Dewatering for the Construction of a New Sealock at Zeebrugge

Rabattement pour les Travaux de l'Ecluse de Zeebrugge

E. BERLEUR Engineering Department Manager, S.B.B.M. Entreprises, Belgium
E. De BEER Professor, State University Ghent, Belgium
J. MAERTENS Engineer, Ministry of Public Works, Belgium
S. van MARCKE Engineer, S.B.B.M. Entreprises, Belgium

For the construction of the new sealock at Zeebrugge, it was necessary to lower the groundwater table over more than 20 meters. As in the Zeebrugge area one finds very compressible peat and clays overlaying a coarse sand layer and a very heterogeneous substratum special measures had to be taken in order to limit the drawdown of the water table in the borough of Zeebrugge located in the neighbourhood of the construction site. Different tests were performed in order to obtain an accurate determination of the hydraulic constants. For limiting the drawdown outside the deep construction pits for the lock heads, resp. for the cellars of the basculing bridges, among several possibilities finally the solution, consisting in the installation of vertical bentonite-cement screens, resp. reinforced concrete diaphragm walls, and in refeeding the watertable has been retained. The design of refeeding installation was principally based on a judicious extrapolation of the results of the pumping tests.

I. INTRODUCTION

At Zeebrugge, a new sea lock for 125,000 ton ships is being built. It consists of a lock chamber with a length of 500 m, a width of 52 m and whose apron is located at the level - 15.00.Each lock head is provided with two gate chambers, with a length of 64,50 m and a width of 10,50 m, for housing of the rolling gates (fig. 1). The water level in the inner harbour will be maintained at + 3,50, while at seeside by exceptional tides, it can vary between + 7,00 and - 1,00

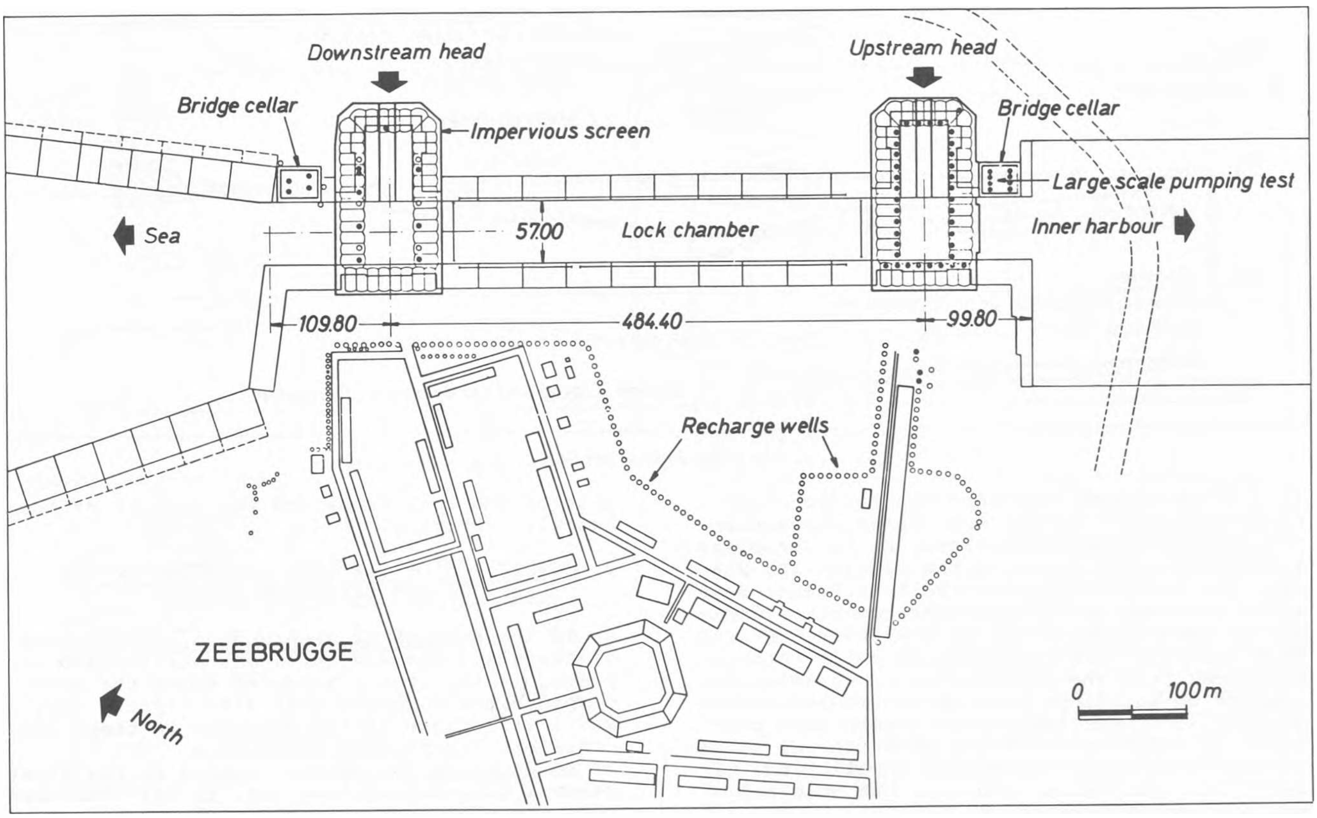


Fig.1 Location plan of the sea lock

The construction of four basculing bridges with deep cellars was also decided.
At the site of the lock, the following geological formations are encountered, starting from the soil surface (fig. 2) :

- quaternary holocene formations, consisting of clay and peat layers, and locally also sandy formations.
- quaternary pleistocene deposits of coarse sands and gravels, with a large amount of shells and locally some clayey inclusions.
- tertiary eocene clay and sandy clay (Assian), with a very variable thickness, and even disappearing underneath a part of the upstream head.
- tertiary eocene fine sands (Ledian and Wemmelian) with layers of clay and marl.
- tertiary eocene clayey and sandy layers (Paniselian).

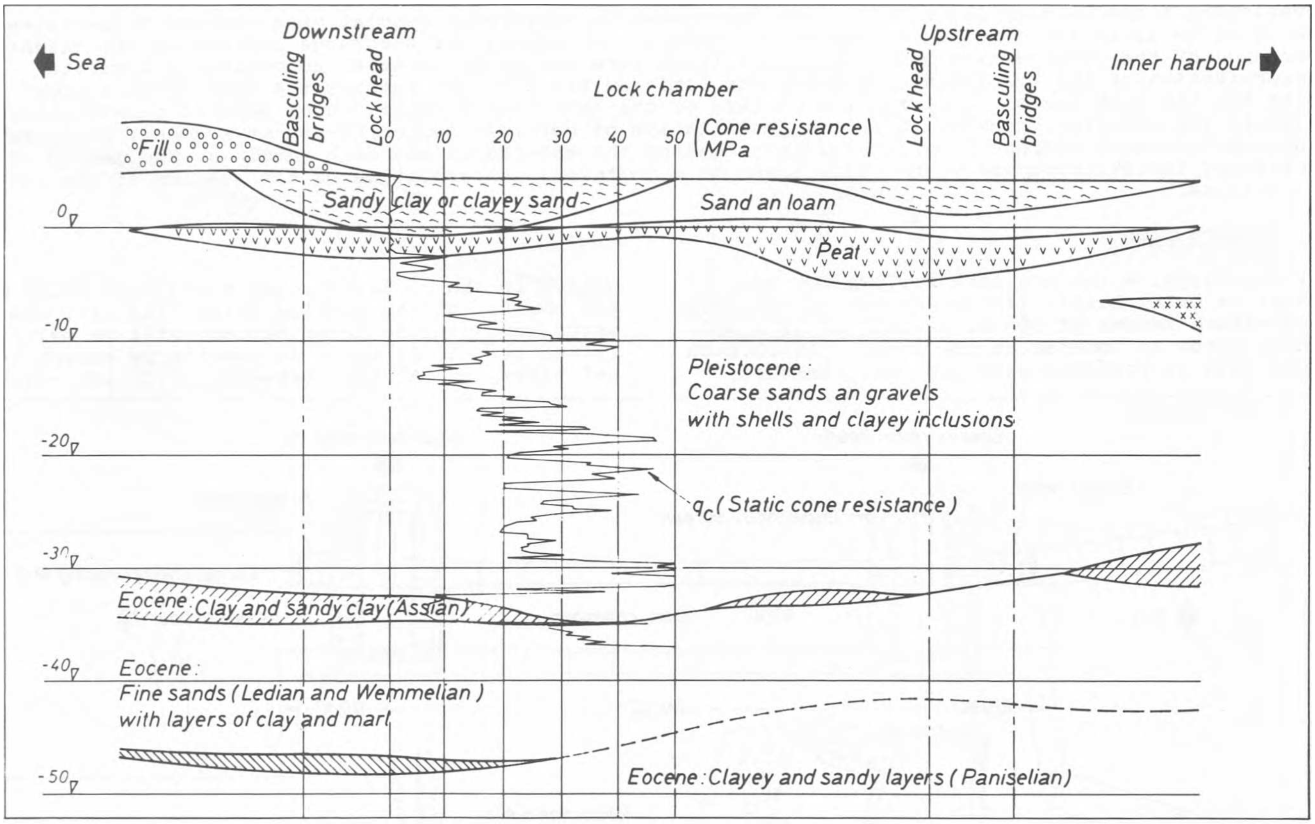


Fig. 2 Geological section

In a first design, the lock should have been built completely in the dry, under protection of a general groundwater lowering in the pleistocene deposits, reaching the level - 16,50 and even the level - 20,00 at the lock heads. Soon after starting the groundwater lowering, damages at several buildings in the Zeebrugge area were observed, some occuring at quite large distances from the building site, especially however on buildings with heterogeneous foundations, or under which peat layers were present. In order to avoid the extension of these damages with time, especially to all kinds of utilities (gas mains, etc...), the original project was abandonned.
A new project was introduced by "The Entreprises S.B.B.M." (Berleur and Fremout, 1978). The main line of the project is to limit as much as possible deep groundwater lowerings by using under water methods of execution; therefore the walls of the lock chamber are designed as retaining structures, consisting of a platform on piles and a wall of concrete sheetpiles, necessitating only a small drawdown of the watertable.
The lock chamber is realized by dredging between the walls to the level - 16,50 and placing under water a continuous filterbed and prefabricated concrete blocks.
As for the execution of the lock heads and the cellars of the basculing bridges which had to be executed with great accuracy, under water techniques were not applicable, controlled dewatering techniques had to be applied in order to limit the drawdown outside the construction pits of the lock heads and the cellars of the basculing bridges.

2. CONCEPTION OF THE LOCK HEADS AND OF THE CELLARS OF THE BASCULING BRIDGES

As in the borough of Zeebrugge a large number of installations and utilities are founded or placed in the layers situated above the very compressible holocene peat clay layers, only small variations of the stresses in these compressible layers were acceptable.
After studying the damages caused by the first general groundwater lowering, it was concluded that the piezometric level in the pleistocene coarse sands and gravels should not be lowered

under the level 0,00 at the location of the nearest houses distant of ca. 100 m from the edge of the lock heads.
This condition should have been rather easy to fulfil, if at the site of both lockheads (dimensions : 100 x 180 m) a sufficiently thick layer of Assian clay should have been present. Indeed, in such a case, it should have been sufficient to surround both lock heads and the bridge cellars (dimensions 32 x 39 m) with an impervious screen reaching the Assian clay.
However, as indicated in fig. 2, this solution could not be generalised, as over a large part of the upstream lock head, the Assian clay was not present.
So only at the downstream head a bentonite-cement screen, with a thickness of 0,80 m and reaching to the level - 32,50, formed together with the Assian clay a relatively watertight box. For the upstream head the installation of a bentonite-cement screen reaching to the level - 32,50 was also designed.
It was however necessary to perform a detailed study in order to determine the measures which were to be taken in order to limit the drawdown underneath the borough of Zeebrugge, as here there was no watertight bottom.
The construction pit of the lock heads was limited by cellular retaining structures made of flat steelsheet-piles, with a free height of 17 m and surrounded by the impervious bentonite-cement screen (fig. 3).
The separation between the two chambers of each head was formed by a balasted construction (fig. 4).
The cellars of the basculing bridges are constructed inside an enclosure of reinforced-concrete diaphragm walls, reaching to the level - 32,50. These diaphragm walls act as watertight screens and are lowered into the Assian clay at the downstream side and into the tertiary eocene fine sands at the upstream side.

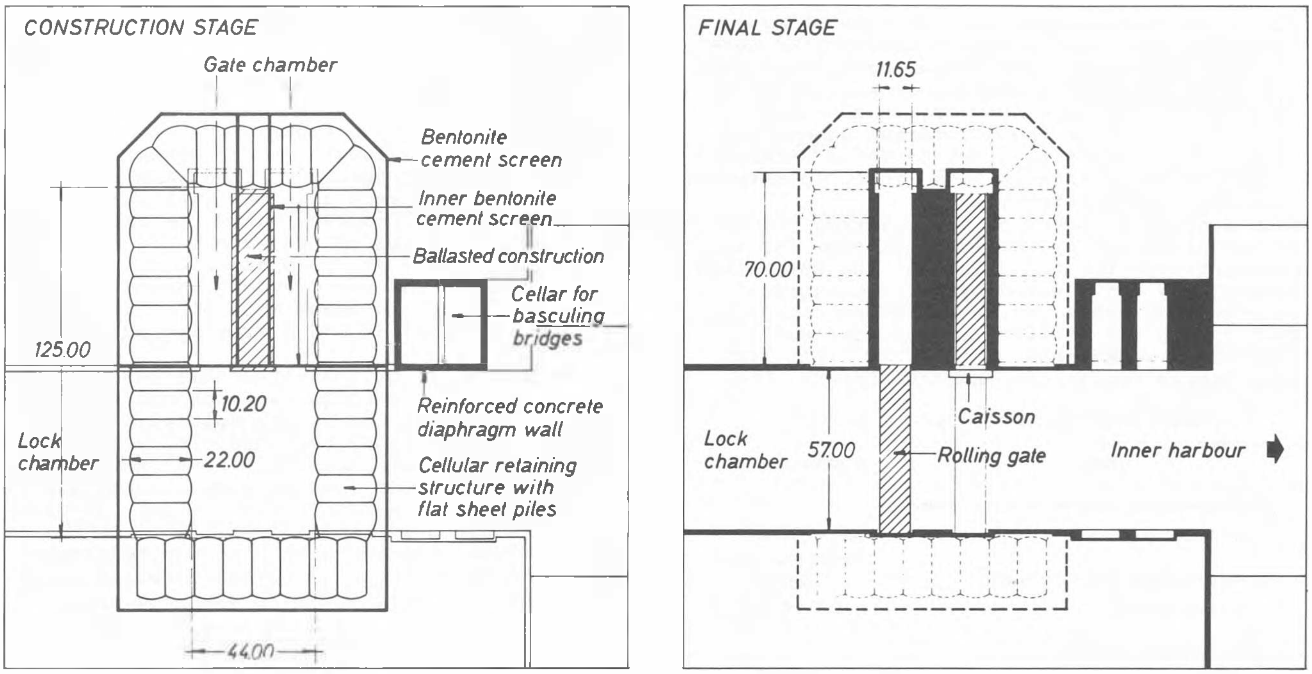


Fig. 3 Gate chambers (Upstream head) Horizontal cross section

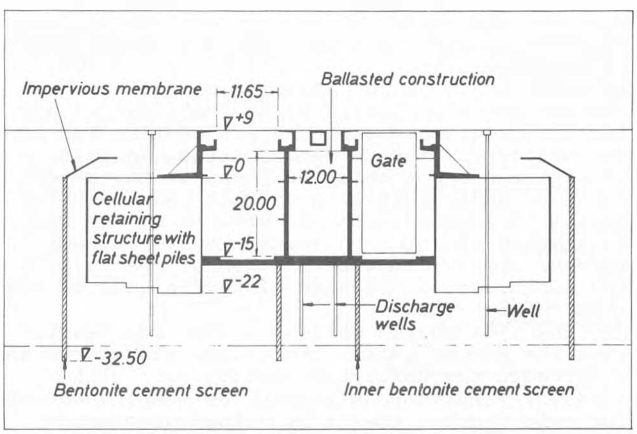


Fig. 4 Gate chambers_Vertical cross section

3. DETERMINATION OF THE HYDRAULIC CONSTANTS

The hydraulic constants were determined from the following observations :
- The drawdown curves registered during the first general groundwater lowering.
- Permeability tests in boreholes.
- A large scale pumping test was performed inside the diaphragm wall enclosure, installed at the upstream side for the construction of the basculing bridges. The enclosure had a rectangular shape (32,00 x 39,00 m) and reached to the level - 32,50. During the pumping test, the water level inside the diaphragm walls enclosure was lowered with ca. 14 m and the piezometric level was controlled in different piezometers. Prior to the test, a complete picture of the waterlevel changes of the coastal aquifer has been registered.
The results of this large scale pumping test

permitted to determine values for different hydraulic constants :

- From the readings of the piezometers located at different levels within the enclosure, simple using of the Darcy law permitted to determine the mean vertical permeability of the Pleistocene coarse sands and of the tertiary eocene fine sands.
- When considering the enclosure as a large discharge well with an equivalent diameter of ca. 19 m, the formula of Dupuit permitted to determine the upper limit of the permeability of the packet formed by the Pleistocene coarse sands and the eocene fine sands.
- The application of the formula of Davidenkoff (Davidenkoff and Franke, 1965) made it possible to determine a lower limit for the vertical permeability of the packet formed by the Pleistocene coarse sands and the eocene fine sands.
- In another attempt, measurements were made on an electrical model for different values of the ratio between the permeability of the Pleistocene coarse sands and of the tertiary eocene fine sands. The measurements were made on a two dimensional model, consisting of an isotropic material. However, an adequate transformation of the horizontal dimensions made it possible to simulate three dimensional flow conditions in an anisotropic material (De Beer and Maertens, 1976).

Table I gives a survey of the obtained values for the different hydraulic constants. The representativity for the values of the horizontal permeability was checked by performing small scale pumping tests in the Pleistocene coarse sands and in the tertiary eocene fine sands. Precautions were taken in order to limit the water extraction to the examined layers. The

Table I Determination of the hydraulic constants

Test		k values (m/sec)
_ Drawdown curve. First groudwaterlowering		
Pleistocene sands		$k_h = 1,38.10^{-4}$ Radius of influence 1200m
_ Permeability tests in two boreholes		
Eocene sands		$k = 0,10$ à $0,24.10^{-4}$
_ Large scale pumping test		
_ Darcy's law		
Pleistocene sands		$k_v = 0,79.10^{-4}$
Eocene sands		$k_v = 0,10$ à $0,23.10^{-4}$
_ Formula of Dupuit		
Pleistocene and eocene sands		$k_h = 1,50.10^{-4}$
_ Formula of Dandenkoff		
Pleistocene and eocene sands		$k_v = 0,19.10^{-4}$
_ Electrical model		
Assumed : pleistocene sands		$k = 1,04.10^{-4}$
Measured : eocene sands		$k = 0,29$ à $0,44.10^{-4}$
_ Small scale pumping tests		
Pleistocene sands		$k_h = 1,47$ à $1,72.10^{-4}$
Eocene sands		$k_h = 0,18$ à $0,29.10^{-4}$
Constants retained for the final calculations	k_h	k_v
Pleistocene sands	$1,50.10^{-4}$ m/sec	$0,80.10^{-4}$ m/sec
Eocene sands	$0,30.10^{-4}$ m/sec	$0.20.10^{-4}$ m/sec

values retained for the final calculations are also indicated.
In a last stage, the large scale test was simulated with the help of a finite element program, valid for axisymmetric flow in anisotropic media (Rammant and Backx, 1978).
When introducing the retained values of the hydraulic constants, a very good agreement was obtained between the calculated and registered values of the discharge and piezometric levels. So these results confirmed the validity of the retained values for the hydraulic constants and of the used computerprogram.

4. DIFFERENT CONSIDERED SOLUTIONS

At the downstream head the bentonite-cement screens were lowered into the Assian clay layer. At the upstream head the absence of a continuous clay-layer with a sufficient thickness, made it necessary to develop measures in order to limit the discharge rates underneath the initially designed screens which reach the level - 32,50. Different measures were considered, for instance

1° grouting a complete or partial impervious horizontal layer at the base of the watertight screens;
2° refeeding the watertable;
3° increasing the length of the watertight screens in the tertiary formations.

All calculations were executed with the help of a finite element program, valid for axisymmetric flow. The refeeding of the watertable was supposed to take place at 200 m and 350 m from the edge of the head and the drawdown curves were calculated in order to check more especially the piezometric level at 100 m from the edge of the head.
As the refeeding of the watertable was only necessary near the borough of Zeebrugge, the calculated refeeding charge was multiplied with a factor 0,65 which has been deduced by anology between F.E.M. calculations and the method of the drawdown curves (see further program RABAT). The data, obtained from the different calculations are given in table II.
Finally, the solution with a bentonite-cement screen to the level - 32,50 and a refeeding of the watertable with 400 m3/h was retained.

5. DRYING OUT OF THE GATE CHAMBERS

The gate chambers have to be separately pumped out for repair and maintenance of the gates. Then the gate chamber is closed with a steel caisson; the cellar structures and the ballasted construction act as retaining structures (fig.3 Final stage).
In order to limit the reaction of the apron against the sheet piles of the cellular structure, a programmed groundwater lowering had to be applied for the permanent stage. In this concept the opportunity to regulate the water pressure underneath each chamber and under the ballasted construction was created by the installation of intermediate bentonite-cement screens namely along both sides of the ballasted construction and also along the side of the lock chamber (fig. 4).
Permanent discharge wells are used for dewatering the ground within the cellar structures and underneath the ballasted construction till - 18,00, simultaneously with the pumping out of the gate chamber itself by using submersible pumps.

It has been controlled that for the upstream head where the Assian clay disappears, the influence of the programmed dewatering has no significant influence at the reference point in the borough of Zeebrugge.

6. EXECUTION OF THE GROUNDWATER LOWERINGS

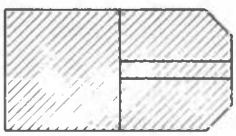
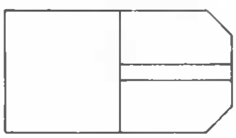
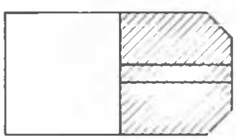
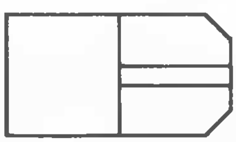
At the construction site the initial groundwater level was situated at + 3,00, this is near the mean sea level. When starting the construction of the sea lock (July 1976), due to groundwater lowerings at other construction sites, the groundwater level was still lowered to + 1,40.
For the groundwater lowering within the enclosure of the upstream head, 38 discharges wells with an overall diameter of 500 mm, a length of 30 m and a capacity of 25 m3/h were installed.
For the refeeding of the watertable, a total number of 146 refeeding walls were installed by the firm Tjaden (fig. 1) (Tjaden, 1979).
For these refeeding wells, the overall diameter was 400 mm, the length 30 m and the estimated mean long term capacity 5 m3/h. All possible precautions have been taken in order to prevent the aeration of the water used for the refeeding of the watertable.
Special filter tanks were provided in order to filter the ferruginous salts and the sand particles out of the refeeding water before entering the refeeding wells. The refeeding wells were cleaned by airlift and by pumping, up to twice a week.
For the groundwater lowering within the downstream head and the bridge cellars at the upstream and downstream side, resp. 10, 8 and 4 discharging walls with a capacity of 25 m3/h were provided. For the head and bridge cellar at the downstream side, a slight number of discharge wells ensure a lowering of the watertable in the eocene fine sands in order to guarantee the stability of the bottom of the excavation.

7. DESCRIPTION OF THE PERFORMED MEASUREMENTS

In order to be able to predict the changes of the piezometric level over a rather large area, caused by an additional pumping or refeeding, a computerprogram "RABAT" was elaborated from the observations already at disposition in previous stages of the works (Entr. S.B.B.M.,1976).

Table II. Different considered solutions

General assumptions : Groundwaterlowering till - 21.00m
Permeability of the bentonite cement screen 1.10^{-8} m/sec
Permeability of the injected horizontal layer 5.10^{-7} m/sec

Considered solution		Discharge m^3/h	Refeeding charge m^3/h (calculated x 0.65)	Piezometric level	
				At the edge of the head	At 100m of the edge of the head
Injected horizontal layer (thickness 3m) over the complete head and bentonite cement screens reaching at - 32.50m		125	—	-1.10	+0.20
Bentonite cement screens reaching at -32.50m		466 540 615 628	— 182 375 400	-8.43 -6.45 -4.39 -4.07	-5.55 -3.03 -0.40 0.00
Injected horizontal layer (thickness 3m) underneath the chambers only and bentonite cement screens reaching at - 32.50m		408 464 525	— 159 320	-7.76 -5.94 -4.08	-4.69 -2.36 0.00
Bentonite cement screens reaching at -45.00m		300 330 350	— 117 191	-4.80 -3.16 -2.13	-3.07 -1.19 0.00

Bentonite cement screen -32.50m   Bentonite cement screen -45.00m   Injected horizontal layer

This program was based on :
- the drawdown curves deduced from the different pumping tests, performed in the Pleistocene coarse sands and the eocene fine sands;
- the theories of Dupuit and De Glee (Kruseman and de Ridder, 1970)
- the principle of superposition.

Important to be noted that the piezometric levels could be plotted on different scales for all points of the site and neighbourhood.
The validity of the program RABAT was checked when the groundwater table within the bridge cellar at the upstream side was lowered to the level - 21,00. After a pumping period of ca. one month, a stabilised discharge of 135 m3/h was obtained.
The drawdown curve registered outside the diaphragm wall enclosure of the cellar is given on the fig. 5 for the directions parallel and perpendicular to the axis of the sea lock (curves 3) On the fig. 5, these curves can be compared with the drawdown curves, calculated for a discharge of 135 m3/h and for the hydraulic constants given in table I, by the following methods :
a) the computerprogram RABAT :
- drawdown curve parallel to the axis of the sea lock (curve 2a of fig. 5).
- drawdown curve perpendicular to the axis of the sea lock (curve 2b of fig. 5).

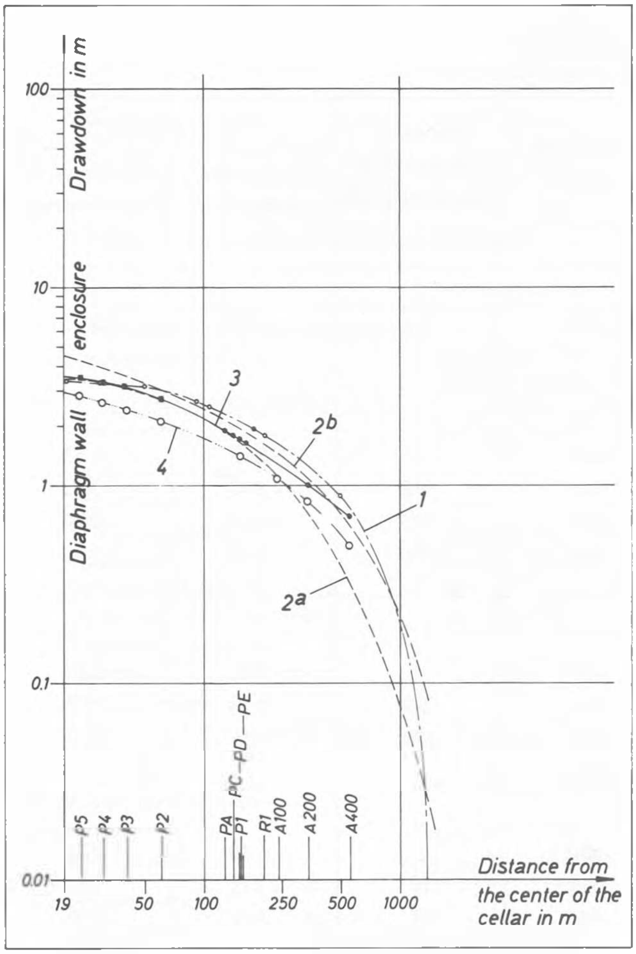


fig.5 Cellar of the basculing bridges at the upstream side Drawdown curves

These two curves can be considered as limiting the zone of the normally expected drawdown.
b) the mathematical axisymmetric model based on the finite element method (curve 1 of fig.5)
c) the measurements on an electric model where the presence of a small pound situated in the vicinity of the bridge cellar was taken into account (curve 4 of fig. 5).

The comparison of the different drawdown curves indicates that the curve obtained by the extrapolation of the disponible drawdown curves (program RABAT) provided the best agreement with the observed drawdown curves.
During the execution of the groundwater lowering at the upstream head, the discharges and refeeding charges were systematically observed. So it was always possible to compare the measured discharges and refeeding charges to the values calculated with the help of the program RABAT (see table III).
The refeeding charges are in complete agreement The difference between the computed and observed values of the discharges is due to clay lenses which are still present over a part of the head. Furthermore it could also be pointed out that from the 146 installed refeeding wells, only 70 wells were in continuous use, having each a mean refeeding charge of 5,7 m3/h. So it could be stated that the registered refeeding charge was only half the disponible refeeding capacity.

Table III

	Discharge m^3/h	Refeeding charge m^3/h	Piezometric Level at 100 m from the edge of the head
Calculated by program RABAT	725	418	0.00
Calculated by mathematical model (FEM)	628	400 *	0.00
Observed	440	325	0.00

*(* calculated x 0.65)*

8. EFFICIENCY OF THE "WATERTIGHT" SCREENS

From the registrations performed during the groundwater lowerings at the downstream head, it could be deduced that the permeability of the bentonite-cement screens varied between 5 and 10 x 10^{-8} m/sec.
The efficiency of a peripherical impervious screen not reaching an impervious substratum largestly depends on the degree of anisotropy of the soil layers concerned.
In the supposition that the eocene fine sands have not been present, the discharge rate without refeeding would have been 1900 m3/h with screens reaching - 32,50 and 1400 m3/h with screens reaching - 45,00.
With an "impervious" screen of k = 1 x 10^{-6} m/sec the discharge rate should have been 900 m3/h and the recharge rate 560 m3/h.
In case no screen should have been designed, the discharge should have increased to

1550 m2/h, the recharge rate to 1200 m3/h and should have no longer been realisable.

9. CONCLUSIONS

- The experience of the site of Zeebrugge is that in similar groundconditions it is well possible to make dry excavations of the importance of 100 x 200 m under the protection of watertight screens reaching the eocene fine sands. Due to the absence of the bottom and the presence of constructions at a small distance of the construction site, a refeeding of the watertable has been necessary and well effective.

- This solution was favorised by the anisotropy of the encountered layers. The accurate determination of the hydraulic constants (horizontal and vertical permeability) of these layers allowed to obtain a rather good agreement between the elaborated prognoses and the observed values

- Based on these values, the influence of the different groundwater lowerings has been satisfactorily extrapolated by using a FEM program and the RABAT program based on the theory of De Glee

- The efficiency of the impervious screens is relatively important especially when there is no natural watertight bottom.
In the conception of such a refeeding installation, an appropriate security coefficient has to be imposed taking the great inaccuracy of the hydrological values and the method of calculations into account, and last but not least to permit the regeneration of the refeeding wells which, in the case of Zeebrugge, has been satisfactorily working during more than two years.

REFERENCES

Berleur E & Fremout J, (1978)
Quay walls for the access channel to the new sea lock at Zeebrugge - Quay walls for the containerdock,
7th International Harbour Congres - Antwerp

Berleur E & Fremout J, (1978)
New Sea Lock at Zeebrugge,
7th International Harbour Congres - Antwerp

Davidenkoff E & Franke O, (1965)
Untersuchung der raumlichen Sicherströmung in eine umspandete Baugrube in offenen Gewässern,
Die Bautechnik (42), 9, 298-307.

De Beer E & Maertens J, (1976)
Metingen op een electrisch model van de keldering aan het bovenhoofd van de Zeesluis te Zeebrugge,
Report 6113-76/376, State Geotechnical Institute,

Entreprises S.B.B.M. (1976)
Computerprogram RABAT for the calculation of groundwater lowerings",
Internal Note,

Kruseman G & de Ridder N, (1970)
Analysis and evaluation of pumping test data,
Bulletin II, International Institute for Land Reclamation and Improvement,
Wageningen

Rammant J & Backx E, (1976)
GROW 1. A program for anisotropic soil fluid flow with free or artesian surface. Application to excavations at lock of Zeebrugge.
Publication of the Catholic University at Leuven,

Tjaden H, (1979)
De techniek van de retourbemaling,
Publication Tjaden Ltd.

ZEEBRUGGE - Sealocks - Aeroview May 1980