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Quay Walls of the Port of Rotterdam

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ABSTRACT

This paper gives an overview of the development of the design of the Rotterdam quay walls. Because of the increasing dimensions of the ships during the last 40 years the retaining height of the quay walls increased as well. The larger height of the wall required heavier sheet piling, higher anchor forces and thus more intensive pile fields. This process of up-scaling resulted in the 70s in large displacements of some of the sea quay walls for coal and ores. The behavior had a decisive influence on the later designs. In the paper the present design philosophy and a new safety concept is presented. Furthermore different optimisations are shown. It appears that the constructions built according to the actual design are functioning properly and that the costs did not increase over the last ten years.

Key Words: Harbor, quay wall, sheet piling, tension pile

1. INTRODUCTION

The Netherlands is situated in the Western part of Europe at the boundary of the North Sea. This basin was filled up with sediments during the Pleistocene (diluvium), and the Holocene (alluvium). These predominantly fluvial and marine sediments were deposited by the rivers Rhine and Maas. In the Western part of the Netherlands the ground level is about or some meters below mean sea level.

The city of Rotterdam is located in the Western part of the Netherlands at the banks of the Rhine. The situation of these rivers and Rotterdam is shown in Fig. 1. The harbor of Rotterdam is the largest in the world as far as cargo throughput is concerned.

The main factor for development of any harbor and its facilities are the shipping activities within the harbor area. Therefore the Port of Rotterdam has been developing its facilities over the past 100 years in accordance to the development in shipping. These developments reached its peak in the end of the 60s resulting in larger ships and thus greater water depths. This in turn had major consequences for the construction of harbor basins, quay walls and waterways which are mainly determined by the dimensions of the ships and the cargo handling facilities. In this paper the development in the design of the quay walls will be presented.

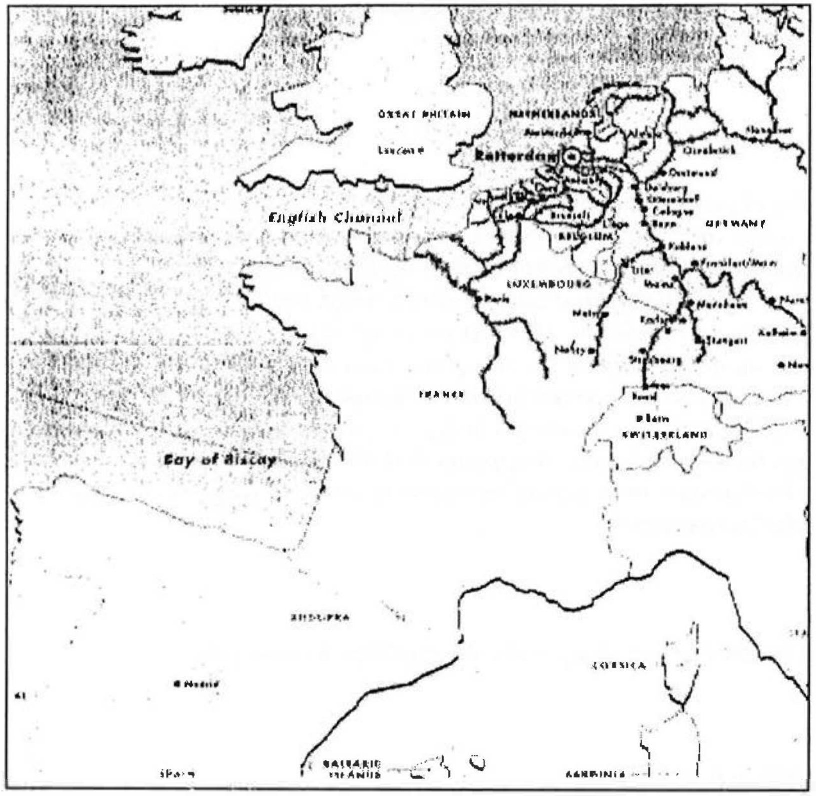


Fig. 1. The Netherlands, Rotterdam and the rivers Rhine and Mass

First an overview will be given of the history of the Port of Rotterdam with its shipping activities, important in understanding the development in quay wall design. Next the soil conditions which are always important for the design of retaining structures are described. Subsequently the design process is illustrated from the initial stages of a project up to the monitoring activities when the quay wall is in use. The construction of this kind of infrastructural facilities requires knowledge of several disciplines to optimise the design. The main disciplines involved being geotechnical, hydraulic and structural engineering. The knowledge and experience gained during the past decades has resulted in a so-called "design philosophy" for quay walls. This also includes more recent developments in safety philosophy and computation methods. Finally several aspects of quay wall design are discussed and some examples are presented.

2. HISTORY OF THE PORT OF ROTTERDAM

In the old days landing stages or moorings for vessels were constructed as close to a port's town centre as was possible and were typically simple wooden structures of piles, jetties and quays. Also the origin of harbor of Rotterdam can be found a few centuries ago in the center of the present city of Rotterdam when it was a port of refuge for fishing boats. The 'Buizengat', one of the oldest harbors in Rotterdam, offers a reminder of that time. In the second half of the 19th century a number of harbors were dug on the southern side of the river. As a result of its privileged geographic position the port of Rotterdam has seen ever increasing volumes of incoming and outgoing goods, therefore in time the extent of these harbor areas became insufficient. Furthermore due to increasing dimensions of the ships these harbors did not have enough water depth. Expansion took place in western direction where the river had a greater water depth. Early this century new port basins like the Maashaven, Waalhaven and Merwehaven were built. The actual situation of the harbor is presented in Fig. 2.

The Waalhaven harbor constructed around 1930, was the largest dredged harbor in the world at the time. Initially used for bulk transport of coal the Waalhaven harbor at present is still used as a general cargo and container harbor. After II World War harbors and harbor basins have been constructed to create optimal conditions for merchandising and shipment. This expansion is presented in Table 1. Goods transhipment between the wars reached 40 million tons. Scale grew, and with it the importance of the investments and the thinking. Current annual port transhipment levels are 300 million tons. The quay walls played a major role in making it happen.

A feasibility study and environmental impact studies for the land reclamation for Maasvlakte II are now being carried out. It is foreseen to construct a new industrial area in the period of 2004-2006 of about 1000 ha.

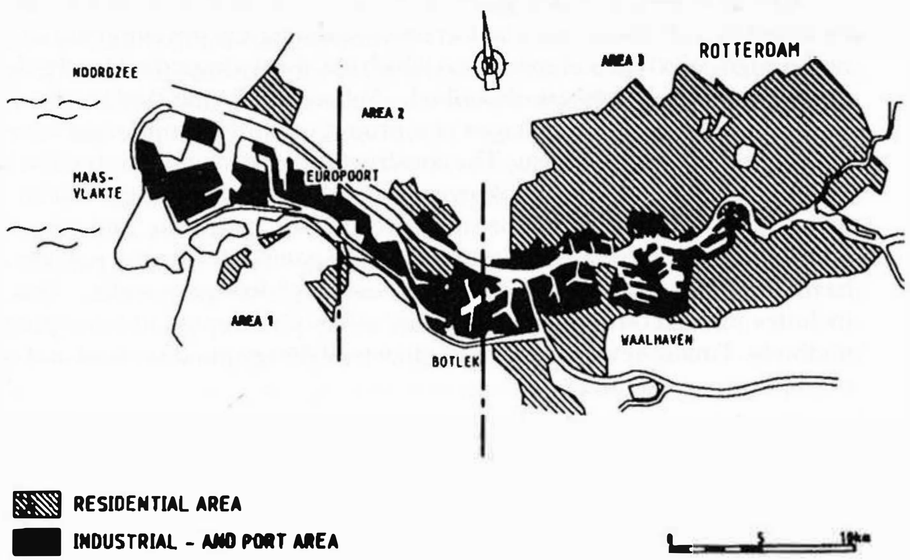


Fig. 2. Situation Rotterdam Port area

Table 1. Expansion of the Rotterdam harbor

Waalhaven	1930
Eemhaven	1950
Botlek	1955
Europoort	1958
Maasvlakte I	1968
Maavlakte II	2004-2006

In the past centuries the transportation of merchandise by ships has changed a great deal. Well into the 19th century ships were made of wood and propulsion took place with the help of sails. Around 1850 the first engines were used for propulsion and wood was replaced by iron. This development lead to the construction of specialised ships. For a long time the so-called General Cargo Ship predominated, a ship with holds containing bales, cases, crates and drums. In fact it was an expanded version of the wooden ships from former days. With the increasing goods carried the economic possibility of specialising the ships also grew. This specialisation reduced loading and unloading times considerably. Besides, it brought an improvement in the utilisation rate. The largest ships in operation at present are bulk carriers which have a length of over 400 m, a width of 65 m, a draught of about 24 m and have a tonnage of 450,000 dwt.

The dimensions of the ships largely determine the design of harbor basins. The draught of the ship determines the required depth of the quay wall; the length of the ship dictates the dimensions of the berths and of the turning

circles that are necessary; and the width of the ship determines the width of the harbor basins. In Fig. 4 the increasing depth of the constructed harbors in time is presented. The earlier mentioned peak in the 60s, with increasing dimensions of the ships, is clearly visible in this graph.

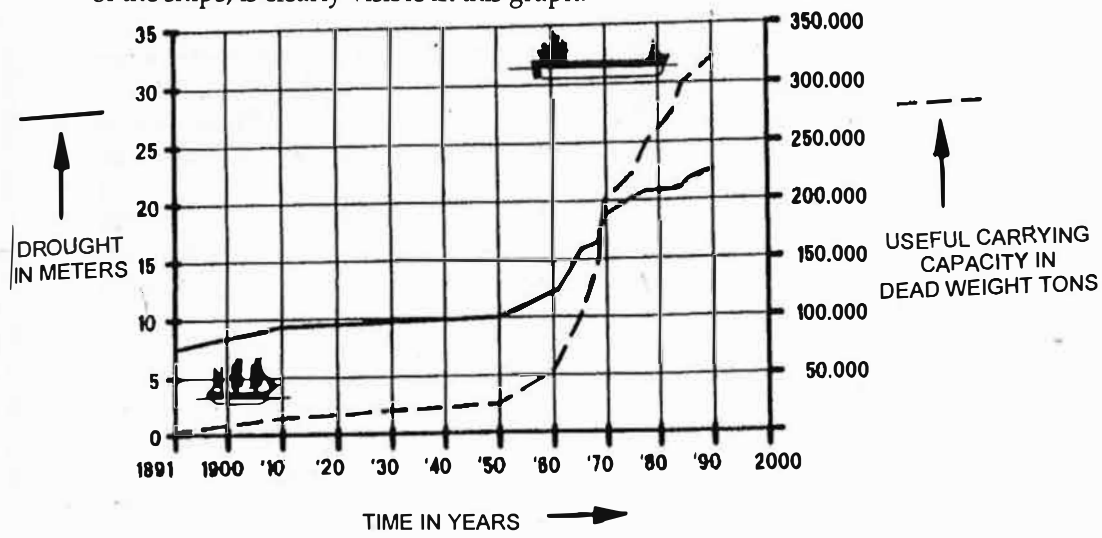


Fig. 3. The development of ship dimensions in time is shown

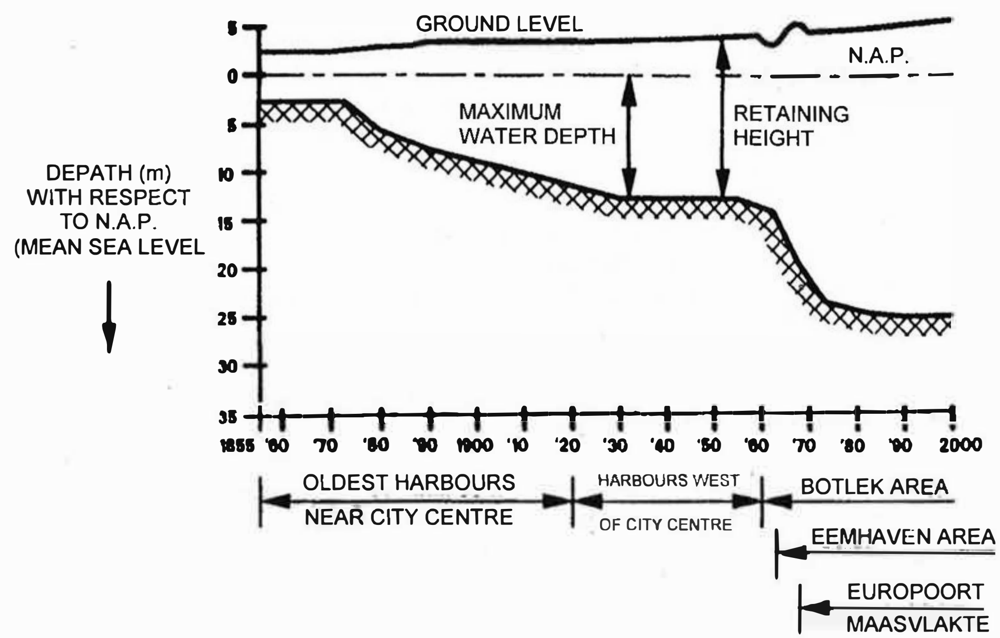


Fig. 4. Development of water depth and retaining height

It is expected that the dimensions of the bulk carriers will remain the same or even become smaller. However for the middle class ships, especially container ships, it is anticipated that their width will increase up to 50 m. The length and draught of these ships are not likely to increase that much in the future.

Quay wall constructions at the end of the 60s and 70s require therefore significantly greater dimensions due to the anticipated increase in ship dimensions. Other determining factors for the design of these type of structures is the increasing retaining height, the development in cargo handling facilities and the type and amount of surcharge on and behind the quay walls. Another development to be considered is the decreasing use of tugboats as ships will increasingly use their own engines for mooring purposes. This may consequently cause additional erosion at the bottom near the quay walls.

All these developments imply that new creative solutions and techniques had to be developed, and will have to be developed in the future, for the design and construction of new quay walls. So far the progress in the requirements for the quay walls. The other important condition is determined by the soil and (ground)water conditions. The subsoil conditions in the Rotterdam area are very important in this respect.

3. SOIL CONDITIONS

The Port of Rotterdam is located near the North Sea in the Rhine-Maas Delta. Characteristic for this area are its meandering rivers and the rise and fall of the sea level in the past. As a result soil conditions can vary significantly over short distances. From about 1700 onward geologically speaking little has changed. However since that time human activities, e.g. several land reclamations such as the Maasvlakte I, have been taking place changing the natural soil conditions.

In geotechnical respect (e.g. soil profile) the Port of Rotterdam, see also Fig. 1, can at present be subdivided in the following three areas:

1. The city area up to the river Oude Maas
2. The area between the river Oude Maas and the Maasvlakte
3. The Maasvlakte area

For each of these areas the result of a typical Cone Penetration Test (CPT), with a corresponding boring log, is shown in Fig. 5. From the boring logs it is evident that the soil profile under the city area consists mainly of very soft Holocene layers to a depth of 15 m beneath Reference Level (RL). Beneath this depth a densely packed medium to coarse Pleistocene sand layer is found. In westward direction the quality, in terms of strength and stiffness, of the Holocene layers improves due to an increasing sand content. Furthermore the top of the Pleistocene sand layer slopes downward, from RL –15 m to RL –22 m, in the direction of the North Sea.

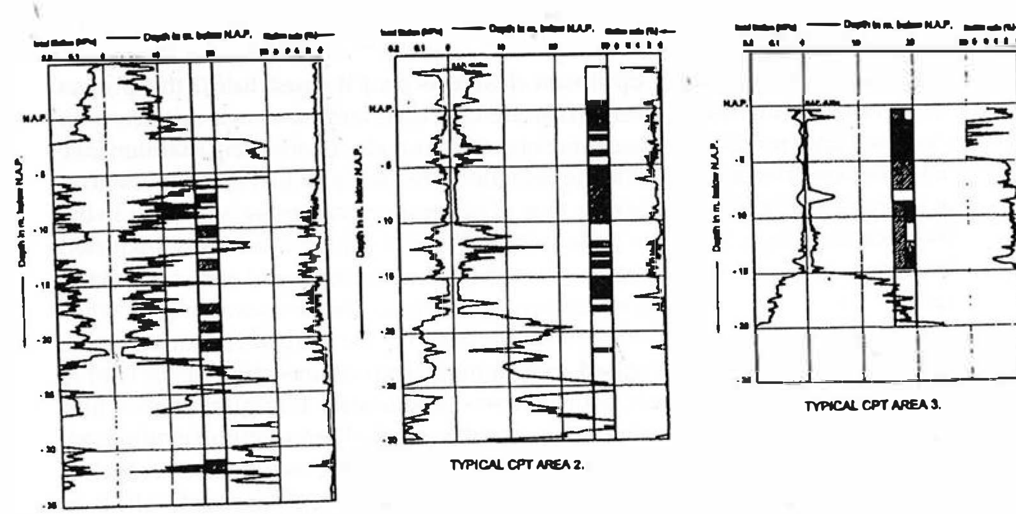


Fig. 5. Typical Cone Penetration Test (CPT) results for areas 1, 2 and 3, in Fig. 2

4. DESIGN PROCESS

The following levels can be distinguished for the design process:

- feasibility level
- preliminary design level
- final detailed design level
- preparation for tender level
- selection of contractor level
- construction level

In the feasibility level the project requirements are established, this is done in consultation with the client. In the Port of Rotterdam the client is usually the Rotterdam Port Authority. The Rotterdam Port Authority manages the harbor infrastructure, leasing the land and water (including the quay constructions) to their clients. Special requirements of these clients can be implemented in the design process.

During the preliminary design phase several alternative design options are considered. After careful consideration one of these options is selected which is then worked out in the final detailed design level. The selected option will be prepared for tender, the tender documents include a work description and drawings. For these kind of projects an open EC-tender is used. In these tender documents criteria are given which the contractor has to fulfill. After a contractor has been selected construction activities can be started. During construction the contractor is required to present detailed working plans, e.g. for sheet pile driving and concreting activities.

5. DESIGN PHILOSOPHY

5.1 General

The main principle of an optimum design is that it must fulfill the project requirements (including the environmental considerations) at a minimum of costs, not only for the design and construction but also concerning maintenance in the future. This requires that the designers are highly skilled and experienced in designing as well as in construction. The required expertise is present in the Engineering Division of Rotterdam Public Works which is a highly qualified institute as most of the quay walls, piers and jetties are designed and constructed under their supervision. Experience in the design of the deep-water quay walls has been gained since the late 60s when the depth of the waterways and retaining height of the quay walls increased considerably. The obtained experience lead to a design philosophy which is of course continuously being evaluated and improved. The design concept in Fig. 6 is the result of this ongoing evaluation. The first quay wall, the 2nd phase of the EKOM quay wall, constructed according to this design concept was constructed in 1982. Since then nearly all deep water quay walls have been constructed according to this design concept.

5.2 Development of Design Concept

The design concept applied at present has been developed through a study of different construction types of quay walls. In the comparison of these construction types factors such as reliability, multi-functionality, corrosion, maintenance, erosion, installation, construction time and costs were considered.

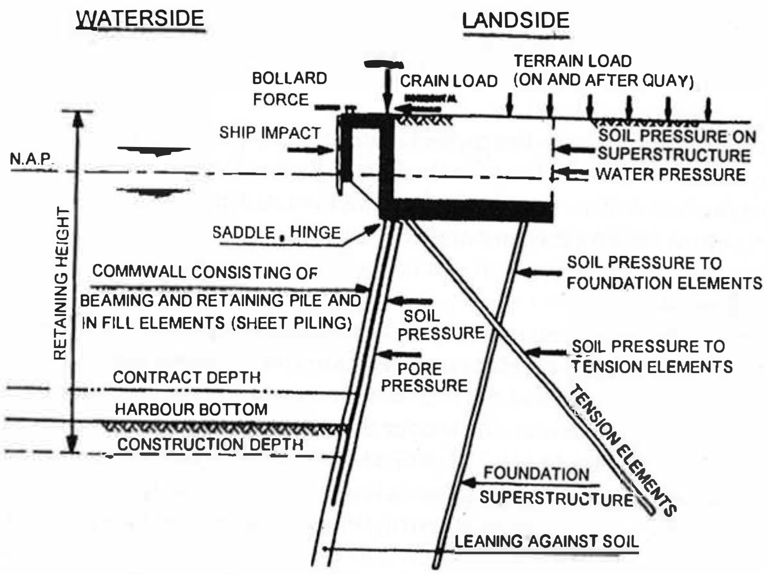


Fig. 6. Design concept for a quay-wall construction.

A quay wall, according to the present design concept, consists of a concrete superstructure supported by a sheet pile wall (e.g. combi-wall). The concrete superstructure, which also serves as a relieving floor, reduces the earth pressure on the sheet pile wall. The sheet pile wall has a supporting and retaining function. Concrete piles and steel tension piles with grout-injection (M.V.-pile; M.V. stands for Muller Verfahen) under an angle of 45° carry the main part of the horizontal load. The anchor force is considerably reduced by placing both the sheet pile wall and the concrete piles under an angle of inclination in such a fashion that the quay wall is "leaning" against the ground. The angle of inclination of the sheet pile wall also has the advantage that the soil pressures on the wall are substantially reduced. Thus with a relatively deep relieving floor and an inclined sheet pile wall a considerable reduction of the costs for the sheet pile wall and the total construction can be achieved. Furthermore the connection between the sheet pile wall and the superstructure consists of an eccentric hinge with a cast iron saddle so as to reduce the bending moments in the wall. Another advantage of an inclined sheet pile wall is that together with the M.V.-pile a kind of A-frame is formed that is very beneficial in respect to horizontal loading and creates the necessary space to place concrete piles with a large angle of inclination. The location of the M.V.-piles on the waterside of the superstructures is also beneficial because placement in the rear would require a denser field of concrete piles. The axial bearing capacity of a combi-wall is generally not critical. Other aspects to be considered are:

- the steel parts of the quay wall are always beneath the level of low tide, this in order to reduce the corrosion process.
- drainage behind the quay wall is be provided to equalise the water pressures over the quay wall as much as possible. If necessary vertical sand drains are installed behind the quay wall.
- the above presented construction is a static determined system.
- to reduce the risk of damage to the sheet pile wall during mooring of ships and in case of a collision it is necessary to have a robust concrete structure located in front (at the water side) of the sheet pile wall with a wooden guidance wall and fenders for a possible ship impact.
- with a deep lying position of the relieving floor the drivability of the combi-wall increases because the driving length is reduced.

The actual design concept as illustrated in Fig. 6 implies an important difference in the design for sea quay wall for bulk handling before 1982, as can be seen in Fig. 7 with the design of the 1st phase of the "EKOM" quay wall (a bulk terminal especially for coal and iron ores) constructed in 1974. The design concept uptill then shows a relatively high position of the relieving floor with

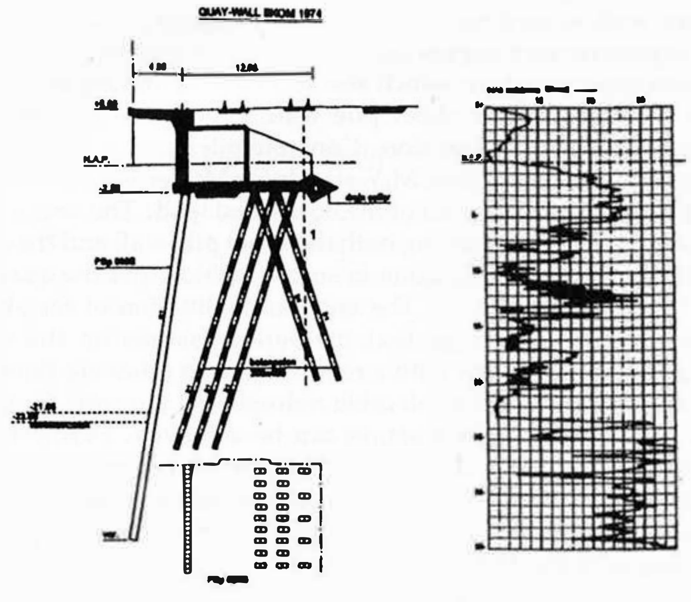


Fig. 7. Design of EKOM quay wall from 1974

a foundation consisting of a sheet pile wall and concrete compression and tension piles placed at a steep angle of inclination of 3 to 1. This type of pile frame bears the high horizontal forces which require a dense field of piles. In Fig. 7 also a top view of the piles is given.

One of the main reasons to study the different design alternatives was the fact that the design concept presented in Fig. 8, based on an up-scaling of earlier constructions to account for the increasing retaining height, showed that high surface loads of coal and iron ore in combination with the soil conditions resulted in considerable continuing displacements of the wall. This made additional anchoring for the quay wall necessary. Analysis of these displacements, much higher then expected, gave two possible explanations:

1. The toe of the concrete tension piles were placed just above the clay layer at RL-21 m, the high surface loads resulted in settlement of this clay layer and thus at the location of the tension piles (Parent, 1983, b). This lead to a situation where the pull of the tension piles on the construction was nearly equal to the force of the total shaft friction of the tension piles. This means a possible overloading of the construction which is equal to the factor of safety of the tension piles, in this case the factor of safety was equal to three. The consequence was that the compression piles were loaded above their ultimate bearing capacity. This resulted in a mainly horizontal and also vertical downward displacement of the rear of the superstructure corresponding to the horizontal and vertical upward displacement of the outer extreme of superstructure due to rotation at, and horizontal displacement of, the top of the combi-wall. This is illustrated in Fig. 8.

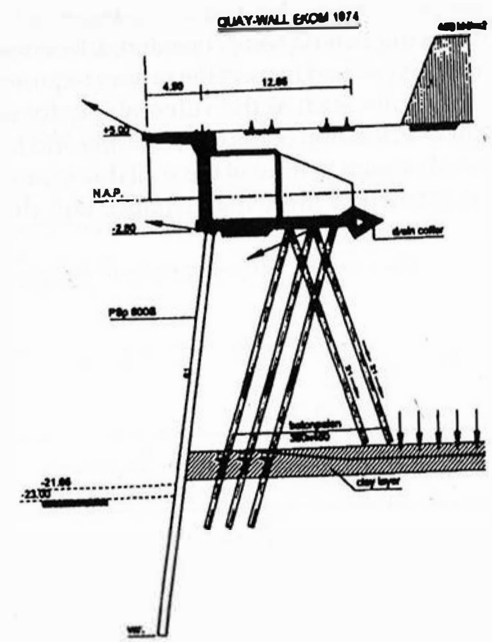


Fig. 8. Displacements of EKOM quay wall

2. The reduction of the horizontal earth pressure on the sheet pile wall, through the relieving floor, can be counteracted by earth pressure on the tension and compression piles, because the dense pile field acts as a fictive wall as illustrated in Fig. 6 (see the dotted line at the rear of the superstructure). This can lead to substantially higher anchor forces than calculated and overloading of primarily the compression piles because they have a lower safety factor than the tension piles (Parent, 1983a).

Both explanations give a similar displacement behavior of the wall and can occur simultaneously.

Reviewing the EKOM quay wall design it was concluded that by placing the toe of the concrete tension piles well below the compressible clay layer, e.g., at a depth of R.L.-25 m, the first situation as described above can be avoided. The disadvantage of this solution is that this probably results in higher bending moments in the concrete tension piles which may require steel tension piles to be used. Avoiding the second situation described above is more difficult and requires another design concept. These considerations and the constant increase of the retaining height was the main reason to study several construction types.

5.3 Comparative Study

The comparative study was conducted by the Engineering Division of Rotterdam Public Works (Parent, 1983a). Seven constructions types were designed and compared, as indicated in Table 2. For the comparison of the different construction types the following criteria were considered: besides the costs and the basic requirement that all types had to meet the project requirements, quality aspects played an important role, such as the vulnerability for mooring ships and collisions, but also for overloading of the construction and for the stability of the construction in case of erosion in front of the wall due to scouring through ship propellers. Also construction time, maintenance and durability were considered.

Table 2. Comparison of seven different quay wall designs

Type of quay wall	Method of execu-tion	Comp arison cost in %	Remarks on quality
Cellular Cofferdam	dry	120	Large risks in execution and use, low resilience, corrosion sensitive
Diaphragm wall	dry	150	Great resilience
Quay wall with high relief floor	dry	140	Sensitive to overload with vertical loads, sensitive to corrosion
Quay wall with high relief floor	wet	150	Sensitive to overload with vertical loads, sensitive to corrosion
L-wall at R.L.-20 m on a soil improvement	dry		Not possible in connection with raft founded objects in the neighborhood
L wall on piles floor at R.L.-10 m	dry	100	Depth yet to be optimized, great resilience, insensitive to overload with vertical loads not sensitive to corrosion
Jetty	wet	180	Vulnerable low resilience in the event of collision, sensitive to overload with vertical loads

In nearly all situations in the Rotterdam harbor, it is possible to built the superstructure of the quay wall in a dry building pit. The equipment for the installation of the sheet piles, the MV-piles and the concrete piles is working in this building pit.

The importance of the different aspects may be not equal and therefore weighing factors were introduced. The results of the study are presented in Table 2. Remarkable aspects are: the quay-wall with a deep position of the relieving floor (L-wall) founded on piles is the most cost effective; the difference between positing of the relieving floor (L-wall) at R.L.-10 m or at R.L.-6 m is minimal and has to be optimized for each specific situation. Also according to the other above-mentioned criteria a deep lying relieving floor is advantageous. The final choice of the type of construction to be designed is of course selected in consultation with the client and furthermore depends on the project requirements, composition of the subsoil and the specific situation at the building site. Thus for every new project a verification is made to confirm that the chosen design concept is also the most appropriate for that specific situation.

5.4 Safety Concept

Until recently a deterministic design method was used based on the EAU-code, with some adaptations for the situation in the area of Rotterdam and based on evaluation of monitoring data. Overall safety factors were used which take into account the uncertainties in the design method as well as the uncertainties in the loading data.

Since 1992 a semi-probabilistic safety concept has been introduced in most EC countries. This design method starts with a probability of failure. This requires a so-called reliability index β which is usually based on legislation, at least for buildings. In the Netherlands this is also the case for dikes for which the reliability index has been standardised by the authorities. For quay walls which are not a primary water barrier this index has been established in consultation with the Rotterdam Port Authorities. In the semi-probabilistic concept the probability of failure is translated into load factors, material factors, geometrical factors etc., this has been done with probabilistic calculations (Spierenburg et al., 1994). For the design of sheet pile constructions in the Netherlands different safety classes have been defined for the following construction types:

- class 1: simple retaining constructions where failure only causes limited damage, the β-value is equal to 2.5.
- class 2: retaining structures where failure results in considerable damage but a low risk for humans, the β-value is equal to 3.4.
- class 3: retaining structures where failure results in risk for human live and important economical loss, the β-value is equal to 4.2.

For these classes partial factors were calculated and published in the CUR-handbook for Sheet Pile Constructions (CUR, 1993). Nowadays these factors are commonly used in the Netherlands. Because the configuration of the Rotterdam quay walls, especially the bulk and container terminals with very high surface loads and retaining height, were not included in the CUR study an additional analysis was carried out to define partial factors for the design of these quay walls (Huijzer, 1996). The β-value for the quay walls in the Rotterdam area has been determined to be at least 3.6. The main failure made for this analysis has been the failure of the combi-wall and the anchor system. The probability of failure of exceeding the passive earth pressure was in this study much lower because of the fact that the sheet pile wall is considerably longer than the minimum length, due to optimizing the bending moments (Tol, 1995). Tables 3 and 4 give an overview of the results of this study. Table 3 presents the material factors and the factors to be applied on the action effects (bending moment an anchor force) and in Table 4 the geometrical factors are presented. In Table 3 the last column gives the "former approach" (the adapted EAU concept), column 4 the values from the CUR-handbook, column 2 the results of a pure statistical analysis based on β=3.6 and column 3 the factors to be applied for the design of the quay walls from now on. The last mentioned values were chosen after a reconsideration and between the "former approach" and the "pure" results of the analysis. An important item in this study was the requirement of a relatively low probability of failure for the connection between the anchor and the superstructure, this is because of the brittle behavior of such a connection and severe consequences of such a failure.

Table 3. Material and other factors

Parameter	Factors from prob. calculations β-3.60	Definitive factors	Factors (CUR 166, safety class II/III)	EAU factors (with some adaptations)
ϕ'	1.00	1.00	1.18	1.00
c'	1.00	1.00	1.05	1.00
δ	1.00	1.00	1.00	1.00
Section modules	0.97		-	
Yield stress	1.17		-	
Bending moment	1.15	1.30	1.00	1.50
Strength of the anchor: - connection - rot	1.46 -	1.70 1.50	1.10 -	1.8 (2.0/1.1)
Ultimate tensile capacity of the anchor	0.98	1.20	1.25	1.4 (2.0/1.25) × 0.85
$E_{pas;max} / E_{pas;mob}$	1.10	1.30	1.00	1.5

Table 4. Geometrical factors

Parameter	Definitive partial factor	Standard deviation [m]	Δ_{min} [m]	Partial factors, according to CUR 166	Δ_{min} [m]
Bottom level	1.2	0.34	0.4	2.4	0.35
Ground water table	2.0	0.33	0.7	1.2	0.25
Water Level	0.6	0.18	0.1	1.9	0.05

5.5 Computation Methods for Combi-Wall

For the design of a combi-wall the following computation methods are available:

- methods based on the limit earth pressure theory, e.g. Blum method;
- methods based on a Winkler-type of model, also called a sub-grade reaction model, with multi or bilinear springs;
- finite element methods (FEM-models), e.g. PLAXIS.

In general the Blum method is used for a quick scan of the required minimum length of the combi-wall. Next a sub-grade reaction model is used to determine the exact length and the action effects. Subsequently the FEM-model is applied to check all other failure modes.

6. ASPECTS IN QUAY-WALL DESIGN

6.1 General

In this paragraph the different aspects (sheet piles, foundation and concrete piles) which play an important role in the design of a quay wall are described. For each of these aspects examples and values will be given if possible.

6.2. Sheet Pile Wall

Since the first combi-walls were utilised as sheet pile walls for quay walls in the Rotterdam area in the early 50s, the combi-wall has been developed further mainly to improve the driving of course also to resist the retaining height increased.

In Fig. 9 the development of the combi-wall in time is presented (Gijt et al.,

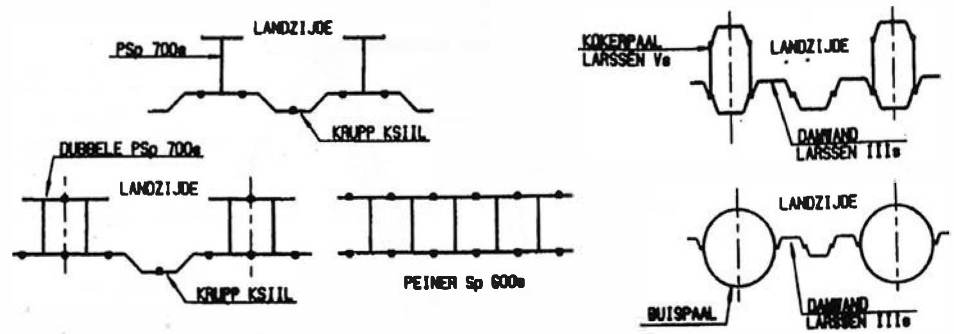


Fig. 9. Development of combi-wall systems

1996). With the extreme length of the sheet pile walls it is of eminent importance, but difficult to achieve, to prevent the occurrence of interlock openings between the sheet piles. Interlock openings have in several occasions resulted in great financial and operational loss. In sandy soil conditions under the influence of tidal waters these interlock openings produce, after a short time or sometimes after years, as liquefied sand flowing through the opening causing extreme settlement at the surface behind the quay wall, see also Fig. 10 (Weijde, 1992).

The experience gained with sheet pile walls has emphasised importance of the type of profile used when it comes to the risk of interlock openings. In Rotterdam the following systems have been applied during the last 40 years (see Fig. 8): H-piles, combi-systems of H-piles and Box-piles with three single sheet pile in between. In the latest designs however a combi-wall of the tubular piles with a triple Larssen sheet pile element in between has been chosen. In the combi-systems the primary elements have a bearing and retaining function and the sheet piles in between, which are considerably shorter, have to retain the resulting water pressure and have to be "soil-tight". To avoid piping these profiles have to penetrate far enough, in practice more then 4 m, under the bottom and take into account erosion as well as future activities.

In most quay walls in the Rotterdam harbor the tubular piles of the combi-walls have a diameter between 1.066 and 1.420 mm and a wall thickness of 16 to 22 mm, the triple piles are Larssen III profiles or equivalent. Also the type of interlock is important, for the combi-wall a Larssen type of interlock is welded onto the tubular pile.

Furthermore in the prevention of interlock openings the installation method is of vital importance. The open ended tubular piles, with a length of 30 to 35 m, are installed by vibration (when very dense layers are encountered the required energy reaches values of 1600 kN) down to the Pleistocene sand layer. The

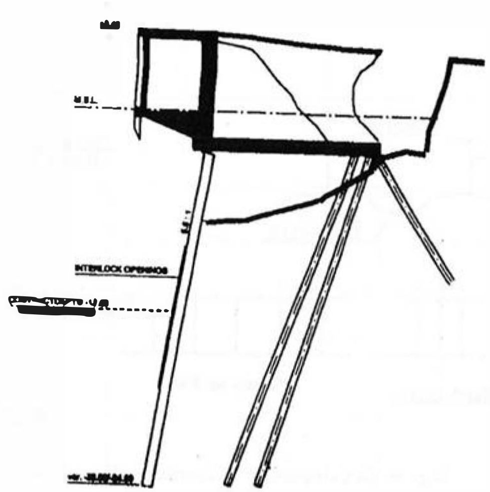


Fig. 10. Damage due to interlock openings (Brittaniehaven)

tubular piles are then driven by impact into the Pleistocene sand layer to the required depth with at least a Diesel D62 or a S200 hydraulic hammer. This installation procedure is adopted for practical reasons as well as for accuracy, driving time and required effort, and to obtain an adequate bearing capacity. A guiding frame is used during the installation of these piles. After the installation of the tubular piles the triple sheet piles, with a length of about 20 to 25 m, are installed by the following procedure: 1) vibrating, 2) vibrating with jetting, and 3) in exceptional situations driving is permitted.

From the driving and vibration behavior of the combi-wall it is very difficult to obtain data indicating if interlock openings are present. This means that interlock openings can only be traced during dredging operations in the harbor basin in front of the quay wall. Repair of interlock openings is a very costly operation and should therefore be minimised. In order to achieve this detectors in the interlock have been developed and tested.

In practice this procedure gives satisfactory results, interlock openings are now nearly completely avoided.

6.3 Foundation

6.3.1 General

The superstructure is supported by the tubular piles of the combi-wall and by prefabricated and concrete piles and anchor piles. For anchor piles with an angle of inclination of 45° a modified M.V.-pile is used. Table 5 gives an overview of the loads and dimensions of these piles in a situation for a sea quay wall with a retaining height of 25 to 30 m and for a wall with a retaining height of 15 to 20 m.

Table 5. Overview of characteristics of the foundation

	Retaining height			
Type of pile	25-30 m		15-20 m	
Concrete pile 450 * 450 or 500 * 500 mm	$F_{s;d}$ [kN] 1800	Length [m] 20-26	$F_{s;d}$ [kN] 1800	Length [m] 20-25
Tubular pile diam. 1066-1420 mm	7000	30-35	4000	25-30
M.V.-pile	3000	35-45	2000	30-36

6.3.2 Anchor piles

The horizontal forces which act on the structure must be transferred to the subsoil. This can be realized by several types of construction elements. In the period in which the study was conducted to improve the design of large quay walls also the type of anchoring was considered. In Table 6, a summary of the

Table 6. Summary of anchoring systems

Types of anchor	Design tensile capacity ($F_{r;hor;d}$) in kN	
A frame construction (concrete)	400 to 600	per pile frame
Anchor wall	500 to 1,000	per m[1]
M.V.-piles (modified)	1,500 to 2,500	per pile
Injection anchors	4,00 to 1,500	per anchor

different types of tension anchors are presented, indicating also the range in ultimate horizontal tension load capacity.

Because of the high tensile load capacity, the rigid behavior and the relative insensibility for corrosion the M.V.-pile was chosen and has since then been used for the construction of quay walls in Rotterdam. The M.V.-pile consists of a steel H-profile pile with grout injection behind an enlarged tip (Gijt et al., 1991).

The original M.V.-pile had a full tip and was difficult to drive to depth. The modified M.V.-pile comprises a reduction of the tip area, facilitating pile-driving and reducing grout consumption. The pile-drivability was also improved by selecting a larger steel cross-section than necessary for absorption of the tensile forces. The pile is equipped with two grout pipes behind the enlarged tip (see Fig. 11), after injection the cross-section of the pile is about 405 * 525 mm. In all the projects in which M.V.-piles have been used, load tests have been conducted on at least 1% of the installed piles. The ultimate shear strength derived from the load tests is equal to 1.4% of the cone resistance q_c of the CPT with a maximum of 250 kN/m². The M.V.-pile has a relatively high working load. This force is transferred to the concrete construction by means of numerous (up to 80) dowels with a diameter of approx. 16 mm.

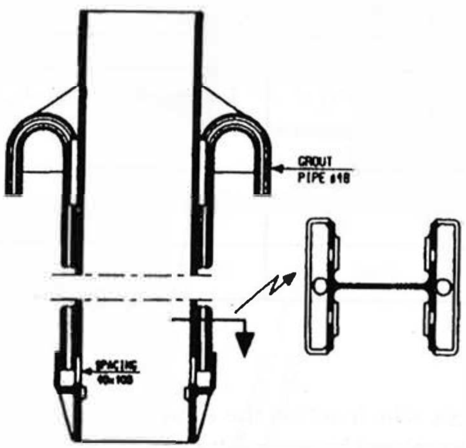


Fig. 11. Construction of the tip of the MV-pile

The M.V.-piles are driven with an S70 or S90 hydraulic hammer, because with an angle of inclination of 45°, this is more effective than using Diesel hammers.

6.3.3 Concrete piles

The prefabricated pre-stressed concrete piles have diameters of 450 and 500 mm and vary in length from 22 m to 30 m. The ultimate bearing capacity is determined according to the Dutch Code NEN 6743 for compression piles. The calculation is based on empirical relationship with the cone resistance measured with CPTs. In the Dutch Code limit values for base resistance and shaft resistance are respectively 15 MPa and 150 kPa. For the very dense Pleistocene sand at Maasvlakte it has been proved by an extensive test program that these limits are too conservative (Opstal, 1996). In the test program new limit values, as presented in Table 7, were established. It was agreed with the local authorities that these values could be used for determining the bearing capacity of prefabricated concrete pile for quay walls at the Maasvlakte location.

Table 7. Limits for shaft resistance and point resistance

Shaft resistance		Point resistance	
Dutch code	Opstal et al. (1996)	Dutch code	Opstal et al. (1996)
α_s [-] limit kPa 0.01 150	α_s [-] limit kPa 0.01 230	α_p [-] limit [MPa] 1.0 15	α_p [-] limit [MPa] 1.0 15 or[1] 0.8 20

[1] it is allowed to use the highest value of either value.

The factors in the table have the following meaning:

α_s pile class factor for shaft resistance according to NEN 6743
α_p pile class factor for point resistance according to NEN 6743

These factors give an empirical relationship between the ultimate resistance and the cone resistance of a CPT. The point resistance is determined by applying a factor α_p to a weighed average of the CPT cone resistance over a trajectory under and above the pile point level is used. For the shaft resistance the following relationship is used (Everts, 1997):

$$p_{r;shaft;max;z} = \alpha_s \cdot q_{c;z}$$

with:

$p_{r;shaft;max;z}$ the maximum shaft resistance at depth z:
$q_{c;z}$ the cone resistance at depth z.

Experience with several projects in the Maasvlakte area has shown that utilising water jetting to install the piles down to the top of the Pleistocene sand layer is necessary. With this installation method adhesion is substantially reduced so that pile driving in the Pleistocene sand layer can be prolonged without damaging the pile head. Consequently, it is then possible to achieve the desired penetration in the Pleistocene sand layer, approximately 4 to 7 m by driving with a D55 or D62 diesel hammer. An additional advantage of this installation method is that the negative skin-friction is reduced due to the water jetting, which loosens the soil around the pile in this area, and thus improves the bearing capacity.

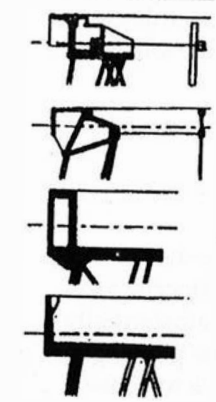
7. COSTS

The construction costs for a sea quay wall with a retaining height of 30 m are at present approx. 60.000 fl per m. The cost breakdown is as follows (Parent, 1990a).

For the Euroterminal, a sea quay wall of 800 m, constructed in 1989, the total construction costs were 42.4 Mfl (million dutch guilders); the steel and pile deliverance amounted to a total of 12.6 Mfl and the construction, including the driving, to 29.8 Mfl. These figures do not include dredging activities.

An interesting aspect of the improvements and optimisation of quay wall design is the development of construction costs. The costs for construction can be divided into three important parts: concrete works, steel and pile deliverance for the combi-wall and piles and all the other costs such as for pile driving, the building pit and so on. The costs for the concrete works amount to about 40% of the total costs. The important cost factor in concrete work is related to the following figures: m^3 of concrete over m^2 of formwork; the man-hours per m^2 of formwork the man hours per m^3 of formwork and the man-hours per m^3 concrete. In Fig. 12 these figures are presented for four type of quay walls. Regarding the costs of the concrete works it appears that the former EMO-wall is too complicated and that the ECT-wall has several inclined parts and therefore the superstructures of these walls are costly. It is apparent that a straight forward design, like the Euroterminal-wall and ARCO, results in is a considerable saving of labor costs (Horst et al., 1992).

Material use



	Formwork/ concrete m-h/m2	Man-hour/ formwork m-h/m2	Man-hour/ formwork m-h/m3	Man-hour/ concrete m-h/m3
EMO 1974	100	100	100	100
ECT 1981	75	65	50	45
EUROTERMINAL 1989	35	50	20	30
ARCO 1991	25	40	10	25

Fig. 12. Development of cost of the concrete works

The costs of the combi-wall, composed of tubular piles and sheet pile elements are, including the installation, about 30-40% of the total quay wall construction.

The development of the combi-wall resulted in only a limited reduction of costs due to weight reduction.

8. CONCLUSIONS

The design of quay walls in the harbor of Rotterdam has been developed and optimized during the last 25 to 30 years. This is partly because of changing requirements, especially the increasing depth of the waterways and high surface loads, and partly by improving the design process based on the experience obtained through monitoring and evaluation. This was made possible because of the enormous growth of the harbor of Rotterdam during this period. The quay walls that were constructed since the 80s are among some of the biggest in the world.

The constructed quay walls meet the requirements and, due to a optimisation process of the constructions, are very cost effective.

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