support the concept of a unique EOP void ratio-effective stress relationship for any soft clay.

INTRODUCTION

One-dimensional settlement results from the compression of voids. Thus by definition

$$s = \frac{\Delta e}{1 + e_o} L \tag{1}$$

The problem is to evaluate Δe. The most general constitutive equation for one-dimensional compression is

$$\frac{de}{dt} = \left(\frac{\partial e}{\partial \sigma'}\right)_t \frac{d\sigma'}{dt} + \left(\frac{\partial e}{\partial t}\right)_{\sigma'} \tag{2}$$

The term $(\partial e/\partial\sigma')_t$ represents the compressibility of the soil structure with effective stress, and $(\partial e/\partial\sigma')_t\, d\sigma'/dt$ is the void ratio decrease with time associated with an effective stress increase. The term $(\partial e/\partial t)_{\sigma'}$ represents the compressibility of the soil structure with time, and it also represents the void ratio decrease with time following a structural disruption caused by the effective stress increase. Equation (2) can be integrated to obtain Δe in Eq. (1).

$$\Delta e = \int_0^t \left[\left(\frac{\partial e}{\partial \sigma'}\right)_t \frac{d\sigma'}{dt} + \left(\frac{\partial e}{\partial t}\right)_{\sigma'}\right] dt \tag{3}$$

Unfortunately, it is not possible to readily evaluate $(\partial e/\partial\sigma')_t$ and $(\partial e/\partial t)_{\sigma'}$ when $d\sigma'/dt \neq 0$. Equation (3) may be rewritten as

$$\Delta e = \int_0^{t_p} \left[\left(\frac{\partial e}{\partial \sigma'}\right)_t \frac{d\sigma'}{dt} + \left(\frac{\partial e}{\partial t}\right)_{\sigma'}\right] dt + \int_{t_p}^{t} \left(\frac{\partial e}{\partial t}\right)_{\sigma'} dt \tag{4}$$

The time t_p represents the period of consolidation during which $d\sigma'/dt \neq 0$. For an incremental loading in the field or in the laboratory, t_p is the duration of the primary consolidation stage. The time beyond t_p, when $d\sigma'/dt = 0$, is the secondary consolidation stage, Fig. 1. Thus, Eq. (4) computes total compression as the sum of the primary compression and the secondary compression:

$$\Delta e = (\Delta e)_p + \int_{t_p}^{t} \left(\frac{\partial e}{\partial t}\right)_{\sigma'} dt \tag{5}$$

where

$$(\Delta e)_p = \int_0^{t_p} \left[\left(\frac{\partial e}{\partial \sigma'}\right)_t \frac{d\sigma'}{dt} + \left(\frac{\partial e}{\partial t}\right)_{\sigma'}\right] dt \tag{6}$$

Rewriting Eq. (3) in the form of Eq. (5) would be useful only if it were possible to directly evaluate $(\Delta e)_p$.

A fundamental question can be raised as to whether or not the value of $(\Delta e)_p$ from a thin laboratory specimen can be directly used to compute the settlement of thick clay layers in the field. This has been an important question in soil mechanics (Leonards, 1972; Bjerrum, 1972; Ladd et al., 1977; Mesri, 1977). The fundamental difference between the consolidation condition of a laboratory specimen and a similar clay sublayer in the field is in the length of time it takes to complete the primary consolidation stage. The values of t_p for the laboratory specimens are in hours or at most in days, whereas in the field situations, t_p often exceeds many years. If a model of soil structure is assumed in which separate and independent mechanisms control $(\partial e/\partial\sigma')_t$ and $(\partial e/\partial t)_{\sigma'}$, then the conclusion from Eq. (6) inevitably follows that the longer the duration of the primary consolidation stage, the larger is $(\Delta e)_p$. In other words, during a larger t_p, more contribution is made by $\int_0^{t_p} (\partial e/\partial t)_{\sigma'}\, dt$ to compression. However, an interrelationship between $(\partial e/\partial\sigma')_t$ and $(\partial e/\partial t)_{\sigma'}$ could produce a value of $(\Delta e)_p$ independent of the value of t_p. The interrelationship between $(\partial e/\partial t)_{\sigma'}$ and $(\partial e/\partial\sigma')_t$ in terms of C_α and C_c has been well established during the secondary consolidation stage (Mesri and Godlewski, 1977, 1979) and such an interrelationship during the primary consolidation stage appears to be most likely (Mesri, 1977). In the absence of data on $(\partial e/\partial\sigma')_t$ and $(\partial e/\partial t)_{\sigma'}$ during the primary consolidation stage, the most direct approach is to compare the end-of-primary (EOP) void ratio-effective stress relationships of thin and thick layers. In a state-of-the-art report Ladd et al. (1977) concluded that little definitive data existed on this important question.

EXISTING EXPERIMENTAL EVIDENCE

The uniqueness of the EOP void ratio-effective stress relationship has been previously investigated by comparing the behavior of either laboratory specimens of different thicknesses or laboratory specimens to field layers under the same loading. Hanrahan (1954) compared the one-dimensional compression of 0.75 and 5.8 in. thick samples under the same load increment, and found little difference in the EOP strain. Schroeder and Wilson (1962) performed consolidation tests on peat samples of different thicknesses. A comparison of the reported e-log σ' curves is complicated by the wide range of initial void ratios of the specimens (8-32). Wilson (1963) applied a single load increment (0-0.275 kg/cm^2) to two samples of amorphous granular peat (L_o = 9.3 and 16.3 cm) that had the same initial void ratio of 14, and found that the EOP strain of the thicker sample was 7 % less than that of the thinner sample. Akai (1963) loaded three "undisturbed" samples of a silty clay (L_o = 0.5, 1 and 2 cm) from 3.2 to 6.4 kg/cm^2 and concluded that the EOP strain increased with specimen thickness. Akai's conclusion is questionable because of the unknown contribution of sample disturbance to the behavior of very thin samples. Also, when the uniqueness question is investigated by means of a single pressure increment, there is a serious potential for misleading results. This point will be explained in detail later. Lee and Brawner (1963) compared EOP $\Delta L/L_o$ of laboratory (L_o = 0.75 to 1.0 in.) to field layers (L_o = 11.5 to 31 ft) for six amorphous and fibrous peats. Although there is not a consistent trend, the small difference between the $\Delta L/L_o$ of thin laboratory and thick field layers suggests EOP strain independent of the duration of the primary consolidation. Adam (1965) compared the EOP compression of a peat in the laboratory and in the field. Adam's data showed a unique $\Delta L/L_o$ - log σ' relationship that was independent of peat thickness. Berre (1969) performed oedometer tests on undisturbed normally consolidated clay with two different initial thicknesses (L_o = 0.75 and 3.0 in.). At the consolidation pressure of 20 psi the EOP strain was 5.1 % and 5.4 % for the thin and thick samples, respectively. Barden (1969) loaded two samples of remolded fibrous peat (L_o = 0.37 and 5.5 in.) from a slurry consistency to a pressure of 3 psi. The EOP strain was 6 % higher for the thicker sample. The previous criticism with respect to sample disturbance is also true for Barden's 0.37 in. sample. Berre and Iversen (1972) performed a series of special consolidation tests on undisturbed specimens of the Drammen clay with four different initial thicknesses ranging from 0.74 to 17.7 in. To obtain the 17.7 in. specimen, three 5.9 in. specimens were connected in series. After an examination of the data, Mesri (1977) stated that the results were inconclusive. Samson and LaRochelle (1972) compared laboratory and field EOP $\Delta L/L_o$ values for a peat. Most of the field strains plotted below the laboratory $\Delta L/L_o$ - log σ' curve. They suspected, however, that the field strains included lateral deformation. Aboshi (1973) applied a single pressure increment from 0.2 to 0.8 kg/cm^2 to reconstituted marine clay samples of thicknesses ranging from 2 to 100 cm. According to Aboshi's interpretation of the data, final primary strain ranged from 8.2 % (L_o = 2 cm) to 8.9 % (L_o = 100 cm). The authors' interpretation suggests EOP strain independent of sample thickness. The serious concern with respect to the single-increment tests, which was expressed earlier, also applies to Aboshi's results. Lefebvre et al. (1979) compared, for a sensitive clay, the void ratio predicted from laboratory tests to the average value observed in the field under the same loading condition. The field void ratio near the completion of excess pore pressure dissipation agreed quite well with the laboratory value taken from the 24-hour e-log σ'_v curve.

When the question of the uniqueness of the EOP void ratio-effective stress relationship is investigated by using a single pressure increment on thin and thick samples, there is a serious potential for reaching a misleading conclusion. The problem is illustrated in Fig. 2. In preparing to compare the behavior of a thin and a thick sample under a pressure increment from σ'_j to σ'_{j+1}, if the samples are allowed an arbitrary period of secondary compression under σ'_j, then the compression versus time curves of the two samples for the pressure increment from σ'_j to σ'_{j+1} can not be directly compared.

Unless the investigator is aware of this problem and the duration of consolidation under σ'_j is carefully controlled, there is a strong possibility that the thin sample will experience larger secondary compression under σ'_j than the thick sample. The reason is the much shorter duration of the primary consolidation stage for the thin sample. During a consolidation period of days or weeks, the thick sample may just barely complete the primary consolidation stage, whereas the thin sample, in addition, may experience significant secondary compression. Even though Fig. 2 was based on the assumption of a unique EOP e-log σ' relationship, the thick sample is expected to compress more, from point 1 to 3, than the thin sample which compresses from point 2 to 3. A comparison of the d-log t curves would then lead to the misleading conclusion that the thick sample experiences more primary compression than the thin sample. Without an e-log σ' plot it may not be realized that the two samples were not identical at σ'_j.

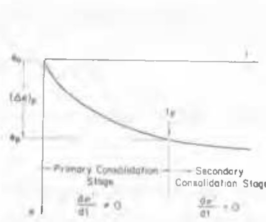

Fig. 1 Consolidation Stages

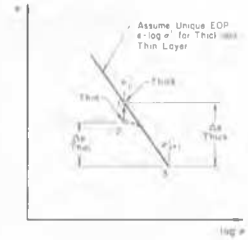

Fig. 2 Single Increment Test

TESTING PROGRAM

After a review of the existing data in the literature, it was decided in 1976 to initiate long-term research on the question of the uniqueness of the EOP void ratio-effective stress relationship. A laboratory testing program was set up to determine the EOP e-log σ' curves of thin and thick specimens. The main guidelines were to test undisturbed samples of natural soft clays and for the thick sample to use a maximum drainage distance of 1/2 m. These requirements together could not be met for the one-dimensional consolidation test. In order to minimize ring friction, a 1/2 m-thick specimen requires a diameter of at least 1 1/2 m. It was not possible to obtain natural soft clay samples of this large size. It was decided to perform consolidation tests on cylindrical specimens which are subjected to equal all-round pressure. The compression is three-dimensional, whereas the drainage is one-dimensional in the axial direction. This method of loading avoids the ring friction and lateral stress variables, and it is possible to maintain a constant total stress condition over a period of years.

During a period of 8 years three series of tests have been completed using undisturbed samples of three natural soft clays (Table 1). The undisturbed samples of Saint Alban and Louiseville clays were taken with the 20 cm-diameter Laval sampler (LaRochelle et al., 1981). Therefore it was possible to cut 1 1/2-, 2 1/8- and 2 1/2 in.-diameter samples side by side from single blocks. The apparatus and testing procedure have been described in detail by Choi (1982). The tests were performed either in special triaxial cells for 1 1/2- and 2 1/8-in.-diameter specimens, or in a specially designed pressure cell which encloses four 2 1/2 in.-diameter specimens. Each specimen was enclosed in a rubber membrane sealed to the pedestal and top cap with O-rings, and it was surrounded by mercury to prevent transfer of fluid through the membrane and end seals. Although, depending on the availability of samples, tests with maximum drainage distances of 1, 2, 3, 5 and 20 in. have been performed, the 5- and 20-in. tests form the main series and are the most comparable because the 20-in. tests were obtained by connecting four identical 5-in. specimens in series, Fig. 3. The 5-in. specimens in the 5- and 20-in. tests, with a L/D of 2, were subjected to identical degree of end constraint.

TABLE 1 Natural Clays Used in the Investigation

Natural Clay	w_o (%)	w_L (%)	w_P (%)	k_o (cm/sec)	C_k	C_α/C_c	σ'_p/σ'_{vo}	$\frac{[\sigma'_p]_{3-D}}{[\sigma'_p]_{1-D}}$
Saint-Alban	39-57	31	18	3×10^{-7}	0.51	0.024	1.9-2.7	0.65
Bay Mud	86-98	89	37	3×10^{-7}	0.77	0.05	1.2	0.71
Louiseville	64-71	65	28	6×10^{-8}	0.87	0.03	2.6-2.9	0.74

In all of the tests Tetko Polyester screens (HD-7-6) rather than filter paper, separated the specimens from the end porous disc. This procedure avoids the deterioration and interaction of the filter papers with the soil specimen. The pore water pressure was measured at the bottom and drainage was allowed from the top of the specimens. For the 20-in. tests pore water pressure was measured at the bottom of all four specimens and drainage was allowed from the top of the first specimen in the series. The pore water pressure was measured with electrical pressure transducers. All tests were performed inside constant-temperature water baths at 26 ± 0.3°C.

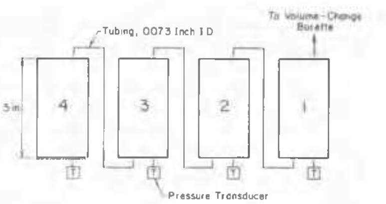

Fig. 3 20 in. Test Obtained by Connecting Four 5 in. Specimens in Series

TEST RESULTS

Each test was set up and back-pressured by 40 psi. Then, the all-round pressure was increased in increments and consolidation was observed in terms of volume change and pore water pressure measurements. The end of primary consolidation was defined at 98 % excess pore water pressure dissipation, corresponding to 0.1 to 0.2 psi excess pore water pressure, at the impervious boundary. Only at selected pressures was secondary compression measured.

The Saint Alban clay series, consisting of one 5-in. and one 20-in. test, required 16 months. The duration of the primary consolidation stage for the 5- and 20-in. tests was in the range of 1 to 12 days and 1/2 to 2 months, respectively. The excess pore water pressure isochrones of the 20-in. test, for one pressure increment, are shown in Fig. 4a. The EOP e-log σ' curves are compared in Fig. 5a.

The San Francisco Bay Mud series, consisting of three tests, required 29 months. The duration of the primary consolidation stage for the 3-, 5- and 20-in. tests was in the range of 1 to 5 days, 8 to 20 days and 1 1/2 to 6 months, respectively. The excess pore water pressure isochrones for one pressure increment of the 20-in. test are shown in Fig. 4b. The EOP e-log σ' curves are compared in Fig. 5b.

The Louiseville clay tests included four pressure increments completely within the recompression range. Therefore, it was possible to observe the excess pore water pressure response upon loading and the very rapid pore pressure dissipation for the pressure increments in the recompression range. An example is shown in Figs. 4c and 6a. For a soft clay layer with a maximum drainage distance of 20 in. and initial coefficient of permeability, k_{vo}, of 0.6×10^{-7} cm/sec, because of very small compressibility, primary consolidation is completed in about a week. In contrast, Figs. 4d and 6b show excess pore water pressure behavior for the pressure increment spanning the preconsolidation pressure. The behavior in Fig. 6b reflects small compressibility in recompression changing to large compressibility in the compression range. In contrast to the 7-day increment duration in the recompression range, the duration of the primary consolidation for this pressure increment is 10 1/2 months. The primary consolidation stage required 1/2 hour to 1 1/2 days, 1 hour to 8 days, 10 hours to 28 days and 4 days to one year for the 1-, 2-, 5-, and 20-in. tests, respectively. This series required 28 months. The EOP e-log σ' curves are compared in Fig. 5c.

For the three natural soft clays, if only the 5- and 20-in. tests are compared, one would conclude that the EOP e-log σ' relationship is independent of the duration of primary consolidation stage. The 3-in. test on San Francisco Bay Mud also supports this conclusion. The 1- and 2-in. Louiseville tests, however, suggest a trend of increasing preconsolidation pressure with the decrease in sample thickness. This behavior develops in the recompression range where both $(\partial e/\partial t)_{\sigma'}$ and t_p are small, and it disappears in the compression range where they are large. Although a more definitive conclusion has to await the results of tests on other soft clays, it is believed that the observed behavior of the 1- and 2-in. samples mostly reflect a testing problem. The explanation is suggested in Fig. 5d which compares the e-log σ' curve of the 20 in. test to that of a one-dimensional consolidation test on a 3/4 in. sample. Since natural soft clays have been formed under a one-dimensional compression condition, they show more resistance to one-dimensional than isotropic loading. The ratios of the isotropic and one-dimensional preconsolidation pressures of the three clays, in Table 1, are within the previously observed range for soft clays. Thus, it is speculated that since the values of L/D for the 1- and 2-in. tests were 2/3 and 4/3, respectively, the end constraint produced a loading condition somewhere in between isotropic and one-dimensional.

CONCLUSIONS

The results of the present laboratory investigation and the existing reliable data in the literature support the concept of a unique EOP e-log σ' curve for any soft clay. Additional indirect support is provided by the agreement between the values of preconsolidation pressure from the oedometer test and those obtained by the interpretation of

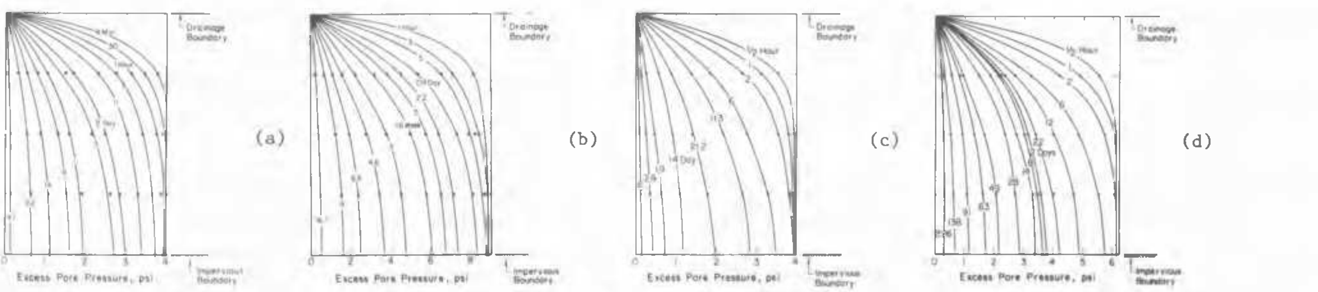

Fig. 4 (a) Saint Alban Clay, 4.1 to 8.1 psi; (b) San Francisco Bay Mud, 20.1 to 29.1 psi; (c) Louiseville Clay, 4 to 8 psi; (d) Louiseville Clay, 16 to 22 psi.

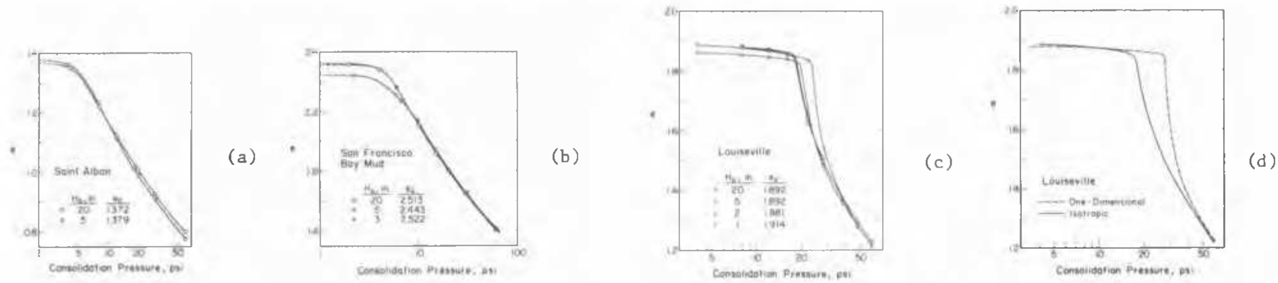

Fig. 5 EOP e-log σ' Curves

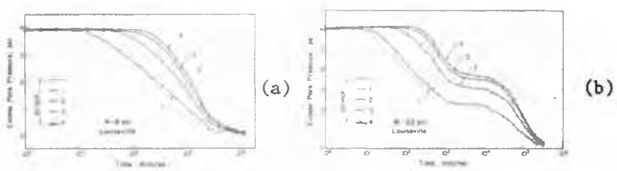

Fig. 6 Pressure Increment (a) in the Recompression Range (b) Spanning the Preconsolidation Pressure

settlement and pore water pressure observations in the field (Sallfors, 1975; Samson et al., 1981). The uniqueness of the EOP e-log σ' relationship is a very powerful concept which allows us to develop rigorous and practical methods for settlement analysis of foundations and embankments on soft clays.

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