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Mathematical model for a controlled groundwater lowering during the construction of the Berendrecht Sealock at Antwerp

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The Berendrecht Sealock at Antwerp is built in a dry open trench. Therefore the groundwater is lowered over about 25 meters. Special measures had to be taken in order to limit the groundwaterlowering underneath a nearby refinery where several large storage tanks are founded on a ca three meters thick very compressible peatlayer. These measures consist of the installation of a bentonite cement diaphragm wall combined with artifical watertable recharge.

For the diversion of conduit pipes which cross the future lock, a number of groundwater recharge wells was installed. A mathematical model following the finite element method has been elaborated in order to control if the existing groundwater recharge system would still continue to satisfy.

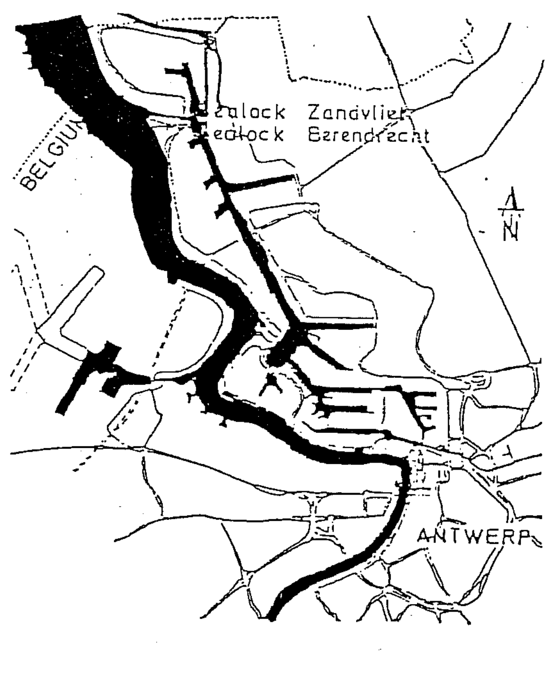
Different calculations have been performed to verify the assumed values for the different parameters and to calculate the expected waterlevels during the different construction stages.
The correspondance between the measured and calculated waterlevels is discussed.

Introduction

Due to the traffic increase to the Antwerp harbour the existing sealock Zandvliet, situated some 20 km down- stream of Antwerp has to be doubled. The location of the new sealock Berendrecht is given on fig. 1. The new sealock is built in a dry open trench. Therefore the groundwaterlevel has to be lowered over about 25 m. As a refinery and several large storage tanks, founded on very compressible Holocene deposits, are located in the immediate neighbourhood of the construction site, special measures had to be taken to limit the lowering of the groundwatertable. These measures consist of the installation of a bentonite-cement cut-off screen and of a groundwater recharge system.

The installation of the bentonite-cement screen and of the groundwater recharge system started in 1982. The construction of the new sealock itself started in 1983. The achievement of the new sealock is foreseen for the summer of 1987.

FIG.1 LOCATION MAP



Subsoil Conditions

From different investigation campaigns the general composition of the subsoil at the construction site of the new sealock Berendrecht was well known. Following layers are encountered :

- from ca. +9.00 m to ca. +2.00 m (above sealevel) =
  Fill, mainly installed during the construction of the existing sealock and the extension of the inner harbour.
- from ca. +2.00 m to ca. -2.00 m =
  Quaternary Holocene deposits, consisting of the former polder deposits, a compact peat layer with a maximum thickness of 3m and stratified clayey and sandy layers.
- from ca. -2.00 m to ca. -22.00 m (below sealevel) =
  Quaternary Pleistocene fine eolian sands, with a maximum thickness of 7 m and Tertiary Upper Pliocene fine glauconitic sands (Zandvliet sands and Merksem sands)
- from ca. -22.00 m to ca. -24.00 m =
  Tertiary Upper Pliocene fine glauconitic sands with shells and numerous clay layers (Kruisschans sands). The thickness of the clay layers generally varies between 0.10 and 0.15 m, but may locally reach 0.20 m.
- from ca. -24.00 m to -50.00 m =
  Tertiary Upper Pliocene sands (Oorderen Sands)
  Tertiary Lower Pliocene (Kattendijk Formation) and Tertiary Neocene (Berchem Formation) fine to medium fine glauonitic sands, with shells dispersed in the sand and concentrated in different layers.
- from ca. -50.00 m =
  Tertiary Oligocene stiff fissured overconsolidated clay (Boom Clay)

Pumping tests have been performed in the sand layer between -2.00 m and -22.00 m and in the sand layer between -24.00 m and -50.00 m to determine the hydrogeological conditions. From the results of these pumping tests the following informations have been deduced :

Groundwaterlevels at rest within the different strata :

- In the fill perched watertabels are encountered. Locally waterlevels between +7.50 m and +7.00 m were observed.
- In the sandy layers between -2.00 m and -22.00 m due to external influences (variations of the dock level and the tide in the River Scheldt) the waterlevel varied between +4.00 m and +4.50 m.
- In the sandy layers between -24.00 m and -50.00 m due to external influences (variations of the dock level and the tide in the River Scheldt) the waterlevel varied between +4.00 m and +4.50 m.

Hydrogeological constants :

- Hydraulic resistance of the Holocene deposits between +2.00 m and -2.00 m =

  $$c = \frac{D}{k_v} = 690 \text{ to } 750 \text{ days}$$

  with D : aquifer thickness
  k_v : vertical permeability of the layer.

- Horizontal permeability of the waterbearing stratum between -2.00 m and -22.00 m =

  $$k_h = 7.9 \text{ E-5} \quad \text{to} \quad 1.04 \text{ E-4 m/s}$$

- Hydraulic resistance of the clayey layers between -22.00 m and -24.00 m =

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

  $$k_v = 5.06 \text{ to } 5.79 \text{ E-8 m/s}$$

The vertical permeability determined at the laboratory on five undisturbed sampels varied between 1.49 E-7 m/s and 8.31 E-9 m/s

- Horizontal permeability of the water-bearing stratum between -24.00 m and -50.00 m =

  $$k_h = 4.88 \text{ to } 6.86 \text{ E-5 m/s}$$

Bentonite Cement Diagphragm wall

The bentonite-cement diaphragm wall has been installed untill the level -25.00 m, which is 1.00 meter beneath the clayey layer encountered between -22.00 m and -24.00 m. So the bentonite-cement screen reaches to a depth of abt. 34 m. The screen has only a cut-off function and will never be subjected to horizontal forces. A width of 0.80 m has been imposed (fig. 2)

The position of the screen, given in fig. 3 has been choosen to minimize the length of the wall still maintaining a safe distance to the construction pits. The screen has been continued over ca 150 m along the dock in order to limit the waterflow around the extremity of the cut-off screen.

Based on former experience (Berleur et al. 1981) a permeability of the screen of 1.00 E-7 m/s has been imposed. During the execution of the cut-off screen the bentonite-cement slurry has been controlled extensively. The permeability measured on

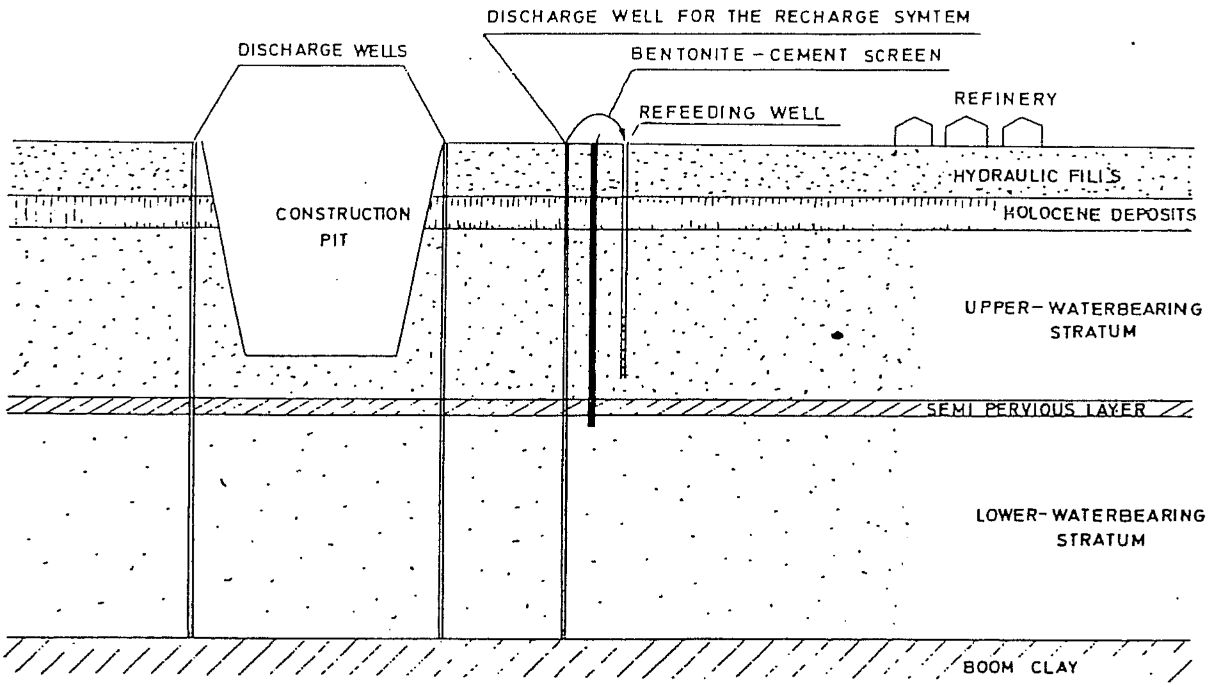


Fig. 2. — Berendrecht: schematic cross-section of the groundwater refeeding system and of the groundwater lowering system for the construction of the new sea-lock.

sampels of the slurry, taken at the top, mid height and bottom of the trench, varied between 1.02 E-7 and 4.97 E-8 m/s after 56 days.

Artificial Groundwater Recharge system

Due to the permeability of the screen, the permeability of the clayey layer between -22.00 and -24.00 and the waterflow around the extremities of the screen, artificial groundwater recharge was necessary behind the wall to minimise the groundwater-lowering underneath the nearby refinery.

As no data were available concerning the influence of the existing sealock, of the River Scheldt and of the inner dock, the elaboration of a mathematical model was not considered in the design stage.

Based on former experience the installation of 70 refeeding wells has been imposed before the start of the groundwater-drawdown. The capacity of the recharge wells has been fixed at 5 m3/h.

The whole protection system of the refinery (bentonite-cement screen and groundwater recharge) has been settled by means of a separate tender, before the construction of the sealock itself was started.

So preference has been given to a completely independent system, with discharge wells installed at a short distance before the bentonite-cement screen (fig. 2). In this way any possible confusion between the groundwater recharge system and the groundwater drawndown for the construction of the new sealock could be avoided.

Observation system

For the permanent observation of the groundwaterlevel 28 piezometers have been installed in which the waterlevel 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 displayed within a short period in graphical or tabular form. The location of the different piezometers is given on fig. 3. The piezometers nrs. 1 to 23 are installed in the upper waterbearing stratum and the piezometers 24 to 28 in the lower waterbearing stratum.

For the observation of the groundwaterlevel at the construction site, a large number of piezometers has been installed in which the groundwaterlevel is measured manually every two weeks.

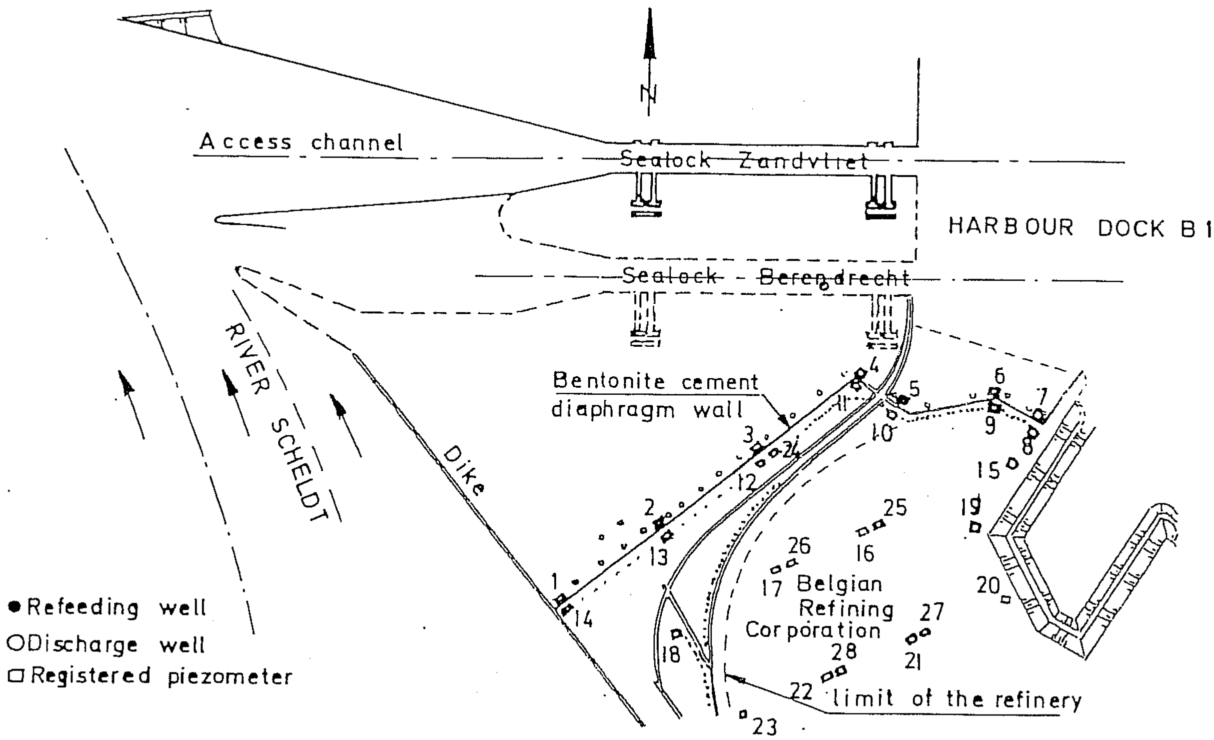


Fig. 3. - Situation of the bentonite-cement diaphragm wall, groundwater recharge system and registered piezometers

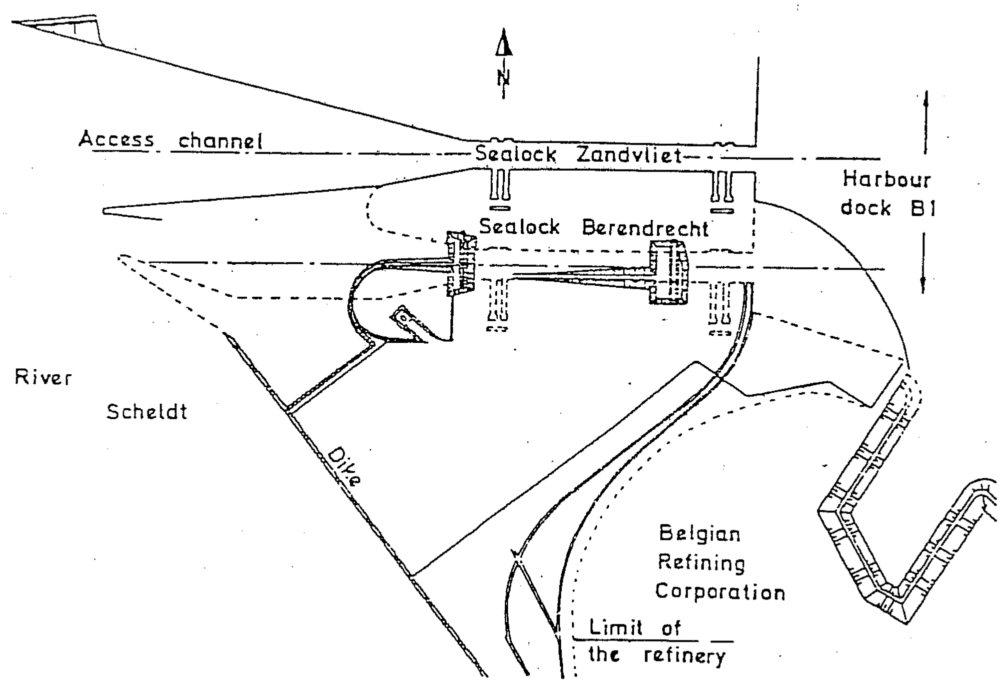


Fig. 4. - Situation of the two excavations for the divertment of conduit pipes

Groundwaterlowering for the diversion 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 m) were made (see fig. 4).

Shortly after the start of the dewatering of these two excavations a sharp lowering of the waterlevel underneath the nearby refinery was observed.

As the waterlevel underneath the refinery continued to decrease even when the groundwater recharge system was put into operation, it has been decided to stop a certain number of discharge wells and to install 70 additional recharge wells.

Because an excessive lowering of the waterlevel was observed in the piezometer 16, the additional recharge wells were installed along the Scheldelaan as near as possible to this piezometer. During the installation of the additional recharge wells it appeared that the difference between the waterlevels in the piezometers 16 and 25, respectively installed in the upper and lower waterbearing stratum remained very small. Additional CPT tests confirmed the supposition that the clayey layers between -22.00 m and -24.00 m didn't exist in the vicinity of the piezometer 16.

When the additional recharge wells were under operation, the groundwaterlowering for the two deep excavations could be performed without any problem. The groundwater- lowering underneath the nearby refinery was limited to less than 1 meter.

For the construction of the new sealock itself the groundwaterloweringsystem installed for the diversion of the conduit pipes had to be extended. In order to verify if the existing groundwater refeeding system would still satisfy, a mathematical model, based on the finite element method, has been elaborated.

Description of the mathematical model

All calculations were made with a finite element program in which the water-bearing strata are simulated by plate elements and the interjacent semi-permeable layers by line elements. Several units composed by a waterbearing stratum and covered with a semi-permeable layer can be superimposed on each other. The base of the lowest layer is always considered impermeable and the upper waterbearing stratum is always covered with a semi permeable layer.

For the line elements simulating the semi-permeable layers one has the following basic equations :

$$k_x \left(\frac{\delta\varphi}{\delta x}\right)^2 = 0 \qquad k_y \left(\frac{\delta\varphi}{\delta y}\right)^2 = 0$$

$$k_z \left(\frac{\delta\varphi}{\delta z}\right)^2 = K_z \left(\frac{\Delta\varphi}{\Delta z}\right)^2$$

$$\Delta U = \iiint_{lbh} k_z \left(\frac{\Delta\varphi}{\Delta z}\right)^2 dxdydz$$

$$= \frac{1}{c} \iiint \Delta\varphi \; dxdy$$

For the plate elements, simulating the waterbearing strata one has the basic equations =

$$k_z \left(\frac{\delta\varphi}{\delta z}\right)^2 = 0$$

$$\Delta U = \iiint_{lbh} \left\{ k_x \left(\frac{\delta\varphi}{\delta x}\right)^2 + k_y \left(\frac{\delta\varphi}{\delta y}\right)^2 \right\} dxdydz$$

$$= k_x H \iint \left(\frac{\delta\varphi}{\delta x}\right)^2 dxdy + k_y H \iint \left(\frac{\delta\varphi}{\delta x}\right)^2 dxdy$$

The general structure of the n-layer program is very similar to that of a classical three dimensional program. The head (or potential) in each node is deduced from the values in all 27 adjacent nodes (= the number of the adjacent nodes in a block structure, the considered node included). In this way the flow through the semi permeable layer can be simulated very accurately.

Following boundary conditions can be introduced :

1. the intensity of flow is prescribed.
2. the head (or potential) is prescribed.
3. the head (or potential) is prescribed with a given entrance resistance.

The flow through the boundary of the considered volume is supposed to be zero when no other specifications are communicated.

The 3rd boundary condition is the most general one. In some cases it can be interesting to introduce the 1st condition with the help of the 3rd one by combining

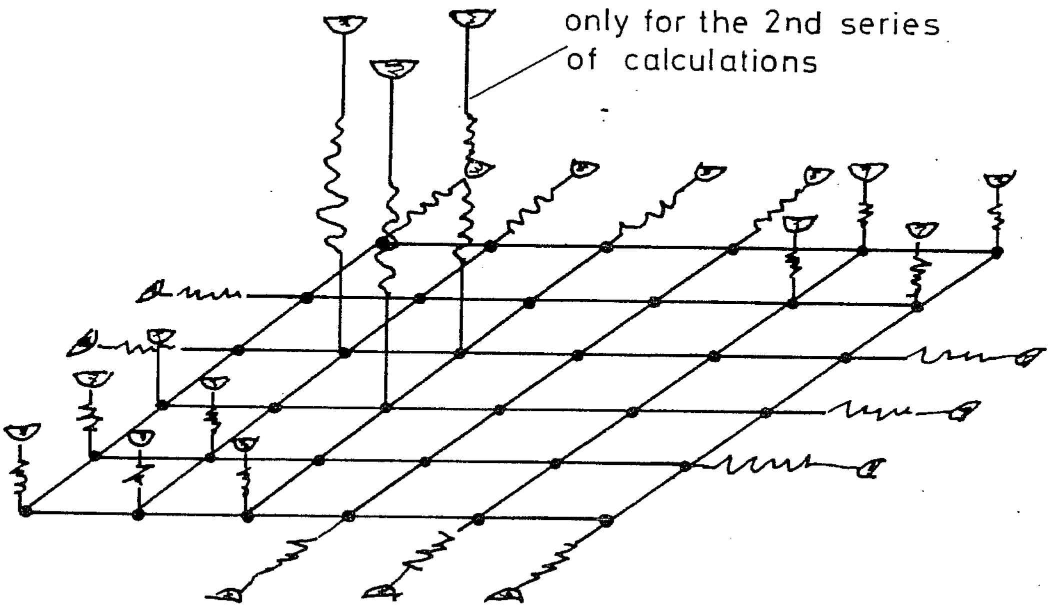


Fig. 5 Scheme of the different considered boundary conditions

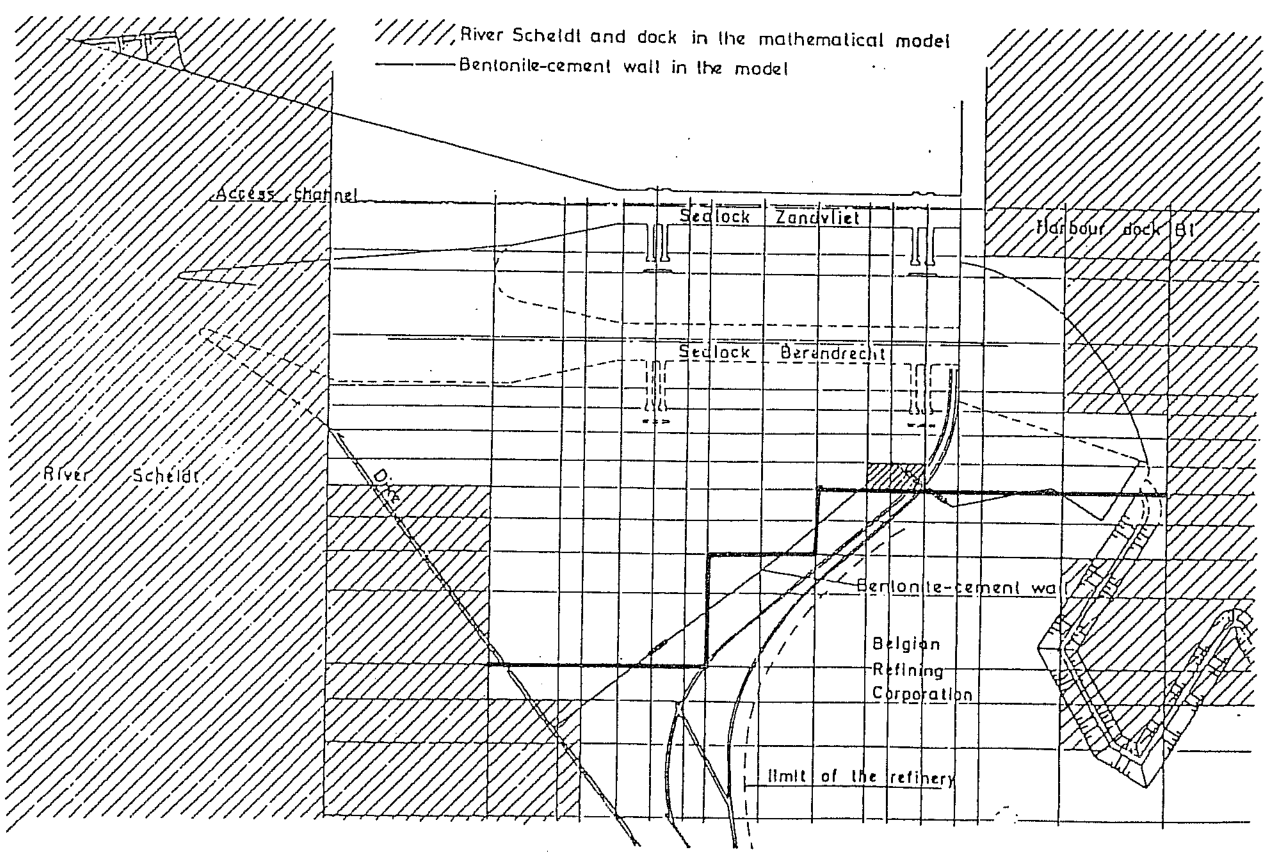


Fig6 Mesh for the 1st. series of calculations

a very high head with a very high entrance resistance, so that the required flow is obtained. Small variations of the head inside the model results then only in a negligable variation of the flow. Following this method a fixed head can be introduced and the node can be omitted during the iteration processus. So a considerable reduction of the computer time is obtained.

Rectangular elements have to be used throughout. The width or the heighth of the elements can be varied for each row or column. Different material constants can be dedicated to all elements.

In order to limit the extent of the model, for some sides, a given head is introduced with an entrance resistance (figure 5). The value of the entrance resistance is calculated in this way that at the limit of the considered model a good agreement is obtained between the measured and the calculated values.

The influence of rainfall can be introduced in different ways, i.e. by means of a prescribed flow in each node of the upper semi-pervious layer or for instance by means of a head of 100 m and an entrance resistance of 100 000 days.

When an aquifer becomes phreatic, the transmissivity of that layer varies considerably with the waterlevel. Therefore the transmissivity of the waterbearing stratum is modified every 20 iterations, according to the calculated waterlevels.

Two series of calculations have been performed with the same program :

- a first series to determine if the existing groundwaterrecharge would still continue to satisfy when the groundwaterlowering system, installed for the diversion of the conduit pipes would be extended for the construction of the quay walls in the access channel to the River Scheldt, the downstream head and the lock chamber

- a second series to check the influence of additional recharge wells installed at the site of the nearby refinery.

First series of calculations :

The used mesh is represented in fig. 6. The bentonite-cement screen is introduced into the model.
Two different waterbearing layers have been considered, namely between 0.00 m and -22.00 m and between -24.00 m and -49.00 m. For these layers a transmissivity kD (with D : thickness of the aquifer) of 6.87 m2/h resp. 4.58 m2/h has been introduced. For the bentonite-cement screen a kD value of 0.008 m/h has been introduced corresponding to the contractual permeability of 1.00 E-7 m/s

For the former polder layers between +3.00 m and 0.00 m a hydraulic resistance of 16 560 hours has been introduced ; this corresponds to a vertical permeability of 5.06 E-8 m/s. The influence of rainfall has not been considered. For the clayey layers between -22.00 m and -24.00 m a hydraulic resistance of 11 040 hours has been introduced, this is the value deduced from the results of the pumping test.

For the waterlevel in the Inner dock a constant value of +4.20 m has been prescribed. In a first approach the same hydraulic resistance has been introduced for the bottom of the Inner dock as for the former polder layers i.e. 16 560 hours. For the River Scheldt the mean waterlevel of +2.70 m has been prescribed. Again a hydraulic resistance of 16 560 hours has been introduced for the bottom of the River Scheldt. At the southern limit of the model a waterlevel varying between +2.70 m and +4.20 m has been prescribed with different entrance resistances.

In a first stage the groundwaterlowering during the diversion of the conduit pipes (situation on Nov. 15, 1982) has been simulated in order to verify the value of some parameters.

Five runs were necessary to obtain an acceptable agreement between the calculated waterlevels and those measured on Nov. 15, 1982. The values of the different parameters for each run are given in tabel 1. For run 5 the comparison between some calculated and measured values is given in table 2. For the upper waterbearing stratum a rather good agreement is found for the piezometers 15 and 16, which were giving the lowest waterlevels and were therefore the most critical. Within the lower waterbearing stratum larger divergences are found.

It probably would have been possible to obtain a better agreement between the measured and the computed values by transferring the window in the layer between -22.00 m and -24.00 m more to the north and by decreasing the dimensions

Table 1 - Hydraulic resistances c (in hours) considered for the different calibration runs

	Run 1	Run 2	Run 3	Run 4	Run 5
former Polder layers	16 560	16 560	16 560	16 560	16 560
layer between -22 and -24	11 040	11 040	11 040	5 555	5 555 *
bottom of River Scheldt	16 560	16 560	16 560	72 000	12 000
bottom of innerdock	16 560	12 000	2 400	2 400	2 400

* In the vicinity of the piezometer 16 an area of 300 m x 270 m with a hydraulic resistance of only 1 111 hours has been introduced

Table 2 = Comparison between some measured and the computed watelevels (Run 5)

Upper waterbearing stratum :

Piezometer	Waterlevel measured on Nov. 15, 1982	Waterlevel calculated for Run 5
15	+3.53m	+4.03 to +3.43m
16	+3.56m	+3.73 to +3.08m
17	+5.28m	+5.14 to +3.56m
20	+4.13m	+3.23 to +3.47m
21	+4.26m	+3.39m
22	+4.70m	+3.62 to +3.77m
23	+5.30m	+5.33 to +4.04m

Lower waterbearing stratum :

Piezometer	Waterlevel measured on Nov. 15, 1982	Waterlevel calculated for Run 5
25	+3.28m	-2.73 to +1.32m
26	+3.81m	+1.05 to +2.02m
27	+4.32m	+3.36m
28	+4.59m	+3.52 to +3.59m

of this window and also the hydraulic resistance of the layer between -22.00 m and -24.00 m within this window. However, due to a lack of time, no further calibrations were made and all further computations were performed with the parameters adapted for run 5.

The extension of the groundwaterlowering system, necessary for the construction of the quay walls in the access channel to the River Scheldt, the downstream head and the lock chamber, was first considered. The performed calculations show that the expected drawdown of the groundwatertable underneath the nearby refinery, compared to the situation on Nov. 15, 1982 was limited to ca 0.40 m.

A comparison between the calculated and the measured drawdown is given in Table 3. The measured drawdown of the watertable obtained as the difference between the waterlevels on Nov. 15, 1982 and June 30, 1983 is considerably larger than the calculated one.
This is due to the fact that after a modification of the construction planning a supplementary grondwaterlowering to the level -20.00 m was started for the constuction of a Pier in the River Scheldt. This drawdown was not considered in the model.

Additional computation was performed with the same model to obtain an idea about the expected waterlevels underneath the refinery when the groundwater would be lowered over the whole construction area i.e. for the construction of the upstream head and the quay wall in the access channel to the Inner dock. Following these calculations an additional groundwaterlowering of ca. 1.30 m had to be expected at the location of the piezometer 16. As such a lowering was considered to be inacceptable, further calculations were performed to study the influence of additional grondwater recharge on the site of the refinery.

Table 3 : Comparison between calculated and measured groundwaterdrawdown during the constuction of the quay walls in the access channel to the Scheldt River, the downstream head and the lock chamber.

Upper waterbearing stratum

Piezometer n°	Computed drawdown	Observed drawdown
15	0.17 m	0.23 m
16	0.34 to 0.51 m	0.91 m
17	0.39 to 0.48 m	0.88 m
20	0.10 m	0.13 m
21	0.22 m	0.30 m
22	0.14 to 0.22 m	0.43 m
23	0.17 to 0.27 m	0.50 m

Lower waterbearing stratum

Piezometer	Computed drawdown	Observed drawdown
25	0.39 to 0.62 m	0.93 m
26	0.41 to 0.57 m	1.01 m
27	0.17 m	0.32 m
28	0.14 to 0.23 m	0.57 m

Second series of calculations :
In order to study the influence of additional groundwater recharge on the site of the refinery different additional computations have been performed. Therefore a slightly different mesh pattern has been choosen.

The hydrogeological parameters from the previous model have been retained. The influence of rainfall has been introduced by means of a constant head of +100.00 m and calculated entrance resistances.

First of all a calculation has been performed for the situation existing on Jan. 4, 1984 in order to compare the computed and observed waterlevels for the new mesh pattern. As the agreement was considered to be satisfactory, further computations have been performed without modification of the model.

Finally the solution, consisting of the installation of 56 additional recharge wells on the site of the refinery, was found the most appropriate. In this way an expected waterlevel of +3.35 m could be maintained at the location of the piezometer 16.

On Dec. 31, 1985 the groundwater was lowered over the whole construction site. Due to practical constraints the configuration of the recharge wells differs somewhat from the configuration introduced in the model. In this way a direct comparison between computed and observed groundwaterlevels is not possible. The deepest waterlevel on Dec. 31, 1985 was of +2.96 m (piezometer 15). It can however be stated that the number of additional recharge wells, necessary to limit the groundwaterlowering underneath the refinery, was predicted very accurately. It has to be remarked that the groundwaterflow around the extremity of the bentonite-cement screen near the Inner dock is not predicted very well.
Rather large differences are found between the computed and observed waterlevels. Probably these differences are due to the choice of the mesh pattern in this area. Also the permeability of the bentonite-cement screen has to be considered. Suspections exist that within the zone between the Scheldelaan and the Inner dock the bentonite cement screen contains some heterogeneities.

Conclusions
During the different stages of the construction of the new Berendrecht sealock at Antwerp a mathematical model following the finite element method has been used with very good results.

Although the situation was very complex due to the presence of the River Scheldt, the existing Zandvliet lock, the Inner dock, the bentonite-cement screen and the groundwaterrecharge system, the mathematical model was very useful for predicting the influence of the successive extensions of the groundwaterlowering and groundwater recharge system. The autors estimate that a good balance has been found between the available geotechnical and hydrological data, the costs and duration of the calculations and the obtained accuracy. For groundwater problems an accuracy of approximately 0.50 m seems reasonable.

The experience obtained with the construction of the Berendrecht sealock shows that a controlled lowering of the groundwatertable is possible, even in very difficult conditions. The step by step approach helps to obtain an economical solution. Due application of a mathematical model makes it possible to adapt the initial project continuously to the existing situation and to guide the different execution stages in a more appropriate way.

Bibliography

Berleur E., De Beer E., Maertens J. and Vanmarcke S. (1981).
Controlled dewatering for the construction of a new sealock at Zeebrugge, Proc. Xth Int. Conf. on Soil Mechanics and Foundation Engineering, Stockholm, 1981.

De Meuter F.J. and Laga P. (1976).
Lithostratigraphy and biostratigraphy based on benthonic foraminifera of the neogene deposits of Northern Belgium, Bulletin van de Belgische Vereniging voor Geologie, Deel IV, p. 133-152.

Goetinck R., Thomas H., Maertens J. and Vanmarcke S. (1983).
Controlled dewatering for the construction of the Berendrecht Sealock at Antwerp, Proc. 8° Int. Havenkongres Antwerpen, 1983.

Goetinck R. (1985)
The Berendrecht lock, Publication of the Ministry of Public Works, Brussels.

Van de Cotte F. and Vanmarcke S. (1981)
Het vermijden van zettingen door aanwending van retourbemaling : Toepassingen in de praktijk.
Nationaal Colloquium Belgisch Comité voor Ingenieursgeologie, Gent 1981.