GEOLOGY AND GEOTECHNIQUE OF THE SCHELDT SURGE BARRIER, CHARACTERISTICS OF AN OVERCONSOLIDATED CLAY

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ABSTRACT

Flood protection of the upstream part of the Scheldt basin, including the city of Antwerp, requests the construction of the Scheldt Surge Barrier. The geological sequence on the dam site consists of three main units   Quaternary sandy deposits, Late Tertiary glauconitic sands and the Rupelian clay, known   as the Boom clay, which is a stiff, fissured Tertiary clay.

Geotechnical properties of these units have been investigated by extensive laboratory and field tests including seismic surveying, cone penetration testing, self boring pressuremeter testing, MENARD pressuremeter testing and sampling in borings.   The main part of this testing programme has been confined to the Rupelian clay.

Some main aspects of the stratigraphic subdivision of the Rupelian clay on the dam site are discussed and related to observations on other sites.   A few particular features of this clay deserve some special attention with regard to civil engineering works :

- large loaf-shaped carbonate concretions or septaria, which have been investigated by reflection seismics, boring and cone penetration testing,
- clay diapirs, of which some geotechnical properties have been investigated for the first time,
- clay fissures, which deserve some attention regarding their origin.

For practical purposes, the Rupelian clay has been subdivided into some main units, defined on base of lithological and geotechnical properties.
A tentative correlation with some other sites is proposed.

Preconsolidation pressure and past burial depth of the Rupelian clay have been investigated both by laboratory testing and in situ testing, with seismic techniques and the self boring pressuremeter.   The obtained figures are evaluated for their geological plausibility.

INTRODUCTION

Throughout time, the Scheldt basin has suffered from widespread, severe floods and the frequency of flooding has increased over the past century.

Consequently, the Department Waterways of the Ministry of Public Works has elaborated the so-called Sigmaplan. This over-all plan aims to protect the low-lying regions of the Scheldt basin against floods, generated by rising North Sea waters, often due to the enhancement of spring tides by persistent westerly or north-westerly winds. A key element in this plan is the storm surge barrier at Oosterweel, Antwerp (cfr. map, fig. 1).

Preliminary investigations of this project have been committed to a joint venture, called "STORMVLOEDKERING   SCHELDEBEKKEN"   or "SVKS".

At the design level of such a dam, which has to account for severe dynamic loading from storm waves, earthquakes and accidental shocks, major attention has to be paid to extensive and deep foundations.

The geological detail structure and geotechnical properties of the dam site have been investigated by extensive field tests : a high-resolution seismic survey, 85 cone penetration tests, 3 self boring pressuremeter tests, 27 borings for the collection of undisturbed samples for laboratory tests, MENARD pressuremeter
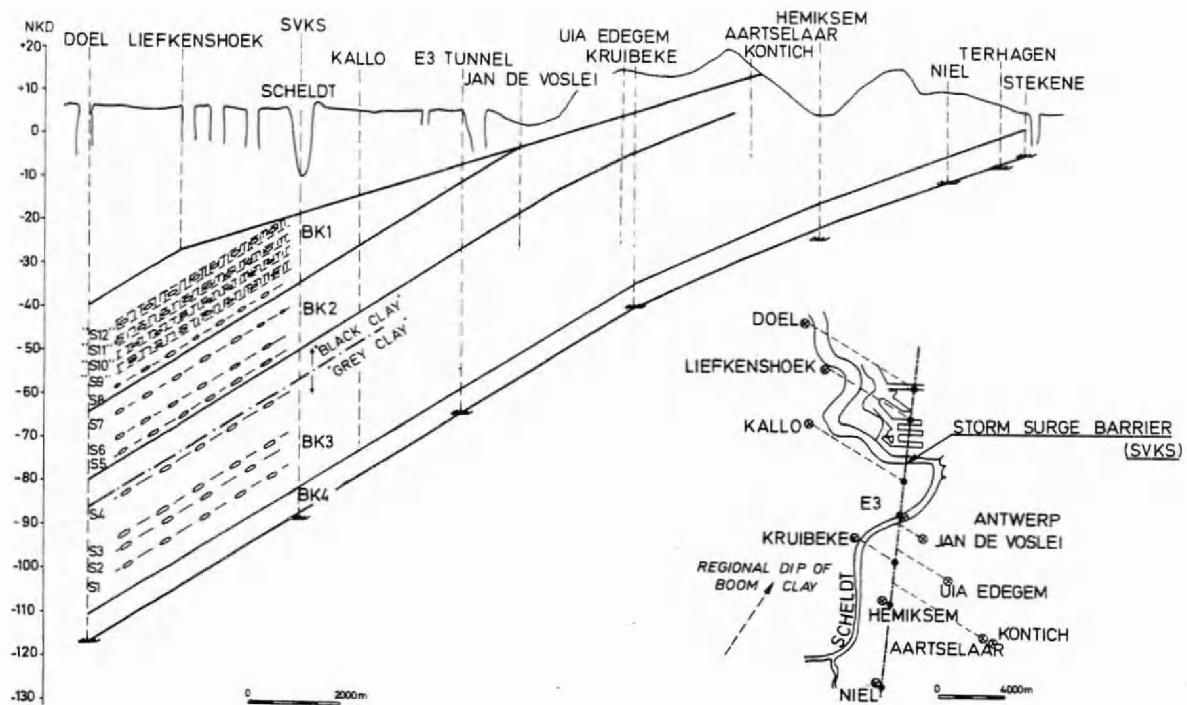
Fig. 1 — General north-south geological section and localisation map.

tests. Since most of these tests have been performed in the Boom clay, their interpretation might contribute to a better knowledge of this formation.

GEOLOGY

The subsoil of the dam site is built up of a series of Rupelian clays, about 70 m thick, covered by Neogene sands, 3 to 7 m thick under the river channel and about 8 m thick under the banks (fig. 2). These sands are known as "Kattendijk" sands (DE MEUTER & LAGA 1976), belonging to the Lower Pliocene. These sediments are covered by Quaternary formations with a thickness varying between 1 and 6 m in the river channel and ranging up to 20 m on the banks (fig. 2).

The Rupelian clay, or Boom clay is a marine deposit of Middle Oligocene age (35 m.y.). The total thickness of the original clay formation could have been well above 100 m as suggested by the 150 to 200 m thick Boom clay in the South Netherlands, just north of the Belgian boundary (PANNEKOEK 1954).
Later erosion has removed part of the Boom clay to leave typical thickness of 69 to 72 m in Oosterweel. Furthermore, there is evidence that in Neogene times, it was covered by thicker deposits than those left today. Consequently, the Boom clay may be considered as an overconsolidated clay.

The Rupelian strata have a gentle dip of about 4°/∘∘ towards north-east.
Although the clay thickens to the north (e.g. 76 m at Doel, 127 m at Meer), it is probable that it has been partly eroded in the whole area south of the West Netherlands Basin and the Netherlands Central Graben (KEIZER & LETSCH 1963).

In former studies, VANDENBERGHE (1978) has correlated the clay sequences cropping out in the Waasland, Boom and southern Kempen and established a stratigraphic subdivision of the lower part of the Boom clay (lower 55 m). The denomination of the lower septaria levels (levels "1" tot "8") and the subdivision into "grey clay" and "black clay" (fig. 1) are borrowed from this work.

SPECIAL STRUCTURAL FEATURES

Some particular structural features of the clay could have an important influence on the foundation design.

* Small sized fissures occur in the heavy clay, generally displaying polished surfaces and striations or slickensides. In the Boom area, such smooth and grooved shear surfaces have also been observed in septaria. This might suggest that the fissures originated at a compaction stage closely approaching the lower plasticity condition, the septaria however still being in a deformable state.
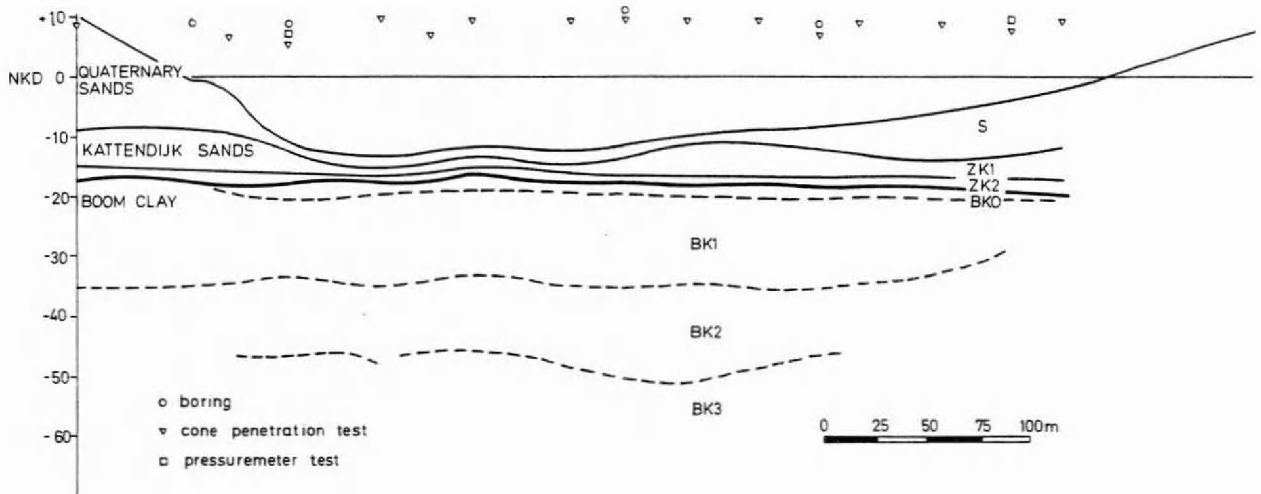
Fig. 2 - Geological cross section over Scheldt river at the dam site and geomechanical subdivision.

* Septaria (fig. 3) are loaf shaped carbonate concretions, showing internally nearly vertical open cracks, closing towards the rim and sometimes water bearing. They occur in distinct horizons, characterized by a relatively higher carbonate content. These septaria levels form stratigraphical key horizons which can be used for correlation purposes (VANDENBERGHE 1978). Individual concretions may have a diameter up to 1 m and a thickness ranging between 10 and 30 cm.

Fig. 3 - Septaria fragment, height 20 cm.

On the dam site, septaria have been sampled in many borings and probably account for most peak values in the cone resistance graphs, measured in the clay.

A frequency analysis of septaria indications over more than 40 cone penetration tests and 20 borings is presented in table 1.

On fig. 4, the bulk of septaria indications has been grouped into depth belts, each of which representing a major concentration of observations. The depth range (height) of the belts could either be due to a scatter in depth observations of a singular level or to a lack of resolution of proximate levels, the latter hypothesis being corroborated by some further evidence. Septaria level 8 for instance turned out to be composed of two distinct levels. The lower one is sideritic and may be correlated with septaria level 8 of the Boom clay stratigraphy by VANDENBERGHE (1978). Septaria belts 7 and 6 correspond to levels 7 and 6, recognized by the same author.
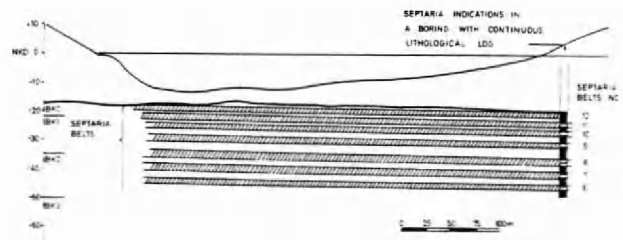

Fig. 4 - Septaria belts

* Diapir structures in the top zone of the Boom clay have already been reported by several authors. LAGA (1966) described a diapir structure observed during excavations on the right bank of the Scheldt in Antwerp.

TABLE 1

SEPTARIA FREQUENCY

Septaria belt	Frequency C.P.T.	Frequency borings	Total frequency (%)
12	8/41	12/24	31
11	13/41	5/24	28
10	5/40	4/24	14
9	9/30	5/19	19
8	8/27	4/15	29
7	11/27	1/15	29
6	11/26	4/15	37

Subbottom profiling by WARTEL (1980) confirmed the occurence of diapirs under the Scheldt. Seismic reflection profiling with a high resolution boomer source showed how such diapir structures fade out with depth, undeformed reflections generally being found between 25 and 35 below chart datum (HENRIET, SCHITTEKAT and HELDENS 1983). A typical diapir structure is shown on fig. 5. This picture has been recorded upstream Antwerp during a reconnaissance seismic reflection survey on behalf of the Rijksinstituut voor Grondmechanica and the Bijzondere Studiedienst Pre-Metro (1982).

As an additional observation on this picture it may be observed how septaria horizons clearly stand out as alignments of diffraction hyperbola, each concretion acting as a singular point reflector.

Curiously enough, comparable updoming clay structures have never been observed in any of the numerous clay pits in the Waasland, Boom and Southern Kempen regions. It is hence thought that diapirism is related to the Scheldt river incision and the associated clay relaxation phenomenon.

This hypothesis might be corroborated by the observation of analogous diapirs of Ieper or London clay, piercing into Quaternary sands in the Thames valley or in paleovalleys in the North Sea (HENRIET e.a. 1982).
Diapirism might be geomechanically interpreted in terms of the TRESCA yield criterion ($\emptyset$ = 0 and Cu), the horizontal stress inherited from past burial conditions largely exceeding the vertical stress after erosion and hence inducing vertical clay flow.

As shown by HEDBERG (1974), over-pressure in clay might also be enhanced by methane gas generation, the development of a gas phase in the pore fluid impeding to some extent fluid expulsion and normal consolidation. In the neighbourhood of the dam site, some seismic sections displaying noise bursts flanked by uplifted or down bending reflection terminations could suggest the presence of deformations of diapiric nature. The axis of one of these structures has been tested by cone penetration, but the derived geomechanical properties turned out to be not significantly different from neighbouring undisturbed regions.
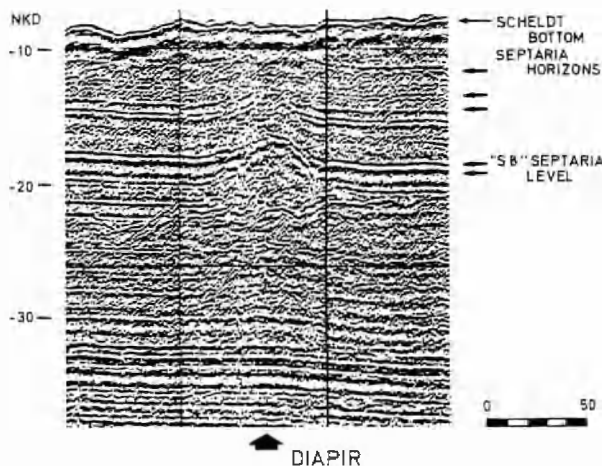
GEOMECHANICAL SUBDIVISION

From a geomechanical point of view, the Boom clay can be subdivided into the following units (fig. 2).

* A weathered top layer, with a thickness varying between a few decimeters and 4 m. In this top layer, the clay is more light coloured and it is characterized by some degree of degradation in geomechanical properties. It is called BKO and is found between 18 and 22 m below chart datum, which is the zero level of NKD ("Nieuw Krijgsdepot").

* A complex unit is situated between -22 m and -35 m. It is called BK1 and consists of a banded sequence of mainly silty and some clayey horizons. This complex unit is characterized by geomechanical properties which are definitely more favourable than those of the underlying unit.

Fig. 5 - Diapir structure under the Scheldt river. upstream Antwerp.

* This underlying unit, situated between -35m and -50 m, is called BK2 and is more clayey than BK1.

* Between -50 m and -80 m, one distinguishes a banded sequence of silty and clayey horizons, called BK3. This unit has not been thoroughly investigated on the dam site, but its properties are well known from outcrops or subcrops in the vicinity of Boom. It includes the "grey clay" and the lower part of the "black clay", described by VANDENBERGHE (fig. 1).

* Between -80 m and -90 m, the Boom clay progressively grades into the underlying sands through a transitional sequence of silty to clayey fine sands.

It is remarkable that the Boom clay sequence investigated in a boring performed on behalf of the Studiecentrum voor Kernergie in Mol (DETHY e.a. 1983) displays a gross zonation of some main physical properties (natural gamma, resistivity, unit weight of dry soil, grain size fraction smaller than 2 um) which might fit the geomechanical subdivision proposed at Oosterweel, as suggested by fig. 6.
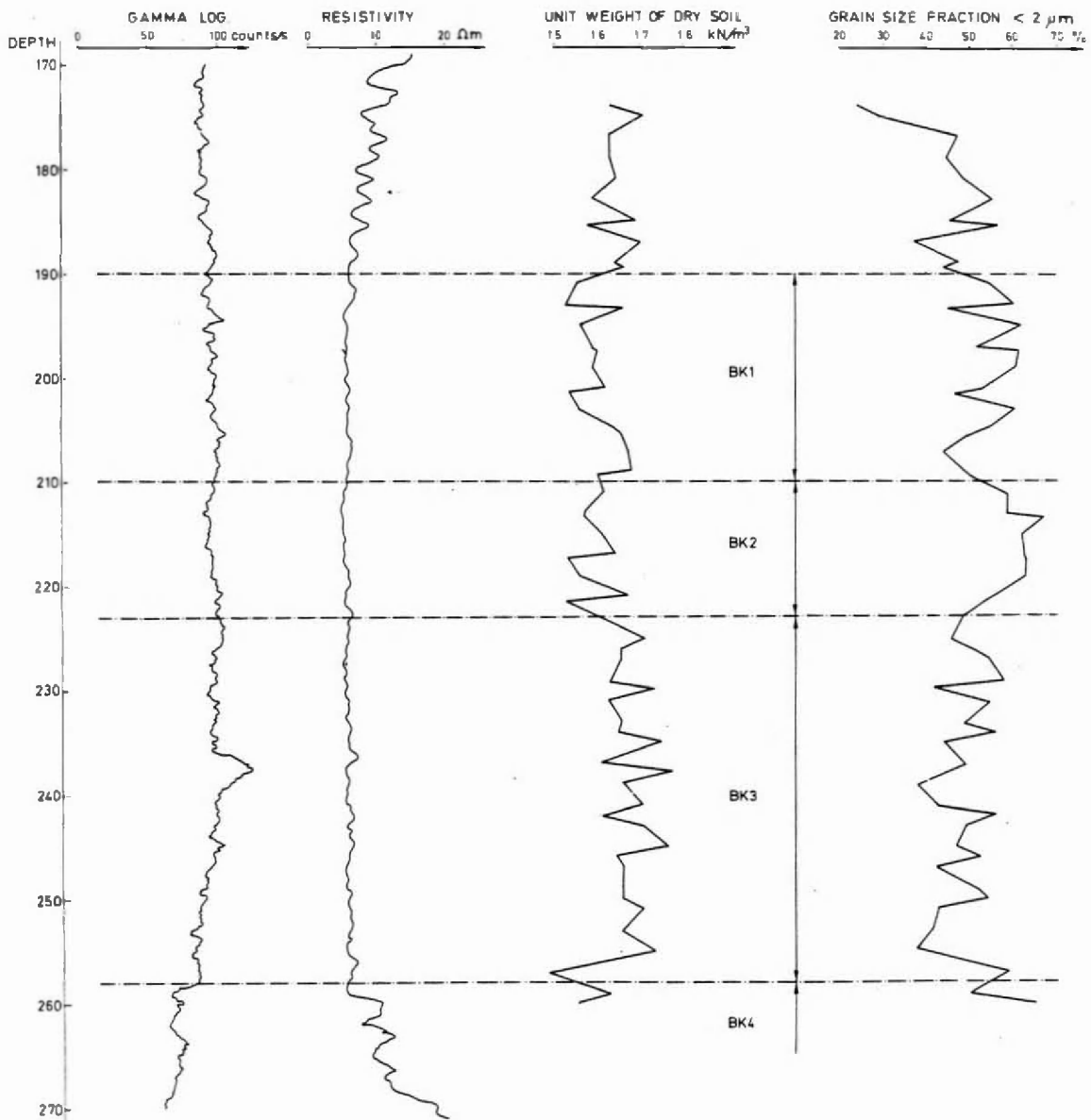

Fig. 6 - Possible correlation between the geomechanical subdivision at Oosterweel and the zonation of some main properties of the Boom clay in the S. C. K. boring. Mol.

Most units identified at Oosterweel have been subject of geotechnical investigations on various sites, shown on fig. 1.

Unit BK3 has been investigated near Aartselaar and Reet.

Unit BK2 has been intensively investigated for some major civil engineering works in the neighbourhood of Antwerp (E3 tunnel, Jan de Voslei, Edegem) (DE BEER 1967). It has also been described in the clay pits of Kruibeke ("black clay"). On the borehole logs in Mol, BK2 is characterized by a higher content of particles smaller than $2 \mu m$ and a somewhat lower unit weight of dry soil, confirming its more clayey character (e.g. compared with BK3), but it is curiously enough also characterized by a somewhat lower natural radioactivity.

Unit BK1 has a thickness of about 20 m in Mol. At Oosterweel, erosion has stripped some of the top meters, leaving a thickness of about 17 m (including BK0). In Doel, somewhat further north of Oosterweel (fig.1), its thickness amounts also to 20 m. Only a few meters of the base of BK1 are found on the site of the E3 tunnel, forming the very top of the Boom clay on that spot.

LABORATORY TESTING

Various laboratory tests have been performed by the Rijksinstituut voor Grondmechanica, the Laboratoire du Génie Civil de Louvain la Neuve and the Laboratoire des Matériaux de Construction de l'Université de Liège.

Identification tests

Current identification testing has been performed on more than hundred disturbed and undisturbed samples.
Resulting mean values of the physical proper-

ties of the different geomechanical units are summarized in table 2. The variation with depth of the grain size fractions smaller than $2 \mu m$ and $20 \mu m$, the plastic limit, the liquid limit and the plasticity index is shown on fig. 7.

U.U. triaxial tests

Unconsolidated undrained (U.U.) triaxial tests have been performed on a large number of undisturbed clay samples, most of which having a diameter of 10 cm and a height of 20 cm. The undrained shear strength (c_u) values, derived from these experiments, are plotted versus depth (z) on fig. 8.
Samples having undergone brittle rupture are indicated by a special symbol.
The resulting set of c_u values displays a large amount of scatter.
A linear regression on all data yields the following relationship :

$$c_u = 175 + 0,8 \ z \ (kPa) \qquad (1)$$

C.U. triaxial tests

Consolidated undrained (C.U.) triaxial tests have been performed on undisturbed samples with a diameter of 3.8 cm and a height of 10 cm. Resulting effective shear parameter values are presented in table 3. Fig. 9 shows the p, q-diagrams for the BK1 and BK2 units, illustrating again the important amount of scatter on the data.

FIELD TESTING

Static cone penetration tests (C.P.T.)

With exception of a few mechanical static penetration tests (type M4) performed on the left bank, all static cone penetration tests have been performed with an electrical cone. Only the latter measurements will deserve further attention.

TABLE 2

IDENTIFICATION TESTS

			BK0	BK1	BK2	BK3
% particles 20 μm		%	-	80	80	80
% particles 2 μm		%	-	50	57	54
Natural water content	W	%	-	26,93	29,40	29,50
Liquid limit	Wl	%	-	66	73	60
Plasticity index	Ip		-	40	44	35
Unit weight of dry soil	γ_d	kN/m³	-	15,31	14,6	14,74
Unit weight of soil	γ	kN/m³	19,31	19,42	19,17	19,31
Unit weight of solid particles	γ_s	kN/m³	-	26,54	26,52	26,52
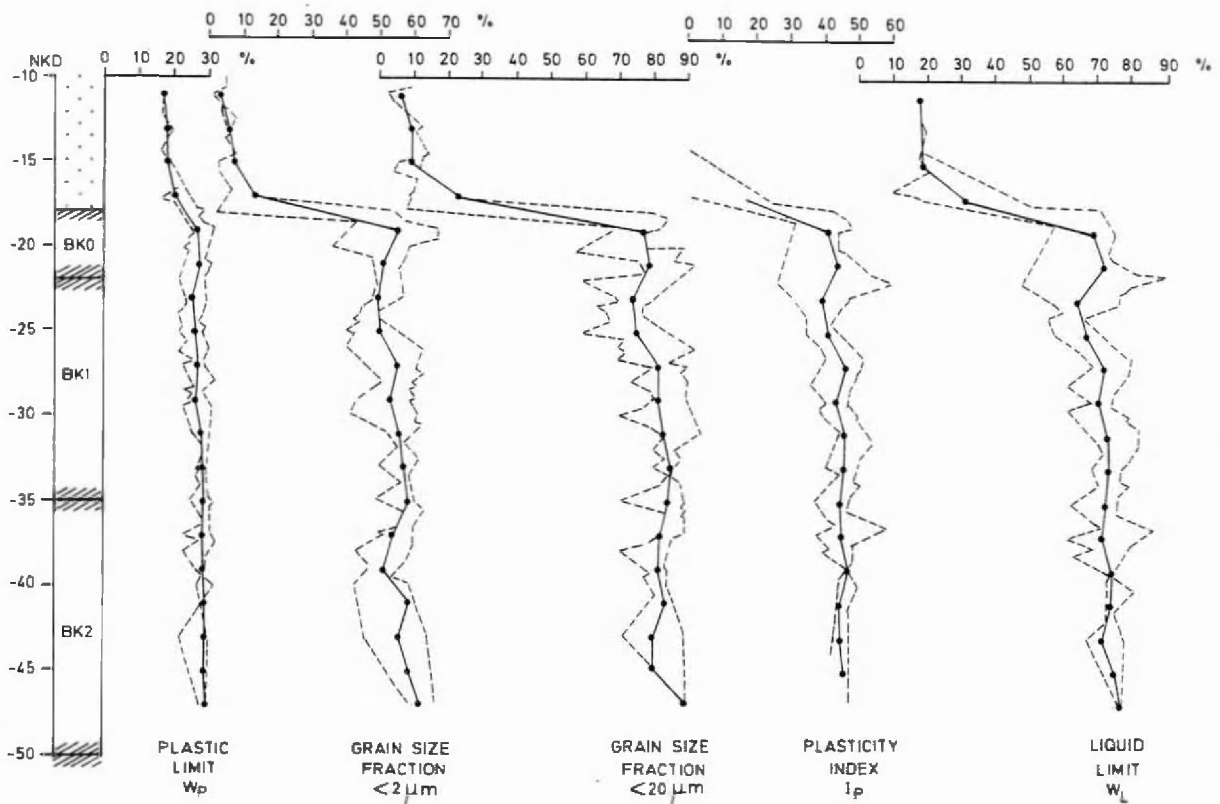
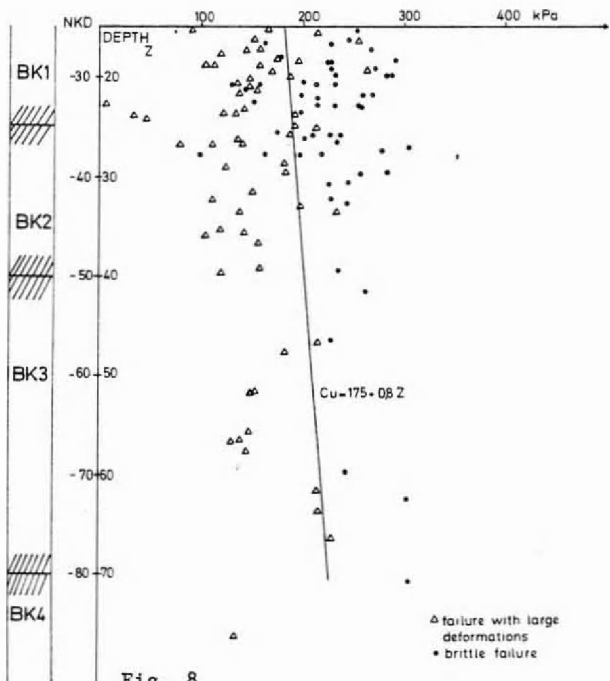Fig. 7 - Profiles of some main sediment properties at Oosterweel.

PLASTIC LIMIT W_P — GRAIN SIZE FRACTION $<2\,\mu m$ — GRAIN SIZE FRACTION $<20\,\mu m$ — PLASTICITY INDEX I_P — LIQUID LIMIT W_L

MEAN
ENVELOPE

Fig. 8

- Undrained shear strength versus depth - U.U. triaxial tests.

$Cu = 175 \cdot 08\,Z$

△ failure with large deformations
• brittle failure

Mean and minimum values of the cone resistance q_c and mean values of the local skin friction f_s are summarized in table 4. An example of a cone penetration graph is presented on fig. 10.

It should be remarked that, on the base of some former investigations, it was generally admitted that cone resistance values should steadily increase with depth in the Boom clay. At Oosterweel, none of more than 50 electrical and mechanical cone penetration tests displays an increase of cone resistance with depth, at least above the level of −50 m (i.e. within BK1 and BK2). Below −50 m, cone resistance do steadily increase with depth, a trend which is continued down to the bottom of the formation. It should be noted that a similar absence of increase of cone resistance with depth in the considered clay units had already been reported on some other sites too, such as at Kallo and Jan de Voslei (fig. 11).

MENARD pressuremeter tests

Borings for the MENARD pressuremeter tests have been performed by the direct flush method, a bentonite slurry being used as drilling mud. The pressuremeter tests have been carried out with a MENARD type B equipment, in accordance with standard procedures.
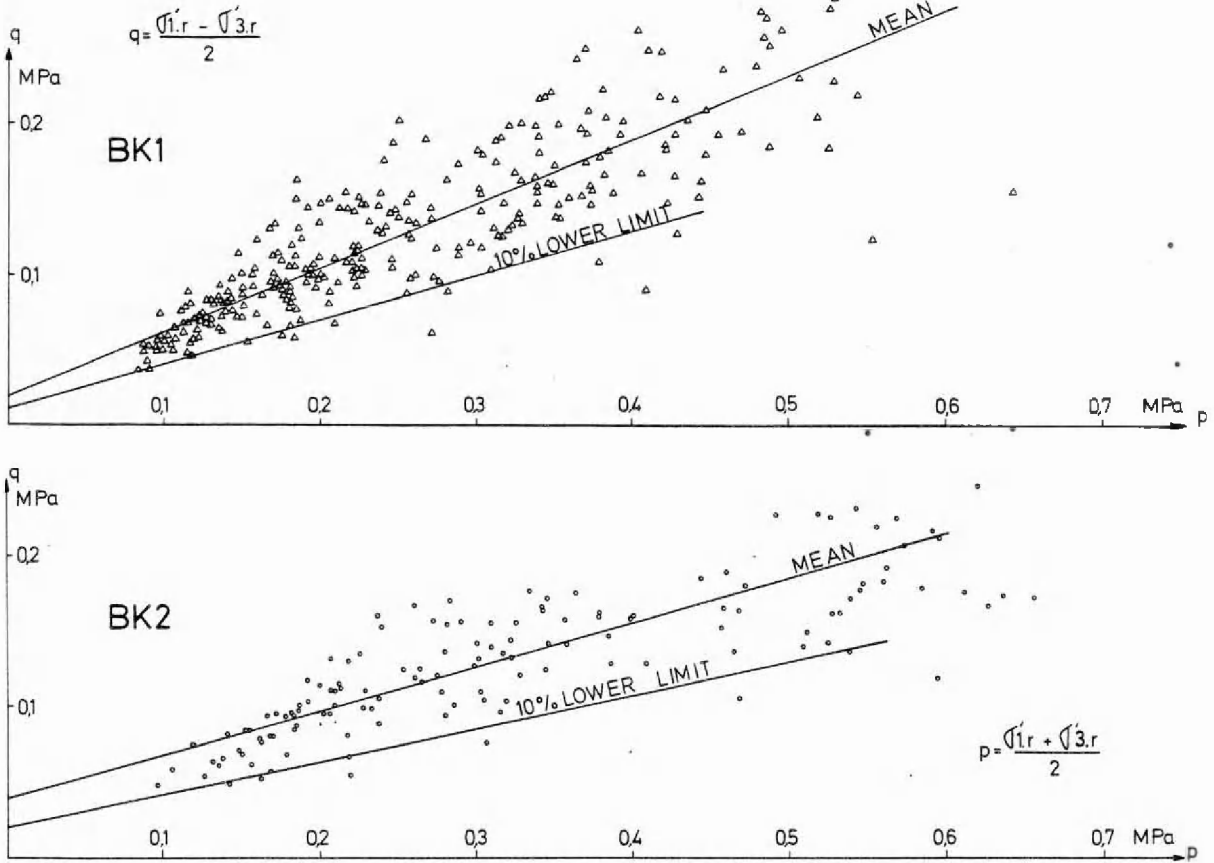
Fig. 9 — p,q,— diagrams for C.U. triaxial tests.
$\sigma_{1.r}$ $\sigma_{3.r}$ = principal stresses at rupture.

TABLE 3

EFFECTIVE SHEAR PARAMETERS

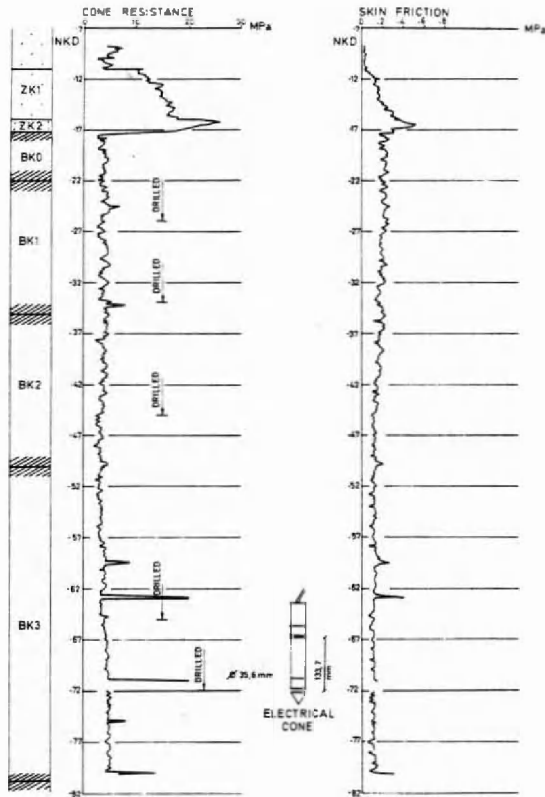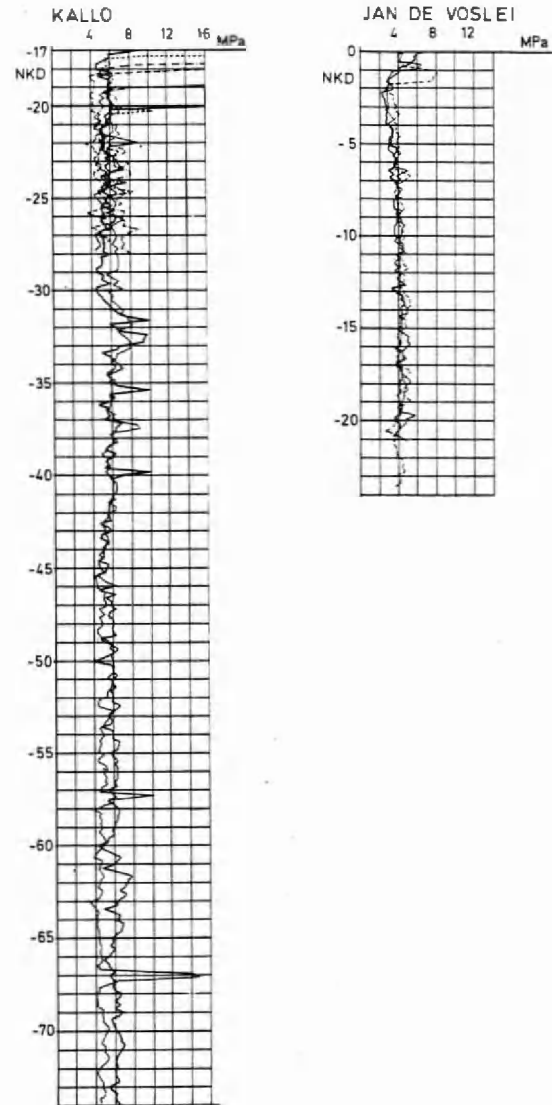|	Number of samples	c'(kN/m²)	∅'	c'min (kN/m²)	∅' min.
BK0		–	–	–	–
BK1	327	22	25,04°	13	17,00°
BK2	162	40	17,05°	21	12,92°
BK3	18	48	20,11°	43	15,32°

Fig. 10 - Typical static cone penetration test at Oosterweel.

Fig. 11 - Static cone penetration tests at Kallo and Jan de Voslei, Antwerp.

TABLE 4

C.P.T RESULTS

	$\overline{q}_{c,min}$ (MPa)	$\overline{q}_c$ (MPa)	$\overline{f}_s$ (MPa)
BK0	3,1	3,7	0,20
BK1	3,2	4,2	0,22
BK2	2,6	3,6	0,16
BK3	2,8 4.6	3,88 5,45	0,20

with

$\overline{q}_c$ the mean value of the cone resistance within the considered layer

$\overline{q}_{c,min}$: the mean value of the minimal cone resistances

$\overline{f}_s$. the mean value of the local skin friction within the considered layer
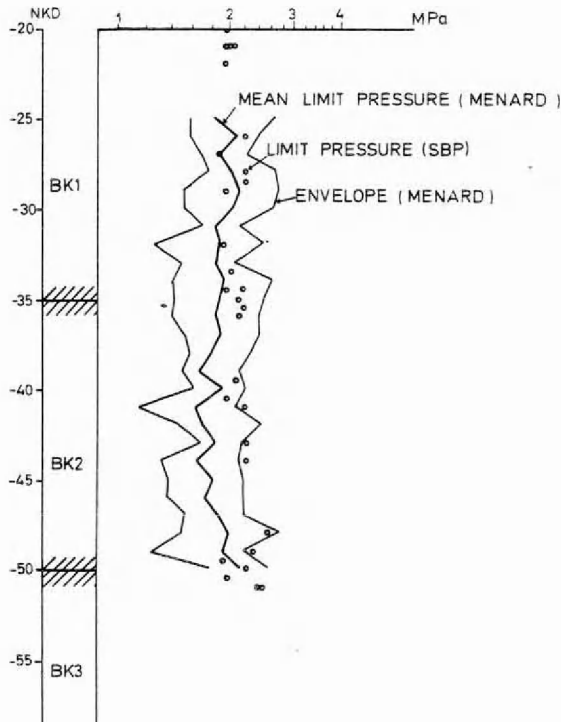
Fig. 12 – MENARD pressuremeter tests. Comparison with self boring pressuremeter data (SBP).

It turned out that the results could be influenced by the drilling procedure. Drilling in short stages (4 to 6 m) and execution of pressuremeter tests immediately after drilling yielded higher pressure modulus values than when drilling was performed in long runs or when testing got delayed. Leaving a borehole open for about 12 hours yielded a limit pressure of 1.05 MPa, while the mean value of 8 measurements carried out shortly after drilling in the same depth interval amounted to 1.86 MPa.

The results of the MENARD pressuremeter tests are summarized in Table 5. Fig. 12 shows the variation with depth of the limit pressure (mean value and envelope of data) ; self boring pressuremeter data, discussed in the next paragraph, have been added for comparison purposes.

Self boring pressuremeter tests (S.B.P.)

Lateral in situ stress and in situ shear modulus being of obvious relevance for the design of friction piles and for predictive finite element modeling, it was decided to carry out self boring pressuremeter tests on the left bank of the Scheldt, down to a depth of 80 m (NKD-72). The contractor was PRESSUREMETER IN SITU TECHNIQUES (P.M.I.T., Cambridge, U.K.). A standard undrained self boring pressuremeter test directly yields three soil parameters (WINDLE & WROTH 1977) : the in situ total lateral stress, the undrained shear strength and the shear modulus.

TABLE 5

PRESSUREMETER RESULTS

	$\overline{p}_1 - \sigma$ (MPa)	$\overline{p}_1$ (MPa)	$\overline{E}_M$,min(MPa)	$\overline{E}_M$(MPa)
BK0	–	1,97		
BK1	1,54	1,97	20	60
BK2	1,46	1,82	17	60
BK3	–	1,83		

with

$\overline{p}_1 - \sigma$ = the mean limit pressure value less one standard deviation

$\overline{p}_1$ = the mean value of the limit pressure

$\overline{E}_M$,min = the mean pressuremeter modulus, when clay is disturbed (drilling with long runs)

$\overline{E}_M$ = the mean pressuremeter modulus without clay disturbance.

Ambient pore water pressure has not been measured. Under the assumption that the pore water pressure distribution is hydrostatic, the effective lateral stress σ'_h may be derived too.

CLARKE (1981) and WROTH (1982) have shown that optimal positioning of the self boring pressuremeter cutter might yield a minimum disturbance of the surrounding medium. HANDY e.a. (1982) also found a good agreement between the determination of in situ horizontal stress with the S.B.P. and with the Iowa stepped blades. The profile of the lateral stress determined through S.B.P. testing should thus, with a fair amount of confidence, be considered as representative for the variation of the in situ horizontal stress with depth. Fig. 13 presents the profile of the effective lateral stress data versus depth.
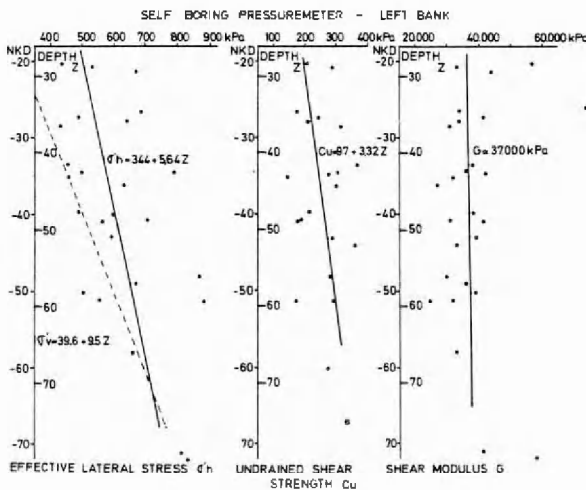

Fig. 13 - Self boring pressuremeter data

A linear regression yields the relationship :

$$\sigma'h = 344 + 5,64\ z \quad KPa \qquad (2)$$

For comparison purposes, the profile of effective vertical stress versus depth has been added to this graph .

$$\sigma'v = 39,6 + 9,5\ z \quad kPa \qquad (3)$$

This profile has been constructed assuming a submerged unit weight of 9,3 kN/m^3 for the Boom clay, of 9,5 kN/m^3 for the overlying saturated sands and a unit weight of 16,0 kN/m^3 for the dry sand cover.

By further processing of the self boring pressuremeter data, the undrained shear strength C of the clay has been determined in accordance with the method GIBSON and ANDERSON (1961). The resulting profile is shown on fig. 13 and might be approximated by

the expression :

$$c_u = 97 + 3,32\ z \quad kPa \qquad (4)$$

The values of the shear modulus G do not show any significant variation with depth (fig. 13) ; the mean value amounts to 37 MPa.

Limit pressure data may be inferred from the stress displacement diagrams of the self boring pressuremeter tests. Limit pressure values defined as the total pressure corresponding with an axial displacement of 10 % are shown on fig. 11. It appears from this plot that these S.B.P. limit pressures are confined between the mean value and the upper envelope boundary of the limit pressures, determined with the MENARD technique.

A synopsis of profiles of some major geotechnical properties on the dam site (cone resistance, pressuremeter modulus and effective shear parameters) is schematically presented on fig. 14.

PAST BURIAL DEPTH

The evaluation of the degree of overconsolidation and hence the maximum past burial depth is of particular importance for the evaluation of the bearing capacity and allowable settlement. Several approaches have been followed.

* Continuous, single channel reflection profiling has been carried out in PVC-lined boreholes. Tube wave reflection patterns thus generated have been interpreted in terms of shear wave velocities (HENRIET, SCHITTEKAT & HELDENS 1983).
Shear waves in marine sediments are known to display a strong velocity gradient in the first tens or hundred meters depth. Following an empirical relation between shear wave velocity and depth in marine silts and clays (HAMILTON 1976), the most representative value of shear wave velocity (355 m/s at the average level of -28 m) suggests a maximum burial depth of about 83 m in clayey or silty sediments.

* Over a hundred oedometer meter tests have been carried out on undisturbed clay samples and have been processed through a graphical approach (CASAGRANDE 1936) in order to determine the preconsolidation pressure.

Various processes, and not the least the sampling process itself and subsequent manipulations, might have caused some loss of preconsolidation but in no case any gain.
Hence, those samples displaying the highest measured preconsolidation might be considered as to represent the closest available approximation of the real preconsolidation. Processing of the data shown on fig. 15 yields a past burial depth of at least 88 m for 1 % of the samples and of at least 69 m for 5 % of
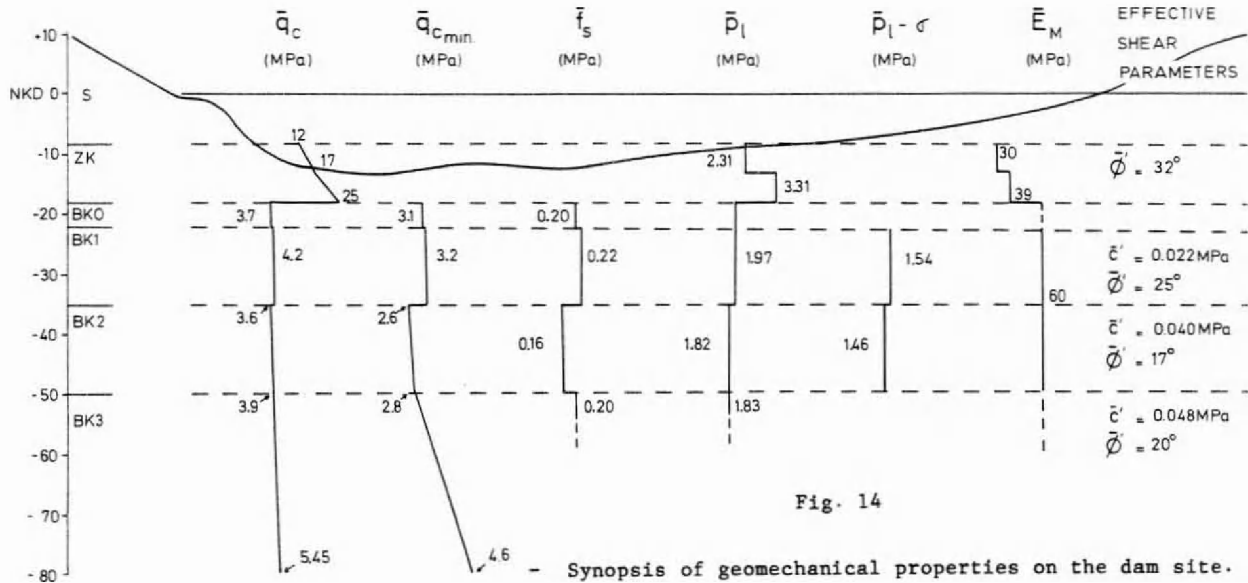
Fig. 14

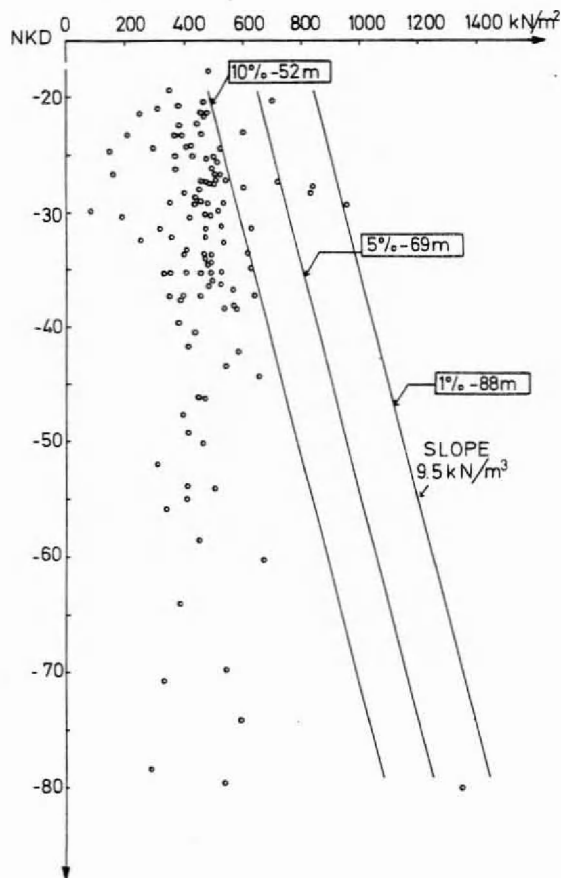— Synopsis of geomechanical properties on the dam site.

Fig. 15

— Interpretation of consolidation tests in accordance with CASAGRANDE.

the samples.

* The lateral stress in the Boom clay has been determined by the self boring pressuremeter tests (equation 2). This lateral stress σ_h' cannot be larger than the upper limit stress, which presumably should be the passive earth pressure σ_p' :

$$\sigma_h' \nleq \sigma_p' = K_p \cdot \sigma_v' + (K_p - 1)c' \cotg \phi' \qquad (5)$$

with K_p = the coefficient of passive earth pressure
$$= tg^2 (45° + \phi'/2)$$

and σ_v' defined by equation (3).

Considering that for BK1 :

$c' = 22 \ kN/m^2$

$\phi' = 25°$

and thus $K_p = 2,46$

equation (5) might be written as

$$\sigma_h' \leq \sigma_p' = 166,3 + 23,4 \ z \ (kPa) \qquad (6)$$

Under the banks of the river, the depth of the top of the clay amounts to about 28 m, which value, introduced in equations (6) and (2), respectively yields

$\sigma_p' = 821,5 \ kPa$ and

$\sigma_h' = 501,9 \ kPa$

At this depth it is thus verified that σ_h' is truely smaller than σ_p'.
The same might be verified for depths larger than 28 m. If $_h$ is not equal to the

upper limit stress, it must be presumed to be equal to the past pressure at rest, which is obtained from the semi-empirical expression defined by JAKY (1948)

$$\sigma_h' = (1-\sin \phi') \; \sigma_{v.o}' \qquad (7)$$

with $\sigma_{v.o}'$ = the past vertical stress.

Assuming a submerged unit weight of about 9,5 $kN/^3$ for the past overburden, $\sigma_{v.o}'$ may be defined by the expression :

$$\sigma_{v.o} = 9,5 \; (z + z_o) \qquad (8)$$

with z_o = the difference in height between the past soil surface and the present soil surface

or $z + z_o$ = the past burial depth.

Substitution of (8) in (7) yields

$$\sigma_h' = (1-\sin \phi') \; 9,5 \; (z+z_o) \qquad (9)$$

The linear functions (9) and (2) must fully coincide, hence

$$9,5.(1-\sin \phi') = 5,64 \qquad (10)$$
$$\text{and } 9,5 \; (1-\sin \phi') \; z_o = 344 \qquad (11)$$

Equation (10) yields a ϕ' value of 24°, in fair agreement with the average internal friction angle deduced from triaxial tests. Substitution of (10) in (11) yields z_o=61 m and the past burial depth is 28 + 61 = 89 m.

* Comments

Apparently, the past burial depth figures derived from seismic data (83 m), consolidation testing (88 m) and S.B.P. testing (89 m) are in fair agreement. The question may be raised whether such figures are geologically plausible or not.

A priori, one might consider two possible past overburden situations : a maximum consolidation during the Rupelian through an originally thicker Boom clay accumulation, or a maximum consolidation induced by a thicker overburden of Neogene glauconitic sands on top of the partly eroded clay.

Glauconitic sands of Miocene and Pliocene age are common around Antwerp. Only a few meters of Kattendijk sands (Pliocene) are found at the dam site, but maximum thicknesses of Mio-Pliocene sands of almost 70 m (of which 45 m of Pliocene sands) do occur northeast of the dam site. Pliocene deposits are built up of a series of small lithological units. There are no arguments to suppose that any of these units might have had a larger thickness than the total preserved Mio-Pliocene sand series. The same consideration holds for the Miocene deposits of the Antwerp area. Taking into account that no major stratigraphic hiatus is known in the pliocene series of the

area (DOPPERT e.a. 1979) and that Miocene deposits have been removed from the area before the deposition of the Pliocene sands, a maximum overburden of Neogene deposits is estimated at less than 70 m.
Younger, Pleistocene overburden deposits are supposed to be significantly less important than the Neogene overburden. Their local thickness rarely exceeds 20 m.

Regarding the first hypothesis, it should thus be concluded that Neogene and Pleistocene overburden only meet the obtained preconsolidation figures by considering maximal thickness estimates for the Plio-Pleistocene deposits.

Regarding the second hypothesis, it has already been remarked that the Boom clay thickens towards north, reaching 170 m south of Breda (Netherlands). Although some stratigraphic thickening of the clay strata might not be excluded, the major part of the thickness increase cannot be accounted for by this phenomenon and it has to be considered that an important clay column has been eroded on the dam site. Various arguments tell strongly in favour of such a major erosion event : an important Oligocene-Neogene stratigraphic hiatus in northern Belgium, the observation of deep erosion gullies on top of the Boom clay on seismic sections in the Southern Bight of the North Sea (HENRIET, D'OLIER, AUFFRET unpublished) and the known major global drop of the sea level during Late Oligocene times (VAIL e.a. 1978). Hence the erosion of 90 to 100 m of Boom Clay at the surge barrier site seems, in view of recent geological evidence, by no means to be excluded.

DYNAMICAL SOIL PARAMETERS

Several elastic constants have been determined by borehole seismic transmission and reflection measurements. All seismic work has been carried out in and between boreholes, lined with PVC casing. The seismic source was a single electrode sparker, fired at 100 to 300 Joules.

Horizontal P-wave velocities have been measured by crosshole shooting at regular intervals. The vertical distribution of vertical P-wave velocities has been determined by uphole and downhole shooting analysis of P-wave reflection patterns generated by continuous vertical reflection profiling in boreholes and by wide angle profiling on the Scheldt.

Tube wave reflection patterns generated by the borehole seismic profiling have been interpreted in terms of shear wave velocity and dynamic shear modulus G (HENRIET, SCHITTEKAT, HELDENS 1983). POISSON's ratio has been determined from the ratio of vertical P wave velocity and S (shear) wave velo-

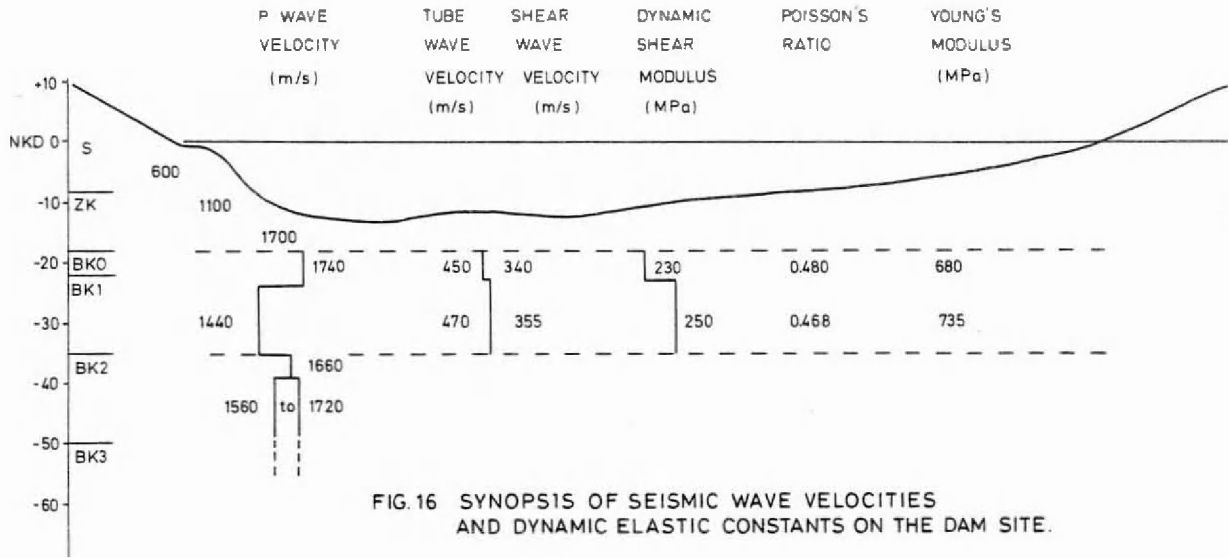|	P WAVE VELOCITY (m/s)	TUBE WAVE VELOCITY (m/s)	SHEAR WAVE VELOCITY (m/s)	DYNAMIC SHEAR MODULUS (MPa)	POISSON'S RATIO	YOUNG'S MODULUS (MPa)
+10						
NKD 0 S	600					
-10 ZK	1100					
	1700					
-20 BKO	1740	450	340	230	0.480	680
BK1						
-30	1440	470	355	250	0.468	735
BK2	1660					
-40	1560 to 1720					
-50 BK3						
-60						

FIG. 16 SYNOPSIS OF SEISMIC WAVE VELOCITIES AND DYNAMIC ELASTIC CONSTANTS ON THE DAM SITE.

city.   YOUNG's modulus got determined from the shear modulus and POISSON's ratio.

A synopsis of seismic wave velocities and dynamic elastic constants is presented on fig. 16.

P-waves velocities of 600 to 1100 m/s, measured below the water table, are definitely abnormal for saturated conditions (the P-wave velocity in water amounts to 1480-1500 m/s). They have to be related to the presence of sediment gas (methane). As a matter of fact, two gas eruptions have been observed during penetration testing on the left bank, between 10 and 18 m depth.

In the Boom clay itself, a P wave velocity low of approximately 1440 m/s should also be traced to the presence of a minor gas concentration, probably trapped in the silty horizons of BK1 and related to the presence of organic matter in the clay.

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