GEOTECHNIQUE OF THE SCHELDT SURGE BARRIER
FOUNDATION DESIGN OF THE MAIN PIERS

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ABSTRACT

The subsoil on the site of the planned Scheldt Surge Barrier consists of Holocene sandy deposits, Tertiary glauconitic sands and Rupelian clay, who is a stiff fissured overconsolidated Tertiary clay.

On base of the results of 85 cone penetration tests, 24 borings with Menard pressuremeter tests, 3 borings with selfboring pressuremeter tests, a seismic survey and 27 borings for sampling and laboratory tests, soil design values have been determined for the different strata.

The main piers have a height of 126 m and are submitted to important horizontal forces.

Different foundation types have been examined : a raft foundation, a mixed raft-slurry walls foundation, a mixed pile-raft foundation, and a pile groupe foundation.
The adopted criteria for the judgement of the different foundation types and for the proper design of foundation are given.

Finally the pile group foundation has been retained. The design of this foundation type is discussed in this contribution :

- determination of the ultimate bearing capacity of a pile
- bearing capacity of a pile group
- horizontal stability of the pile group foundation
- pile group effect
- prediction of the short and long term settlements
- estimation of the possible tilt from uneven settlement of a pier.

A synopsis of the execution sequence of the foundation is given.

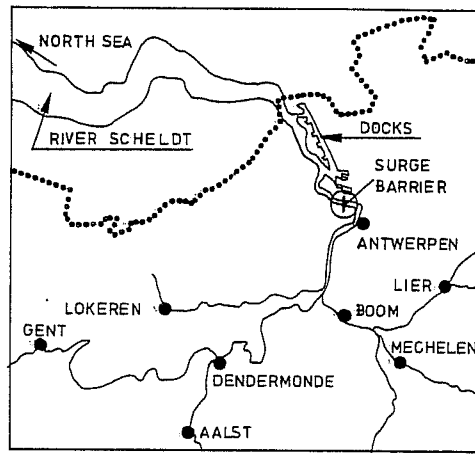


Fig 1 : Situation Surge Barrier

INTRODUCTION

The storm surge barrier on the Scheldt near Antwerp (fig.1) will consist of a series of mobile metallic drop gates, lifted between concrete piers and abutments (fig. 2). In open position the drop gates hang in mid-air and do not interfere with the daily tidal movement.

When a storm surge occurs the gates are lowered.

High waterlevels can only occur downstream the barrier and the entire upper basin of the Scheldt basin is protected against flooding.

Extensive foundations are necessary requiring an intensive site investigation.

The subsoil consists of Quaternary deposits covering Tertiary glauconitic sands and Rupelian clay also known as the Boom clay.

Considering in the Scheldt River the Rupelian clay, which has a thickness of ca 70 m, is encountered at a depth of less than 10 m it is clear that the foundation of the main piers ought to be installed in the Rupelian clay. Furthermore an extensive research program on the bearing capacity of piles, driven in the Rupelian clay at Kontich, revealed a rather large scatter of the characteristics of the Rupelian clay (De Beer e.a., 1977)

Starting from these data an extensive soil investigation campaign, was planned in order to determine not only the mean value but also the scatter of the different characteristics of the Boom clay.

Within the framework of this campaign, 85 static cone penetration tests, 27 borings with prelevation of disturbed and undisturbed samples, 24 borings with Menard pressuremeter tests and 3 borings with self boring pressuremeter tests have been performed.

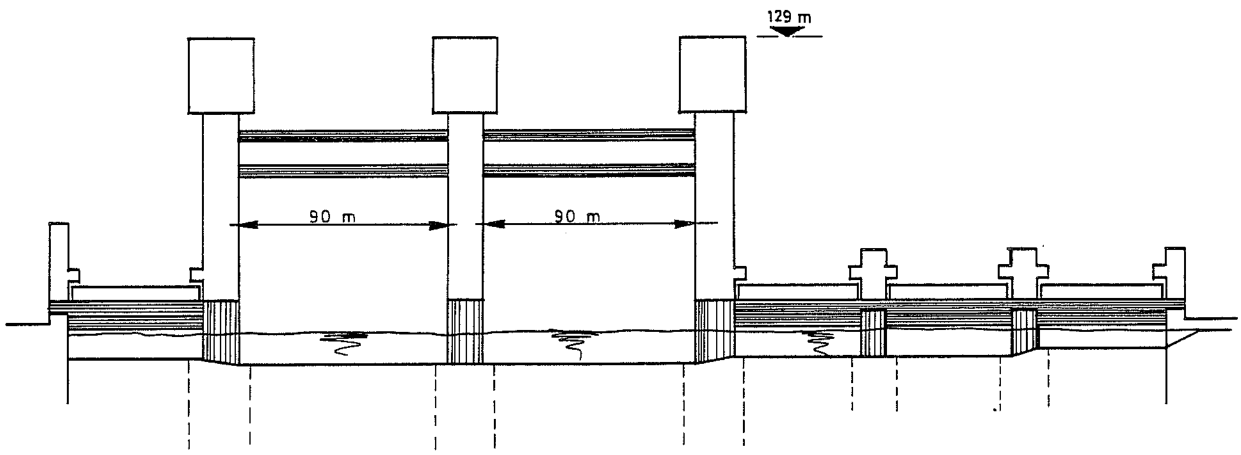


Fig 2 Design storm surge barrier

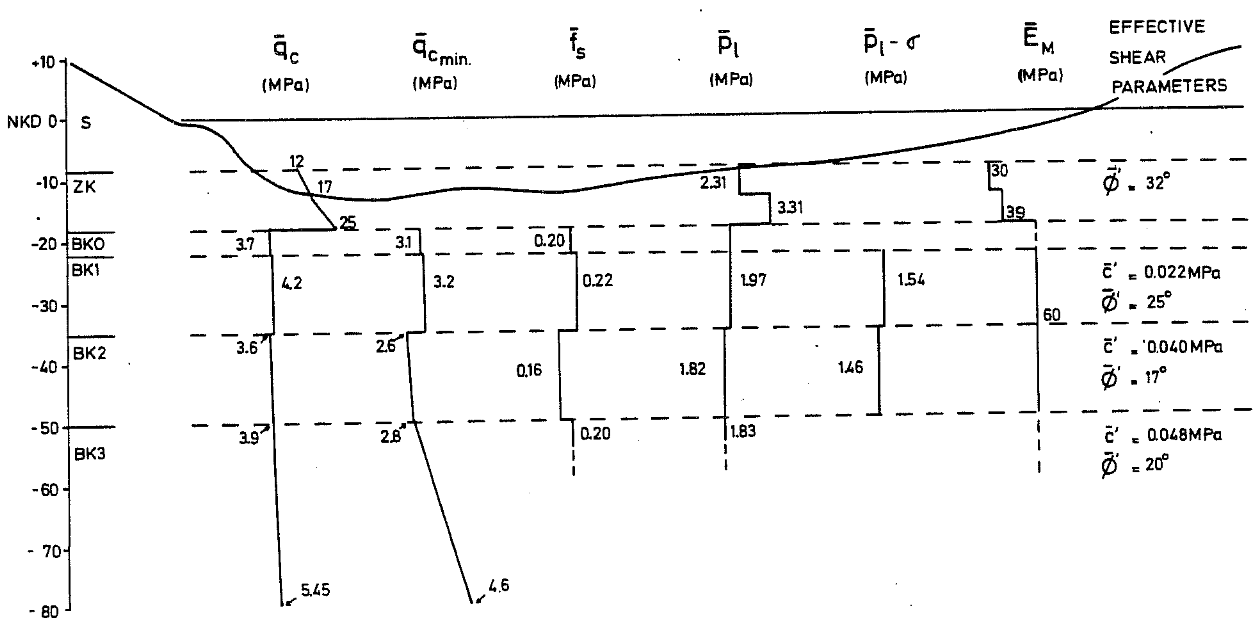


FIG. 3 SYNOPSIS OF GEOMECHANICAL PROPERTIES ON THE DAM SITE

Schittekat, Henriet and Vandenberghe (1983)report the results of the soil in better knowledge of the Boom clay.

The Boom clay has been subdivided in five main geotechnical subdivisions called BKO which is the weathered top of the clay, BK1, a silty clayey unit from level -20 to -35 m, BK2 from -35m to -50 m and which is more clayey, BK3 from -50 to -80 m and BK4 from -80 to -90 m, which consists of silty clayey fine sand.

The results of the soil survey are summarized in fig. 3 for each unit of the geotechnical subdivision.

DETERMINATION OF DESIGN VALUES

1. Undrained shear strength c_u

In the research program at Kontich, dealing with the bearing capacity of different piles driven in the Rupelian clay, a marked decrease of the undrained shear strength has been observed when the tested volume was increased.

The same observation had been previously made by Marsland (1973) for the London clay and Kérisel (1967) for the clay at Bagnolet.

The observations made at Kontich are summarized on the figure 4 taken over from De Beer e.a. (1977)
On this figure, one observes

$c_{u,b}$ = the undrained shear strength corresponding to a diameter b of the loading body
$c_{u,peak}$ = the peak value of the undrained shear strength
$c_{u,res}$ = the residual value of the undrained shear strength.

From these observations the following rule was deduced for the variation of the undrained shear strength (peak value) with the base diameter of a pile driven in the Rupelian clay.

$$c_u = c_u(\emptyset\,C.P.T.) \times \frac{1}{1-0,012(\emptyset/3,6-1)} \quad (1)$$

with $\emptyset$ = diameter of the loading body in cm.
$\emptyset$,CPT = diameter of the cone =3,6cm
The results of the static cone penetration tests and vane tests performed within the scope of this research program at Kontich presented a rather large scatter. The scatter of the $c_u$peak and $c_u$res values deduced from the results of these tests and corresponding to diameters of 3,6 cm and 4 cm are indicated on fig. 3.

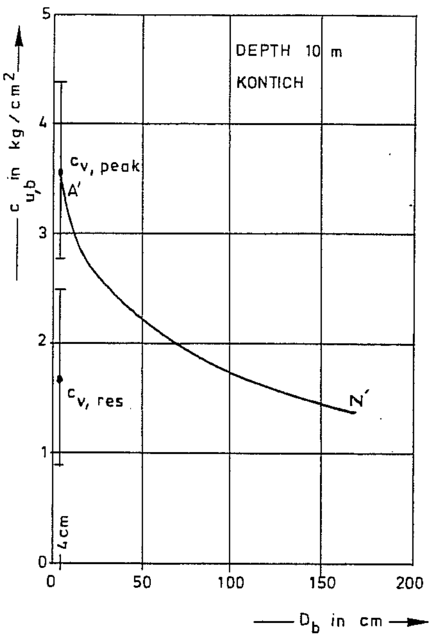


Fig :4
INFLUENCE OF THE DIAMETER OF THE LOADING BODY ON THE IN SITU UNDRAINED SHEAR STRENGTH OF A STIFF FISSURED CLAY. (AFTER DE BEER ET AL. 1977)

The value c_u,res-Δ has been defined as the lower boundary of the value of c_u,res,Δ being the dispersion of the values of c_u,res for a diameter of 4 cm. It was observed that the dispersion Δ of the peak is identical with the dispersion of the residual values of the undrained shear strength.

Considering that on the site of the planned Scheldt Surge Barrier the top of the Rupelian clay is encountered at a depth of at least 30 meters below the deck of the self elevating platform, it was not possible to perform vane tests, taking into account the stiffness of the clay and the torsion of more than 30 m of rods.

Consequently the values of c_u,peak, c_u,res and c_u,res $-\Delta$ had to be deduced from :

- the results of the laboratory tests (UU and CU triaxial tests)
- the cone penetration resistances
- the limit pressures, determined by the Menard pressuremeter tests
- the results of the self boring pressuremeter tests performed on the left bank
- the velocity of the S-waves in the seismic tests.

TABLE 1

UNDRAINED SHEAR STRENGTH - LABORATORY TESTS

	$c_{u,res} - \Delta$ (MPa)	$c_{u,res}$ (MPa)	$c_{u,peak}$ (MPa)
UU triaxial tests			
BK1	0,10	0,18	0,38
BK2	0,13	0,19	0,40
CU triaxial tests			
BK1	0,08	0,13	0,27
BK2	0,10	0,13	0,27

* The values of the undrained shear strength deduced from the results of the UU triaxial tests are given in table 1. Since only a slight increase with depth of the mean values was observed (Schittekat, Henriet, Vandenberghe 1983), it is convenient to consider a constant value for each geotechnical subdivision.

The value for which only 10 % of the available values are smaller has been considered as c_u,res$- \Delta$. Finally the value of c_u,peak has been obtained by using the relationship (2) :

c_u,peak = St.c_u, res (2)

where St is the sensitivity of the clay (=2,1 as determined at Kontich).

The values of the undrained shear strength, deduced from the results of the CU triaxial tests for $\sigma_3' = 2/3\ \sigma'_v$ are also presented in table 1.

* From the measured cone resistances, the values of c_u,peak and c_u,res can be obtained from relationship (3) where q_c is the mean cone resistance and N the coefficient of Skempton (N = 9 for clay)

c_u,peak = qc/N (3)

Admitting that the value of c_u,peak $- \Delta$ corresponds to q_c,min (mean minimum cone resistance) one obtains with the data of fig. 2, the values of c_u,res and c_u,res $- \Delta$ given in table 2.
They remain constant for each subdivision since no significant increase of cone resistance with depth has been observed (Schittekat, Henriet, Vandenberghe 1983), see also fig. 5.

CONE PENETRATION TESTS ENVELOPE

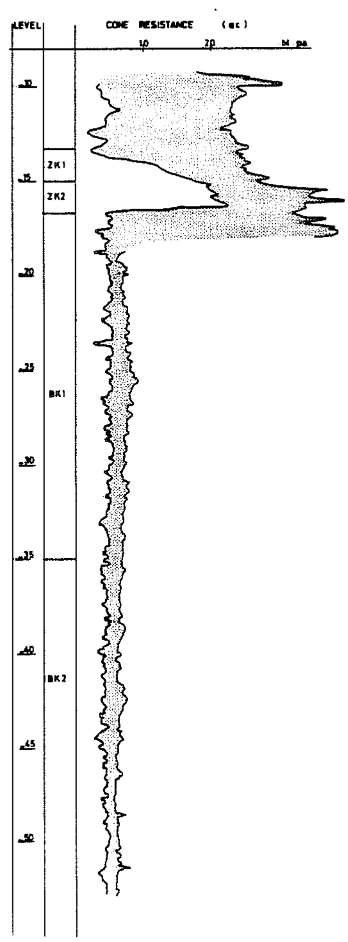


Fig : 5

TABLE 2

UNDRAINED SHEAR STRENGTH - C. P. T.

	$c_{u,res} - \Delta$ (MPa)	c_u res (MPa)	$c_{u,peak}$ (MPa)
BK1	0,11	0,22	0,46
BK2	0,08	0,19	0,40

TABLE 3

UNDRAINED SHEAR STRENGTH - PRESSUREMETER TESTS

	$c_{u,res} - \Delta$ (MPa)	$c_{u,res}$ (MPa)	$c_{u,peak}$ (MPa)
BK1	0,14	0,17	0,36
BK2	0,12	0,15	0,31

* From the results of the Menard pressuremeter tests the value of the undrained shear strength has been determined with the empirical formula elaborated by S. Amar and J. Jézéquel (1972)

c_u =(pl-po)/10+0,025 (MPa) (4)

By applying this formula to the results of the pressuremeter tests, given on fig. 2 one obtains the values of table 3 by considering the value corresponding to p_l as c_u,res and the value corresponding to $p_l - \sigma$ as $c_u$res $- \Delta$.

* From the results of the self boring pressuremeter tests the undrained shear strength has been determined according to the method proposed by Gibson and Anderson (1961) From the values obtained in this way, the values, given in table 4 have been deduced, considering the value of $c_u$res as corresponding to the mean value of all available data and c_u,res $-\Delta$ as corresponding to the value, for which only 10 % of the available values are smaller.

In contrast to the other tests, the self boring pressuremeter tests clearly outlined an increase with depth of the undrained shear strength. For convenience, only mean values of the geotechnical subdivision have been retained.

* From the results of the seismic tests the value of the shear modulus G has been determined from shear wave velocities. The value of the undrained shear strenght can be obtained by the relation proposed by Seed and Idriss (1970) for clay :

$G = 2000\ c_u$.

The shear modulus G has been directly determined from tube wave velocities (Henriet, Schittekat, Heldens 1983) :

G = 230 MPa between -20 and -30 and
G = 250 MPa between -23 and -35 m.

From these data, Cu values of resp. 0,11 and 0,13 MPa have been deduced.

The values of the undrained shear strength deduced from the results of the different investigation methods, are related to different volumes of tested

TABLE 4

UNDRAINED SHEAR STRENGTH - S. B. P.

	$c_{u,res} - \Delta$ (MPa)	c_u res (MPa)	$c_{u,peak}$ (MPa)
BK1	0,13	0,23	0,49
BK2	0,16	0,26	0,55

soil. Consequently a transformation had to be made to relate these values to a standard diameter. For the pressuremetertest, an equivalent diameter of 30 cm has been introduced.

By applying relationship (1) one obtains
c_u ($\emptyset$ = 3,6 cm) = 1,05 x c_u ($\emptyset$ = 10 cm)
c_u ($\emptyset$ = 3,6 cm) = 1,22 x c_u ($\emptyset$ = 30 cm)

The adjustment of the c_u values for the diameter has always been done for the c_u,peak or c_u,res values, the value of c_u,res - Δ being considered as unaffected by the dimensions of the testing probe.

The values of c_u,res - Δ , c_u,res and c_u,peak obtained in this way and all related to a diameter of 3,6 cm are given in table 5. For the interpretation of these results one should have in mind that the self boring pressuremeter tests have been performed on the left bank, while the largest part of the other tests have been performed in the Scheldt River, where the bottom level is 18 m lower than the soil level on the bank, involving corresponding slightly higher undrained shear strength values. One has observed over numerous C.P.T. that the mean cone resistance is 3 % higher on the bank than on the river.

The cone resistances in the Kattendijk sands are 20 % higher on the bank.

The values of the undrained shear strength are estimated at an average of :

	BK1	BK2
c_u,res - Δ =	0,11 MPa	0,12 MPa
c_u,res =	0,21 MPa	0,20 MPa
c_u,peak =	0,44 MPa	0,43 MPa

2. Compressibility

The compressibility of the Rupelian clay has been deduced from the results of the

- Oedometer tests
- Menard pressuremeter tests
- Self boring pressuremeter tests

* Oedometer tests - Although more than a hundred of classical oedometer tests have been performed, the value of the compression modulus introduced in the settlement calculations has been deduced from the results of the so called geological oedometer tests, formerly proposed by J. Brinch Hansen (1964). Following this procedure, the sample is in a first stage charged to the maximum effective stress σ'_{max}. that existed during the geological history of the considered layer ; afterwards the sample is unloaded to a minimal stress σ'_{min}, deduced from the maximum erosion, that has taken place after the maximal load was exerted. Then the load on the sample is increased untill the natural stress, existing at the level where the sample has been taken. Starting from this stress, different loading or unloading stages are applied and the compression or unloading moduli,are determined in accordance with the working stresses, supposed to act during and after construction of the piers.

The values obtained in this way present a rather large scatter. The mean value of the compression modulus was C = 75 and curiously enough, it only sligthy increased with depth. This value has been introduced in the settlement calculations.

* Menard pressuremeter tests - The pressuremeter moduli deduced from the performed Menard pressuremeter tests presented a very large scatter. From the obtained results it could be seen that they were influenced by the drilling procedure.

TABLE 5

UNDRAINED SHEAR STRENGTH - (Ø 3,6 cm)

	$c_{u,res} - \Delta$ (MPa)	$c_{u,res}$ (MPa)	$c_{u,peak}$ (MPa)
BK1 layer			
UU-triaxial tests	0,10	0,19	0,40
CU-triaxial tests for '3 = 2/3 'v	0,08	0,14	0,29
CPT	0,11	0,22	0,47
Menard pressuremeter tests	0,14	0,21	0,44
Selfboring pressuremeter tests	0,13	0,29	0,61
Tube waves	0,11 - 0,12 (very large Ø)		
BK2 layer			
UU-triaxial tests	0,13	0,20	0,42
CU-triaxial tests for '3 = 2/3 'v	0,10	0,14	0,29
CPT's	0,08	0,19	0,40
Menard pressuremeter tests	0,12	0,18	0,38
Selfboring pressuremeter tests	0,16	0,32	0,67

When making the hole in small passes (4 to 6 m) and performing the tests immediately afterwards, higher values of the Menard pressuremeter modulus were obtained than when larger drilling travels were adopted. The results were also influenced by the time interval between the end of drilling and the execution of the pressuremeter tests.

Consequently, the pressuremeter moduli are bimodally distributed. One population with median value equal to 20 MPa corresponds to tests performed in slighty disturbed holes, the other with median value significantly higher than 50 MPa corresponds to tests in nearly undisturbed boreholes. Following the experience in the use of the results of the pressuremeter, it is recommended to use the lower average value of E_M equal to 20 MPa for the determination of the subgrade reaction modulus of drilled piles and to restrict the pressuremetermodulus (E_M) to 60 MPa in settlement predictions.

* Self boring pressuremeter tests. From the results of the self boring pressuremeter tests the value of the shear modulus G was determined at different levels. Considering that there exists no sensible variation of these values with depth, the mean value G=37MPa has been adopted.

The Boom clay behaves as a stratified or transversely isotropic (orthotropic) material in which a rotational symmetry of properties exists within the plane of the strata. Such a material possesses five independent elastic soil parameters. The equations of deformation and restrictions on the parameters are discussed by Gibson (1974).

E_V and E_H are the Young's moduli in vertical direction, ν_{HV} and ν_{HH} are the poisson's ratios for transversely isotropic material and G_{VH}, G_{HH} are the corresponding shear moduli.

The parameter G_{HH} gives the shear stiffness for shearing in the horizontal plane and is thus relevant to the analysis of the effects of cylindrical cavity expansion due to pressuremeter tests, so that G of the pressuremeter is equal to G_{HH}.

$$G_{HH} = 37 \text{ MPa}$$

$$G_{HH} = E/(2(1 + \nu_{HH})) \qquad (5)$$

$$n = E_H/E_V \qquad (6)$$

$$\nu_{HH} = 1 - n/2 \qquad (7)$$

1.144 From previous tests, performed for the construction of the Kennedy tunnel under the Scheldt River, at Antwerp (De Beer 1971) it was found that n = 1,5 at 40 m depth.
From relations (5), (6), (7) and the general stress-strain relations, it is found that with $\nu_{VH} = 0,5$: E_H = 92,5 MPa ;
E_V = 61,7 MPa ; ν_{HH} = 0,25 and
G_{VH} = 20,6 MPa. The value of E_V = 61,7 MPa is in fair agreement with the value of 70,6 MPa found by compression tests on large blocksample cutted of by tunnel excavating at 80 m depth in the neighbourhoud of the site (De Beer, Buttiens 1966).

For drained conditions, a general admitted value of the Poisson ratio of
$\nu'_{HH} = \nu'_{HV} = 0,25$ has been retained. Using the relation (8), whose validity has been demonstrated for clays by Absi (1965).

$$\frac{E}{1+\nu} = \frac{E'}{1+\nu'} \qquad (8)$$

one obtains : G_{VH} = 20,6 MPa ;
E'_V = 51,4 MPa ; G_{HH} = 37 MPa and
E'_H = 92,5 MPa.

GENERAL DESIGN OF THE FOUNDATIONS

In this paper, only the foundation design of the main piers will be discussed.
These piers have a height of 129 m, a weight of 730 MN and are submitted to important horizontal forces (116 MN).
The maximum dimensions of the foundation were imposed by the hydraulical conditions and were of
25 m x 100 m. (See fig. 6)

* Raft foundation.

The first examined foundation method was that of a raft foundation installed at the levels -23, -25 or -27 m.
These raft foundations had to be installed by excavating and concreting under water. Although the overall stability was assured, this foundation method has been rejected as no guarantee could be obtained concerning the expected swelling of the clay during the excavation and concerning the absence of mud inclusions during the pouring of the concrete. The risk of differential settlements and a relative inclination of the construction could not be taken.

The installation of a raft foundation was examined a second time in a later stage. Then a foundation level of -18,00 (or about 0,50 m above the upper limit of the clay) was considered, and the planned execution method provided in the installation under water of a drain layer and of a concrete slab. After the construction of the pier, an injection of the drain layer was foreseen. This foundation method has not been retained as the predicted inclination of the construction due to differential settlements was not compatible with the imposed limitation of the designed electromechanical equipments for the lifting of the gates.

* A foundation of prefabricated caissons.

A foundation of prefabricated caissons, sunk to the desired level under compressed air, in order to obtain a direct or raft foundation, or placed on previously installed bored piles, has been considered next. Although it seemed that a solution could be found for the many technical problems, imposed by this foundation method, it has been rejected for nautical reasons, as the many interruptions of the navigation traffic on the Scheldt River during the transport and the sinking operation of the prefabricated caissons was considered to be incompatible with the normal exploitation of the harbour of Antwerp.

* A mixed raft slurry walls foundation.

In order to avoid the difficulties, which were encountered for the installation of a direct foundation and especially concerning the good contact between the foundation basement and the underlaying clay, a foundation supported by slurry walls was examined. The following execution procedure was foreseen :

- installation of a cofferdam with sheet piles down to the level -20,00 m ;

- backfilling of the cofferdam with sand to a level situated above the mean water level in the Scheldt River and installation of a groundwater lowering system within the cofferdam ;

- installation of slurry walls around the foundation and parallel to the long side in the middle of it. Due to the presence of septaria in the Rupelian clay and to the fact that the slurry walls had to be installed at a short distance of the sheet pile bases, a secant pile wall was envisaged with piles of 2,30 m diameter and having their base at the level -35,00 m

- under water excavation until the level -18 m or -20 m and installation of a concrete slab with a thickness of 5 m. During these processes the slurry walls act as retaining walls.

- construction of the pier in the dry.

This foundation method has not been retained as the installation of the secant pile walls was very expensive and a pile raft foundation seemed finally to be more economic.

* A mixed pile raft foundation with 94 bored piles of diameter 1,30 m and at a depth of 29,10 m depth has been considered. The structural concept followed in the analysis of this foundation is that the foundation block and piles sheare in transmitting both vertical and horizontal loads to the soil. The load distribution between the block and the piles depends on the stiffness of the piles with respect to the subgrade soil.

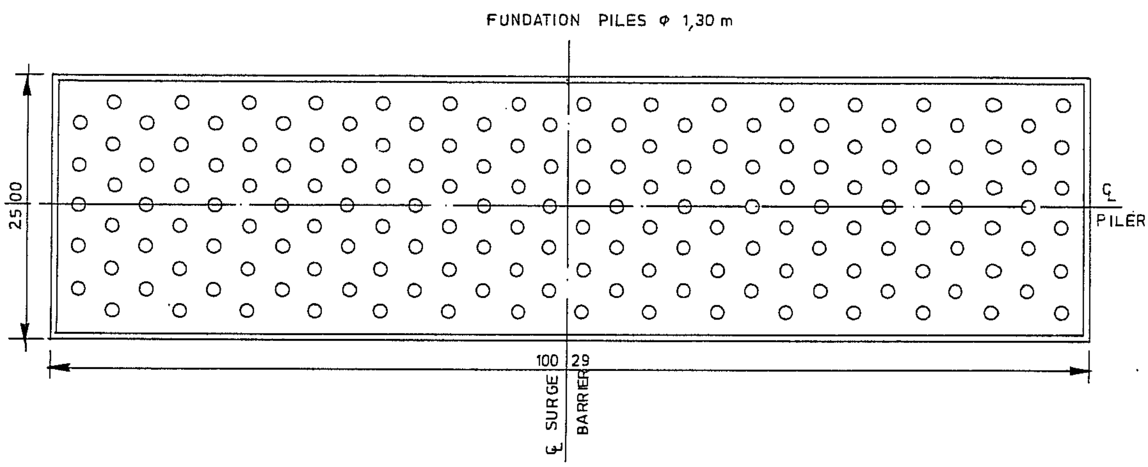
A primary simplified analysis, complemented with pile group structural analysis has shown that the edge and corner piles, even if admitting an uniform repartition of the load on the piles would be submitted to important forces by closing gates or exceptional wind so that the clay beside the piles would be loaded till plastic deformation might possibly undergo irreversible deformations, increasing at each additional cycle.

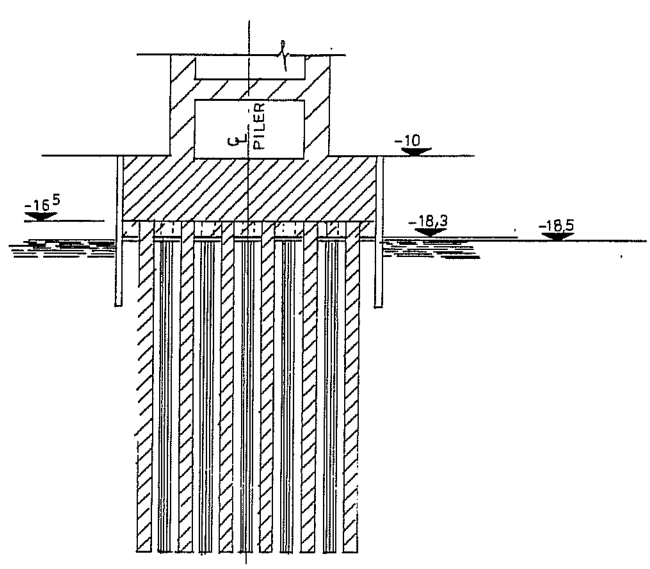
Moreover if no uniform repartition of loads is admitted and the pile group effect is introduced, the corner and edge piles are additionally loaded. This has been recently observed and measured (Joustra e.a. 1977, Levy 1981, Coole e.a. 1981). Hence the pile raft foundation system has been rejected.

* A foundation consisting of a raft with a thickness of 5 m supported by 165 bored piles with a diameter of 1,30 m and having their base at the level -49 m has finally been retained (see fig. 6).

The piles are built under use of bentonite throughout the piling operations. The design and the planned execution method of this foundation are discussed hereafter.

Fig 6 : Fundations main piers





A possible alternative of cylindrical piles could consist of an equal number of underreamed piles. Underreamed piles are generally used in London clay which is in some respects similar to the Boom clay. It is clear, however, that the basic properties of both clays are different and because there is no experience of constructing underreamed piles in the Boom clay, should the case arise it should be careful to carry out some further investigations such as drilling and underreaming prototype piles. The main advantage of this pile design is that the piles are essentially point bearing and it prevents the possible plastic deformations of the soil below the corner or edge piles. On the other hand they request drilling without bentonite, which would affect the friction (Cambefort 1963).

DESIGN OF THE PILE GROUP FOUNDATION

Concerning the design of the pile group foundations only some main points will be discussed.

The bearing capacity of a single pile has been determined following the existing calculation methods.

Considering however that no experience concerning the installation of long bored piles (as exists f.i. for the London clay) is available for the Boom clay, the installation of trial piles will be necessary, in order to control that the parameters, introduced in the calculations, are in good agreement with the adopted execution procedure. Based on the results of these loading tests the definitive length of the piles has to be optimalised.

DETERMINATION OF THE ULTIMATE BEARING CAPACITY OF A SINGLE PILE

The bearing capacity of a single pile has been calculated following the method of De Beer (1977)

* Point resistance : the ultimate base resistance was calculated from the results of the static cone penetration tests. Therefore the following values have been determined :

$q_{c,min,min}$ = the mean value of the minimal cone resistances, measured for the most unfavourable test between -40 and -50 m.

$q_{c,min,max}$ = the mean value of the minimal cone resistances, measured for the most favourable test between -40 and -50 m.

η = dispersion (for the Boom clay η = 1,5 was deduced from the tests at Kontich)

q_c^* = the lower value of $q_{c,min,min}$ and $q_{c,min,max}/\eta$

These values have been determined for each pier by taking into account the results of the tests, performed at the spot or near the considered pier.

The unit ultimate bearing capacity qb,r at the base of a bored pile with a diameter D is then given by the formula (De Beer 1977)

$$q_{b,r}=0,8\left[1-0,01((D/0,036)-1\right]\cdot(q_c^*-\sigma_v)+\sigma_v \qquad (9)$$

where :

0,8 = a reduction coefficient deduced from the theoretical comparison between the bearing capacity of a driven and a bored pile in clay (De Beer, 1964

σ_v = the total vertical stress at the pile base.

The value of the unit ultimate bearing capacity calculated following this method is the lower limit of the ultimate bearing capacity, as it has been deduced from the minimal measured ($q_{c,min,min}$) or calculated ($q_{cmin,max}/\eta$) cone resistances.

For the determination of the admissible unit load only a safety factor of 1,4 was introduced, as it was assumed that even at the most unfavourable vertical, no excessive deformations can occur when a safety factor of 1,4 is applied to the calculated minimal ultimate bearing capacity. The admissible unit load is then given by the relation :

$$qb,a = (qb,r - \sigma_v)/1,4 + \sigma_v \qquad (10)$$

The values of $q_{b,r}$ and $q_{b,a}$ obtained in this way for the central pier and for piles with a diameter of 1,3 m and installed at level -49m are :

$q_{b,r}$ = 1,53 MPa
$q_{b,a}$ = 1,35 MPa

These values have been determined for each pier by taking into account the results of the tests, performed at the spot or near the considered pier.

* Shaft resistance - the ultimate shaft resistance $f_{s,r}$ of the pile has been determined following three methods.

1) by the relation (De Beer et all 1977)

$$f_{s,r} = \alpha_a \, c_{u,peak,min} \qquad (11)$$

with cu,peak,min = the minimal peak value of the undrained shear strength α_a = 0,17 coefficient extrapolated from the results of the pile tests at Kontich.

For the determination of the value of c_u,peak,min it has been assumed that this value is given by the expression

$$c_{u,peak,min} = c_{u,peak} - \Delta$$

and that Δ is the same for the peak and for the residual shear strength and unaffected by the diameter. As the dispersion becomes smaller for increasing diameters, the values obtained following this assumption are rather conservative. So one obtains for the BK1 layer.

$$c_{u,peak,min} = 0{,}455 - 0{,}105 = 0{,}350 \text{ MPa}$$

and for the BK2 layer =

$$c_{u,peak,min} = 0{,}390 - 0{,}110 = 0{,}280 \text{ MPa}.$$

As nearly the same pile length is situated in the BK1 and the BK2 layer the mean value of the $c_{u,peak,min}$ values determined for the BK1 and BK2 layers, has been introduced =

$$c_{u,peak,min} = \frac{0{,}35+0{,}28}{2} = 0{,}315 \text{ MPa}$$

Hence the value of the ultimate shaft resistance is :

$$f_{s,r} = 0{,}17 \times 0{,}315 = 0{,}054 \text{ MPa}$$

The admissible shaft friction $f_{s,a}$ obtained by introducing a safety factor of 1,4. One obtains :

$$f_{s,a} = fs,r/1{,}4 = 0{,}054/1{,}4 = 0{,}039\text{MPa}$$

2) by the relation : (De Beer e.a. 1977)

$$f_{s,r} = \alpha_d \cdot \frac{1}{\eta} \cdot \alpha_{ss} \cdot f_{s,}\max \qquad (12)$$

with :

α_d : a coefficient introduced to transpose the values, measured on driven piles into bored piles, it is assumed α_d = 0,8 (De Beer 1964)

η : dispersion of the lateral skin friction (from the results of the pile tests at Kontich one obtained η = 1,3)

α_{ss} : coefficient, determined by the execution method by extrapolating the results of the pile tests at Kontich, one puts α_{ss} = 0,67

$f_{s,}$max : the maximal value of the unit lateral skin friction, measured for the mechanical cone penetration tests (type M4)

As from the results of a mechanical static cone penetration test, performed on the left bank we deduced as an average of the skin friction of BK1 and BK2.

$$f_s = f_s\max = 0{,}15 \text{ MPa}$$

This value is 20 % smaller than the local skin friction measured with the electrical cone on the river ; 0,16 MPa for BK2 and 0,22 MPa for BK1. One obtains for the ultimate lateral skin friction

$$f_{s,r} = (0{,}8 \times 1 \times 0{,}67 \times 0{,}15)/1{,}3 = 0{,}062\text{MPa}$$

The admissible unit skin friction $f_{s,a}$ is then given by introducing a safety factor of 1,4. One obtains :

$$f_{s,a} = f_{s,r}/1{,}4 = 0{,}062/1{,}4 = 0{,}044\text{MPa}$$

3) from the effective earth pressure on the pile shaft :

$$f_{s,r} = p'_o \cdot \text{tg}\phi'_{res} \qquad (13)$$

with :

a) p'_o = the effective horizontal earth pressure on the pile shaft.

It has been established elsewere (Schittekat, Henriet, Vandenberghe 1983)

$$\sigma'_h = 344 + 5{,}64 \, Z \; (\text{kN/m}^2) \qquad (14)$$

with Z = depth

This simple equation is only valid for the Boom clay under the riverbank. In the Scheldt the effective vertical stress (σ'_v) is :

$$\sigma'_v = 1{,}6 + 9{,}3 \, Z \; (\text{kN/m}^2) \qquad (15)$$

Equation (15) has been derived assuming the submerged unit weight of the Boom clay is 9,3 kN/m^3 and that of the overlying sands is 9,5 kN/m^3.

The lateral stress can not be larger as the upper limit stress, which presumably must be the horizontal passive pressure.

$$\sigma'h = K_p \cdot \sigma'_v + (K_p - 1)\ c'\ \text{cotg}(\phi') \qquad (16)$$

K_p is the coefficient of passive earth pressure.
Equation (16) is representative of the horizontal stress up to the depth Zcr so that (16) = (13) at this depth.

With c'=22 kN/m^2 and ϕ'=25° for BK1, one has : Zcr = 15,7 m (depth below the bottom of the Scheldt)

From 10 to 15,7 m depth :

$$\sigma'h = 72{,}8 + 22{,}9\ Z\ (kN/m^2) \qquad (17)$$

and from 15,7 m depth : equation (14) is valid. The average horizontal stress on the pile shaft is 471 kPa.

b) ϕ'_{res} = the residual value of the effective friction angle in order to take into account the disturbance of the clay due to the drilling.
ϕ'_{res} = 11° (De Beer 1971)

So the following value of the ultimate shaft friction was obtained :

$f_{s,r}$ = 0,47.tg 11° = 0,091 MPa.

The admissible unit skin friction $f_{s,a}$ is then obtained by introducing a safety factor s = 2. One has :
$f_{s,a}$ = fs,r/2 = 0,045 MPa

The calculated values of the unit lateral skin friction are given in Table 6.

The average value of the admissible shaft friction is 0,043 MPa. Bustamente and Gianeselli (1981) have shown on test pile in stiff clay that the maximum ultimate shaft resistance is 0,08 MPa even though special technology would be used in order to avoid clay disturbance. Hence a value $f_{s,a}$=0,04 MPa has been adopted. Prototype pile tests would enable to state accurate values of ultimate shaft friction.

TABLE 6

SHAFT RESISTANCE

	$f_{s,r}$ in MPa	s (safety factor)	$f_{s,a}$ in MPa
from "$c_{u,peak}$"	0,054	1,4	0,039
from "skin friction"	0,062	1,4	0,044
from lateral stress	0,091	2	0,045

Admissible pile load : starting from the calculated values of $q_{b,a}$ and $f_{s,a}$, the admissible pile load Q_a has been calculated for the different piers, the piles are designed on the most heavy loaded piles by excentrical load as f.i. by closing gates or lateral wind. The whole pile length within the clay has been considered and piles are mainly working on friction.

One obtains for a central pier :
Q_a = 7,29 MN (without buoyancy effect).

OVERALL STABILITY OF THE FOUNDATION

Centric load

The whole of the foundation slab, piles and soil between the piles has been considered as a direct foundation installed at the level - 49 (fig. 4), and the stability of this foundation has been controlled. In this case the admissible vertical load V_a of the pier is given by the expression :

$$V_a = Q_{b,a} + F_a - G_g \quad (18)$$

with : $Q_{b,a}$ = admissible bearing capacity of the basement at -49m.

F_a = admissible friction on the foundation sides

G_g = weight of the soil between the piles.

The value of $Q_{b,a}$ and F_a has been determined for undrained conditions which are more critical than drained conditions. The ultimate bearing capacity $Q_{b,r}$ of a direct foundation of 22 x 100 m, installed at the level -49 m, is then given by the expression :

$$Q_{b,r} = S \times (Pb + S_c \cdot V_c \cdot c_u) \quad (19)$$

with : S = the surface of the foundation (=2200 m^2)

p_b = the vertical pressure at the foundation level

s_c = shape factor (assumed is s_c=1)

V_c = bearing capacity factor (=5,14 for $\emptyset$ = 0)

c_u = undrained shear strength.

In order to obtain a safe value of $Q_{b,r}$, the mean value of the $c_{u,res} - \Delta$ values for the BK2 and BK3 layers has been introduced ($c_{u,res} - \Delta$ = (0,10+0,15)/2=0,125 MPa

This conservative assumption for the undrained shear strength has been compensated by a reduction of the applied factor of safety. A safety factor s = 1,5 has been introduced.
Cosequently one obtains :

$$Q_{b,a} = S.(p_b + s_c.V_c.c_u)/1,5 = 1778\text{MPa} \quad (20)$$

The ultimate value of the unit friction on the foundation sides is given by the expression

$$fr = \alpha . c_u \quad (21)$$

with : α = reduction factor (following Vesic (1967): α = 0,36)

$C_u = C_{u,res}$ = 0,20 MPa

One obtains : f_r=0,36x0,20= 0,072 MPa.

The admissible friction on the foundation sides is then obtained by introducing a safety factor s = 2, or
fa = fr/2 = 0,072/2 = 0,036 MPa

For the calculation of the total admissible friction on the foundation sides only three sides, namely the two short sides and one long side are considered, and only the layer between level -49m and the top of the clay (-20m). So one has :

F_a = (22 + 22 + 100)(50-20).f_a=156MN

G_g = 630 MN hence from (18), (20) and (22) V = 1778+156-630=1304 MN

Since the vertical load is 840 MN the overall stability is assured by centric load.

Excentric load

V = 777 MN (vertical load) and H=10MN (horizontal force by exceptional wind).

The turning moment around the length (1) axis is 852 MN.m and 3530 MN.m around the width (b) axis.

The excentricity is :
e_b=852/777 = 1,10 m in the width axis

e_l = 3530/777 = 4,56 m in the length axis

Reduced length and width of the foundation are introduced :

b' = b-2e_b = 19,80 m

a' = 1-2e_a = 90,87 m

A reduction factor (X_c) on the undrained shear strength is introduced
$X_c = 0,5+0,5.\sqrt{1-H/a.b.c_u}$

The reduced undrained shear strength is 0,98x0,125 MPa.

 By substitution of these values and reduced area in relation (18), (19) and (20):

V_a = 1071 MN.

Hence the overall stability is ensured at excentric load.

BENDING MOMENTS IN THE PILES DUE TO THE HORIZONTAL FORCES

The bending moments in the piles, due to the horizontal forces, have been calculated by the method proposed by Gambin (1979).

Following this method different moduli of subgrade reaction are introduced for the inner piles, the edge piles and the front piles. As the horizontal loads are mostly transmitted to the soil in the upper part of the clay a Menard pressuremodulus E_M=20MPa has been introduced for the calculation of the subgrade reaction moduli. Following this method horizontal reactions varying between 1287 and 647 kN and bending moments between 2,935 and 1,854 MNm had to be taken by the piles. The calculated horizontal displacement of the foundation slab was of 0,62 cm.

The bending moments of the piles have also been calculted following a method proposed by Prof. De Beer, and based principally on the calculation of the horizontal displacements under the shear forces transmitted to the clay by the foundation piles.

Following this method the horizontal reaction was assumed to be the same for all the piles (=712 kN), the maximum bending moment was of 2,85 MNm and the calculated horizontal displacement of 1,10 cm.
For the final design of the pile reinforcement, the results of both methods have been taken into account. A last control was performed following the finite elements method.
As the horizontal forces and bending moments to be taken by the piles are very important, a heavy reinforcement of the piles was necessary.

PILE GROUP EFFECT

Informations concerning the load distribution within a pile group were communicated by Joustra e.a. (1977) and recently by Hooper and Levy (1981) and Cooke e.a. (1981). Based on these data a special study has been performed to determine the load distribution within the pile group.

Therefore a special computer program has been elaborated following the PIGLET-method, developped by M. Randolph (1980) at Cambridge University. With this computerprogram the load distribution within the pile group has been calculated for the following assumptions :

$G = G_o$ = 37 MPa
ν = 0,4
(EI)piles = $4{,}1.10^2$ MNm^2
H = 116,7 MN (horizontal force on the pier)
V = 740,6 MN (vertical load of the pier).

The load distribution over the piles obtained for this load case, is given in fig. 7. From this figure we see that following this method the load on the corner piles is larger than the calculated ultimate bearing capacity, and the load on the side piles is larger than the admissible load.

Although the obtained results offer interesting information concerning the load distribution within the pile group, certain reticences exist as the calculations are only based on the elastic behavior of the soil ; consequently the load on some piles become twice the calculated minimum value of the ultimate bearing capacity of a single pile. The introduction of the elasto-plastic behavior of the clay was however not possible within the scope of this study.

For horizontal loading the pile group effect has also been analysed, following the method proposed by Randolph (1981). A synopsis of the results are given in fig. 7.

The assumption of an elastic behavior of the clay is not applicable at all for the last edge piles and overestimates the turning moments in the pile heads.

For the inner piles the results are comparable with the values obtained according to the method proposed by Gambin. All results for lateral loaded pilegroup are compared in table 7.

SHORT TERM SETTLEMENTS

The short term settlement s_i, due to an undrained deformation of the clay, is given by the formula :

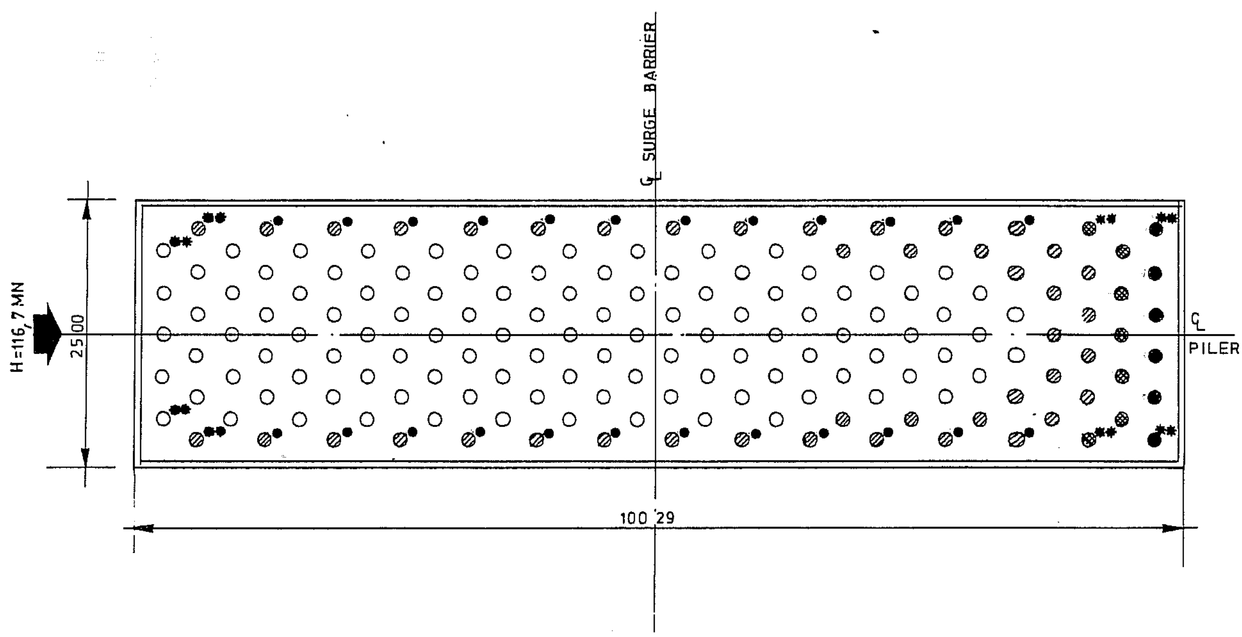
$$s_i = f.(1-\nu^2).q.B/E$$

with : f = shape factor
ν = coefficient of poisson
E = Modulus of elasticity
B = width of the foundation.

TABLE 7

MAXIMUM TURNINGMOMENT IN PILEHEAD, HORIZONTAL DISPLACEMENT

	"GAMBIN"	"DE BEER"	"RANDOLPH"
last row piles	2,94 MN.m	2,85 MN.m	4,35 MN.m
penultimate row piles	2,27 MN.m	2,85 MN.m	2,46 MN.m
edge piles	2,14 MN.m	2,85 MN.m	1 to 1.8 MN.m
inner piles	1,85 MN.m	2,85 MN.m	0,6 to 1.65 MN.m
horizontal displacement	0,6 cm	1,1 cm	0,6 cm



HORIZONTAL LOADING
H = 116,7 MN

VERTICAL LOADING
V = 740,6 MN

M	TURNING MOMENT	Q	VERTICAL LOAD OF INDIVIDUAL PILE
○	$M < 1$ MNm	✱✱	$Q > 11{,}9$ MN $= Q_r$
⊘	1 MNm $< M < 2$ MNm	●	$11{,}9$ MN $> Q > 8{,}5 \left(= \frac{Q_r}{1{,}4}\right)$
⊗	2 MNm $< M < 3$ MNm		
●	4 MN $> M$		

Fig 7 : Pile group effects

For the calculation of the short term settlements the foundation slab, piles and soil between the piles have been considered as an undeformable body, installed at level -49 m. An average value of 56.6 MPa and 0.5 has been respectively taken for the modulus of elasticity and a Poisson ratio for undrained conditions as determined before and $\nu = 0.5$ for undrained conditions.

The values of the shape factor have been taken from Caquot and Kerisel (1966).

On the value of the short term settlement calculated in this way a reduction factor has ben applied in order to take into account the anisotropy of the clay. From a camparison between the characteristics of the London clay and the Boom clay a value of 0,46 has been deduced for this reduction factor.

HYDRODYNAMIC SETTLEMENTS

For the calculation of the total or long term settlements and of the hydrodynamic settlements, different methods have been used :

- the method proposed by De Beer (1971)
- the method suggested by Skempton and Bjerrum (1967)
- the method of Menard (1962)
- the method of Terzaghi and Peck (1967)
- the finite elements
- the pile group analysis according to Randolph (1979).

For the calculation of the total settlement, different assumptions have been introduced for the load transfer between the foundation slab at level -18 m and the base of the piles at -49 m.

* in a first approach the load of the pier is considered to be applied completely at the base of the piles (no lateral friction)

* for a second calculation a lateral friction of 40 kN/m^2 has been considered over the perimeter of the foundation and a spreading of the friction load over a width equal to half of the height was introduced ;

* in a third approach a lateral friction of 80 kN/m^2 was introduced and again a spreading of this friction load over a width equal to half of the height ;

* for the settlement calculation following the method of Terzaghi the complete load is considered to be applied at a depth beneath the foundation slab equal to two thirds of the pile length, and a lateral friction on the circumference of 80 kN/m^2 was considered.

The calculation of the total settlement, based on the results of the oedometer tests was performed for C = 75.
Just in case other calculations were made with values of C increasing with depth, as it has been determined for the E3-tunnel (De Beer 1971), there was no significant difference.

For the calculation of the settlement following the method of Menard a pressuremetermodulus E_M = 60 MPa has been introduced.

Plane strain finite element modelling was used. The pile group is replaced by of block of pile-reinforced material of the same overall dimensions of the actual group.
The clay was assumed to behave orthotropically as previously discussed. The elastic properties of the pile-reinforced block were assumed isotropic and estimated using relationship

$$E = (E_{soil})^{1-n} (E_{concrete})^{n}$$

with n = ratio volume concrete / total volume=0,1 and E = 180 MPa.

Calculation for drained and undrained behavior have been performed : the obtained results are summarized in Table 8 and 9.

Finally it has been assumed that a lateral friction exists on the perimeter of the pile group, and an average value of 8 cm and 5 cm has been considered for respectively the total and hydrodynamic settlements.

ESTIMATION OF THE DIFFERENTIAL SETTLEMENTS

In order to obtain an idea concerning the possible tilting of the piers, an estimation of the differential settlements has been made.

Therefore both the classical method and the finite elements have been used.

* Classical method : the variation of the compressibility of the clay has been assumed to be the same as the variation of the cone resistance. So the ratio between the results of the most favourable and the most unfavourable cone penetration tests, performed at the spot or near the different main piers has been determined. The following value has been obtained for the central pier

$$q_{c,max}/q_{c,min}=3,4/2,3=1,48$$

TABLE 8

TOTAL SETTLEMENTS

Method	No lateral friction	Lateral friction 40 kN/m^2	Lateral friction 80 kN/m^2
De Beer Skempton-Bjerum Menard (after 100 years) Terzaghi	18,7 cm 15,3 cm 4,8 cm	13 cm 11,9 cm 3,8 cm	6,4 cm 7,1 cm 2,6 cm 9 cm
Finite Elements Piglet		6,6 cm 3,3 cm	

TABLE 9

HYDRODYNAMIC SETTLEMENTS

Method	No lateral friction	Lateral friction 40 kN/m^2	Lateral friction 80 kN/m^2
De Beer Skempton-Bjerum Menard (after 100 years) Terzaghi	11,7 cm 8,3 cm -	7,5 cm 6,4 cm -	2,4 cm 3,1 cm - 5 cm
Finite Elements		1,1 cm	

Further it was assumed that the scatter determined from the results of the static cone penetration tests existed also for the compressibility of the clay at the spot of the singular points (cfr. De Beer, 1954).

Hence, starting from the average hydrodynamic settlement of 4 cm, one has 5 cm in one singular point of the foundation and 3 cm at the other corresponding to an inclination of 1,5 10^{-3} which is since more conservative assumption were made an overestimate value of the inclination due to differential settlements.

* Finite elements : the heterogeneity of the clay has been taken into account by introducing for all soil elements at one side of the foundation with soil characteristics that are 25 % lower than the calculated mean values, and for all soil elements at the other side, soil characteristics that are 25 % higher than the calculated mean values. For the elements layed in between both sides of the foundations, the soil elements have properties passing gradually from the lower to the higher values.

The calculations have been performed for drained and undrained conditions, and the transversal moment on the foundation (M=3112 MN.m) due to the wind forces, has been introduced.

From the calculations, the following results were obtained :

- for the undrained conditions :
  differential settlement = 0,61 cm
  or 2,8 10^{-4} inclination

- for the drained conditions = 0,44 cm
  or 2,0 10^{-4} inclination.

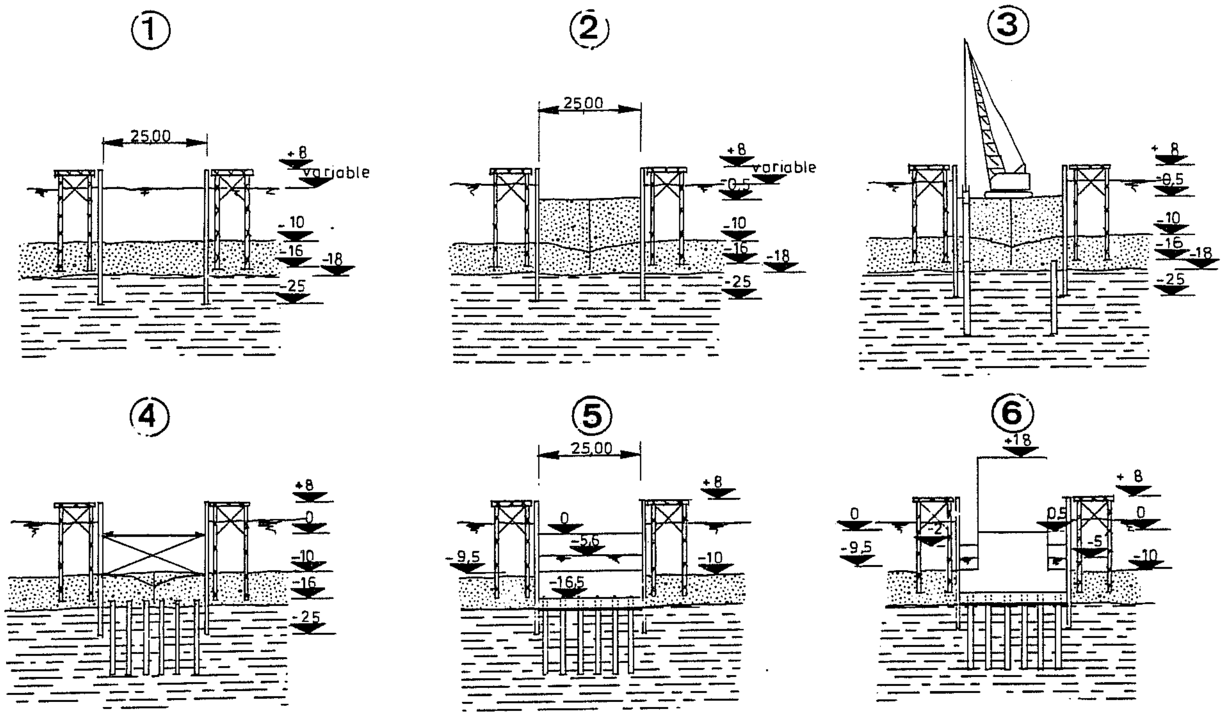


Fig 8 : Execution sequences

1) - Cofferdam driving
   - Construction of wharf

2) - Filling up with sand
   - Water lowering

3) - Bored piles execution

4) - Shoring up
   - Soil excavation

5) - Further soil excavation to level -18,5 m and shoring up
   - Water rising to level -5,6 m
   - Laying of water permeable geotextil on level -18,5 m
   - Concreting between levels -18,0 m and -16,5

6) - Pier construction

EXECUTION SEQUENCES

A synopsis of the different execution sequences of the foundation is given on fig. 8.

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REFERENCES

ABSI, E. 1965 - Recherches sur la rhéologie des argiles, Annales I.T.B.T.P., série "Essais et mesures", n° 213 septembre.

AMAR, S., JEZEQUEL, J.F. 1972 - Essais en place et en laboratoire sur sols cohérents : Comparaison des résultats, Bulletin de Liaison des Laboratoires des Ponts et Chaussées, n° 58, mars/avril pp. 97-108.

BRINCH HANSENS, J., MISE TADASHI 1964 - An empirical evaluation of consolidation Tests with Lille Belt clay, The Danish Geotechnical Institute, Bulletin n° 16, Copenhagen.

BUSTAMENTE, M., GIANESELLI, L. 1981 - Prévision de la capacité portante des pieux isolés sous charge verticale. Bulletin de liaison des Laboratoires des Ponts et Chaussées n° 113, mai-juin pp. 83-108.

CAMBEFORT, H. 1971 - Geotechnique de l'ingénieur et reconnaissance des sols, Ed. Eyrolles p. 127.

CAQUOT A., KERISEL J. 1966 - Traité de mécanique des sols - Gauthier-Villars.

DE BEER, E.,E. 1954 - Grondmechanica deel II NV Standaard Boekhandel, Brussel.

DE BEER, E.,E. 1964 - Some considerations concering the point bearing capacity of bored piles, Proc. of the Symposium on bearing capacity of piles. Central Building Research Institute, Roorkee, India, February, vol. I, pp. 178-204.

DE BEER, E., E., BUTTIENS, E. 1966 - Construction de réservoirs pour hydrocarbures liquéfiés dans l'argile de Boom à Anvers. Etude des mouvements du sol provoqués par cette réalisation, Revue Travaux n° 379 et 380, sept.-oct. 15p.

DE BEER, E. 1971 - Problèmes posés par la Construction du tunnel sous l'Escaut à Anvers Annales de l'Institut Technique du Bâtiment et des Travaux Publics - Série Sols et Fondations n° 83 - octobre pp. 22-50.

DE BEER, E.,E., LOUSBERG, E., WALLAYS, M., CARPENTIER, R., DE JAEGER, J., PAQUAY, J., 1977 - Bearing capacity of displacement piles in stiff fissured clays, I.W.O.N.L. Verslagen over navorsingen nr. 39 maart 135 p.

DE BEER, E., E. 1977 - Sonderen en funderingen op palen in klei Fugro, sondeersympo- sium pp. 135-154.

COOKE, R.W., BRYDEN-SMITH, D.W., GOOCH, M.V., SILLET, D.F. 1981 - Some observations of the foundation loading and settlement of a multi-storey building on a piled raft foundation in London clay, Proc. Instn. Civil Pub. Works, Part 1, 1981,70, Aug. pp. 433-460.

GAMBIN, M. 1979 - Calculation of foundations subjected to horizontal forces using pressuremeter data, Sols Soils n° 30/31 - pp. 17-59.

GIBSON, R.E., ANDERSON, W., F. 1961 - In situ measurement of soil properties with the pressuremeter, Civ. Eng. & Publ. Works Rev. Vol. 56, n° 658, May pp. 615-618.

GIBSON, R. E., 1974 - The analytical method in soil mechanics, Geotechnique, Vol. 24, n°2 pp. 115-118.

JOUSTRA, K., DE SITTER, W., DEN OUDEN, N. 1977 - Tall building settlement and pile load measurements - Proceeding of the IXth International Conference of SMFE, Tokjo pp. 577 - 580.

HENRIET, J.P., SCHITTEKAT, J., HELDENS, P. 1983 - Borehole seismic profiling and tube wave application in a dam site investigation, Geophysical Prospecting 31, 15p.

HOOPER, J.A., LEVY, J., F. 1981 - Behavior of a flexible piled foundation, proceedings of the X th International Conference on SMFE, Stockholm pp. 735-740.

KERISEL, J. 1967 - Scaling laws in soil mechanics, Proc. 3rd Panamerican Conf. Soil Mech. and Found Eng., Caracas, Vol. III pp.69-92.

MARSLAND, A. 1973 - Large in-situ tests to measure the properties of stiff fissured clays, Building Research Establishment, Current Paper CP1/73.

MENARD L., ROUSSEAU J. 1962 - Evaluation des tassements - Tendances nouvelles - Sol-Soils n° 1, 26 p.

RANDOLPH, M.F. 1980 - Piglet a computer program for the analysis and design of pile groups under general loading conditions, Cambridge University Research Report, CUED - Soils TR91.

RANDOLPH, M.F. 1981 - The response of flexible piles to lateral loading, Geotechnique, Vol. 31 pp. 247-259.

SCHITTEKAT, J., HENRIET, J.P., VANDENBERGHE, N. 1983 - Geology and geotechnique of the Scheldt surge barrier, characteristics of an overconsolidated clay, Proceeding 8th International Harbour Congress, Antwerp, 15p.

SEED, H.B., IDRISS, I.M. 1970 - Soil moduli and damping factors for dynamic response analysis Report n° EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley.

SKEMPTON, A.W., BJERRUM, L. 1967 - A contribution to the settlement analyses of foundations on clays, Geotechnique - December

TERZAGHI K., PECK R.B. - 1967 Soil mechnics in engineering practice, Ed. Wiley International pp. 548-550.

VESIC, A. 1967 - Ultimate of bearing capacity of deep foundations, Final report, School of Civil Engineering, Georgia Institute of Technology, Atlanta, Georgia pp. 1-254.