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# Test areas with several vertical drainage systems on state highway no. 19 at Schipluiden, NL

# Versuchsgelände mit verschiedenen Vertikaldränsystemen im Autobahn Nr. 19 in der Nähe von Schipluiden, Niederlände

ZUSAMMENFASSUNG

In der niederländischen Autobahn Nr. 19 (im Bau) ist ein Versuchsgelände angelegt zum Studium des Setzungsherganges eines Autobahndammes bei der Anwendung verschiedener Vertikaldränsysteme. Die Geländemessungen und die Berechnungen zeigen eine schnellere Konsolidierung bei Verkürzung des Abstandes zwischen den Vertikaldränen. Es erweist sich, dass die drei angewandten Vertikaldränsysteme in der beschriebenen Situation eine gleiche Wirkung haben.

## 1. INTRODUCTION

A road which has been constructed on a strongly compressible subsoil, will be sinking for a considerable time after it has been opened for the traffic. This settlement will have to be kept within certain limits, however, so that the maintenance costs do not increase to a disproportionate level.

One of the methods of influencing the process of settlement of a road, is the application of vertical drainage.
After the embankment has been installed the soil particles in the weaker layers will tend to group themselves more compactly and therefore diminish the pore volume. However, the pore water will prevent this change in volume; until the surplus of pore water can flow off the settlement will not occur. As the pore dimensions are small and the pore water will have to travel a relatively long way, this process of flowing off will be a slow one, so will be the process of settlement.
By application of vertical drainage, the way to be travelled by the pore water, is considerably shortened and therefore the process of settlement is accelerated.

The degree to which vertical drainage influences the process of settlement, will depend on the type of vertical drains and the distance at which they are placed. The influence of these factors can be appointed theoretically; it remains, however, to what extent this theory is realistic.
In order to get more knowledge in this matter, the Highway Department of the Ministry of Public Works in cooperation with the Delft Soil Mechanics Laboratory have installed 11 test areas in state highway no. 19, which is under construction between Rotterdam and The Hague.
The following report gives briefly the design, the purposes and the greater part of the results of these test areas.

## 2. LOCATION

The test areas are situated south of the connection of the state highway nr. 19 with the Kruithuisweg near Delft (see figures 1 and 2).
Surfaces of the test areas vary from 1600 to 2500 $m^2$ and locations are chosen both inside and outside the main lane.
All areas are temporarily charged with comparable loads; the surplus of sand will be removed before

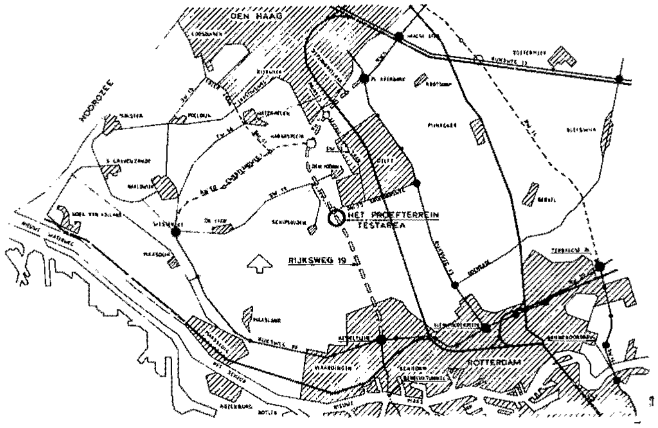


Figure 1

the construction of pavement (in about 1980). The oblique locations of the areas prevented from being intersected by old polderditches.

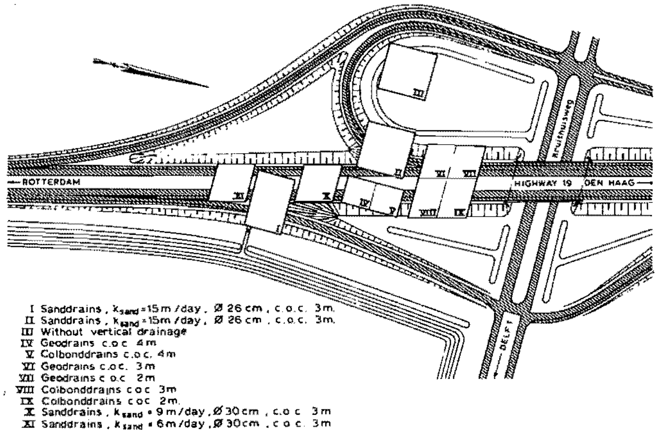


Figure 2

## 3. EXECUTION OF THE EMBANKMENT

The original surface in the area of the road was at about 3 m - N.A.P. The construction height of the test area is about 2 m + N.A.P., including a settlement of about 2 m, the embankment will have a thickness of about 7 m.

The sand was applied by hydraulic fill in layers of about 1 to 1.5 m, except for the first layer, which had a thickness of about 2 m.
In figure 3 the speed of loading is indicated.
The plan to construct the whole test area on the same level was not feasible for 100% for practical reasons.
Before the installation of the embankment cunets, about 10 m wide and 2 m deep, are dug out at the future slope and filled with sand for stability-support.

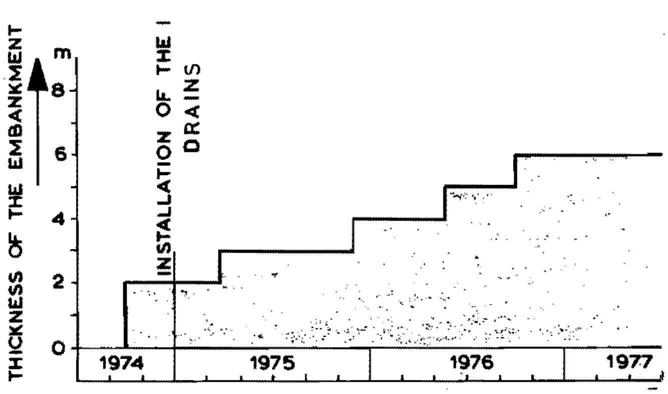


Figure 3

After the first sand layer of 2 m thickness has been applied, the vertical and horizontal drains were installed.
For the data of the different test areas we refer to figure 2.
The vertical drains in the test areas reach to a depth of 8 m - N.A.P.

The sand drains are made with a jetting system at 3 m c.o.c. distances in a pattern of equilateral triangles.

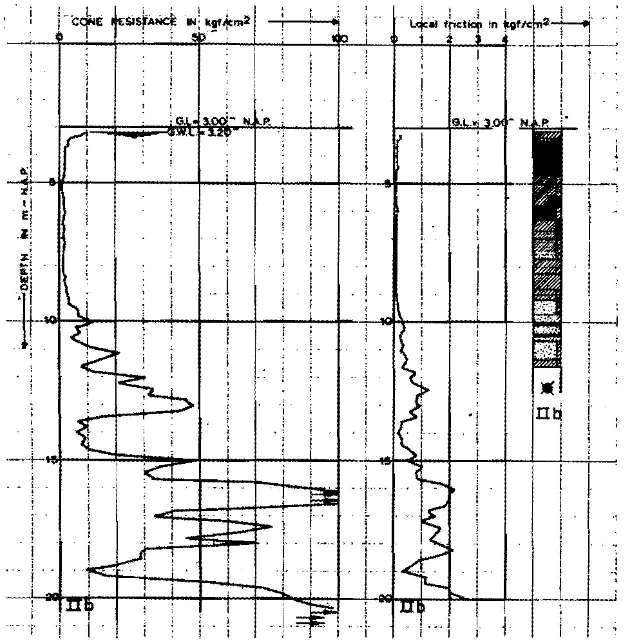
Geodrain is a product which is developed in Sweden, as an improvement to the paper drain as developed by Kjellman. It consists of a profiled plastic strip of about 10 cm wide inside an envelope of special impregnated paper.
The drains are pushed into the ground by means of a converted Volvo-loader. As expected, this machine has proved to be too light to penetrate the first sand layer of 2 m thickness, in which cone resistances have been measured of more than 80 kgf/ $cm^2$. By lack of time a heavier machine could not be used. At places where the machine could not reach through the sand layer the sand jetting machine assisted.

The A.V.-Colbond drain is a fabric (non-woven), developed in Holland, with a width of 30 cm and a thickness of 4 mm.

The Colbond drains are installed with a jetting system with a large weight. Remarkable is that the use of water gives problems in some cases.

### 4.1. Soil structure

In the territory of the test areas of highway 19, we found a soil strata which consists of an amount of layers with different composition; it also appears from recent executed control-borings that some



| SOIL DESCRIPTION | $\gamma$ (t/m³) | $W_0$ | AT A LOADING OF $\Delta p = 32$ KN/m² | | | | $\varphi'$ | C' N/mm² |
|---|---|---|---|---|---|---|---|---|
| | | | $C_v$ | $C_h$ | $1/C'_p$ | $1/C'_s$ | | |
| POLDERCLAY / TOPCLAY | 1.5 | 81.5 | $1.28 \cdot 10^{-5}$ | $608 \cdot 10^{-6}$ | 0.0332 | 0.0039 | 20° | 0.004 |
| PEAT | 1.01 | 497.3<br>652.2 | $1.52 \cdot 10^{-4}$<br>$2.5 \cdot 10^{-4}$ | $8 \cdot 10^{-4}$<br>$4.76 \cdot 10^{-5}$ | 0.1024<br>0.2204 | 0.0321<br>0.0208 | 24° | 0.002 |
| CLAY WITH ORGANIC MATERIAL | 1.35 | 136.3 | $4.44 \cdot 10^{-5}$ | $4.44 \cdot 10^{-5}$ | 0.1034 | 0.0106 | 18° | 0.002 |
| PEAT MORE OR LESS CLAYEY | 1.00<br>1.08 | 500.7<br>397.4 | $4 \cdot 10^{-5}$<br>$2.22 \cdot 10^{-5}$ | $2.22 \cdot 10^{-3}$<br>$8.33 \cdot 10^{-6}$ | 0.1856<br>0.1645 | 0.0271<br>0.0560 | 16° | 0.001 |
| CLAY WITH ORGANIC MATERIAL | 1.4 | 113.2 | $2.5 \cdot 10^{-4}$ | $6.06 \cdot 10^{-5}$ | 0.1196 | 0.0086 | 19° | 0.002 |
| CLAY LAMINATED WITH SAND | 1.65 | 71.7 | $1.05 \cdot 10^{-4}$ | $1.67 \cdot 10^{-3}$ | 0.0672 | 0.0078 | 24° | 0.001 |
| SAND LAMINATED WITH CLAY | 1.7 | 51.2 | | | 0.0551 | 0.0065 | 27° | |
| SAND LAMINATED WITH CLAY AND PEAT | 1.85 | | | | 0.0323 | 0.0050 | 30° | |
| SAND | | | | | | | | |
| CLAY LAMINATED WITH SAND | | | | | | | | |
| SAND | | | | | | | | |
| BASIC PEAT | | | | | | | | |
| PLEISTOCENE SAND | | | | | | | | |

LEGEND :

G.L. = GROUND LEVEL
G.W.L.= GROUNDWATER LEVEL
$\gamma$ = UNIT MASS IN t/m³
$W_0$ = INITIAL WATER CONTENT
$C_v$, $C_h$ = COEFFICIENTS OF CONSOLIDATION
$1/C'_p$, $1/C'_s$ = COEFFICIENTS OF VOLUME COMPRESSIBILITY (DIRECT) (SECONDARY)
$\varphi'$ = ANGLE OF INTERNAL FRICTION
C' = COHESION
NAP = NORMAL AMSTERDAM LEVEL

Figure 4

layers vary considerably in thickness.

From the ground level (originally at about 3 m - N.A.P.) we found after the crust of polderclay of about 0.5 to 0.6 m thickness, a system of soft layers, successively consisting of a first peat layer (thickness 1 to 1.5 m), an intermediate clay layer with organic material (thickness and composition considerably varying), a more or less clayey peat layer with a thickness of 1.3 to 1.5 m, passing over into a 1 to 1.5 m thick clay layer with some organic material, in which to the depth thin sand layers appear.

The bottom of these soft layers is at 8.5 to 9 m - N.A.P. Next layers with higher cone resistances are found consisting of holocene deposits of sand more or less intersected by thin clay layers. For the above-mentioned intermediate clay layer it is proved that the thickness within one test area can vary considerably between 0.4 to 1.2 m.

The volume weight varies between 1.2 and 1.3 to 1.35 t/m$^3$.

It also appears that in area VIII and presumably in area IX the thickness is a lot more, maybe originally over 2 m, with values of the volume weight of 1.5 to 1.55 t/m$^3$. The sounding and the boring profile of area II with the reported soil properties illustrate the above-mentioned description. (Figure 4).

## 4.2. Groundwater tables

The polderwater level is about 3.2 m - N.A.P.

In the holocene deposits of sand groundwater levels are measured of 3 to 3.5 m - N.A.P.; in the deep sand the levels will change between 3.5 and 4.5 to 5 m - N.A.P., caused by pumping of coolwater for industry in Delft.

## 4.3. Soil properties

The permeability in horizontal and vertical direction can be influenced strongly by occurrence of rush-stalks, particularly in the first peat layer and the intermediate clay layer.

Originally the first peat layer is strongly permeable; with the increase of the consolidation the permeability in both directions decreases. As the coefficient of compressibility $m_v$ (= $\alpha$) does not diminish in the same rate with $k_{v,h}$ also $c_{v,h}$ decreases considerably (factor 10 to 20).

For the intermediate clay layer we can calculate for higher loadings with a small reduction of $c_{v,h}$ (factor 1.2 to 1.5).

The more or less clayey peat layer is strongly impermeable in both directions; for an increasing rate of consolidation for the loading the $c_{v,h}$ decreases with a factor 4 to 8.

## 5. DEFINITION OF THE TEST AREAS

The test areas have to enlarge the knowledge for the next points:

1) The influence of vertical drainage on the time of the hydrodynamic period for different c.o.c.-distances.
2) The difference in operation of the applied drainage types.
3) The difference between theory and practice. Studying of the results can lead to an adaption of the existing settlement theory and an improvement of the methods of calculation for cases with loading in steps.

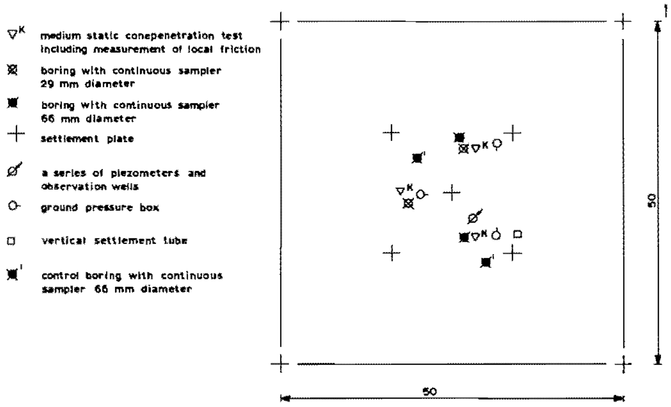


Figure 5

## 6. FIELD AND LABORATORY INVESTIGATION AND INSTRUMENTATION IN THE FIELD

In May 1974 medium soundings and continuous borings Ø 29 mm and Ø 66 mm are executed (see figure 5). Directly after the installation of the first fill medium soundings with measurements of local friction are executed in the test areas IV through X. In March 1975 we carried out control continuous borings Ø 29 mm next to the drains in the test areas IV, VI and VIII; in December 1976, 5 continuous borings Ø 66 mm were carried out in the test areas I through III, VI and VIII.

The laboratory investigation has existed i.a. of a big amount of oedometer and permeability tests with continuous registration during the first 400 to 1000 minutes.

The instrumentation in the field exists i.a. of settlement plates in all test areas, series of piezometers and observation wells in the test areas I through IX, ground-pressure boxes in the test areas I through III and vertical bellowhoses in the test areas II, III and VI.

## 7. JUSTIFICATION OF THEORETICAL CALCULATIONS

Based on the described layering with the soil properties per layer settlement calculations are executed for the loading in steps.

The plots of the oedometer tests are used as test on scale for the appointment of the direct and secular settlement per layer.

For the calculation of the hydrodynamic period we used the α/k-values, taken from the permeability tests, in which α and $k_{v,h}$ are appointed in the consolidated situation. These α/k-values appear to increase strongly with higher loadings for peat, clayey peat and peaty clay.

Based on the formula

$$t_{ev} = \left(\frac{H}{2}\right)^2 \cdot T_v \cdot \frac{\alpha}{k_v} \cdot \gamma_w$$

and accounting the increasing of the α/k-value during the consolidation period we will get an unfavourable value of $t_{ev}$.

The registrated course of the settlement in the laboratory tests during the hydrodynamic period justifies the use of a reduction-coefficient, for which we have taken a value diminishing with the increase of the loading from 2 to 1.8. Using the appointed $\frac{\alpha}{k_v}$ -values per layer the vertical hydrodynamic period is calculated per step of loading with a composed layer computation.

In the case without drains the time-settlement curve after Terzaghi-Fröhlich was the base for the calculation of the course of the settlement in the time. We assume that this curve applied to the calculated direct settlement and that the secular settlement will occur from the first day.

So the settlement on day $t_x$ is:

$$Z_{tx} = \frac{U}{100} \cdot Z_{direct} + Z_{secular} \cdot \log t_x$$

(Terzaghi-Koppejan).

For the stepwise loadings the settlement curve is calculated per step and added to the preceding settlement curve.

In the case of application of drains the calculation of the horizontal hydrodynamic period $t_{e_h}$ and the course of the direct settlement caused by consolidation only in horizontal direction we based on a combination of the formulas of Barron:

$$t_{eh} = \frac{D^2}{c_h} \cdot T_h \qquad (1)$$

and of Kjellman:

$$t_{eh} = \frac{D^2}{8c_h} \left[\ln \frac{D}{d} - \frac{3}{4}\right] \cdot \ln \frac{100}{100 - U} \qquad (2)$$

these give::

$$T_h = \frac{1}{8}\left[\ln\frac{D}{d} - \frac{3}{4}\right]\ln\frac{100}{100 - U} \qquad (3)$$

If:

$$\frac{1}{8}\left[\ln\frac{D}{d} - \frac{3}{4}\right] = \frac{1}{B}, \text{ then } \frac{U}{100} = 1 - e^{-B \cdot T_h}$$

$$\text{From (1) } T_h = \frac{c_h \cdot t}{D^2}, \text{ so } \frac{U}{100} = 1 - e^{-B \cdot \frac{c_h}{D^2} \cdot t}$$

$$c_h = \frac{1}{\frac{\alpha}{K_h} \cdot \gamma_w} \text{ and } A = -\frac{B}{\gamma_w \cdot D^2} \text{ gives}$$

$$\frac{U}{100} = 1 - e^{A \cdot \frac{k_h}{\alpha} \cdot t} \qquad (4)$$

$$\Delta Z = \frac{U_\alpha}{100} \cdot Z_{direct} = (1 - e^{A \cdot \frac{k_h}{\alpha} \cdot t}) \cdot Z_{direct} \qquad (5)$$

Further, a combined curve of the direct settlement is constructed with the results of the settlement course for the vertical and horizontal consolidation period per load step with the basic formula $\bar{u}_c = \frac{\bar{u}_v \cdot \bar{u}_h}{100}$ at different moments, where $\bar{u}$ = percentage excess pore water.

To design a comparative calculation with regard to the sand drain we computed an aequivalent diameter for both fabric-drains in view of the form.

## 8. COMPARISON OF CALCULATIONS AND MEASUREMENTS

Basic fact for all computations, except that for test area III, is the system of layering as given in boring IIB, as data from laboratory tests for the tests areas IV through IX are not available. This simplification appears acceptable for the greater part of the test areas.

- In test area V possibly the influence of the nearby cunet, is one of the causes of the accelerated settlements, while also deviations in the subsoil can influence.

In test area IX, where settlements show clearly a delay in regard to the comparable areas, presumably deviations in the layering system turn the scale.

- Taking into account the performed activities - the fillings 1 through 4 and the installation of the drains - we can divide the measured consolidation time in five periods.

When comparing it appears that during the first 40 to 50 days in all areas the measured settlement clearly delays to the calculated settlement, however, in test area II it seems less.

In the next table 1 the measured (M) and calculated (C) settlements, divided per period for the periods 2 through 5, speak for themselves.

| | | Install. of drain - fill 2 | | fill 2 - 3 | | fill 3 - 4 | | fill 4 till 620 d | |
|---|---|---|---|---|---|---|---|---|---|
| | | M | C | M | C | M | C | M | C |
| II | s.d.-3 m | 17 | *34* | 57 | *59* | 30 | *28* | 23 | *17* |
| VII | G - 2 m | 31 | *39* | 64 | *63* | 30 | *24* | 24 | *18* |
| VI | G - 3 m | 27 | *31* | 64 | *61* | 25 | *25* | 30 | *17* |
| IV | G - 4 m | 20 | *33* | 55 | *50* | 29 | *24* | 34 | *20* |
| IX | C - 2 m | 28 | *49* | 72 | *64* | 23 | *25* | 29 | *17* |
| VIII | C - 3 m | 22 | *36* | 67 | *61* | 33 | *28* | 21 | *18* |
| IX | C - 4 m | 21 | *36* | 69 | *54* | 33 | *26* | 37 | *18* |
| III | no drains | 14 | *13* | 54 | *45* | 20 | *17* | - | - |

Table 1: settlement per period in cm.

Curves showing a pertinent delay to the calculated figures in the second period the measured settlements in the third and fourth period are about equal or a little bigger and at last exceed abundantly the calculated settlements in the fifth period.

Differences in the executed loading steps cause deviations in the tendency connected to the influence of the c.o.c.-distances; this is visible in the calculated curves for geodrain c.o.c. 2, 3 and 4 m (figure 6).

Comparison of the measurements of these 3 areas (figure 7) shows the influence of the c.o.c.-distance on the course of settlement.

The deviating tendency in test area V makes the effect of the c.o.c.-distance for the A.V. Colbond-drain untransparant in figure 8, however, in table 1 it can be demonstrated for the 2 and 3 m distances.

Comparison of the different types of vertical drain for the 3 m-distance (figure 9) proves that:

1. all drains are effective;
2. the relative differences are small, however, the sand drains show at the end of the period a little delay to the fabric-drains, perhaps partly caused by differences in the subsoil.

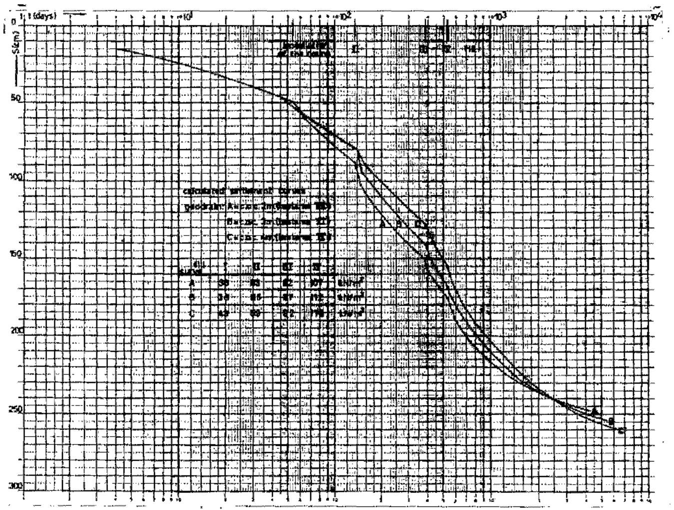


Figure 6

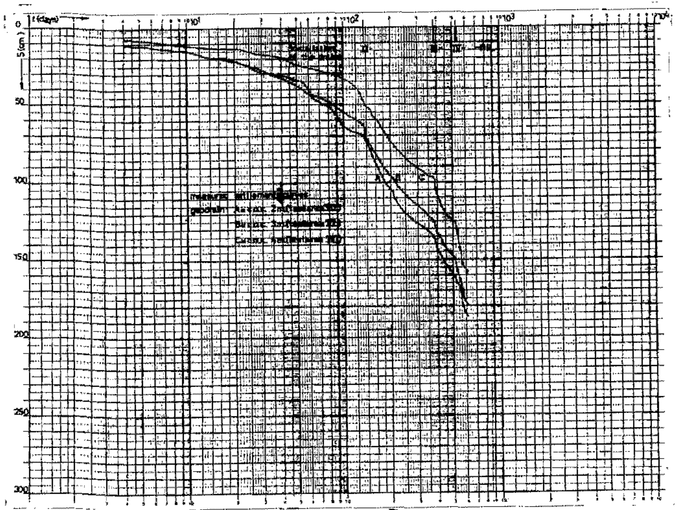


Figure 7

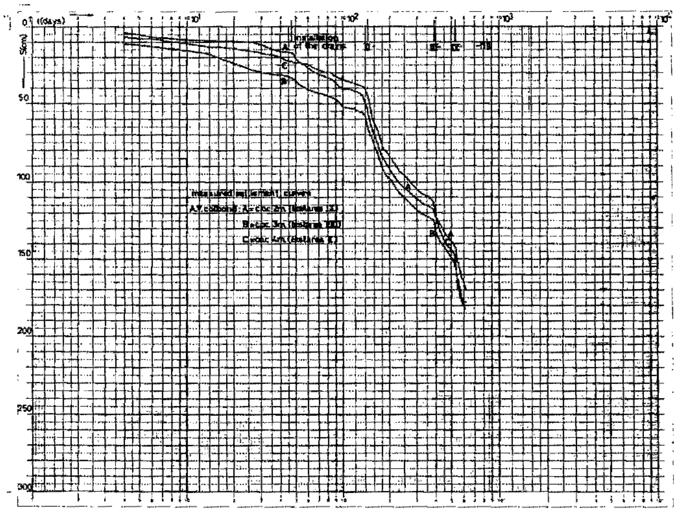


Figure 8

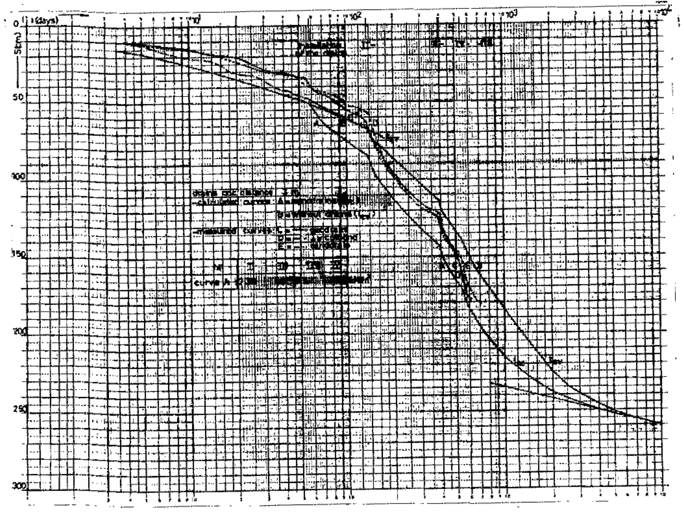


Figure 9

## 9. CONSOLIDATION OD EXCESS PORE PRESSURE BETWEEN THE DRAINS

In test area VI (geodrains c.o.c. 3 m) five piezometers are installed on different distances between two drains with the filter on the same level of 6.5 m - N.A.P. in the more or less clayey peat layer in the beginning of April 1976, just before the performance of the fourth fill.

In figure 10 the measured excess pore pressure is drawn on two dates, respectively 21 days (11-5-1976) and 152 days (16-9-1976) after the installation of the fourth fill. The constructed lines are the calculated curves for these dates. These curves are calculated with the theory of Barron (1). The basic values are represented next to the figure.

From the studies of the measured and calculated development of the excess pore pressure between the drains we conclude that the applied fabric drains still function about two years after installation.

To answer the questions if theory fits in with the measurements and if the chosen aequivalent diameter is correct requires further studies and a longer period of measuring.

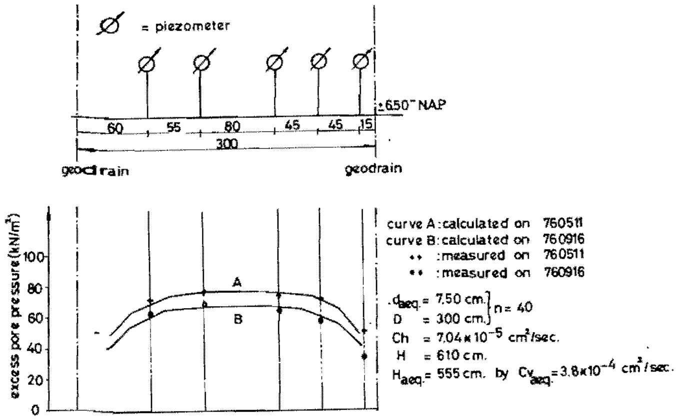


Figure 10

## 10. EVALUATIONS

1. Respecting the increasing value of $\frac{\alpha}{K}$ with higher loads we had to apply in the computation of the

hydrodynamic period a reduction coefficient (r.f.) essentially an artifice.
In our opinion we have to aim for future calculations to a solution of the formula:

$$t_{e_V} = (\frac{H}{2})^2 \cdot T_V \cdot \gamma_w \cdot \int_o^{t_e} \frac{d\frac{\alpha}{K}}{dt} \cdot dt \text{ instead of}$$

the formula $t_{e_V} = (\frac{H}{2})^2 \cdot T_V \cdot \frac{\alpha}{K} \cdot \gamma_w \cdot \frac{1}{r.f.}$ .

A different method of calculation can be developed by application of the values of the registrated consolidation period in laboratory tests on scale.

2. Basic for the computation of the settlement course for the vertical consolidation period is the curve of Terzaghi-Fröhlich.
Studying the registrated course of settlement in a big amount of oedometer and permeability tests does suppose that considerable deviations in regard to the above-mentioned curve appear.
We have taken into account that in the results of the registrated course the secular settlement has to be eliminated.
Combined with results of laboratory tests from the control-borings of December 1976, not yet available, further research and calculations will be reported later.

3. Caused by lack of time we have to drop a comparison of the measured settlements and excess pore pressure on several points in the consolidation period. A first impression is that in the soft layers the course of settlement passes quicker than the reduction of the excess pore pressure.

4. The assumption $d_{aeq} = \frac{\text{perimeter drain}}{\pi}$ has to be studied; $d_{aeq}$ possibly also depends on i.a. the horizontal and vertical drainage capacity and the procedure of installation.

5. In the described test areas only 3 types of vertical drains are tested. In the meantime other types are and more types will be deliverable. To analyse the quality of all drain types with field tests is very expensive. To limit these field tests, therefore, we have to search for a number of standard laboratory tests to check the quality.
The drain type should be inspected on:
- horizontal and vertical drainage capacity, strength, durability and persistancy against chemical and bacteriological influences.

NOTATIONS

$\alpha$ = coefficient of compressibility ($cm^2$/kgf)
c = cohesion ($N/mm^2$)
c' = coefficient of volume compressibility:
 $c'_p$ = directly; $c'_s$ = secondary
C = coefficient of consolidation ($cm^2$/sec)
 $C_{v,h}$ = for vertical resp. horizontal flow
$\gamma$ = unit mass ($t/m^3$) .
D = distance between the drains (cm)
d = diameter of the drain (cm)
$d_{aeq}$ = $\frac{\text{perimeter of the drain}}{\pi}$ = equivalent diameter of the drain (cm)
$\phi$ = angle of internal friction ($^0$)
H = thickness of the soft layers (cm)
k = coefficient of permeability; (cm/sec)
 $k_{v,h}$ = in vert. resp. hor. direction
$m_v$ = coefficient of compressibility ($cm^2$/kgf)
n = a ratio = $\frac{D}{d_{aeq}}$
N.A.P. = Normal Amsterdam Level
$\Delta p$ = loadstep ($kN/m^2$)
r.f. = reduction factor
s = settlement (cm)
t = time (sec)
 $t_e$ = hydrodynamic period
  $t_{e,v}$ = for vertical flow
  $t_{e,h}$ = for horizontal flow
T = time factor
 $T_{h,v}$ = for hor. resp. vert. flow
$\bar{u}$ = excess pore water pressure (%)
 $u_{v,h}$ = due to vert. resp. hor. flow
U = average consolidation (%)
 $U_{v,h}$ = due to vert. resp. hor. flow
$W_o$ = initial water content

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