seen that

$$R_1 < \left(1-\frac{8}{\pi^2}\right)e^{-9\pi^2\frac{T}{4}} \tag{16}$$

which gives the third column of Table 1.

TABLE 1

T	R<	R_1<
0.1	2.0×10^{-6}	2.1×10^{-2}
0.2	8.4×10^{-4}	2.2×10^{-3}
0.3	8.2×10^{-3}	2.4×10^{-4}

By virtue of this Table we can write

$$\left.\begin{aligned} \mu &= 2\sqrt{\frac{T}{\pi}} \quad , \quad 0 \leqslant T \leqslant 0{,}2 \\ \mu &= 1-\frac{8}{\pi^2}e^{-\pi^2\frac{T}{4}} \quad , \quad T > 0{,}2 \end{aligned}\right\} \tag{17}$$

as a simple composite solution correct to errors of 0.002 or less in μ. When T = 0.2, μ = 0.504 and since the order of error is relatively unaffected by a small alteration of μ or T, we can equally take μ = 0.5 as a convenient change-over point.

For less than 50 per cent consolidation, the parabolic form (8) is accurate to an absolute error of less than 0.001 (or a fractional error of less than 1/500th) in μ and a plot of μ against $\sqrt{t}$ will be correspondingly linear. On this basis Cooling and Skempton (reference 5) have used (8) to estimate the coefficient of consolidation from the initial portion of experimental consolidation - root time curves.

It is perhaps worth noting that equation (8) corresponds to a total settlement

$$\rho = m_v q_o h\mu = \frac{2}{\sqrt{\pi}} m_v q_o \sqrt{ct} \tag{18}$$

which is independent of d, and is in fact the exact theoretical total settlement for a layer of infinite depth. The high accuracy of the parabolic form for a finite layer thus implies physically that for T ⩽ 0.2, i.e. $t < h^2/5c$, the layer behaves as one of infinite depth so far as total settlement is concerned. Conversely, we can say that to the same high accuracy for any given time, all layers of thickness $h > 5\sqrt{ct}$ will have the same total settlement.

The present note has been written to indicate the derivation of the parabolic form (8) and in particular to give numerical estimates of the error involved in its use. Apart from this, it may be noted that the operational calculus can equally be applied to other consolidation problems to give alternative forms of solution which together will generally enable the theoretical consolidation-time curve to be evaluated with small numerical effort.

ACKNOWLEDGMENT.

The work described in the paper was carried out at the Building Research Station of the Department of Scientific and Industrial Research, and the results are published by permission of the Director of Building Research.

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I d 4

ON THE COMPRESSIBILITY OF PRECONSOLIDATED SOIL-LAYERS

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I. DEFINITION OF THE PROBLEM.

The formation of a proper judgement as to the possibility of consolidation of the subsoil is the main problem of settlement predictions, both qualitatively and quantitatively. It is therefore desirable to obtain a better idea of the essence of the compacting process, and an endeavour has been made to do this by means of a simple model, which may serve as extension to the model investigations carried out by von Terzaghi 1) 2). In addition to that, in the regions affected by the last ice-age, the difference between the soils once loaded by glaciers, i.e. preconsolidated ground, and the fresh deposits made after the ice age, with regard to their compressibility is particularly great. It is therefore necessary to form a clear idea of the influence of the preconsolidation load on a soil. In connection with this, from the point of view of soil mechanics among the questions arising, the three following ones are of fundamental importance:

1) How does the magnitude of the preconsolidation load affect the compressibility of a loose sediment slowly relieved to a given pressure?
2) How does the magnitude of the relieved load affect the compressibility of a loose sediment for a given preconsolidation load?
3) What conclusions can be drawn from the replies to questions(1) and(2) with respect to the judgement of "undisturbed samples" and how consolidating tests must be arranged and evaluated in order to obtain a perfect basis for settlement analysis?

Before replying to these questions some fundamental conceptions have to be explained, and the results of certain series of investigations discussed.

II. THEORETICAL CONSIDERATIONS.

If a saturated loose sediment, prevented from extending laterally, is subjected to a load increasing by steps (oedometer test), the specific settlement may be represented as a function of the natural logaritm of the pressure with sufficient accuracy by a straight line (straight line of settlement), of the form 3):

$$\Delta_i = \Delta_e \ln \lambda_i \qquad (1)$$

where x):

Δ_i = primary specific settlement = $\frac{h_1 - h_i}{h_1}$ (cf. Fig 1)

Δ_e = coefficient of compressibility

λ_i = loading index = $\frac{\sigma_i}{\sigma_1}$, where σ_1 = 1 kg/cm^2.

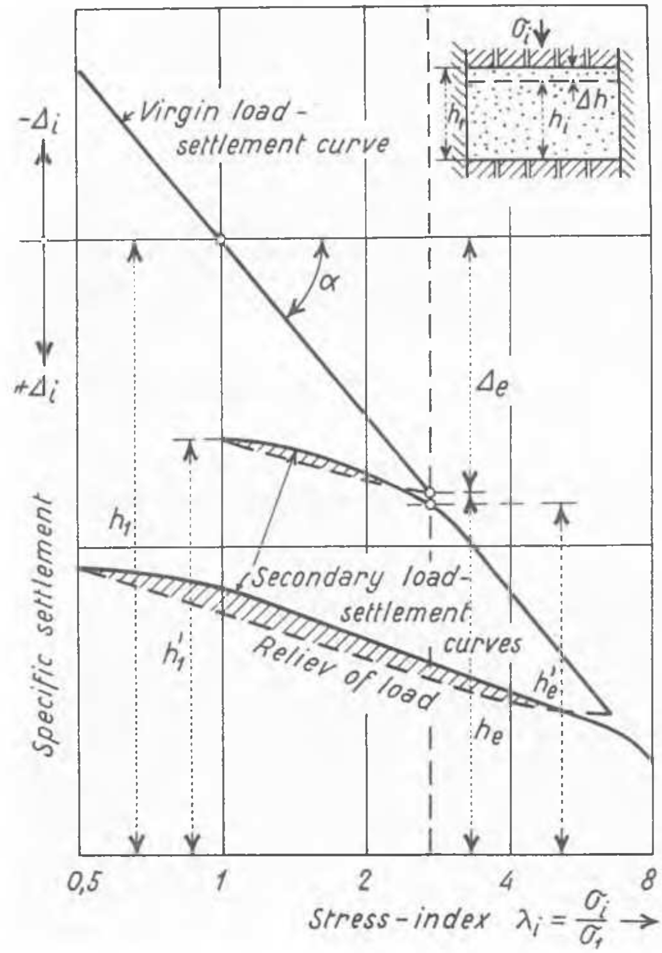
The coefficient of compressibility Δ_e is consequently that specific settlement which the sample experiences by increasing the compacting pressure from σ = 1 to σ = e = 2,72 kg/cm^2. As is known, for a given material no definite relation exists between the specific settlement and the consolidating pressure, since a different settlement straight line is obtained for each initial water content. Thereby it holds good that, with increasing initial water content, also the inclination of the corresponding settlement straight line becomes greater until a certain limiting value is reached. This limiting value is obtained when the water content is chosen equal to the Atterberg flow limit. According to that, also Δ_e is not to be regarded as a material constant, but as a constitutional magnitude whose value also increases with increasing initial water content.
If the latter is chosen equal to the Atterberg flow limit, a maximum Δ_e value is obtained, which is best suited for comparing the compressibility of different loose sediments 5).

If the behaviour of the plastic loose sediment is to be compared with that of elastic solid stone or other material, it will be found suitable to characterise the compressibility of the loose sediment by a magnitude corresponding to the modulus of elasticity, this magnitude being obtained as follows by drawing tangents to the settlement curve:

$$\frac{d\Delta_i}{d\lambda} = \Delta_e \frac{1}{\lambda_i} = \Delta_e \frac{\sigma_1}{\sigma_i} = \frac{\sigma_1}{\frac{\sigma_i}{\Delta_e}} = \frac{\sigma_1}{M_E}$$

$$M_E = \frac{\sigma_i}{\Delta_e} \qquad (2)$$

This equation shows the well-known fact



Specific settlement as a function of stress (load-settlement curves)

FIG. 1

that the modulus of plasticity M_E, which is expressed in the same units as the modulus of elasticity E, is directly proportional to the compacting pressure σ_i and indirectly proportional to the coefficient of compressibility Δ_e of the loose sediment in question. In order to compare the different kinds of soils on one common basis, we select as consolidating pressure σ_i = 1 kg/cm^2 and form the corresponding modulus of plasticity:

$$M_{E_1} = \frac{\sigma_1}{\Delta_e} \;;\; \sigma_1 = 1\,kg/cm^2 \qquad (3)$$

The two coefficients Δ_e and M_{E_1} of the different kinds of soil are in reciprocal relation to each other. and vary approximately between the limits given in table 1:

TABEL 1

Type of soil	Δ e in%	M_{E_1} in kg/cm^2
Peat	10— 30	10 - 3
Clay	5 - 15	20 - 7
Loam	2 - 10	50 - 10
Muddy sand	1 - 5	200 - 20
Sand	0,5 - 2	200 - 50
Gravel sand	0,4 - 1,5	250 - 70
Molasse marl	0,02 - 0,5	5000 - 200

x) This fundamental law was discovered in the Institute for Hydraulic Research in 1934 absolutely independent of the classic investigations of Terzaghi onnthe pressure void ratio diagram, thus confirming the latter 2) and 3)

If then a material compacted under any desired pressure $\sigma_i > e$, with admission of wa is gradually relieved and then loaded again is known that the hysteresis loop illustrat in Fig. 1 is obtained, the rising branch of which is designated as swelling curve and t descending branch as reloading curve or cur of secondary setting, or consolidation. In manner analogous to the primary consolidati being characterised by the Δ_e-value, the be viour of the material when relieved and reloaded can be characterised by the followin two magnitudes which may be calculated from the two branches of the hysteresis loop (cf. Fig. 1):

$$\Delta_e' = \frac{h_1' - h_e'}{h_1'} \quad = \text{secondary coefficient of compressibility} \tag{4}$$

$$\Delta_e'' = \frac{h_1' - h_e}{h_1'} \quad = \text{swelling coefficient.}$$

In order to standardise also this test, and thereby make it possible to compare the different materials, the consolidating and relieving pressures which bound the hysteresis loop were chosen by us not arbitrarily but uniformly, namely σ_i = e and 1 kg/cm² respectively. From this standard test also the moduli of plasticity can be derived for the relieved loading and the reloading. But since the law of the logarithmic straight lines for the two branches of the hysteresis loop is only partly complied with, to determine M_E' we use not the tangents to the corresponding curves but the chords between the points of the specific settlement σ_1 and σ_i = e kg/cm². From this we have the corresponding moduli of plasticity:

$$M_E' = \frac{e-1}{\Delta_e'}\sigma_1 = \frac{1{,}72}{\Delta_e'}\sigma_1 \tag{5}$$

for reloading

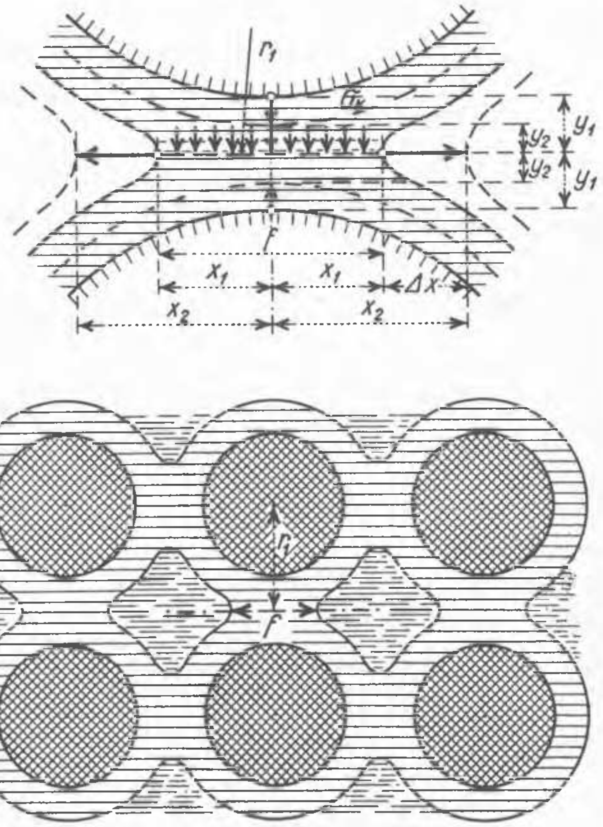
$$M_E'' = \frac{e-1}{\Delta_e''}\sigma_1 = \frac{1{,}72}{\Delta_e''}\sigma_1$$

for relieving

Since the Δ_e' and Δ_e'' values are only a relatively small fraction of the Δ_e values (about 1/3 to 1/10, depending on the kind of material), the M_E' and M_E'' values become as a rule a multiple of the modulus of plasticity M_{E_1} of the primary consolidation. This accounts for the considerable influence of preconsolidation load on the magnitude of the settlement.

As an example, the coefficients of the plastic behaviour spoken of are brought together in table 2 for the loose sediment used in the following investigations:

separate grains are not directly in contact, the transmission of force from grain to grain takes place mainly through the film lying between them, this film corresponding to the molecularly combined water of the loose sediment. For this film there is a certain critical compressive stress σ_K (boundry loading) and when it is exceeded, the material yields laterally. If our system is loaded, the closest force-transmitting surfaces f must accordingly be formed in such a way that the boundry loading σ_K is just reached in them. If the external loading is increased, a fresh equilibrium can only be established by the grains moving together, thus giving rise to a corresponding increase in the contact surfaces f (compare Fig. 2, top). The volume of the plastic inter-



Model (idealized loose soil)

FIG. 2

TABLE 2

Coefficients of compressibility and moduli of plasticity.

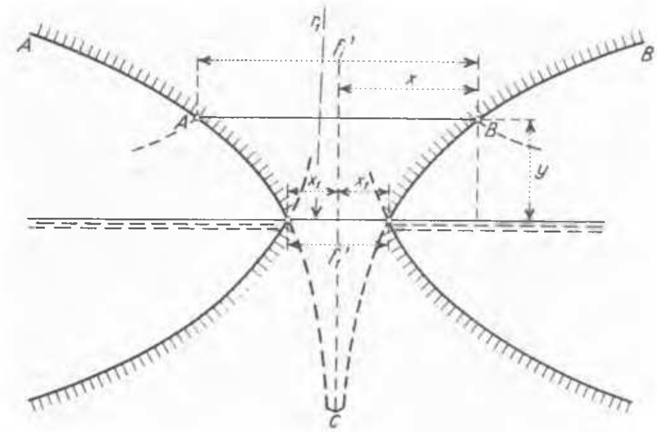
Material	Δ_e %	Δ_e' %	Δ_e'' %	M_{E_1} kg/cm²	M_E' kg/cm²	M_E'' kg/cm²
Brick clay 1632	9,12	1,40	1,23	11	123	140
Loam No. 2101	5,08	0,52	0,38	20	331	453

After that an endeavour shall be made to explain the fundamental law of primary consolidation (equation 1) with the help of an imaginary model, without taking into consideration the influence of time which will be included later 4). This model consists of separate grains which, as shown in Fig. 2, are surrounded by a tough, plastic film. Since the mediate substance then remains practically unchanged. Since the contact surface increases proportionally to the external pressure, the decrease in distance between two grains with the same increase in pressure will always be smaller the greater the consolidating pressure. Our model must therefore have a compressibility decreasing with the pressure, i.e. an increasing modulus of plasticity, as is the case with loose sediment or with snow.

In order to pass over from purely qualitative considerations to analysis, we designate with f_1 the sum of the contact surfaces per unit of surface for the consolidation pressure σ_i = 1 kg/cm², and with f_i the corresponding sum for $\sigma_i = \sigma_1$. Then according to the above, f_i must be proportional to the compacting pressure, i.e.

$$\frac{f_i}{f_1} = \frac{\sigma_i}{\sigma_1} = \lambda_i \qquad (6)$$

For the contact zone (meridian section) illustrated idealized in Fig. 3, the contact sur-



Contact zone of two separate particles of the model.

FIG. 3

face f_1' has been established under the compacting pressure σ_1, whilst half the distance between the centre points of the grains amounts to r_1. If the external pressure is increased, the two grains approach each other by the amount $2y$, whilst the contact surface of f_1' increases to f_i'. We assume that the ratio of f_i' to f_1' increases with y according to an exponential function of the form:

$$\frac{f_i'}{f_1'} = e^{\beta(\frac{y}{r_1})} = \frac{f_i}{f_1} \qquad (7)$$

For reasons of similarity it also follows:

$$\frac{y}{r_1} = \frac{\Delta h}{h_1} = \Delta_i \qquad \text{and consequently:} \qquad (8)$$

$$\frac{f_i}{f_1} = e^{\beta\Delta_i} = \lambda_i \qquad \text{according to equation 6.}$$

By taking logarithms, from equation(8) we obtain finally:

$$\left.\begin{aligned} \Delta_i &= \frac{1}{\beta}\ln\lambda_i \\ &= \Delta_e \ln\lambda_i \end{aligned}\right\} \quad \beta = \frac{1}{\Delta_e} = M_{E_1} \qquad \text{q.e.d.} \qquad (9)$$

Equation(9) is identical with equation 1). The exponential function chosen for the ratio $f_i' : f_1'$ consequently leads to the known logarithmic law of primary settlement. The equation of the curves illustrated in Fig. 3 is accordingly as follows:

$$\frac{x}{x_1} = \sqrt{\frac{f_i}{f_1}} = \sqrt{e^{\frac{1}{\Delta_e}(\frac{y}{r_1})}} = \sqrt{e^{\frac{\Delta_i}{\Delta_e}}} = \sqrt{e^{\ln\lambda_i}} = \sqrt{\lambda_i} \qquad (10)$$

If one had to deal not with plastic masses, but with the transmission of pressure between two solid bodies, whose form would correspond to the symmetrical solids of revolution according to Fig. 3 and whose compressive strenght amounts to σ_k the compressibility of the system would also have to be expressible by equation(1). The considered model may therefore be transmitted also to sand, whose behaviour - except of time effect - differs only slightly from that of loams and clays, although the solid bodies to a large extent are in direct contact with each other. On the other hand, if a metal point is constructed of the shape ABC and, as in the cone test, pressed under different loadings into a horizontally bounded material consisting of loam or clay with a given flow limit, penetration and pressure must also depend on each other as formulated in equation(1), only with the difference that, instead of σ the pressure P has to be substituted. Such penetrations will also occur when materials of mixed grains are pressed together (for instance moraine), and the pressure of sharp-sided grains is transmitted to fine clay laminas packed with liquid films.

III. ARRANGEMENT AND RESULTS OF THE TESTS.

a) Test arrangement.

Corresponding to the questions 1) and 2) posed in the definition of the problem, two test groups were to be distinguished from the start, namely a group I with variable preconsolidation load (preloading) and constant relieving loading, and a group II in which on the contrary the preloading is kept constant but the relieving loading is varied. In the third test group (III) the preloading as well as the relieving loading was varied, and in such a way that the latter always amounted to one half of the former. These three groups of tests were further subdivided as shown in table 3:

TABLE 3

Summary of the conditions of the tests

Group	Designation	Material No.	Preconsolidation load σ_I in kg/cm²	Relieving load σ_u in kg/cm²
I	I a	1632	0,25-0,5-1-e-4-8-16	0,04
	I b	1632	4-6-10-12-14-16	1,00
II	II a	1632	e = 2,72	1,0-0,5-0,25-0,125 - 0,04
	II b	2101	16	8-4-1-0,5-0,25- - 0,04
III	III	2101	0,5-1-2-4-8-16	0,25-0,5-1-2-4-8

For the tests Ia, Ib and IIa the number of samples had to be the same as the number of loading combinations. Tests No. IIb and III on the other hand were carried out each on one sample by cyclic alteration of load in one single testing operation. The duration of one load increment amounted normally to 24 hours. The influence of the duration of loading was only more closely investigated during test IIb.

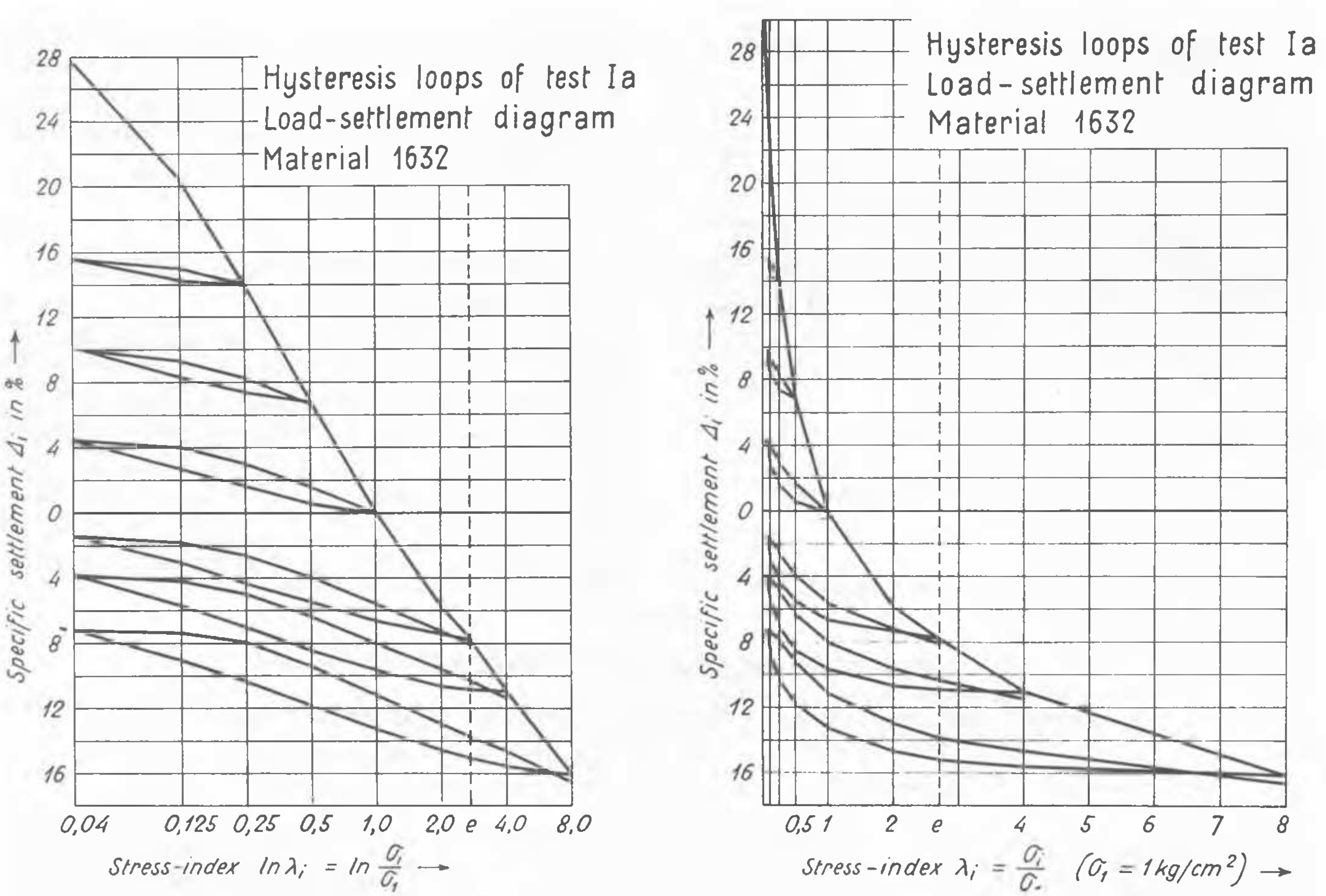
The apparatus used was the oedometer known from publications, with a cross-sectional area of 25 cm^2 and taking samples 2 cm high. 6)

The most important properties of the materials used for the tests are summarised in table 4.

TABLE 4

Characteristics of the test materials.

Property		1632	2101
Flow limit (Ateerberg) f in %		55,7	32,6
Plastic limit a in %		22,9	17,9
Coefficient of plasticity p = f - a in %		32,8	14,7
Coefficient of compression Δ_e in % (disturbed)		9,12	5,08
Coefficient of permeability k_{10^0} cm/sec		$2{,}0.10^{-8}$	$3{,}6.10^{-8}$
Grain distribution, % by weight	0 - 0,002	36,6	18,5
	0,002 - 0,02	40,5	42,9
	0,02 - 0,2	22,6	38,6
	0,2 - 2,0	0,3	-
Lime content approx. in %		27,5	38,2



Hysteresis loops of test Ia

FIG. 4

b) Test results.

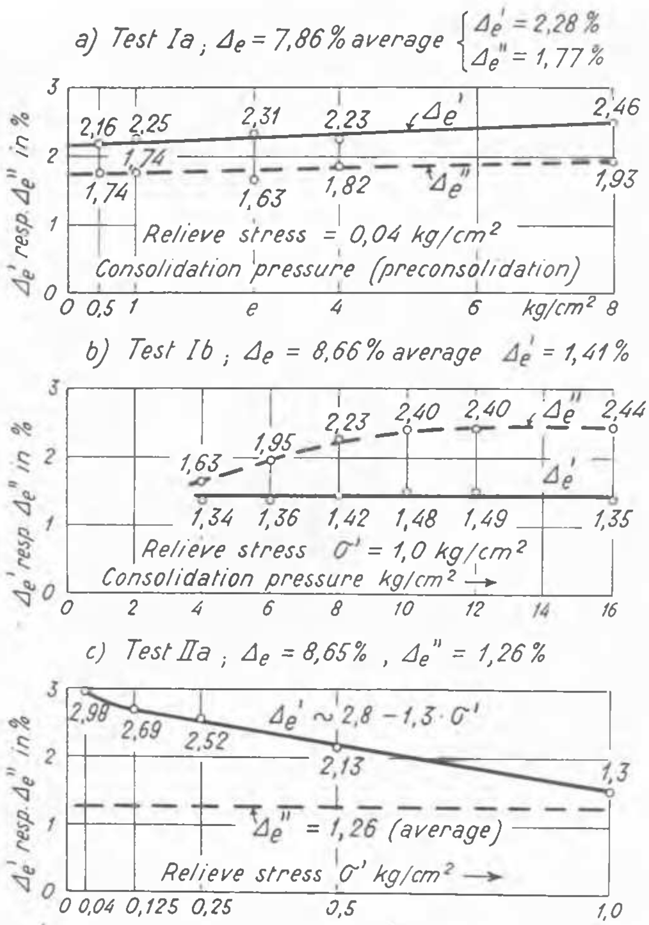
Test group I.

For test Ia the curves of the primary compressing, of the relieving and of the reloading have been plotted in Fig. 4 and compared with each other to normal or semi-logarithmic scale. It should be noted that the swelling curves in the logarithmic scale lie closer to the straight lines than the secondary load settlement curve when reloading. The hysteresis loop therefore originates because, for obvious reasons, a departure from the straight line occurs at the beginning of the relieving as well as at the beginning of the reloading. The essential result of the tests consists in that the values Δ_e' and Δ_e'' scarcely vary with preconsolidation load, i.e. they may be regarded as practically nearly independent of the height of preconsolidation load, as can clearly be seen from Fig. 5a. With higher preconsolidation loads and less relieving (test Ib), this constancy holds good only for the secondary coefficients of compression Δ_e', whilst the swelling coefficients exhibit an evident increase with increasing preloading (cf. Fig. 5b).

Test group II.

According to fig. 5, test IIa, in which in contrast to test group I the preloading was kept constant at e = 2,72 kg/cm² whilst the relieving on the other hand varied between 0,04 and 1 kg/cm², gave exactly the polar reverse of the result obtained with test Ib: the Δ_e' values showed an approximately linear increase as relieving proceeded, i.e. with increasing relieving stress. This is to be explained by the relieving favouring the regeneration of the original structure, due to the swelling process connected with the relieving. Therefore, the closer the relieving stress approaches the value 0, and the longer it is effective, the more is the compressibility with reloading. On the other hand, it is self-evident that the Δ_e'' values must be independent of the magnitude of the relieving.

With test IIb, in which the preloading was raised to 16 kg/cm², fundamentally similar conditions as in test IIa resulted. For illustrating them the whole load cycle was plotted in Fig. 6, both to normal as well as to semilogarithmic scale. The logarithmic representation shows clearly that the hysteresis loops rapidly become broader with increasing relieving load or smaller relieving stress. Connected therewith we have the secondary settlement curves (reloading) becoming steeper the more intense the relieving; this has already been shown in test IIa. The comparison



Secondary coefficients of compressibility (Δ_e') and swelling coefficients (Δ_e'') from tests Ia, Ib and IIa.

FIG. 5

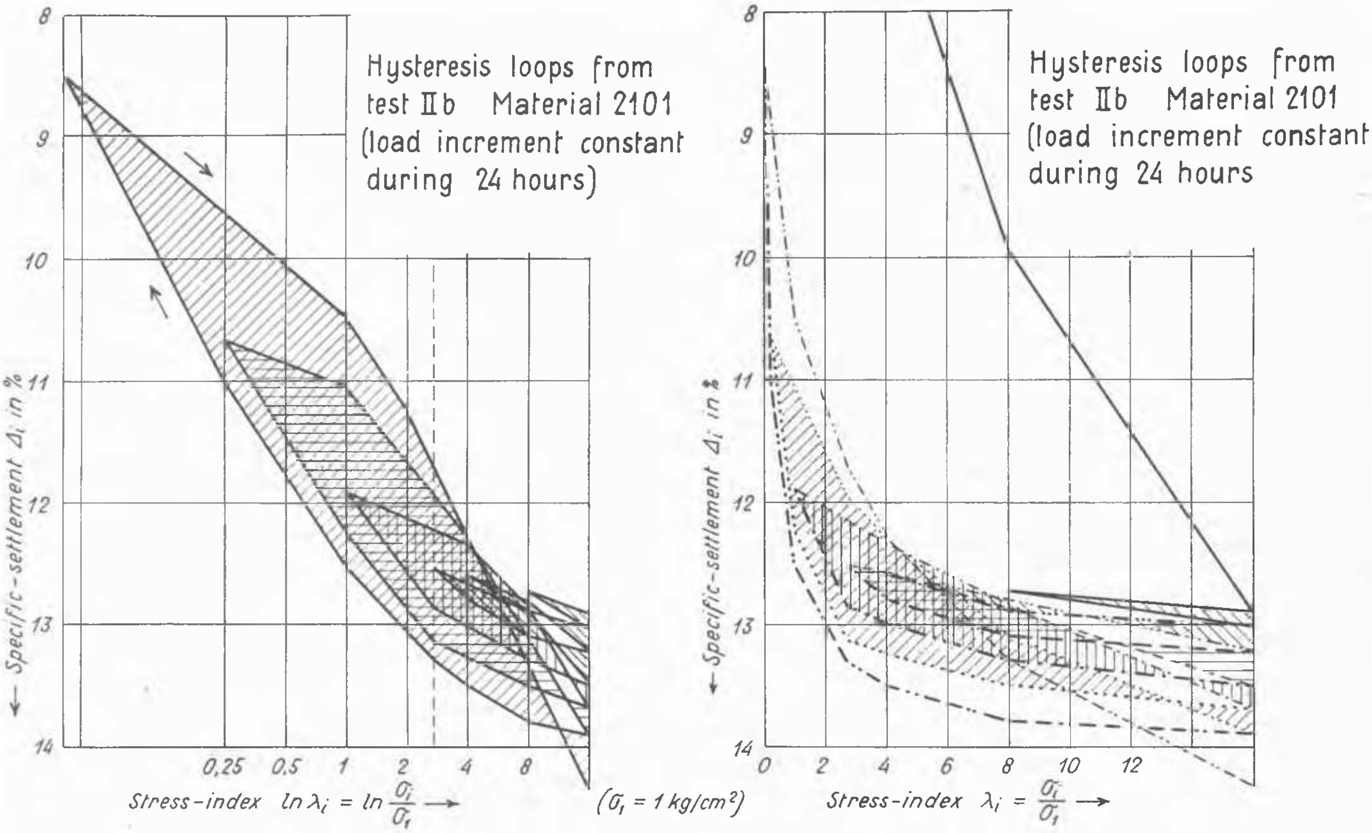
of the two representations to normal and semi-logarithmic scales shows also that with slight relieving the secondary specific settlement is nearly proportional to the pressure, but with much relieving on the other hand proportional to the logarithm of the pressure. In order to recognise the dependence of the compressibility on the relieving stress - always with constant preloading - it is preferable to compare the M_E' - values for the load range from 8 - 16 kg/cm² according to table 5.

TABLE 5

M_E'-values for the loading range of 8-16 kg/cm² as a function of the relieving stress σ_u with constant preloading σ_I

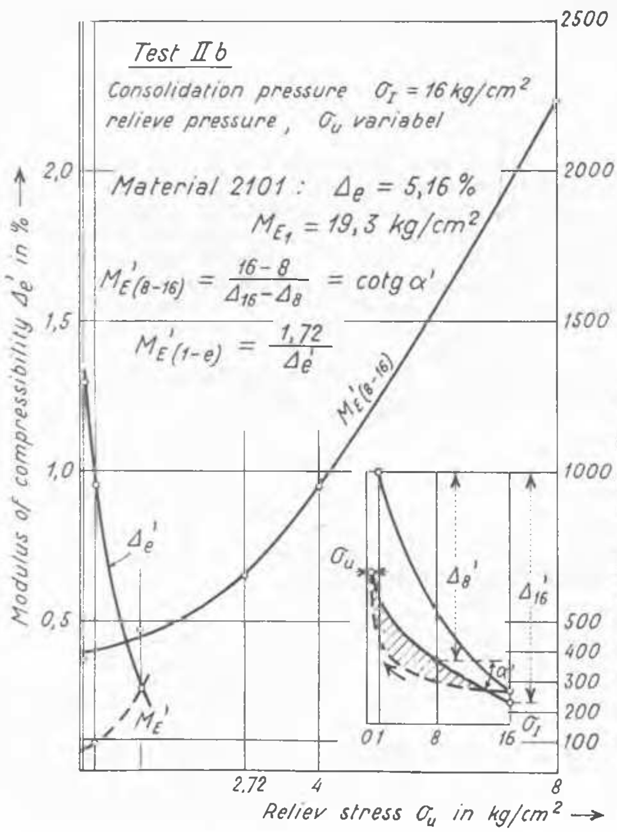
(material 2101 ; test IIb)

σ_u in kg/cm²	8	4	e = 2,72	1	0,25	0,004
$M_E'(8\text{-}16)$ $\frac{16-8}{\Delta 16 - \Delta 8}$	4450	1900	1310	950	-	775
Δ_e' in %				0,27	0,95	1,31
Δ_e'' in %				1,03	1,02	0,90
$M_E'(1\text{-}e) = \frac{1{,}72}{e'}$ kg/cm²				637	181	131



Hysteresis loops from test IIb.

FIG. 6



Modulus of plasticity M_E, as a function of the relieving stress.

FIG. 7

In contrast to the standard test with M_{E1} = 20 and M_E' = 331 kg/cm², the moduli of plasticity for the load increment of 8-16 kg/cm² fluctuate here, according to the degree of relieving, between 775 and 4450 kg/cm². Very striking is the great increase in compressibility with increasing relieving, or diminishing of the relieving stress (Fig. 7).

The influence of the duration of the relieving time is seen particularly clearly on the other hand in Fig. 8, which illustrates the hysteresis loops for two parallel tests with relieving times of 24 hours and 5 minutes respectively per load increment. In order to show that the swelling process is in the first line decisive for all these occurrences, a analogical loading cycle was carried out with dry material powder for the purpose of comparison (Fig. 8). The corresponding moduli of plasticity gave the following values:

Saturated, load alteration every 24 hours
M_E' = 575 kg/cm²
Saturated, load alteration every 5 minutes
1430 kg/cm²
Dry powder, load alteration every 5 minutes
3400 kg/cm²

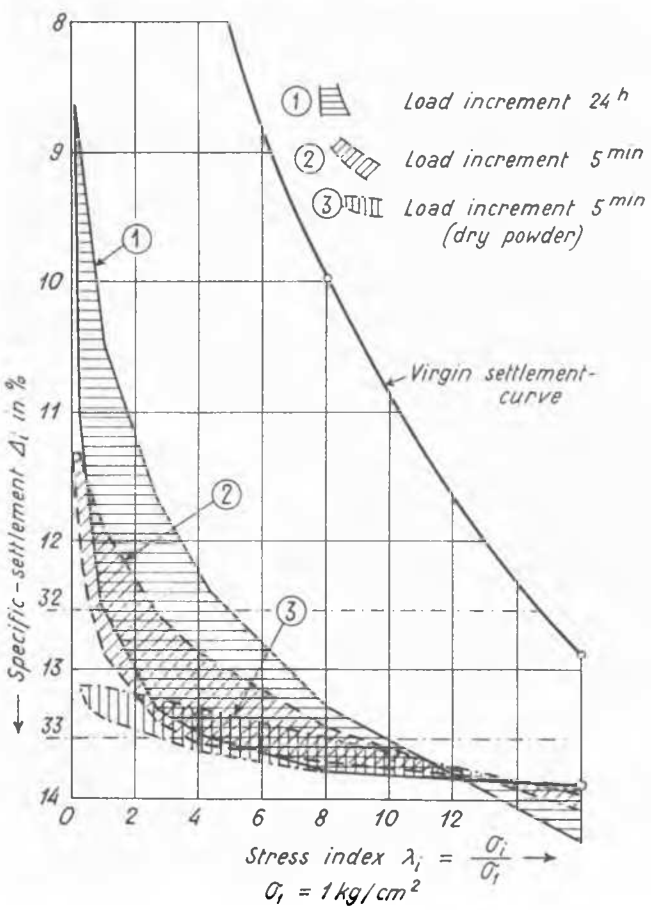
Test group III.

In test group III, in which the relieving stress σ_u was always the half of preconsolidation load σ_I, increase of the moduli M_E' and M_E'' was approximately proportional to preconsolidation load σ_I, the moduli referring each time to the whole range of the relieving (table 6).

TABLE 6

Results of test III (Material 2101; $\Delta_e \sim 5{,}0\,\%$).

Preconsolidation load	kg/cm²	1,0	8,0	16,0
Relieving stress	"	0,5	4,0	8,0
M_E' $(\sigma_u \div \sigma_I)$	"	114	840	1290
M_E'' $(\sigma_I \div \sigma_u)$	"	177	1510	2600
$M_E = \frac{\sigma_I}{\Delta_e}$	"	~ 20	~ 160	~ 320



Dependence of the hysteresis loop on the duration of the test.

FIG. 8

From table 6 it can be seen that the secondary modulus of plasticity M_E' is here about 4 - 6 times greater than the primary one.

In carrying out and evaluating the tests, we are indebted to the engineers F. Germann and R. Cebertowicz for their collaboration.

IV. CONCLUSIONS.

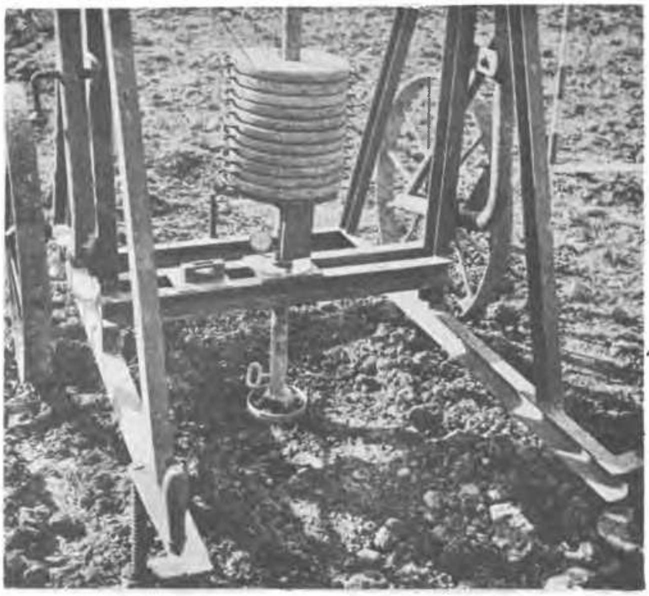
When investigating the compressibility of loose sediment it will be found preferable to start from the specific settlement. Through the model described in section II, it becomes comprehensible that the specific settlement or the void ratio increases in proportion to the logarithm of the pressure. Based on the results of tests carried out up to the present, which refer for the moment only to the behaviour of two typical representatives of fine-grained loose sediment, the questions posed at the start may on the other hand be answered as follows:

1) The secondary coefficient of compression Δ_e' is, with constant relieving stress, practically independent of the height of preconsolidation load.

2) With a given preconsolidation load, the material is all the more compressible, or the Δ_e' value is all the greater, the more the material has been relieved before the reloading, or the more intense the swelling process was. The last-mentioned causes a partial regeneration of the original-structure.

3) The relieving due to the removal of the sample from the natural layer causes a disturbance which can hardly be avoided. The sample should be taken in such a way that the swelling process is absolutely prevented, and this gives new points of view for the further development of the samplers 7). Also the packing of the samples into the oedometer often entails disturbances. The results of the compression tests with so-called undisturbed soil samples must therefore be judged with very great care. Much more reliable are carefully conducted settlement tests on undisturbed layers of soil in order to determine the modulus of plasticity M_E^x.

The result of field tests will preferably be checked by laboratory tests, the oedometer being used to determine the secondary coefficient of compression Δ_e' or the corresponding modulus of plasticity M_E'. The experience made by us up to the present shows that, with preconsolidated soils the two values M_E^x and M_E' as a rule to a large extent agree with each other, which is to be attributed on the one hand to the slight dependence of the compressibility on the height of preconsolidation load, and on the other hand to the fact that in one method of testing as well as in the other, only a relatively slight relieving of the material takes place. The determining of M_E^x is based on the accurate measurements of settlement by means of special appliances. Their adoption in a borehole makes it possible to test the subsoil with respect to its compressibility. Fig 9 shows an appliance designed by the Soil Mechanics Department with the collaboration of W. Schaad, engineer, for testing the subsoil when constructing the Kloten airport 8).



Light, movable settlement recorder, Kloten

FIG. 9

The connection between settlement and M_E-value is given by the following equation:

$$y = \frac{\sigma}{M_E^x} D \tag{11}$$

where D is the diameter of the circular-shaped loaded surface.

Equation(11) gives at the same time the relation between the modulus of plasticity, the coefficient of subgrade reaction C (ratio of soil pressure to settlement), and the diameter of the loaded surface, the following holding good:

$$M_E^x = \frac{\sigma}{y} D = C.D \tag{12}$$

C = coefficient of subgrade reaction

Finally, the methods illustrated here must be regarded as symptomatic for that new development of soil mechanics which is making efforts for further extension and increased adoption of field tests on undisturbed layers of soil, since the mere removing of the separate samples from the entirely of the soil entails a sensible disturbance in consequence of alterations in stressing. The coordination of field and laboratory tests leads to mutual extension and checking. This is the middle course which avoids the danger of onesidedness that may occur when one or other of the methods of testing is adopted exclusively.

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