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CONTROLLED DEWATERING TECHNIQUES FOR THE CONSTRUCTION OF THE BERENDRECHT SEALOCK AT ANTWERP

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ABSTRACT

The Berendrecht Sealock at Antwerp will be built in a dry open trench. The groundwater level needs to be lowered over about 25 meters. Such an important drawdown of the watertable will influence the groundwater level within a range of several hundreds of meters. The composition and the general structure of the subsoil is well known by the experience from the Zandvliet Sealock and further from soil tests and geological investigation.
Unfortunately in this area is found a very compressive peatlayer, three meters thick, in which the watertable drawdown will cause serious settlements.
As a refinery and several large storage tanks which are founded in this peatlayer are located in the immediate neighbourhood of the construction site, such settlements cannot be tolerated and special measures had to be taken.
These measures consist of the installation of a bentonite-cement diaphragm wall combined with artificial watertable refeeding.

In a first part the investigation of the geological and hydraulic soil constants will be discussed.
The second part describes the design, execution and control of the bentonite-cement diaphragm wall.
The third part describes the design and execution of the groundwater refeeding system. The execution of a preliminary refeeding test and the design of a permanent control system are discussed.
Finally the execution of the groundwaterlowerings for the divertment of a number of conduit-pipes which cross the future lock is described and the calculations with a mathematical model for the prevision of the final groundwaterlowering will also be discussed.

INTRODUCTION

Due to the increase and the differentiation of the traffic to the Antwerp Harbour it is necessary to enlarge the locking capacity in order to maintain a good accessibility to the inner harbour on the right bank of the River Scheldt.

Therefore the existing sealock Zandvliet situated ca 20 km downstream of Antwerp, will be doubled. The new sealock Berendrecht will be constructed under the direction of the Ministry of Public Works. The general location of the new sealock Berendrecht is given of figure 1 and a more detailed situation of the sealock is given on figure 2.

Fig. 1 LOCATION MAP

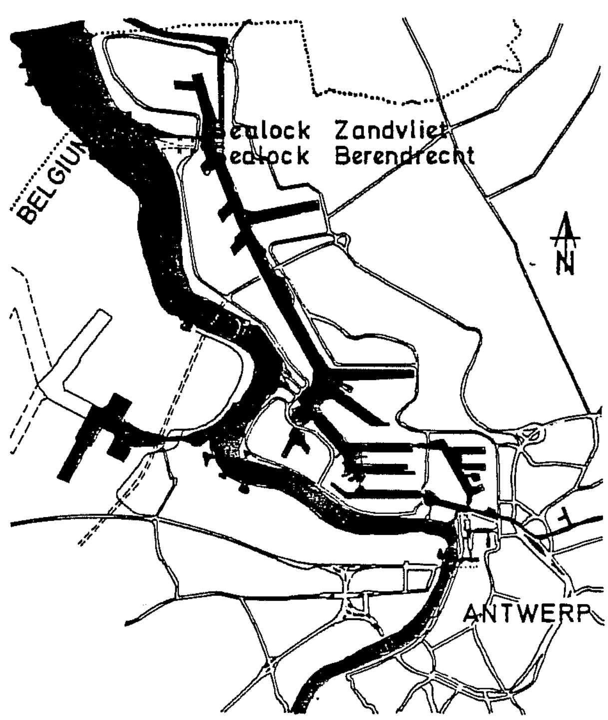
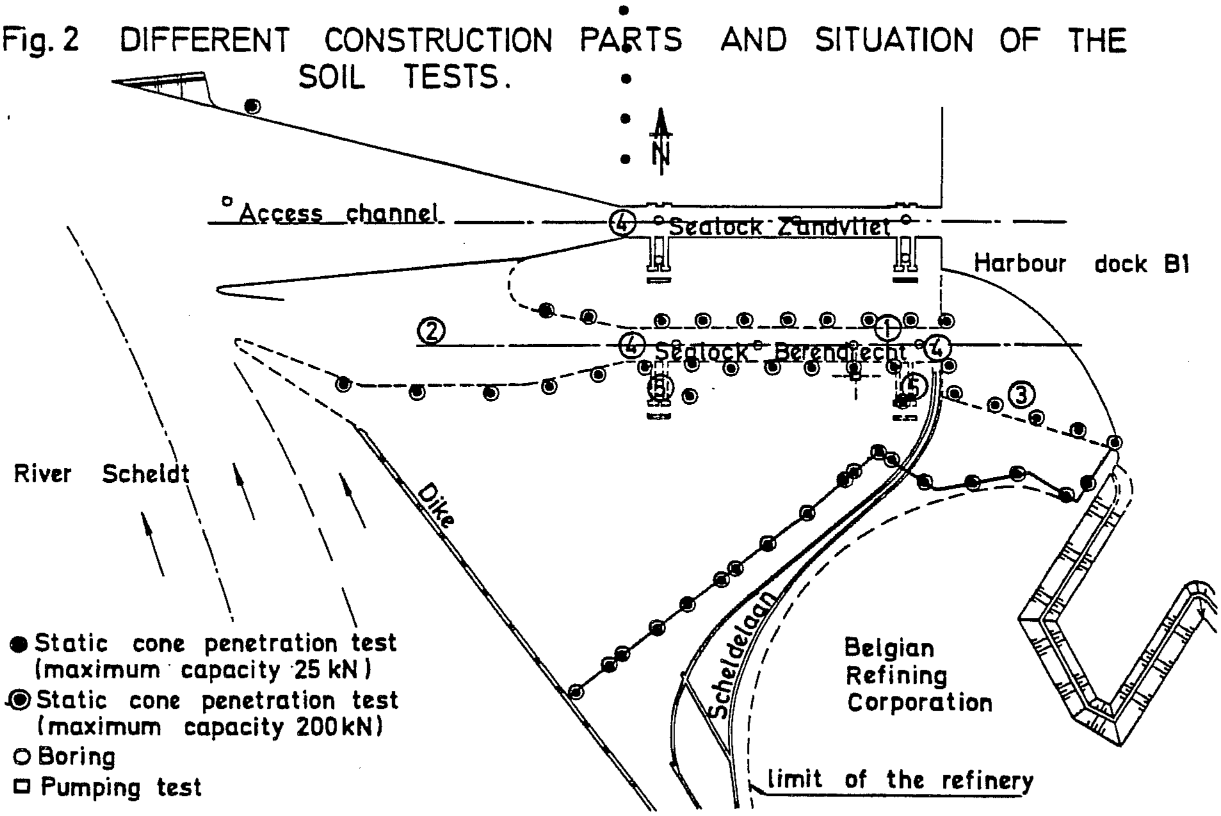


Fig. 2 DIFFERENT CONSTRUCTION PARTS AND SITUATION OF THE SOIL TESTS.



For the construction of this new lock an execution period of 4,5 years is provided. The project consists of:

1. The sealock itself.
2. The access channel to the River Scheldt with demolition of the southern quay walls of the present access channel to the existing sealock Zandvliet.
3. A quay wall on the dockside.
4. Metal bascule bridges: two over the new sealock Berendrecht and one over the existing sealock Zandvliet.
5. The lock doors.
6. The electromechanical equipment.
7. The dredging works.

The items 1 till 5 are indicated on the figure 2.

The sealock Berendrecht will be built in a dry open trench. The subsoil at the construction site consists of fills, Holocene deposits, an upper waterbearing stratum between ca. 11 and 31 m. depth, a semi-pervious layer between ca. 31 and 33 m. depth, and a lower waterbearing stratum between ca. 33 and 59 m. depth.

For the installation of a dry open trench, the groundwaterlevel needs to be lowered over about 25 meters. Furthermore, in order to avoid the bursting of the bottom of the excavation, the waterpressure within the lower waterbearing stratum needs to be lowered also considerably. Such an important drawdown of the watertable within both the upper and the lower waterbearing stratum will influence the groundwaterlevel within a range of several hundreds of meters.

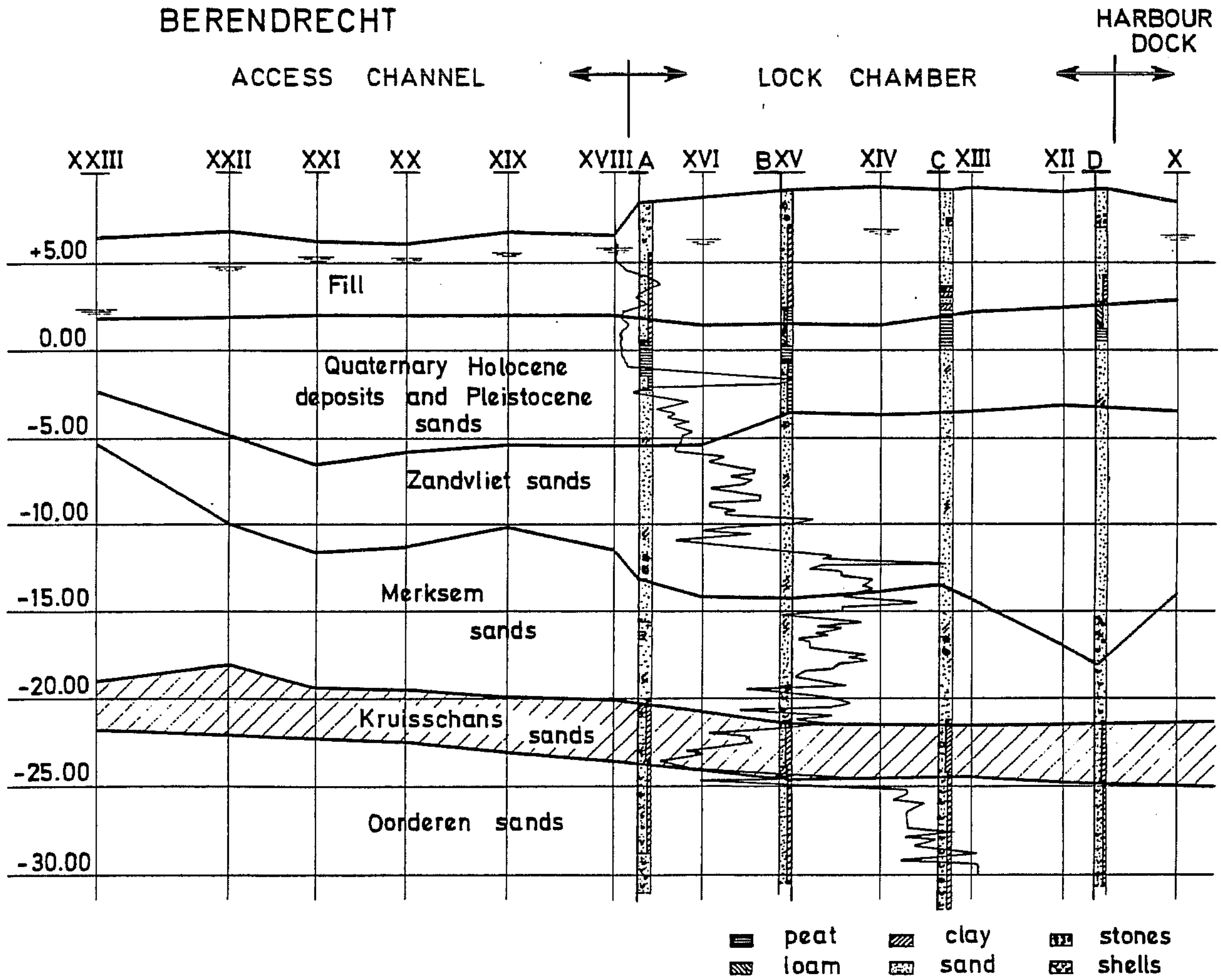
Due to the presence of the very compressible Holocene deposits, in which a thick peat layer is encountered, the watertable drawdown will cause serious settlements. As a refinery and several large storage tanks, which are founded on these Holocene deposits, are located in the immediate neighbourhood of the construction site, such settlements cannot be tolerated and special measures had to be taken. These measures consist of the installation of a bentonite-cement cut-off screen and of a groundwater refeeding system.

SUBSOIL CONDITIONS

From the different investigation campaigns performed for the construction of the existing sealock Zandvliet with its access channel and inner dock, the general composition of the subsoil at the spot of the new sealock Berendrecht was known. A succession of clayey and sandy layers is encountered.
Starting from these data a detailed site investigation has been planned. It consisted of 23 static cone penetration tests, of 4 borings to 40 and 60 m depth with prelevation of disturbed and undisturbed sampels, and of 2 pumping tests. Some additional tests were performed for the design of the roadway extension in the vicinity of the new sealock.

The location of the different tests is given on fig. 2. A schematic cross section along the new sealock Berendrecht is given on fig. 3.
At the spot of the new sealock the groundlevel is situated at ca + 9,00. Before the construction of the existing sealock in 1962, the groundlevel corresponded with the polderlevel and was situated at + 2,00 a + 3,00.
During the construction of the existing sealock and the extension of the inner harbour a 7 m thick fill layer has been installed.

Fig. 3 GEOLOGICAL SECTION ALONG THE SEALOCK BERENDRECHT



a) Geology of the site:

At the spot of the new sealock, the following layers are encountered:

- from ca + 9,00 to ca + 2,00:
  Fill, mainly installed during the construction of the existing sealock and the extension of the inner harbour.
  This fill layer is very heterogeneous due to the different adopted installation methods (in the dry and hydraulic) and to the type of the infilled materials.

- from ca + 2,00 to ca - 2,00:
  Quaternary Holocene deposits, consisting of the former polder deposits, a compact peat layer with a maximum thickness of 3 m and stratified clayey and sandy layers.

- from ca - 2,00 to ca - 22,00:
  Quaternary Pleistocene fine eolian sands, with a maximum thickness of 7 m, and Tertiary Upper Pliocene fine glauconitic sands (Zandvliet sands and Merksem sands), with sideritic sandstone concretions, and shells and sideritic clayey concretions in the lower part.

- from ca - 22,00 to ca - 24,00:
  Tertiary Upper Pliocene fine glauconitic sands (Kruisschans sands) with shells and numerous clay layers. The thickness of the clay layers generally varies between 1 and 1,5 cm, but may locally reach 20 cm. The vertical permeability, determined at the laboratory on five undisturbed sampels, varied between $1,49.10^{-7}$ and $8,31.10^{-9}$ m/s

- from ca - 24,00 to ca - 50,00:
  Tertiary Upper Pliocene sands (Oorderen sands), Tertiary Lower Pliocene (Kattendijk Formation) and Tertiary Miocene (Berchem Formation) fine to medium fine glauconitic sands, with shells dispersed in the sand and concentrated in different layers.

- underneath - 50,00:
  Tertiary Oligocene stiff fissured overconsolidated clay (Boom clay).

For the subdivision of the different layers, the litostratigraphical subdivision, given in a detailed study of F.J. De Meuter and P. Laga (1976) has been followed.

b) Hydrogeological conditions:

For the determination of the hydrogeological characteristics of the water bearing strata, encountered between the levels -2,00 and -22,00 and between the levels - 24,00 and -50,00, two discharging wells with a diameter of 25 cm have been installed at an intermediate distance of 5 m. Their filter sections were respectively situated between the levels - 2,00 and - 20,00 and between - 25,00 and - 45,00.
All necessary precautions were taken in order to limit the waterextraction to the layer, in which the filter was installed (see fig. 4 a).

After the installation of the discharging wells and piezometers, the waterlevel was measured within these wells and piezometers every half an hour during a period of 12 hours, in order to determine the influence of the tide within the Scheldt River on the groundwaterlevel within the different strata.

So it could be observed that at the spot of the pumping test, the waterlevel within the upper waterbearing stratum was only influenced by the waterlevel in the harbour dock. Within the lower waterbearing stratum the measured amplitude, due to the tide within the Scheldt River, was of ca 20 cm. The distance from the piezometers to the Scheldt River varied from 950 to 1150 m.

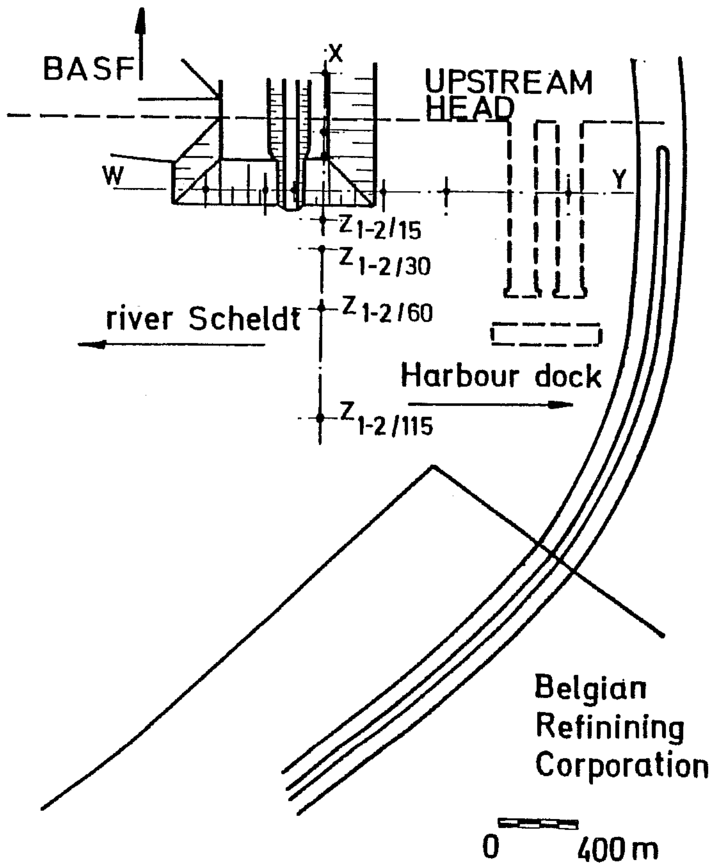
A first pumping test was performed in the sandy layers between the levels - 2,00 and - 22,00. During a period of 12 days, a rate of ca 70 m^3/h was extracted. The waterlevel in the different piezometers was measured systematically.

On the schematic cross section of fig. 4 b the waterlevels, observed within the two waterbearing strata at the end of this pumping test, are given.

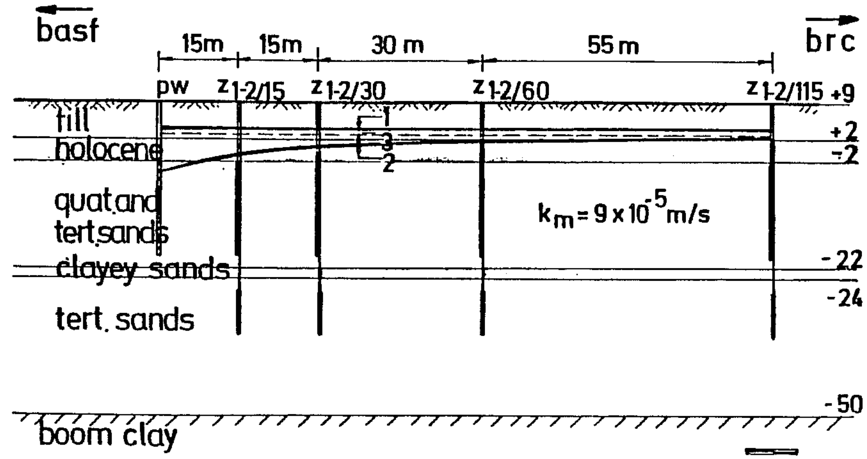
A second pumping test was performed in the sandy layers between the levels - 24 and - 50 ca 1 month after the end of the first pumping test. During a period of 9 days a rate of ca 66 m^3/h was extracted. Again the waterlevel in the different piezometers was measured systematically.
On the schematic cross section of fig. 4 c the waterlevels, observed within the two waterbearing strata at the end of the latter pumping test, are given.

FIG. 4 PUMPING TESTS

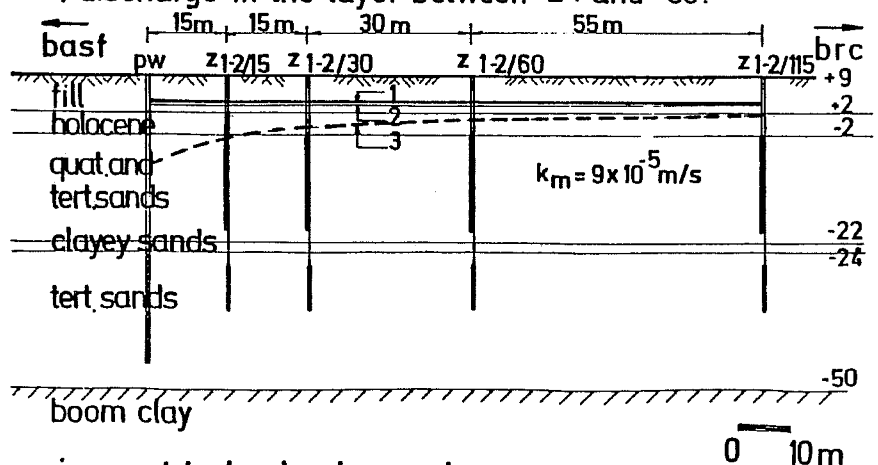
a) situation



b) discharge in the layer between - 2 and - 22.



c) discharge in the layer between -24 and -50.



piezometric levels observed:

1. at rest.
2. in the upper waterbearing stratum at the end of the test.
3. in the lower waterbearing stratum at the end of the test.

From the results of these pumping tests the following informations have been deduced:

Groundwaterlevels at rest within the different strata:

- In the fill hanging watertables are encountered. Locally waterlevels between + 7,50 and + 7,15 were observed.
- In the sandy layers between the levels -2,00 and -22,00: due to external influences (variations of the dock level), the waterlevel varied between + 4,00 and 4,50.
- In the sandy layers between the levels - 24,00 and - 50,00: due to external influences (variations of the dock level and tide within the Scheldt River), the groundwaterlevel varied between ca + 4,00 and + 4,40.

Hydrogeological constants:

- Hydraulic resistance of the Holocene deposits between + 2,00 and - 2,00:

  $c = \frac{d}{k} = 750$ days.

- Horizontal permeability of the water-bearing stratum between - 2,00 and -22,00:

  $k = 7{,}9.10^{-5}$ a $1{,}04.10^{-4}$ m/s

  (mean value $k_m = 9.10^{-5}$ m/s)

- Hydraulic resistance of the clayey layers between - 22,00 and - 24,00:

  $c = \frac{d}{k} = 400$ a 460 days

  (mean value c = 430 days).

  or $k_v = 5{,}06$ a $5{,}79.10^{-8}$ m/s

- Horizontal permeability of the water-bearing stratum between - 24,00 and - 50,00: $k = 4{,}88$ a $6{,}86.10^{-5}$ m/s

  (mean value $k_m = 5{,}9.10^{-5}$ m/s)

BENTONITE-CEMENT DIAPHRAGM WALL

a) Design:

In order to prevent an excessive lowering of the groundwaterlevel underneath the nearby refinery, a bentonite-cement diaphragm wall and an artificial watertable refeeding system had to be provided. The position of the bentonite-cement diaphragm wall, given on fig. 5, has been chosen to minimize the lenght of the wall, though assuring a safe distance to the construction wharf.
The bentonite-cement diaphragm wall has been continued over ca 150 m along the dock in order to limit the waterflow around the extremity of the cut-off screen.

Without any doubt the best solution would have consisted of the installation of a bentonite-cement screen reaching the impermeable Boom Clay layer, situated at a depth of ca 60 m.
As however the unit price of a bentonite-cement screen increases very rapidly with its depth, this solution is very uneconomic. Therefore it was decided to limit the bentonite-cement screen at the level - 25,00 which is 1 meter beneath the clayey layers, encountered between the levels - 22 and - 24. So the bentonite-cement screen reaches a depth of ca 34 m.

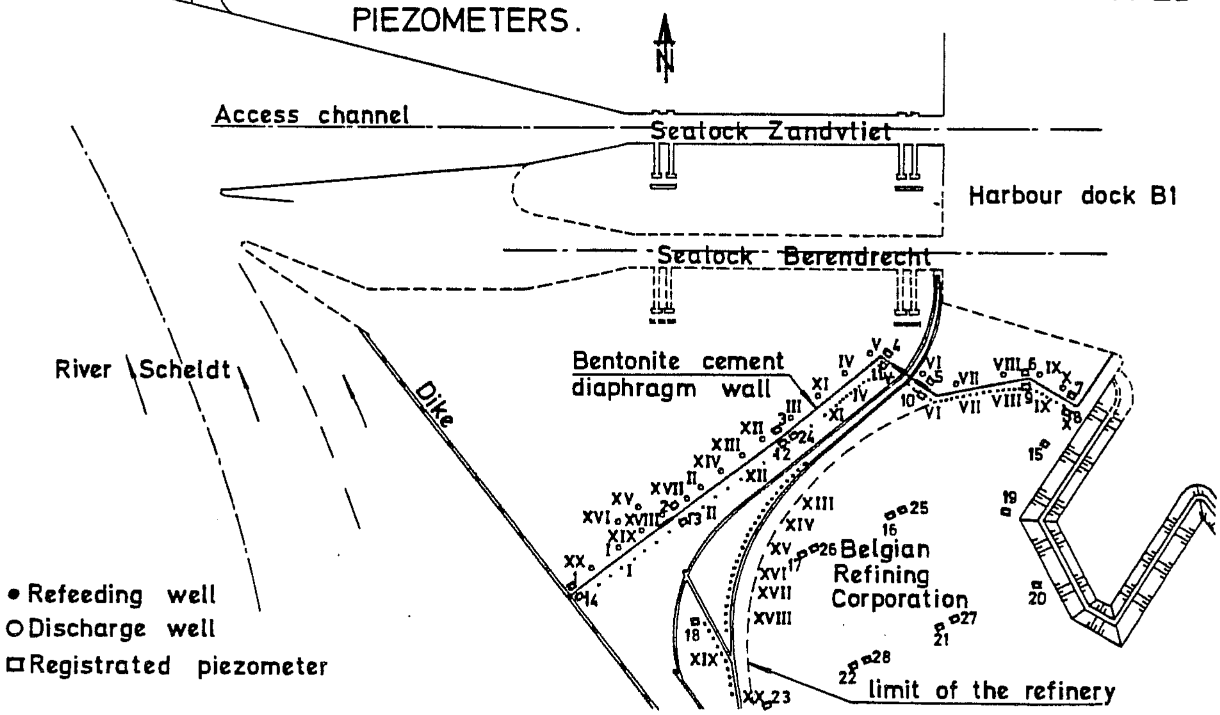
The bentonite-cement screen has only a cut-off function, and will never be sollicitated by horizontal forces.
Therefore it would have been possible to adopt the minimal available width of 0,60 m. As however the screen had to be installed to a rather large depth through relatively resistant layers, it seemed preferable to impose a width of 0,80 m. In this way the risk of discontinuities in the joints between the adjacent panels is decreased.

Based on former experience (BERLEUR et al, 1981) a permeability of the screen of 10^{-7} m/s has been imposed.
An extensive control system has been provided, consisting of the systematic control of the characteristics of the used bentonite-cement slurry and of regular measurements of the groundwaterlevel behind the bentonite-cement screen, in order to determine the location of eventual discontinuities.

b) Execution:

Before starting the execution of the bentonite-cement screen, 15 static cone penetration tests have been performed at intermediate distances of ca 100 m in order to determine the exact depth of the clayey layers encountered between the levels - 22 and - 24.

Fig.5 SITUATION OF THE BENTONITE-CEMENT DIAPHRAGM WALL, GROUNDWATER REFEEDING SYSTEM AND REGISTRATED PIEZOMETERS.



The results of these tests have been determinative for the depth of the bentonite-cement screen. During the excavation of the different panels it has systematically been controlled that the clayey layers were encountered and the depth of the screen has been adapted continuously to reach a depth of ca 1 m beneath the lower limit of the clayey layers.

In order to determine an adequate composition of the bentonite-cement slurry, different mixtures have been tested at the laboratory. These mixtures were prepared with different quantities and types of blast-furnace cement and portland cement and with different quantities and types of bentonite. Based on the results of these tests the following mixture was proposed:

35 kg Bentonite CV 15
200 kg Blast-furnace cement LK 30
925 l water

During the excavation of the first panels some difficulties occurred concerning the stability of the bentonite-cement slurry within the trench.

An excessive water-loss was observed. Comparative tests on mixtures, prepared with tap water and groundwater showed very divergent results. So new tests have been performed on different bentonite-cement mixtures, prepared with groundwater. Finally the following mixture has been used:

35 kg Bentonite CEBO
150 kg Blast-furnace cement LK 30
925 l water

The excavation of the panels has been performed with cable-suspended grabs. The grabs were equiped with skirts of ca 3 m height for the guidance of the grab. The excavation was made between guide walls.
A discontinue excavation sequence has been followed.
The length of the primary panels was of 2,80 m and the length of the secondary panels of 2,20 m. The time interval between the excavation of the primary and secondary panels was of ca 4 days. The excavation rate varied between 7 and 14 m^2/h.

1.170 c) Control of the bentonite-cement slurry:

For the control of the bentonite-cement slurry on the site a field laboratory has been installed and the slurry has been sampled systematically:

- just after leaving the bentonite-cement mixer;
- at the top of the trench;
- at the mid-height of the trench;
- at the bottom of the completed trench.

Following characteristics of the slurry were determined:

- volumic weight γ
- Marsh viscosity
- filter loss and cake thickness
- pH

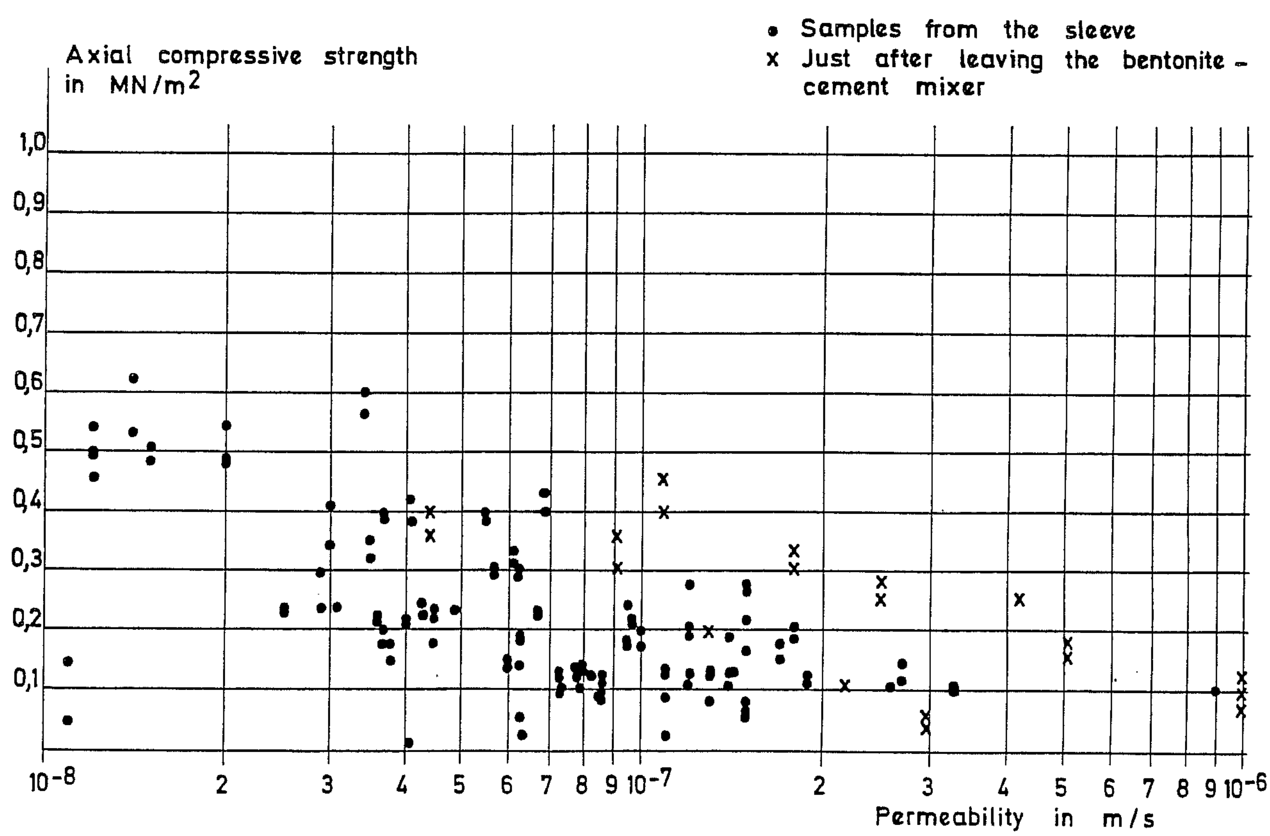
Furthermore samples with a diameter of 10 cm and a height of ca 10 cm have been formed with the slurry taken (only for a certain number of panels) just after leaving the bentonite-cement mixer, and at the top, mid-height and bottom of the trench.

These samples were conserved under water for a period of 28 and 56 days. After this period the volumic weight, the water content, the permeability and the axial compression resistance have been determined at the State Geotechnical Institute.

The mean values of the characteristics of the fresh bentonite-cement slurry are given in table 1. The pH of the bentonite-cement slurry varies in all cases between 11,5 and 12,5.
The mean values of the characteristics of the samples, tested after 28 and 56 days are given in tables 2 and 3.
The results of the performed tests presented a rather large scatter.

On fig. 6 the measured values of the axial compressive strength are given as a function of the measured permeability. This figure illustrates that the axial compressive strength increases when the permeability decreases. Although the relation between the permeability and the axial compressive strength presents a large scatter.

Fig.6 RELATION BETWEEN PERMEABILITY AND COMPRESSIVE STRENGTH OF THE FORMED BETONITE – CEMENT SAMPLES



d) Control of the completed screen:

A large number of piezometers, having a length of ca 19 m, an inner diameter of 5 cm (2") and a filter length of 2 m, have been installed 5 m behind the bentonite-cement screen at intermediate distances of 15 m. This was done in order to make it possible to localize eventual discontinuities in the screen.
During the first months of the groundwater-lowering, the waterlevel in these piezometers has been measured weekly or two times a week.
Afterwards, when a permanent situation was reached, these measurements were performed monthly or two times a month.
The results of these measurements revealed rather large differences in the groundwater-level behind the screen, as given in fig.7.
However,the observed differences are influenced by the different density of the refeeding wells. On the same figure the waterlevels, measured in the upper waterbearing stratum on the wharf side of the screen, are also indicated.

TABLE 1

Characteristics of the fresh bentonite-cement slurry

	Volume weight in kN/m^3	Marsh viscosity in s	Filter loss in ml/7,5"	Cake thickness in mm
just after leaving the mixer	10,84	35,8	91	3,25
at the top of the trench	11,87	---	104	11,8
at the midheight of the trench	12,32	58,7	103	14,3
at the bottom of the trench	12,31	68,2	105,6	13,63

TABLE 2

Characteristics of the samples tested after 28 days

	Permeability in m/s	Axial compressive strength in MN/m^2	Volume weight in kN/m^3	Water content in %
just after leaving the mixer(n=5)	$2,55.10^{-7}$	0,196	11,06	452
at the top of the trench (n=3)	$1,63.10^{-7}$	0,131	11,84	235,7
at the midheight of the trench (n=13)	$1,81.10^{-7}$	0,139	12,41	208,2
at the bottom of the trench (n=13)	$1,21.10^{-7}$	0,187	12,35	200,1

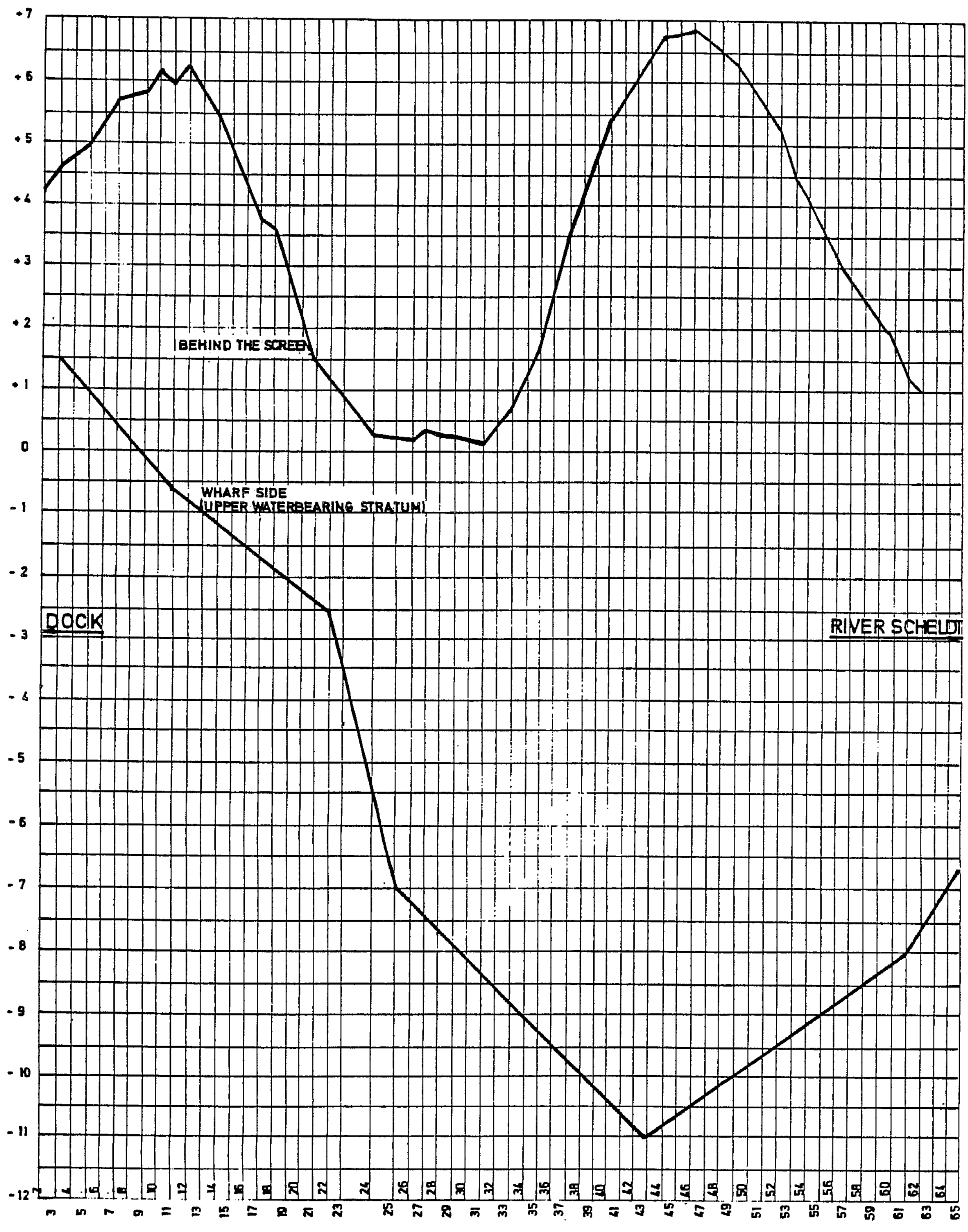
TABLE 3

Characteristics of the samples tested after 56 days

	Permeability in m/s	Axial compressive strength in MN/m^2	Volume weight in kN/m^3	Water content in %
just after leaving the mixer (n=5)	$1,81.10^{-7}$	0,287	11,04	446
at the top of the trench (n=3)	$1,02.10^{-7}$	0,206	11,89	235,5
at the midheight of the trench (n=13)	$6,2.10^{-8}$	0,219	12,29	206,5
at the bottom of the trench (n=13)	$4,97.10^{-8}$	0,289	12,34	200,3

n = number of tested samples

FIG.7 GROUNDWATERLEVEL BEHIND THE SCREEN AND IN THE UPPER WATERBEARING STRATUM ON THE WHARF SIDE.



ARTIFICIAL GROUNDWATER REFEEDING SYSTEM

a) Design:

In spite of the presence of the bentonite-cement cut-off wall, still a groundwater-lowering was expected even in the upper waterbearing stratum behind this bentonite-cement screen, due to the permeability of the screen, the permeability of the clayey layer between - 22 and - 24, and the waterflow around the extremities of the screen. Therefore an artificial refeeding of the watertable in the upper waterbearing stratum behind the cut-off wall was necessary.

As no data were available concerning the influence of the different phenomena, it was decided to install only a limited number of refeeding wells in a first stage. Due to the presence of the existing sea-lock, of the Scheldt River and of the inner dock, the elaboration of a mathematical model was very difficult in the design stage as no data were available concerning the boundary conditions to be introduced in the model. Finally the installation of 70 refeeding wells has been imposed before the start of any groundwaterlowering at all. Based on former experience (Berleur et al., 1981), the capacity of the refeeding wells has been fixed at 5 m^3/h.

The refeeding wells had to be installed 15 m behind the bentonite-cement screen at intermediate distances of 10 resp. 30 meters. A small intermediate distance has been chosen where the bentonite-cement screen is located at a short distance of the excavation pits.

The whole protection system of the refinery (bentonite-cement screen and groundwater refeeding system) has been settled by means of a separate tender, before the execution of the sealock itself was started. So preference has been given to a completely independent system, with discharge wells installed in the lower waterbearing layer, at a short distance before the bentonite-cement screen. In this way eventual confusion between the groundwater refeeding system and the groundwaterlowering for the construction of the new sealock could be avoided.

The installation of an extensive control system for the groundwaterlevel underneath the nearby refinery has also been imposed.

b) Preliminary refeeding test:

Before the final design of the groundwater refeeding system, a preliminary refeeding test has been performed in order to control:

- the capacity and the influence radius of the refeeding wells;
- the ability of the water extracted from the deep waterbearing layer for refeeding;
- the type and evolution of the well clogging;
- the frequency of the necessary cleaning of the wells and the efficiency of the planned cleaning method.

For the execution of the preliminary refeeding test a discharge well, four refeeding wells and several piezometers in the two waterbearing layers have been installed.

The discharge well consisted of a PVC-tube with an inner diameter of 235 mm and a filter length of 11 m, between the levels - 38 and - 49. The borehole, with a diameter of 40 cm was drilled by the direct flush method. A short time pumping test was performed to determine the characteristics of this discharge well.

The refeeding wells were installed at intermediate distances of 10 m and consisted of a PVC-tube with an inner diameter of 150 mm and a filter length of 10 m, between the levels - 11 and - 21. The borehole with a diameter of 40 cm was also drilled by the direct flush method.
In order to avoid the aeration of the refeeded water, the depth of the PVC injection tube, with an inner diameter of 50 mm reached till the lower limit of the well filter. The injection pipe was especially designed to assure a sufficient overpressure in the adduction pipes.

A complete analysis of the pumped water showed that the content of Fe^{2+} ions varied between 50 and 250 mg /liter and that the oxygen content was very low ($<$ 0,24 mg/liter).
Consequently, special precautions were necessary to avoid the aeration of the pumped water, in order to prevent the deposit of ferrugenous salts.

During a period of 2 months water has been recharged into the four refeeding wells at a total rate varying between 50 and 60 m^3/h. During this period a close examination of the refeeding wells has been carried out.
The waterlevel in the two waterbearing layers has also been measured regularly at different distances of the refeeding wells.

Furthermore a large number of tests were performed to control the eventual clogging of the refeeding wells.
The presence of fine particles and the gas content have been controlled regularly.

The obtained results showed that the pumped water contained only very few fine particles and that the gas content was very low.
After a period of 40 days the refeeding water for one refeeding well has been aerated on purpose.
After 1 day this refeeding well was already completely clogged.
After cleaning of this refeeding well by extensive pumping, its capacity was restored to only 85 % of its initial capacity.
This proves again that special precautions to avoid aeration are an absolute necessity.

c) Execution:

The situation of the first 70 refeeding wells installed before any groundwater-lowering, is given on fig. 5 (sections Ito X). On this figure the situation of the discharge wells and of the 70 additional refeeding wells (sections XI to XX) installed at the beginning of the groundwater lowering, is also given.

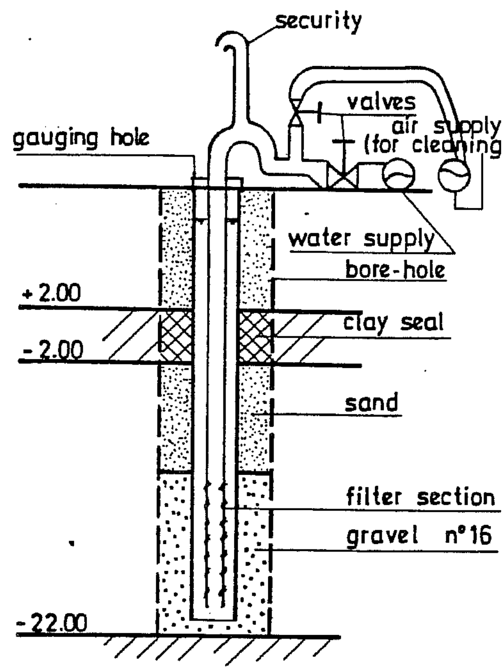
The discharge wells were identical to the discharge well, installed for the preliminary refeeding test. In order to avoid the aeration of the pumped water, the capacity of these wells was limited to 35 m^3/h, and preference was given to a direct connection between the discharge wells and the refeeding wells.
So each discharge well was connected directly with only 7 refeeding wells.
In this way the eventual aeration of the water of one discharge well could only result in the clogging of 7 refeeding wells and not of the whole system, which would have been the case if all discharge wells and all refeeding wells would have been connected to a central collector.

Each section, formed by one discharge well and seven refeeding wells, was equiped with the necessary valves for the adjustment of the refeeding rate of the different wells.
Special features were provided for cleaning the refeeding wells by airlift.

d) Control system:

For the control of the groundwaterlevel underneath the nearby refinery 28 piezometers have been installed, in which the groundwaterlevel is measured automatically every quarter of an hour. The results are stocked for one week in a central memory on the construction site and can be reproduced within a short period in a graphical or tabular form. The situation of the different piezometers is given on fig. 5. The piezometers nrs 1 to 23 were installed in the upper waterbearing stratum and the piezometers 24 to 28 in the lower waterbearing stratum.

Fig. 8 REFEEDING WELL



The piezometers consist of PVC - tubes with a diameter of ca 20 cm (8"), having a filter length of 4 m, in which a vibrating wire pressure transducer is suspended. Preference was given to large diameter PVC-tubes, because the piezometers will be used for a period of at least 5 years and the cleaning up of the piezometers cannot be performed in the normal way, due to the presence of the pressure transducers.

EXECUTION OF THE GROUNDWATERLOWERINGS

a) For the divertment of conduit pipes

Before the construction of the new sealock was started, a certain number of conduit pipes had to be diverted. Therefore two excavations with a depth of ca 30 m (till the level - 20,50) were made. The first one was situated at the end of the access channel to the new sealock and the second one near the gate chamber at the upstream head (see fig. 9). For the installation of the conduit pipes a working platform was necessary at the level - 15,10 as indicated on the cross section of the second excavation (fig. 10).

The groundwaterlowerings at the spot of these excavations have been performed with the help of 39 discharge wells.

Fig. 9 SITUATION OF THE TWO EXCAVATIONS FOR THE DIVERTMENT OF CONDUIT PIPES

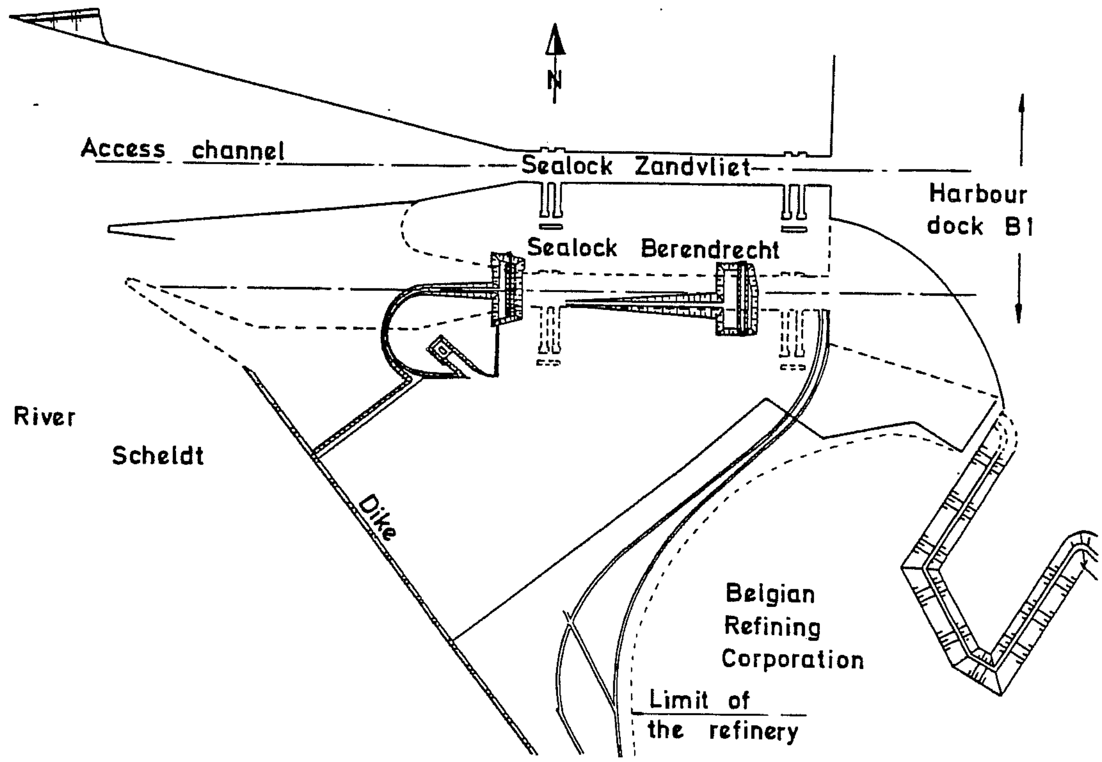
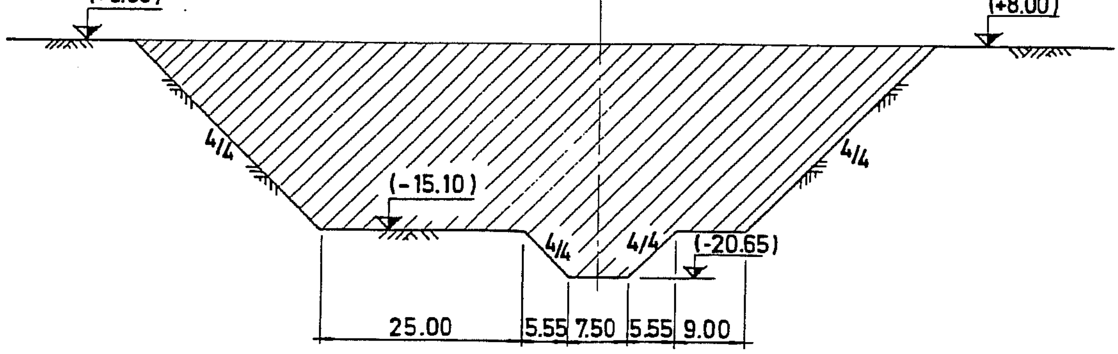


Fig. 10 CROSS - SECTION OF THE EXCAVATION AT THE UPSTREAM HEAD



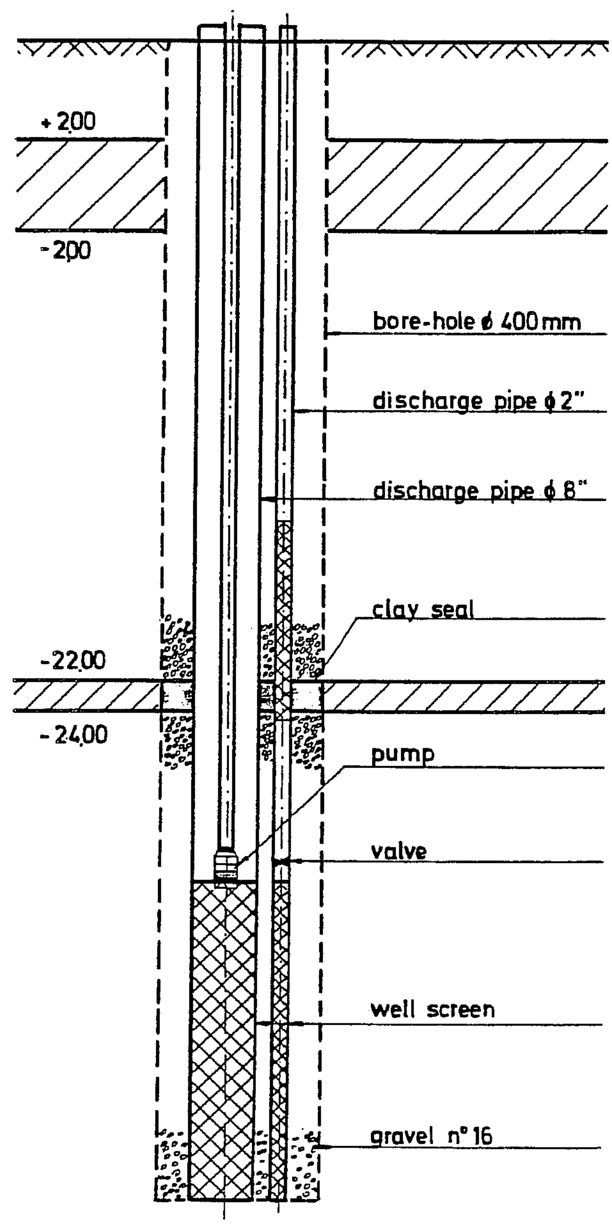
1.176 Most of the installed discharge wells are of a special type, patented by the NV Smet, sub-contractor for the groundwaterlowerings (see fig. 11).
These "combined " wells permit a simultaneous water extraction from the two waterbearing layers. Therefore, in what follows, these wells are indicated as double wells.
At the first excavation 11 double wells and 5 common discharge wells in the upper waterbearing stratum have been installed. The second excavation required the installation of 16 double wells and 7 common discharge wells in the upper waterbearing stratum.

Shortly after the start of the dewatering a sharp lowering of the waterlevel underneath the nearby refinery has been observed. As the waterlevel underneath the refinery continued to decrease even when the groundwater refeeding system was put into operation, it has been decided to stop a certain number of discharge wells and to install the 70 above-mentioned additional refeeding wells. Again preference has been given to a complete separation between the groundwaterlowerings and the refeeding system.

As the registrated groundwaterlevels showed an excessive lowering of the level in the piezometer 16, it was feared that the permeability of the clayey layers, encountered between the levels - 22 and - 24, was larger in the vicinity of the piezometer 16 than on the rest of the site. In order to avoid a direct flush between the refeeding wells and the discharge wells, it has been decided to install all the necessary discharge wells before the bentonite-cement screen. The situation of the additional refeeding wells and of the corresponding discharge wells is given on fig. 5 (sections XI till XX).

For the boring of the refeeding wells on the central verge of the Scheldelaan a discharge well has been installed in the lower waterbearing stratum. From the waterlevels registrated during the pumping in this well, it could be observed that the difference between the waterlevels in the piezometers 16 and 25, respectively installed in the upper and lower waterbearing stratum, remained very small. This observation confirmed the supposal that the permeability of the clayey layers between the levels - 22 and - 24 was locally larger than on the other parts of the site.

Fig.11 CROSS-SECTION OVER A DEEP WELL OF THE „COMBINED" TYPE.
(patent : Smet - Boring N.V.).



When the additional refeeding wells were operational, the groundwaterlowering of the two excavations could be performed without any problem. The groundwater-lowering underneath the nearby refinery could be limited to ca. 0,50 m.

Due to the stratification of the layers between the levels - 16,50 and - 22, composed by a succession of rather thin clay and sand layers, the installed discharge wells didn't permit the complete lowering of the waterlevel in the upper waterbearing stratum till - 21,00. So additional vacuum filters and horizontal drains had to be installed.

b) For the construction of the new sealock

For the construction of the new sealock the groundwaterlowering system, installed for the divertment of the conduit pipes, has to be extended. In order to control if the existing groundwater refeeding system would still continue to satisfy, a mathematical model, following the finite element method, has been elaborated (see fig. 12). In this model three waterbearing layers are considered, namely the fill, the layer between the levels - 2 and - 22 and the layer between the levels - 24 and - 50.
The refeeding from the inner dock and from the River Scheldt have been considered, and a hydraulic resistance (= d/k, with d thickness of the layer and k permeability of the layer) of 500 days has been introduced for the bottom of the inner dock and of 3000 days for the bottom of the River Scheldt.

With this model different calculations have been executed.

1. In a first stage the groundwaterlowering during the divertment of the conduit pipes has been calculated in order to verify the assumed values for the permeability of the different layers. Differences of several meters were found between the measured and the calculated waterlevels. Additional calculations have been performed with modified values for the permeability of some layers.

   An acceptable correspondance between the measured and the calculated waterlevels has been found for the following assumptions:

   - hydraulic resistance of the bottom of the inner dock = 100 days.
   - hydraulic resistance of the bottom of the River Scheldt = 500 days.
   - hydraulic resistance of the clayey layer between - 22 and - 24 = 231 days. In the vicinity of the piezometer 16 an area of 300 m x 275 m with a hydraulic resistance of only 46,3 days has been introduced.

2. Based on these assumptions a second model has been elaborated for the prevision of the groundwaterlevel during the first stage of the execution of the new sealock, namely for the installation of the access channel, the downstream head and the lock chamber. The obtained results indicated that the existing groundwater refeeding system is sufficient to limit the groundwater-lowering underneath the nearby refinery to ca 0,50 m.

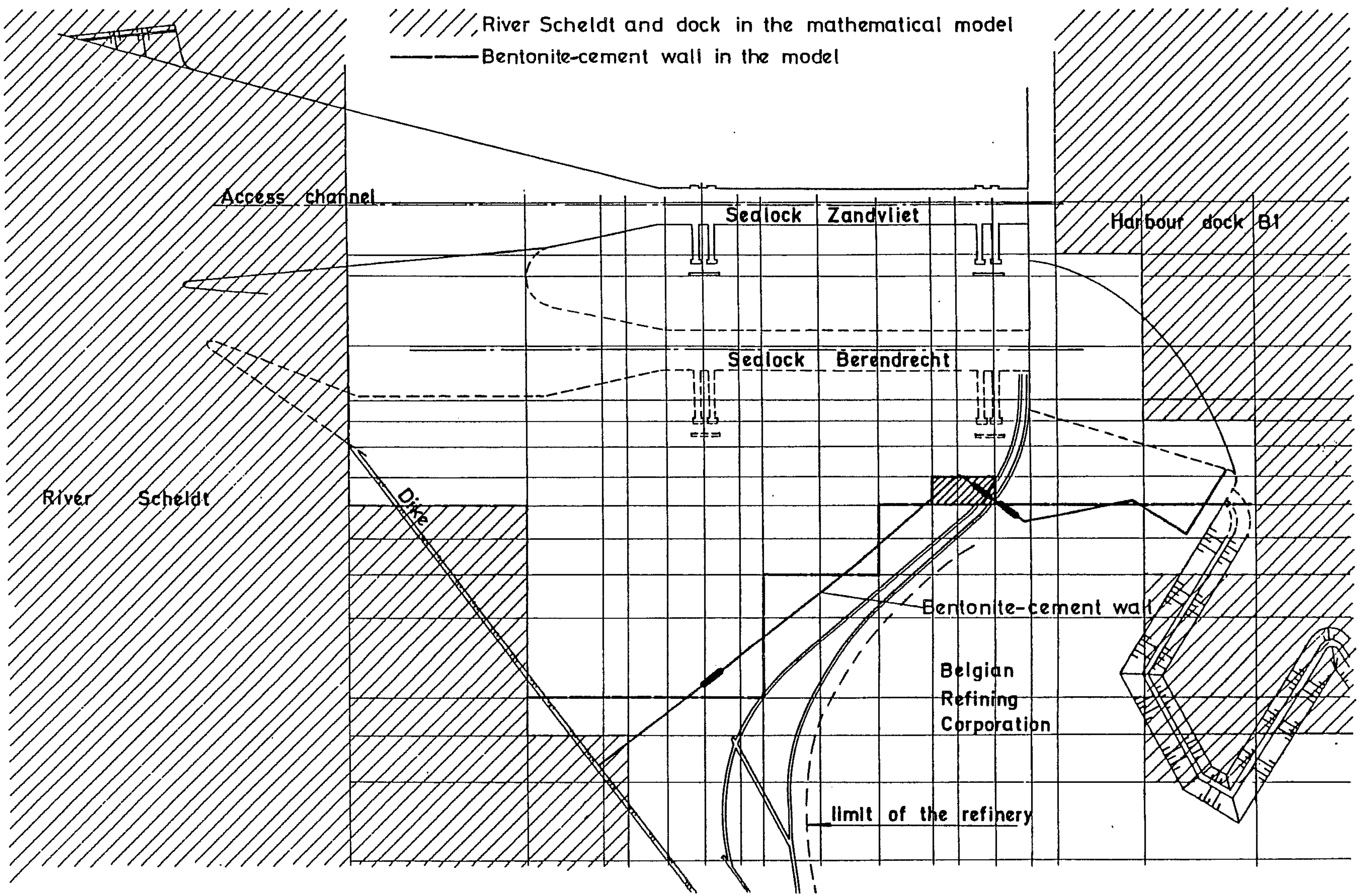
3. A third model has been elaborated in order to obtain an idea about the waterlevels that can be expected during the execution of the upstream head and the quay wall to the inner dock. The obtained results indicate for this stage a total groundwaterlowering in the upper waterbearing stratum between 1,50 and 3,00 m. Additional refeeding of the groundwater will then be necessary.

   Additional calculations have been performed to control how much the waterlevel will rise when the discharge wells IV to X are eliminated, and the refeeding sections IV to X are fed with the water of the discharge wells situated near the excavations. The obtained results indicated a rise of the waterlevel varying between 0,50 and 1,50 m.

   From the results of these calculations it has been deduced that special precautions will be necessary to avoid an excessive lowering of the groundwaterlevel underneath the nearby refinery. A certain rise of the waterlevel can be obtained by the elimination of some discharge wells, located near the bentonite-cement screen. However the installation of more refeeding wells will probably be necessary.

   For the optimalization of the location of these wells, further calculations with the mathematical model have to be performed.

Fig. 12 MODEL FOR CALCULATIONS FOLLOWING THE FINITE ELEMENT METHOD



CONCLUSIONS

For the construction of the new sealock Berendrecht a bentonite-cement diaphragm wall combined with artificial watertable refeeding had to be installed in order to prevent an excessive lowering of the groundwatertable underneath a nearby refinery.
During the installation of the bentonite-cement diaphragm wall the characteristics of the bentonite-cement slurry were controlled systematically.
The permeability and the axial compressive strength of the bentonite-cement mixture were determined at the laboratory after 28 and 56 days. From the results of these tests it could be deduced that only a poor relation exists between the permeability and the axial compressive strength of the tested bentonite-cement samples.

The execution of a groundwater refeeding test permitted to control the quality of the pumped water and the efficacy of the designed system.

An extensive control system has been installed for the registration of the waterlevels underneath the nearby refinery. Calculations with a finite element model have been performed in order to evaluate the eventual necessity of additional refeeding wells for the first phase of the construction of the new sealock.
Further calculations will be carried out for the optimalization of the additional refeeding system, which will probably be necessary for the final phase of the construction of the new sealock.

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