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.

Acknowledgements. The author wishes to express his thanks to the managing director of the Laboratory of Soil Mechanics, Delft. The Netherlands, ir. T. K. Huizinga and its staff, for assistance in many respects and to ir. N. Nanninga for his help and valuable remarks.

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

ir. J. C. N. Ringeling, Engineer of the "Rijkswaterstaat" direction "Wegenverbetering"

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).

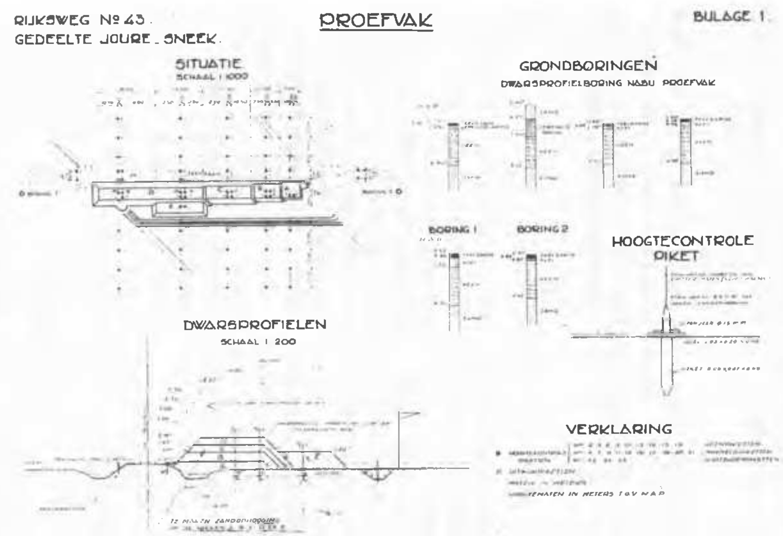
The relation between loads and computed settlements were plotted in Fig. 4, while the sand heights belonging to these loads have been plotted above the settlements, so that immediately can be seen, which settlement can be expected if a certain height of sand is applied. The expected end-settlements in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

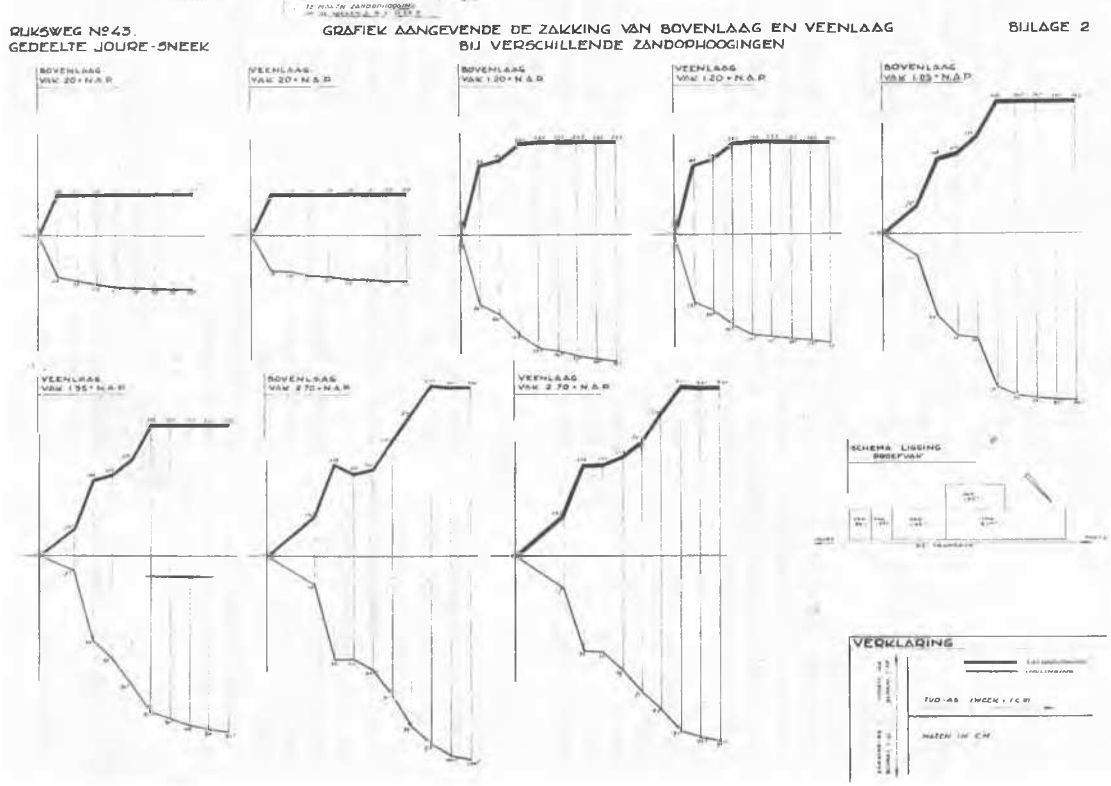
It appears that the settlements of the experimental road section do not agree with the computed settlements, they are lower by lower loads and greater by higher loads than the computed ones.

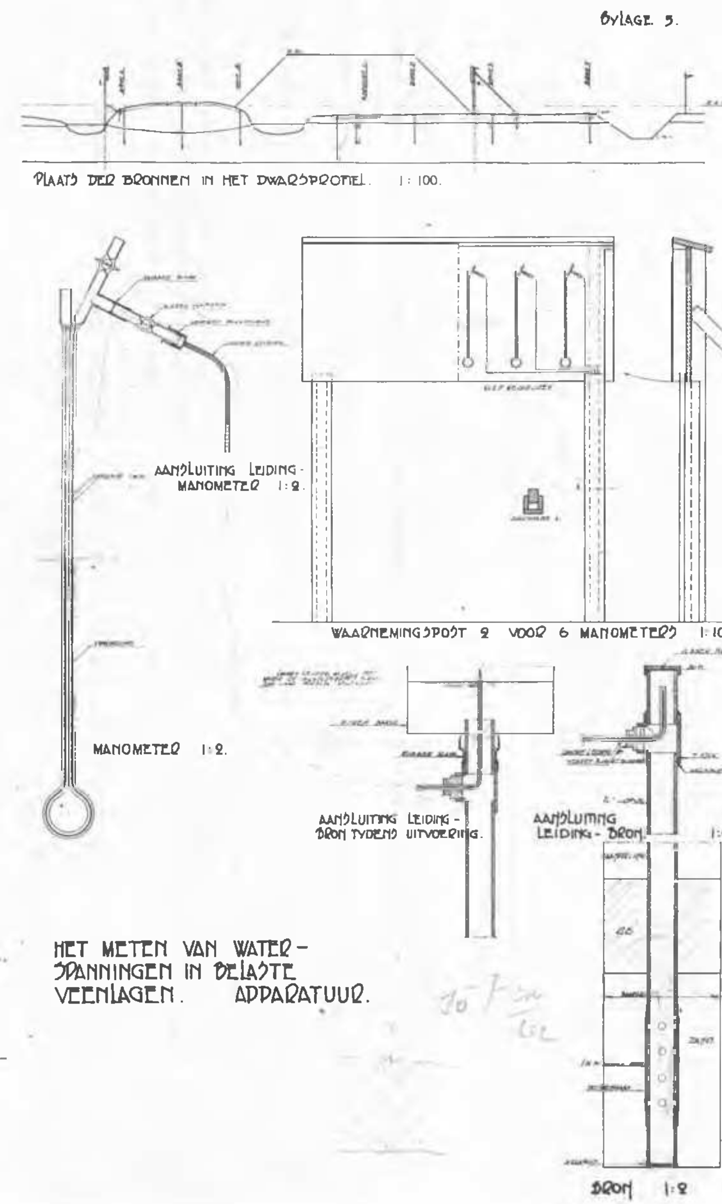
Even appears that the settlements of the experimental section rather well agree with settlements computed with the premise, that the peat follows the law of Hooke ($E = 0.8$ kg/cm^2).

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.


RIJKSWEG No 43.
GEDEELTE JOURE - SNEEK.
PROEFVAK
BIJLAGE 1.
SITUATIE
SCHAAL 1:1000
GRONDBORINGEN
DWARSPROFIELBORING NABIJ PROEFVAK
BORING 1
BORING 2
HOOGTECONTROLE PIKET
DWARSPROFIELEN
SCHAAL 1:200
VERKLARING


RIJKSWEG No 43.
GEDEELTE JOURE - SNEEK
GRAFIEK AANGEVENDE DE ZAKKING VAN BOVENLAAG EN VEENLAAG BIJ VERSCHILLENDE ZANDOPHOOGINGEN
BIJLAGE 2
BOVENLAAG VAK 20 + N.A.P.
VEENLAAG VAK 20 + N.A.P.
BOVENLAAG VAK 1.20 + N.A.P.
VEENLAAG VAK 1.20 + N.A.P.
BOVENLAAG VAK 1.95 + N.A.P.
VEENLAAG VAK 1.95 + N.A.P.
BOVENLAAG VAK 2.70 + N.A.P.
VEENLAAG VAK 2.70 + N.A.P.
SCHEMA LIGGING PROEFVAK
VERKLARING
TIJD-AS 1 WEEK = 1 CM
MATEN IN CM


BYLAGE 5.
PLAATS DER BRONNEN IN HET DWARSPROFIEL. 1:100.
AANSLUITING LEIDING - MANOMETER 1:2.
WAARNEMINGSPOST 2 VOOR 6 MANOMETERS 1:10.
MANOMETER 1:2.
AANSLUITING LEIDING - BRON TIJDENS UITVOERING.
AANSLUITING LEIDING - BRON 1:2
BRON 1:2
HET METEN VAN WATER-SPANNINGEN IN BELASTE VEENLAGEN. APPARATUUR.

F-8

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In order to measure exactly, no water may be withdrawn from the layer, therefore a capillary mercury-gauge was used. The gauges could not be fixed directly on the wells, therefore a lead line was fixed between the wells and the gauges, which could follow the settlements of the road embankment sufficiently.

The wells existed of a $\frac{3}{4}$" gaspipe, closed at the bottom and at the bottom end provided with holes over a length of ± 10 cm. Hereover some windings of galvanised steel wire are fixed and thereover fine wire-netting was welded. The wells were placed in a hole bored with a screw-borer. The filter was formed by sand and the lower part of the well. This sand was covered with a watertight clay layer. Further details give Fig. 5.

Results of the measurings. Some results are plotted in the Figures 6 and 7. In Fig. 6 the observations of the wells are plotted. Generally a fast increasing and afterwards a slow decreasing of the pressure can be observed.

The increasings agree with the applying of a new load, while the pressure between two loadings decreases.

The wells sunk with the field, so that in the end the pressure in the middle of the peat layer was no more measured.

In the end differences were hardly found.

In Fig. 7 the load at a certain time is given by a dotted line and the sum of all pressure-increasings in consequence of this load by a full line. This shows that immediately after loading about 75% of the load is taken up by the water. A sideward divergence of the water pressures can be observed.

It is remarkable that in spite of the small permeability of the peat, water pressures decrease practically immediately.

Translation of notations in figures.

Fig. 1:

Proefvak	experimental road section
situatie	situation
grondboringen	borings
dwarsprofielen	cross sections
hoogte controle piket	height control peg
trambaan	tramway road
bovenkant rail	top rail
dwarsprofiel aarden baan v.d.toekomst.weg	cross section of planned road
uitwijkpiketten	lateral displacement pegs
teelaarde	mould
klei	clay
veen	peat
zand	sand
zandophooging	sand tipping

Fig. 2:

Grafiek aangevende de zakking van de bovenlaag en veenlaag bij verschillende zandophoogingen	record of settlements of the mould and moor layer under different sand-tippings
bovenlaag	upper layer
veenlaag	peat layer
vak	section
zandophooging	sand-tipping
inklinking	settlement

Fig. 3:

zakkingsberekening	calculation of the settlement
teelaarde + klei	mould and clay
veen	peat
zand	sand
belasting	load

Fig. 4:

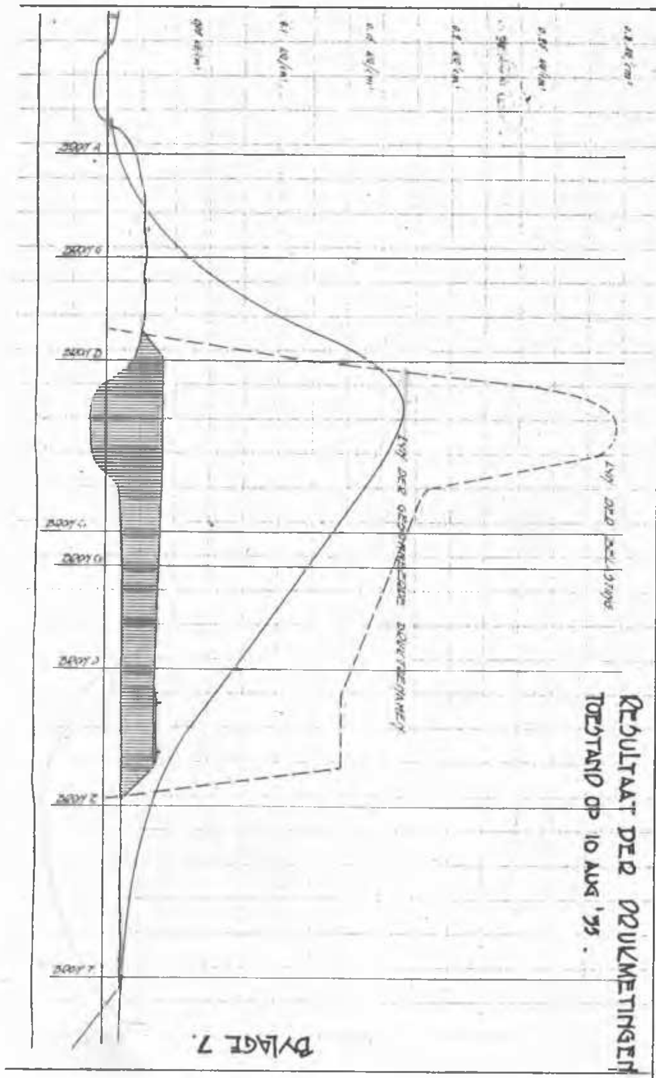
uiteindelijke hoogte boven maaiveld	final height above soil surface
belasting	load
grondwater	groundwater
diepte v.d.inzinking	settlement
zakking trambaan	settlement tramway road
proefvak	experimental road section
uiteindelijke zakking proefvak met inachtname der tijdzakkings	final settlement of the experimental road section derived from time-settlement-diagram

Fig. 5:

Het meten van waterspanningen in belaste veenlagen	the measuring of water pressures in loaded peatlayers
plaats der bronnen in het dwarsprofiel	situation of the wells in the cross section
aansluiting leiding manometer	connection between lead-line and manometer
aansluiting leiding-bron tijdens uitvoering	making the connection between well and lead-line
aansluiting leiding-bron	connection well and lead-line
klei	clay
zand	sand
gelascht	welded

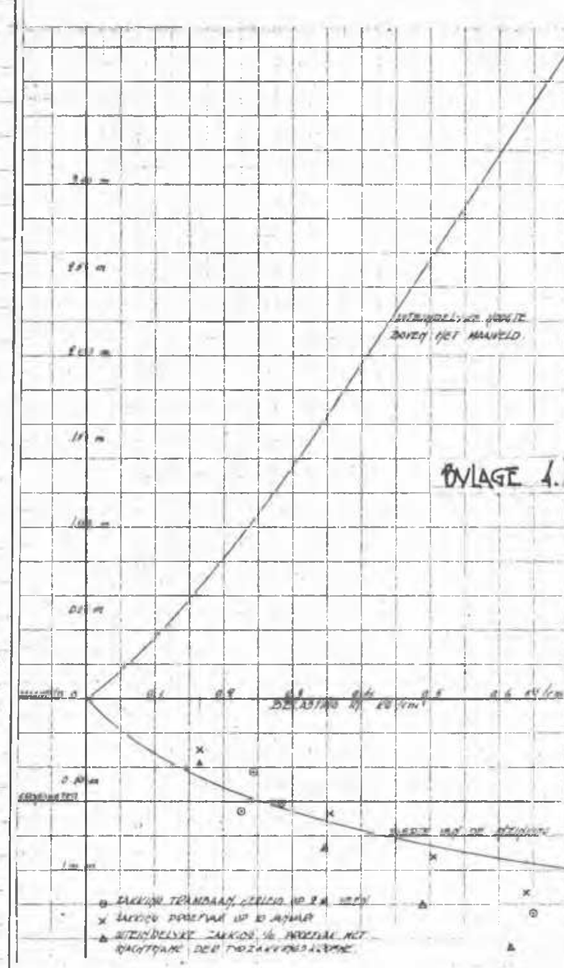
Fig. 6:

Bron	well
waterdruk	waterpressure
na deze tijd zal het filter zich in de zandlaag bevonden hebben. De druk is dus daar gemeten.	after this time the filter will have been situated in the sand layer. So the pressure is measured there.



RESULTAAT DER DRUKMETINGEN
TOESTAND OP 10 AUG '55.

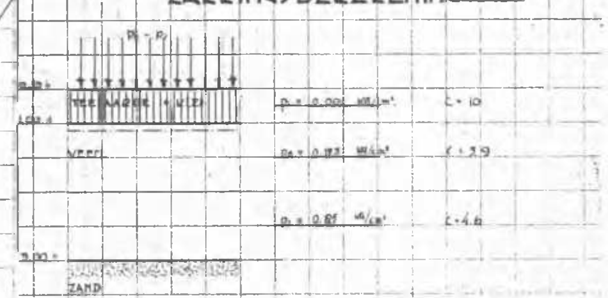
BYLAGE 7.



BYLAGE 4.

ZAKKINGSBEREKENING

BYLAGE 5



BELASTING 0.105 kg/cm²

LAAG	h	c	p_1	p_2	ln p_2/p_1	Δh
TEELAARDE + KLEI	0.5	10	0.095	0.2	0.755	0.038
VEEN	1.0	3.9	0.125	0.23	0.610	0.156
VEEN	1.0	4.6	0.125	0.23	0.610	0.132
					TOTAAL	0.33 m

BELASTING 0.255 kg/cm²

LAAG	h	c	p_1	p_2	ln p_2/p_1	Δh
TEELAARDE + KLEI	0.5	10	0.095	0.35	1.300	0.067
VEEN	1.0	3.9	0.125	0.38	1.110	0.285
VEEN	1.0	4.6	0.125	0.38	1.110	0.239
					TOTAAL	0.59 m.

BELASTING 0.365 kg/cm²

LAAG	h	c	p_1	p_2	ln p_2/p_1	Δh
TEELAARDE + KLEI	0.5	10	0.095	0.46	1.580	0.079
VEEN	1.0	3.9	0.125	0.49	1.370	0.352
VEEN	1.0	4.6	0.125	0.49	1.370	0.298
					TOTAAL	0.73 m

BELASTING 0.485 kg/cm²

LAAG	h	c	p_1	p_2	ln p_2/p_1	Δh
TEELAARDE + KLEI	0.5	10	0.095	0.58	1.800	0.090
VEEN	1.0	3.9	0.125	0.61	1.570	0.403
VEEN	1.0	4.6	0.125	0.61	1.570	0.342
					TOTAAL	0.84 m

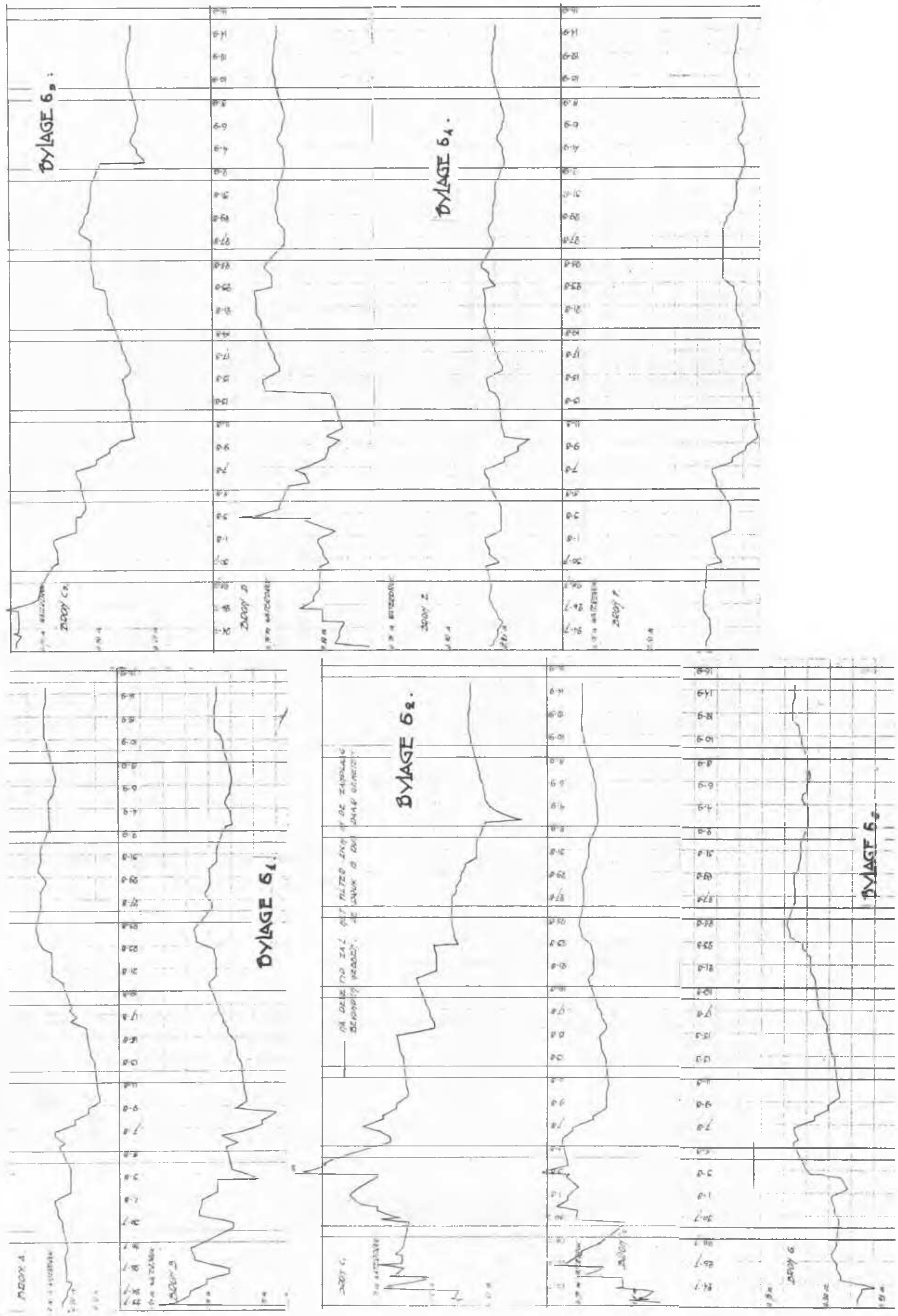


Fig. 7:

resultaat der drukmetingen	result of the pressure measurings
lijn der belasting	load-diagrams
lijn der gesommeerde druktoename	line of summed pressure increased

No. F-9

DIRECT MEASURING OF INTERNAL WATER PRESSURES IN CLAY

ir. C. Biemond, Chief Engineer of Public Works at Amsterdam

Geological survey. The Dutch soil, to the depths that play a sensible part in engineer's work, is very recent formation; it belongs to the holoceen and the young-pleistoceen. In Amsterdam the top of the pleistoceen layer of sand is found about 14 m under the surface at A.P. (A.P. if the Amsterdam Level and about equal to the average sea-level); above the mentioned layer there is a layer of d.m. thickness consisting of compressed peat, above which we find fine sand mixed with marine clay up to a level of 10 m ÷ A.P. Above that height the soil consists of loose layers only, marine clay with little sand up to 5 to 6 m ÷ A.P. and above that loose peat.

General description of construction. This paper treats mainly of works executed in the Amsterdam ports along the Dijksgracht, where the surface was 3 to 6 m under A.P. (Amsterdam Level) and the peat-layer therefore was mainly missing. This work was executed for the railways and consisting of the construction of a broad high embankment (7 m above A.P. - Amsterdam Level) adjacent to an existing low embankment and in the construction of a new high-level siding with a width of about 100 m. For the greater part these works were executed on virgin soil but in parts use was made of an old fill destined for industrial purposes. Fig. 1 shows the situation and Fig. 2 gives the most typical sections I - V and VI. To prevent side-slipping of the embankment the subsoil under the new toe was artificially improved by dredging out the loose subsoil and replacing this with sand. As the embankment on the other side rested on a former seadike more than 100 years old the core of the embankment still consisting of loose soil could be considered sufficiently encased from both sides. The phenomena under consideration can therefore be compared with a consolidation test but then on a much bigger scale.

The work was executed as hydraulic fill; first the subsoil-improvement was carried out; on this a dike was filled against the water and after that the whole embankment was raised to A.P. (Amsterdam Level) and then raised section by section and layer by layer to 7 m above A.P.

Kinds of soil encountered. Fig. 3 shows grain-size of 9 different soil-samples. These samples were taken by screw-borings; 5 of them come from the Dijksgracht and 4 from the Watergraafsmeer. These samples are in complete mutual agreement and also agree with the surmise, based on our geological knowledge of this district, that a depth of 6 - 7 and 9 - 10 m under A.P. we should find layers of fine-grained clay whilst from 7 to 9 m under A.P. there should be a somewhat greater mixture with sand. The samples shown were taken at:

f	- 6.30	÷ A.P.	- Watergraafsmeer
a	- 7.00	"	- Dijksgracht
g	- 7.30	"	- W
b	- 7.50	"	- D
c	- 8.30	"	- D
h	- 8.40	"	- W
d	- 8.80	"	- D
i	- 9.40	"	- W
e	-10.00	"	- D

The physical properties of soil samples are shown in Table I and the paper will further give a description of how these figures were arrived at. A rational analysis was made of different fractions of sample h with the results, shown in Table II.

Direct measuring of internal water pressure. At various places in the clay layers at 6 to 11 m ÷ A.P. $1\frac{1}{2}$" pipes were sunk ending in filterheads composed of about $\frac{1}{2}$ m perforated pipe encased in fine copper-gauze, which were surrounded by course sand reaching from $\frac{1}{2}$m under the filter to $\frac{1}{2}$ m above it, therefore over a length of $1\frac{1}{2}$ m. This sinking was done by boring down with a 6" pipe, which was gradually withdrawn from around the filter-head as the course sand was introduced. On further withdrawing the boring-pipe the opening above the filterhead was completely sealed with a layer of clay at least 1m thick. In some cases the same boring contained an additional filter at a higher level. During the growth of the embankment the water level in the ascending pipes from the filterheads was seen to rise. A new layer of sand of 1 to $1\frac{1}{2}$ m of thickness caused the water level in the pipe to rise 2 to 3 m the very same day, after which the internal pressure in the clay layers and therefore the level of the water decreased gradually. Fig. 4 to 9 show the pressure variation in various filter heads situated as shown in Fig. 1. The graphs further show the water level in the raised sand embankment and the compression of the sub-soil