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PORT OF ANTWERP

Renovation of the existing port infrastructure
Making the America Dock, Albert Dock and 3rd Harbour Dock accessible for Panamax vessels

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Authors:

— irs. **G. Thues** and **W. Thibaut** of the Technical Service of the Antwerp Port Authority;

— irs. **P. De Schrijver**, **R. Charlier**, **E. Laforce**, **L. Meyvis** and **L. Corluy** of the Ministry of the Flemish Community, Environment and Infrastructure Department.

— irs. **P. Boucquet**, **Sol**, **Dupont**, **J. Himpe**, **F. Elskens**, **L. Ponnet**, **F. Augusteyns** of the contractors Van Laere, Bencor, Fundex, Herbosch-Kiere, Bitumar, H.S.S. and Dredging International;

— ir. **R. Dedeyne** of Seco;

— Prof. ir. **Bourgois** of the Department of civil engineering of the Royal Military Academy.

Pictures:

— **Guido Coolens**

— Site Inspectors of the Antwerp Seaport Service.

— the Topography and Photogrammetry Service of Environment and Infrastructure Department of the Ministry of the Flemish Community.

Drawings:

— the drawing-offices of the Antwerp Seaport Service and several contractors.

General co-ordination and translation:

— the Antwerp Seaport Service

1. OBJECTIVE

Since the end of the 70's the Antwerp port area has been accurately defined in the regional territory planning. The available space is described as industrial zone and comprises 14,000 hectares, of which 7,600 hectares on the right river bank of the Scheldt and 6,400 hectares on the left bank. 3,670 hectares has been allocated to port related industries, mostly petrochemical plants, oil refineries, ship repair and car assembly.
Transhipment of goods from sea-going vessels and the related storage and distribution takes place in an area of about 14,400 hectares.

In 1992 maritime trade totalled about 103 million tons and this volume is still increasing along with the expansion of the economic activities of the port's hinterland.

Year:	1975	1980	1985	1990	1995	2000 (prognosis)	2005
Goods:	60.4	81.9	95.4	102	114	127	139.5

This hinterland comprises continental Europe, Great Britain, Ireland and Scandinavia, as well as overseas like Africa, the Middle East, the other side of the Atlantic and Pacific Oceans.

To allow for continued expansion within the confined space every square metre of the port area must be completely utilized.
Every area made available induces many candidate investors and operators, and the waiting list is getting longer and longer.

However productivity in certain port areas has been stagnating or even falling off for ten years. This productivity is measured on the basis of parameters such as the movements of the sea-borne goods per square metre port area and per year. Compared with other seaports, the scores have been relatively high in Antwerp. The only way to increase the flow of goods, leaving the expansion on the left bank apart, is to improve area productivity on the right bank.

The above-mentioned stagnating areas are located, as would be expected, in the older port, i.e. the docks built before the First World War (1914-1918). The docks south of the Royers Lock and the Albert Canal, built between 1811 and 1880, proved to be too small in length, width and draught to be adapted in situ. Limited quay size does not allow modern goods handling either. Therefore these docks are now assigned to inland and coastal navigation. The land area will house support industries for the port such as repair shops for ship equipment, ship's chandlers and offices of port-related enterprises.

The northern docks such as the America Dock and especially the Albert Dock and 3rd Harbour Dock are adaptable to present-day requirements.

This adaptation has 2 aspects:
- on land: by extending the area of the port terminals, i.e. by increasing the size of the area (the distance perpendicular to the quay wall);
- on water: by deepening the docks to enable modern standard size vessels to moor.

To meet the first requirement, the present size of the handling area (60 to 80 m) has to be increased to more than 100 m, preferably even 250 m. To this end roads and railroads must be relocated a greater distance from the quay and existing sheds and facilities demolished. In the case of the 1st Harbour Dock, built in 1906, the only solution is the filling up of the dock and adjacent shelter dock for lighters, and the construction of a new deep founded quay wall on the eastern side of the Albert Dock.

To increase water depth, the dock bottom must be dredged and the quay wall adapted. The studies and works involved are described extensively hereafter.

Once the concept had been examined thoroughly by a working group of the Port Advisory Board eight years ago, the need for implementation became more urgent, since port expansion on the left bank alone could not meet the increasing trade. Several large port companies located in the Albert Dock and America Dock zone were faced with the dilemma of either leaving the premises (most probably to a destination abroad), or staying in their facilities worth several billion belgian francs, on condition that they could rely on the planned renovation.

Their disappearance would cause considerable capital reduction but also the laying off of many hundreds of specialised workers.

Commodities mainly affected use middle-sized bulk vessels, so-called Panamax vessels: grains and related commodities, fertilizers, sugar and cement. For this reason the Panamax vessel was taken as the standard for the redesigning of the docks.

A few years ago the **Technical Service of the Port of Antwerp** drew up a masterplan for the renovation of this part of the port, and the accompanying construction of a fairway for sea-going vessels with a draught of 40' to 42', type Panamax. Different alternatives were examined, in particular the construction of a northern and a southern sailing route.

After examination of the alternatives using varying criteria, such as the total cost, road accessibility of the terminals, adaptation of the rail links, the time schedule etc., the construction of a fairway via the southern route, i.e. through the Hansa Dock, 5th Harbour Dock and America Dock, was given the highest priority by the Advisory Board.

Indeed, the adaptation of the port infrastructure along the northern route would take much longer (at least 6 to 7 years) and is much more expensive (at least 8 billion Bfr.). This means that the northern route will only be studied when the southern route has been completed.

To realize the fairway via the southern route, the docks involved must be systematically deepened and bottlenecks removed.

Very important infrastructure works are under way or must soon be started and completed before mid-1994; otherwise major parts of the present infrastructure would become useless, and the global renovation project would be a failure.

The most recent completions of infrastructure, those in the course of construction or to be started soon are described herein.

Indeed these radical renovations require the availability and expenditure of significant sums of money, required in the short term and strictly controlled.

Therefore **Prof. Dr. G. Blauwens** was commissioned by the City of Antwerp to carry out a cost-benefit analysis to estimate the socio-economic impact of the investments.

Terminal operator **Manuport** commissioned a similar cost-benefit analysis from Prof. Blauwens regarding the infrastructural requirements to modernize their terminal in the 3rd Harbour Dock.

These analyses clearly show the effects of either proceeding with the renovation or not and prove their economic profitability, but are not further discussed here.

2. PLANNING OF THE PROJECT

(see Fig. 1A — Port of Antwerp)

Several works have already been completed, e.g.

1. The widening and deepening of the fairway between the Hansa dock and 5th Harbour Dock, including a service tunnel.
2. The construction of new quay walls on the south side of the America Dock.
3. The connection of the 5th Harbour Dock and the America Dock.
4. The construction of the Noordkasteel bridges.
5. The construction of a service tunnel under the passage between the America Dock and the Albert Dock.
6. The deepening of the 5th Harbour Dock, the America Dock, including dredging for the newly built quay walls in the America Dock.

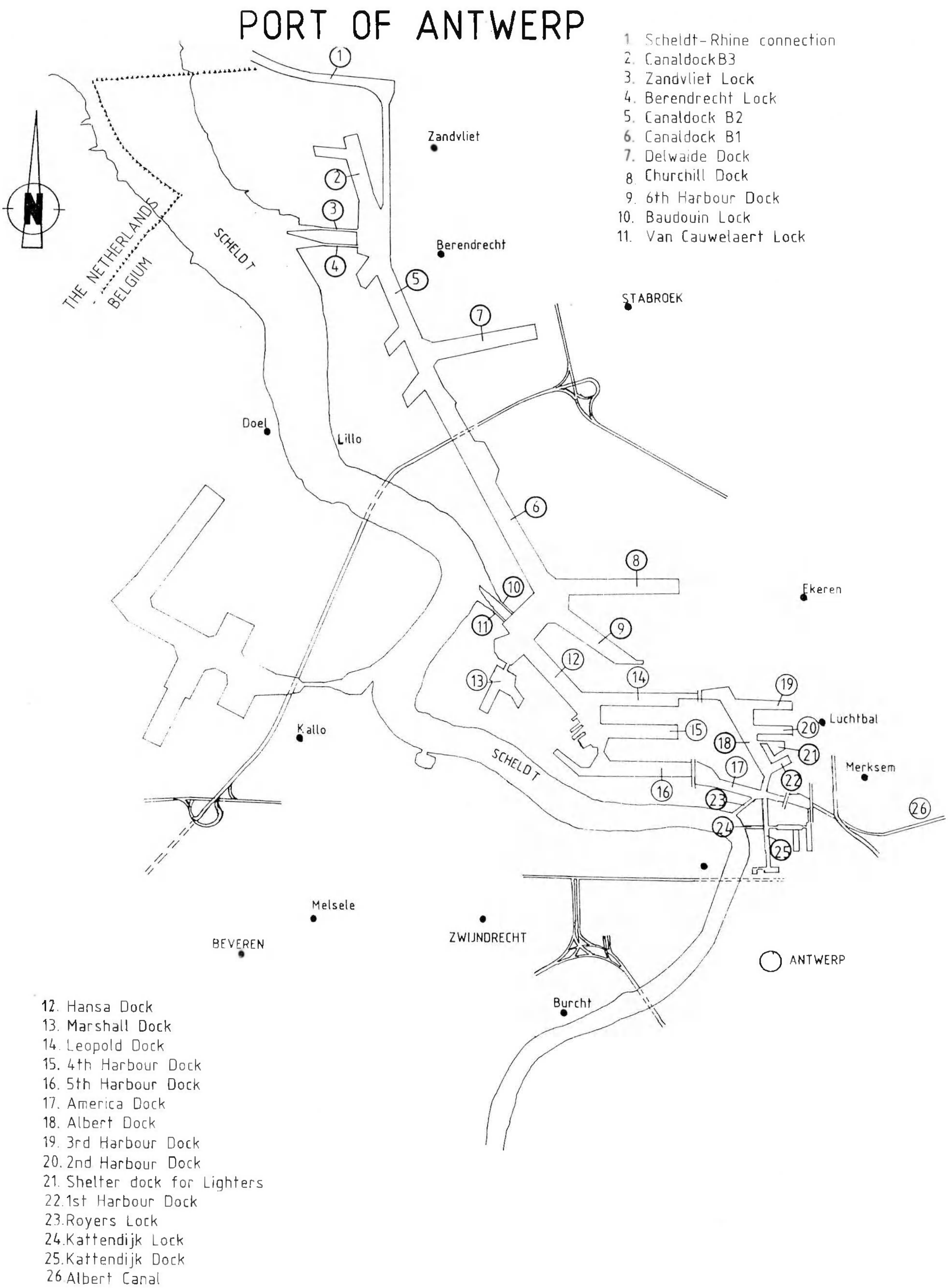

Fig. 1A — Port of Antwerp.

IN PROGRESS 1991-1993

1. Passage widening and deepening
2. Service tunnel
3. Shelter dock for lighters
4. Renovation north quays 3rd Harbour Dock
5. VHP technique
6. Deepening dredging works+ miscellaneous
7. Refilling of existing docks

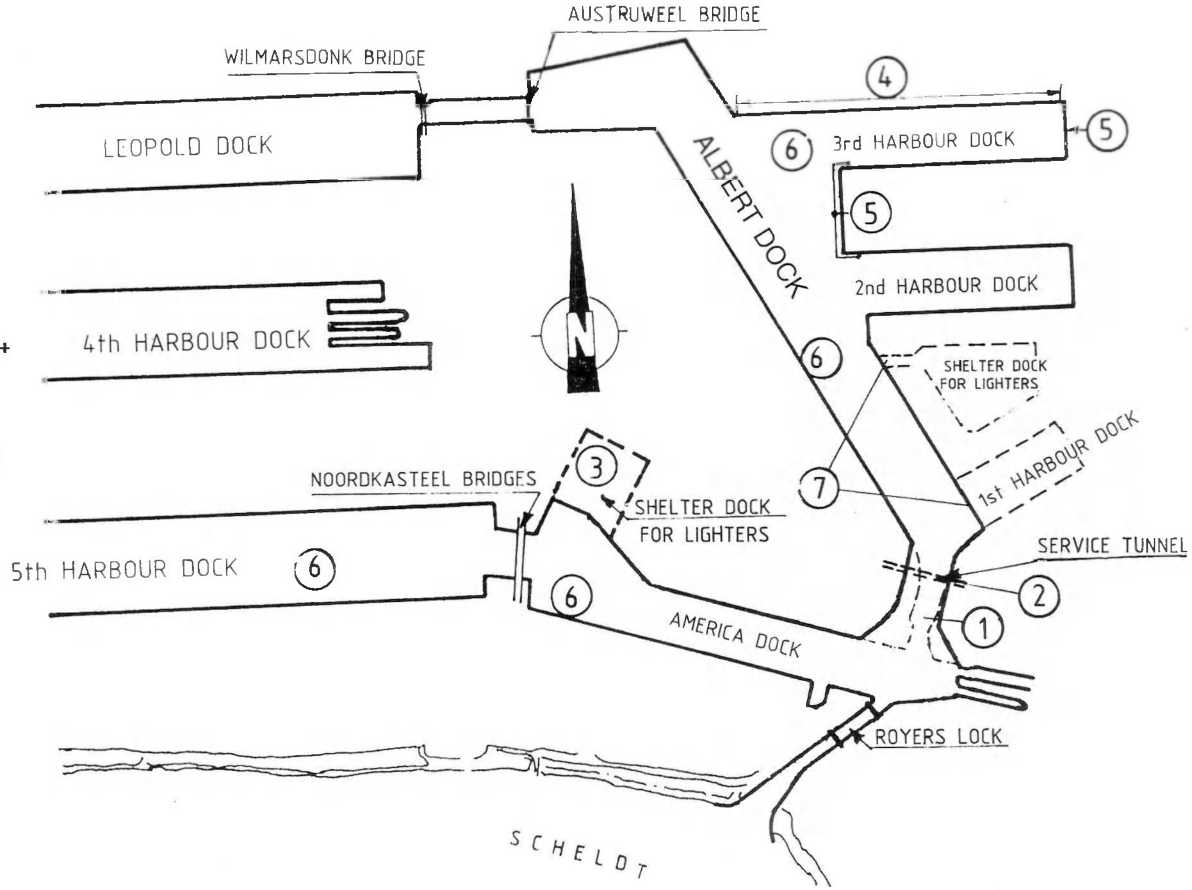

Fig. 1B — Port of Antwerp. Zone to be renovated.

3. DESCRIPTION OF THE PROJECT UNDER CONSTRUCTION

The present renovation project consists of the following major divisions:

1. The widening of the fairway between the America Dock and the Albert Dock. This includes the further deepening of the America Dock, construction of a swinging basin for Panamax vessels in the America Dock and a safe passage for ships heading to and from the Albert Dock, America Dock, Albert Canal and the Royers Lock (either renovated or not).

   For this reason about 1,000 m old quay walls, including attendant facilities, sheds and warehouses have to be demolished and replaced by new bank reinforcements.

   The spoil produced by the dredging work is used to fill the old shelter dock for lighters and the First Harbour Dock. In this way new areas are created for modern commodities handling. The remainder is reserved for sand stocks along the Sea-Scheldt for later use in the construction and maintenance of dikes in the framework of the Sigmaplan, and for the construction of new industrial or distribution zones.

2. The construction of deep 3,100 m long quay walls along the northern and southern bank of the 3rd Harbour Dock and the east side of the Albert Dock.

3. Deepening the Albert Dock and the 3rd Harbour Dock for vessels with a draught of 40 to 42 feet. The spoil is used to conserve sand stocks for the construction of the A-12 Harbour Road, industrial areas and distribution zones (Renaval). The remainder to increase sand stocks for the dike works of the Sigmaplan.

4. Construction of a new shelter dock for lighters with accompanying facilities to meet modern inland navigation requirements.

5. Re-shaping and relocating Vosseschijn Street and road works in the vicinity of the new terminals.

6. Relocation of the rails and reconstruction of the shunting yards and sheds north of the 3rd Harbour Dock.

7. Construction of the required superstructure including demolition of existing sheds, paving new terminals, building of adapted sheds, cranes, conveyor belts etc. to implement the modernisation of the terminals operated by **Manuport**, **Northern Shipping**, **Westerlund** and others.

The last mentioned investments are to be made by the stevedores and terminal operators themselves. Therefore these investments are not discussed here.

It is worth noting, however, that some of these terminal operators have already made major investments in cranes, conveyor belts, sheds etc., which constitute a much larger expenditure than that of the Authority, thus anticipating the adaptation of the marine infrastructure.

The profitability of these investments may be increased by extending and further implementing the present programme of renovations to the quay walls on the west bank of the Albert Dock, northern quay wall of the America Dock and completion of the northern fairway etc.

These topics are not covered by the present renovation project.

4. DESCRIPTION OF THE PROJECT PARTS

4.1. The passage between the America Dock and the Albert Dock

The lay-out of the passage — simulation study.

The passage was built around the turn of the century and links the present America Dock (the former Lefebvre Dock), built in 1887, with the Albert Dock and the 1st Harbour Dock, both brought into service in 1907.

The passage has a length of 300 m and a minimum width of 52 m. The passage is at right angles to the America Dock and the draught is minimum 8 m.

The water level in the docks is TAW + 4,25.

Further this passage links up with the access to the Albert Canal and to the Royers Lock, which is the main link for inland navigation between the Scheldt and the port.

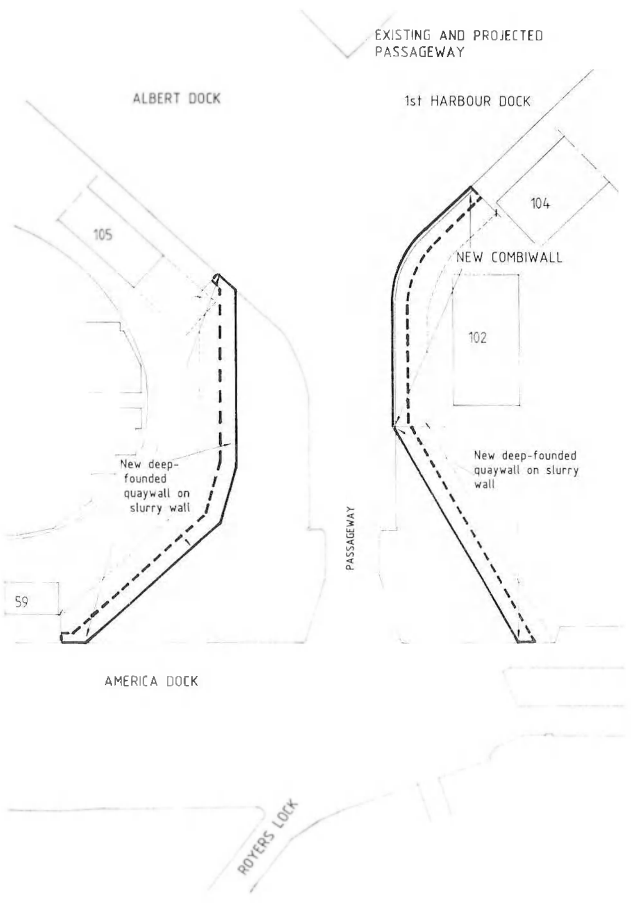

Fig. 2 — Existing and projected push tow passage.

It is clear that the restricted size of the passage does not allow for Panamax vessels. Based on the experience of different port users such as **Brabo dock pilots**, **Harbour Captain Services** and a few push tow companies a first lay-out of the new passage was designed.

The widening of the passage to the Albert Canal was taken into account as well, so that large push convoys (4 boxes, i.e. 22.80 m x 185.00 m) will be able to enter the Albert Dock and the rebuilt Royers Lock from the Albert Canal.

At the same time lay-outs were examined for different sizes and locations of a renovated Royers Lock. To a large extent the situation and size of this lock have determined the basic concept.

Then the nautical properties of the approach route and the projected passage were tested on the ship simulator of the **Hydraulics Laboratory** at Borgerhout, Antwerp.

The simulations were carried out with two types of vessels, namely a Panamax with a length of 230 m and a width of 32.2 m (approx. 70,000 DWT) heading for the Albert Dock with a draught of 40 feet and leaving in ballasted with a draught of 26 feet, and a bulk carrier with a length of 260 m, a width of 38.4 m and a draught of 40 feet (approx. 110,000 DWT).

Immediately after the passage to the Albert Dock the vessels have to pass through the Noordkasteel bridges with a navigable width of only 50 m. Also, the bridge passage is off centre from the axis of the America Dock. The latter has been extended to facilitate navigating and to allow smaller vessels to swing round, but the remaining navigable channel was only approx. 280 m long. First an extension of the distance between the bridge and the shore at level TAW to 330 m –10.50 m was foreseen. This means that when a 230 m vessel leaves the passage with aft tugs, a clearance of approx. 50 m between the bow and the shore remains. Therefore the navigation of these bridges was also part of this study.

The simulator consists of a ship's bridge with steering gear, monitoring instruments, radar image and an outside view. The outside view reflects the view through the windows of the bridge either in the centre or on the starboard or port side.

The pilot reacts to the ship's behaviour by instructing the engine, the rudder position, the tugs etc. to carry out the manoeuvre. The monitors are connected to a computer which is continuously calculating the impact of the hydrodynamic forces and the environment. The speed, position and heading of the ship are computed and shown on the instruments, the radar and the outside view. In this way the pilot has the illusion that he is navigating on a real vessel, so that he will react in as natural a way as possible.

During manoeuvre tests 40 parameters are continuously being measured and the real course is drawn.

The wind data in the America Dock before the Royers Lock show that the average wind speeds over a period of 10 minutes exceed 10 m/s only during only 1.5 % of the time. Therefore this parameter was set at an average gusty wind of 5 Beaufort. The influence of buildings along the America Dock such as high grain silos, the sheds, moored vessels etc. was also included in the calculations. The evolution of the windflow was measured in a special set-up on a small-scale model, scale 1/500, and the results were entered into the ship manoeuvre simulations.

Also the variability of the wind was simulated. The turbulent structure of the wind was calculated with a Von Karman spectrum and superimposed over the average wind-force. In this way the wind-force could reach 7 Beaufort at certain times or even fall back to 3 Beaufort.

The axial and lateral forces as well as the moment to the hull caused by the wind were calculated in their turn, in determining the apparant wind-force and direction, making use of the wind factors for large tankers. Further special effects such as bank suction, limited navigable width and draught were also entered into the mathematical model.

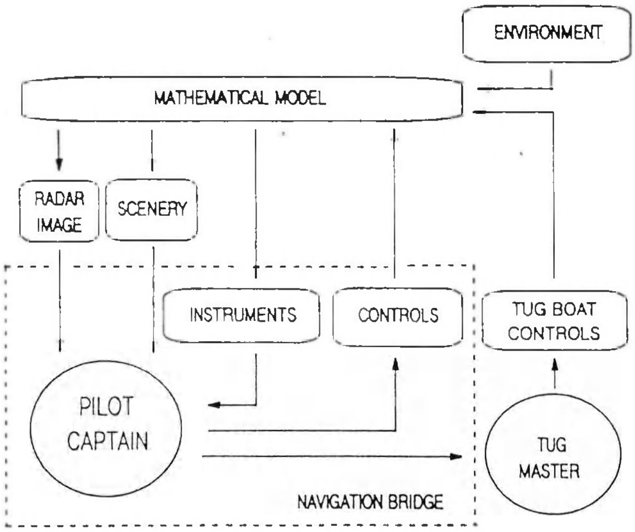

Fig. 3 — Ship simulation principle.

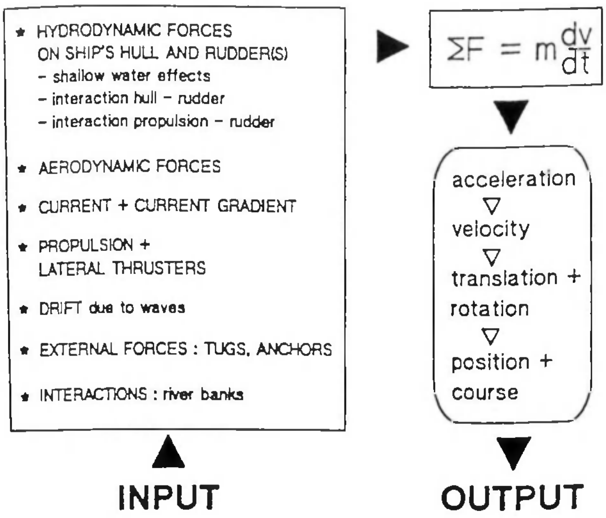

Fig. 4 — Simulation — Mathematical model.

Tug assistance consisted of 2 to 3 municipal tugs propelled by Voith Schneider with a spud tractive power of 22 tons. The tugs were steered by tug captains of the **Municipal Tug Company**.

The navigation tests were carried out by Brabo dock pilots. The study was implemented in different stages. The original design of the channel between the America Dock and the Albert Dock was gradually adapted in the course of the study. The minimum navigable width was eventually extended to 160 m, allowing for 260 m bulk vessels to pass the channel and even to swing around 180° ballasted.
It also proved necessary to widen the swinging basin near the Noordkasteel bridges.

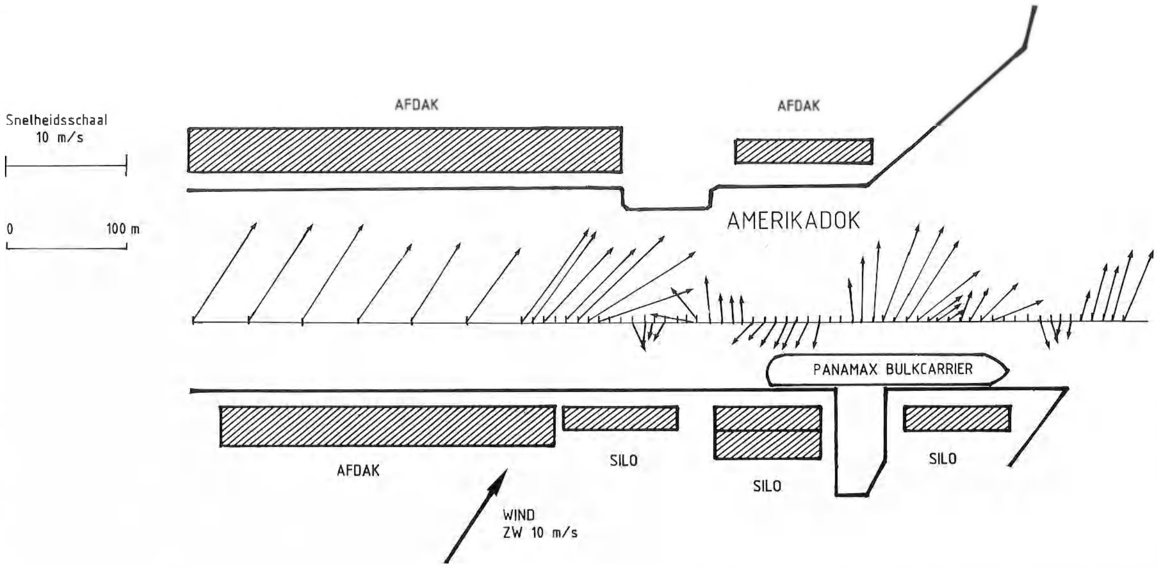

Fig. 5 — Windflow in the America Dock.

Simulations proved that with an average wind-force of 5 Beaufort and gusts up to 7 Beaufort departing Panamax vessels with a length of 230 m and ballasted sometimes had difficulties when passing the Noordkasteel bridges, whereby the pillars of the bridge were hit.

Based on further simulations this problem was solved by widening the America Dock up to 380 m before the bridges around the axis of the bridge.

In this way a departing vessel can be steered around the axis of the bridge at a distance of 150 m from the bridge.

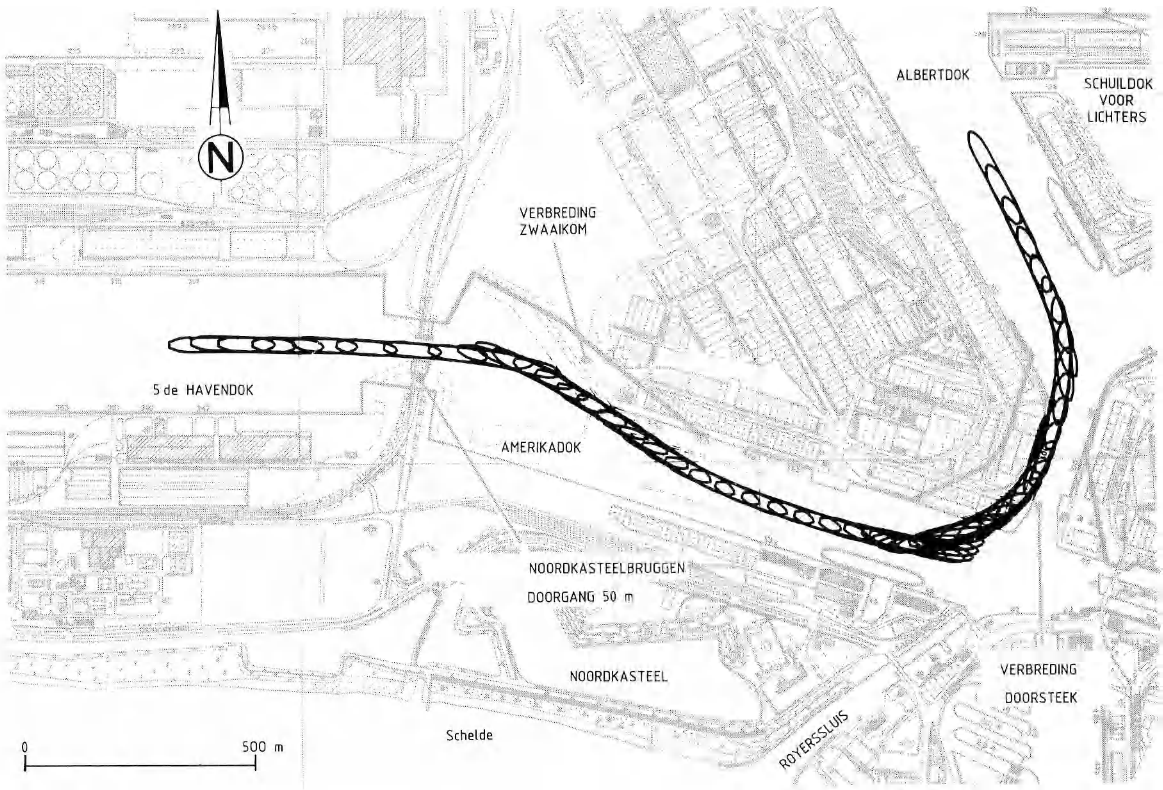

Fig. 6 — Path of a bulk carrier with a length of 260 m heading for the 3rd Harbour Dock.

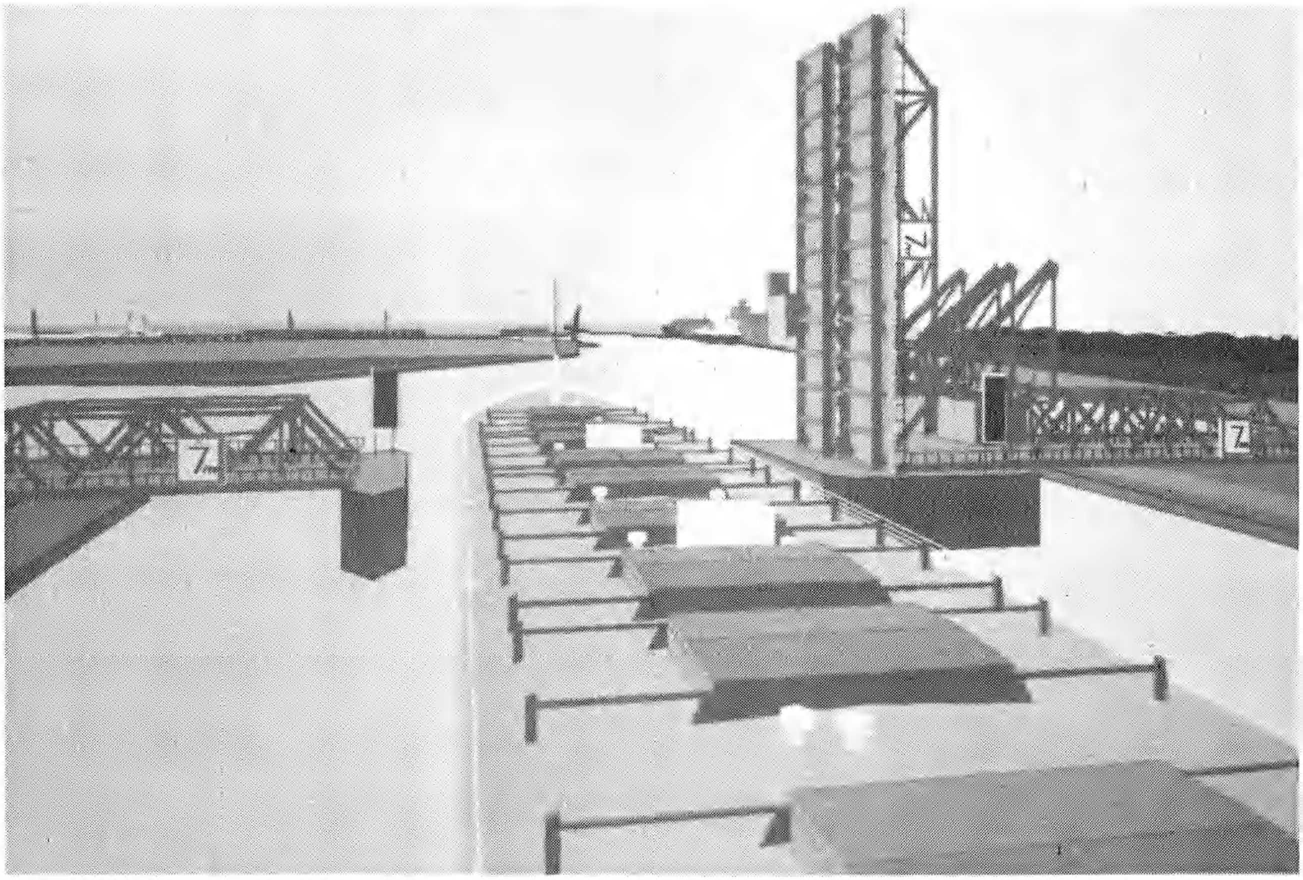

Fig. 7 — Noordkasteel bridges. Computer screen as seen from the simulator.

After this adaptation the passage appeared to be safe. The ship simulation allowed pilots and captains to gain experience in carrying out manoeuvres in a non-existent environment.

The step-by-step evaluation and adaptations to the lay-out of the channel could be worked out thanks to experience with actual vessels, and on the simulator and also the statistical analysis of the navigation paths sailed.

4.2. Passage between the America Dock and the Albert Dock

Construction of a service tunnel under the widened passageway between the America Dock and the Albert Dock.

Contractor: Joint venture **M.B.G. — Wayss und Freytag**.

Introduction

Before widening the passage between the America Dock and the Albert Dock a number of cables and conduits which cross the navigation channel had to be laid deeper and relocated.

The terms of reference included the construction of a tunnel with a minimum diameter of 2.80 m which could either be drilled, pushed, or sunk.

The tunnel had to be made accessible on both banks through reinforced concrete access shafts.

The shafts had to be constructed without the surrounding groundwater level being altered.

The most economical offer provided the construction of two sunken shafts and a drilled and pushed tunnel in

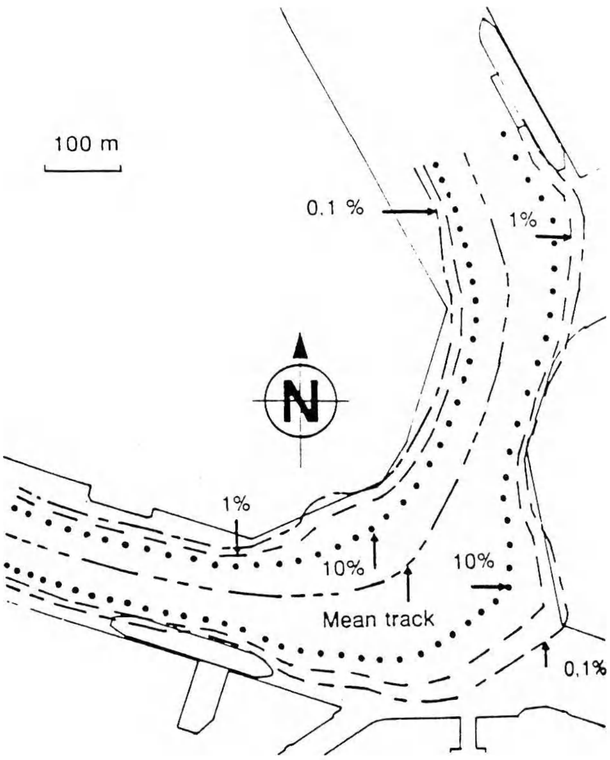

Fig. 8 — Statistically extrapolated boundaries of the navigation lanes.

reinforced concrete with an inside diameter of 3.00 m. This bid was also selected.

Construction

The shafts on both banks have a double aim: firstly to provide a vertical access to the service tunnel, and to enable the drilling shield, tunneling plant and pipes to be placed and secondly, after the drilling is finished, to bring the drilling shield back to the surface.

As a result the sizes of the shafts were determined mainly by the construction method of shafts and tunnel.

— quays 101-103 (opposite bank), the entry shaft:

outside diameter: 11.40 m
inside diameter: 9.00 m
height: approx. 27.00 m

— quay 102 (even bank), the exit shaft:

outside diameter: 8.30 m
inside diameter: 6.50 m
height: approx. 26.00 m

Each shaft was provided with a 2 m concrete plug to prevent it from rising.

The shafts were made of reinforced concrete and sunk. They were constructed at different concreting stages.

The concrete was cast in a mold using the layered system. An expansion joint was inserted between each concrete layer. The reinforcement consists of pre-shaped welded nets.

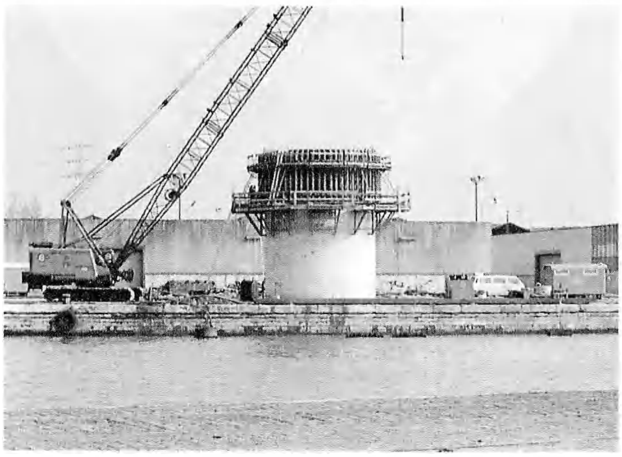

Fig. 9 — Construction of the entry shaft with a jumping formwork.

As no draining of groundwater was allowed because of the presence of buildings sensitive to settlement in the immediate vicinity, the wet soil within the shafts was excavated with a heavy grab crane, thus enabling the shaft to be sunk under permanent monitoring conditions.

To reduce the friction of the casing against the cylinder cheek a bentonite lubrication was applied.

After the shafts had been sunk to their required level they were provided with a reinforced concrete base slab which can absorb the full hydrostatic uplift.

Special attention was given to the joints between tunnel and shaft, so that deformations and differential settlements could be absorbed without endangering the waterproof condition of the construction.

To this end openings were made in both shafts for anchors to fit the joints of the shield. The openings in the shaft wall to allow the passage of the shield were constructed in a cylindrical shape and filled before sinking with mortar having a limited strength after 28 days.

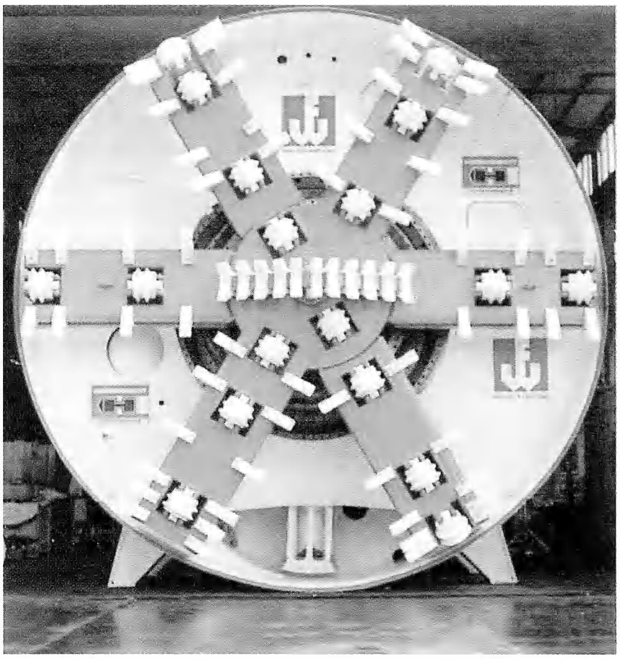

Fig. 10 — Wayss und Freytag Hydromix shield.

The tunnel itself with an outer diameter of 3.60 m and a length of 212 m lies at a depth (base of the pipe) of 23.50 m below groundlevel.

This tunnel was constructed of reinforced concrete and built by means of a pushing technique.

Its overall length is reached by juxtaposition of 70 concrete sections of 3 m length each. These pipes are 30 m thick and have a steel core (5 mm thick) covered with a 5 cm concrete layer.

For the pushing of the pipe elements a **Wayss und Freytag** mixshield was used. The hydromix shield is an excavator consisting of a horizontal cylinder with a diameter of 3.60 m. Fully equipped it weighs 60 tons.

The front, the drilling chamber, is separated hermetically from the chamber behind it. A diaphragm extending under the axis divides the drilling chamber into two partitions.

The bentonite suspension fills the two partitions. An air cushion in the aft upper chamber supplies the needed pressure on the liquid. As a result the liquid will mount up to the crest of the shield in the front chamber. In this way the bentonite suspension fully supports the drilling front. It balances ground and water pressure. By increasing cohesion it also improves the soil structure. The star-shaped drilling head with 4 forked arms is equipped with cutting and grinding bits. This cutting head can turn in two directions with a continuously controllable velocity of 0 to 2 r.p.m. The sand and clay are ground off regularly by the drilling head.

The depth location is such that during construction there is a ground covering of 20 m on top of the conduits and 8 m in the channel, with a water pressure of 22 m watercolumn.

The tunnel is drilled partly in a tertiary sand stratum in the upper layers of the Boomse clay.

The tunnel was constructed in the following way:

— the cutting head of the machine turns clockwise or counter clockwise;

— at the same time the shield and the reinforced concrete pipes behind it are hydraulically pushed forward from the main compressing station in the entry shaft, whereby the mantle friction on the outside of the pipes and the shield is overcome;

— the soil in front of the drilling head is ground away by the forward movement of the drilling head and the simultaneously turning cutting wheel;

— the ground soil is transported hydraulically through a pipeline system (flush conduct) and is achieved by means of sand dredging pumps;

— the soil/bentonite suspension is passed through a washing and screening plant and a separation plant, so that solid and liquid parts are separated;

— the solids are removed by truck, whereas the regained liquids are recycled to the shield;

— this cycle is repeating itself permanently during the tunneling and is only interrupted to introduce a new pipe section in the entry shaft;

— in this way the tunnel is extended one pipe section at a time from the entry shaft, whereby the entire pipe is moved during the tunnelling operation;

Since, with this tunnel length, the power of the main compression station is not sufficient to surmount the mantle friction, two additional intermediary stations are inserted along the tunnel length;

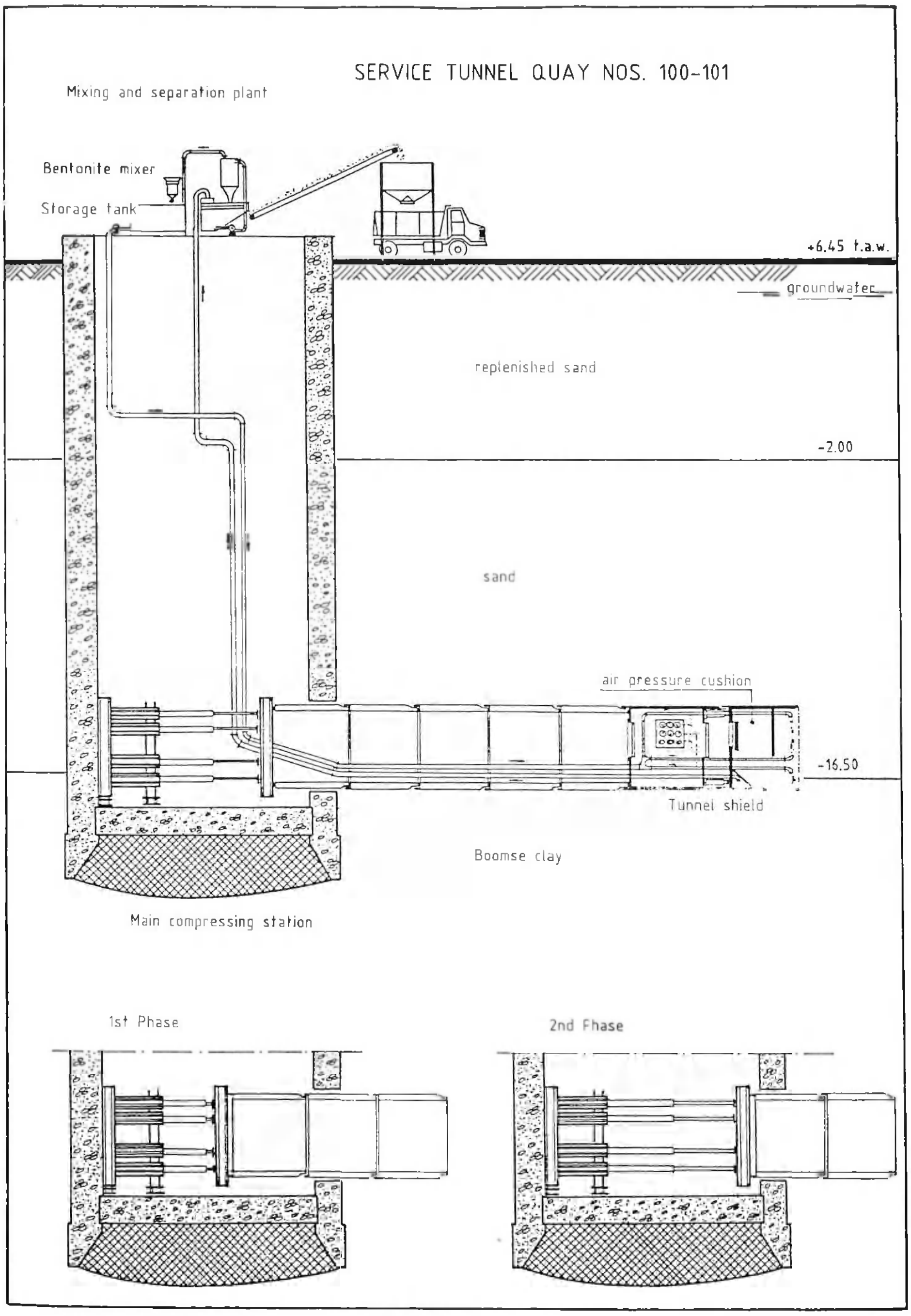

Fig. 11 — Principle of pushing the tunnel sections with the hydromix shield.

- the stability of the 3.60 m high drilling front is brought about by the combined action of air pressure and bentonite;
- the air pressure is adjusted to the soil and water pressures. Measurements during the drilling are carried out by laser beams emitted from the entry shaft;
- Piercing through the entry and exit shafts with the hydromix shield took place after the soil bulk behind it had been stabilised by freezing. The ice plug was formed by introducing liquid nitrogen into the ground.

To this end metal lances were drilled into the ground.

In this way liquid nitrogen from a cryogene storage tank can be pumped into the soil through the pipes and lances. The evaporation and warming up of the nitrogen freezes the ground water between the soil grains.

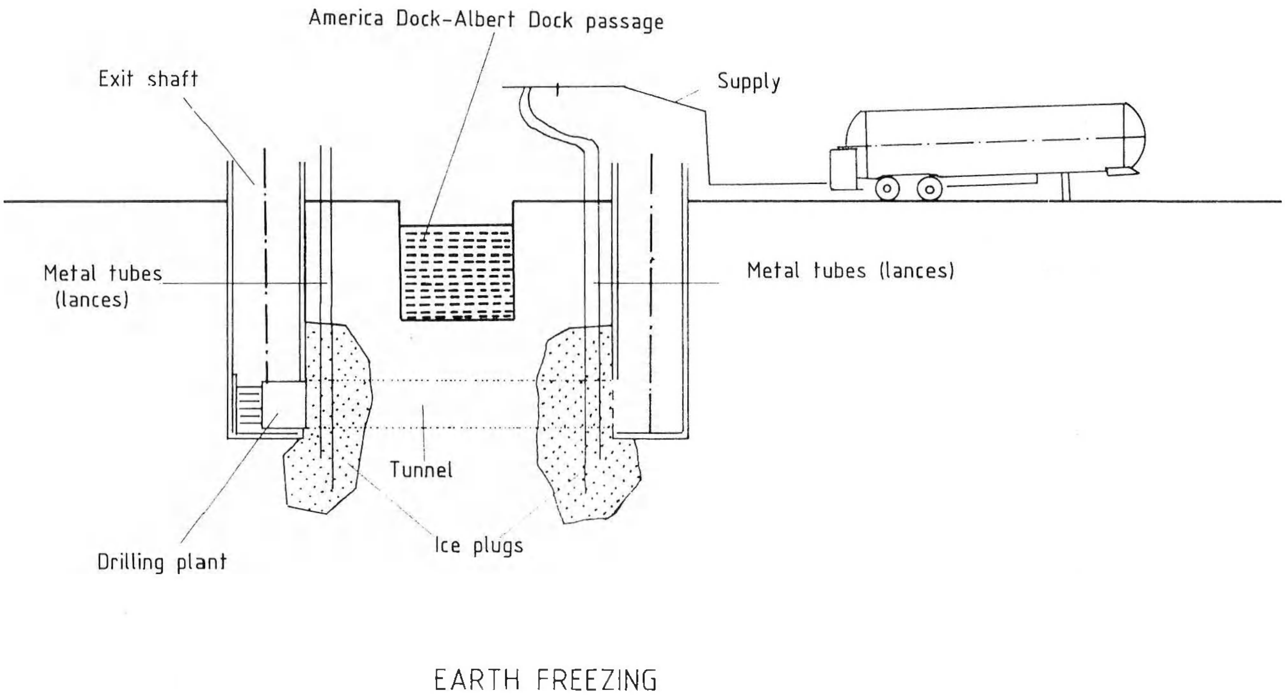

Fig. 12A — Introduction of liquid nitrogen into the ground.

Above the lances the nitrogen, now gaseous and warm, flows out through an opening. The temperature of the escaping nitrogen is gauged and computer regulated to keep it low enough.

Also a number of temperature gauges are introduced into the ground to monitor the process.

Initially the largest quantity of nitrogen is consumed, since the groundwater is being frozen at this time. Then the ice plug must be maintained to enable the drilling work. This consumes much less nitrogen. Conversely nitrogen consumption is one of the best indicators to check whether the ice plug has been formed.

Some 20 days after the lances and nitrogen are introduced, the concrete and the soil are drilled through. Finally the soil is defrosted and the lances are immediately removed.

— After the entry shaft had been pierced the pushing of the 212 m long tunnel took less than 5 weeks.

After the exit shaft has been reached the joints of the pipes are fitted.

A special joint was fitted in the tunnel at the height of the new quay wall in order to absorb the transmitted load.

The tunnel and tunnel shafts were designed to use high vacuum cathode tubes to electrically insulate conduits from the steel parts of the tunnel.

The attachments and cable racks are galvanised and can absorb forces caused when conduits are empty and the tunnel is filled with water.

Fig. 13 — Exit shaft viewed from above.

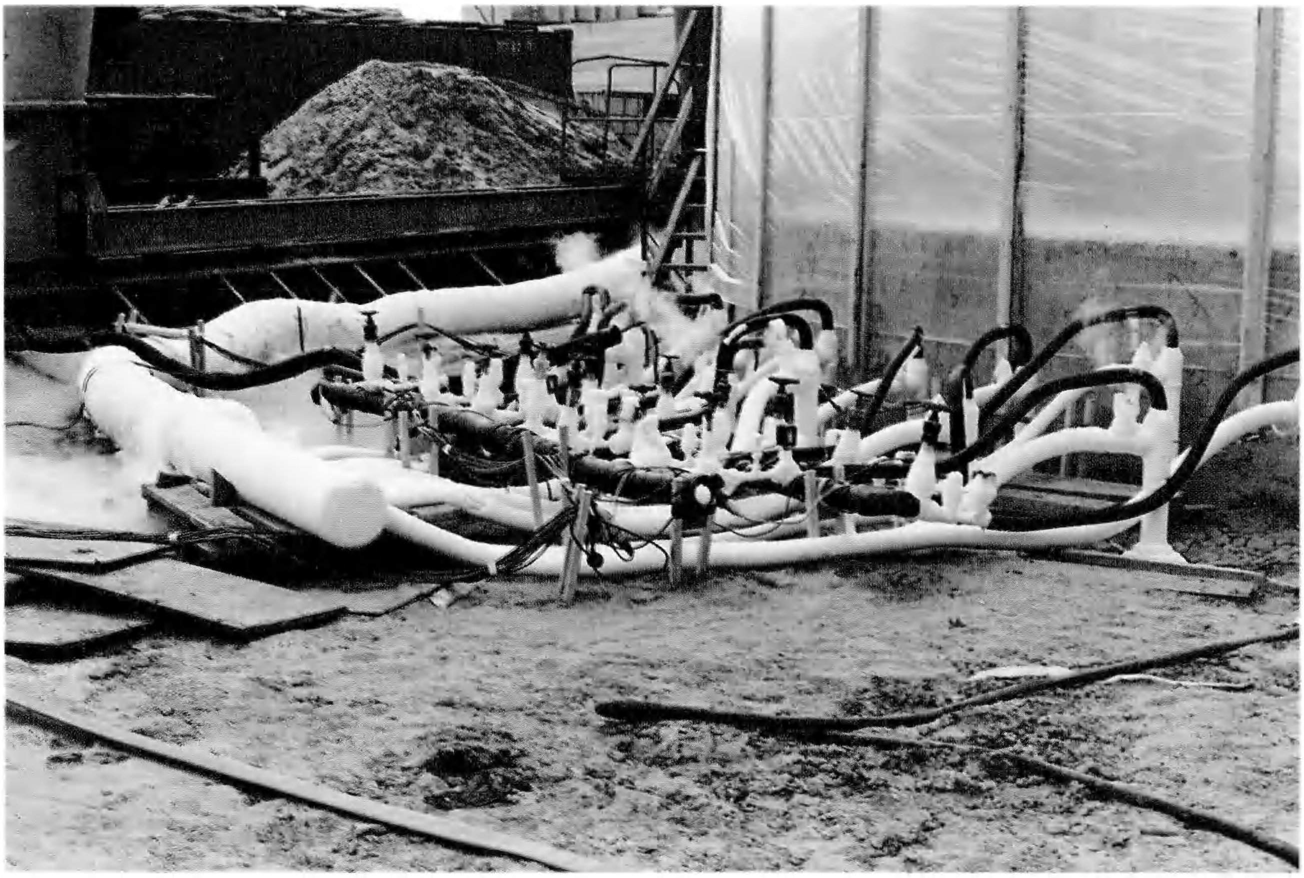

Fig. 12B — Metal lances near the shaft.

4.3. Passage between the America Dock and the Albert Dock

Widening and deepening

Contractor: Joint venture "AMAL" (**Van Laere N.V. — Jan De Nul N.V. — S.B.B.M. and Six Construct N.V.**)

4.3.1. *Existing situation*

The existing quay walls are all gravity walls of different types. Fig. 15 and fig. 16 show sections of the quay wall types used. These quay walls are mainly composed of masonry or of walls constructed with sunken metal caissons filled with "concrete" (trass, lime, chippings, etc.).

Some quay walls were founded on heavy quarry stone (with or without mortar).

The foundation depth of these quay walls is almost equal to the dock bottom.

The construction of the new passage for Panamax vessels not only requires widening the old channel but also deepening it by 5 to 6 m. Therefore the old quay walls have to be removed and replaced by new constructions. In some places the old quay walls can be incorporated in the new ones.

4.3.2. *New quay walls*

The following quay walls have to be constructed:

— the quay walls to be removed in the America Dock, channel and Albert Dock are replaced by a concrete quay wall construction composed of a continuous slurry wall covered by a concrete sill and floor slab anchored with a number of cast-in-situ tension and load bearing piles;

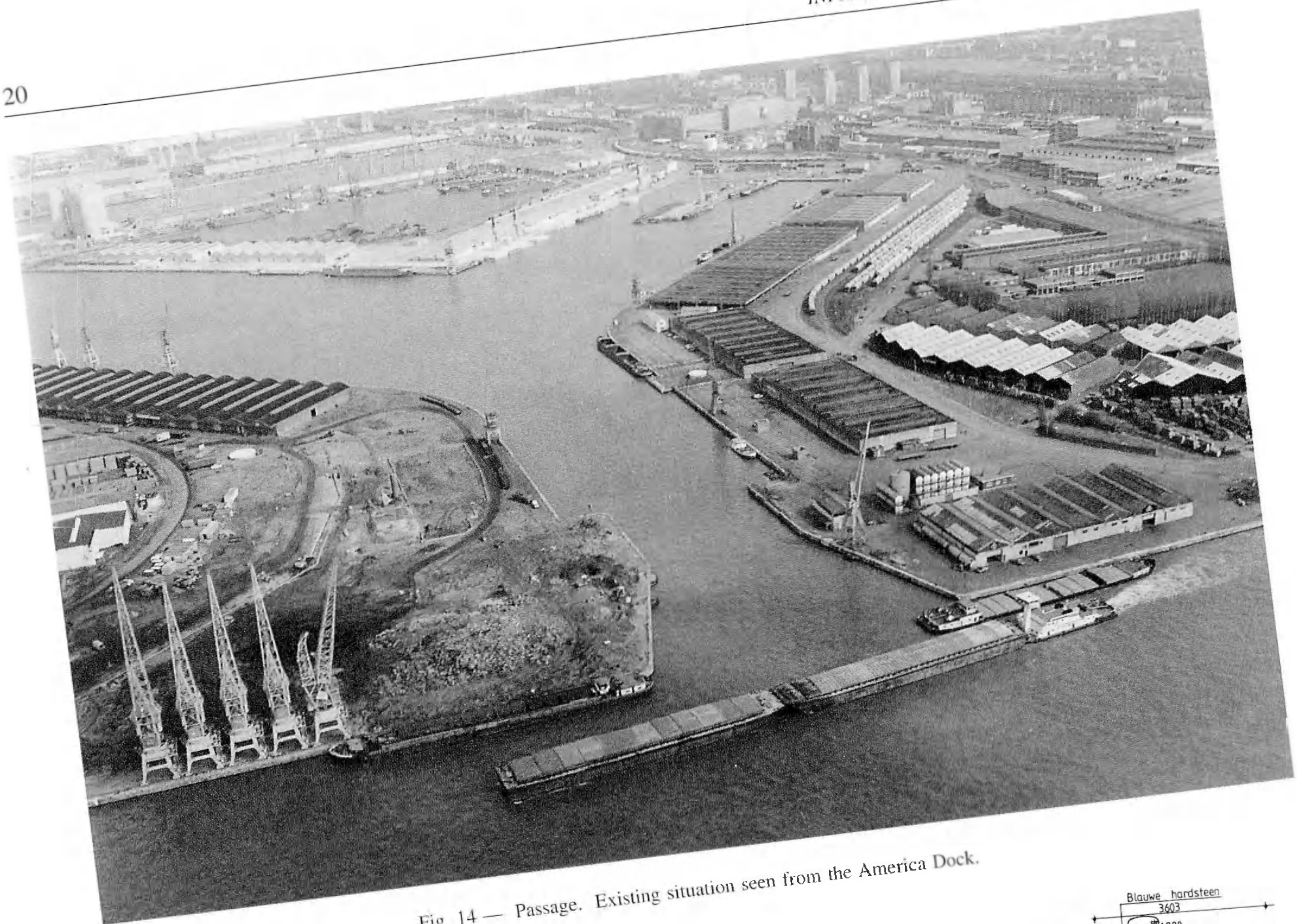

Fig. 14 — Passage. Existing situation seen from the America Dock.

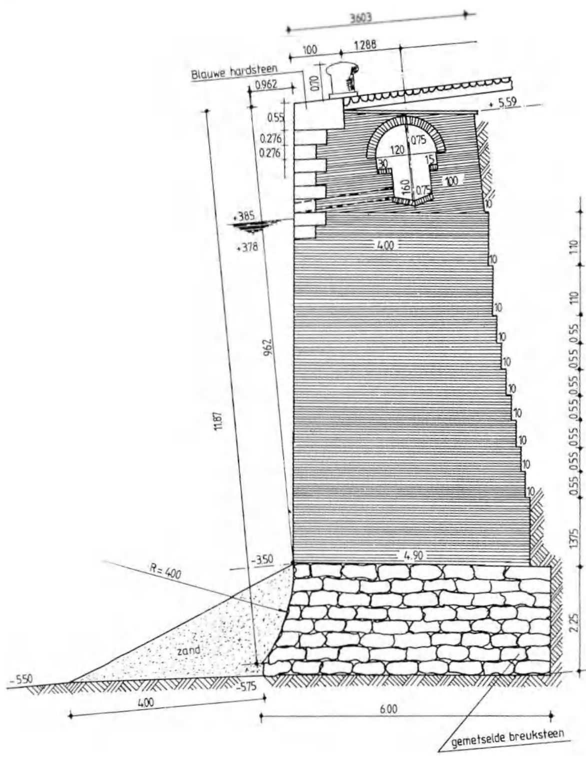

Fig. 15 — Existing quay wall.
Masonry

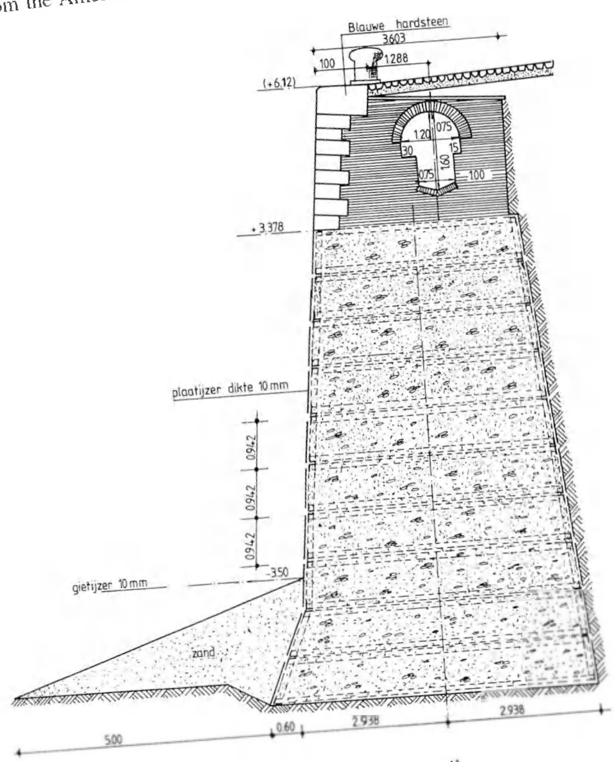

Fig. 16 — Existing quay wall.
Caisson type

Fig. 17 — Opposite bank. Demolition of the buildings above ground level.
Situation January 1992.
Situation August 1992.

— the connection of the new quay walls with the existing ones along the opposite banks in the America Dock and the Albert Dock is made by reinforcing and deepening the existing quay walls (each time over a length of 50 m).

This is done by underpinning these quay walls with piles using the VHP (very high pressure) grouting technique. On the dockside a double soilproof screen up to level TAW — 14.50 is placed and on the land side a single screen. Both screens are connected by means of lateral screens.

This application is an extrapolation of a test project that was successfully completed along the transverse quay wall in the 3rd Harbour Dock.
This reinforced and deepened quay wall is also covered by a concrete slab which connects the whole structure. At the same time the quay wall is anchored with a row of cast-in-situ tension and load bearing piles.

Quantities

The total quantity of the work to be completed is:

— Slurry walls : 700 running m with a depth of 24 m
— Pile rows : 260 running m with a depth of 22 m
— Sheet piling to separate 1st Harbour Dock : 180 m
— Slurry wall concrete : 17,000 m³
— Slurry wall reinforcement steel : 1,300,000 kg
— Superstructure concrete : 22,000 m³
— Superstructure reinforcement steel : 2,200,000 kg
— Total length tension and bearing piles : 34,000 m
— dredging works:
  — cutter-suction dredge : 800,000 m³
  — dipper dredge : 400,000 m³
  — hopper dredge : 600,000 m³
— demolition quay wall (concrete masonry)80,000 m³

4.3.3. *Demolition works.*

The demolition work consists mainly of breaking up and removing the existing quay walls and jetties with all their accessories.

Further, a quay wall previously blown up has to be removed from a trench in the America Dock.

At the same time various foundations, road pavings, sewerage pipes etc. have to be cleared away. Furthermore the contractor has to seal off all cut off pipes and sewerage pipes and to extend the drains. The passage is crossed by various utility pipes, sinkers and debris.

Since the pipes have been redirected through the service tunnel constructed in 1990, the debris etc. remaining in the channel has to be removed.

A number of sheds which lie within the perimeter of the new passageway also have to be entirely or partly dismantled.

In fig. 17 the state of the demolition work in August 1992 on the opposite bank is compared to the situation in January 1992 (before the works started).

Fig. 18 — Demolition of shed no. 105. Exterior wall.

The part demolition of shed no. 105, consisting of a number of galleries with shell constructions, was an especially delicate undertaking. Demolition was carried out up to a first row of reinforcing cross-beams, which were fitted into these shell constructions a number of years ago to enhance the stability of the whole structure. These cross-beams divide the adjacent shell roofs into a number of limited and independent shell sections.

After completing a number of core drillings in the existing shell roofs checking calculations, it was decided to fit a new reinforced concrete edge beam before starting the demolition work to absorb and spread the forces evenly over the cross-beams.
Because of safety considerations a similar edge beam was fitted near the cross-beams.
The new exterior wall is a brick wall.

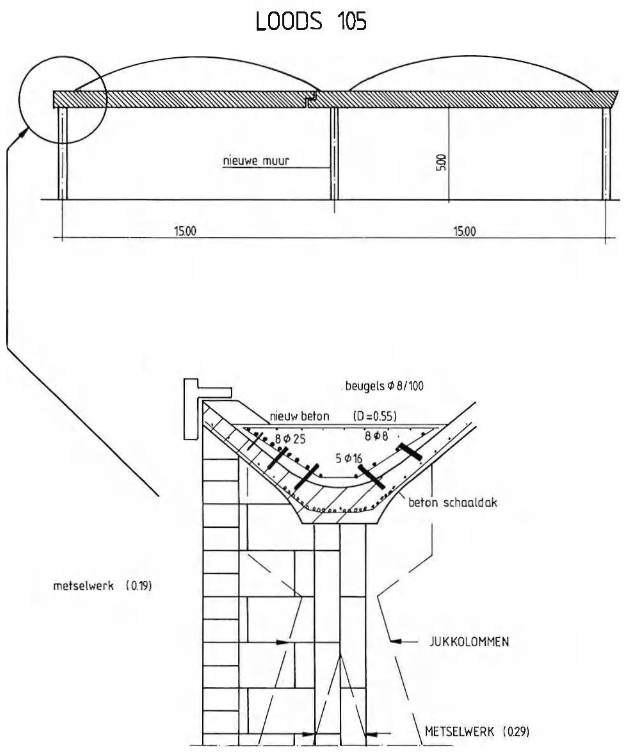

Fig. 19 — Shed at quay 105. Shell roofs — Fitting of edge beam.

The demolition is partly carried out with explosives. In the following paragraphs the applied method, the monitoring-network, the evaluation of the measurements etc. are described in more detail.

Recycling of old materials

The debris from the demolition work is largely recycled. The rubble to be recycled is separated from all foreign elements such as wood, reinforcing iron etc.

In accordance with the terms of reference the rubble is then crushed to different calibres for the construction of the base foundation of the quay platforms and roads. This is in accordance with draft directives for the processing of concrete and rubble for the construction of roads, stated in the draft of standard specifications no. 150 by the **Ministry of Public Works**.

The remainder of the debris is crushed for applications in hydraulic engineering works, as the calibre 2 to 50 kg is used e.g. to fill some deep bottom erosions at Container Quay South *(Europe Terminal)* near the Berendrecht Lock.

The rubble was dumped by the hopper barge MS. "Pompei", operated by a consortium of 4 hydraulic engineering contractors (**Herbosch-Kiere N.V.**, **Dredging International N.V.**, **J. De Nul N.V.** and **Decloedt N.V.**).

Monitoring of the explosion effects

The Joint Venture "AMAL" decided in favour of demolishing the existing quay walls by means of explosives. In this case the use of explosives is a time and cost effective method, particularly taking into account that approx. 1 km quay wall (80,000 m³ masonry and concrete) had to be demolished.

Demolition using explosives in a highly industrialised zone, however, requires certain precautions regarding effects on the surroundings, such as ground vibrations and projectiles.

The **Laboratory of the Royal Military Academy Architecture Faculty** was asked for advice. This lab had already carried out similar studies in the past in connection with the demolition of the quay wall at Berendrecht and the widening works of the Hansa Dock passageway.

The demolition study itself was done by **N.E.B. (Nobel Explosifs Belgique)**, the implementation by **Enterprises Jan De Nul N.V.**

Fig. 20 — MS. "Pompei" heading for Container Quay South.

A second problem facing the Architecture Faculty on this building site was the impact of the pile driving works on the surroundings during the construction of the new quay wall.

Follow-up and supervision of the dynamiting work

As already mentioned before, demolition with explosives requires certain precautions with respect to its impact on the surroundings.In order of importance these effects, as far as this building site is concerned, are ground vibrations and projectiles.

Because the quay wall is dynamited almost entirely under water, projectiles have not posed a problem so far.

Therefore, apart from creating a safety zone during the explosion, no special measures are taken.

The ground vibrations caused by the dynamiting are a more important problem, which is solved in a careful manner.

By making use of delayed-action explosions the ground vibrations can be reduced to a large extent. But in a complex environment it is not always possible to make concrete predictions without measurements.

For that reason four test explosions with a restricted demolishing effect were carried out to allow the contractor to optimize his explosion plan. These explosions took place at a relatively safe location.

Parallel to this an extensive vibration monitoring network was set up by N.E.B. over a wide surrounding area.

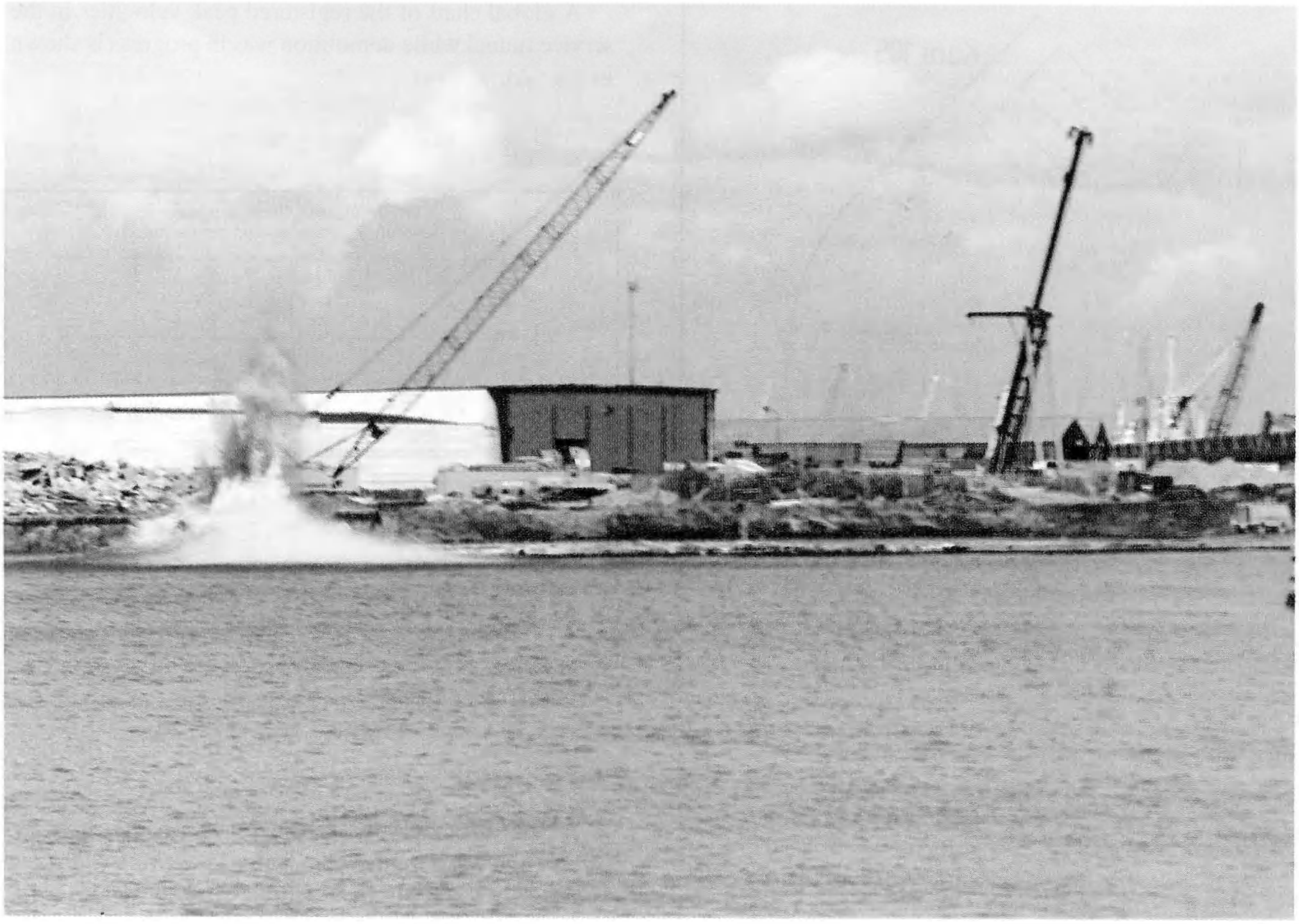

Fig. 21 — Dynamiting a massive quay wall.

The stations were selected in agreement with the enterprises in the neighbourhood and in consultation with the **Technical Service of the Antwerp Port Enterprise**. At the same time a good information campaign was launched informing all parties having plants in the vicinity of the site.

A survey of the site shows that there are three critical constructions in the immediate vicinity, two of which (service tunnel and shed no. 105) are situated on the opposite bank.

First there is the newly-built service tunnel under the passage. This tunnel lies 7 m under the quay wall to be demolished.

Calculations and monitoring carried out on the occasion of similar works in the Hansa Dock proved that normal limits for buildings are not applicable here and that velocities up to 250 mm/s measured on the tunnel wall are acceptable. (*Response to shock-wave of a microtunnel in soft ground*, **Legrand C., Vyncke J., Bourgois R., Thibaut W.**, proceedings *Tunnels et Micro-tunnels en terrain meublé*, Paris, 7-10/02/1989).

The second critical point is the sheds near berth no. 105. The stability of the shell roofs is uncertain. Following foreign directives an upper limit of 10 mm/s was set for the velocity measured on the shell roofs, for any component and for all frequencies.

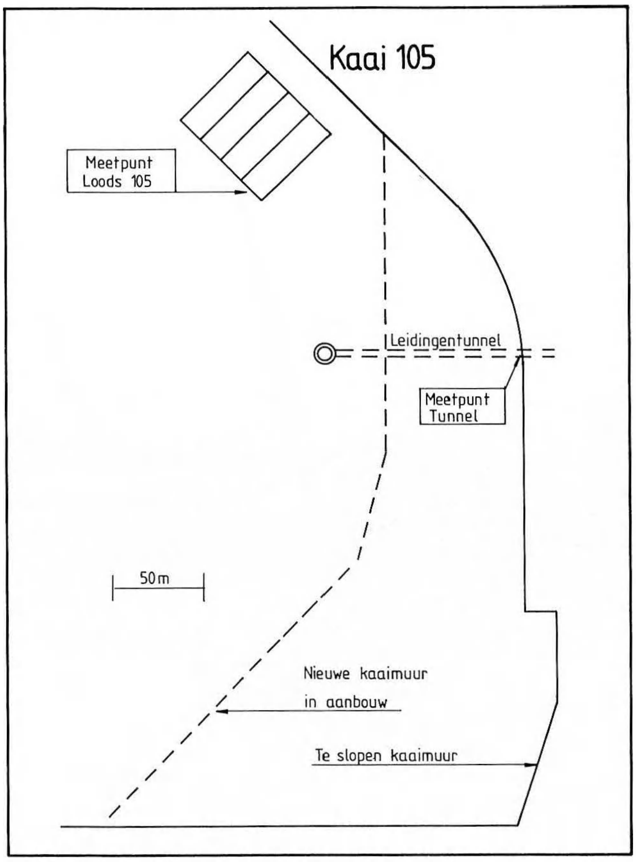

Fig. 22 — Stations on the opposite bank.

A third critical point is the grain silos of **Samga N.V.** at the South side of the America Dock.

At these checkpoints measurements were being carried out by the Architecture Faculty during the demolition work.

The evaluation after the test explosions allowed firstly, to set the optimal time of delay at 42 ms and secondly, to obtain indicative vibration values at the two above-mentioned locations which could be used to forecast further dynamiting.
The actual demolition activities started end of August 1992, whereby an average 30 m of quay wall was dynamited once a week.

The explosion schedule to demolish the quay wall sections adjacent to the service tunnel was set up, using the measurements already gathered on the tunnel wall and the prognoses based thereon.

A global chart of the registered peak velocities in the service tunnel while demolition was in progress is shown in Fig. 23.

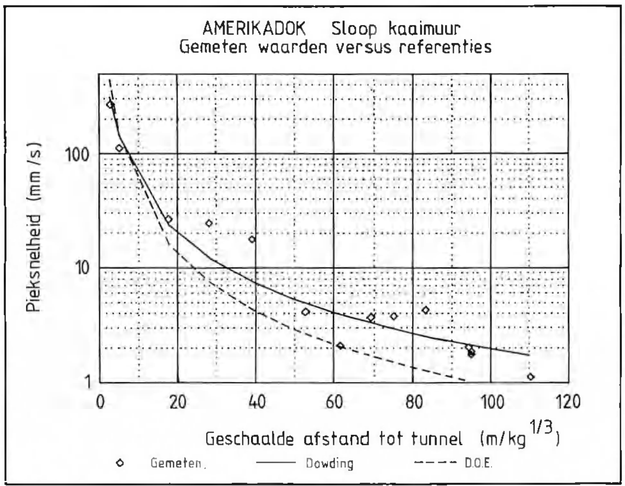

Fig. 23 — Global chart of registered peak velocities.

At the same time the measurements in the chart are compared to two empirical prediction laws (*Blast Vibration Monitoring and Control*, **Dowding Ch. H.**, Prentice Hall, 1985 and *A Manual for the Prediction of Blast and Fragment Loadings on Structures*, Department of Energy, USA, 1980), which are used for the prognosis.

Immediately above the tunnel a vertical peak velocity of 264 mm/s was registered at the time of the explosion, whereby no damage was ascertained.

The recorded signal, on which the exact time of the explosions delayed by milliseconds are clearly visible, is shown in the next chart.

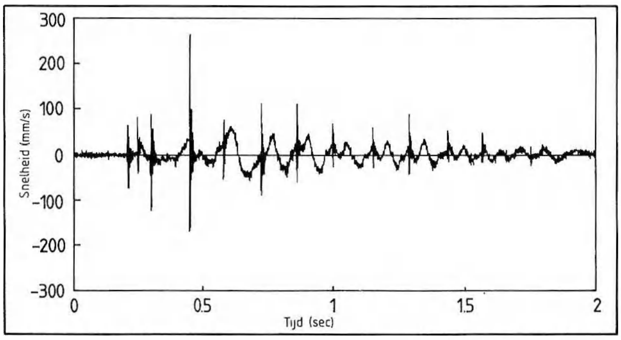

Fig. 24 — Recording velocities.

At this stage it is too early to draw conclusions with regard to the sheds at berth no. 105. On the basis of the collected data, however, an acceptable explosion plan will be attempted.

Monitoring of vibrations during pile driving activities

For the construction of the new quay platform, piles have to be driven over the whole length of the new quay wall. Here, also ground vibrations may be caused which can have adverse effects on surrounding constructions. Therefore measurements were taken at the above-mentioned checkpoints.

During pile driving in the immediate vicinity of the sheds at quay 105 the velocities measured on the nearest shell roof remained at all times below 0.5 mm/s. As a result no special measures were taken.

A more delicate problem was the service tunnel, where piles had to be driven within a few metres' distance. An alternative working method for this location was the use of screwed piles. To minimize this expensive and time consuming method a test campaign was organised.

At ever closer distances to the tunnel test piles were driven, while the vibrations on the tunnel wall were being recorded. For lack of concrete data in the literature, and bearing in mind the above-mentioned study, it was decided to stop pile driving at velocities of 40 mm/s, since this was not a one-off operation and since there was a real danger of fatigue.

Fig. 25 shows a pile row on which the measuring directions (Y-Z directions) in the service tunnel are indicated. The piles are rammed up to level TAW — 13.00. The upper side of the pipe lies at TAW — 13.47. The distances from the pile axis to the pipe axis are also indicated. When pile no. 10 was being driven at a distance of 5 m from the tunnel wall, a peak velocity of 15.4 mm/s was recorded.

As a result of this monitoring campaign the planned pile driving actions could be continued right up to the service tunnel without alterations.

Above and next to the tunnel vibrationless screwed piles were used.

The carefully planned measuring campaign, together with good co-operation between contractor and principal has allowed the delicate demolition with explosives to be finished on schedule.

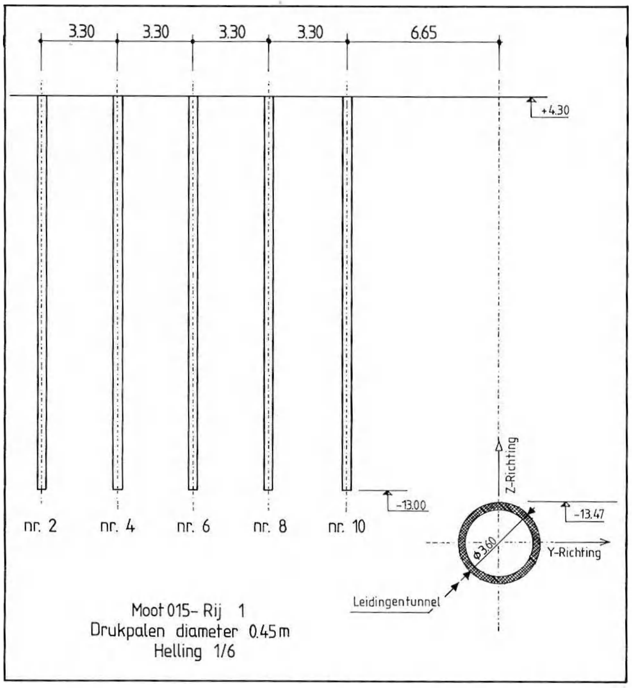

Fig. 25 — Situation of pile row near the service tunnel. Indication of measuring directions.

Vibration monitoring is of fundamental importance when faced with such problems.

The dismantling of the superstructures and sheds was subcontracted to **Van Kempen N.V.** and posed few problems with the exception of the part disassembly of the shell roof at berth no. 105.

4.3.4. *Design of the high-founded quay wall on wall in trench and on cast-in-situ piles*

Previous history and soil characteristics of the building site.

In view of the reclaiming of industrial areas and the construction of quay platforms, large amounts of soil have been dumped into the original low polders in the zones of the envisaged work. The density of these topsoils is very low.

Moreover the whole area is crossed by former water channels or moats.

The moats to the north of the America Dock, the Lefebvre Dock and the Straatsburg Dock as well as those around the Noordkasteel all belonged to the nineteenth century fortifications (Brialmont circle).

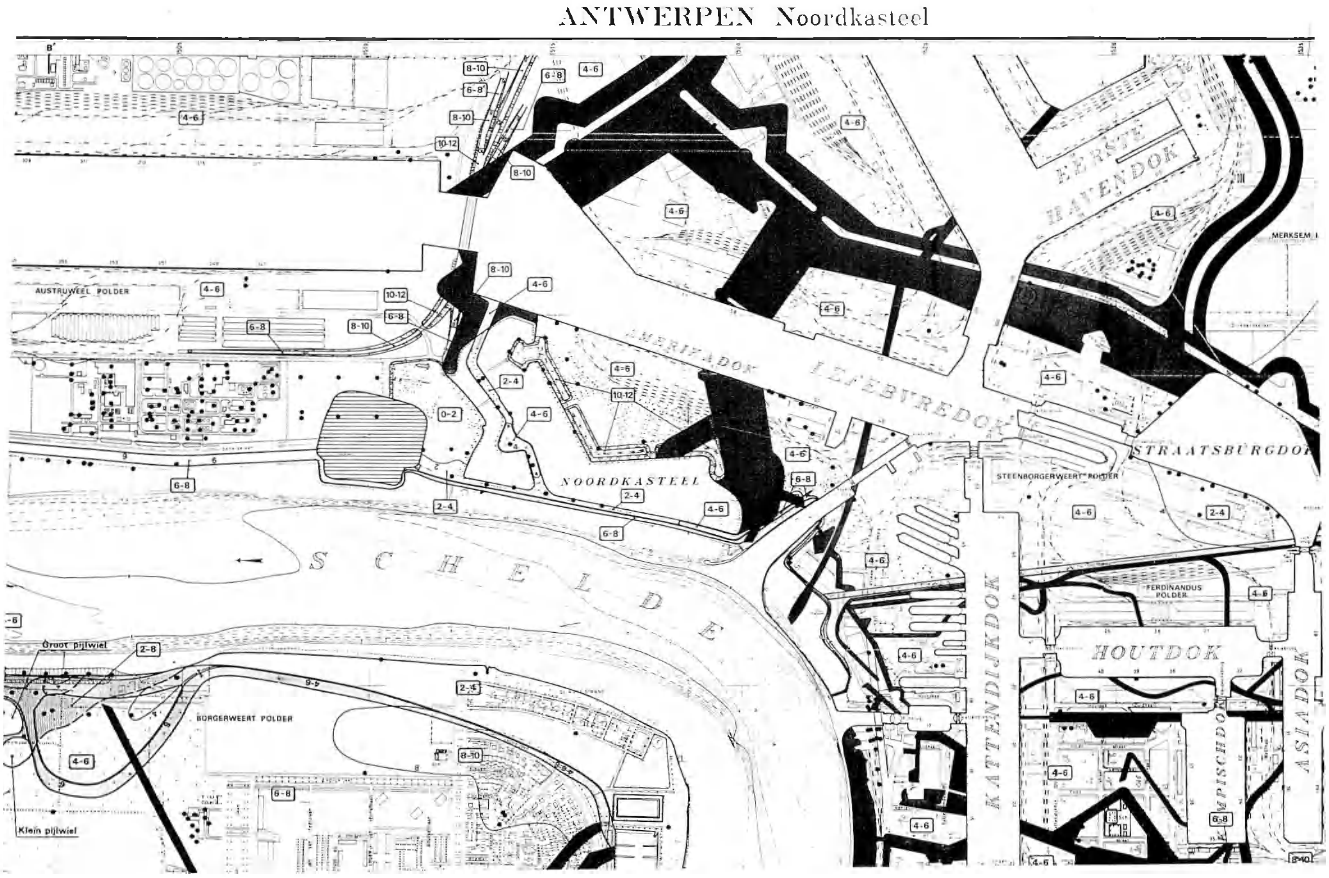

Fig. 26 — Antwerp Noordkasteel. Location of moats with respect to the passageway.

On top of this the Vosseschijn used to flow between the Steenborgerweert and the Austruweel polders. This water channel, which was connected to the Scheldt before the area was reclaimed, flowed into the river through several little gullies in the salt pasture.

These moats and water channels, now replenished, traverse the entire present construction zone.

The **Geotechnics Department** carried out an extensive study to be able to draw up the quay wall calculation sheet with soil characteristics which could be accounted for.

a) Opposite berth nos. (west side of the passageway)

On Fig. 27 the positioning of the soundings S6, S7 and S26 to S43 are indicated.

Besides the soundings S6 and S7 also the drillings B6 and B7 were carried out.

The cone penetrations of the soundings S26 to S43 are shown in chart 51. The area in question can be divided into two zones.

1st zone

Between starting section 01 (America Dock) and corner section 011 the following ground layers are found:

— from the ground level TAW + 6.20 to level TAW – 1.00 there are loosely compacted, heterogeneous filling soils;

— between TAW – 1.00 and TAW – 5.00: a quaternary layer of glauconite, i.e. sand mixed with peat, clay lumps and shell fragments;

— between level TAW – 5.00 and TAW – 17.00 there is a very compact sand layer;

— below level TAW – 17.00 there is Boomse clay.

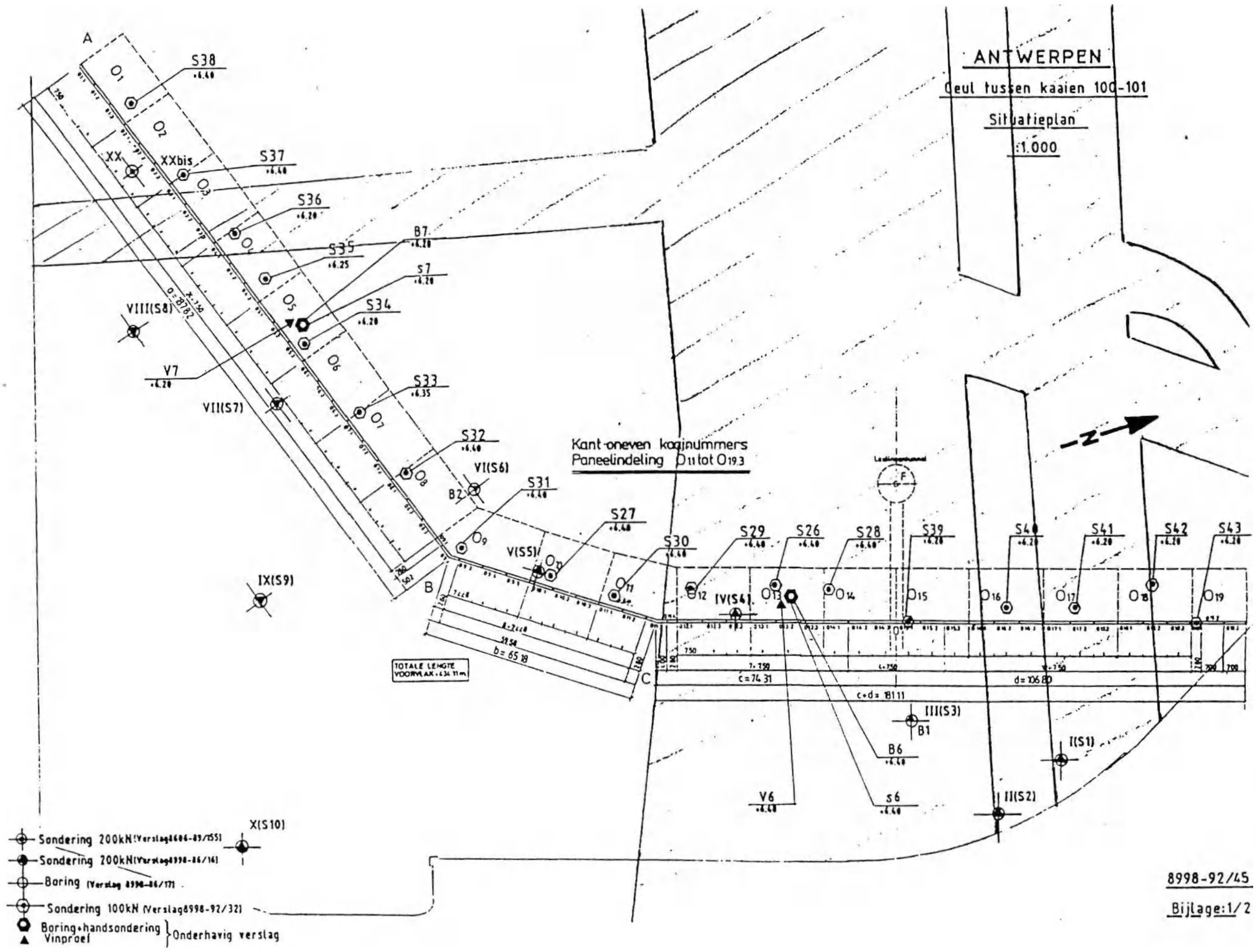

Fig. 27 — Positioning of soundings and drillings.

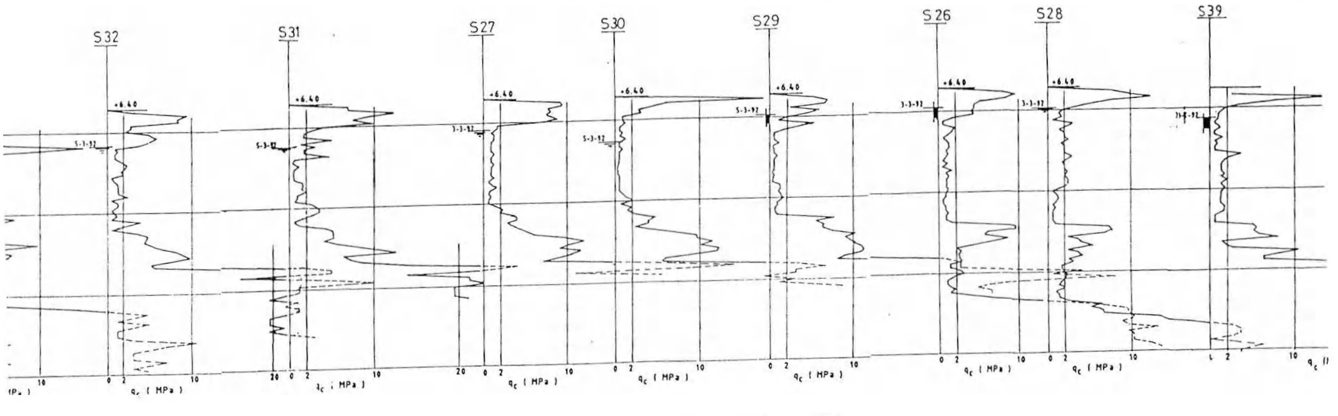

Fig. 28 — deep-soundings S26 — S31.

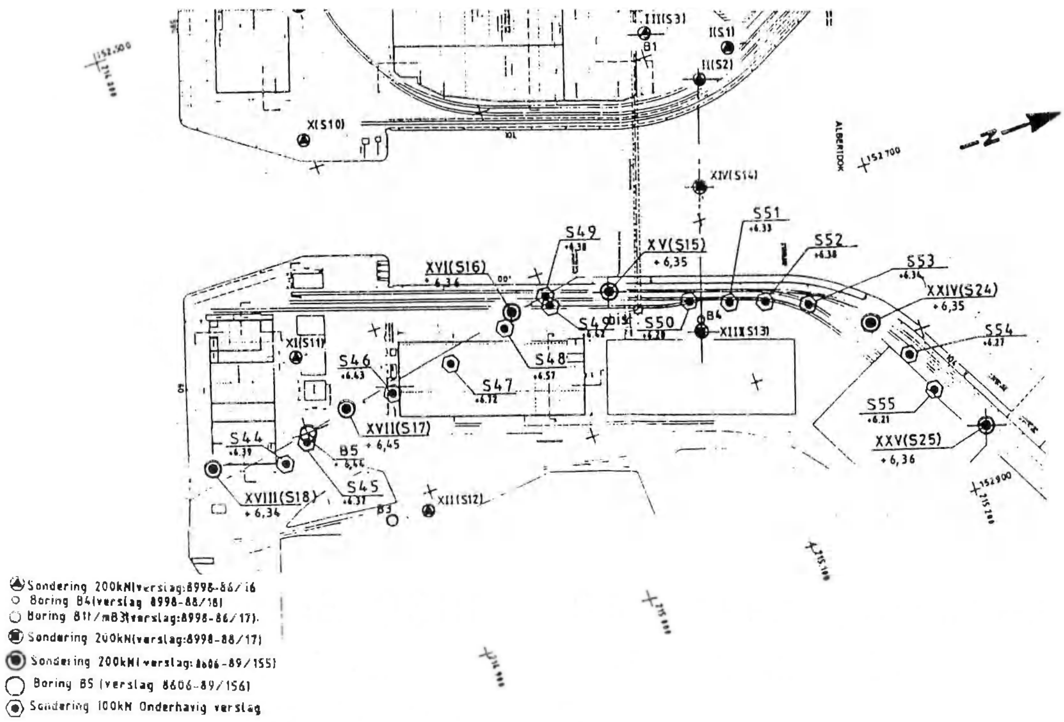

Fig. 29 — Indication of soundings and drillings.

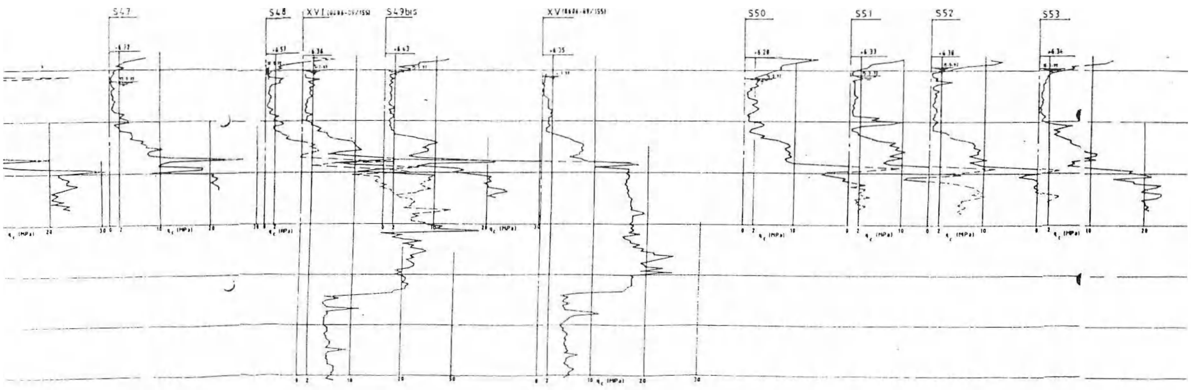

Fig. 30 — Deep-soundings opposite bank.

2nd zone

Between corner section 011 and end section 019 (Albert Dock) there are the following ground layers:

- from TAW +6.20 to TAW –5.00: see zone 1;
- between TAW –5.00 and TAW –8.00 there is a transistional layer composed of moderately compacted sandy clay with shell waste;
- below level TAW –8.00: see zone 1.

b) Even berth nos. (East side of passageway)

- on Fig. 29 the location of the soundings and drillings is indicated.
- Fig. 30 shows the cone-penetration values. From the groundlevel to TAW –2.00 there are loosely compacted, heterogeneous filling soils;
- between TAW –2.00 and TAW –4.00 there is a transitional layer composed of moderately compacted sandy clay with shell waste;
- between TAW –4.00 and TAW –17.00 there is a very compact sand layer;
- below level TAW –17.00 there is Boomse clay.

c) Fig. 31 (table) gives a survey of the soil characteristics which served as a basis for the calculations.

NIVEAU	ONEVEN KAAINUMMERS Zone met goede grond	ONEVEN KAAINUMMERS Zone met slechte grond	EVEN KAAINUMMERS Volledige zone
Maaiveld	Phi=20' Delta=0 K_{ah} = 0,50	Phi=20' Delta=0 K_{ah} = 0,50	Phi=20' Delta=0 K_{ah} = 0,50
(-2,00)			
(-3,00)			Phi=30' Delta=2/3Phi K_{ah} = 0,28
(-4,00) (-5,00)	Phi=25' Delta= 1/2 Phi K_{ah} = 0,36	Phi=30' Delta= 1/2 Phi K_{ah} = 0,29	
(-8,00)		Phi=25' Delta= 1/2 Phi K_{ah} = 0,36	
	Phi=34,2' Delta=1/2Phi K_{ah} = 0,243 K_{ph} = 3,65	Phi=34,2' Delta=1/2Phi K_{ah} = 0,243 K_{ph} = 3,65	Phi=34,2' Delta=2/3Phi K_{ah} = 0,23 K_{ph} = 3,65
(-17,00) (-20,00)	Phi=17,1' Delta=1/2Phi K_{ah} = 0,5 C = 20kN/m² K_{ph} = 1,46	Phi=17,1' Delta=1/2Phi K_{ah} = 0,5 C = 20kN/m² K_{ph} = 1,46	Phi=17,1' Delta=2/3Phi K_{ah} = 0,48 C = 20kN/m² K_{ph} = 1,46

Fig. 31.

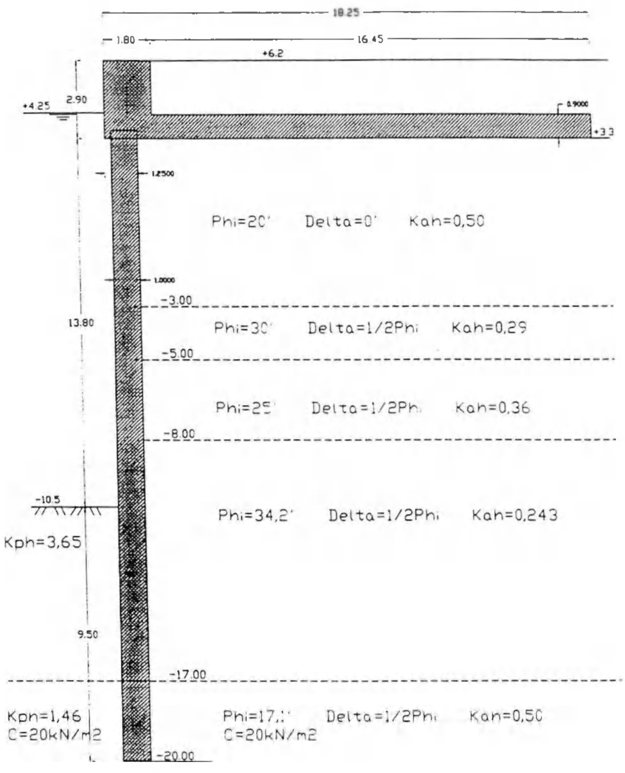

Fig. 32A — Quay wall on slurry wall — Calculation — General basic data.

d) On the passive ground pressure below TAW – 10.50 dockward a reduction factor s = *1,8* has been applied. This reduction factor was imposed in the terms of reference in order to reduce the dockward movements of the quay wall.

Loads to be observed:

— specific weight:

— soil above the p.s. (phreatic surface) :	18 kN/m³
— soil below the p.s.:	10 kN/m³
— concrete above the p.s.:	25 kN/m³
— concrete below the p.s.:	15 kN/m³

— overload on the quay platform : 40 kN/m³

— horizontal bollard force on the quay wall : 50 kN/m³

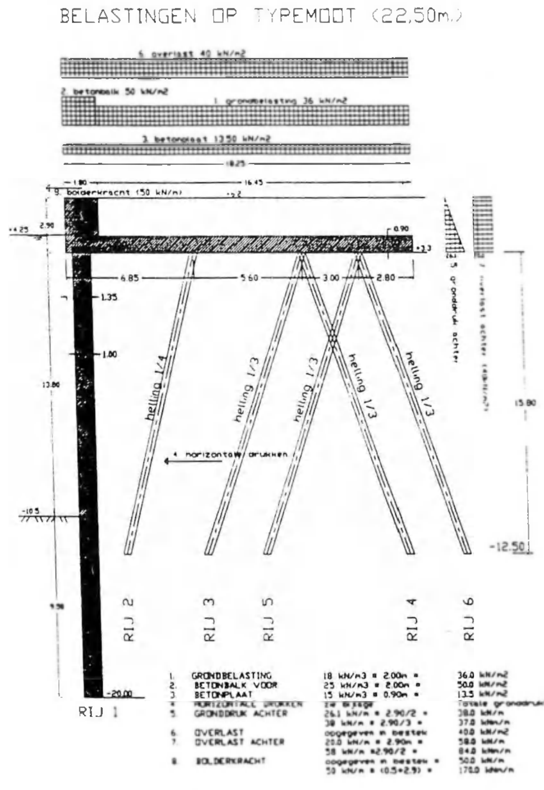

Fig. 32B — Quay wall on slurry wall — Loads on standard section (22.50 m).

Load on the slurry wall

The horizontal soil pressure on the landside is active ground pressure. The protecting effect of the top slab and the influence of the tension piles were taken into account.

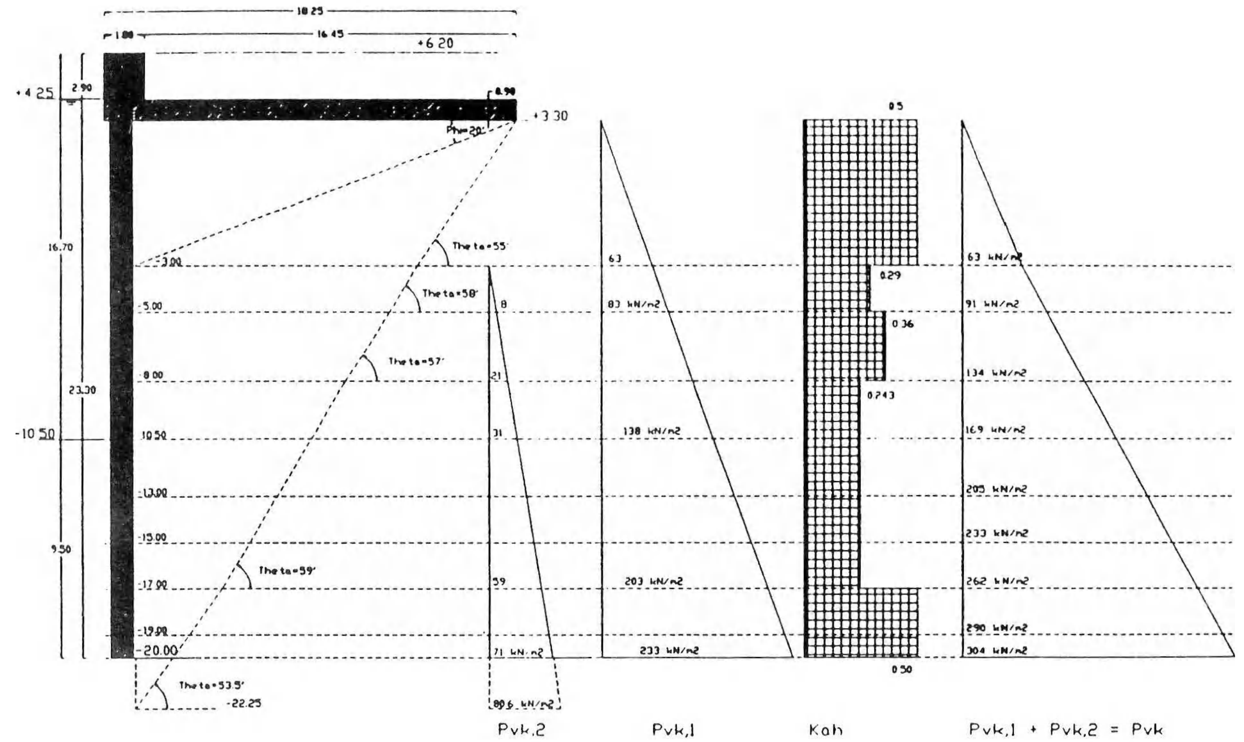

Fig. 33 — Curve of the vertical particle pressure.

The horizontal ground pressure on the dockside is a reduced passive ground pressure.

Below level TAW – 17.00 the cohesion of the Boomse clay and an effective water pressure of 0.50 m were also entered into the calculations.

By way of clarification the results of the ground pressure onto the slurry wall (opposite quay nos. zone 2) are indicated.

P_{vk1} vertical particle pressure under influence of the dead weight of the ground under the base of the superstructure (level TAW + 3.30)

P_{vk2} vertical protected particle pressure under influence of the overload and the dead weight of the ground behind the superstructure (level TAW + 3.30)

K_{ah} coefficient of the active ground pressure.

Calculation sheet for the slurry wall

The above diagram shows the slurry wall is flexibly held in the superstructure. The latter is a flexible slab on spring-mounted supports (the slurry wall and the piles).

The calculations for the whole structure were computerized. The flexibility of each part was taken into account.

On chart 36 the results of the calculations are presented in a graphic form.

The chemical concrete calculation is made for a concrete quality R_{wk} = 30 N/mm² and an admissible steel tension of 24 kN/cm².

The slurry wall is 1 m thick and the steel quantity in it amounts to 80 kg per m³ concrete.

A check calculation was made taking into account a possible erosion of the dock bottom up to level TAW – 12.00, whereby the reduction factor on the passive ground pressure was reduced to 1.4 instead of 1.8. The admissible steel tension is 32 kN/cm².

This control calculation led to the construction of the slurry wall up to level TAW – 20.00.

Calculation sheet for the pile foundation

The piles are cast in situ with a diameter of 45 cm. The pile tips reach the level TAW – 12.50 in the very hard and compact sand layer.

a) Tension piles

Axial reinforcement : 5 rods with a diameter of 20 mm, (1 % concrete section);

Transverse reinforcement : spiral diameter 8 mm, pitch 20 cm.

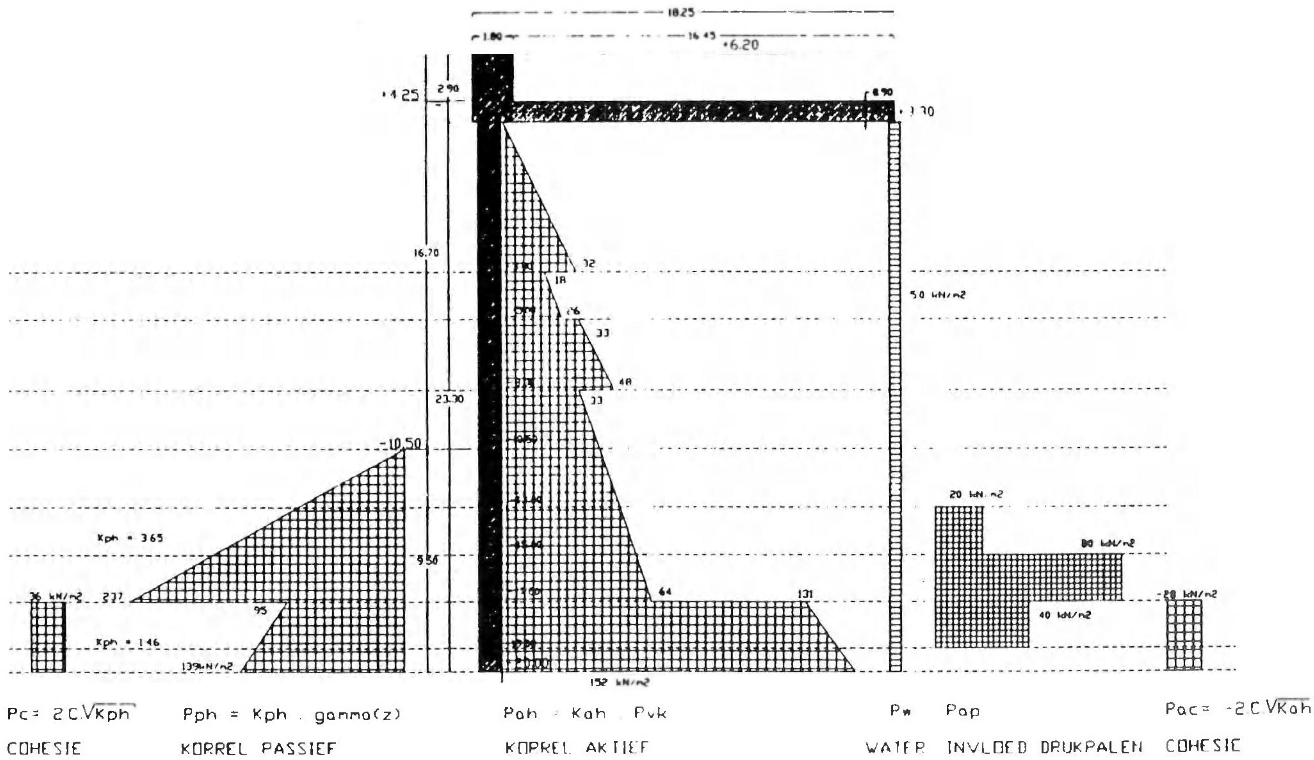

Fig. 34 — Load diagram on slurry wall (resulting pressure in kN/m³).

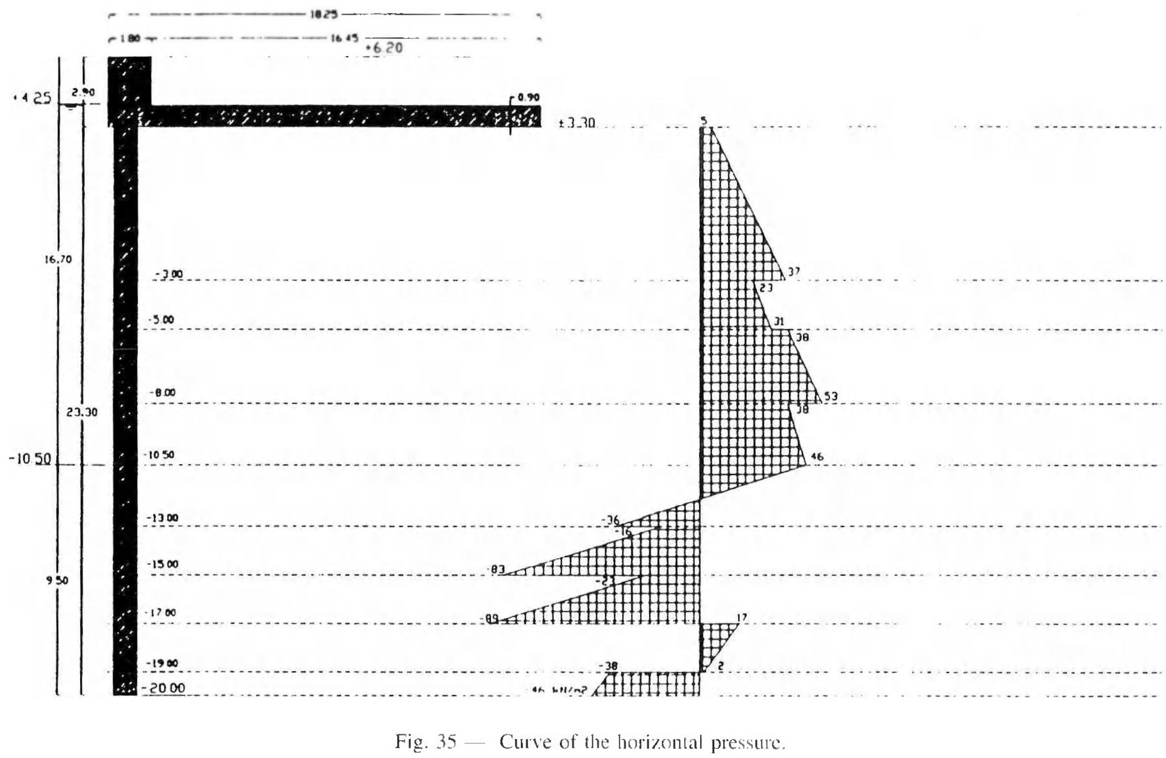

Fig. 35 — Curve of the horizontal pressure.

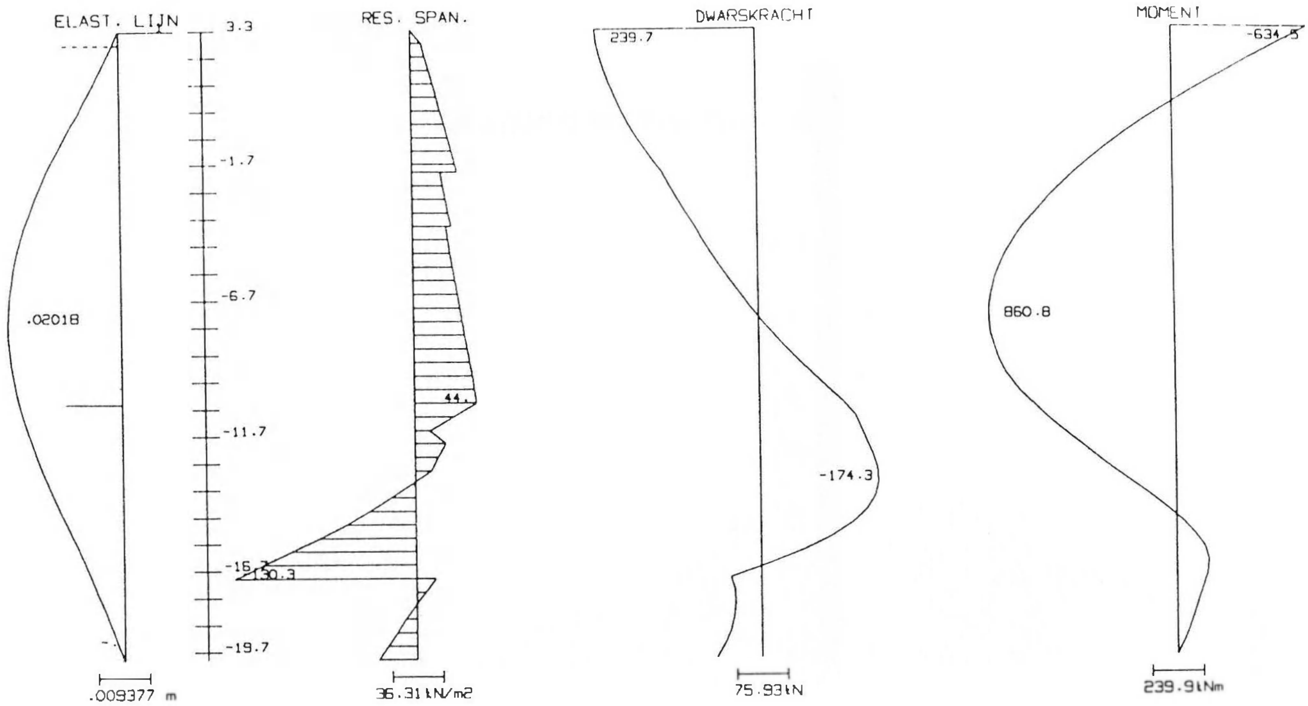

Fig. 36 — Graph of the calculation results.

The maximum tension force is 1.300 kN per pile. Of this 300 kN is absorbed by the axial reinforcement and 1.000 kN by the concrete section.

The tension force on the pile tip is 8.1 N/mm². Cone resistances measured are greater than 17 N/mm².

b) Load bearing piles

Axial reinforcement : 5 rods with a diameter of 25 mm for the whole pile length and 5 bars with a diameter of 32 mm in the upper 12 m. Transverse reinforcement: spiral 12 mm, pitch 20 cm.

This strong axial reinforcement (4 % concrete section) is necessary to absorb the bending moment that will occur when the soft top layer is compressed from above (40 kN/m²).

Insufficient flexibility in the tension piles would cause flaking of the concrete, leading to corrosion of the tensile reinforcement. This would have a serious adverse effect on the strength of the tension piles.

An interaction calculation proves that with a tensile force of 450 kN a bending moment of 150 kNm is still absorbed (BE 500 S steel).

.1 SLIBWAND

		ONEVEN		EVEN
		zone 1	zone 2	
Moment in steek	kNm/m	-209	-158	-350
Moment in veld	kNm/m	965	1087	845
Moment in inklemming	kNm/m	-886	-1008	-632
Normaalkracht	kN/m	288	268	322
Dwarskracht	kN/m	286	308	241

.2 PAALBELASTINGEN

		ONEVEN		EVEN
	aantal	zone 1	zone 2	
drukpaal 1ste juk	12	1208	1249	1160
drukpaal 2de juk	8	1270	1310	1161
drukpaal 3de juk	8	1277	1315	1195
trekpaal 2de juk	9	-429	-465	-362
trekpaal 3de juk	9	-415	-455	-333

.3 BOVENBOUW

	ONEVEN		EVEN	
	Me	N	Me	N
Moment onder				
Zone 1+2	1094	302	912	238
Zone 3	21	345	55	291
Zone 4+5	100	180	100	153
Moment boven				
Punt A	300	140	300	150
Punt B	300	140	248	150
Punt C	395	333	385	279
Punt D	431	180	431	153

Fig. 38 — Comparison of calculation results.

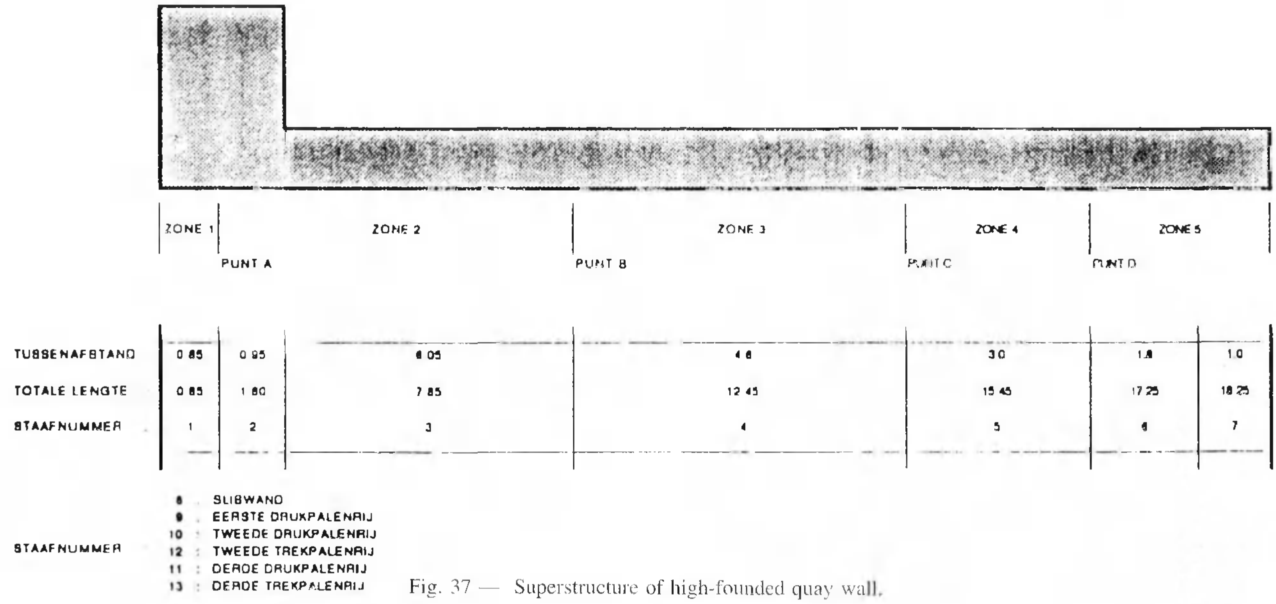

Fig. 37 — Superstructure of high-founded quay wall.

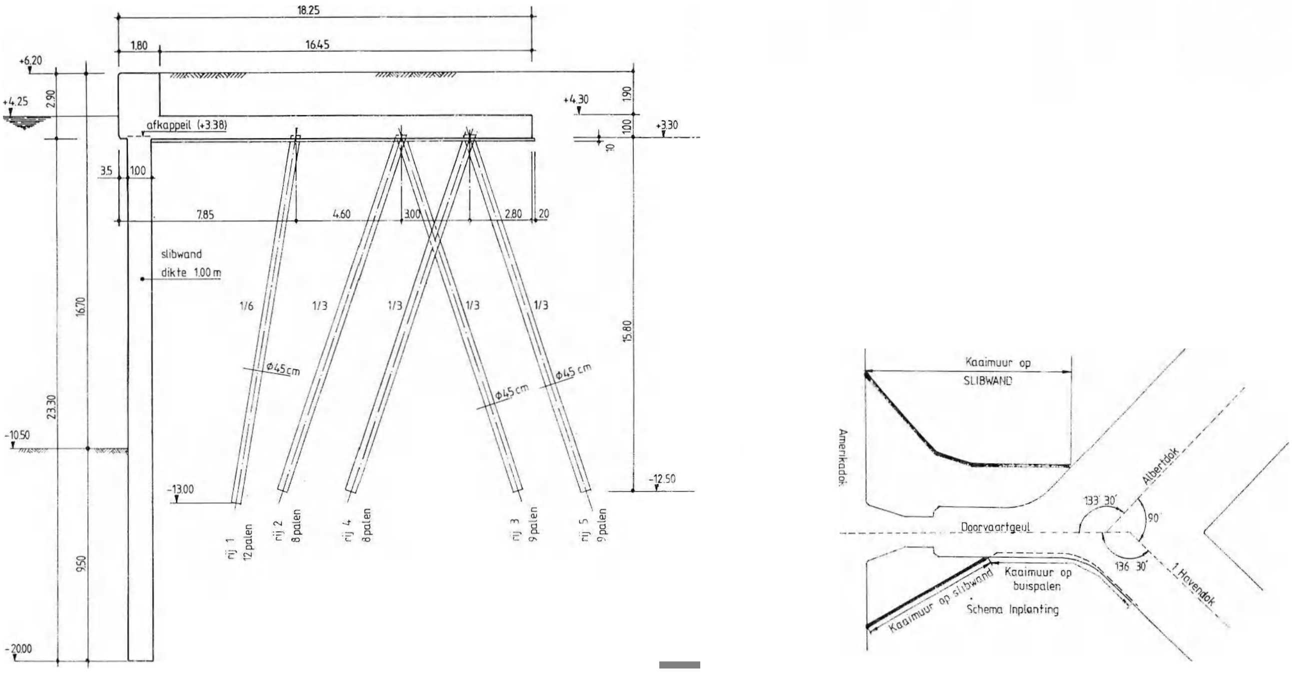

Fig. 39 — Cross-section of the slurry wall.

To determine the behaviour of the tension piles three test loads have been carried out. The test load, the set-up, the results and the calculations derived from it are further developed when the construction of the pile foundation is discussed below.

In any case the geotechnical strength of the piles, derived from these tests, is the key element for the stability of the high-founded quay wall.

Calculation sheet of the superstructure of the high-founded quay wall

For the strength assessment of the top slab a pressure of 60 kN/m² was entered.

In Fig. 37 the supports are marked with capital letters and the fields with figures.

Fig. 40 — Slurry wall construction — panel concreting. Background: excavation of panel.

Fig. 41 — Slurry wall construction — Excavation of trench with a grabcrane between guide walls.

The chemical concrete calculation is made for a slab thickness of 1 m. The lower reinforcement is concentrated in the fields between the slurry wall and the second pile trestle (for the opposite quay nos.: dia. 32 mm, pitch 11 cm; for the even quay nos.: dia. 32 mm, pitch 12.5 cm). The upper reinforcement is concentrated above the pile trestles (dia. 25 mm, pitch 15 cm). The steel quantity amounts to 100 kg/m³.

The results of the computer calculations are shown in table 38 for slurry wall, pile loads and superstructure.

4.3.5. *Construction of the slurry walls*

General

The quay walls with slurry walls are used at those locations where a new quay wall is projected behind the old quay wall which is to be demolished.

The slurry walls are constructed by the subcontractor **Bencor N.V.**

Between prefabricated guide walls and protected by a bentonite suspension a trench of the intended width and depth of TAW – 20.00 m is excavated, with an adapted excavator.
The guide walls are two reinforced concrete beams each 0.80 m high and 0.20 thick, with a clearance equal to the width of the slurry wall plus a few centimetres to facilitate the movement of the excavator.

The supporting liquid used in this project is bentonite sludge, prepared on the building site with bentonite powder, tap-water and aggregates.

The projected quay wall, a cross-section of which is shown, is constructed in an axial direction, in sections having a length of 22.50 m. Each section is composed of 3 slurry wall panels of 7.5 m length, 1 m thick; the foundation base lies at 26 m below groundlevel.

Since a slurry wall panel is usually longer (in our case 7.5 m) than the length of the grab used to excavate (2.8 m), a panel is excavated in two stages by the grabcrane, i.e. one stage at each end of the panel. Then the remainder, i.e. 1.9 m is excavated.

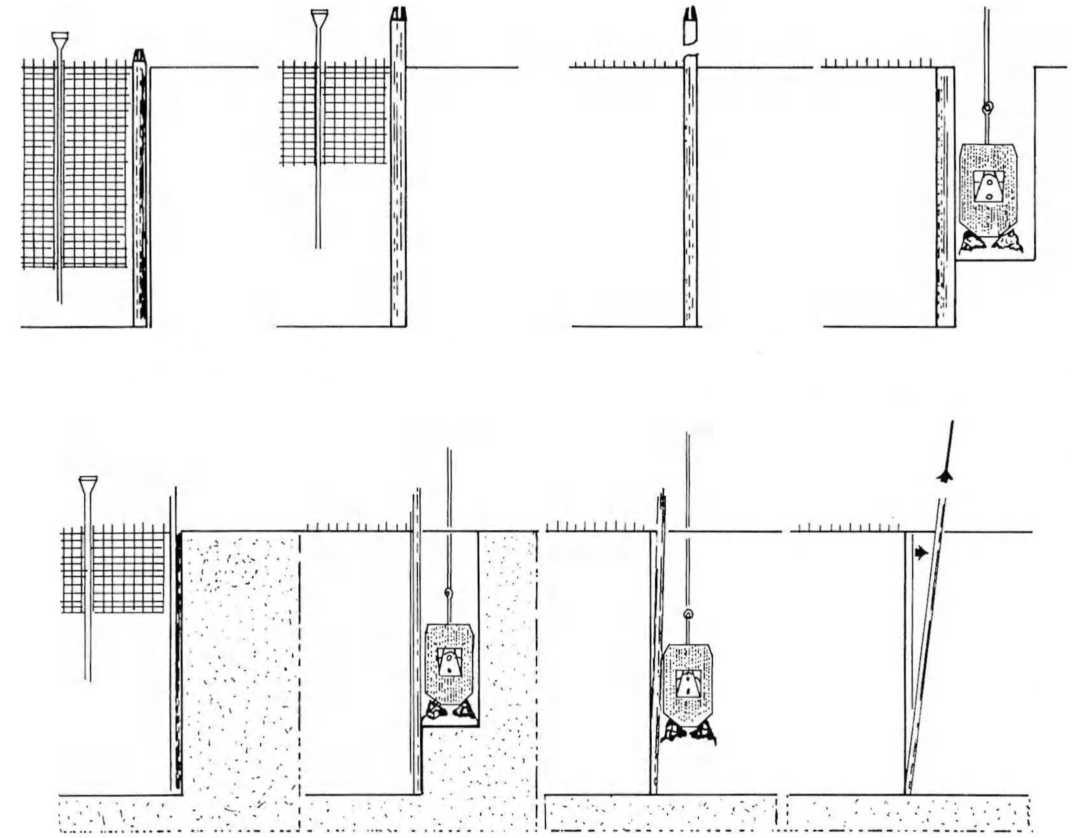

Fig. 42 — Principal design of the diaphragm wall construction.

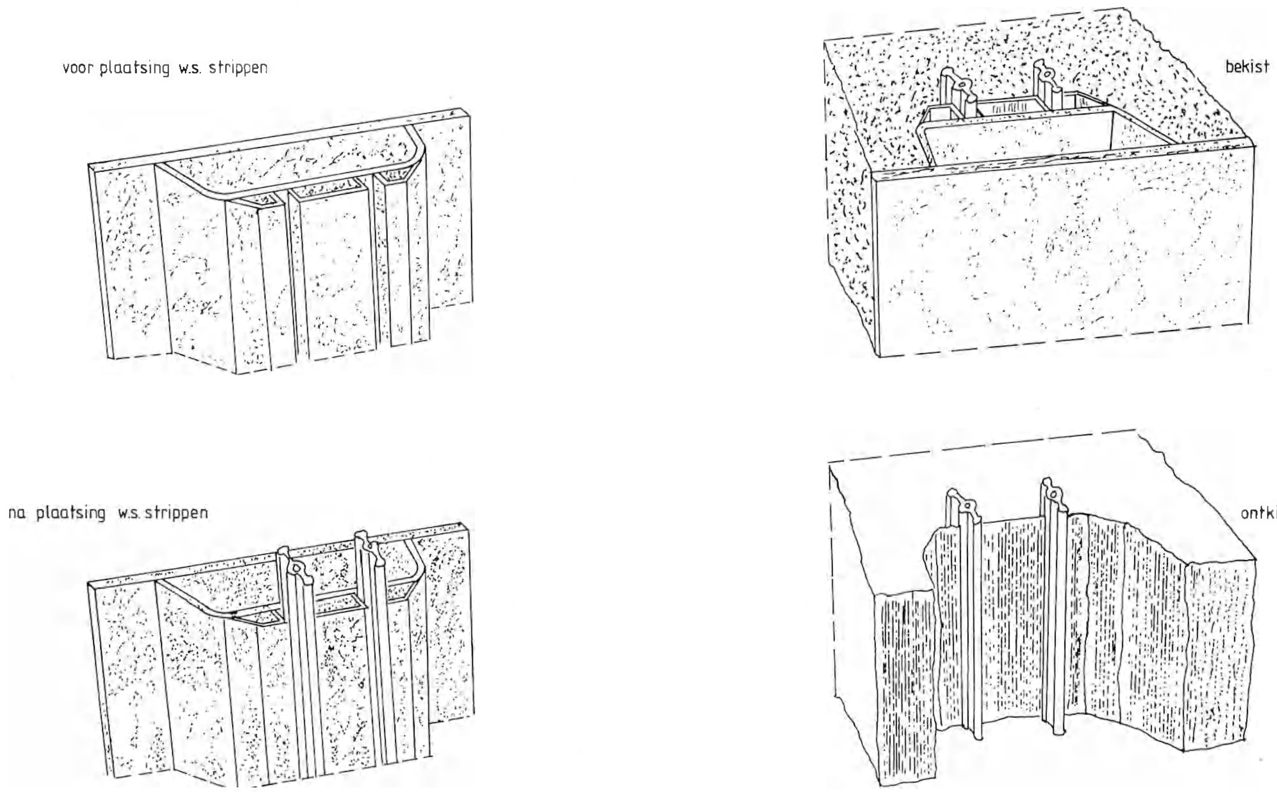

Fig. 43 — CWS joint system — Principle.

This configuration is selected to load the grab symmetrically during the excavation work so as to render the longitudinal excavation as vertical as possible. The suspension used during the construction of the slurry wall has a double function. Firstly it must form a impermeable foil on the sides of the trench, secondly it must support these sides.

The first requirement the liquid must meet is to prevent groundwater from mixing with the suspension, since this would form a watery solution in the trench. This would reduce the resistance of the excavated walls to crumbling and undermine the entire stability of the excavation, which could lead to collapse.

The impermeable skin on the sides of the excavation is formed sufficiently starting 2 to 2.5 below the top level of the suspension.

In the zone where the new quay walls for the widening and deepening of the passage between the America Dock and the Albert Dock have to be constructed, the water in the dock comes to approx. 2 m below groundlevel. At some locations along the new quay wall a groundwater level of barely 1.5 m below the groundlevel was measured. Basically the kind of soil to be excavated for the construction of the slurry wall readily allowed a stable excavation.

At certain places, however, the top 6 m was composed of mud and silt, originating from channels and gullies in the former mud-flats and saltings around the former "Noordkasteel" fortress, and from the uncompressed ground mixed with rubbish used to fill these gullies.
The groundwater level was quite high here, which sometimes caused the upper parts of the panels to collapse.

Infilling with stabilized sand, thick and thin concrete and more excavations made the construction of the slurry wall possible.

When the excavation of an entire panel is finished preparations are made to join it to the previous one already concreted.

In the case of diaphragm walls the joints between the different panels have to be watertight. The joint system used in this quay wall construction is the CWS-joint.

CWS joint system

The CWS-joint, which is protected by several patents and for which **Bencor N.V.** obtained the licence, brings both an elegant and effective solution to problems which are difficult to solve with other systems.

Operation of the CWS-joint

The joint is installed at each end of a panel and is pulled out after concreting like a sliding frame.
This method allows the combination of three functions which guarantee a better construction of the joints, namely:

- the concrete framing can take place independently from the site's organization and planning;
- the guide function is carried out in optimal circumstances;
- the watertight function is improved by fitting impermeable seals in the joints between the panels.

Stripping

The joint is not removed until the concrete has reached enough rigidity. On the contrary, the frame is left in place until the concrete is set and the next panel has been fully excavated.
After this the joint frame is extracted sideways mechanically, the grab being equipped with hooks. Thus the construction of the joint is totally independent from the circumstances in which the concreting is executed.

Excavating Guide

The CWS-frame is left in place while the next panel is being excavated. In this way it is used as a guide for the excavation equipment, thus ensuring the geometrical continuity of the successive panels of the wall.

Impermeability

During the excavation of the adjacent panel the frame protects the previous panel from being damaged. This improves alignment of the joints between the panels. Furthermore the system offers the undeniable advantage that additional seals can be installed in the joints.

After installing the frame and before introducing the reinforcement cage into the excavated panel, the bentonite of this panel, which has been contaminated with sand during the excavation work, is purified to an acceptable standard. This operation is necessary to prevent the sand in the suspension from settling out during the introduction of the cages and concrete chutes, which would cause an unstable sand layer at the base of the panel.

Fig. 44 — Bentonite containers.

The foundation base could be jeopardised in this way. Usually the bentonite is purified after the excavation and re-used as extra liquid during the concreting. This method, however, has disadvantages. As it takes several hours to process the bentonite repeatedly through the purification plant to reduce the sand concentration to an acceptable level of about 2 %, this operation is usually curtailed.

But incomplete purification is the result. To safeguard quality the standard purification principle was abandoned and replaced by substituting the retaining fluid. Two separate supplies of fluid are used, one for the excavating and another for the concreting. Between the excavation of one panel and the concreting of another, one bentonite is exchanged for the other.

Each kind has its own characteristics and must be used accordingly. The bentonite used for the excavation is contaminated by sand; incomplete purification poses no problem here. The fluid used for the concreting, however, is not contaminated with sand permitting correct placement of the concrete.

When this substitution operation is completed the reinforcement cages and the concrete chutes are fitted and the panel is filled with concrete following the underwater concreting method.

Fig. 45 — Construction of slurry wall.
Fitting of reinforcement cage in the excavated trench.

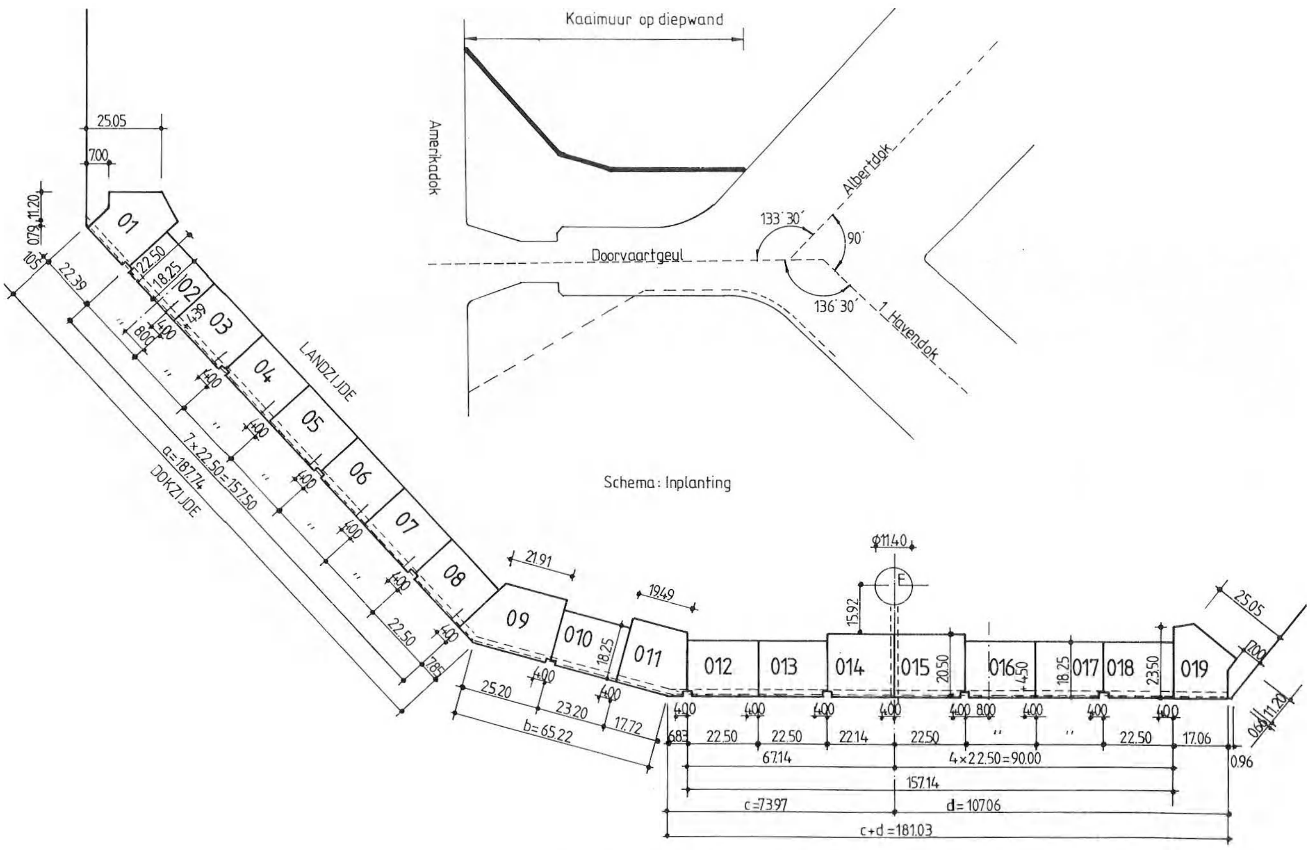

Fig. 46 — Sectioning on the opposite bank.

Single points : junctions — service tunnel

Drilling through the existing quay walls at both ends of the slurry wall. On each bank two junctions with different types of quay wall (combiwall) are projected. These junctions coincide with the crossing of the old quay wall with the new one. To this end the old quay wall is destroyed with heavy cutting heads protected by bentonite during the construction of the slurry wall panels.

The top of the service tunnel lies at level TAW – 13.45. Therefore a clearance of 60 cm is allowed between the slurry wall and the tunnel where the slurry wall panels are situated, next to and above this tunnel, so that no direct load is transferred onto the tunnel.

A standard section is 22.50 m long and is composed of 3 panels of 7.50 m each. The sections connected with the existing quay walls and the corner sections are of a different type. On the opposite bank the panels at both sides of the tunnel are sunk to level TAW – 24.00. The workpace was adjusted so that one panel of 7.5 m was excavated and filled with bentonite every day while another panel was being concreted.

Pile driving was carried out as far away from the excavated trench as possible, to minimize disturbance caused by shock-waves.

Three reinforcing cages were fitted in each panel. These cages arrived prefabricated at the construction site. Given the depth of the slurry wall, each cage was composed of two parts that were connected to each other on site.

Concrete

In order to comply with circular no. 225/910/31 of the **Ministry of the Flemish Community, Supporting Studies and Assignments Administration**, concerning alkali-silica reactions in concrete, the concrete composition listed in the terms of reference was altered.

The following composition was chosen :

— cement HL 30 :	320 kg
— marine sand 0/5	670 kg
— Scheldt river sand :	130 kg
— sea gravel 4/28	1,150 kg
— plasticiser :	3 l
— water :	100 l

Fig. 47 — Delivery of granulates.

Fig. 48 — Sieving plant.

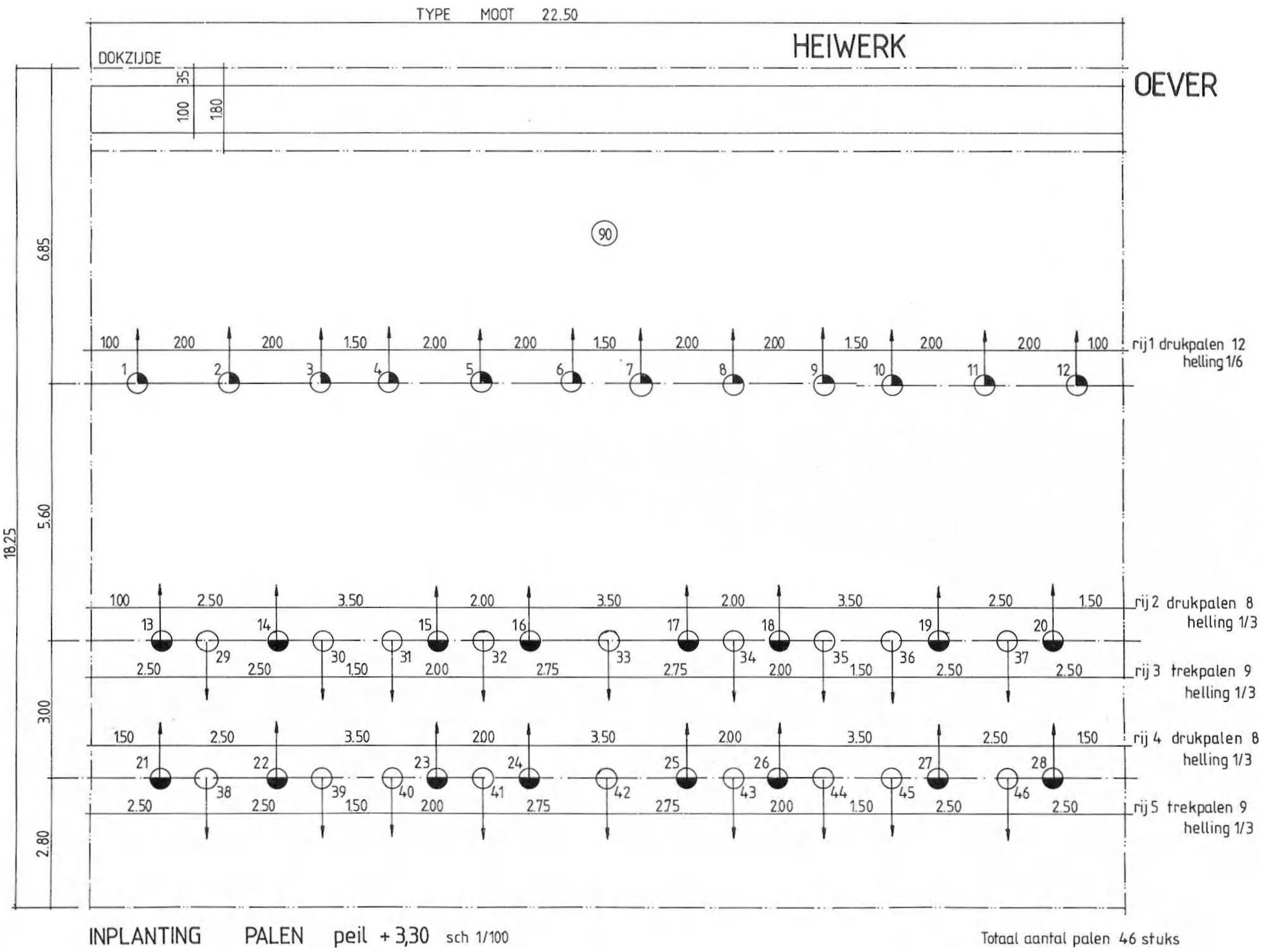

Fig. 49 — Pile configuration of a standard section of the slurry wall.

The concrete has an average strength of 40 N/mm² and a specific strength of 35 N/mm², after 28 days. Its W/C factor is 0.306.

The granulates are dredged at sea near the mouth of the river Thames and sieved and washed at the construction site. At delivery they are separated into three grades: sand 0/5, gravel 4/28 and coarse gravel. The latter is used in the drain structures behind the quay wall, and to replace the low density spoil between the combiwall and the existing quay wall.

The concrete is prepared on the construction site in the two plants there, with a capacity of 80 m³/h and 60 m³/h respectively.

4.3.6. *Construction of the pile foundation*

The piles are of the cast-in-situ type and are constructed by the subcontractor **Fundex N.V.**

Per standard section of 22.5 m (slurry wall type quay wall) 46 piles of 16 m were driven. There are 18 tension piles and 28 load bearing piles. The piles are 46 cm across.

In total 30 piles per 19.76 m section were used (quay wall of combiwall type together with existing quay wall), 14 of these being tension piles and 16 load bearing piles.

In the vicinity of the service tunnel a number of screw piles (26 in total) were installed.
It appears from the vibration measurements, however, that the number of drilled piles could have been reduced without any risk and one could have carried on using driven piles.

The reinforcement consists of a prefabricated BE 500 reinforcement cage.

There are two pile driving rigs on the construction site. Each piece of equipment rammed about 9 piles per day.

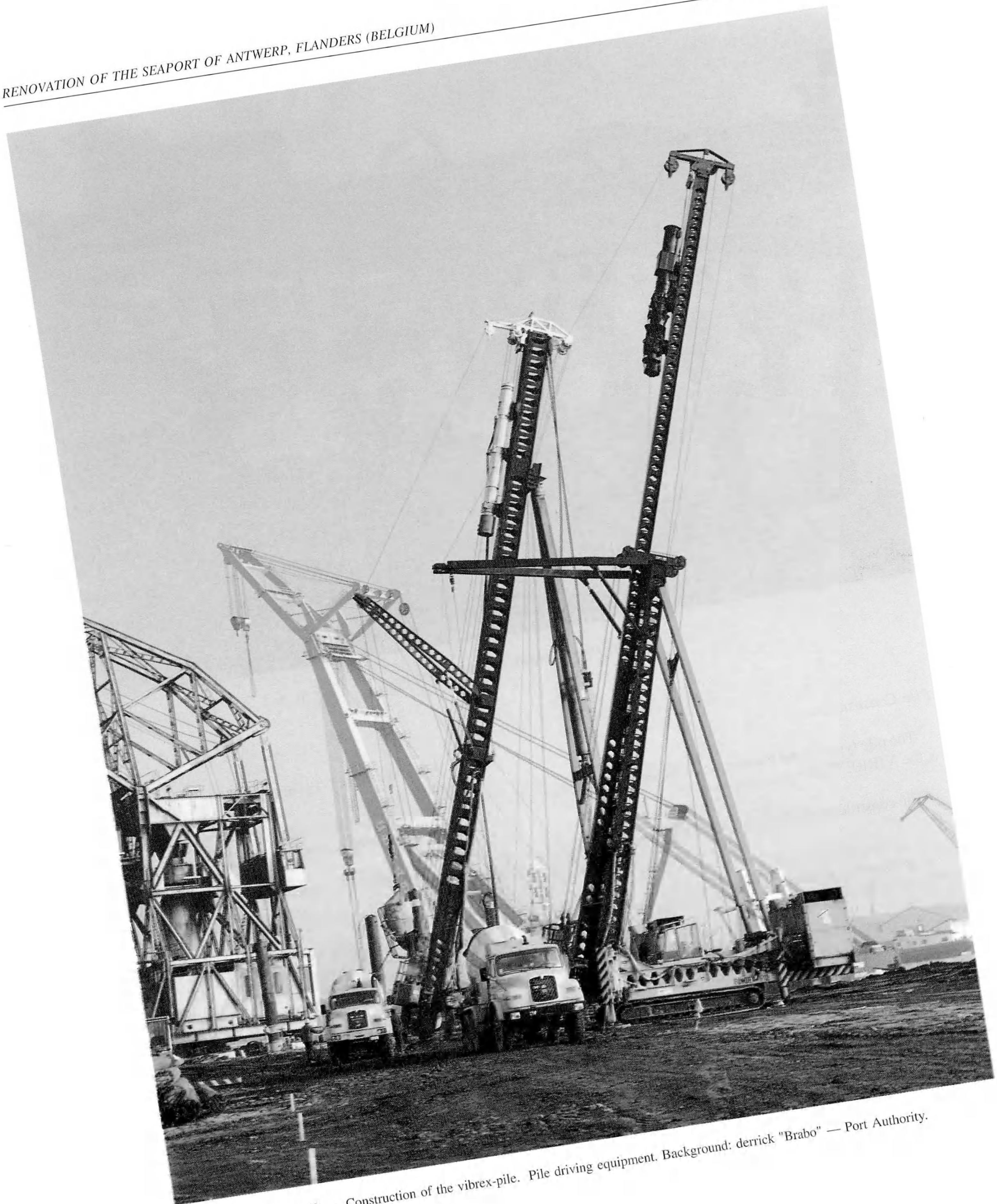

Fig. 50 — Construction of the vibrex-pile. Pile driving equipment. Background: derrick "Brabo" — Port Authority.

Fig. 51 — Pile heads after the excavation.

Pile Construction

The pile type for the foundation of the new quay walls is the VIBREX-pile: a cast-in-situ pile.

The construction of the pile takes place in three stages:

1) driving of the pile;
2) concreting and reinforcing;
3) forming of the pile.

Pile driving

Heavy driving could be expected, as soundings indicated. The tertiary Pliocene sand layers in which the piles had to be based, showed q_c-values of over 20 MPa over a considerable depth. Some soundings could not be continued because of *hard inclusions*.

Nevertheless a deep penetration in the foundation layer is required to resist the required tensile force.

For this heavy driving work the following equipment was used:
* an hydraulic pile hammer: IHC S70 and
* a diesel pile hammer DELMAG D36-32

The pile hammers are mounted on hydraulic pile driving rigs.

The base of the pile tube is closed off by a base plate fitted with a raised rim that glides over the pile tube. A compound is placed between the tube and this rim to ensure impermeability during pile driving.

The inclination of the piles is hydraulically adjusted by the machine and is manually verified for the chosen angle.

Concreting and reinforcing of the piles.

Once on the required depth the driving tube, completely empty and free of any impurity like water or soil, is filled with reinforcement and concrete, identical to the kind used for the slurry wall. Reinforcement was supplied to the site in prefabricated form. Since the foundation work consisted only of raked piles and the use of an auxiliary crane for the concreting and reinforcing would be neither cost nor time effective, a special jib was designed so that each machine could independently reinforce and concrete its own piles. The efficiency gained was considerable.

Forming of the piles

Hydraulic vibrators are used which are clamped around the driving tube. They completely encompass the pile tube used as a temporary formwork for the future concrete pile and remain in operation until the whole pile is made. The combined action of vibrating and lifting the vibrator in which the tube is clamped, results in the tube being extracted from the ground and the pile being formed.

Pile driving in the vicinity of the service tunnel

To avoid any damage to the service tunnel, a non-vibrating system was applied.

The pile driving rig was transformed into a drilling device with which Fundex screw piles were drilled.

These are non-vibrating, cast-in-situ, drilled screw piles, which press the soil aside. The drilling head remains in the ground and forms the pile foot. The resulting empty tube has a large cross-section allowing fitment of a reinforced cage comparable to the one used with the driven piles.

Because all the earth is pushed away and the pile can be screwed to the same depth, the load capacity of these piles is comparable to that of the cast-in-situ driven piles originally selected.

In this category of piles, namely those of the total soil displacement type, only this screw pile using drill tables with considerable torque can drill up to 46 cm diameter drilling tubes to the desired depth together with the fitting drillhead (and do so without help from earth moving Archimedean screws).

Pile reinforcement cages having a large external diameter can be placed in the drilling tube before the concreting is begun. The cages are placed over the whole pile length. Thanks to these characteristics this pile type can be used as a tension pile and can resist important horizontal forces and bending moments.

Static load test — Tension piles

In the projected quay walls the piles are subjected to considerable tensile forces. Based on the available geotechnical data it was impossible to accurately predict whether the demanded tensile force could be taken up by the piles in question or not. Therefore it was decided to execute the static load test(s) under tension as foreseen in the terms of reference.

In order to keep the test construction simple, four extra piles were driven for the load tests. These piles were fitted with special reinforcement cages to transfer the tensile forces onto the pile to be tested using the ground friction.

The load test equipment used and the test set-up

The test set-up is shown in fig. 52

To transfer the tensile force onto the pile we used 2 parallel hydraulic hollow jacks which were independently calibrated before use, together with the hydraulic pump and manometer. The calibration was carried out by the **MAGNEL laboratory** for reinforced concrete at the **University of Ghent**.

These hydraulic jacks were connected to a manometer from which the applied load could be read, making use of the rating table compiled during the calibration.

The measurements were taken by an independent surveyor: **BnS Engineering N.V.** at Zelzate.

The device used for the measurements is the precision instrument WILD N3. The invar-laths used were mounted directly onto the pile.

Method and description of the test

The pull load test was carried out in compliance with the German DIN standard 1054.

Determination of Q_g and Q_{max}.

Q_g = ultimate tension load of the pile or the tensile force at failure. Under this tension load the elevation of the pile is continuous.

There are two ways of determining this Q_g:

1) Q_g is the force whereby the elevation line resulting from the tension test with constant tensile force is almost vertical.

Fig. 52 — Pull load test — Test set-up.

2) If this vertical line is not reached, Q_g is considered as the tensile force corresponding to a resulting permanent elevation of 0.025 times the diameter of the pile.

Q_{max} = the maximum force with which can be pulled on the tension pile. This is determined by the test set-up and amounts to 1,200 kN (1,320 kN) in this case.

If Q_g is not reached during the tension test, then for this test $Q_g = Q_{max}$.

Conform with the specifications 1/3 of Q_g is an acceptable tensile load.

During the entire test the exerted tensile force is kept constant.

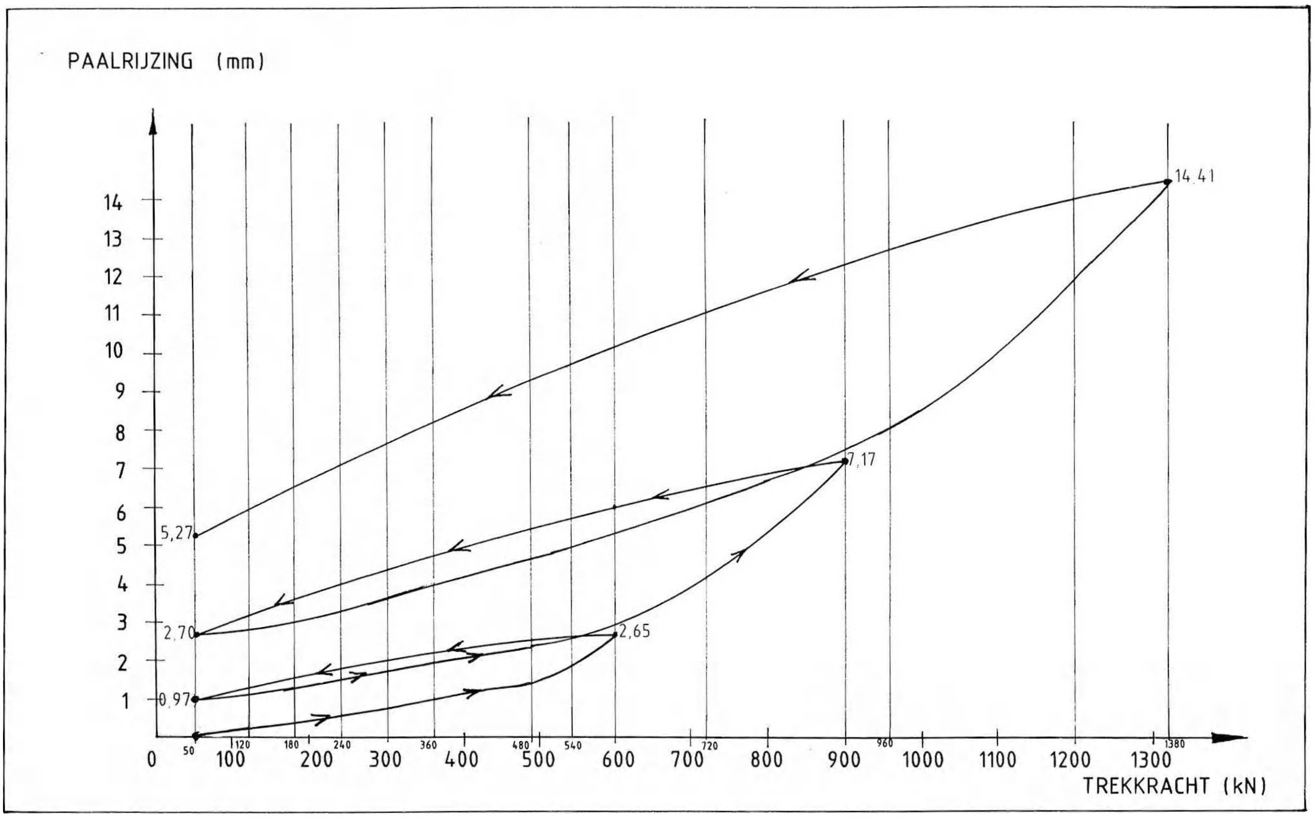

Fig. 53 — Pull load test on pile 3. Diagram tensile force/pile elevation.

Test procedure in function of the time

Loading stages

— The zero-measurement takes place at a tensile force of 50 kN, so that the load can be centered if necessary.

— The Q_{max} load will be transferred to the test pile in three stages, the so-called loading stages:
— 1st loading stage = 0.5 x Q_{max} = 600 kN
— 2nd loading stage = 0.75 x Q_{max} = 900 kN
— 3rd loading stage = Q_{max} = 1,200 kN

— Duration of each loading stage :
* first loading stage : the maximum load for this stage (600 kN) of 0.5 x Q_{max} will be reached after 1 hour in 5 stages of 12 minutes; this load will be kept constant for 20 minutes.

0 — 50 kN
1 — 120 kN
2 — 240 kN
3 — 360 kN
4 — 480 kN
5 — 600 kN

* second loading stage: the maximum load for this stage (900 kN) of 0.75 x Q_{max} will be reached after 2 hours in 5 stages of 24 minutes; this load will be kept constant for 1 hour.

0 — 50 kN
1 — 180 kN
2 — 360 kN
3 — 540 kN
4 — 720 kN
5 — 900 kN

* third loading stage: the maximum load for this stage (1,200 kN) of Q_{max} will be reached after 2 hours in 5 stages of 24 minutes; this load will be kept constant for 1 hour.

0 — 50 kN
1 — 240 kN
2 — 480 kN
3 — 720 kN
4 — 960 kN
5 — 1,200 kN
6 — 1,320 kN (extra stage)

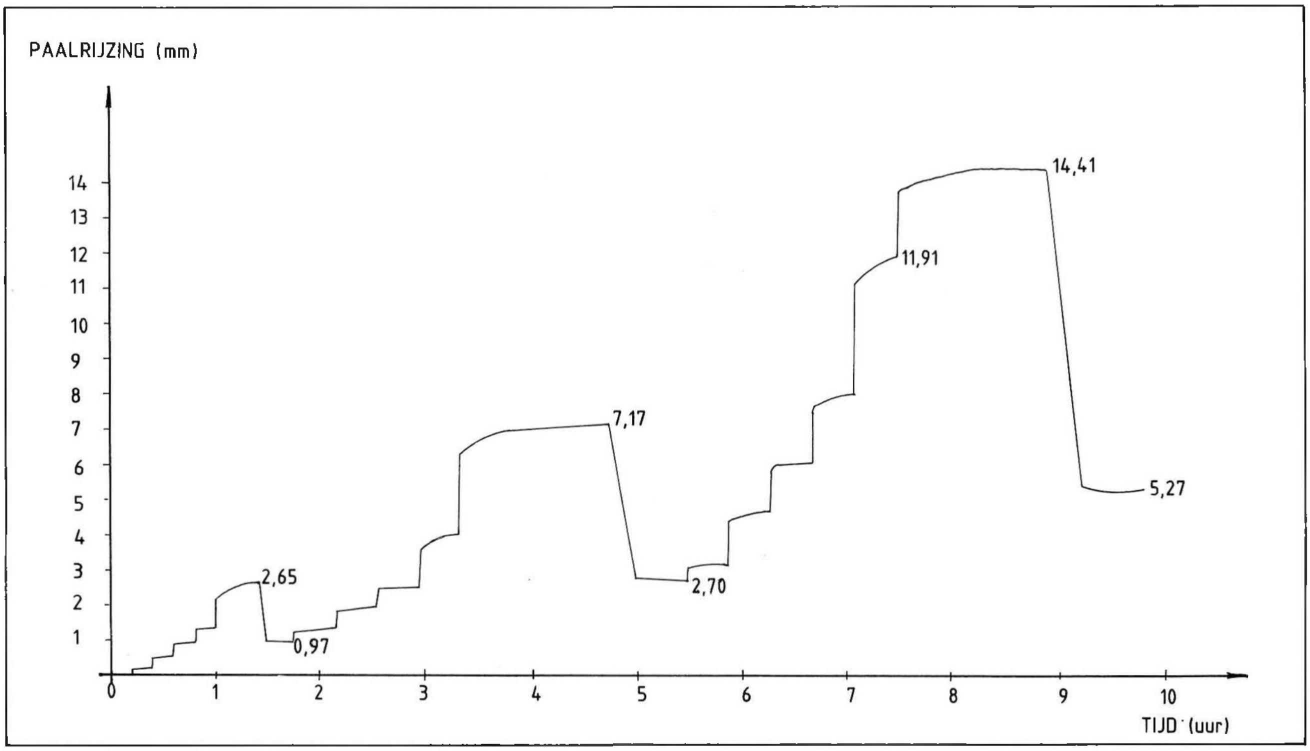

Fig. 54 — Pull load test on pile 3. Diagram : time/pile uplift.

Releasing stages

After every loading stage we proceed to release the tension.

The releasing stages are:

— after the 1st loading stage : 10 minutes
— after the 2nd loading stage: 15 minutes
— after the 3rd loading stage : 20 minutes

As soon as tension on the test pile has been released, every stage is kept constant for :

— after the 1st loading stage : 15 minutes
— after the 2nd loading stage : 30 minutes
— after the 3rd loading stage : 45 minutes

Note that released pile tension is equivalent to a remaining tensile force of 50 kN, so that the zero measurements and the reference measurements can be made at this remaining tension load of 50 kN.

The calculation of the tensile strength of the pile is as follows:

TABLE: measurements of tension tests

T	P1		P2		P3	
	S1	S2	S1	S2	S1	S2
600	2,650	970	2,630	760	2,170	880
900	7,170	2,700	8,190	2,500	6,750	2,130
1,200	14,410	5,270	13,230	3,700	—	—
1,320	—	—	—	—	12,210	3,550

P1, P2, P3 : numbering of the tension tests.
T : tensile force in kN
S1 : instantaneous rise in micrometer
S2 : rise after release (permanent deformation) in micrometer

According to DIN 1054 — 5.4.1.1. the failure load is reached when the permanent deformation is equal to:

$$S_{kr} = \text{diameter pile}/40$$

The pile diameter is 460 mm, therefore

$$S_{kr} = 460/40 = 11.5 \text{ mm}$$

To determine the failure load the linear relation of **Chin Fung Kee** is applied:

s/T = a + bs
s = rise in micrometer
T = tensile force in kN
a = constant in µ/kN
b = constant in 1/kN
T_{kr} = failure load according to DIN 1054
T_{toel} = $(T_{kr} - P)/3 + P/1.1$
T_{toel} : admissible tensile force in kN
P : effective weight of the tension pile in kN
3 and 1.1 : safety coefficients

Table : summary of results of tension tests

	a	1/b	T_{kr}	P	T_{toel}
	µ/kN	kN	kN	kN	kN
P1	1.25	2,200	1,775	41	610
P2	0.90	1,600	1,420	37	500
P3	1.25	1,800	1,500	37	520

From these tension tests we may conclude that the maximum tensile force (470 kN) is taken up with sufficient safety *(s = 3)* from the geotechnical point of view.

The geotechnical tension strength of these piles is the key element for the stability of a high-founded quay wall.

Construction of the superstructure: concrete slab and sill

The quantity of reinforced concrete for the superstructure of the quay wall amounts to 2,200 m³. The reinforcing iron to be processed totals 2,200,000 kg.

The top level of the piles lies at TAW + 3,38 m.

The top slab is 18.25 m wide and 1 m (quay wall on slurry wall) or 1.10 m (quay wall on combiwall) thick.

The header beam consists of a massive concrete block anchored to the top slab.

The sharp upper edge of the head piece is finished with a heavy metal coping profile.

Behind the header on the top slab a drain is projected to remove excess water under the quay platform paving to the dock.

Concerning the combiwall, the part above the water is concreted to reinforced prefabricated slabs. The front of the concrete header is prefabricated apron elements supported by reinforced prefabricated slabs.

Hence no shuttering is required above water.

4.3.7. *Design of a combiwall (even quay nos.) on Raymond cast-in-situ piles*

General information

The upper part of the quay wall is demolished to level TAW + 4.40.

The lower part of the existing gravity wall is not destroyed, but is bridged by the top slab of the new quay wall and horizontally anchored.

First the ground pressure on the covered existing quay wall is determined.

The horizontal retaining force at level TAW + 4.25 is equal to 50 kN/m.

Then the reactions on the base of the existing quay wall are estimated (V = 518 kN/m, u = 3.10 m, H = 233 kN/m).

These reactions are transferred to the combiwall.

The vertical reaction (V = 518 kN/m) is reduced by the active ground pressure coefficient.

The total horizontal reaction (H = 233 kN/m) is transferred to the combiwall.

The pressure behind the old quay wall at level TAW – 6.35 is, reduced by the active ground pressure coefficient, transferred to the lowest zone of the combiwall.

In fig. 60 the horizontal pressures originating from these different sources are shown separately.

For the passive ground pressure coefficient the absence of the partitions below level TAW – 13.50 has been taken into account.

In Fig. 61 the calculation results are presented graphically.

Fig. 55A — Reinforcement and shuttering of the top slab (standard section).

Fig. 55B — Reinforcement and shuttering of the top slab (corner section).

Fig. 56 — Quay wall section after being concreted.

Combiwall

The combiwall has a hinged bearing to the superstructure. The retaining force of 75 kN/m is taken up by the superstructure. In the combiwall there is a maximum flexibility of 810 kN/m.

The combiwall is comprised of the principal elements, tubes with an external diameter of 1,220 mm, a thickness of 14.3 mm and a length of 22.80 m. The steel quality is X65 (AE 450). The profile of the partition sheets is PU 12 with a length of 10 m. The distance between the tubes is 2.47 m.

The characteristics of the combiwall are the following:

— drag value : $W = 6{,}704$ cm³/m
— cross-section of the tube : $A = 541.7$ cm²
— radius of the tube: $i = 42.6$ cm

Taking the flexibility into account, the admissible steel tension $\sigma_{toel} = 24$ kN/cm². The slimness of the tubes and the second-order effect were also considered when checking the tension.

The maximum tension $\sigma_{max} = 17$ kN/cm².

To transmit the vertical load exerted on a tube (2,000 kN) to the ground (Boomse clay), the upper 10 m of the tube is filled with concrete.

Piles

All piles have been driven in at an inclination of 1/4 and grouped in pairs. There are 30 piles per section of 19.76 m.

The maximum bearing force is 1,230 kN per pile and the maximum tensile force is 380 kN per pile.

The top slab of this type of quay wall has width of 18.25 m and a thickness of 1.10 m. The maximum overload is 60 kN/m².

The maximum field moment is 1,600 kN/m, the support moment above the tube pile is 430 kNm/m and above the harnessed piles 360 kNm/m.

The base reinforcements consist of two layers of 32 mm diameter rods 19 cm apart. The upper reinforcement on the harnessed piles and the tubular piles consists of 25 mm diameter rods spaced 17 cm apart.

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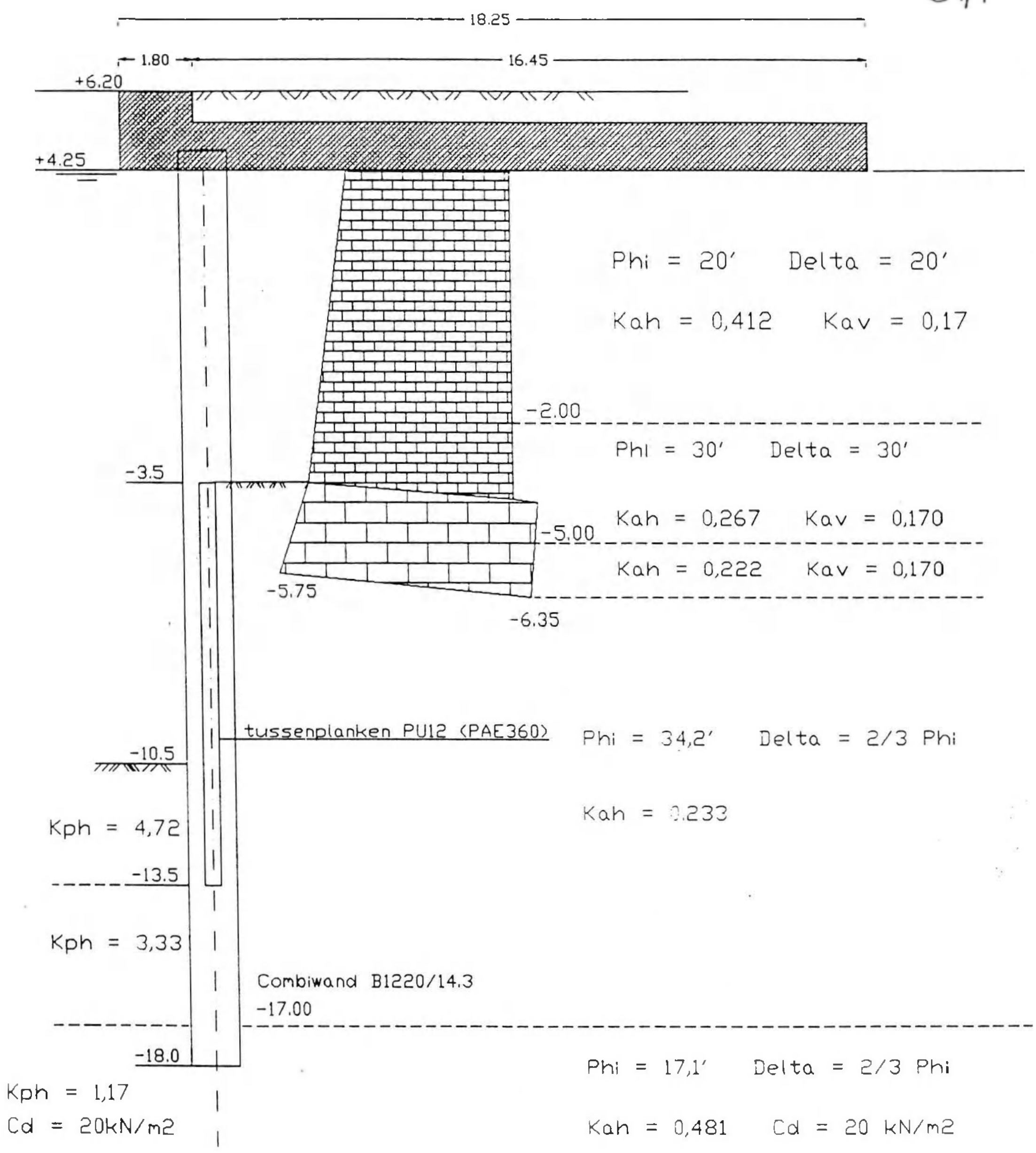


Fig. 57 — Calculation of quay wall with combiwall. Basic data.

4.3.8. *Construction of the combiwall (even nos.)*

Tubular piles — Partition boards

The combiwall consists of metal tubular piles. Between these piles 10 m long partition boards are driven from level TAW – 3.50 to TAW – 13.50.

The 22.80 m piles reach TAW – 18.00.

The axis of the partition wall tubes is 3.5 m in front of the existing quay wall. The partition piles are filled with concrete in order to increase the vertical load bearing capacity of the construction.

Between the existing quay wall and the partition piling a stone/asphalt protection shield for the sea-bed is fitted.

The combiwall is connected with the previously completed quay wall (i.e. at the time of the construction of the service tunnel).

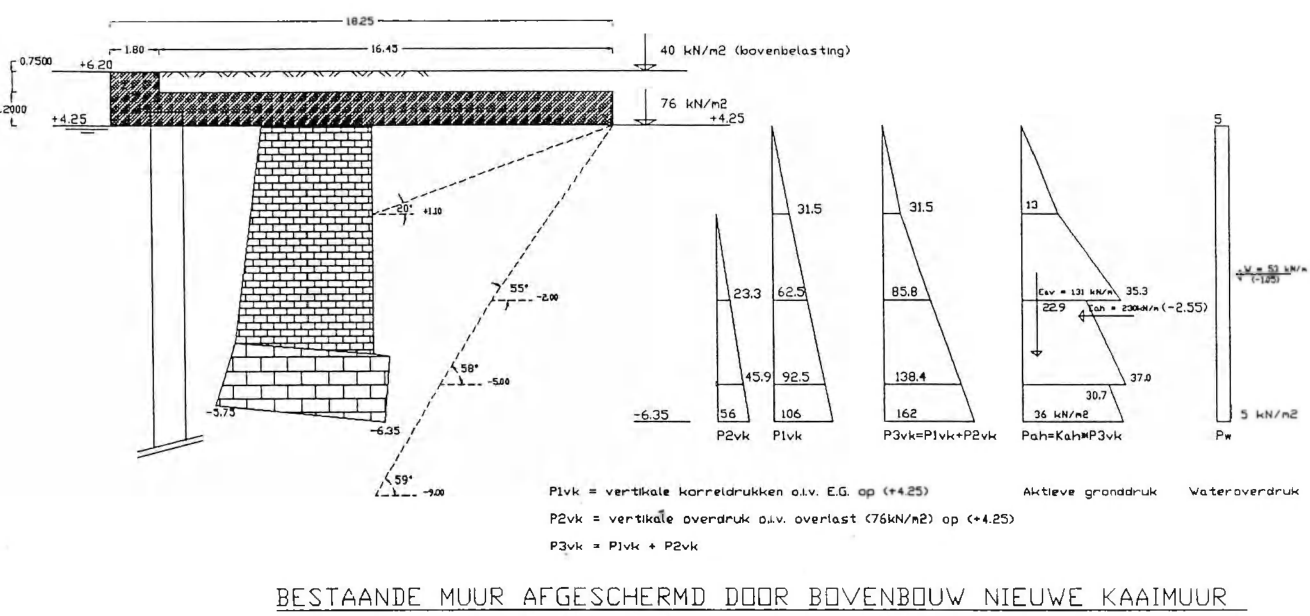
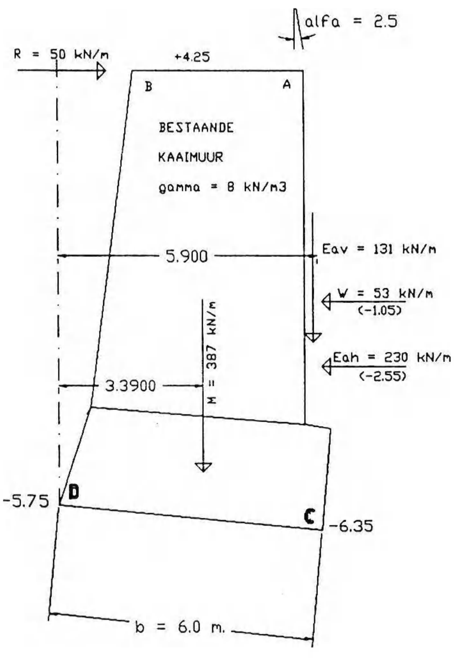


Fig. 58 — Ground pressure on covered quay wall.



VERTIKALE LASTEN

	LAST	HEFBOOM	M(D)
	kN/m	m.	kNm/m
M	387	3.39	1312
Eav	131	5.90	773
V	518		2085

HORIZONTALE LASTEN

	LAST	HEFBOOM	M(D)
	kN/m	m.	kNm/m
Eah	230	-3.20	-736
W	53	-4.70	-249
R	-50	-10.0	+500
H =	233		-485

M(D) = 1600 kNm/m

u = M(D)/V = 3.10 m.

e = 1/2b - u = -0.10 m.

e / (1/6b) = 0.10 H/V = 0.45

Sigma(m) = V/b = 86.5 kN/m2

Sigma1 = 86.5 * 0.90 = 78 kN/m2

Sigma2 = 86.5 * 1.10 = 95 kN/m2

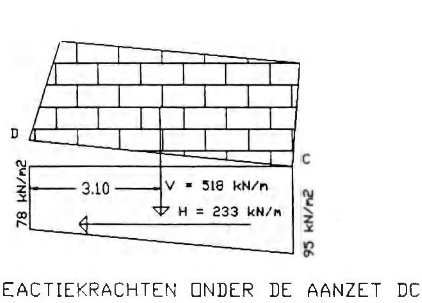


Fig. 59 — Reactions on base of existing quay wall.

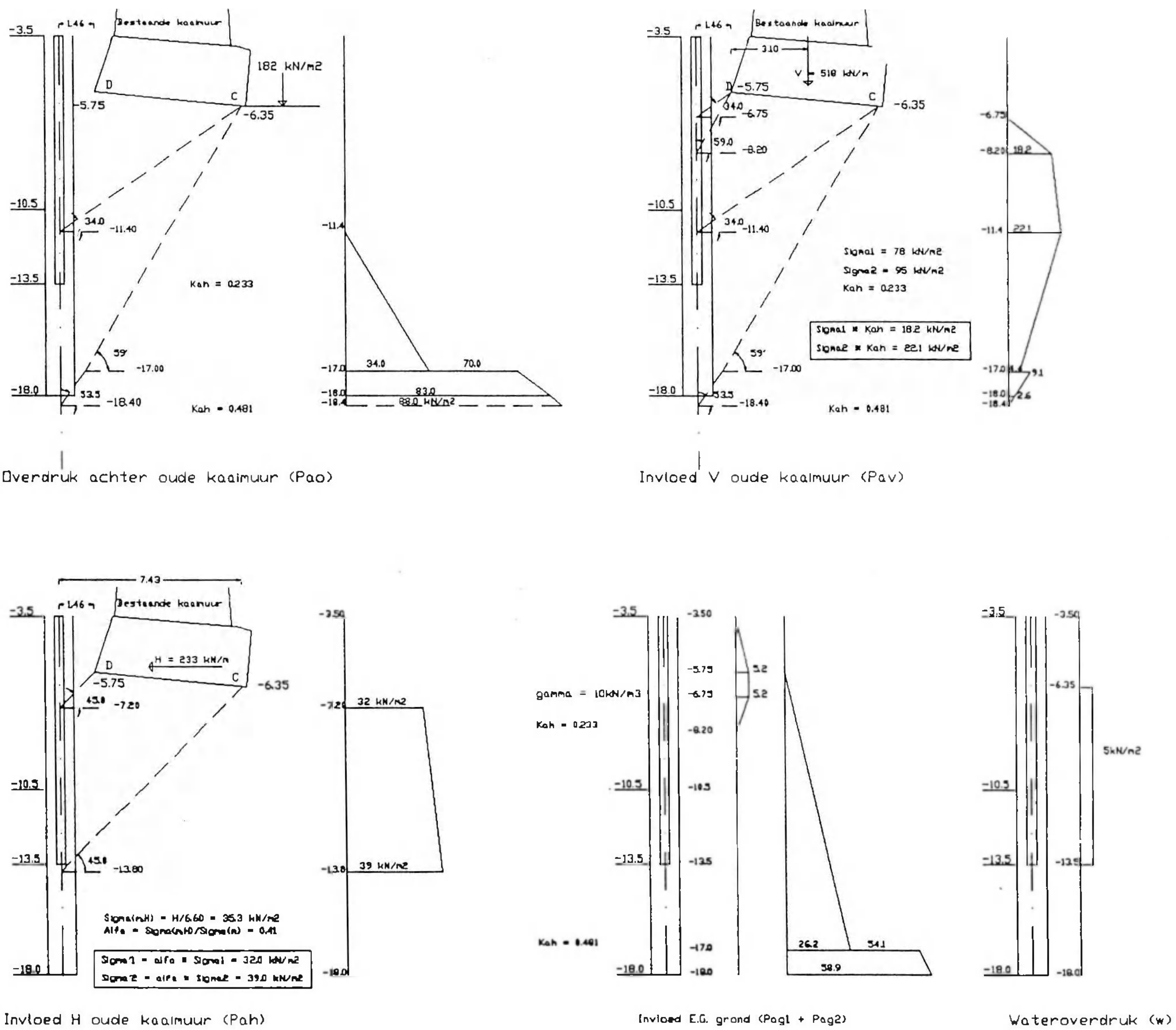


Fig. 60 — Components horizontal pressure on quay wall.

As shown in Fig. 63, in front of the superstructure resting on the partition piling and existing quay wall, reinforced concrete prefab elements are mounted, which are concreted to the top slab later.

The apron of this quay wall is also partly composed of prefabricated elements.

The tubes are correctly positioned by means of a steel frame, after which the partition boards are fitted (see Fig. 64).

Coating

In the splash zone above level TAW + 3,00 m a coating is applied in to protect the tubes against corrosion. This coating is composed of coal tar epoxy with an epoxy zinc primer.

The application is as follows :

— sand blasting Sa 2 1/2 followed by the application of the following layers:

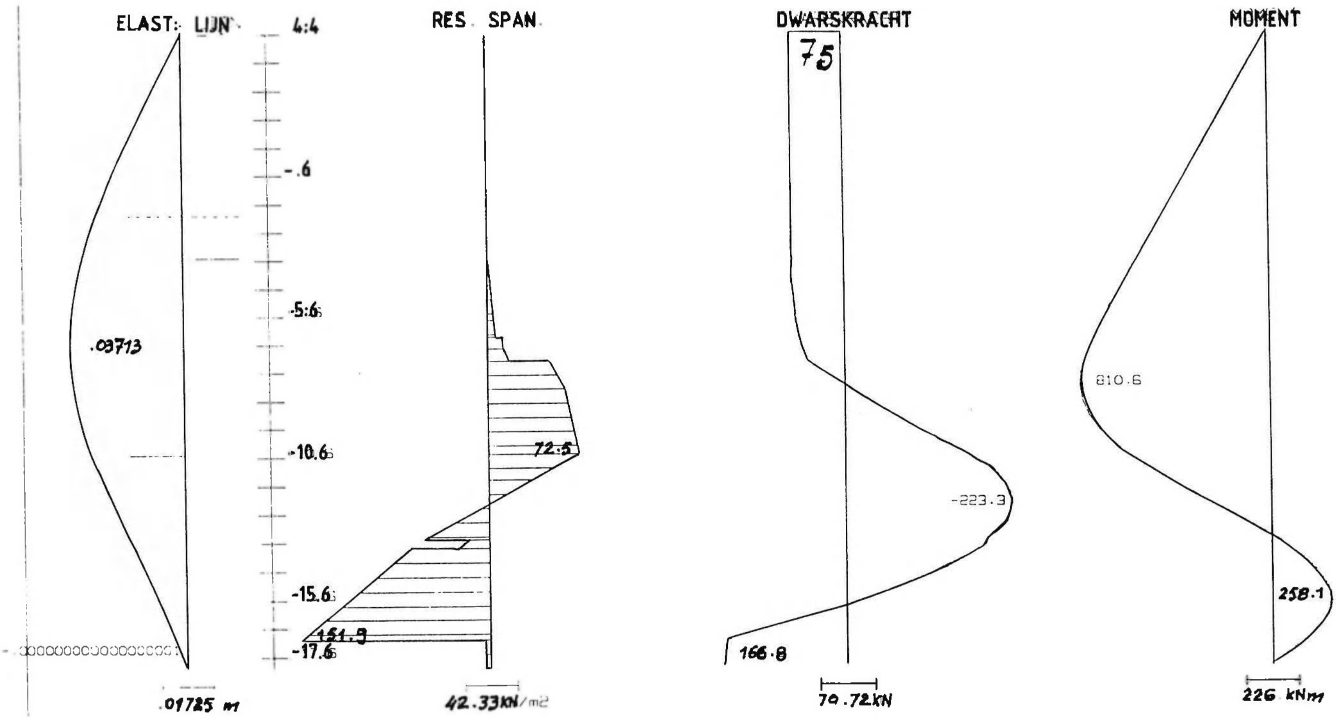


Fig. 61 — Graphic presentation of the calculation results.

— 25 to 30 µm Sigmarite zinc;
— 50 to 70 µm Sigmate sealer;
— 1 layer 150 µm Chemicote T brown;
— 1 layer 150 µm Chemicote T black.

Soil protection.

Between the combiwall and the existing quay wall a stone/asphalt shield is applied.

The open fiber stone asphalt is a warm prepared mixture of crushed lime aggregates 20/40 mm with asphalt mastic, whereby the distribution of the parts has a 80/20 ratio (by mass).

The asphalt mastic has the following composition (expressed in weight percentages):

— 45 to 70 % dry sand;
— 13 to 30 % soft filling matter;
— 12 to 25 % bitumen 80/100;
— 0,1 to 0,7 % cellulose fibers.

4.3.9. *Junctions*

The junction of the new quay wall to the existing quay walls in the America Dock and the Albert Dock, along the opposite bank, had to be constructed as follows: over a limited length (2 x 30 m) the combiwall would be placed in front of the existing quay wall.

This combiwall would serve mainly to make up for the difference between the dock bottom level (TAW – 10.50 m in the passage channel) and the existing dock bottom level (TAW – 5.00 m in both docks).

The disadvantage of this solution, however, is that the possibility of mooring is lost here (approx. 220 m).

The contractor was obliged to tender an alternative for the renovation of these junction sections, whereby the quay wall would be underpinned with piles using the VHP (very high pressure) grouting technique.
This method was not chosen after tendering, since no information on the small scale test project was yet available.

However early in 1992 this technique to deepen and strengthen a quay wall proved to be possible and therefore the decision was eventually taken to choose this solution, for the above-mentioned reason.

The technical construction of this section is dealt with below.

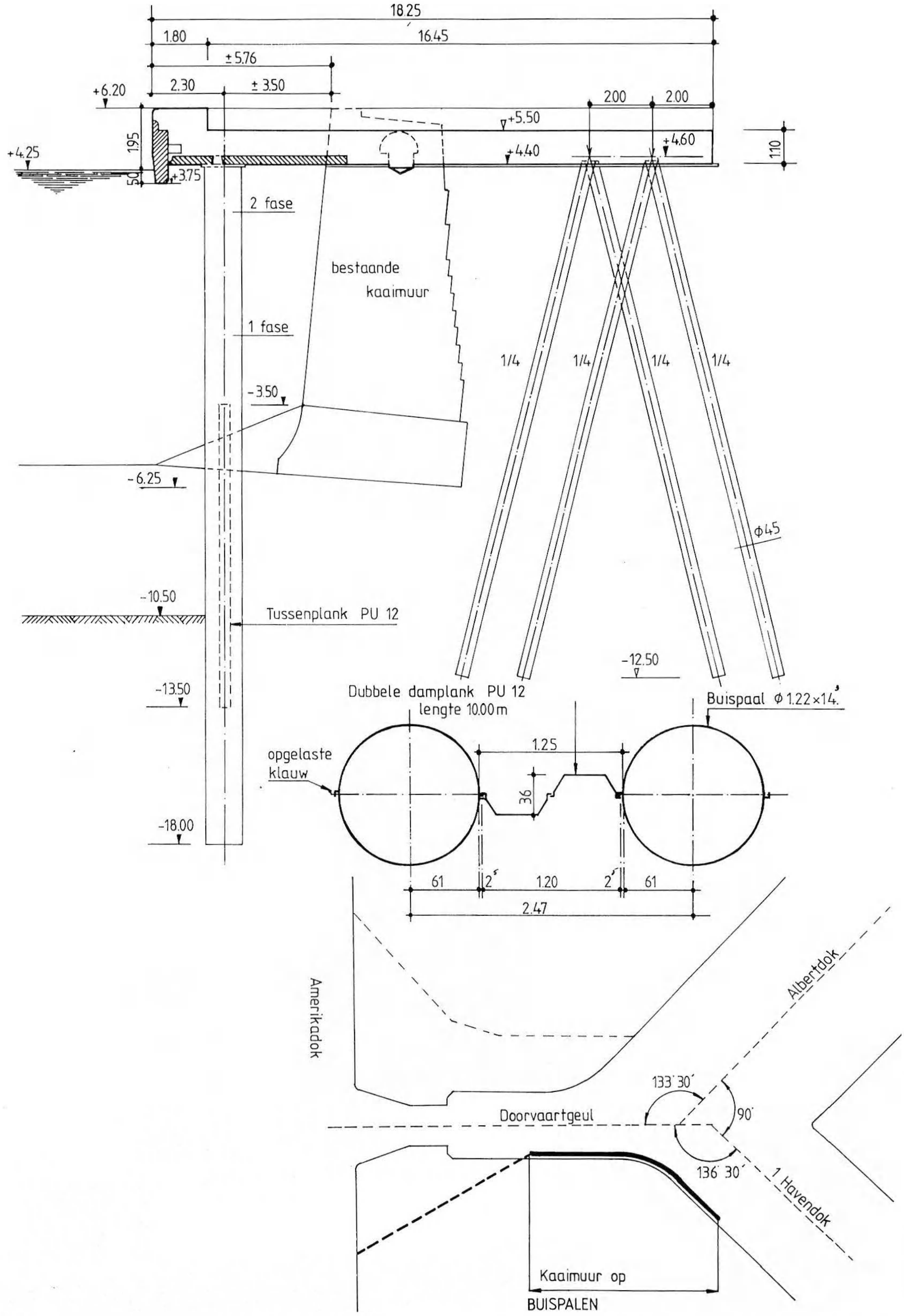


Fig. 62 — Cross-section of the quay wall with combiwall.

DOORSNEDE PREFABELEMENTEN

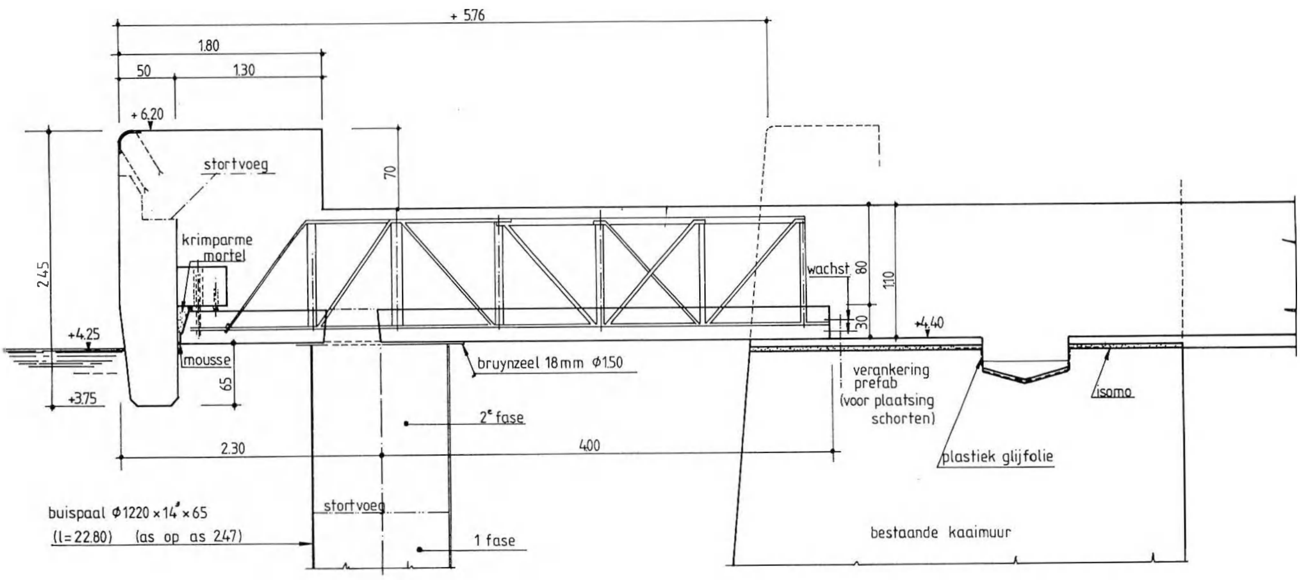


Fig. 63 — Detail of prefabricated elements supported by partition walls and quay walls.

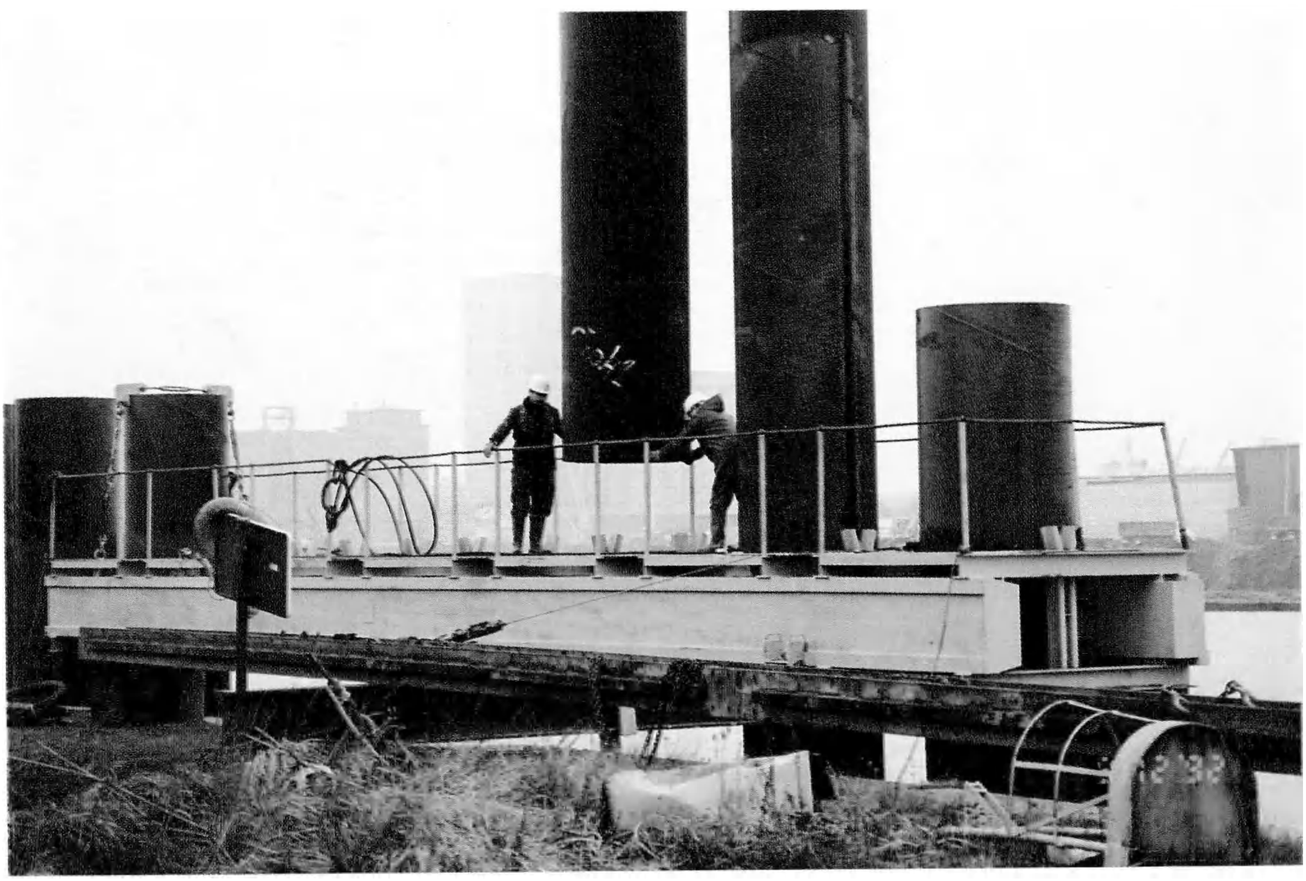


Fig. 64 — Positioning of the tubular piles by means of the frame.

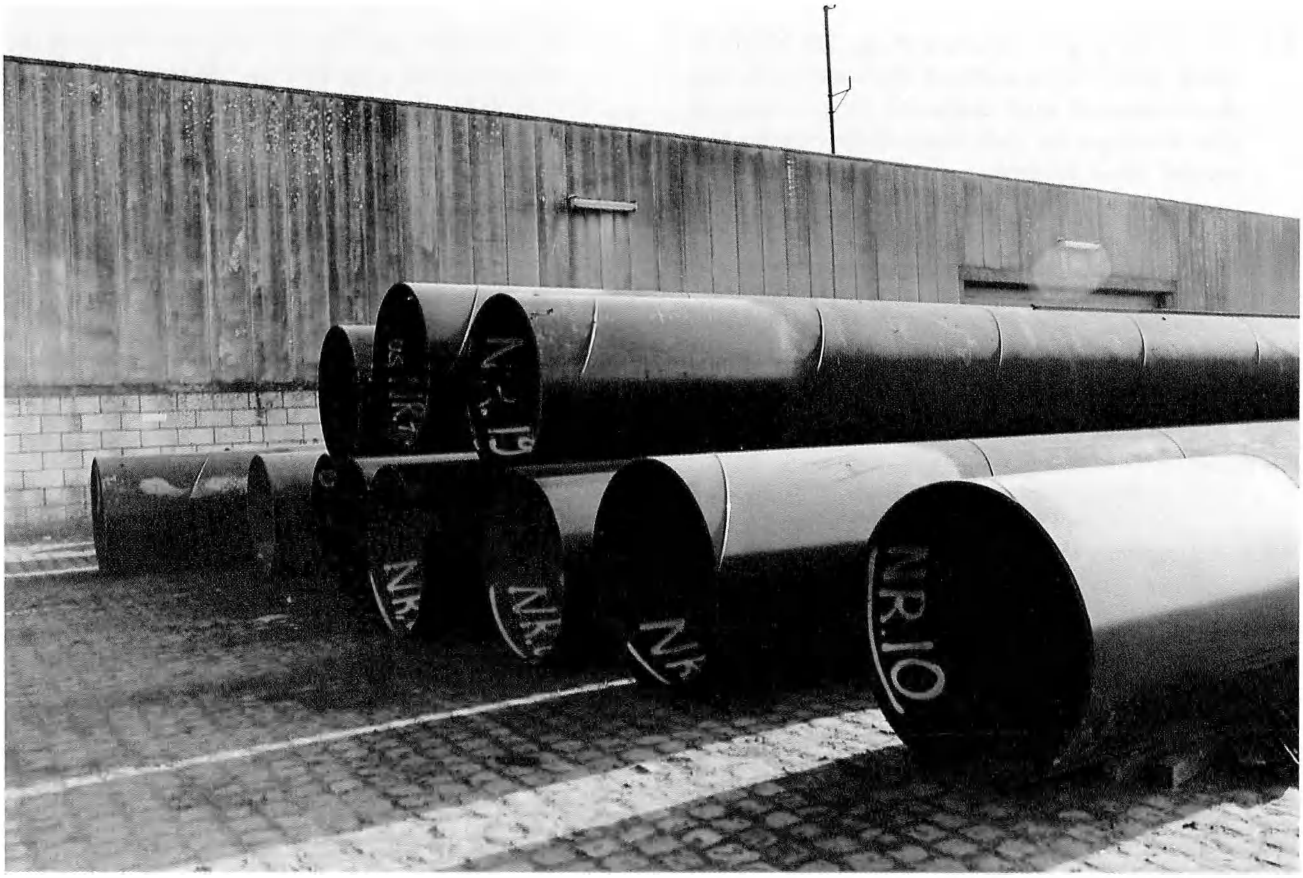


Fig. 65 — Tubular piles.

4.3.10. *Ground and dredging works for the realization of the passageway*

Including the new shelter dock for lighters, the construction of which is planned for early 1993, and the soil to be excavated in the America Dock near the Noordkasteel bridges, the amount of soil to be dredged and disposed of totals about 2,400,000 m³.

This soil will mainly be used for :

— the replenishment of the 1st Harbour Dock and the existing shelter dock for lighters, thus creating new port areas on which modern terminals can be constructed;

— direct dumping into the dike (upstream Kruisschans) to strengthen and to weight the Sigmadike along the Sea Scheldt;

— the creation of sand stocks for future dike constructions near Lillo fortress in the framework of the Sigma plan;

— the reclamation of industrial areas near the Tijsmans tunnel.

By the end of December 1992 300,000 m³ had already been recovered from the excavated quay wall and dumped in :

— the 1st Harbour Dock;

— and approx. 100,000 m³ soil with a restricted geotechnical value was dumped in a dredged pit in the Industry Dock at the back of the 5th Harbour Dock.

The good soil from this pit had first been pumped hydraulically into the dike near Fort Philip with a cutter suction dredger.

4.4. Deepening works America Dock, 5th Harbour Dock, Albert Dock and 3rd Harbour Dock, and the creation of sand stocks for the execution of dike works on the right bank of the Scheldt and several other projects.

Contractor : Joint Venture "Verdieping Havendokken" — **Dredging International N.V.** — **Jan De Nul N.V.**

Prior to or simultaneous with the widening and deepening of the passage channel between the America Dock and the Albert Dock the navigation channel along the southern route was deepened.

4.4.1. *Deepening works 5th Harbour Dock — America Dock*

Fig. 66 — Towed hopper suction-dredger "Krankeloon".

Dredging works.

These works consisted of :

— the dredging of a channel to a draught of TAW – 10.50 in the Fifth Harbour Dock;

— the dredging clear of the new quay walls at the south of the America Dock to TAW – 10.50 and at the north side of the America Dock to TAW – 5.00.

— the clearing of a shallow in the Hansa Dock near the passage channel to the 5th Harbour Dock to level TAW – 10.50 m.

— the deepening of the passage channel between the America Dock and the Fifth Harbour Dock at the Noordkasteel bridges to level TAW – 9.50 m.

The deepening works in the navigation channels and the passages were carried out with towed hopper-dredgers equipped with a draghead to plough up tough and compact sand layers.

The joint venture "Verdieping Havendokken", i.e. **Dredging International N.V.** and **Jan De Nul N.V.**, employed the following towed hopper suction-dredgers:

Name	Length m	Width m	Draught m	Hopper capacity m^3	Power kW
Krankeloon	94.5	17.4	6.5	2,700	6,045
James Ensor	105.2	18.2	6.0	3,600	7,350
Khersones	105.8	18.2	6.5	4,800	6,433

During the period from February to June 1991 these craft dredged a total of 745,000 m^3 from the 5th Harbour Dock and the America Dock, transported it to different locations and through pressurized pipes at quays nos. 373 and 497 to the different earth buildings of the dike sections along the Scheldt banks.

Fig. 67 — Towed hopper suction-dredger "James Ensor".

In order to safeguard the stability of the existing quay walls, dredging took place in the middle of the docks and a 40 m wide shoal along the shore was kept at the original depth.

The Falcon positioning system, computer steered from four standardized transmitters, was used allowing the hopper suction dredgers to work in the dredging area in the middle of the docks.

For the dredging works in the 3rd Harbour Dock and the Albert Dock the hopper suction dredgers are also equipped with monitoring equipment to register the exact location and depth of the dredging head.

In this way the dredging works could be accurately monitored and varied on board ship taking admitted tolerances into account.

Fig. 68 — Towed hopper suction dredger "Khersones".

The dredging clear of the quay walls in the America Dock and the deepening of the passages in the Hansa Dock and the vicinity of the Noordkasteel bridges was executed by the dipper dredger "Zenne", equipped with an hydraulic excavator with a loading capacity of 8 m³.

This combination of a 50 m long and 11.40 m wide pontoon equipped with 3 spuds and an hydraulic excavator loaded the dredged spoil into elevator barges "DI 67" and "DI 67".

These elevator barges were discharged by suction power of 2,265 kW. Suction barge "Warche II" was moored at quay no. 373 in the Industry Dock. It pumped the slurry through a 2,300 m pipeline with many quay and road bridgings into the projected dikes along the Scheldt bank, south of the Van Cauwelaert Lock .

In total 270,000 m³ sand was excavated from the docks by these craft and processed in dike building from March 1991 until December 1991.

Fig. 69 — Dipper dredger "Zenne"
while excavating the renovated quay wall
(in the background the future site for the new shelter dock for lighters).

Dike works and the building up of sand stocks

The dredged sand was hydraulically pumped by the tow and the storage dredgers into two dike sections up- and downstream of the Kruisschans, along the Scheldt bank. The sailing distance for the towed suction dredgers from the dredging areas to the discharging places amounted to 5 km upstream and 2.5 km downstream of the lock complex.

Dike section downstream of the lock complex

Before starting to raise this dike, the soil underneath needed improvement. Rich soil was excavated from the foreshore and salt pasture and stored in the form of slurry walls on the existing dike and edge of the foreshore.

These stocks were used later as a covering layer for the projected embankment.

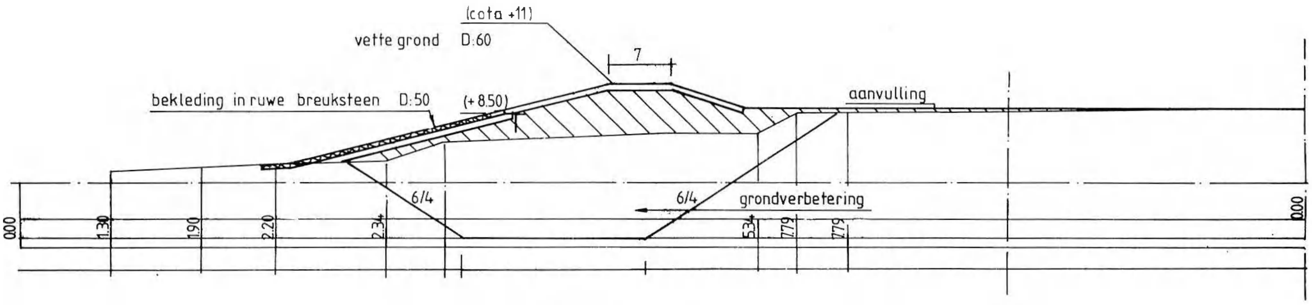


Fig. 70 — Dike profile.

The excavator on pontoon "Zenne" dredged 76,000 m³ from trenches forming longitudinal slurry walls. Dredged sand from the docks filled the cavity, acting as a soil improver.

At quay 497, north of the Baudouin Lock access, a pontoon pressure plant was installed for the towed hoppers. It was connected to a 2,100 m pipeline through which 490,000 m³ sand was hydraulically pumped into the dike which was to be raised over a distance of 920 m.

Dike section upstream of the lock complex.

From the foundation of the projected new dike drained fertile earth was excavated to serve as slurry walls for the infilling work, and later to finish the dike. By draining the excavated soil from the future dike foundation, it was improved though at a lesser depth than in the upstream dike section.

From quay no. 373 the towed hoppers and barge suction dredger pumped 525,000 m³ sand into the dike over a distance of approximately 920 m.

Protection of the bank downstream of the Kruisschans dike section.

To prevent erosion of the downstream raised dike by River Scheldt temporary protection was applied along the bank of the dike under construction.

This bank protection consisted of a geotextile complying with specifications of the **Tidal Waterways Service**, covered by rockfill calibre 5 – 25 kg between TAW 0.00 and TAW + 8.50.

At some places a rubble covering of calibre 20 – 80 kg was applied (wet weight).

Fig. 71A — Sea-Scheldt dike. Slurry wall ditch.

Fig. 71B — Sea-Scheldt dike. Bank construction.

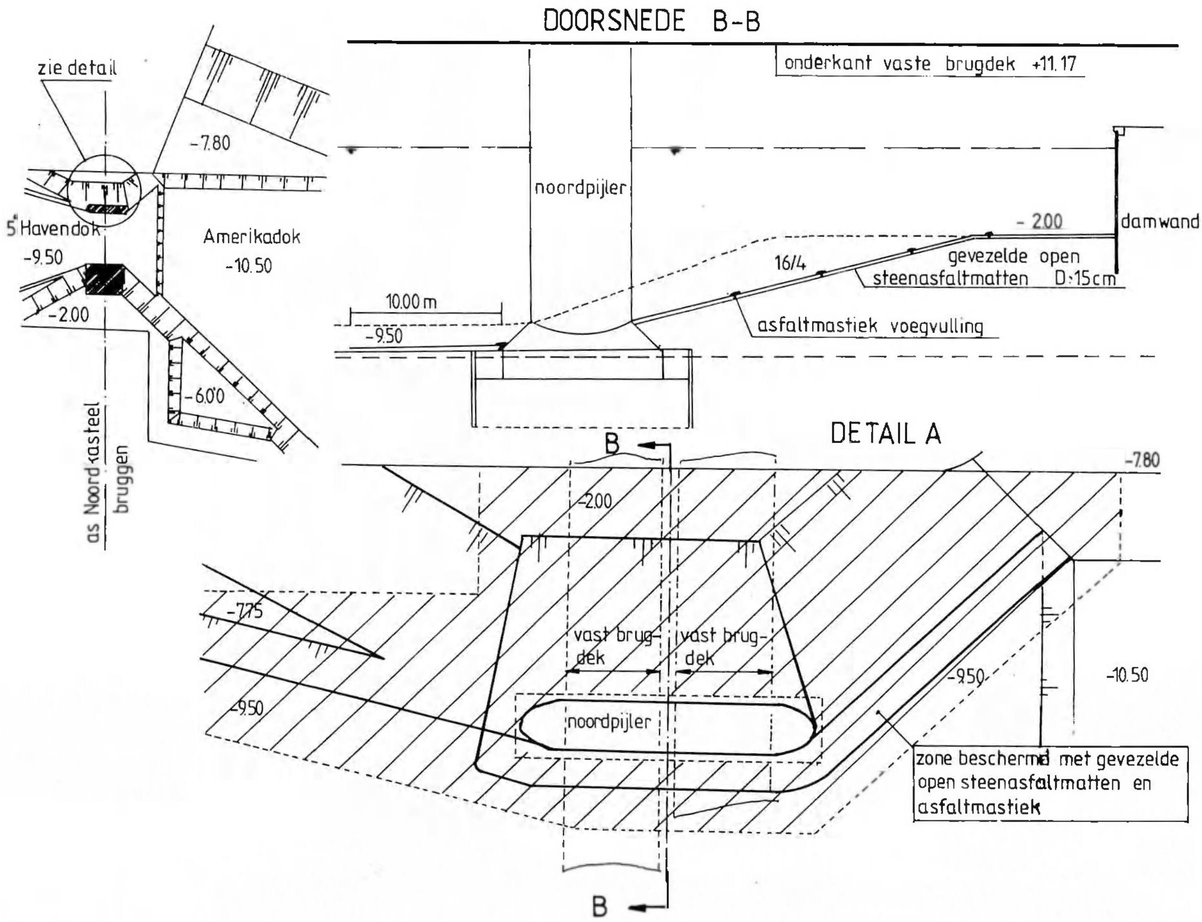


Fig. 72 — Location of the bottom improvement works north of the Noordkasteel bridges.

4.4.2. *Bottom protection of the Noordkasteel bridges passage — structured open stone asphalt mats*

While the 5th Harbour Dock and the America Dock were being dredged, the passage through the Noordkasteel bridges was deepened from TAW – 7.85 to TAW – 9.50.

The lowered bottom and underwater bank near the north pier of these movable bridges required protection to prevent the natural base material from erosion, which would endanger the stability of the bridge foundation.

This was achieved using 15 cm thick prefabricated mats made of structured stone asphalt on an underlying geotextile.

After placing these prefabricated elements around the bridge pier and against the partition walls, all joints between the mats, pier and partition walls were sealed with asphalt mastic poured under water.

Structured open stone asphalt has been used as a durable, water permeable and flexible bottom protection for years in situ or as a prefabricated structure under water.

The shore and bottom of the port of Antwerp and the Sea-Scheldt have been similarly protected against natural and traffic generated erosion.

Bitumar N.V., which has patented the production and laying of structured open stone asphalt, carried out the work as a subcontractor.

Stone asphalt is fiberised asphalt mastic mixed with 16/22 mm or 20/32 mm stone chippings in mass proportion of 20/80 %. It is manufactured in a 3-stage mixing process.

Because of the very open character of the product (hollow space percentage approx. 30 %), it must always be applied in combination with an underlying filter material (natural filter, geotextile or sand asphalt).

Fig. 73 — Prefabrication and laying of the stone asphalt mats under water.

The main characteristics are :

- relatively thin sheetform coating (10 to 15 cm);
- high resistance against water stream (at least 6m/s) and wave effect;
- durable material;
- very flexible (adjusts itself without any problem to the undulations and settling of the bed);
- fast and flexible application (little hindrance to vessel and road traffic);
- maintenance free coating.
- the size and thickness of the mats can be selected to fit local circumstances;

The fiberised open stone asphalt can only be directly processed dry. Under the waterline asphalt mats that have been prefabricated to the required size on a flat surface or pontoon are used.

They are lifted individually by means of re-usable strips, hanging from a special laying frame provided with a pneumatically controlled disconnecting system.

Fig. 74 — Placing of the fiberised open stone asphalt mats under the fixed bridge deck section.

The mats are juxtaposed whereas the underlying geotextile overlaps.

An additional difficulty when laying the stone asphalt mats near and under the Noordkasteel bridges was the limited clearance of merely 7 m under the fixed bridge deck section, i.e. between the north pier and the north abutment.

These asphalt mats were lifted one by one from a pontoon by means of the laying frame, suspending from 2 independent cables, led through openings in the bridge deck by means of two heavy telescopic cranes positioned next to the bridge.

The other mats to be arranged next to the fixed bridge deck were placed by means of the floating cranes "Portunus" and "Titan" and the derrick "Brabo" owned by the **Port Enterprise of the City of Antwerp**.

Deepening works of the Albert Dock — 3rd Harbour Dock

In the Albert Dock the projected deepening from TAW – 6.50 to TAW – 10.50 over a length of 1,700 m and a width of 170 m, produced a 40 m shoal along the existing quay wall required for stability at level TAW – 6.50 m.

To meet the urgent need for sand supplies for the A-12

Harbour Road at Zandvliet and the Renaval project next to the Tijsmans Tunnel, the dredging in the Albert Dock was started early 1992.

The towed hopper suction-dredger "Antigoon" (power 6,500 kW) was used to bridge the pressure pipeline distance between discharging place and sand supply area, particularly because the spoil to be transported consisted of coarse sand and many shells.

Name	Length m	Width m	Draught m	Hopper capacity m^3	Power kW
Antigoon	115	22.5	8.7	8,400	9,735

Dredging with this kind of towed hopper in the 250 m wide dock was no easy task, especially when swinging round after each dredging section and when the suction pipe also had to be lifted from the bottom.

Fig. 75 — Towed hopper suction-dredger "Antigoon".

Sand replenishment for the A-12 Harbour Road

In February 1992 500,000 m^3 sand from the Albert Dock was transported to a sand stock east of the Noordland bridge opposite of the Scheldt-Rhine connection for the A-12 Harbour Road.

The imposed 10 week execution period could be met by employing this heavy dredging equipment.

The sailing distance from the Albert Dock to the B2 Canal Dock, discharge site next to the Scheldt-Rhine connection entrance, was 16 km and one single trip, including mooring time, took approximately 1.5 hours.

Once anchored at the discharging site the "Antigoon" was connected to the pressure pipelines on shore by means of a bow connection, and the sand was pumped to the storage site some 4.5 km away.

The sand was raised by 5 to 6 m and the dredger slurry water was pumped by 2 water pumps back from the stocks into the Scheldt-Rhine Canal through a separate pipeline.

Earth building work for the Renaval-project (Berendrecht distribution zone)

In the first phase of the Renaval project, north of the access roads to the Tijsmans tunnel, 500,000 m^3 of sand, from the deepening of the Albert Dock, was to be supplied.

The sailing distance from the Albert Dock to the discharging site was 11 km and the sand was pumped 3.2 km in a layer approximately 2 m thick.

Although the sailing distance was shorter than for the A-12 Harbour Road, the time difference was small because mooring at the Lillo bridge was more complicated.

Because anchoring near the bridge was prohibited and the effects of occasional strong winds on the "Antigoon", mooring took place at the pontoon along the quay, assisted by a powerful launch.

The slurry water was collected in a basin where any remaining sand could settle, then the water was pumped to natural waterways.

A second phase of these earth building works was started early in 1993. In this phase some 300,000 m^3 of spoil was dredged from the 3rd Harbour Dock and Albert Dock and transported to the same area.

4.5. Renovation of the quays at the north side of the Third Harbour Dock

Contractor : Joint venture **Herbosch-Kiere N.V. — Antwerpse Bouwwerken Verbeeck N.V.**

4.5.1. *Financing — Subsidy arrangement — Principal*

Design

Following the applicable Decision of the Cabinet dated 30th October 1986 the principal of the renovation works of the quay walls in the 3rd Harbour Dock is the **Port Enterprise of the City of Antwerp**, and 80 % of the works is subsidised by the Flemish Region.

Indeed, these constructions comprise basic infrastructure from which the Port obtains revenues, and concern the renovation of an existing structure.

As a result the design of the renovation and the tendering were taken care of by the Port Enterprise of Antwerp, whereas the formerly described works relate to basic infrastructure not yielding any revenue to the port of Antwerp. For the latter works **the Flemish Region** acts as principal.

Thus the contracts and works discussed in the previous chapters were designed and executed under the supervision of the Regional Authorities.

4.5.2. *Existing quay wall — Design requirements — Basic concept*

The present quay walls were built shortly before the 1st World War.

They are gravity walls, made of plain concrete up to half height, on top of this brickwork. These walls are still in a good condition and can still have a ground retaining function.

The north quay of the 3rd Harbour Dock is about 1,140 m long and has been given in concession to two terminal operators: **Manuport N.V.** and **Belgian New Fruit Wharf N.V.**

Manuport, concessionaire of the eastern 800 m, invested a lot in a modern fertilizer terminal. This was planned to go along with the renovation of the quay wall. The co-ordination of the construction works of the quay wall and those by the terminal operator and his operating requirements have caused quite some problems indeed.

The operations on the quay had to be considered.

The most obvious concept of the new quay wall is driving a new sheet piling in front of the existing quay wall, and bridging the space between the sheet piling and the quay wall by means of a concrete slab.

Given the circumstances — the massive wall has a protruding toe on the dock bottom — the top of the renovated wall would protrude at least 6 to 7 m into the dock with regard to the existing quay wall.

By increasing this size up to 10,50 m it is possible to have the concrete slab supported at the other side by the existing massive wall, which saves a pile foundation. Also the sheet piling can be lighter.

In addition to this the surface extension was accepted thankfully by the port operators who are both in need of more space.

Other design requirements

The most important further requirements include the mounting of modern port cranes on the renovated quay.

Considering the present and anticipating future trends, this crane should have an outreach of at least 50 m, rail gauge 11 m, and leg load of 240 tons distributed over 4 bogies of 60 tons each. The crane track itself is to be constructed by the concessionaire.

The mobile overload onto the quay platform amounts to 6 t/m² and it must be possible to put one rail track between the crane rails.

Basic concept

Taking into account all above-mentioned requirements a concept was drawn up by the port enterprise, consisting of a sheet piling carried out as a combi-wall driven into the dock bottom in front of the existing quay wall, and a reinforced concrete slab resting on this combi-wall and on the present massive wall, the top of which had to be cut off in advance.

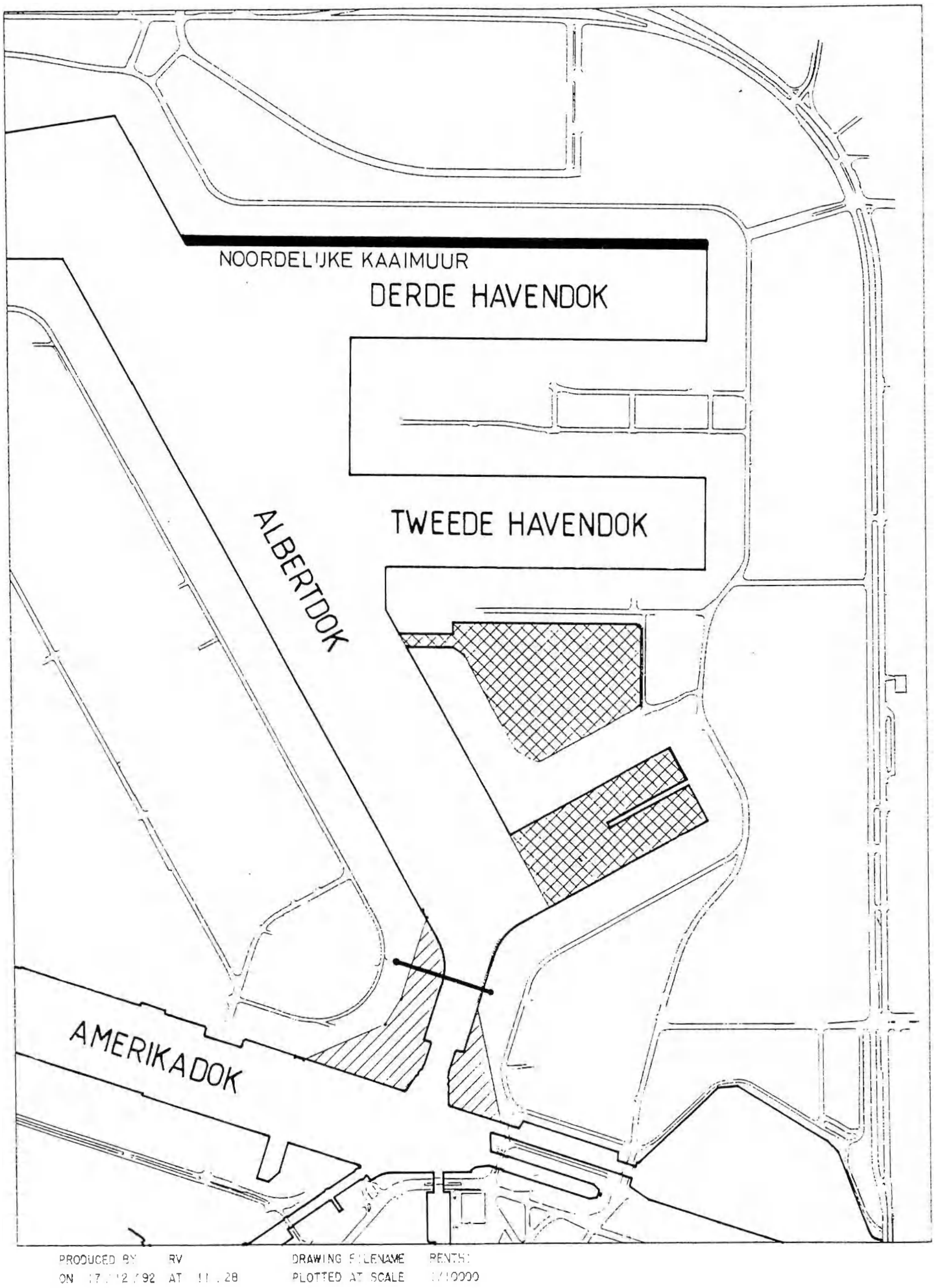


Fig. 76 — Location of north quay 3rd Harbour Dock.

This sunken concrete slab is covered by a layer of sand and the quay paving. The increase of the ground pressure and the bollard forces is taken up by ground anchors.

Further the usual quay attributes are foreseen: mooring-posts, ladders, a drain, sewerage outlets, hydrant recesses, cable slots and, as the quay wall is of a light type, rubber fenders.

The new quay wall protrudes 10,50 m into the dock with regard to the old one.
The space between the sheet piling and the old quay wall remains open; the bottom is provided with a bottom protection.

The construction of the north quay was divided into three phases for budget reasons.

For the 1st phase (in which repetition of award was foreseen) a public tender was called.

The most favourable and lowest bid came from the joint venture **Herbosch-Kiere N.V.** — **Antwerpse Bouwwerken Verbeeck N.V.** with the basic concept described above.

Detailed design

Present situation

The existing quay wall is a gravity wall. The upper 8.70 m is a brick wall, the lower 7.60 m is plain concrete. The stability of this wall is assured by its own weight, which prevents both overturning and shifting. The ground pressure on the back side of this wall and its gravity results in a vertical trapezoidally distributed grain pressure and in a horizontal force in the foundation area of the wall.

Soundings and drillings with sampling had been carried out in advance by the Geotechnics Department. Triaxial tests resulted in a ϕ'-value of 30° or more. Fig. 78 shows a typical deep sounding meassurement.

4.5.3. *Sheet piling : "Combi-wall"*

The sheet piling is to resist vertical reactions caused by the superstructure and the ground pressure. For this sheet piling a "combi-wall" solution was chosen: steel pile tubes between which sheet pilings are driven.

The pile tubes, nearly entirely filled with concrete, have both a big vertical and horizontal load capacity.

The sheet pilings only serve to fill the space between the pile tubes; their vertical and horizontal load capacity is niglectable.

As already said the space between the combi-wall and the old quay wall is not filled up; therefore the sheet pilings are much shorter than the pile tubes.

For the combi-wall three kinds of loads can be distinguished:

1) the ground pressure onto and the own gravity of the massive wall result as mentioned before in a trapezoidally distributed vertical grain pressure in the foundation area of the massive wall. This vertical grain pressure is transmitted onto the sheet piling.

2) the horizontal force in the foundation area of the massive wall.

   To transmit this load onto the combi-wall (a linearised form of) the theory of **Boussinesq** was used.

3) finally there is the active and passive ground pressure caused by the weight of the ground in front of and behind the sheet piling. The sheet piling was calculated with these loads making use of the Standard **Blum** method.

   The horizontal forces at the head of the sheet piling are, together with the tensile forces onto the mooring posts, absorbed by evenly spread ground anchors behind the concrete slab.

   To absorb the landward directed horizontal reactions of the fenders the slab is anchored onto the massive wall by anchor rods drilled at 45°.

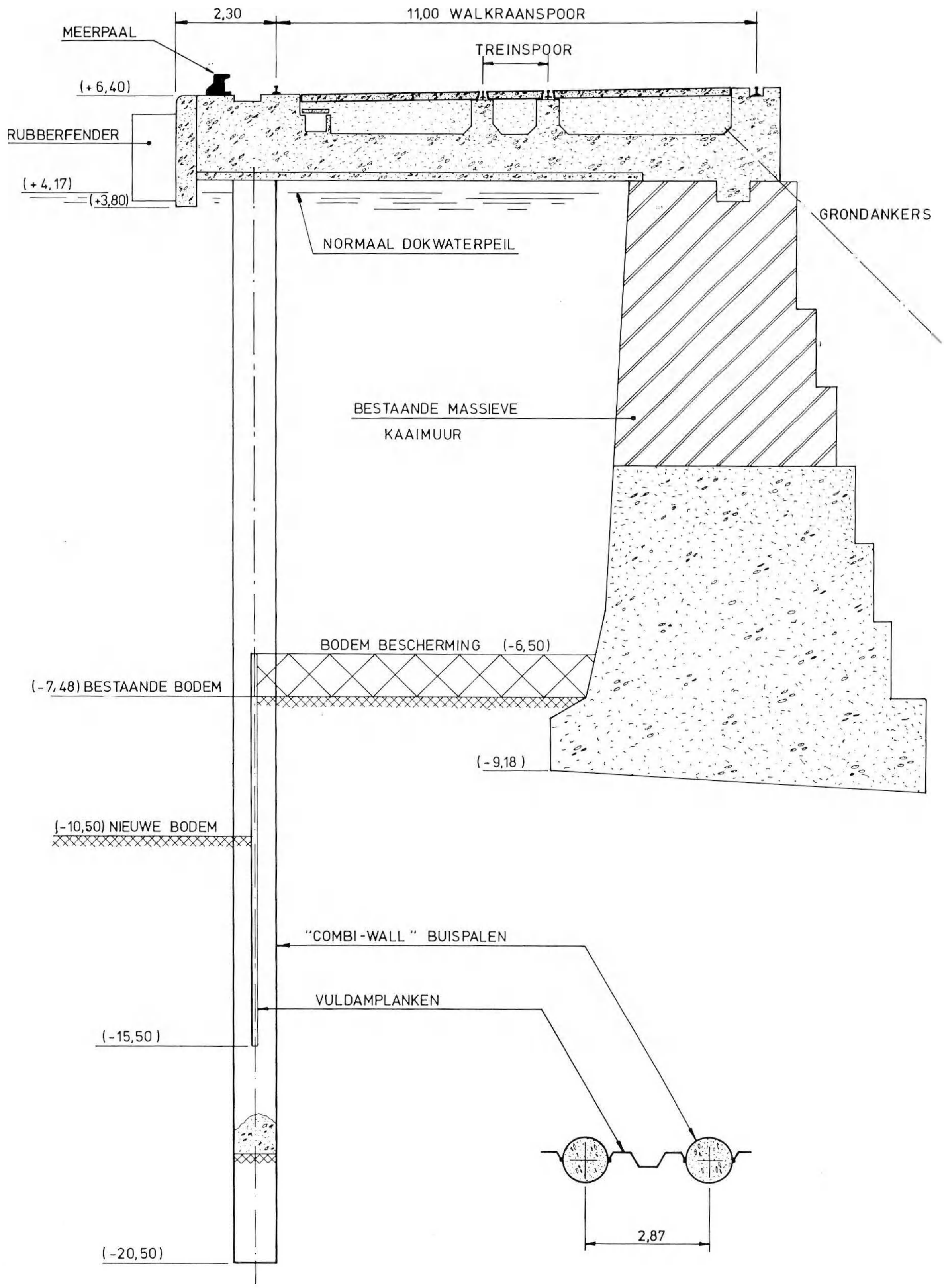


Fig. 77 — typical cross-section of the quay wall renovation.

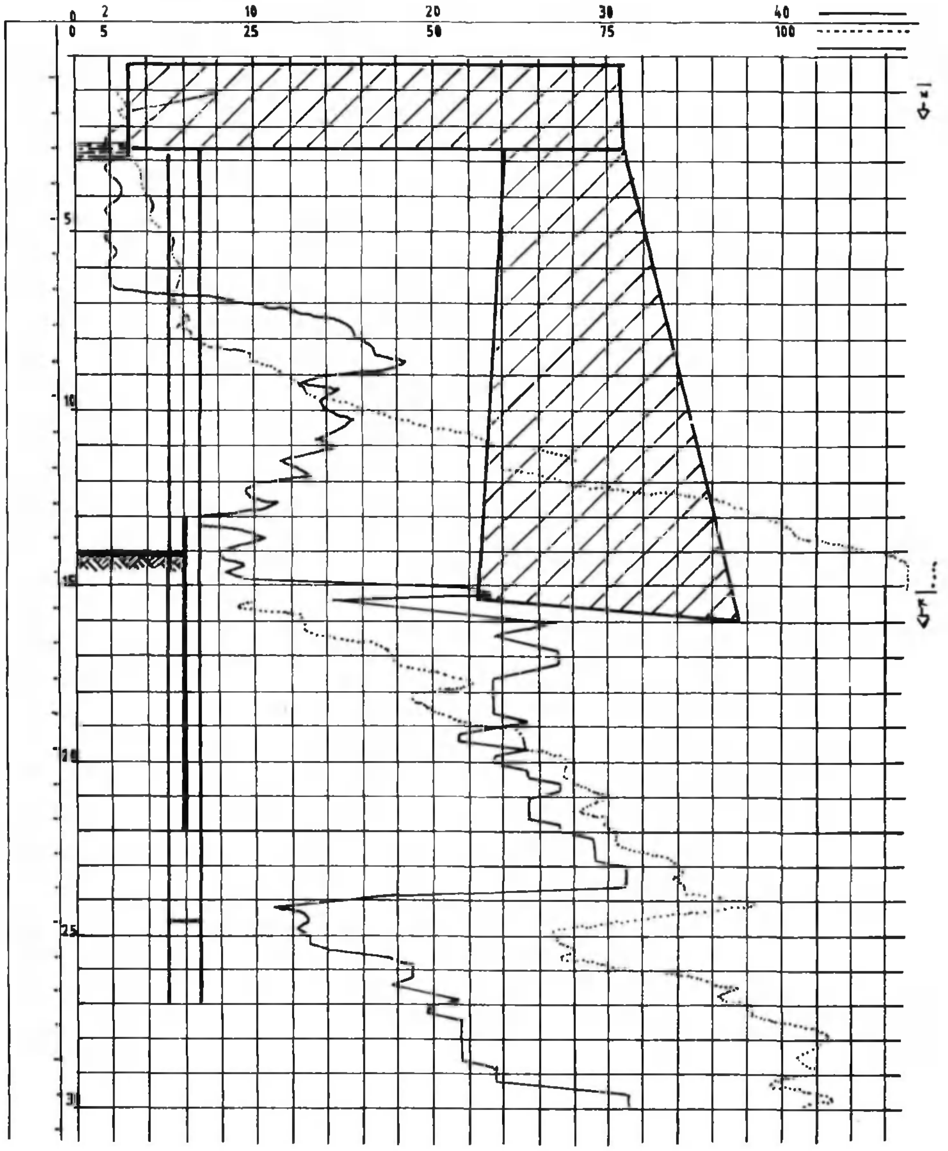


Fig. 78 — Deep-sounding result at the location of the quay wall to be constructed.

4.5.4. *Concrete superstructure.*

The concrete slab rests on the massive quay wall and on the pile tubes 8.70 m in front of it, and is still projecting 1.80 m.

The concrete slab is cast on prefabricated concrete sheets of only 20 cm thick to limit their weight.

The overall thickness of 1.10 m (the prefab sheet included) of the concrete slab was reached in two casts: first a concrete layer of 15 cm was cast into the prefab sheet, and after the concrete had reached full rigidity the second layer of 75 cm was cast.

The first layer was necessary to resist the load of the next one and to limit the bending, and to realise the connection with the front surface of the concrete slab which consisted of a prefab concrete element as well.

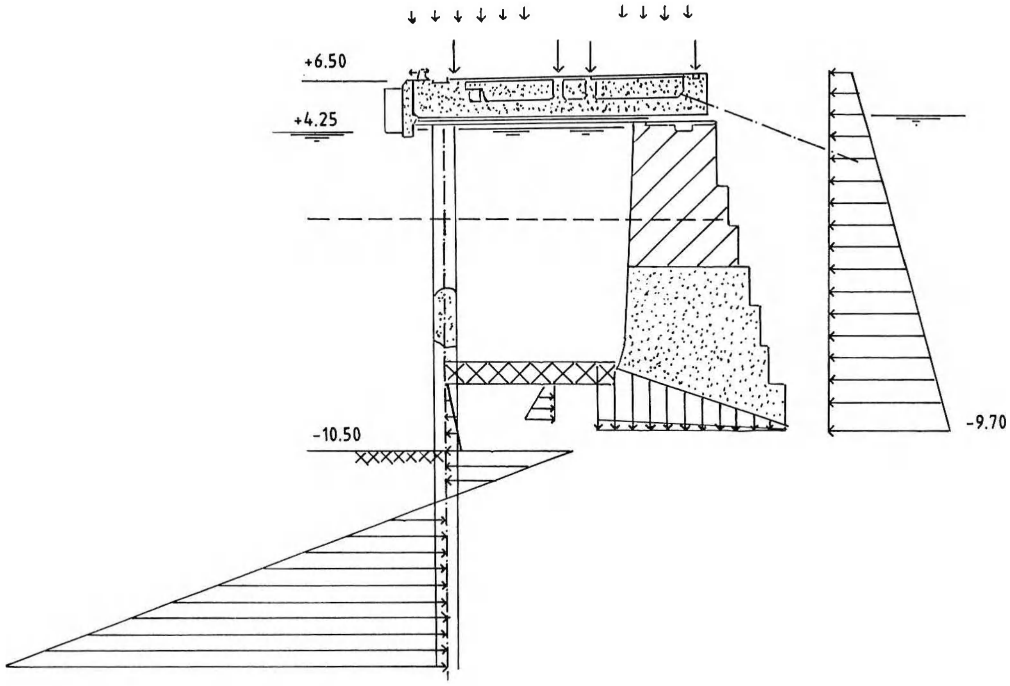


Fig. 79 — Survey of all loads.

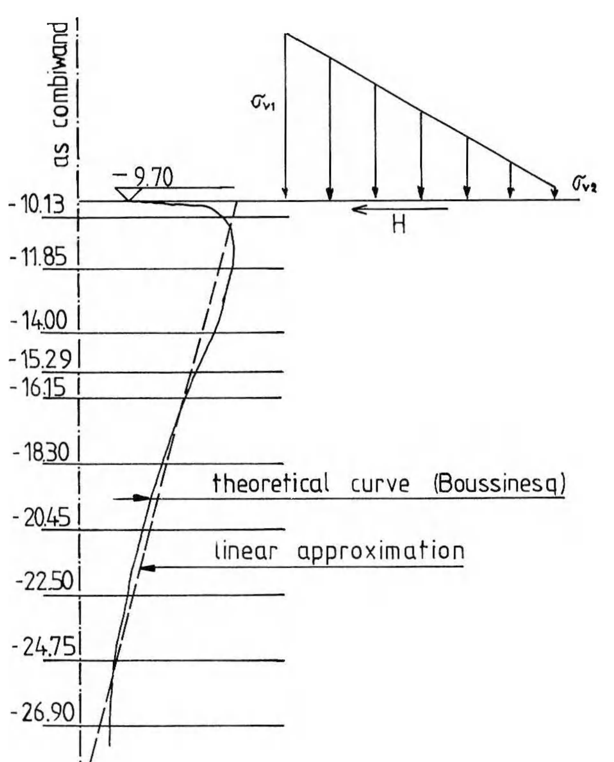


Fig. 80 — Ground pressures according to the theory of Boussinesq.

In view of the aggressivity of the commodities which will be handled (fertilizers), a concrete cover of 7.5 cm was imposed and the admissible crack width limited.

The lower reinforcement in the prefab sheet has not been incalculated for this reason, and is considered to have only a function during the construction phase.

The top reinforcement in the prefab sheet — needed because of its slenderness — together with extra reinforcement bars in the first cast in situ concrete layer, constitutes the lower reinforcement of the slab.

As a result the latter has a coverage of 17 cm, and nevertheless can resist 300 N/mm² in spite of the crack width restriction.

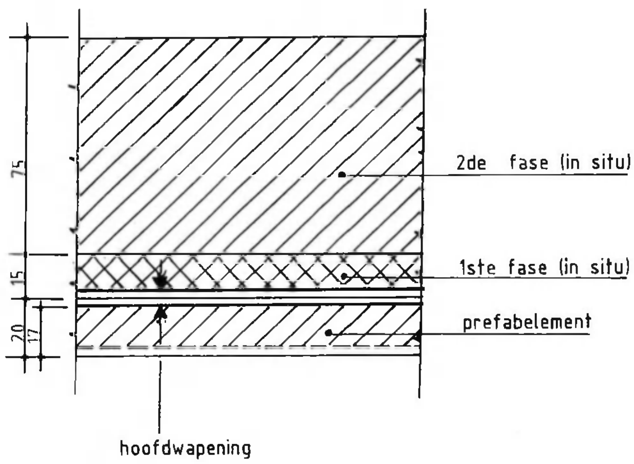


Fig. 81 — Concrete superstructure. Main reinforcement structure.

4.5.5. *Execution*

The whole length of the north quay of the 3rd Harbour Dock is to be executed in 3 stages.

The first stage covers a length of 370 m starting from the eastern end of the dock, and has been completed in the meantime.

It was carried out simultaneously with the investment plan by **Manuport**.

Pretty much co-ordination with this enterprise was needed. It was necessary after all to carry out part of the work from the water with floating equipment.

On top of this a part of the quay wall was put into operation long before it was finished ...

The construction of the quay wall is a sequence of logical steps that proceed "like a train":

— Preparatory works.

Installation of the wharf.

Breaking up of all kinds of pipelines and cables, some of which had to be relocated for the time being.

Breaking up of the quay paving.

Removing and scrapping of obsolete quaycranes.

— Sheet piling and bottom protection.

The pile tubes are made of steel quality AE 355 and have a diameter of 1,016 mm, a length of 24.1 m and a gauge of 14 mm.

In this thickness 3 mm surplus with regard to the statically needed thickness is included. No paint nor any other corrosion protection is foreseen.

The pile tubes were planted with a heavy vibrating hammer (excentrical moment 1,100 Nm) and brought largely to the required depth; part of the pile tubes had to be driven to the definite level by means of a diesel hammer (type D62).

Then the tubes were emptied and immediately filled with concrete.

After the pile tubes the pile sheets were rammed in, i.e. three sheets of 0.60 m by 9 m per opening.

Next the bottom protection was placed consisting of a geotextile with fascines and ballasted with calibrated concrete rubble from the demolished quay wall head.

The connection between the bottom protection and the old wall and the sheet piling was penetrated with hydro-concrete.

In the top surface of the beheaded massive quay wall holes were drilled (depth 3.5 m) to anchor the concrete slab to be constructed afterwards. These holes were alternatively drilled at 40° and 50° angles to avoid a preferential shifting plane.

— Superstructure.

Prefab concrete was used as much as possible. The formwork for the concrete slab consisted of prefab concrete sheets with sizes 10.63 × 2.86 × 0.20 m and a weight of 15 tons. The front surface of the new quay wall was also prefabricated, with sizes 2.60 × 2.80 × 0.50 m and a weight of 9.5 tons.

In these prefab skirts all fittings for the quay attributes had been cast. Beside a standard reinforcement, also 35 kg/m³ steel fibers were added to increase the shock resistance. The prefab skirts were fixed to the prefab sheets with adjustable bolts to enable a perfect lining of the quay wall front.

After the prefab elements had been placed the reinforcement was put into place and the concrete slab was cast, in two layers. Then the upstands on the slab were made: 2 for the crane rails and 2 for the railroad track.

After this the 4 ground anshors were fitted per section of 22.96 m. Each ground anchor has a service load of 575 kN and a gradient of 35°.

Finally, after the fixing of the impermeable skirt the sand layer was placed and the quay paving consisting of 20 cm plain concrete.

2nd and 3rd stage

The second stage has a length of 459 m and is identical to the first stage in all other respects. To avoid the whole length being unavailable for operations, this stage was split up in two parts.

The third stage will cover 314 m. For operational reasons it will also have to be divided into two parts.

4.6. Renovation quays located at Northern Shipping (zone between 3rd Harbour Dock, Albert Dock and 2nd Harbour Dock) : Deepening of quay walls with the VHP technique

Contractors: **Hydro Soil Services N.V. — Van Laere N.V. — Smet Boring N.V.**

In the vicinity of the facilities of **Northern Shipping** a new method for the underpinning of a massive quay wall by means of the VHP technique is applied.

First the principle will be presented; next the execution will be described in more detail based on the test project carried out in Antwerp and the specific advantage of this technique will be mentioned. Finally a description will be given for which quays and why this technique will be applied in the framework of the renovation plan for the port of Antwerp.

4.6.1. *Principle of the method*

A hole is drilled through the massive quay wall from top to bottom. Through this hole a drilling rod is introduced and drilled into the ground under the quay wall.

The underside of the drilling rod is provided with two nozzles through which a mixture of water and cement grout is injected under very high pressure.

The energy of the jet is so high that the structure of the soil is completely destroyed and all soil particles are mixed in with the injected grout. At the same time the rod is being turned and slowly pulled up. As a result of this helicoidal movement a cylindrical volume of sand-cement-water arises, and after has hardening a pile is obtained.

This so-called VHP grouting or jet-grouting technique is applied as follows for the deepening of a quay wall: under the front surface of the quay wall a groundproof sheet is realised by juxtaposing VHP piles. In fact a double screen is realised for the sake of safety.

Additional piles are made under the centre and under the rear side of the quay wall.

In this way the quay wall is put on piles and extended downward.

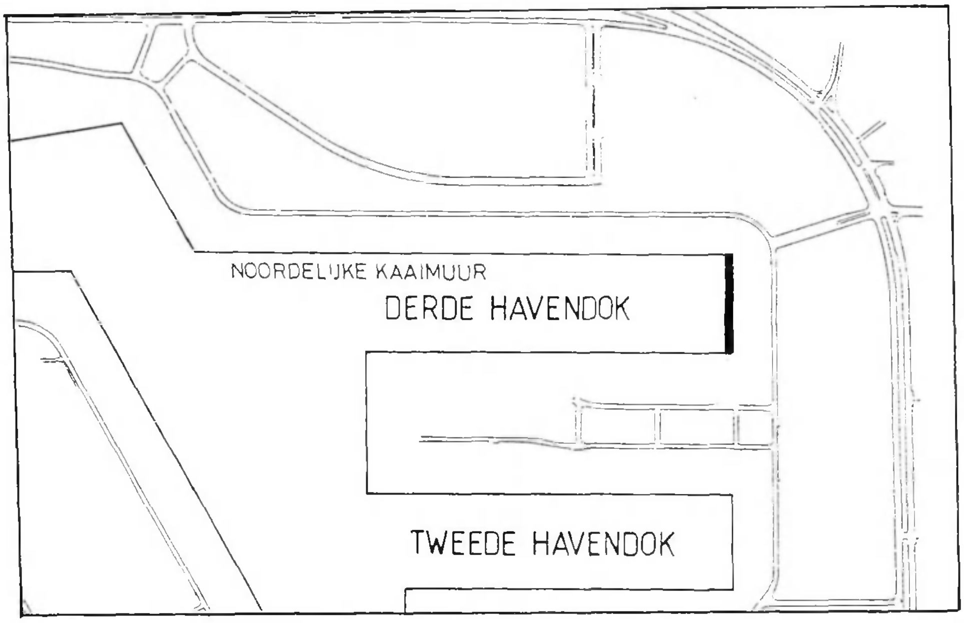


Fig. 82 — Location of the test project.

Because the quay wall has become higher the ground pressure increases. This increase is absorbed by ground anchors or by a superstructure in reinforced concrete on a series of load bearing and tension piles.

Most massive quay walls in Antwerp have a toe that protrudes into the dock under the dock bottom. This toe has to be removed.

4.6.2. *3rd Harbour Dock test project*

Contractor: Joint venture **Hydro Soil Services N.V. (H.S.S.)** — **Van Laere N.V.**

— *Location*

The VHP technique is rather new; this technique applied for the underpinning of quay walls, however, is *completely* new. There is no precedent where the VHP piles come free and become part of the front surface of the quay wall. For this reason the decision was taken to carry out a test project at full scale but over a limited length before this technique — if the test succeeded — was applied on a larger scale.

This test project was carried out at the transverse quay of the 3rd Harbour Dock (quay no. 170).

The underpinning to be implemented is 3 m, the length of the test project is 40 m so that, considering the necessary underwater slopes, 20 m could be dredged clear up to the full 3 m. The existing water depth of 11.70 m was to be brought to 14.70 m.

The ground retaining sheet is double and consists of VHP piles at 0.80 m axis to axis (afterwards it was found that the piles had a diameter of 1.05 m) and at a gradient of 4 degrees. The other VHP piles are grouped in transverse sheets, 6 m axis to axis. The ground anchors are executed 3 m axis to axis and symmetrically with regard to the transverse sheet.

Fig. 83 shows how modest the renovated quay wall is, compared to a Panamax type vessel.

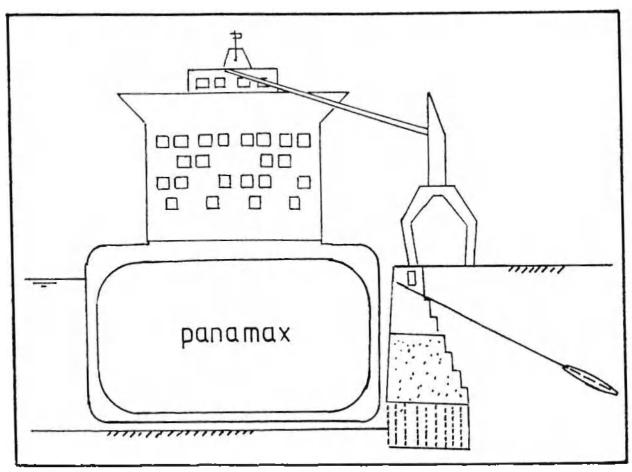


Fig. 83 — Quay wall versus Panamax vessel.

Fig. 84 and 85 give the schematic cross-section of the quay wall before and after the underpinning.

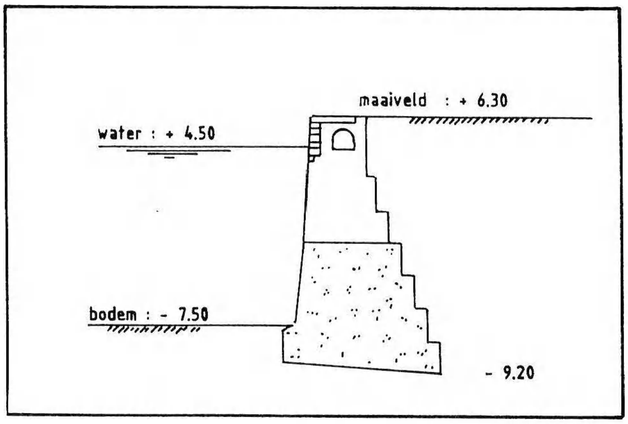


Fig. 84 — Cross-section quay wall before renavation.

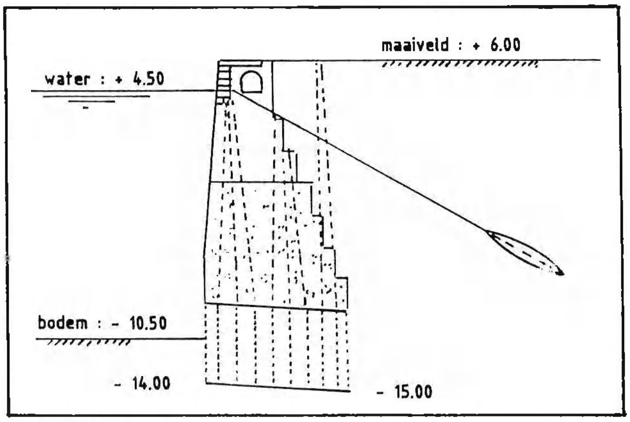


Fig. 85 — Cross-section quay wall after renovation.

Stages of execution

1. Pre-drilling through the quay wall (Fig. 86)

As a consequence of obstacles near the surface and the presence of a cable tunnel, the drilling pattern for the transverse sheet piles is quite complex. At quay level the holes are grouped, but diverge downward.

The holes for the ground retaining front sheets have a gradient of 4° to follow the sloping front of the quay wall.

The precision of the drilling is very important in view of the groundtightness. The direction of each hole was monitored with a computerized inclinometer.

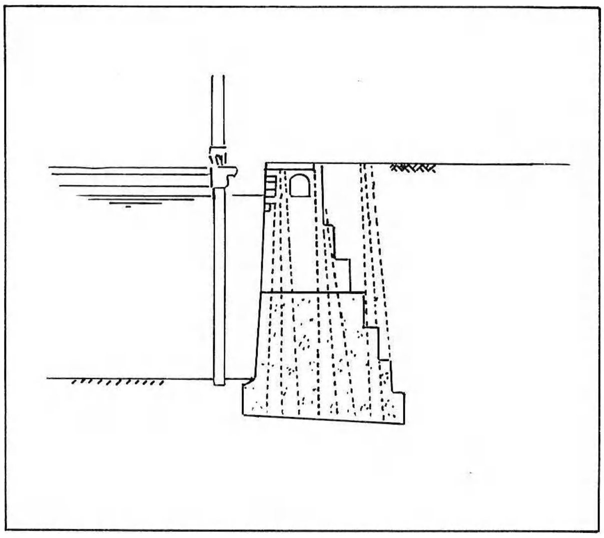


Fig. 86.

2. Pre-drilling for the ground anchors (Fig. 87)

Beside the hole for the ground anchor itself, a core drilling with a dia. = 0.50 m is done for the blind head of the ground anchor. This diameter is determined by the size of the distribution slab, determined itself by the pressure drag of the brickwork.

Afterwards the drilled core was remounted so that the appearance of the wall has hardly been altered.

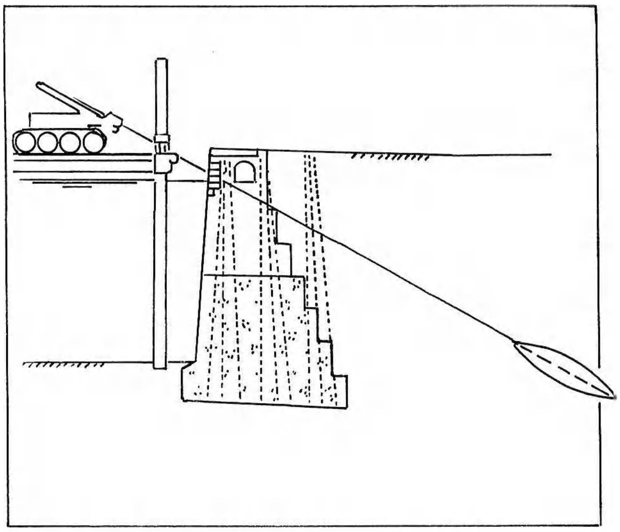


Fig. 87.

3. Execution of the ground anchors

— Introduction of the ground anchor into the hole pre-drilled at 35°;
— Grouting of the anchoring root;
— Tightening the anchor after 28 days hardening;
— Application of corrosion protection of the anchor head and restoring the quay wall front surface.

The works of items 3 and 4 are executed from the water, so no vessel can be moored at that location. But as the execution of these works is functionally independent from the other actions, the planning can be made in function of the arrival of vessels.

4. Construction of the double ground retaining diaphragm (Fig. 88).

— First the piles of the first ground diaphragm are executed. The mono-jet is used in this case. The cement consumption is higher, but the strength obtained from the hardened grout is also greater. This greater strength was considered necessary because this first ground screen comes in direct contact with the dock water, and is subject, among others, to sand loaded jets of water from the vessels' propellers.

This first front screen must be erosion resistant. A pressure drag of 10 to 15 N/mm² was guaranteed. Afterwards a pressure drag of 50 N/mm² was ascertained.

— The second front diaphragm is needed to guarantee the groundproofness of the whole structure. This second screen is carried out with the bi-jet technique. Besides grout also air is injected, leading to a lower cement consumption — and a cheaper production — while the obtained pile diameter does not descrease.

The pressure drag of this second screen is lower, but still amply sufficient to resist the loads.

— The prior quay wall examination had shown that the lowest meter of concrete was of poor quality. Therefore the VHP grouting was continued *into* the concrete of the quay wall.

— A steel reinforment rod, 40 mm diameter, is introduced into the pile immediately after being grouted to connect the brickwork, the concrete and the VHP pile.

5. After the groundproof longitudinal diaphragms the transverse screens were carried out.

Originally isolated piles were proposed under the centre and the rear side of the quay wall, but the study showed that they had to be grouped in transverse screens. The distance between them is 6 m.

Fig. 89 shows the relative position of ground anchors and transverse diaphragms.

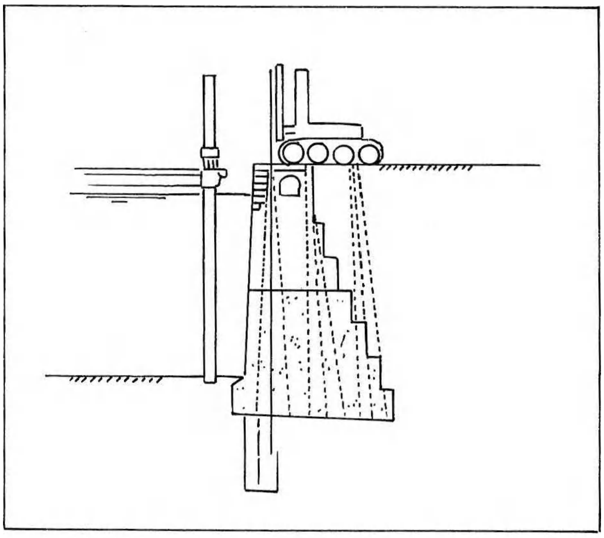


Fig. 88

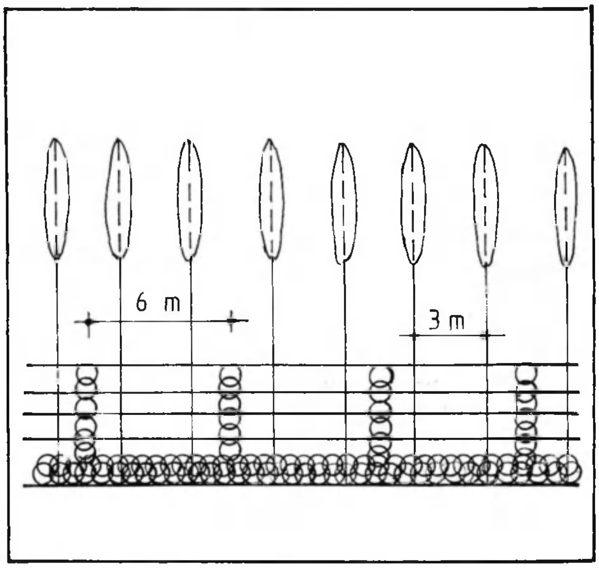


Fig. 89 — Relative position of ground anchors.

6. Demolition of the toe of the quay wall (Fig. 90).

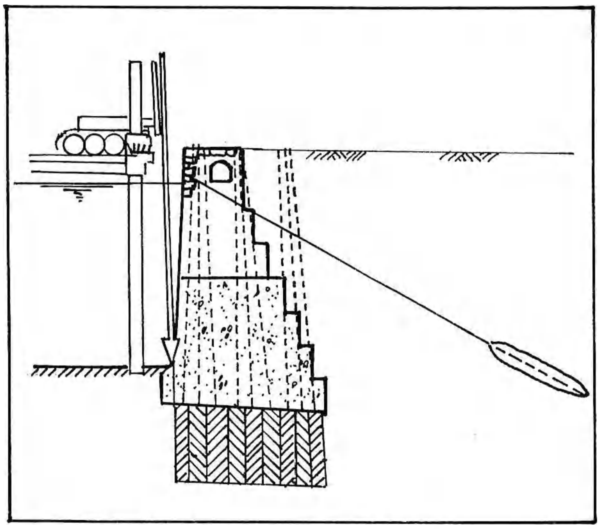


Fig. 90

The toe of the quay wall is removed with the pre-splitting method.

By means of a vertical holes are drilled in the toe. The rig is needed to obtain sufficient accuracy.

7. Dynamiting the toe (Fig. 91).

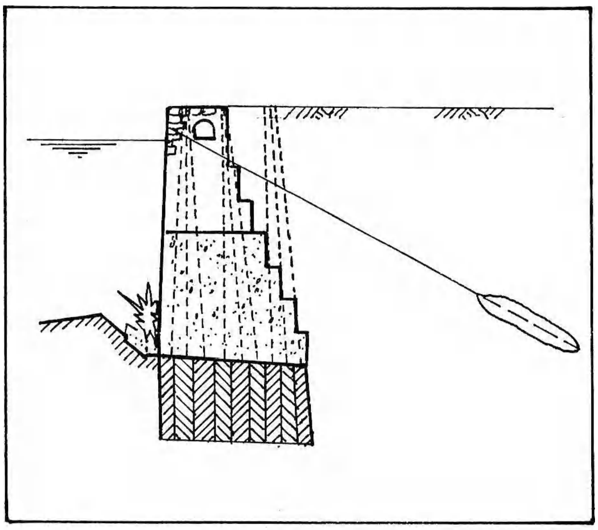


Fig. 91

By way of experiment two kinds of dynamite were used: a standard kind (**Nobel** manufacture) and a dynamite with a very low explosion speed (**Sumitomo** manufacture).

The first type is cheaper and easier to use. Though the vibrations were higher, they remained acceptable.

8. When this was completed the dredging works were carried out.

Calculation — dimensioning — design

In its final state the ground anchors. The admissible particle pressure, the overturning balance, the securing against general gliding was checked under influence of ground and water pressures, own gravity, payload and pre-tension forces in the ground anchors.

The particle pressure onto the foundation foot is acceptable on condition that the overall length of the foundation is bearing.

To dimension the ground anchors a uniform quay load of 60 kN/m² was accepted, just like everywhere else in the port of Antwerp — right bank.

The resulting load is 650 kN per anchor, a value that is acceptable for anchording in a compact sand layer.

To limit the permanent pretension of the ground the anchors are fixed at 520 kN. Each anchor was tested at 1.5 × 650 kN.

One anchor was tested up to 1,300 kN to check the security against failure of the anchoring. The free anchor length was also checked.

The reinforcement of the ground anchors consists of Macalloy 835/1030 bars. Under working conditions the tension ise less than 60 % of the 0,1 % test-tension.

Possible loss of pre-tension in the course of time will be checked.

Since the ground anchors are permanent, much attention was given to the protection against corrosion. A double protection was foreseen both over the free length and over the active and passive zones of the anchoring.

Tests

Many tests and checks were performed for this test project. They are listed briefly:

1) deep-soundings on the water and on the shore, close to the quay wall and further away in the zone of the ground;

2) drillings on the water and on the shore, with ground mechanical tests on undisturbed samples;

3) vertical and slant core drillings over the full length and width of the quay wall, followed by tests on the cores obtained.

   Also diagraphy drillings, providing qualitative data which can be related to the quantitative results of the tests on cores;

4) the precision of the drillings in the quay wall was monitored with a mobile inclinometer coupled to a computer;

5) regarding the ground anchors: fitness test, tensioning and releasing test, measurements of the extension of each anchor;

6) from the VHP piles cores were drilled to check the pressure drag of the grout;

7) during the grouting all parameters were being recorded continously, like rotation speed, injection pressure, speed at which the injection tube was pulled out, cement consumption;

8) the resistance of the VHP piles against erosion was tested in the **Hydraulics Laboratory** at Borgerhout, whereby the speed of sand loaded water under influence of propeller jets was simulated;

9) during the dynamiting of the quay wall toe the vibrations were measured;

10) the movements of the quay wall were followed (and will be) by topographic measurements of the bench marks;

11) in two anchor heads anchor tension meters were built in, so that the evolution of the anchor tension in time can be monitored;

12) after the dredging clear of the deepened section of the quay wall a full examination was carried out by divers, with an underwater video recording of the broken surface of the quay wall toe and of the part of the VHP piles come free.

Advantages of the method

An important advantage of the described method is that the front surface of the quay wall remains in place unaltered. This advantage plays a role when a quay wall must only be partially underpinned, e.g. in case of an underwater slope as transition between a deepened and a non-deepened section of a dock.

If the renovation technique with a sheet piling were applied, the quay wall lining would be offset having as consequence that no vessel could be moored any longer.

A second important advantage is that the quay surface is practically not affected. The quay facilities — crane rails, railway tracks, drain, conveyor belts, conducts, cables etc. — remain untouched.

The working zone is very limited and the cranes can pass by.

In combination with the fact that the front quay surface remains in place this means that vessels can still be moored and that the loading and discharging operations are not disrupted.

The works which have to be carried out from an elevator platform on the water are the drilling and fitting of the ground anchors and the drilling for the demolition of the quay wall toe.

These works are functionally independent from the other works and can therefore be planned exclusively in function of the arrival of vessels.

This second advantage means that the berth can remain in service, leaving apart a minor hindrance. This advantage is of major importance in case of a renovation, which per definition is carried out on an existing quay.

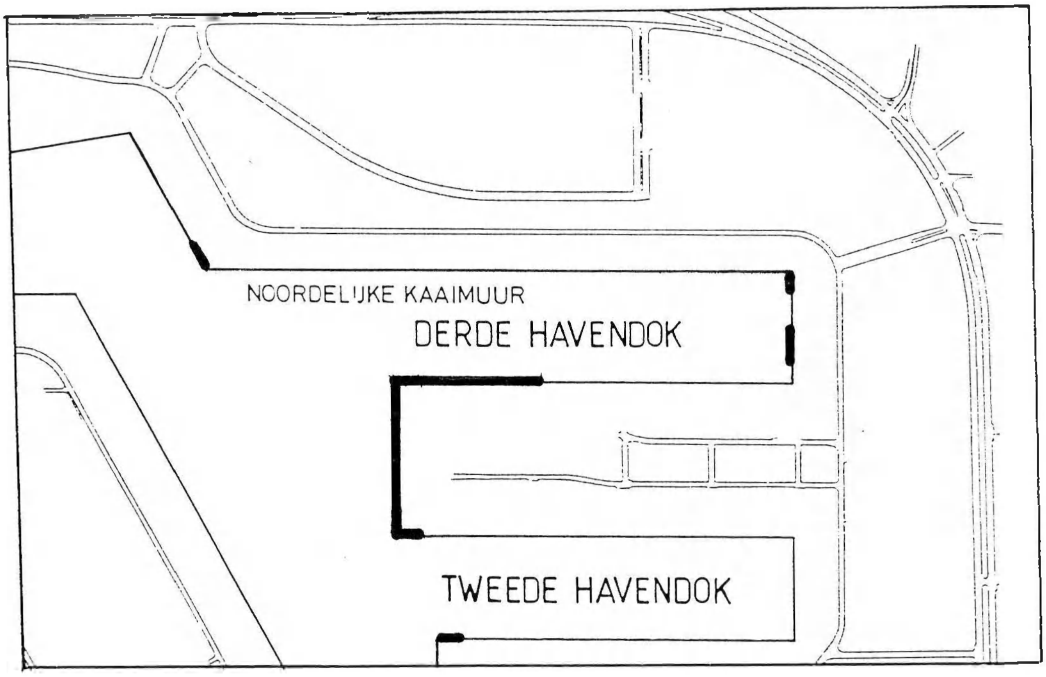


Fig. 92 — Location of the VHP technique in the renovation plan for the 2nd and 3rd Harbour Dock and Albert Dock.

4.6.3. *Application of the VHP technique in the renovation plan*

Contractor : **Hydro Soil Services N.V. (H.S.S.)**

After the test project had led to the conclusion that the underpinning with the VHP method is technically possible and offers sufficient security, the Port Enterprise decided to adopt this technique for the renovation of different quay walls (Fig. 92).

At quay no. 188 and between quays no. 152 and 130 of the 2nd Harbour Dock an underwater slope has to be realised as a transition between the Albert Dock which has to be deepened and sections which do not.

Therefore these quay walls must be underpinned over a certain transition length. Adoption of the VHP technique allows the transition berths to remain in service for the handling of vessels, as the front surface remains unchanged.

This is not so when a sheet piling had been chosen. In the case of quay no. 188 the two berths of quays 188-190 would have been reduced to only one. This cannot be the intention of a renovation project.

Also the transverse quay of the 3rd Harbour Dock has to be underpinned with the VHP method. In doing so the test project could be integrated in a useful manner, no part of the 3rd Harbour Dock is lost, and not the whole transverse quay has to be deepened.

At the south quay of the 3rd Harbour Dock and at the transverse quay between the 2nd and the 3rd Harbour Dock terminal operator **Northern Shipping** has its seat. They handle fertilizers and other bulk goods.

The underpinning with the VHP technique allows vessels still to be handled during the works. The company has an extensive superstructure of specialised silos connected with conveyor belts. It cannot move to another quay — even if one had been available.

Deepening by means of a sheet piling placed in front of the existing quay wall would have led to a considerable exploitation loss caused by the non-availability of berths.

The same technique is used as a transition construction along the opposite bank between existing quays, and for the new quay wall of the widened fairway between the Albert Dock and the America Dock where **the Flemish region** is the principal.

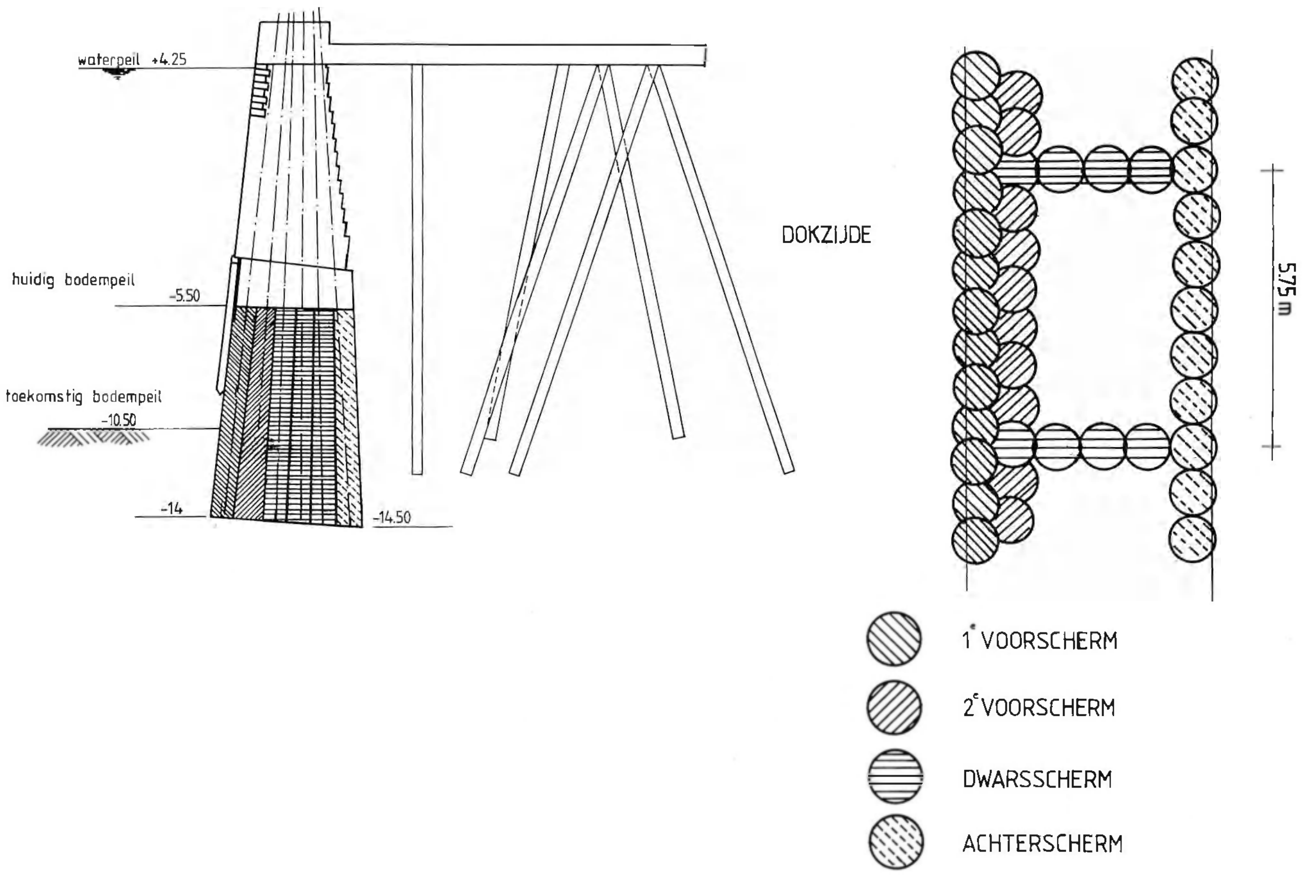


Fig. 93 — Passage America Dock — Albert Dock. Execution of transition sections.

The execution of these transition sections is more complex than the similar project carried out before, mainly because the bottom deepening is 5 to 7 m in this case, whereby the VHP piles themselves have to be dredged clear more than 5 m.

If we adopt VHP grout piles in combination with ground anchors for this deepening, the arising moments and tensions, both in the VHP grout piles and in the quay wall itself, are rather high.

In applying the VHP grout piles in combination with a relief slab, load bearing and tension piles a better transfer of the loads is obtained in this instance, resulting in considerably lower moments (about half compared to the solution with anchors) and tensions in the lower section of the quay wall.

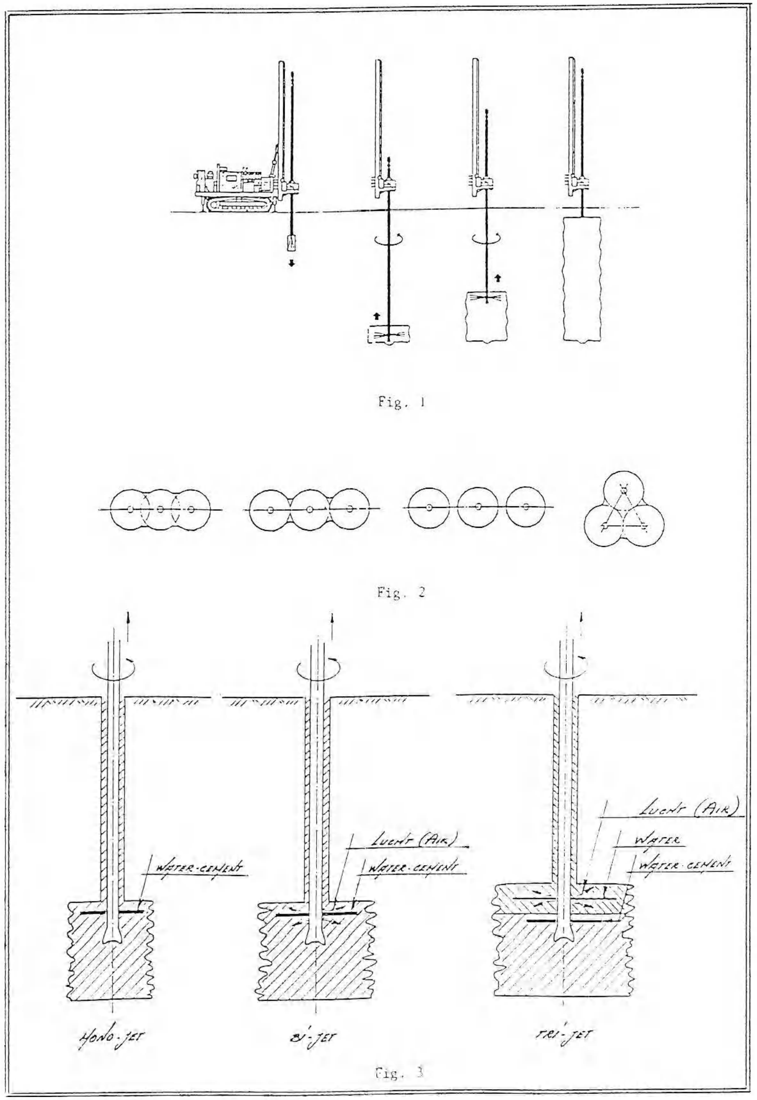
The use of a double row of piles as front surface with a rear diaphragm and transverse screens offers more advantages in this case than the so-called comb form (without rear screens).

The two front pile rows form once again a double groundtight diaphragm, whereby the rear screen guarantees a maximal groundtightness.

By adopting this combination of VHP piles the tensile and bearing forces are minimal and evenly distributed.

The use of a closed form is most similar to that of a gravity wall with the additional advantage that the ground inside this form is co-operating to a maximum extent.

Taking into account the difficult application method of these transition sections, and based on stability calculations and the experience gained from previous renovation works carried out with the VHP grout technique, the closed VHP form in combination with a relief slab, tension and load bearing piles is the best solution for these deepening works.



V.H.P. - GROUTING

Method of execution

General

To execute V.H.P. - grouting, one proceeds as follows (e.g. fig. 1) :

- in a first phase, a drilling rod is lowered to the desired depth.
- as soon as the desired depth has been reached, a mixture of water and cement grout is injected in the ground under very high pressure (200 to 800 bar) - through two nozzles. The energy of the jet is so high that the structure of the soil is completely destroyed and all soil particles are mixed in with the injected grout. After hardening, a homogeneous grout column is obtained.
- before hardening occurs, it is possible to insert a steel bar as long as the underground contains no stones or other hard inclusions.

The grout columns can be placed in arbitrary arrangements, in function of the goal or the expected low. (e.g. fig.2)

To increase the jet radius, the jet-stream can be covered with an air layer. This is called a BI-JET (cement + water and air mixture) (e.g. fig. 3).

A third execution method shows the ground being cut with a waterjet stream coverd with an air layer. A cavity is thus created in the ground. At a certain distance (approx. 30 cm) below the water-air-jet, the water-cement-mixture is injected in the obtained cavity. This way of injecting is usually done with low pressure (approx. 30 bar) and is called TRI-JET (water, air and water-cement mixture).

Fig. 94 — Very High Pressure grouting. Principle.

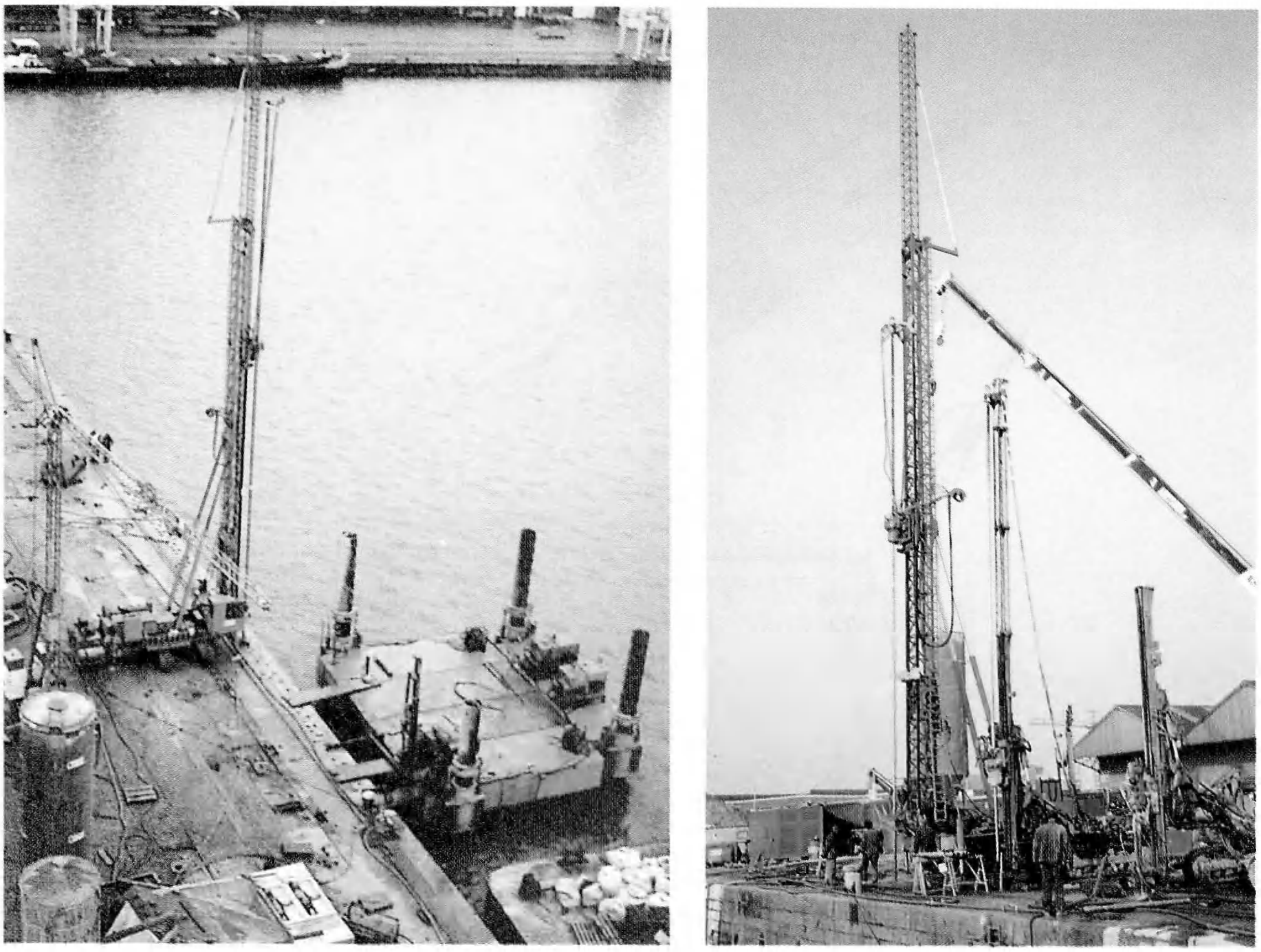


Fig. 95 — Very High Pressure grouting. Application.

4.7. The Shelter Dock for Lighters at the north side of the America Dock

Contractor : Joint venture "AMAL" (**Van Laere N.V. — J. De Nul N.V. — Besix N.V.**)

Necessity

The construction of the shelter dock for lighters north of the America Dock is an essential part of the adopted renovation programme.

In the port of Antwerp there has been a need for a proper shelter dock for inland navigation for many years.

Lighters and push towings which have to remain in the port for some time between loading and discharging operations do not find a mooring place reserved for inland navigation only.

The present waiting dock for lighters between the 1st and the 2nd Harbour Dock is much too small; moreover, it will be refilled soon under of the renovation plan for the port of Antwerp.

As far as mooring opportunities is concerned, inland navigation in the port of Antwerp has always had to give way to ocean navigation.

The docks south of the Albert Canal — Royers Lock line are completely out of date and unsuitable for modern push convoys as they provide a shelter mainly for ships which are no longer in use.

The surface area of the planned dock is 65,000 m².

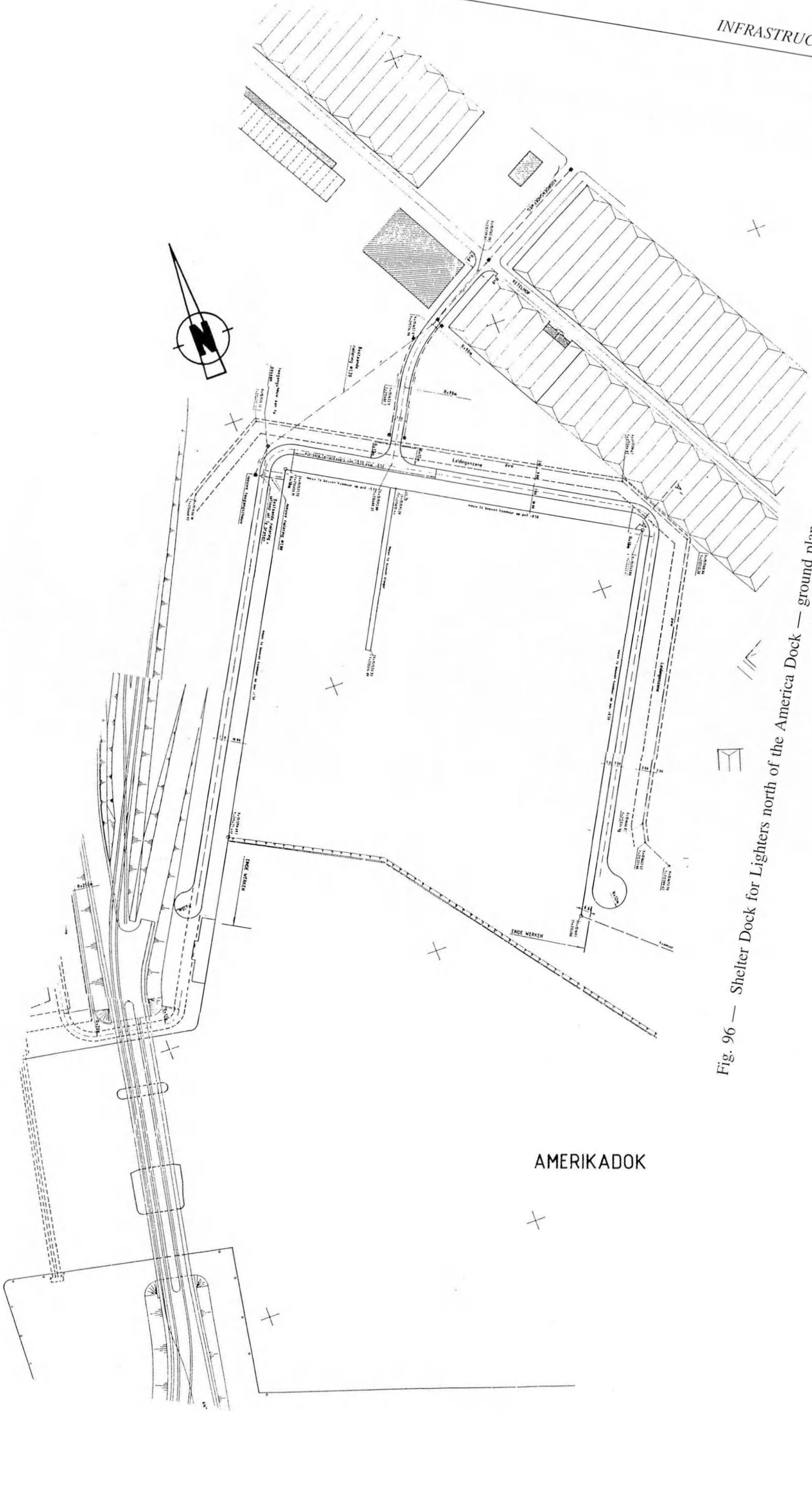


Fig. 96 — Shelter Dock for Lighters north of the America Dock — ground plan.

The shelter dock is constructed with a high-founded quay wall with a quay platform at TAW + 8.50 on one side, and a quay wall with a low quay platform at TAW + 5.50 on the other side.

The high quay wall will be of the "Danish" type, built on metal sheet pilings anchored with tension and load bearing piles.

The low quay wall will be kept in place with drilled anchors.

The zone with the high quay platform will be reserved for push convoys, the other zone for normal inland navigation.

The mooring zones are separated by a landing stage. The stage has 30 m long spans consisting of 2 running metal girders and a floor out of metal grids.

This construction is mounted on metal pile tubes with an outer diameter of 1,480 mm.

The works were started in January 1993 and are to be completed at the same time as the general contract of the widening and deepening of the passage between the America Dock and the Albert Dock, i.e. in January 1994.

4.7.1. *Soil contamination*

During the construction of a new quay wall at quay no. 57 when the access to the Noordkasteel bridges was widened, the ground and the groundwater in the vicinity of the projected shelter dock for lighters were found to be contaminated.

The **Flemish Institute for Technological Research (VITO)** and the engineering consultants **Betech** were commissioned to carry out a thorough investigation on ground and groundwater samples of the Noordkasteel soil. It confirmed the visually ascertained contamination.

This contamination was probably caused by the facilities of the former petroleum harbour which were located there between 1870 and the 1st World War.

Based on **Betech**'s study and in consultation with **O.V.A.M. (Realisation and Sanitation Dept.)** a solution was worked out to clean the soil and store the contaminated soil within the available dumping areas of the whole renovation project in a way harmless to the environment.

An extension of this investigation further revealed that the soil to be excavated within the perimeter of the projected Shelter Dock was also contaminated, but to a lesser extent.

Based on the results and the recommendations of this report and on discussions with O.V.A.M. it is also possible, on condition that a number of measures and precautions are taken, to sanitate and store the soil within the current renovation project. Several alternatives have been examined and discussed with O.V.A.M.

Finally the following working method was selected with the agreement of **O.V.A.M.**:

1) The purification of the groundwater will take place after the new quay wall has been constructed.

   The groundwater, pumped up for dewatering purposes, will be taken to a mobile purification plant with airing and settling basins. The residue will be transported to a specialised processing plant.

2) Soil from the upper layers, where contamination is the greatest, will be treated and placed in storage in accordance with the degree of its contamination.

   A small quantity of the soil may be treated with active lime and transported to a specialised dump.

3) As soon as the groundwater pumped up has the same quality as the dock water, the soil will be used to refill the 1st Harbour Dock and the old shelter dock for lighters.

   The presence of a practically impenetrable mud layer on the bottom of both docks will seal off the lightly contaminated spoil between the existing quay walls and the new quay wall.

   When the contaminated spoil has been dumped the dock will be covered up with a layer of clean sand about 4 m thick, reclaimed from the dredging works of the deepening and widening of the passage.

In concert with O.V.A.M. the whole working method will be followed up permanently by an environmental expert from specialised consultants, so that the necessary tests and measurements can be carried out during the execution, and technical instructions can be given to adjust the method if necessary.

The shelter dock itself will create safe mooring places for modern inland navigation.

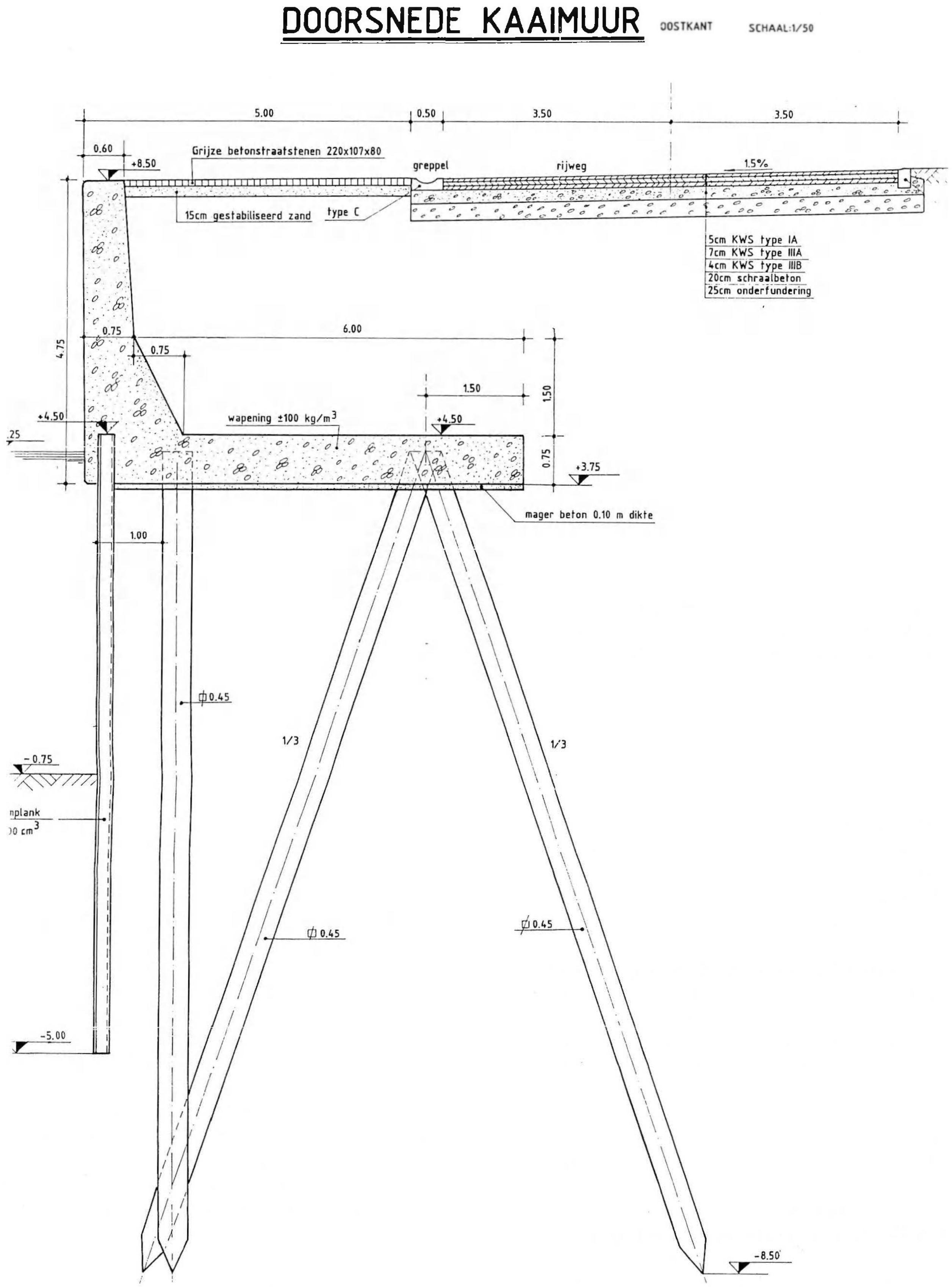


Fig. 97 — Cross-section of the high-founded quay wall.

Fig. 98 — Royers Lock.

5. EXTENSION OF THE PRESENT RENOVATION PROGRAMME

The renovation of the central port area will not be completed with the implementation of the present renovation programme.

As a consequence of reorganisations and take-overs in a number of port enterprises and the termination of some concessions, rearrangements of terminals are planned and new functions and concessions foreseen in the Leopold Dock and the America Dock.

Potential operators are prepared, however, to make large investments if the docks offer berths deep enough to receive modern vessels with a draught of up to 40'.

Considering these requirements, we may assume that in the next few years (1994-1995) the present renovation programme will be continued with the renovation of some hundreds of metres of quay wall in the America Dock and the Leopold Dock.

The most important and urgent work to be undertaken, however, is the renovation of the Royers Lock.

This lock, built in 1907 and situated downstream of Antwerp, offers a direct connection to the renovated port area around the America Dock and the Albert Dock and in particular to the Albert Canal.

The size of the lock (180.00 x 22.00 x 6.40 m) permits the transit of coastal and inland navigation ships. The limited length and width of the lock-chamber, however, cannot accomodate modern push convoys.

Due to its excellent location in the port area, this lock, together with the still older Kattendijk Lock (built in 1870), is used by the majority of barges to leave and enter the port of Antwerp.

The following survey lists the number of ship moves through both locks in the year 1991:

	IN		OUT	
Coastal navigation:	*Qty*	*DWT*	*Qty*	*DWT*
Royers Lock	545	681,178	516	690,103
Kattendijk Lock	—	—	—	—
Inland Navigation:	*Qty*	m^3	*Qty*	m^3
Royers Lock	11,134	9,248,592	13,504	10,672,530
Lockings:	*Qty*			
Royers Lock	8,151			
Kattendijk Lock	1,740			

The lock needs replacing for the following reasons:

1. Both the civil engineering (chambers, gates, etc.) and the electro-mechanical equipment are completely worn out and need replacing. Spare gates and parts are no longer available, since entire replacement has been anticipated for years. Damage to the gates could have disastrous consequences.
2. The current size of the lock does not meet requirements by modern inland navigation, i.e. push convoys.

The construction of the new lock will be a difficult and delicate venture, given the volumes to be handled and a number of extremely strict additional conditions.

A nautical pre-feasibility study stated that the new lock would be best located at the same place as the present lock, both with respect to access from the river and traffic flow in the docks. A new construction in the same place means that other locks will have to take over the workload, leading to longer sailing distances and congestion.

Available land space limits the lock extension, but more so the presence of important industrial constructions which cannot be demolished and have to remain in operation, e.g. the grain silos of **Samga N.V.** to the north and the Vosseschijn pumping-station to the south.

Many road transports to and from the port use the roads around the lock. The location of the lock does not permit a smooth traffic flow, so that traffic congestion will be a concern for years to come. Therefore the method of replacement must also offer a solution to traffic congestion.

Bearing all these factors in mind, special construction techniques are being studied in order to:

1. limit the construction time to the minimum;
2. maximise the use of the present lock during the preparatory works and the construction of the new Lock, particularly during the
   - 2.1. conservation and strengthening of the northern chamber wall;
   - 2.2. construction of the southern chamber wall;
   - 2.3. tunneling for and construction of the gate heads;
   - 2.4. conservation of the existing constructions: pumping-station and silos;
   - 2.5. the study for the continuation of the traffic flows during the whole construction period.

With these pre-conditions the construction of this lock cannot be regarded a standard sea-lock construction. Based on preliminary study work a project team has been formed which is now studying the renovation of the Royers Lock. Construction should start in 1995 at the latest.

A lock accomodating convoys will provide the best opportunities for the development of the southern port area, and is necessary to capitalise on the investments made in the Albert Canal to accomodate large push convoys.

The new lock will relieve the big locks and surrounding docks and contribute to a safer and faster navigation flow, because a number of vessel types, in particular the larger inland navigation, will have a better access to the port.

Finally, it should be mentioned that in the medium term, and especially after the renovation of the Leopold Dock, the realisation of the short and direct route via the north side of the Island will be necessary. This route will be constructed by widening and deepening the narrow passage between the Leopold Dock and the Albert Dock. two bridges, namely the Wilmarsdonk and the Austruweel bridge, span this passage and form the link, both for road and rail, between the Island and its hinterland. The masterplan being drawn up plans replacing both bridges by a single bridge located at the Wilmarsdonk bridge. It will span about 65 m.
This bridge will serve mainly the rail traffic to and from the shunting yard and also the local port and road transport.

For faster and through traffic a tunnel for cars is planned under the deepened and widened fairway between the two docks. Removing this bottleneck, however, requires a large investment, estimated at 5 to 7 billion belgian francs.