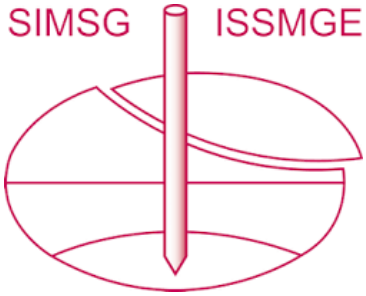
INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING




*This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:*

*https://www.issmge.org/publications/online-library*

*This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.*

Id 6

# TIME EFFECT IN CONNECTION WITH CONSOLIDATION TESTS

R. HAEFELI
W. SCHAAD
Laboratory for Hydraulic Research and Soil Mechanics of the
Swiss Federal Institute of Technology, Zurich

## I. DEFINITION OF THE PROBLEM.

In the case of natural settlement phenomena, often extending over centuries, the engineer is faced with the problem of predicting the final values of such settlements with sufficient accuracy on the basis of relatively brief laboratory tests. For carrying out this task he must correctly judge the compressibility of the respective soil layers. If the layers are of recent date and are to be loaded heavily for the first time, an exact knowledge of the coefficients of compression $\alpha_v$ of the loose sediment in question will be sufficient for this purpose 1), 2). With homogeneous materials, the total settlement is known to be directly proportional to the coefficient of compression.

When determining the coefficients of compressibility in the laboratoty, the problem consequently consists in obtaining accurate values in a short a time as possible. At first it was thought that this aim could only be attained by means of oedometer tests lasting for a relatively long time. Tests carried out in the Laboratory for Hydraulic Research and Soil Mechanics at the Swiss Federal Institute of Technology during several years have, however, shown that this is not absolutely necessary, since accurate results can be obtained even with brief oedometer tests, if certain rules in connection with them are observed.

In order to explain the results of the tests, some of them rather surprising, the different stages of the settlement phenomenon is first of all theoretically considered in connection with the determination of $\alpha_v$. In connection with this, some new points of view were also obtained for forming a judgement on the post-settlement.

## II. THEORETICAL CONSIDERATIONS.

If we study the progress of the settlement in the course of time of a loose sediment prevented from lateral expansion, a difference can at first be made between principal or main and post-settlement 2). Strictly speaking, however, the principal or main settlement can again be divided into two processes, so that we have the following divisions (a, b and c) into three parts:

1. Main settlement
   - a) Short settlement (elastic)
   - b) Hydrodynamic settlement.
2. Post-settlement — c) Settlement in consequence of metamorphosis.

In Fig. 1 this three-part process is diagrammatically represented for the case of a loam or clay saturated with water, the time being plotted to logarithmic scale.

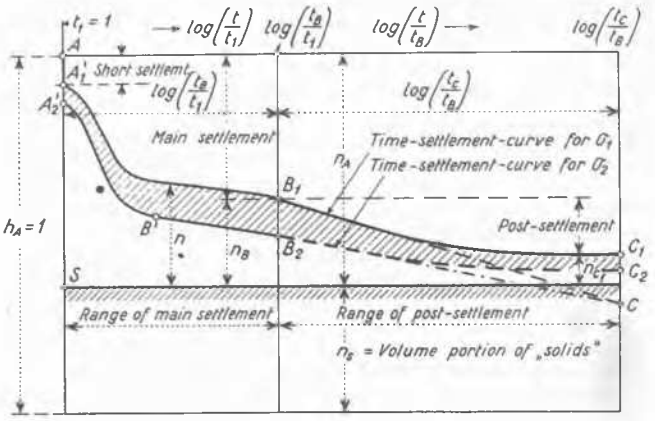
Regarding the three phases of the settlement process the following may in general be stated:

a) The short-settlement, which occurs with sudden loading, is very difficult to express technically by means of measurements. In loams and clays saturated with water it is exclusively determined by the elastic compressibility of the solid and liquid phases, above all by the latter. In comparison with the plastic settlement portions considered under b) and c), this elastic yielding is naturally very small and is therefore as a rule neglected. With loose sediments which are very permeable or not completely saturated (sand, snow), sudden loading is on the other hand often accompagnied by an instantaneous change of structure (breack-down of structure). In this case the short settlement may become quantitatively the main part of the total settlement.

b) The hydrodynamic settlement is conditioned by the unsteady seepage and the gradual relaxing in tension of the pore water with corresponding compacting of the material. This process, which takes place at a gradually diminishing rate, has been closely studied by Terzaghi and Fröhlich, and has been expressed mathematically for certain boundry conditions 4), 5). The time required for its accomplishment was then found to be dependent on the following factors: Drainage conditions, thickness of layers, permeability and compressibility of the material.

c) The post-settlement cannot, in contrast to the main settlement, be explained by mechanical processes. It is in the first line conditioned by colloidochemical processes, which may finally lead in the course of geological epochs over early metamorphosis to the conversion of a clay into a clayslate, in so far as the necessary temperatures of that are available. With constant pressure and a constant temperature lying between freezing and boiling point, the compacting process taking place at a gradually diminishing rate, must finally come to a stillstand before the porous-free state is reached. As an analogous phenomenon we mention that also snow under the application of relatively low pressures and temperatures (under 0° C) may, even after a very long time, not be compacted into ice free from pores 6), 7). Glacier ice originates above all from the collaboration of the liquid phase which forces its way into the pores of the Firn (perpetual snow) and freezes there. In the high-lying Greenland inland-ice, where no melting processes occur on the Firn surface, the formation of ice can therefore occur only at very great depths (in the region of the pressure-melting point).

In order to illustrate the above-mentioned conditions in Fig. 1, the initial height of the sample to be consolidated is taken as $h_A = 1$. The horizontal base line drawn through the point S corresponds to the volume portion $n_S$ of the solid substance, whilst the distance S - A is identical with the volume portion of the initial porosity $n_A$. The upper settlement curve $A\text{-}B_1\text{-}C_1$ therefore gives at the same time the change in the porosity n, which appears as ordinate of this curve above the horizontal line through S. Since the porous-free state is never reached, the curve in the neighbourhood of the post-settlement must approach asymptotically to a horizontal line through $C_1$, whose distance from the base line S is identical with the possible final porosity $n_{C_1}$.

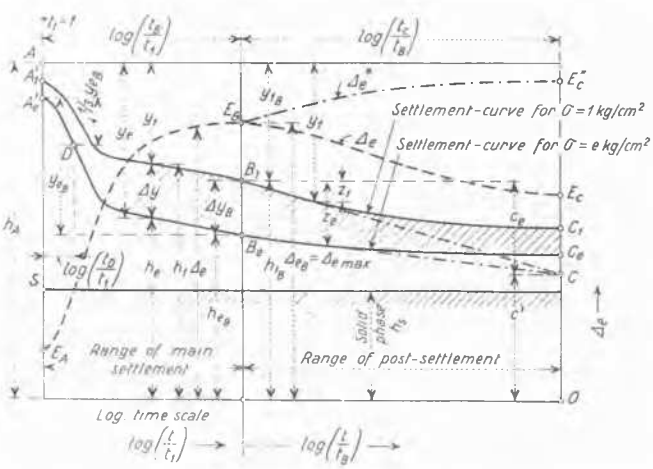



Main settlement and post-settlement.

FIG.1

In order to study the influence of the constant specific loading $\sigma$ on the settlement process, and thereby also the dependence to be sought of the coefficient of compressibility $\Delta_e$ on the duration of the test, in addition to a first settlement curve for $\sigma_1$ also a second curve A-$B_2$-C2, starting with the same initial conditions, was drawn in Fig. 1; it corresponds to a higher consolidation pressure $\sigma_2$. Thereby we were interested above all in the difference between these two settlement curves. Under the assumption that the short settlement is small in comparison with the main settlement, there is first of all no doubt that the two curves must diverge in the region of the principal settlement. Less clear, however, are the conditions in the region of the post-settlement, since a priori nothing definitely can be predicted regarding the relative course of the two lines. Under the assumption that both curves tend towards the porous-free state of the material, but without reaching it, there is a certain degree of probability for the convergence shown in Fig. 1.

To investigate the dependence of the coefficient of compressibility $\Delta_e$ on the duration of the test, the consolidating pressures of the two settlement curves to be compared have been chosen in Fig. 2 as $\sigma_1$ = 1 kg/cm$^2$, and $\sigma_2$ = e = 2,72 kg/cm$^2$.




$\Delta_e$-Values for main and post-settlement.

FIG.2 (diagrammatic)

As is known, $\Delta_e$ signifies the percentage settlement of a sample in consequence of increasing the compacting pressure from $\sigma$ = 1 to $\sigma$ = e kg/cm$^2$. Since we first of all assume that the two load increments act on the sample for exactly the same length of time, $\Delta_e$ can be determined directly from the ordinate difference $\Delta ij$ of the two curves, it holding good, that 8):

$$\Delta_e = \frac{h_1 - h_e}{h_1} = \frac{\Delta y}{h_1} \qquad (1)$$

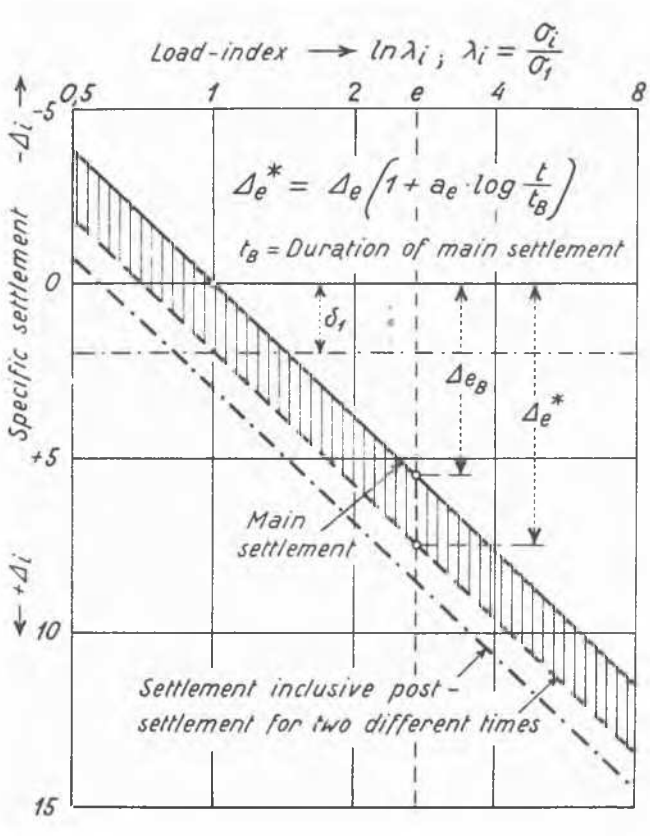
Since the two curves diverge in the region of the main settlement, and thus $\Delta ij$ becomes greater with increasing length of time of the test, whilst $h_1$ simultaneously decreases, the quotient $\Delta_e$ increase with the time, until finally the maximum value $\Delta_{e_s}$ is reached when passing over from principal settlement to post-settlement (normal value for standard test).

For the initial region of the post-settlement Buisman has shown that the settlement is proportional to the logarithm of the time, so that the settlement curve appears as a straight line when adopting the semi-logarithmic scale. A deviation from this straight line is only to be expected in a later phase of the process. Experimentally it is only possible to investigate the initial course of the post-settlement. In this way it can be specially tested whether the convergence between the two lines $B_1$ - O and $B_2$ - C illustrated in Fig. 2 really exists or not. From the ordinate difference of these two straight lines the dependence of the $\Delta_e$ value on the duration of the test is calculated as follows:

$$\Delta_e = \Delta_{e_s} \cdot \frac{\log\left(\frac{t_c}{t}\right)}{\log\left(\frac{t_c}{t_e}\right) - \frac{c_e}{h_{1B}} \cdot \log\left(\frac{t}{t_e}\right)} \qquad (2)$$

If the point of intersection C of the two straight lines is moved downwards until it reaches the point O, $\Delta_e$ becomes = $\Delta_{e_s}$, i.e. independently of the time, which also is shown from equation (2) for the special case $c_e = h_{1B}$, and also directly from Fig. 2.

In Fig. 2 the dependence hitherto considered between the $\Delta_e$ value and the duration of loading is diagrammatically illustrated by the curve $E_A$ - $E_B$ - $E_C$. The weakly pronounced maximum at $E_B$ characterises at the same time the transition from the main to the post-settlement. In the region of the main settlement the question posed at the beginning regarding the time required for determining $\Delta_e$ can be answered on the basis of the $\Delta_e$ curve. On the other hand, in the region of post-settlement, the $\Delta_e$ curve ($E_B$ - $E_C$) serves only to allow a judgement to be formed on the mutual course of the corresponding settlement curves (divergence of convergence). For replying to the practically important question as to the effect of the post-settlement in nature or in a structure, there would be no use in having a $\Delta_e$ value decreasing with the time, whose determination rests on the load increments for 1 kg/cm$^2$ or for e kg/cm$^2$ lasting for equal times. In this case the duration of the load increment for 1 kg/cm$^2$ may only be chosen equal to the duration of the main settlement $t_B$, whilst the load increment e must extend over the time of the post-settlement. The corresponding $\Delta_e'$ value may then be calculated as follows, according to Fig. 2, the specific post-settlement $\delta e'$ occurring during one year being adopted as magnitude for comparison.




Specific settlement as a function of load for main and post-settlement.

FIG.3

$$\Delta_e^* = \frac{z_e}{h_{i_B}} = \Delta_{e_B} \cdot \left[1 + \frac{\delta e'}{\Delta e_B} \cdot \frac{\log t - \log t_B}{\log 365 - \log t_B}\right] ; \quad \delta e' = \frac{z_e'}{h_{i_B}} \qquad (3)$$

This equation for the increase in $\Delta_e^*$ holds good of course only during the first phase of the post-settlement, i.e. in the region where the logarithmic settlement straight lines are guilty. The corresponding curve is shown in Fig. 2. by the line $E_B - E_C'$. In Fig. 3 the post-settlement is plotted as a function of the compacting pressure. The straight full line, shows the known dependence of the specific main settlement $\Delta_1$ on the consolidating pressure $\sigma_1$, which is as follows:

$$\Delta_1 = \frac{h_1 - h_i}{h_i} = \Delta_e \cdot ln\lambda_1 ; \quad \lambda_1 = \frac{\sigma_i}{\sigma_1} \qquad (4)$$

If for each load increment the increase of $\Delta_1$, in consequence of an post-settlement limited to a certain time is plotted, the dotted line displaced downwards is obtained, which cuts out the above-described $\Delta_e^*$ value on the vertical through e. Depending on the mutual course of the corresponding settlement curves (Fig. 5), the two upper lines run parallel or not. Further, the longer the post-settlement lasts, the further the dotted line moves downwards, as shown by the chain-dotted line.

Hitherto we have tacitly assumed that the duration of the main settlement $t_B$, or the beginning of the post-settlement, is independent of the magnitude of the consolidating pressure; this is only permissible in a first approximation. Strictly speaking this is not the case, since the higher pressure at once causes greater consolidation and this leads consequently to a considerable decrease in permeability.

## III. METHODS OF CARRYING-OUT AND RESULTS OF TESTS.

The main tests, extending over a period of three years, were carried out during the war (1939-1941), the complementary tests on the other hand only in 1947 9).

### 1. Test materials.

In the main tests the initial material adopted was an elutriated Kaolinite-clay from Zettlitz, analysis of which showed 98.4 % clay substance, 1,2 % quartz and 0,4 % feldspar. In 1938 Dr. von Moos analysed a Zettlitz Kaolinite-clay with the following results:

Grain fraction 0,02 - 0,005 mm = 13 %
0,005 - 0,002 mm = 28 %
less than - 0.002 mm = 59 %

In the Agricultural-Chemical Institute of the Swiss Federal Institute of Technology, which is under the direction of Prof. Pallmann, the initial material was neatralised with various ions and H-Kaolinite-clay, K-Kaolinite-clay and Ca-Kaolinite-clay were thus produced 8). The plastic behaviour of these materials can be seen from table 1.

TABLE I

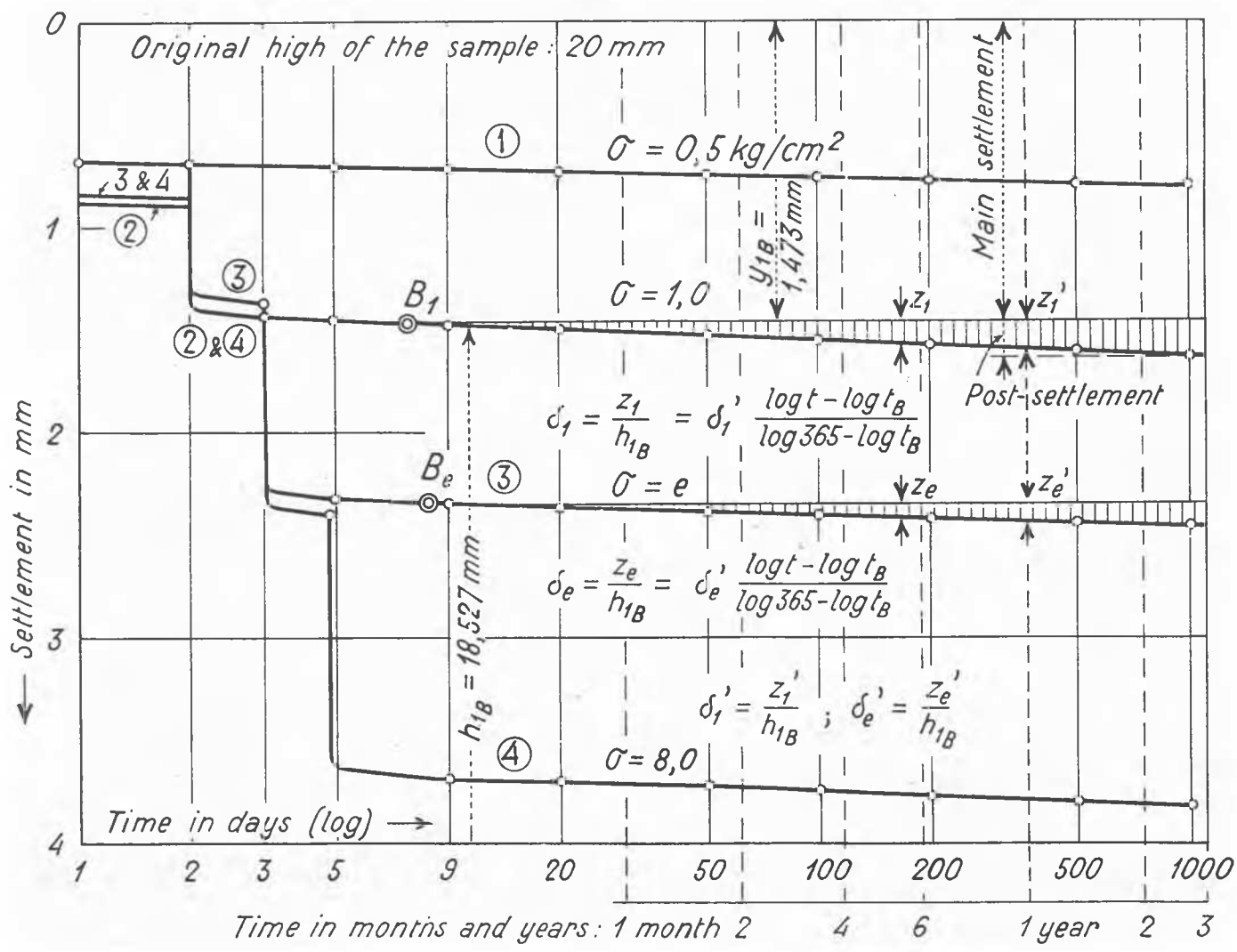
Flow limits and coefficients of compression of the Kaolinite-clays

| Materials | Kaolinite-clay | K-Kaolinite-clay | Ca-Kaolinite-clay | H-Kaolinite-clay |
|---|---|---|---|---|
| Flow limit % | 61,0 | 53,3 | 65,7 | 87,0 |
| Plastic limit % | 33,3 | 38,6 | 34,3 | 33,1 |
| Coefficient of plasticity % | 27,7 | 14,7 | 31,4 | 53,9 |
| e values % | - | 6,23 | 7,04 | 5,07 |

The complementary tests were carried-out with the same clay as was used to form the watertight core of the Bannalp dam. Its characteristics are given in table 2 10).

### 2. Preparing the material.

1 kg of each of the three kinds of Kaolinite-clay was worked up to a paste in vacuum with distilled water. By being loaded in a glass cylinder, this paste was brought to the consistency suitable for filling the oedometer free from air. The water contents were then as shown in table 3.




Main and post-settlement of H-Kaolinite-clay observed during three years (four load increments: σ = 0,5 - 1,0 - 2,72 and 8,0 kg/cm² )

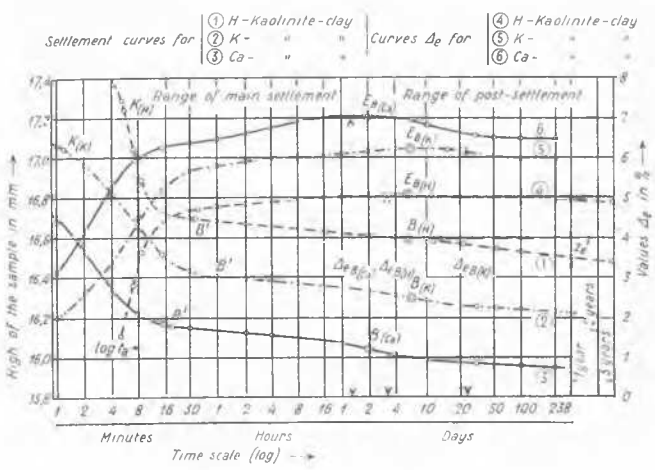
FIG.4

TABLE 2

| Characteristics of the Bannalp clay | | | |
|---|---|---|---|
| Flow limit f in % | | | 76,6 |
| Plastic limit a in % | | | 34,7 |
| Coefficient of plasticity p = f - a % | | | 41,9 |
| Coefficient of compressibility $\Delta_e$ % | | | 8,8 - 9,0 |
| Coefficient of permeability $k_{100}$ cm/sec | | | $3,7.10^{-8}$ |
| Humus content % | | | 1,5 |
| Grain distribution mm | 0,2 - 0,02 | 0,02-0,002 | 0,002 |
| Percentage weight | 18,3 | 50,2 | 31,5 |

TABLE 3

| Water contents in % for different states of consolidation | | | |
|---|---|---|---|
| Material | K-Kaolinite-clay | Ca-Kaolinite-clay | H-Kaolinite-clay |
| Initial water content $w_A$ | 56,25 | 56,20 | 47,14 |
| $w_1$ (for σ = 1 kg/cm²) | 44,15 | 43,10 | 40.15-41,4 |




Time-settlement curves and $\Delta_e$-values of three Kaolinite-clays for main and post-settlement.

## FIG.5

The Bannalp clay was prepared in the normal manner usual for standard tests and filled into the oedometer with a water content corresponding to the Atterberg flow limit.

3. Carrying-out the tests.

Each time when carrying-out the main tests, four different load increments were investigated in parallel at compacting pressures of 0,5 - 1,0 - 2,72 and 8 kg/cm$^2$. For this purpose eight oedometers were filled with H-Kaolinite-clay, four of these samples being subjected to the consolidation test for three years, whilst the settlement of the other samples was studied only during 7 - 8 months. The complementary tests with Bannalp clay were carried out in two series; the methods then adopted will be described later in connection with the test results.

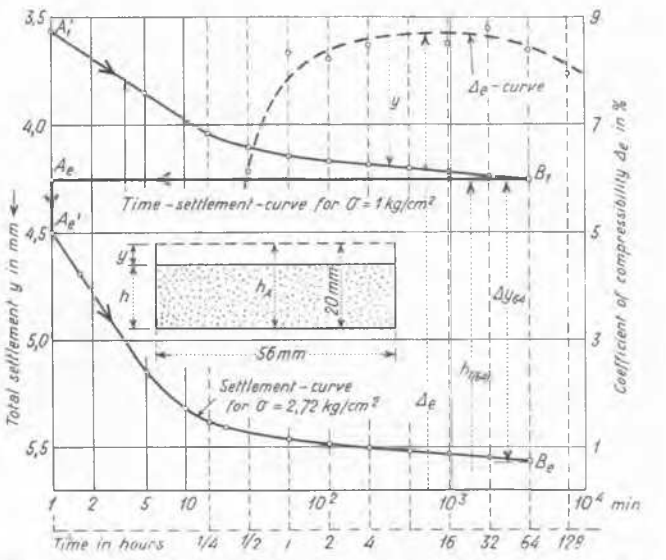
4. Results of the tests.

The settlement curves in Fig. 5 first of all confirm that there is no sharp boundry to be perceived between main-settlement and post-settlement. The duration of the main settlement can therefore not be measured directly. On the other hand, in all settlement curves in Fig. 4, 5 and 6, the abrupt bend at B' is very striking. It lies well within the region of main settlement, and corresponds to that moment of time at which the main settlement is to a great extent completed. The remaining part of the settlement takes place extremely slowly and theoretically requires an infinitely long time. In the diagram the rest of the main settlement, in percentage small but in time very great, is represented by the distance B' - B.

According to Terzaghi and Buisman the time of the main settlement amounts to :

$$t_e = 0,1432 \cdot \frac{a}{k} \cdot h^2 \qquad (5)$$

where h = the height of the sample, a the coefficient of compression and k the coefficient of permeability. Applied to the H-Kaolinite-clay, equation (5) gives the order of magnitude of a few days, consideration being paid to the existing uncertainty of the value of k. It will be observed that approximately after the same time - namely after about 6 days - the coefficient of compression $\Delta_e$ passes through its maximum value. For the sake of simplicity, in our diagrammatic representation in Fig. 2, we have




Time-settlement-curve and $\Delta_e$-values of Bannalp clay.

## FIG.6

assumed that the time of maximum $\Delta_e$ value ($E_B$) is identical with the division B between main and post-settlement.

In Fig. 4 the settlement curves are given of four different load increments for H-Kaolinite-clay in the range of the three years after settlement. The straight-line course of all four lines first of all confirms that, during the first phase of the post-compacting the additional settlement z is directly proportional to the logarithm of the pressure, as already determined by Buisman. The mutual course of the logarithmic straight-lines, however, shows no uniformity; whilst the lines for $\sigma$ = 1,0 and $\sigma$ = e kg/cm$^2$ clearly converge with increasing time, there is a slight divergence between the settlement straight-lines for $\sigma$ = 1 and $\sigma$ = 0,5 kg/cm$^2$. On the other hand, since the straight lines for $\sigma$ = 1 and $\sigma$ = 8 kg/cm$^2$ run practically parallel, all three possibilities are here represented (convergency, divergency and parallelism).

Fig. 5 shows on the one hand the three settlement curves for $\sigma$ = e = 2,72 kg/cm$^2$ in the range of the main and post-settlements of the three Kaolinite-clays, and on the other hand the corresponding development of the $\Delta_e$ values as a function of the time. In all three $\Delta_e$ curves the maximum ($E_B$) reached after 2 - 6 days can be clearly recognised. This occurrence is, as has already been shown in section II, conditioned by the mutual run of the settlement curves, - divergency in the region of the main settlement and convergency in the region of the post-settlement. As previously mentioned, the moment at which the maximum occurs is temporarily regarded as the boundary between main and post-settlement.

The occurrence of a maximum in the $\Delta_e$ curve could also be determined during the first series of tests with the Bannalp clay. The considerably greater scattering of the $\Delta_e$ values is conditioned by the special kind of testing method adopted here. Whilst in the tests hitherto discussed the $\Delta_e$-values were determined on the basis of two parallelly conducted loading tests with 1 and e kg/cm$^2$ respectively, in the case of the Bannalp clay only one test was made at first to determine each $\Delta_e$ value. In Fig. 6 the corresponding settlement curves are

given for a duration of loading of 64 hours per load increment.

In a second series of tests with Bannalp clay, as in the case of Kaolinite-clay, two parallel tests were carried out. After the samples had been preconsolidated at 0,5 kg/cm$^2$, one of them was loaded to $\sigma$ = 1 kg/cm$^2$ and the other at the same time to $\sigma$ = e kg/cm$^2$. The two settlement curves run practically parallel, as is shown by the constancy of the $\Delta h$ and $\Delta_e$ values (cf. table 4).

TABLE 4

$\Delta_e$ values for Bannalp clay as a function of time

(For the same duration of loading of the load increments $\sigma$ = 1 or e kg/cm2 respectively).

| Duration of loading in hours | 0,5 | 1 | 2 | 4 | 8 | 16 | 32 | 128 |
|---|---|---|---|---|---|---|---|---|
| Sample height $h_1$ in mm | 16,056 | 16,022 | 16,005 | 15,985 | 15,970 | 15,953 | 15,933 | 15,888 |
| Sample height $h_e$ in mm | 14,618 | 14,590 | 14,567 | 14,550 | 14,538 | 14,529 | 14,504 | 14,463 |
| $\Delta h = h_1 - h_e$ cm | 1,438 | 1,432 | 1,438 | 1,435 | 1,432 | 1,434 | 1,429 | 1,425 |
| $\Delta_e = \frac{\Delta h}{\Delta h_1}$ in % | 8,95 | 8,94 | 8,97 | 8,98 | 8,97 | 8,93 | 8,96 | 8,97 |

No maximum could be perceived here. Apart from the slightness of the scattering it is above all interesting from the technical point of view of testing that already a duration of loading of half an hour was sufficient to give the exact $\Delta_e$ values. For carrying-out the above-mentioned tests with Bannalp clay we are indebted to Mr. G. Amberg, engineer.

For the quantitative judging of the post-settlement it will be found preferable to calculate the percentage increase in settlement $\delta'$ in the course of the first year after the main settlement. The long-time tests with kaolinite-clay gave the following figures in this connection (cf. Fig. 4 and 5).

ations considered above lead to the following conclusions:

1) In the main settlement, a fundamental distinction may be made between short settlement and hydrodynamic settlement. In those cases where short settlement cannot be neglected, the above-mentioned distinction is the refore justified, since the two settlement parts stress the structure in quite different manners.
2) There is no sharp boundary between main and post-settlement. In spite of that, it is important to separate them, since they are subjected to different laws. The logarithmic straight line determined by Buisman was confirmed in the range of a three-years post-settlement.
3) The coefficient of compressibility $\Delta_e$, expressed as a function of the time, is just as suitable for judging the main and the post-settlement. For comprehending the main settlement, use is made as previously of the maximum value $\Delta_{e_B}$, - in the standard test 2), 8) briefly designated $\Delta_e$, - which is reached during the transition from main to post-settlement.
4) Through the long-duration tests with four different kinds of clay it was proved that sufficiently accurate $\Delta_e$ values for judging the main settlement could be obtained with short-time oedometer tests. While, as previously, one day per loading increment is to be recommended in the standard test, so that for a $\Delta_e$ measuring corresponding to the three load increments of 0,5, 1 and e kg/cm$^2$ at least three days are required, the $\Delta_e$ value in urgent cases may be determined with suffucient accuracy with a load increment of one hour for each load. On the other hand, for testing the post-settlement, long-time tests are for the moment still essential.
5) Since as a rule one must be satisfied with determining the $\Delta_e$ value decisive for the

TABLE 5

Post-settlement in the course of one year.

| Material | $h_{1B}$ mm | $\delta'_1 = \frac{z'_1}{h_{1B}}$ % | $\delta'_2 = \frac{z'_2}{h_{1B}}$ % | $\Delta_{e_B}$ % | $\frac{\delta'_2}{\Delta_{e_B}}$ % |
|---|---|---|---|---|---|
| K-Kaolinite-clay | 17,215 | 0,865 | 0,514 | 6,23 | 8,2 |
| Ca-Kaolinite-clay | 17,377 | 0,805 | 0,640 | 7,04 | 9,1 |
| H-Kaolinite-clay | 18,527 | 0,635 | 0,474 | 5,07 | 9,4 |

The percentage increase due to post-settlement, given in the last column of the coefficients of compressibility vary only slightly during the one-year post-settlement. For H-Kaolinite.clay $\Delta_e^+$ value is calculated from equation (3) as follows:

$$\Delta_e^+ = \Delta_{e_B} \left[ 1 + 0{,}00\,94 \cdot \frac{\log t - \log 6}{\log 365 - \log 6} \right]$$

For 10 years, for instance, the percentage increase of $\Delta_e^+$ in consequence of post-settlement would amount to about 2 x 5,4 = 18,8 %.

IV. CONCLUSIONS.

The theoretical and experimental investig-

principal settlement, the facts mentioned above under 4, make it possible essentially to reduce the laboratory equipment, since the number of oedometers required may be diminished proportionally to the reduction in the duration of the tests.

BIBLIOGRAPHY.

1) E. Meyer-Peter: Berechnung der Setzung von Bauwerken. Schweiz. Bauzeitung, Bd. 112 H.6, 1938.
2) R. Haefeli: Mechanische Eigenschaften von Lockergesteinen. Schweiz. Bauzeitung Bd. 111 H. 24 und 26 1938.
3) Haefeli-Schaad : Setzungen infolge Senkung, Schwankung und Strömung des Grundwassers. Schweiz. Bauzeitung Bd. 123, H. 23, 24 und 25, 1944. (Mitteilung Nr. 8 der Versuchsanstalt für Wasserbau, E.T.H., Zürich)
4) K. v. Terzaghi: Erdbaumechanik auf bodenphysikalischer Grundlage. Wien 1935.
5) Terzaghi-Fröhlich: Theorie der Setzung von Tonschichten. Leipzig und Wien 1936.
6) R. Haefeli: Schneemechanik mit Hinweisen auf die Erdbaumechanik. Promotionsarbeit der Eidg. Techn. Hochschule, Zürich 1939.
7) M. de Quervain: Schnee als kristallines Aggregat. Experientia 1945.
8) R. Haefeli: On the Compressibility of Preconsolidated Soil-layers. Second International Conference on Soil Mechanics. Rotterdam 1948.
9) Schaad-von Moos-Haefeli: Langfristige Oedometerversuche. Interner Bericht der Erdbauabteilung der Versuchsanstalt für Wasserbau an der E.T.H. Zürich 1945 (non publicated.)
10) Haefeli-von Moos: Drei Lockergesteine und ihre technischen Eigenschaften. Schweiz. Bauzeitung, Bd. 112, H. 11, 1938.

-o-o-o-o-o-o-

Id 7

## THE FIRST LAW OF THERMODYNAMICS AND THE CONSOLIDATION PROCESS.

Ir. E.C.W.A. GEUZE
Head of the research Department of the Delft Laboratory of Soil Mechanics.

C.M.A. de Bruyn
Research Associate in Physical Chemistry.

INTRODUCTION

The first law of thermodynamics can be succesfully applied to the consolidation process, in order to evaluate the influence of loading speed on the amount of work to obtain a definite amount of consolidation. The influence of loading speed on the consolidation process has proved to be a practical problem of outstanding importance in soil mechanics.

One possibility of approaching this problem is the analytical method. Terzaghi's theory on consolidation 1) is an example of a mathematical method of analyzing the consolidation process. This theory was elaborated by Terzaghi and Fröhlich 2) to give the proper solution of the rate of settlement for a given relation between load and time. The findings of Buisman 3) a.o. in connection with secular time-effects (or secundary time-effects) complicated the consolidation properties of cohesive soils to such an extent, thet the application of the above-mentioned mathematical theories could not be maintained any longer. Buisman tried to give an approximative solution (I.c. par. 49, p. 119) based on the assumption of the soil constants, $\alpha_p$ resp. $\alpha_s$ expressing the direct and the secular effect of an instantaneous load on consolidation, and the retarding effect of hydrodynamic stresses, as expressed by a set of successive sinusoidal isochrones. One of the authors' principal objections to this point of view is the use of the secular effect $\alpha_s$ as a constant in the computation of settlement. So far no definite proof has been given of the existence of a secular effect of constant magnitude during the hydrodynamic phase of the consolidation process, though some effect must be supposed to be present. Investigations on the magnitude of $\alpha_s$ during the hydrodynamic phase are still in progress (see another report by the senior author for this Conference, entitled: "Consolidation characteristics of cohesive soils").

Meanwhile the authors tried to approach the subject by the application of a generally valid theory from an energetic point of view. The results of a series of consolidation tests with varying speeds of increasing loads gave the opportunity to study the effect of application of the first law of thermodynamics. Results of this study are given in the following report. Use has been made of this opportunity to prove the existence of the link between the above-mentioned process and the physico-chemical properties of soil. These latter are investigated by the authors simultaneously and will be the subject of later publications.

EXTERNAL WORK ON A SOIL SAMPLE.

The work done on a soil sample by a load P during a settlement dz is given by:

$$dW = Pdz$$

and in general, as found by integration:

$$W = \int_0^{z_t} P\,dz$$

Thus W can be easily obtained by graphical integration for any load-time relation. The work done on a sample, which is supposed to be saturated with water or a dilute solution is mainly used to overcome:

i. the side-friction between the sample and the ring in the consolidation apparatus.

ii. the internal frictional resistance dur-