upwards but the corresponding bench marks settled about two inches which should increase the total settlement of Pier 2S from twenty-two inches to twenty-four inches. Theory B is unable, however, to explain why piers 2N and 3N tipped north.

7. Conclusions. The behavior of Pier 2S during the first thirteen years of its existence suggests that the saturated fine sand mass either was in process of consolidation like clay or more probably moved plastically (Fig. 7), for saturated fine sand masses apparently may behave as plastic bodies. Actually, plastic flow may be visualized as a slow movement under the action of an unbalanced shear stress. The slow movement element is undoubtedly present in similar cases and the existence of a shearing stress may be proven by the fact that a movement of this kind may be stopped by increasing the shearing resistance of the mass. Dr. Ing. Otto Mast reports a similar case in Germany where the movement of fine sand running from underneath a bridge pier was discontinued by applying the Joosten consolidation process. ("Der Bauingenieur," vol. 15, No.33/34 (August 17, 1934). Obviously the latter process increases the shearing resistance and not the resistance to compression of the material since a saturated fine sand mass is itself incompressible.

In studying this case, a number of questions arose which have remained unanswered. Some of them are: (a) How is the friction resistance distributed along the length of a so-called "friction pile"; (b) is the stress at the tip of a loaded pile a pressure or a shearing stress, or combination of both; (c) how is the load distributed between the soil surface, S, and the piles, P, driven in it (Fig. 5)?

No. F-7

RESULTS OF LONG DURATION SETTLEMENT TESTS

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Introductory remarks. The existing settlement theory supposes that compressible soils demonstrate a definite degree of compressibility so that an increase of loading, after some time has elapsed, simply results in a definite decrease of the porevolume, to an amount, depending upon the properties of the soil and the magnitude of the applied load increment. If, for some soilsample, it lasted some days or even a week and more before final consolidation resulting from the new loading was attained, this circumstance had to be described to a very small degree of permeability and not to secular effects. Recent American publications already mention the observation of a "secondary" time effect and for this reason we supposed, that the results of long-duration tests, performed in the laboratory of soil mechanics of Delft, might be of interest for the purpose of this conference, as it seems that secular effects cannot be neglected in theoretical and practical treatment of the settlement problem.

Observation of settlements. In consequence of the study of time-settlement diagrams of both structures and laboratory samples, if plotted on a semi-logarithmic scale, it appears, that these diagrams may be represented approximately by a vertical line symbolizing the direct compression effect α_p succeeded by a rectilinear slope representing a secular effect α_s, so that

$$z_t = \alpha_p + \alpha_s \log_{10} t \tag{1}$$

if only the moment of application of the load is taken as time 1.

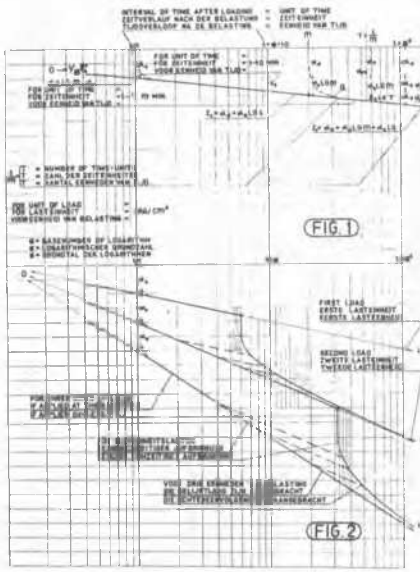
z_t settlement per unit of thickness of layer or sample dependent from relative thickness thereof.
t time of observation of settlement in minutes.
α_p settlement for 1 kg/cm² representing the direct effect and
α_s settlement for 1 kg/cm² representing the secular effect of loading for time intervals 1-10, 10-100 etc.

For these intervals z_t increases with equal amounts α_s and of course the value of α_s will depend upon the unit of time we use.
If instead of 1-10, 10-100 minute periods we prefer to use 1 - m, m-10m, 10m - 100m, minute periods, and indicate the ends of these periods as T, 10 T etc, we must put $t = m.T$, so that

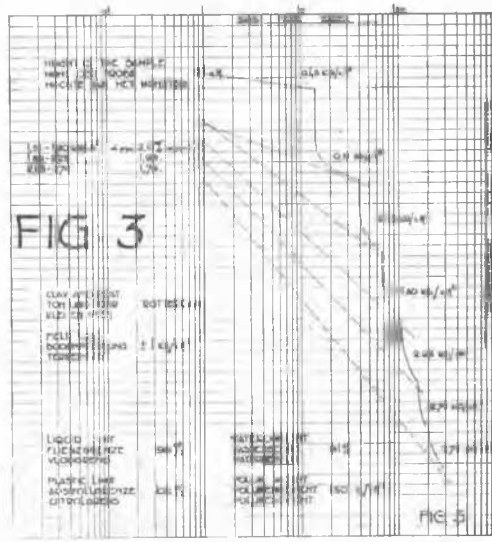
$$z_T = \alpha_p + \alpha_s . \log_{10} mT = \alpha_p + \alpha_s \log_{10} m + \alpha_s \log_{10} T \tag{2}$$

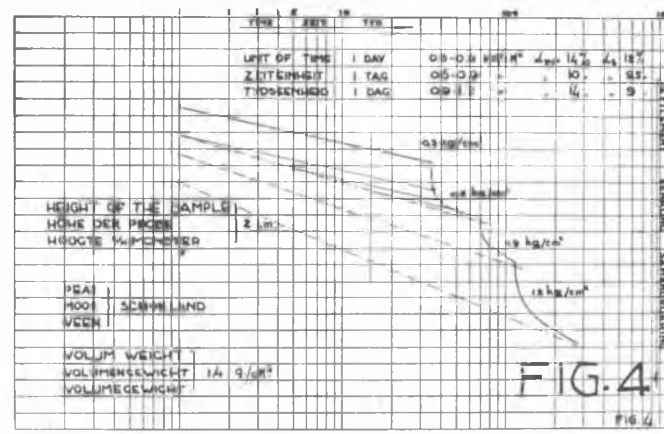
If we start the diagram at $T = 1$, situated upon a part of the diagram that is really rectilinear, we can put

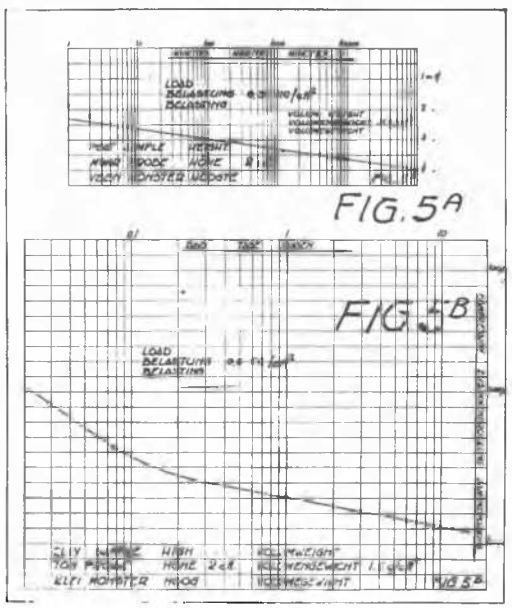
$$\alpha_p + \alpha_s \log_{10} m = \alpha_{pI} \tag{3}$$

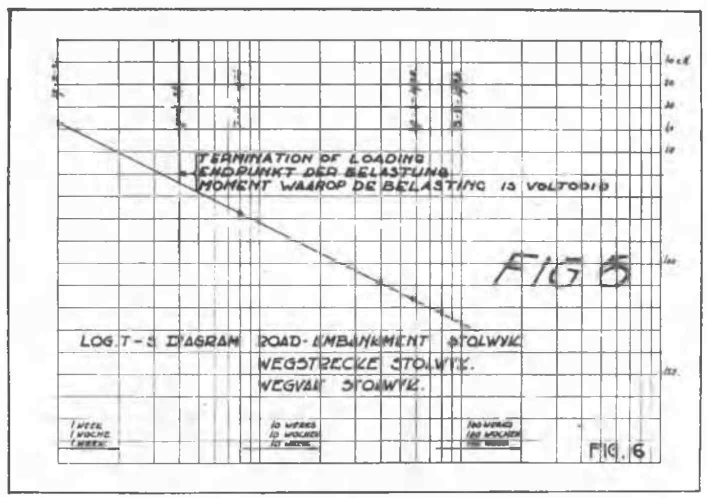


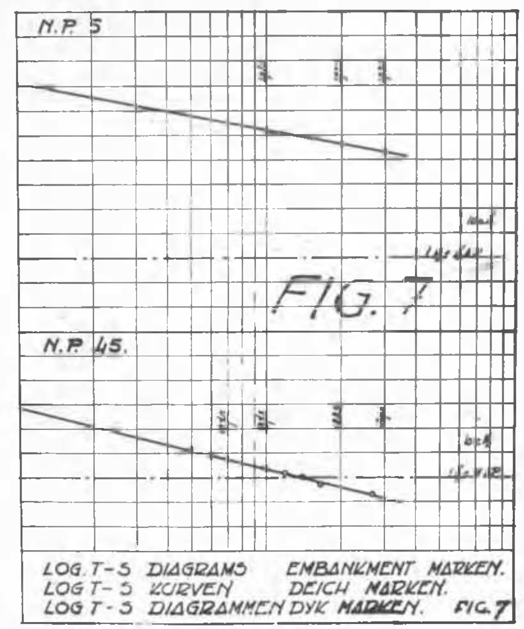
HYPOTHETICAL TIME-SETTLEMENT DIAGRAM
HYPOTHETISCHE ZEIT-SETZUNGS KURVE
HYPOTHETISCH TIJD-ZAKKINGS DIAGRAM











so that

$$Z_T = \alpha_{PI} + \alpha_s \log_{10} T \tag{4}$$

Log_{10} m values for different periods of time are given in Table I.
Simplified diagrams conform these formulae are represented in Figures 1 and 2.

TABLE I

Period from moment of loading until:	Log_{10}m.
10 minutes	1.00
1 day	3.16
1 week	4.00
$2\frac{1}{2}$ months	5.03
1 year	5.72
1 century	7.72
10 centuries	8.72

For the sake of simple representation we supposed in this paper that a proportionality of loadings and resulting effects would exist; might this not appear to be sufficiently exact in a given case, so can correct values for a planned loading be easily determined. Conform to our supposition log t-s lines all intersect the $z_t = 0$ axis in the same point 0, as it is indicated in Fig. 2.

Loads not applied at the same time, but with a certain time interval between them would, if further a superposition that takes into account the different timescale might prove to be justified, cause log t-s diagrams as represented in Fig. 2. The respective diagrams would then have asymptotes, intersecting in the same point 0, already mentioned above.

Verification of hypothetical diagrams by Laboratory tests. In Figures 3, 4, 5 results of laboratory tests of short and long duration on peat and clay samples, taken at random, are plotted on a semi-logarithmic scale. The experimentation took place before the rectilinear diagram was thought of, otherwise some of the loads in the long duration experimentation would surely have been applied at a later time. Fortunately it appeared feasible to find the asymptotes with the help of smaller time scales, as it is represented on some of the figures.

These tests, one of which lasted for 500 days, seem to justify the following tentative conclusions:

1. The semi-logarithmic diagrams, with an exception for the vicinity of the loading point, are rectilinear as long as observations were continued.
2. Observed peat samples of 2 cm of thickness procure rectilinear diagrams about 1 minute or a few minutes after loading (Fig. 5a) and clay samples about 1 day after loading (Fig. 5b).
3. Diagram slopes (α_s values) are approximately proportional to the loads applied, with perhaps some tendency to-ward decreasing increments of slope for equal increments of load.
4. Superposition of loads, successively applied, results in superposition of log t - s diagrams; the same can be said with respect to the direct load effects of which α_{PI} values are given.
5. Higher temperature causes increasing slope of the diagrams.

Comparison of field and laboratory diagrams. Fig. 6 and 7 give log t-s lines for a road embankment as described in the paper of ir. Royer and for a levee as described in the paper of ir. v. d. Burgt. A comparison between laboratory and field results taking into account the thickness of the deposits will be made. It is remarkable, that for the road embankment, that has been observed with great care during two years the log t-s. diagram appears to be straight already at the termination of the process of loading.

Conclusive remarks. If our tentative conclusions in future might find confirmation, this would signify, that our usual conception of compressibility, according to which a consolidation test was continued until a certain small velocity of settlement had been attained, was composed of a mixture of direct load effects and secular time effects, the last one being represented therein to an amount, depending upon the procedure of testing.

Also in theoretical treatment of the settlement problem it then should have to be taken into account, that in every soil element whereupon a new imposed load begins to exert its influence not only the direct load effect, but also a secular time effect begins to develop itself and should be taken into account. Perhaps in some cases also a certain degree of non-homogenity as to permeability should be taken into consideration. This would still more complicate the mathematical treatment of the settlement problem, that already at present is rather intricate.

It seems of interest to determine in future experimentation α_p and α_s values of deposits and of samples of standard thickness (2 cm) and to find the laws to which these quantities obey, especially whether α_s in the reality appears to remain a constant or eventually for how long a time it does so. Tests for practical purposes then should be performed at average soil temperature.

The problems here discussed might be of interest not only in soil mechanics but also from a geological point of view as to age, load and density of deposits.

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No. F-8 MEASURING GROUNDWATER PRESSURES IN A LAYER OF PEAT, CAUSED BY AN IMPOSED LOAD

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Nature of the soil. The soil under the projected road Sneek-Joure consists of a very compressive peat layer thick 2 - 3 m, under which sand-layers of sufficient thickness and bearing capacity were found.

Here the question arose if it was possible to construct a road embankment on the existing underground, regarding the possible settlements, in order to construct a concrete road immediately after finishing the embankment without further expensive upkeep costs.

Also should be investigated:

1. how fast the compressive peat layers will settle under the applied sand-load,
2. when the consolidations will be complete.

As the stability of the tramway-embankment next to the projected road should be assured, it was decided to make an experimental road section, in order to obtain the necessary data.

Short description of the experimental road section. The physical characteristics of the subsoil of the chosen plot can be considered essential for the whole projected road. Data of borings taken at that place, as well as the dimensions of the constructed experimental section are given in Fig. 1.

This experimental road section was divided into 4 sections in order to obtain as many results as possible.

1°. Section A long 10 m, with a sand-load until 0.20+ = common top-height of the projected road.
2°. Section B, long 15 m, with a sand-load to 1.20 +.
3°. Section C, long 25 m, with a sand-load to 1.95 +.
4°. Section D, long 55 m, with a sand-load to 2.70 +, with regard to the ramps to be constructed.
5°. Section E, load of the berm next section D, at the height of section B.

Apparatus for measuring settlements. In order to get an idea of the expected settlements, displacements, etc. height-control-pegs were placed at the bottom of the new digged ditch and at the place of the projected sand-embankment on soil surface and on the separation between the mould and the peat layer.

Pegs were also put up for controlling the lateral displacements as given in Fig. 1.

Before the beginning and during the sand-tippings the height of these pegs were exactly observed.

The results of the found settlements are given in Fig. 2. To this the following explications ought to be given: The horizontal axis is the time-axis, 1 cm = 1 week. The height of the sand-tippings are plotted vertically above the horizontal axis, the measured settlements under the horizontal axis.

Horizontal displacements have not been observed.

Comparison between the computed settlements and the data of investigated undisturbed soil samples. Eight undisturbed soil samples have been tested in the consolidation apparatus of the Laboratory of Soil Mechanics.

For two peat-samples values for the coefficient of compressibility C of respectively 4.6 and 3.9 have been found, while for the top layers consisting of mould and clay, a consolidation coefficient of 10 was found. With the aid of these coefficients the expected settlement under several loads has been computed. (Fig. 3).

The relation between loads and computed settlements were plotted in Fig. 4, while the sand heights belonging to these loads have been plotted above the settlements, so that immediately can be seen, which settlement can be expected if a certain height of sand is applied. The expected end-settlements in the experimental road section are also plotted in Fig. 4.

It appears that the settlements of the experimental road section do not agree with the computed settlements, they are lower by lower loads and greater by higher loads than the computed ones.

Even appears that the settlements of the experimental section rather well agree with settlements computed with the premise, that the peat follows the law of Hooke ($E = 0.8$ kg/cm^2).

Method for measuring water pressures. In loaded soils with a little water-permeability the applied loads will be first supported for the greater part by the water and when the pression in the water decreases by the material itself. The water, which gradually comes under higher pressures as load increases, causes a gradient at the side of the cross-section. These higher water pressures will also be one of the causes of lateral displacements. In order to get an idea of the course of the water pressures after loading and in order to test this theory to the practice, these water pressures were measured. A design of the used measuring-apparatus gives Fig. 5.

The water pressures were measured with the aid of wells, which were placed in the peat layer. The place of the wells in the cross-section is given in Fig. 5. The pressure of the water in the wells was measured with mercury-gauges attached at the wells.