Structures in hydraulic engineering

Port Infrastructure

Lecture notes on Port Infrastructure - CT5313

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1. Introduction

1.1 General

In history harbours were places of 'natural' shelter, ships could be safely anchored, however loading and unloading facilities were generally non existent apart from human labour. On the other hand ports are man made places of shelter purpose built for efficient transfer of cargo. The distinction between the words harbour and port is not always clear cut, obviously many of today's ports once started out as harbours. In the past and nowadays ports play an important role in world trade, they are part of a logistic chain transporting cargoes all over the world. Four main cargo commodities can be distinguished, viz. containers, liquid and dry bulk, and breakbulk or general cargo. The difference shows in the type of vessel being used for maritime transport, the type of handling equipment and storage within the port, and the type of hinterland transport. General reference is made to the courses CT4330 and CT5306 on Ports and Waterways.
The port has to provide shelter and facilities for cargo transfer. The subject of these lecture notes is the port infrastructure that is predominantly required for cargo transfer, i.e. quays and jetties. One could always argue that (e.g.) a breakwater provides the necessary protection to be able to moor a ship along a jetty, however, the breakwater will not unload the LNG carrier whilst there will always be a weather window allowing the LNG carrier being handled at the jetty.
Nowadays port infrastructure types will be described and discussed with regard to structural design and wherever possible a glimpse on the future will be revealed as developments do not stop.
Some specific subjects like fendering and scour in front of the port structures will be dealt with as well.

Future developments

The general practice of designing and constructing for eternity seems to vanish. This is caused amongst others by rapid changing ship dimensions, loads and crane designs. The developments are hard to predict and all of this results in uncertainty being a sure factor.
This means that flexibility is becoming more important when designing new quay structures. Therefore the past designs must be evaluated in relation to life/cycle costs and new designs must be thought of.
In this respect flexibility could mean design of movable quays, new materials, combination of new materials and present available materials. Also ideas could be generated in relation to demolishing techniques. With the afore mentioned aspects for future developments a challenge is implicitly present for young interdisciplinary thinking civil engineers.
These lecture notes are supplemented by an extensive reference list for further reading and/or study.

1.2 History of the Port of Rotterdam

The history of the harbour of Rotterdam began a few centuries ago in the centre of the present city. At that time, it was a port of refuge for fishing boats. The Buizengat, one of the oldest harbours in Rotterdam, offers a reminder of that time.

In the second half of the nineteenth century a leap was made to the southern side of the river, where a number of harbours were dug. In time however, the depth of the area behind the quays became too small. Moreover the water depths at these river locations were not deep enough owing to the increasing ship dimensions. Expansion was effected in western direction, where the river is deeper.

The Waalhaven harbour, at the time the largest dredged port in the world, was constructed ~~in and~~ around 1930. Initially it was a dry bulk cargo port for coal, now it is still in full use as a general cargo and container port. Nonetheless, plans are developed to change the Waalhaven in both an industrial and residential area.

After the Second World War, the following harbour basins have been constructed to create optimal conditions for the transfer of cargoes and (related) port industry, at times generating more added value than the core port businesses.

Eemhaven	1950
Botlek	1955
Europoort	1958
Maasvlakte	1968
Extension Maasvlakte	2010

In the course of time tremendous changes took place in shipping. Until far into the nineteenth century ships were made of wood and were equipped with sails for propulsion. About 1850, the first engines were introduced for propulsion: at the same time wood was replaced by iron. This made it possible to build considerably larger ships than before. Through the years the developments led to construction of special purpose vessels. For a long time, the so-called General Cargo Ship, predominantly a ship with holds containing bales, cases, crates and drums, ruled the seas. In fact it was an expanded version of the wooden ships from former days. Economies of scale resulted in larger quantities, requiring bigger ships and specialised handling in ports, handling times reduced considerable and the berth utilization rate improved.
The largest ships in operation now have a length of almost 400 m, a width of 55 to 60 m, a draught of about 24 m and the tonnage is about 350,000 dwt.

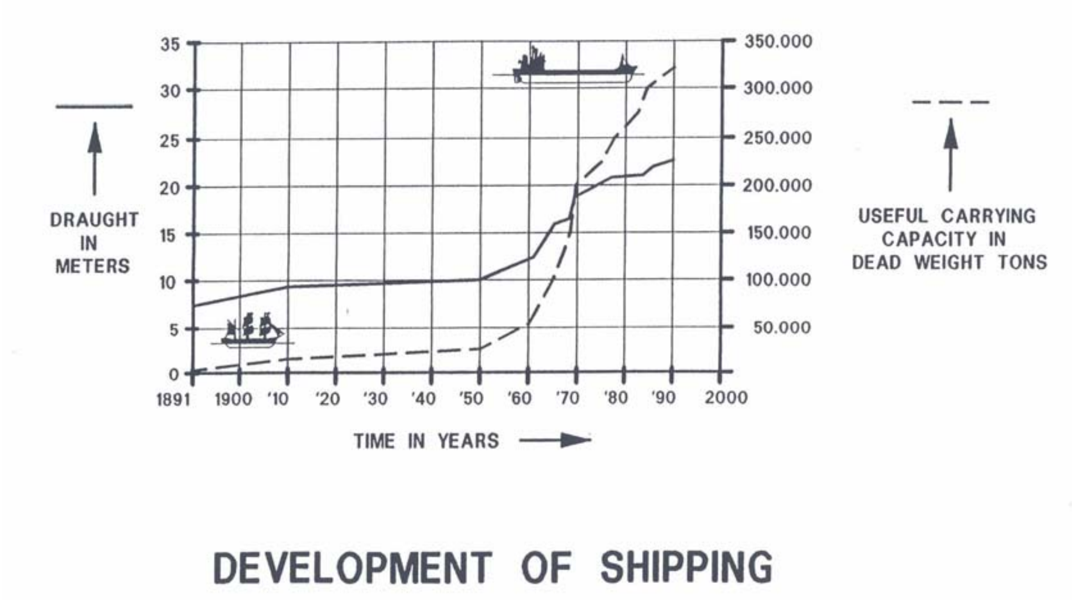


Figure 1 The development of ship dimensions with time.

The dimensions of the ships largely determine the design of harbour basins.
The draught of the ship determines the depth of the quay; 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 harbour basins.

In figure 2 the increase in harbour depth with time and location is presented.

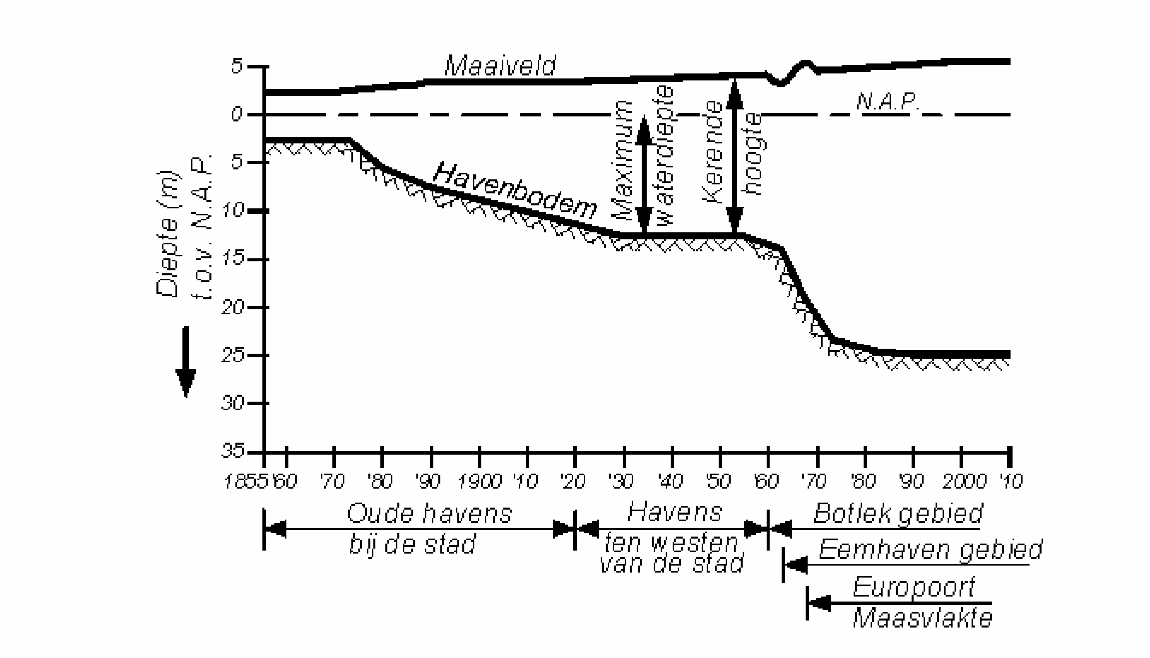


Fig. 2 Increase harbour depth.

It is expected that the dimensions of the bulk carriers remain the same or become smaller. However, for the middle class of ships, especially container ships, it is anticipated that their width will increase probably up to 70 m. The lengths and draughts of these ships are not likely to increase dramatically.

In figure 3 the development of containership dimensions is indicated.

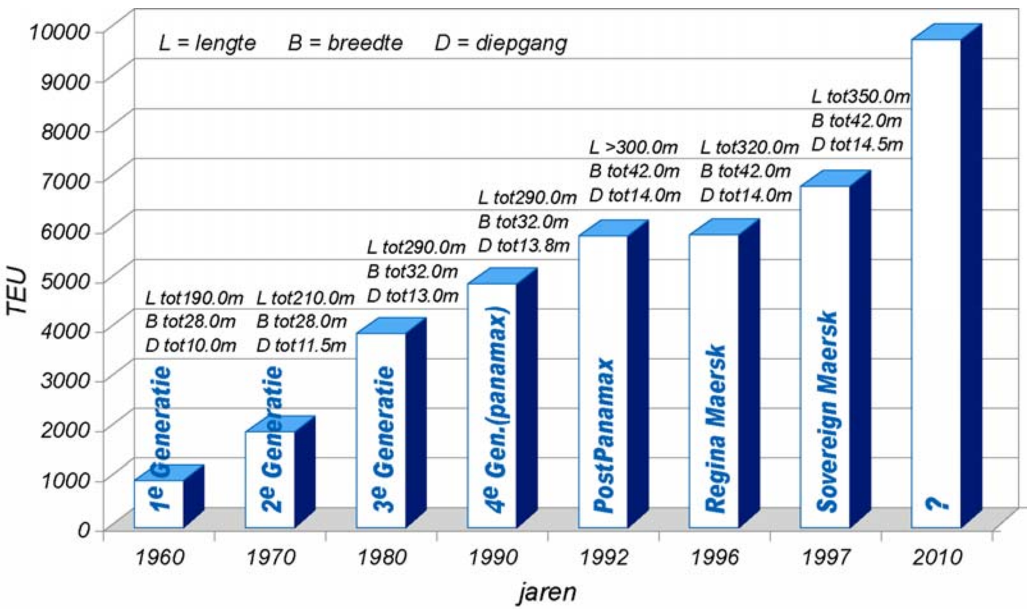


Fig. 3 The development of containership dimensions

Quay-wall constructions therefore need also significantly greater dimensions due to the increase in ship dimensions.
The use of tugboats will probably decrease as the ships are increasingly using their own propellers. This use may cause extra bottom erosion.

This implies that new creative solutions and techniques had to be developed for the construction of quay-walls. The subsoil conditions are very important in this respect.

Soil conditions

Rotterdam is located near the Northsea in the Rhine-Meuse Delta.
Owing to the geological history and location of the area, the soil conditions can vary significantly over a short distance.
Those variations are the result of meandering rivers and rises and drops in the sea level in the past.
Since around 1700 the situation had changed relatively little as far as geology is concerned. However due to human activities several area of extra land have been created, e.g. the Maasvlakte area.

The present situation, *figure 4,* can, as far as soil mechanical aspects are concerned, be divided in three areas with their own typical soil profiles.

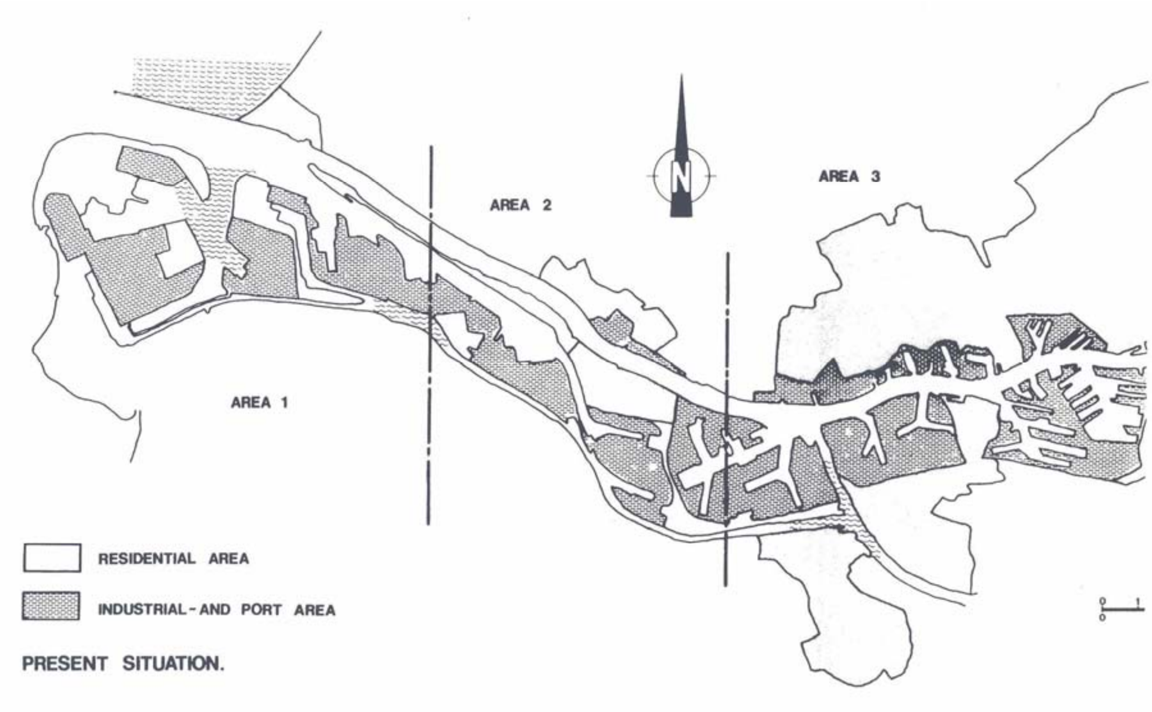


Fig. 4 Soil mechanical characteristics Port of Rotterdam.

The three areas are:

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 a typical cone penetration test result (CPT) is presented in *figures 5 to 7*. In the CPT also a boring log is drawn.

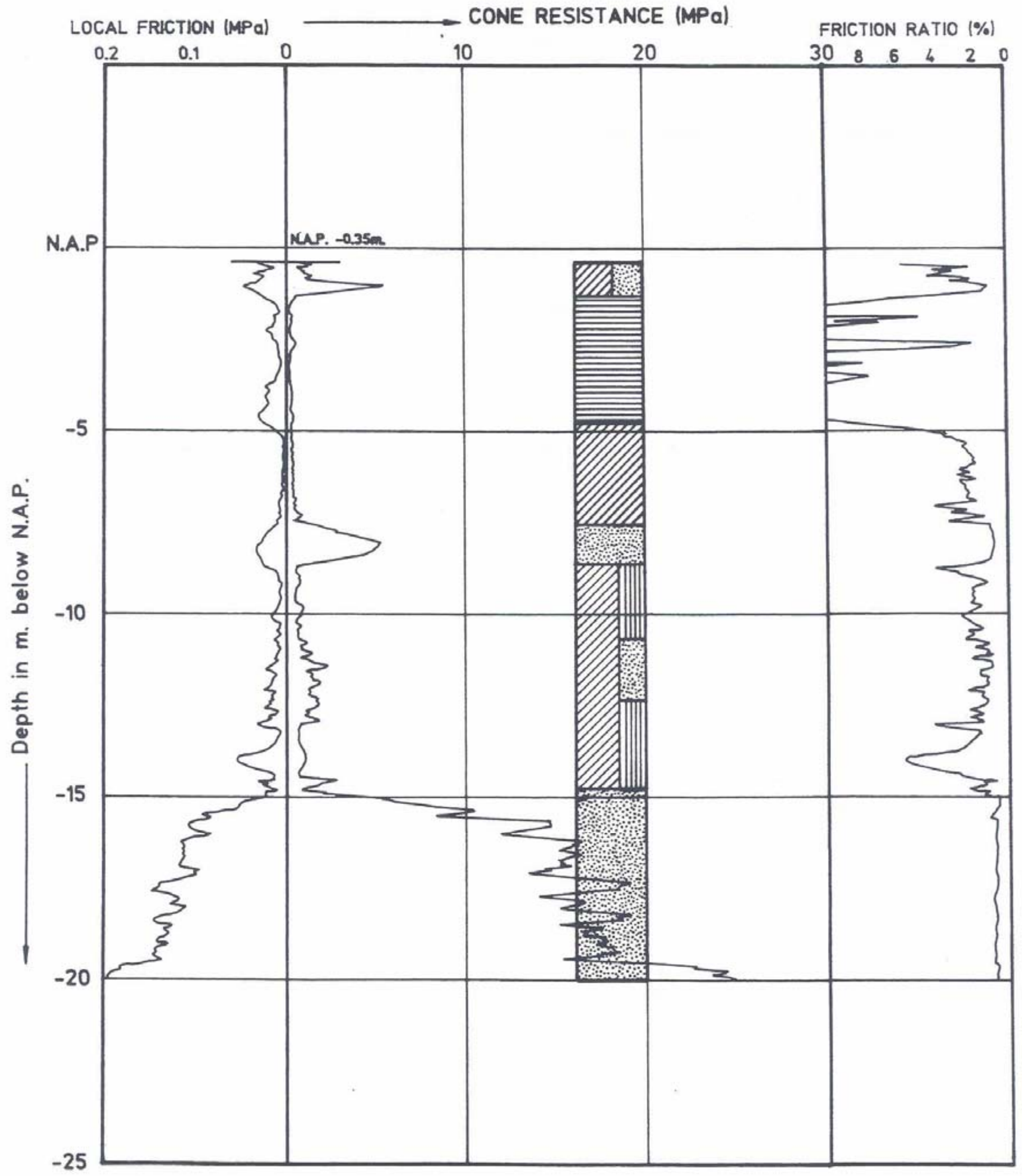
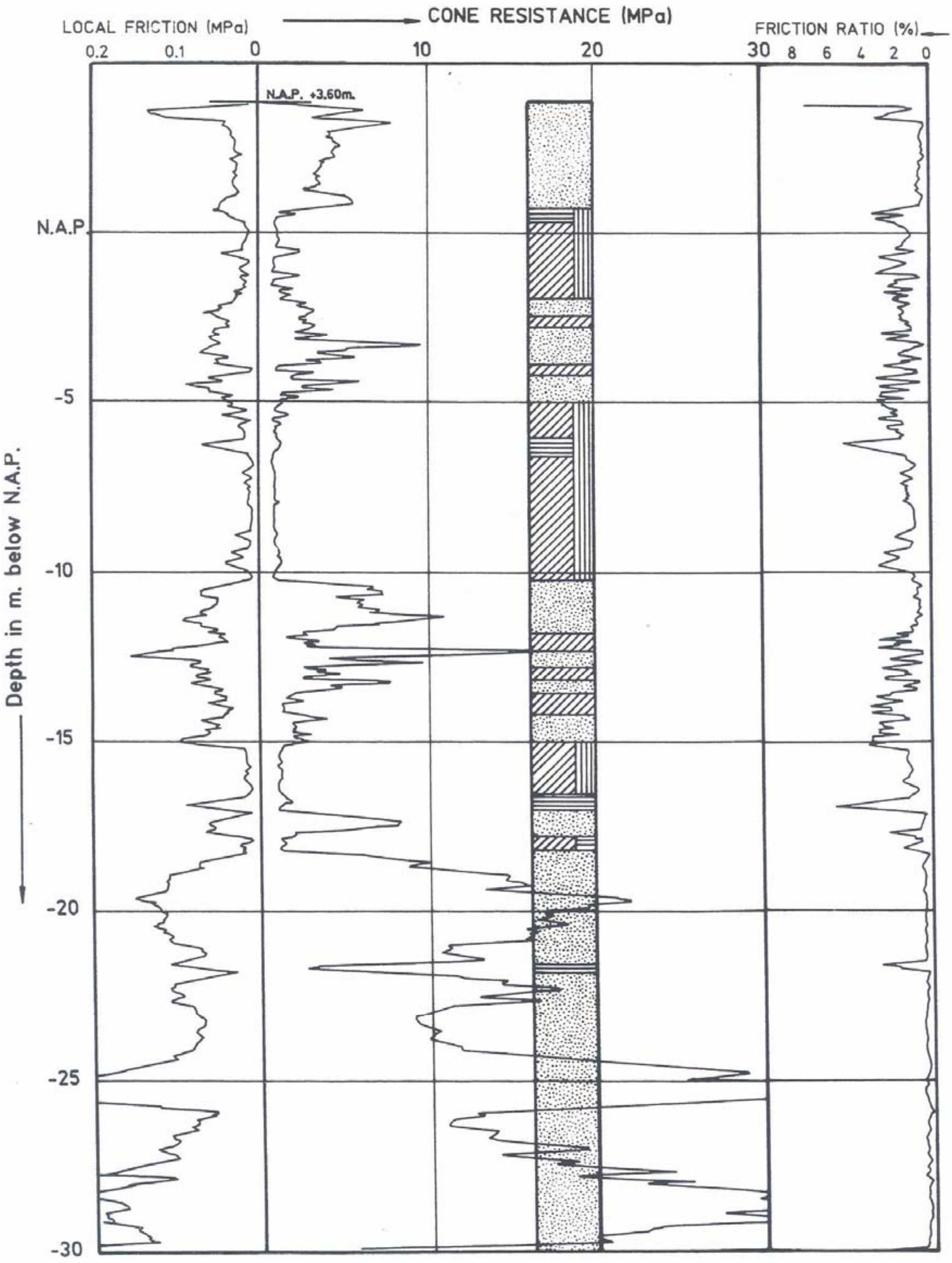


FIGURE 5: TYPICAL CPT AREA 3.

Fig. 5 Typical CPT results for the three areas.



TYPICAL CPT AREA 2.

Fig. 6

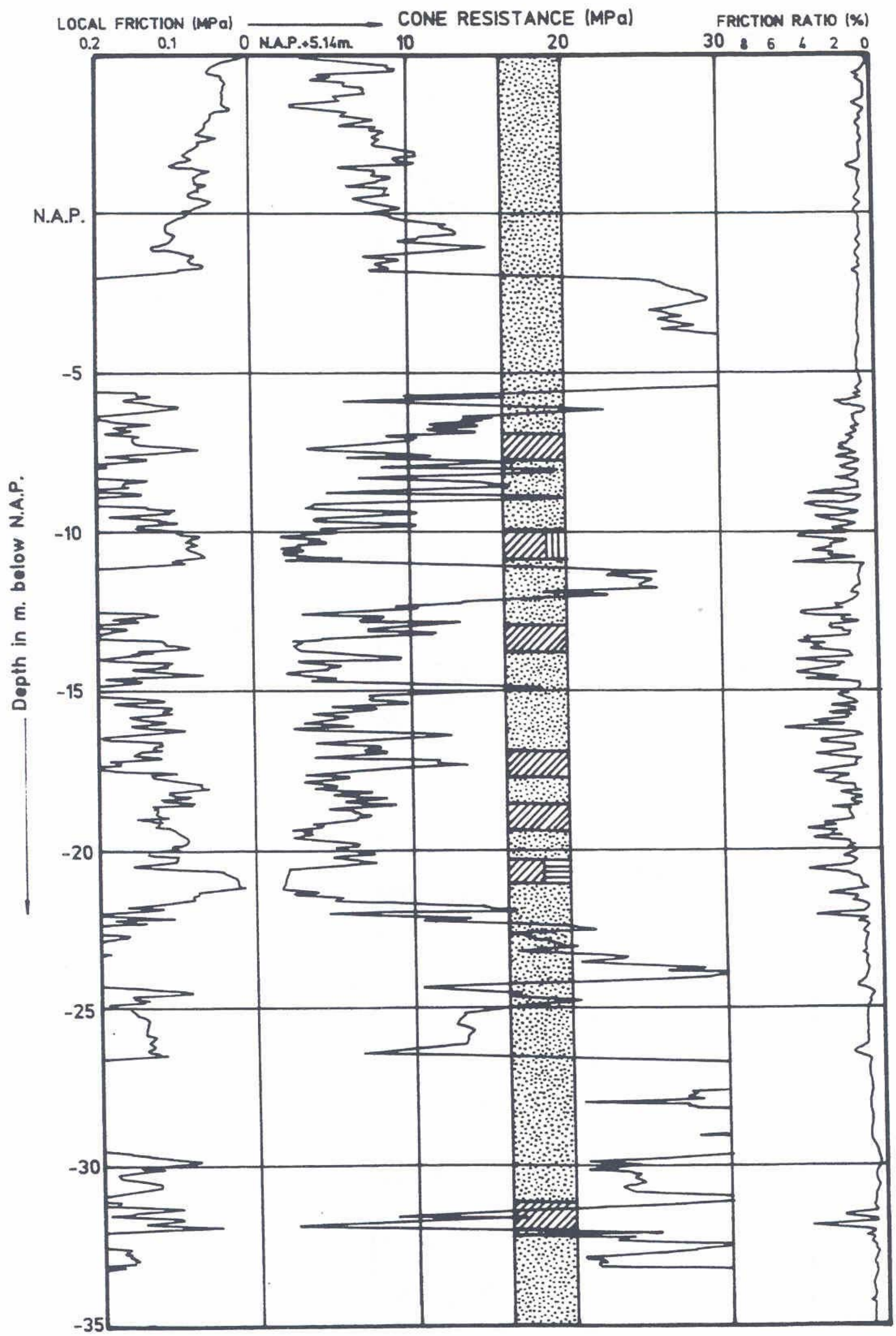


FIGURE 3: TYPICAL CPT AREA 1.

Fig. 7

2. Program of Project requirements

2.1 General

The program of project requirements is a very important document for all involved in the design and constructions process. With this document every party has given commitment to the construction that has to be built. Therefore it is essential that enough time is spent for finalizing this document. The engineer involved must realize him/herself that in this phase in principle everything has been agreed of how to proceed for the rest of the design process.
Within the project requirements difference can be made between functional and technical requirements.
Depending of the contracting strategy the functional or the technical requirements may more or less relative important.

2.2 Functional requirements

Port infrastructure must minimal fulfil the following requirements:

- Retaining function
- Bearing function
- Mooring function
- Protecting function.

<u>Retaining function</u>
The structure has to retain soil and water safely. Typical for this aspect that the retaining height must be agreed and there fore the bottom level as well as the top of structure must be established. This assessment follows from the requirements of the anticipated mooring ships and the minimal anticipated water level.

<u>Bearing function</u>
The loads imposed by cranes, vehicles and the loads by the stored goods must safely beard. For this item it is essential that the handling procedures of the goods are secured, while further also the speed of loading and unloading of the ships is important. In certain circumstances separated store area's and loading and unloading area's are incorporated in the design of the terminal.

<u>Mooring function</u>
The construction must enable the ships to moor safely and subsequently to load and unload there goods efficiently. The space there fore required is depending on number of ships, the dimensions of ships, wind and waves and currents.

Protecting function
This function is related to the safely mooring of ships. To avoid damage to ships some kind of fendering may be needed. Bollards are necessary for fastening the ships.
Further the need for scour protection is depending on the dimensions of the ships propellers and power.

A division is made between functional and technical aspects for the programme requirements; the lists here under illustrate the differences between the two.
Both documents are always required however the use is dependent on the phase of the project and the method of contracting f.e. Design Construct/Classical.

Contents Functional Programme of requirements

1. **Introduction**
   This gives a brief description of the project
2. **Boundary conditions**
   2.1 Determination of the existing situation
   2.2 Natural conditions such as water levels and wind
   2.3 Existing operational situation
3. **Requirements**
   3.1 Nautical requirements
   Types of ship+ characteristic parameters including length, beam and draught
   Number and length of berths
   3.2 Bearing requirements
   Width of the apron area
   Number and types of cranes + characteristic parameters
   Dimensions of the storage zone
   Sort and volume of freight to be handled and storage method
   3.3 Retaining requirements
   Height of the upper surface of the quay
   Depth of the water
   3.4 Protective function
   Berthing facilities
   Bottom protection

2.3 Technical requirements

This document provides all the information required to make the technical computations f.e. steel quality, concrete quality.

Contents Programme of Technical Requirements

1. **Introduction**
   This gives a brief description of the requirements. The objective of the project, the organisation, planning, and possible phasing and functional requirements are described.
2. **Boundary conditions**
   2.1 Description of existing situation
   2.2 Natural conditions
   2.2.1 Topographical
   2.2.2 Hydro-graphic
   2.2.3 Geotechnical
   2.2.4 Hydraulic
   2.2.5 Meteorological
   2.2.6 Environmental
   2.2.7 Disturbance to substrata
   2.3 The presence of cables and pipelines
   2.4 Existing operational situation
3. **Nautical function**
   3.1 Nautical basis
   3.1.1 Usable length of berths
   3.1.2 Type of vessel
   3.1.3 Details of main propellers
   3.1.4 Details of bow thrusters
   3.2 Dimensions of quay wall
4. **Retaining function**
   4.1 Structure of the quay wall
5. **Bearing function**
   5.1 Data on freight
   5.2 Data on cranes and vehicles
   5.3 Crane track equipment
   5.3.1 Details of crane track
   5.3.2 Criteria for use
6. **Protective function**
   6.1 Mooring facilities
   6.2 Harbour bottom
   6.3 Harbour bed protection
   6.4 Bank protection
   6.5 Maintenance requirements and management plan
7. **Diverse**
   Public utilities, lighting, drainage, signage
8. **Safety aspects/reporting and permits**
9. **References**
   Procedures, guidelines, standards, legal aspects
10. **Annexes**
    Drawings, action of the load

3. Boundary conditions

3.1 General

If designing structures one needs data for the type of structure to be constructed.
Knowledge of the site is a prerequisite.
In addition several other data are required to make an optimal design.
One may distinguish the following group of data: geotechnical, nautical, hydraulic and environmental.

Sometimes it appears that not enough data can be obtained: lack of knowledge, lack of money.
In these situation it is the designers responsibility to make a robust design and to inform the client.
Risks which are taken by the designer in the design/tender phase must be taken with care and responsibility.

3.2 Geotechnical

The subsoil conditions are of vital importance during the design of the constructions.
Thus a thorough soil investigation is required. This soil investigation must be directed to the construction types considered and should be related to the design methods to obtain the soil parameters required for the analysis.
Generally the soil investigation is carried in phases to avoid to both for practical as well as financial reasons. By doing the investigation in phases the preliminary design can start at once and the costs can be limited.
After a site survey and gathering data in the vicinity of the project a soil investigation plan is made up. The soil survey generally consists of:

- cone penetration testing (CPT);
- standard penetration testing (SPT): during sounding, both the cone resistance and local friction, as well as the inclination were measured continuously. With sounding depths of 50 m below ground level, the measurement of the inclination is an absolute requirement for this analysis;
- drilling and sampling;
- laboratory testing: classification/strength and deformation testing.

Generally, the following geotechnical aspects need to be investigated for the design of quay-wall structures:

- expected cone resistance in the foundation layers after excavation of the building pit and dredging of the harbour;
- bearing capacity of the quay-wall foundation (open steel tubular piles, prestressed concrete piles and M.V.-piles);
- installation aspects; drivability analysis;
- calculation of the sheet piling as element of the quay-wall;

- calculation of the stability of the total quay-wall construction, including surface loads;
- negative skin friction and settlement;
- piping aspects during installation;
- selection of dewatering system for building pit;
- assessment of influence on surroundings;
- legal aspects, licenses;
- environmental aspects.

For the analysis of these items a thorough soil investigation is performed.

Also the effects of earthquakes when appropriate have to be established.

In figure 8, a very useful relation between CPT and SPT results is displayed in relation to the grain size.

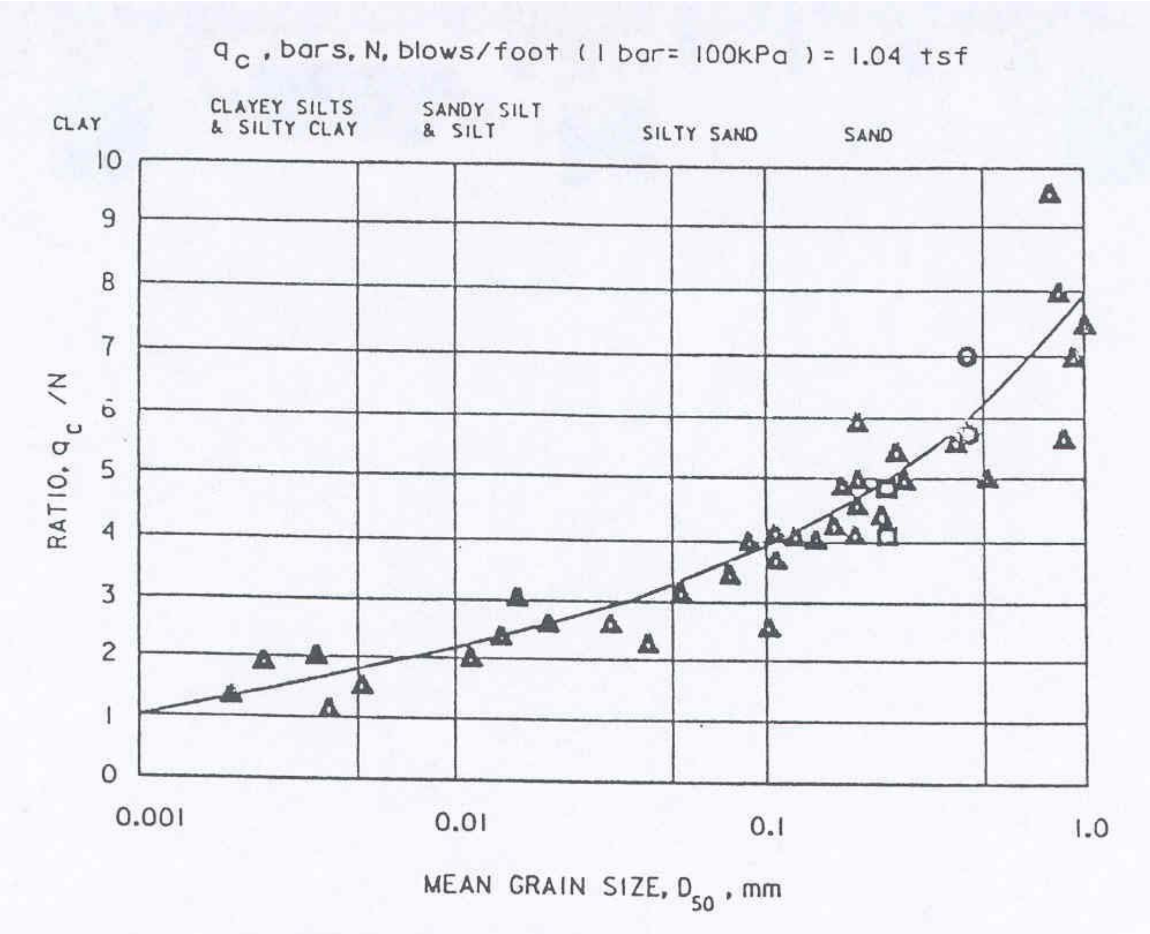


Fig. 8 Relation between CPT and SPT depending on the D50 (after Robertson and Campanella 1983.

3.3 Nautical

In this phase the ship dimensions must be established such as length, width, draught, loading capacity, propellers dimensions and the alike.

These data determine the depth for the quay wall. Also attention can be paid to scour prevention measures.

3.4 Hydraulic

Knowledge of wind, waves and current and some times ice loading is required to assess the mooring conditions.
These factors influence the level of top of the quay wall and also whether safe mooring conditions can be realized.

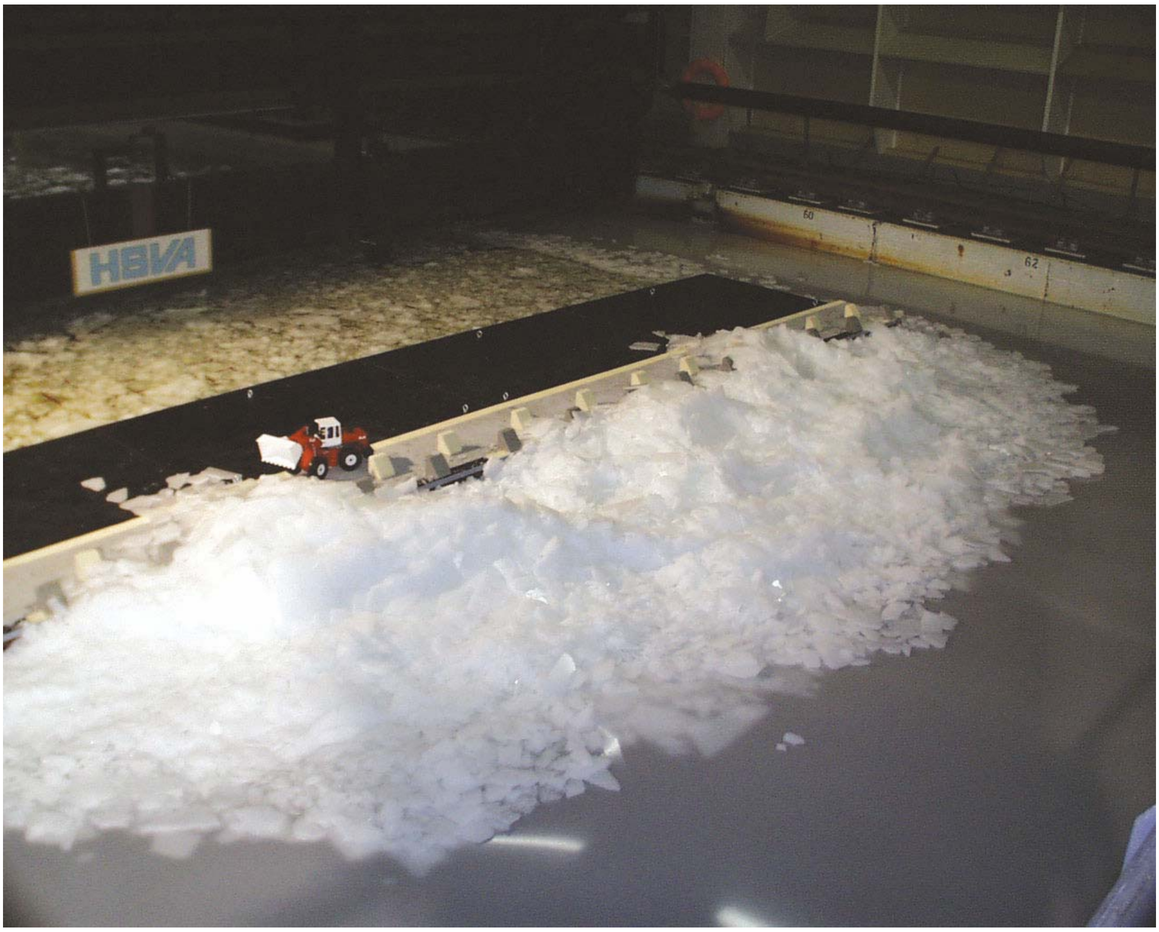


Fig. 9 Laboratory setup of ice-loading.

3.5 Environmental

Both the soil and air conditions must be verified in relation to possible spoiled soil locations. Noise and dust may be a problem in case a terminal is located nearby a city.

5.1.1 Seagoing Vessels

5.1.1.1 Passenger Vessels (table R 39-1.1)

Tonnage	Carrying capacity	Displace-ment *G*	Overall length	Length between perps	Beam	Draft
GT	DWT	t	m	m	m	m
80 000	–	75 000	315	295	35.5	11.5
70 000	–	65 000	315	295	34.0	11.0
60 000	–	55 000	310	290	32.5	10.5
50 000	–	45 000	300	280	31.0	10.5
40 000	–	35 000	265	245	29.5	10.0
30 000	–	30 000	230	210	28.0	10.0

5.1.1.2 Bulk Carriers (table R 39-1.2) (oil, ore, coal, grain, etc.)

–	450 000	524 000	424	404	68.5	25.0
–	420 000	490 000	418	398	67.0	24.5
–	380 000	445 000	407	386	64.5	24.0
–	365 000	428 000	404	383	63.5	23.0
–	340 000	400 000	398	378	62.5	23.0
–	300 000	356 000	385	364	59.5	22.0
–	275 000	326 000	376	355	57.5	21.5
–	250 000	300 000	367	346	55.5	20.5
–	225 000	270 000	356	336	53.5	20.5
–	200 000	240 000	345	326	51.0	19.5
–	175 000	212 000	330	315	48.5	18.5
–	150 000	180 000	315	300	46.0	16.5
–	125 000	155 000	295	280	43.5	16.0
–	100 000	125 000	280	265	41.0	15.0
–	85 000	105 000	265	255	38.0	14.0
–	65 000	85 000	255	245	33.5	13.0
–	45 000	60 000	230	220	29.0	11.5
–	35 000	45 000	210	200	27.0	11.0
–	25 000	30 000	190	180	24.5	10.5
–	15 000	20 000	165	155	21.5	9.5

5.1.1.3 Mixed Cargo Freighters (Full Deck Construction) (table R 31-1.3)

Tonnage	Carrying capacity	Displace-ment *G*	Overall length	Length between perps	Beam	Draft
GT	DWT	t	m	m	m	m
10 000	15 000	20 000	165	155	21.5	9.5
7 500	11 000	15 000	150	140	20.0	9.0
5 000	7 500	10 000	135	125	17.5	8.0
4 000	6 000	8 000	120	110	16.0	7.5
3 000	4 500	6 000	105	100	14.5	7.0
2 000	3 000	4 000	95	90	13.0	6.0
1 500	2 200	3 000	90	85	12.0	5.5
1 000	1 500	2 000	75	70	10.0	4.5
500	700	1 000	60	55	8.5	3.5

There appears to be no trend towards construction of larger cargo freighters. If necessary, the dimensions used in section 5.1.1.2 may be used accordingly.

5.1.1.4 Fishing Vessels (table R 39-1.4)

2 500	–	2 800	90	80	14.0	5.9
2 000	–	2 500	85	75	13.0	5.6
1 500	–	2 100	80	70	12.0	5.3
1 000	–	1 750	75	65	11.0	5.0
800	–	1 550	70	60	10.5	4.8
600	–	1 200	65	55	10.0	4.5
400	–	800	55	45	8.5	4.0
200	–	400	40	35	7.0	3.5

5.1.1.5 Container Ships (table R 39-1.5)

Carrying capacity	Dis-place-ment *G*	Overall length	Length between perps	Beam	Draft	Number of con-tainers	Genera-tion
DWT	t	m	m	m	m	circa	
75 000	90 000	350	335	45.0	14.0	6 000	6th
66 300	80 000	275	262	40.0	14.0	4 800	5th
64 500	77 500	294	282	32.2	13.5	4 400	5th
55 000	77 000	275	260	39.4	12.5	3 900	4th
50 000	73 500	290	275	32.4	13.0	2 800	3rd
42 000	61 000	285	270	32.3	12.0	2 380	3rd
36 000	51 000	270	255	31.8	11.7	2 000	3rd
30 000	41 500	228	214	31.0	11.3	1 670	2nd
25 000	34 000	212	198	30.0	10.7	1 380	2nd
20 000	27 000	198	184	28.7	10.0	1 100	2nd
15 000	20 000	180	166	26.5	9.0	810	1st
10 000	13 500	159	144	23.5	8.0	530	1st
7 000	9 600	143	128	19.0	6.5	316	1st

5.1.1.6 **Car transport Ships** (table R 39-1.6)

Carrying capacity	Displace-ment *G*	Overall length	Length between perps	Beam	Draft	No. of cars
DWT	t	m	m	m	m	approx.
28 000	45 000	198	183	32.3	11.8	6 200
26 300	42 000	213	198	32.3	10.5	6 000
17 900	33 000	195	180	32.2	9.7	5 600

5.1.1.7 **Ferries and Ro-Ro Ships** (table R 39-1.7)

Carrying capacity	Displace-ment *G*	Overall length	Length between perps	Beam	Draft
DWT	t	m	m	m	m
106 400	115 000	253.00	238.00	40.00	15.10
64 400	76 100	225.00	215.00	34.00	13.00
42 500	53 000	182.50	173.00	32.30	12.00
27 750	39 800	177.30	158.10	27.30	11.55
18 000	32 650	181.20	165.00	30.40	9.30
16 000	23 400	178.10	164.00	26.80	7.60
14 000	21 500	163.80	148.60	23.50	8.80
12 000	20 000	190.90	173.00	26.00	7.18
10 000	23 410	192.50	181.00	27.30	6.75
8 000	16 000	156.00	137.00	22.60	7.30
6 000	20 750	179.40	170.00	27.80	6.27
4 000	17 500	163.40	150.00	27.00	6.20
2 000	10 800	164.70	159.60	17.70	5.90

The data in the table vary according to type of load (cars, trucks, trailers, waggons, passengers) and load shares.

5.1.2 **River-sea Ships** (table R 39-2)

Tonnage	Carrying capacity	Displace-ment *G*	Overall length	Beam	Draft
GT	DWT	t	m	m	m
999	3 200	3 700	94.0	12.8	4.2
499	1 795	2 600	81.0	11.3	3.6
299	1 100	1 500	69.0	9.5	3.0

5.1.3 Inland Vessels (table R 39-3)

Designation	Carrying capacity	Displacement G	Length	Beam	Draft
	t	t	m	m	m
Motor freighters:					
Large Rhine ship	4 500	5 200	110.0	11.4	4.5
2600-ton class	2 600	2 950	110.0	11.4	2.7
Rhine ship	2 000	2 385	95.0	11.4	2.7
"Europe" ship	1 350	1 650	80.0	9.5	2.5
Dortmund-Ems-Canal ship	1 000	1 235	67.0	8.2	2.5
Large-Canal-Class ship	950	1 150	82.0	9.5	2.0
Large-"Plauer"-Class ship	700	840	67.0	8.2	2.0
BM-500 ship	650	780	55.0	8.0	1.8
Kempenaar	600	765	50.0	6.6	2.5
Barge	415	505	32.5	8.2	2.0
Peniche	300	405	38.5	5.0	2.2
Large-Saale-Class ship	300	400	52.0	6.6	2.0
Large-Finow-Class ship	250	300	41.5	5.1	1.8
Push lighters:					
Europe IIa	2 940	3 275	76.5	11.4	4.0
	1 520	1 885			2.5
Europe II	2 520	2 835	76.5	11.4	3.5
	1 660	1 990			2.5
Europe I	1 880	2 110	70.0	9.5	3.5
	1 240	1 480			2.5
Carrier ship lighters:					
Seabee	860	1 020	29.7	10.7	3.2
Lash	376	488	18.8	9.5	2.7
Push tows:					
with one lighter Europe IIa	2 940	3 520[1]	110.0	11.4	4.0
	1 520	2 130[1]			2.5
with 2 lighters Europe IIa	5 880	6 795[1]	185.0	11.4	4.0
	3 040	4 015[1]	110.0	22.8	4.0
					2.5
with 4 lighters Europe IIa	11 760	13 640[2]	185.0	22.8	4.0
	6 080	8 080[2]			2.5

1) Push vessel 1 480 kW; approx. 245 t displacement
2) Push vessel 2963–3333 kW; approx. 540 t displacement

TABLE 3.4.2.3.1.1. APPARENT SPECIFIC WEIGHTS AND INTERNAL FRICTION ANGLES OF USUAL CARGOS STORED IN PORT ZONES.

BULK MATERIALS	γ (t/m³)	ϕ (°)
A) SOLID BULK MATERIALS		
– Ores		
Alumina	1.70	35°
Aluminium (Bauxite)	1.40	50° (h) 28° (s)
Copper (Pyrites)	2.60	45°
Chromium	2.60	40°
Tin (Cassiterite)	2.00	38°
Iron (Limonite and Magnetite)	3.00	40°
Magnesium	1.50	35°
Manganese	2.40	45°
Lead (Galena)	2.80	40°
Zinc (Zincblende)	1.80	38°
Roasted pyrite	1.40	45°
– Chemical Products		
Artificial fertilizers	1.20	40°
Mineral fertilizer	1.20	30°
Sulphur	1.20	40°
Carbide	0.90	30°
Phosphates	1.10	35°
Potash	1.10	35°
– Solid Fuels		
Lignite bricks (stacked)	0.80	30°
Charcoal (crushed)	0.40	45°
Coal coke	0.50	40°
Rough coal (moist)	1.00	45°
Pulverized coal	0.70	25°
Residual washing coal	1.20	0°
Other forms of coal	0.85	30°
Spill wood	0.20	45°
Split firewood	0.40	45°
Lignite	0.70	35°
Sawdust (settled)	0.25	45°
Sawdust (loose)	0.15	45°
Construction Materials		
Sand Dry	1.70	30°
Sand Saturated	2.00	30°
Pumice Sand	0.70	35°
Lime powder	1.00	25°
Lump lime	1.00	45°
Kaolin	0.95	35°
Brick rubble or powder	1.30	35°
Cement powder	1.20	25°
Coke ash	0.70	25°
Cement clinker	1.50	30°
Blastfurnace Slag Granular	1.10	25°
Blastfurnace Slag Crushed	1.50	40°
Granite (worked)	1.30	35°
Gravel Dry	1.60	40°
Gravel Saturated	2.00	40°
Marble (worked)	1.30	35°
Limestone	1.70	35°

STACKED MATERIALS	γ (t/m³)
– Ores	
Bauxite (bagged)	0.90
Chromium (boxed)	2.50
Magnesium (bagged)	1.50
Nickel (bagged)	1.65
Nickel (in barrels)	1.45
– Metal. Steel and Iron Products	
Steel (in bars)	3.00
Steel (coiled)	2.80
Steel (in ingots)	3.60
Steel (in plates)	3.50
Aluminium (in ingots)	1.25
Copper (coiled)	1.10
Copper (in ingots)	3.50
Copper (in plates)	3.50
Tin (in ingots)	3.40
Zinc (in ingots)	2.50
– Chemical Products	
Sulphur (bagged)	1.00
Sulphur (in barrels)	0.75
Fertilizers (bagged)	0.90
– Solid Fuels	
Lignite bricks (stacked)	1.30
– Construction Materials	
Sand (in boxes)	0.60
Kaolin (bagged)	0.77
Cement Bagged	1.00
Cement In barrels	0.90
Gypsum (bagged)	0.83
– Wood and Wood Products	
Rubber (in bales. bags or boxes)	0.50
Ruber (in plates)	0.60
Cork	0.24
Wood Soft	0.70
Wood Hard	1.00
Paper (in rolls)	0.40
Paper (parcels)	0.80
Paper paste (compressed bales)	0.60
Plywood	0.65
Timber ties	0.77
– Food Products	
Rice (in barrels)	0.53
Rice (bagged)	0.70
Oats (bagged)	0.43
Sugar (bagged)	0.80
Beverages (in barrels)	0.60
Coffee (bagged)	0.55
Frozen meat (boxed)	0.48
Frozen meat (bagged)	0.44
Canned meat (boxed)	0.60
Barley (bagged)	0.60

TABLE 3.4.2.3.1.1. (Continued).

SOLID BULK MATERIALS	γ (t/m³)	φ (°)
SOLID BULK MATERIALS (continued)		
Crushed stone	1.80	40°
Gypsum and Plaster	1.25	25°
— Waste Products		
Demolition waste	1.30	35°
Urban debris	0.60	—
Compressed manure	1.80	45°
Loose manure	1.20	45°
Heavy scrap	1.60	35°
Light scrap	1.20	30°
— Food Products		
Sugar	0.75	35°
Frozen meat	0.35	—
Cereals Rice	0.60	25°
Oats	0.45	30°
Barley	0.65	25°
Rye	0.80	35°
Corn	0.75	25°
Millet	0.70	25°
Wheat	0.75	25°
Rape	0.70	25°
Fodder	0.17	—
Fruits and Vegetables	0.75	30°
Soybeans	0.85	60°
Cereal or soy flour	0.50	45°
Fish meal	0.80	45°
Ice	0.90	30°
Bones	0.40	—
Legumes	0.80	30°
Crushed malt	0.40	45°
Feed	0.50	45°
Dried sugar beets	0.30	40°
Common salt	0.90	45°
Sunflower seeds	0.55	—
Semolina	0.55	30°
Tubers	0.75	30°
— Plants		
Flax	0.60	25°
B) LIQUID BULK MATERIALS		
— Petroleum products		
Crude oil	0.80	—
Fuel oil	0.80	—
Gas oil	0.80	—
Gasoline	0.75	—
Liquified gases (natural gas. methane. butane. etc.)	(*)	—
— Chemical Products		
Chlorydric acid Up to 40%	1.20	—
Nitric acid up to 40%	1.25	—
Sulphuric acid up to 50%	1.40	—
Acetone	0.80	—
Ethylic alcohol	0.80	—

STACKED MATERIAL	γ (t/m³)
Rye (bagged)	0.63
Coconuts (boxed)	0.40
Coconuts (bagged)	0.53
Citrus fruits (boxed)	0.40
Soybeans (bagged)	0.72
Flour (in barrels)	0.66
Flour (bagged)	0.85
Bones (bagged)	0.60
Condensed milk (in barrels)	0.60
Condensed milk (boxed)	0.50
Dehydrated milk (boxed)	0.50
Dehydrated milk (bagged)	0.53
Corn (bagged)	0.65
Butter (in barrels of boxes)	0.60
Fresh or frozen fish (boxed)	0.50
Banannas (boxed)	0.26
Cheese (boxed)	0.70
Salt (boxed)	0.70
Salt (bagged)	0.90
Sunflower seeds (boxed)	0.50
Sunflower seeds (bagged)	0.48
Tapioca (bagged)	0.65
Tea (in bags)	0.35
Wheat (bagged)	0.65
Tubers (boxed)	0.40
Tubers (bagged)	0.60
Grapes (boxed)	0.25
Plants (boxed)	0.60
Plants (bagged)	0.50
— Animal and vegetable products	
Cotton (baled)	0.37
Esparto grass (baled)	0.25
Wool (compressed bales)	0.60
Moist skins (baled)	0.55
Dried skins (baled)	0.20
Dried skins (compressed bales)	0.24
— Petroleum products (in barrels)	0.50
— Oils	
Fish (in barrels)	0.60
Vegetable (in barrels)	0.55
Latex (in barrels)	0.70
Molasses (in barrels)	0.55
— Containers	0.50-0.70
— Vehicles	
Motor vehicles (empty)	0.25
Motor vehicles (as scrap iron in cages)	1.00

TABLE 3.4.2.3.1.1. (Continued).

SOLID BULK MATERIALS	γ (t/m³)	ϕ (°)	STACKED MATERIAL	γ (t/m³)
LIQUID BULK MATERIALS (continued)				
Aniline	1.00	—		
Bencine	0.70	—		
Carbon sulphide	1.30	—		
— Oils				
Creosote	1.10	—		
Linseed	0.95	—		
Mineral	0.93	—		
Fish	0.90	—		
Castor oil	0.97	—		
Vegetable	0.92	—		
Latex	1.00	—		
Molasses	1.25	—		
— Wines and Beverages				
Water Fresh	1.00	—		
Water Salt	1.03	—		
Beer	1.03	—		
Milk	1.03	—		
Wine	1.00	—		

NOTES:

(*) When determining transmitted loads by the storage of liquified gasses, the specific weight and height of storage shall not be significant parameters.
The significant parameter shall be the pressure used by the storage installation to maintain the liquified gas.

(h) moist.

(s) dry.

5.14.3 Loads for Harbour Cranes

	Rotating cranes	Container cranes and other transhipment gear
Bearing capacity [t]	7–50	10–80
Dead weight [t]	180–350	200–1200
Portal span [m]	6–19	9–45
Clear portal height [m]	5–7	5–13
Max. vertical corner load [kN]	800–3000	1200–8000
max. vertical wheel surcharge load [kN/m]	250–600	250–700
Horizontal wheel load		
transverse to the direction of the rail	up to approx. 10 % of vertical load	
in the direction of the rail	up to approx. 15 % of the vertical load of the braked wheels	
Claw load [1)] [kN]	Mobile crane up to 4800	

1) Prerequisite is a zone of 40 m² subject to no other loads; the claw load can be taken as distributed over 4 m².

Table R 84-1. Dimensions and characteristic loads of rotating and container cranes

5.5.6 Loads Outside the Waterfront Cargo Handling Area

Outside the waterfront cargo handling area, the following live loads are taken as the basis in accordance with [140], working on the basis of 300 kN gross load for 40' containers and 200 kN for 20′ containers.

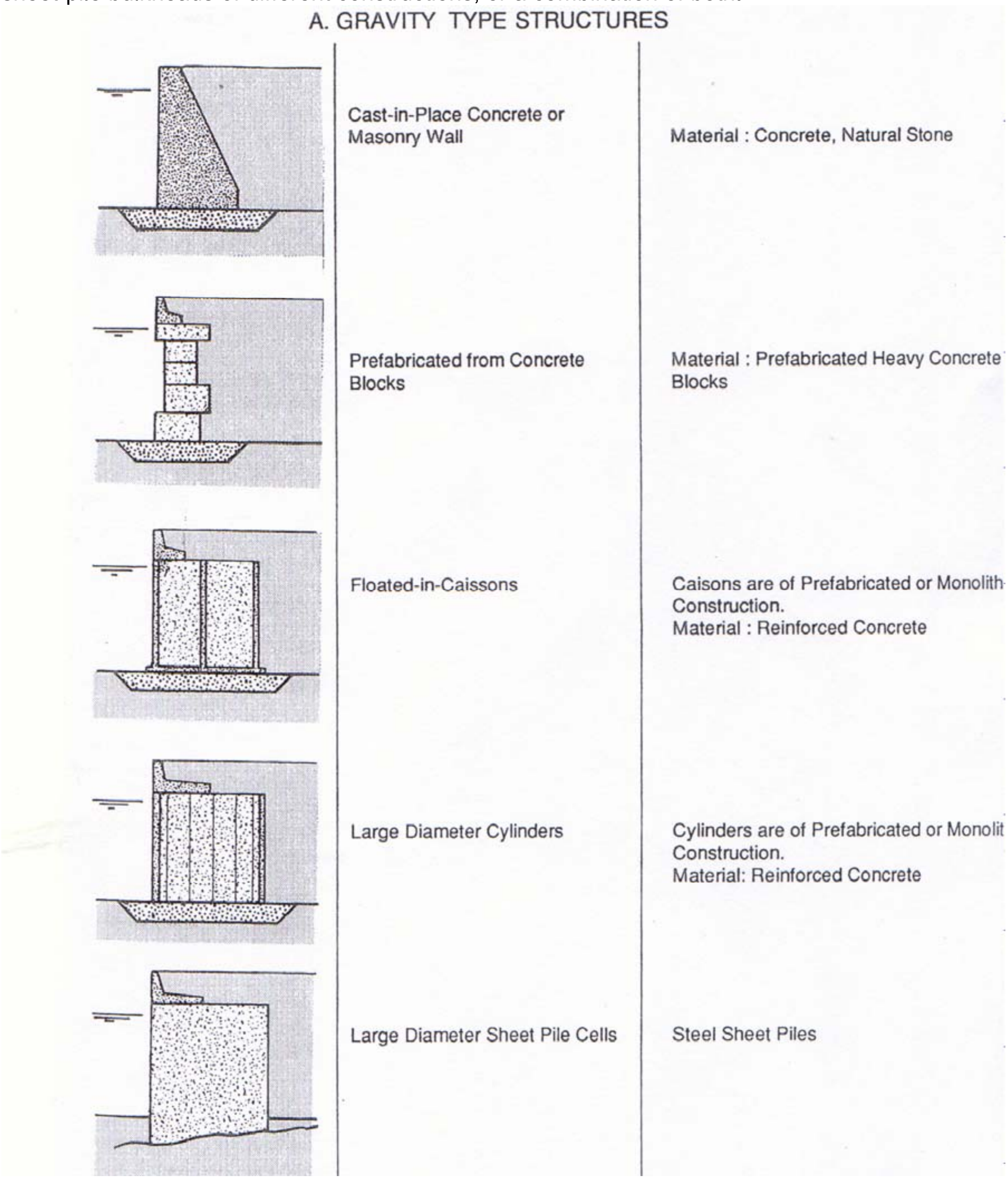
- Light traffic (cars) 5 kN/m²
- General traffic (trucks) 10 kN/m²
- General cargo 20 kN/m²
- Containers:
  - empty, stacked 4 high 15 kN/m²
  - full, stacked 2high 35 kN/m²
  - full, stacked 4 high 55 kN/m²
- Ro-Ro loads 30–50 kN/m²
- Multi-purpose facilities 50 kN/m²
- Offshore feeder bases 55–150 kN/m²
- Paper depending on the
- Timber products bulk/stacking height,
- Steel calculating values of the
- Coal weight density according to
- Ore DIN1055, part 4

Further details regarding the material properties of bulk and stacked goods are to be found in the tables of ROM 02.-90 [197].
When calculating the active earth pressure of retaining structures, as a rule the differing loads in the cargo handling and container area can be grouped together to produce an average surface load of 30 to 50 kN/m².

4. Port Infrastructure

4.1 General

Port marine structures presently in use are either bottom fixed or floating. These course notes are dedicated to fixed structures only. The fixed port marine structure can, in general, be classified as follows: soil retaining - piled - and structures on special foundations. Combinations of all of the above basic types of structures are also used. There are many types of waterfront constructions used to date. The most typical of them are shown in figure 10.
All port related marine structures can be categorized as soil retaining structures, piled structures, and structures that are rested on special foundations.
Soil retaining structures may be subdivided into gravity-type structures, flexible structures such as sheet-pile bulkheads of different constructions, or a combination of both.



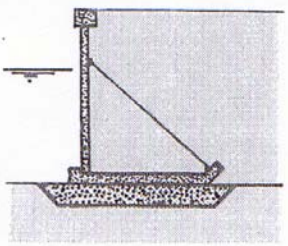
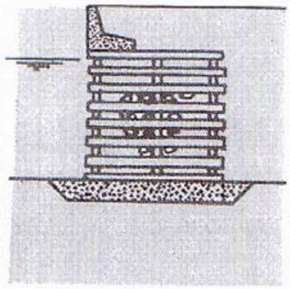
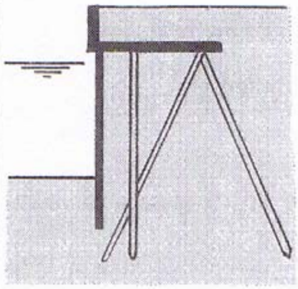
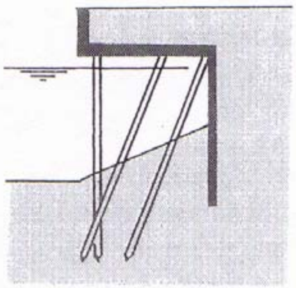
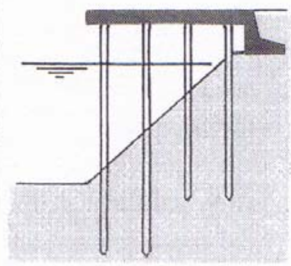
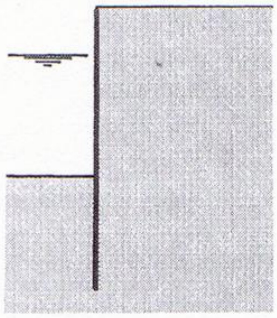
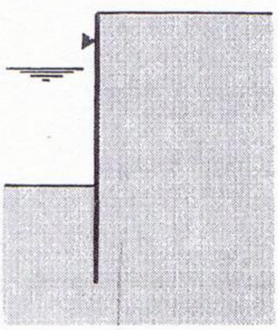
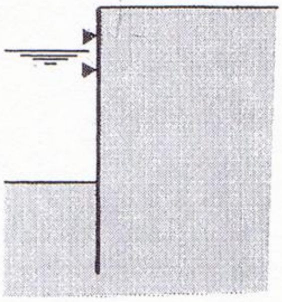
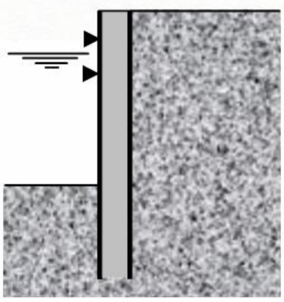
	Angle Type Wall	Built from Prefabricated Elements, or Prefabricated Sections. Material: Reinforced Concrete.
	Floated in or Erected-in-Place Cribs	Material: Timber, Prefabricated Concrete Elements, Natural Stone.
	B.PILED STRUCTURES	
	Relieving Platform with Front Sheet Pile Wall	Monolithical or Prefabricated Concrete fo Platform Construction. Steel or Reinforce Concrete Piles and Sheet Piles
	Relieving Platform with Rear Sheet Pile Wall	Monolithical or Prefabricated Concrete fo Platform Construction. Steel or Reinforc Concrete Piles and Sheet Piles.
	Piles Supported Platform	Monolithical or Prefabricated Concrete fo Platform Construction. Steel or Reinforce Concrete Piles of Regular Construction or Large Diameter (up to 1.6m)

Fig. 10 Overview of main types of quaywalls.

C. SHEET PILE BULKHEADS

	Cantilever Bulkhead	Steel or Reinforced Concrete Piles
	Single Anchor Bulkhead	Steel or Reinforced Concrete Piles. Anchors: Regular Steel Tieback with Deadman, Ground Anchor, Anchor pile, Pile system, Screw Anchor, M.V.-pile, Others
	Multi Anchored Bulkhead	Same as Single Anchor Bulkhead
	Slurry wall Single/multi anchored	Steel or Reinforced Concrete Piles. Anchors: Regular Steel Tieback with Deadman, Ground Anchor, Anchor pile, Pile system, Screw Anchor, M.V.-pile, Others

D. STRUCTURES ON SPECIAL FOUNDATIONS

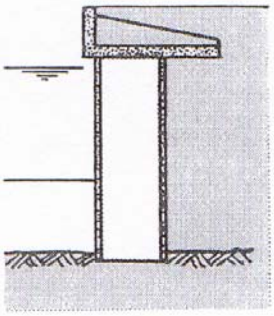
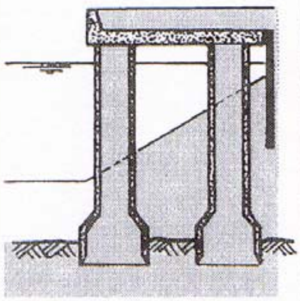
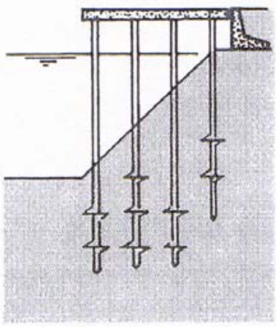
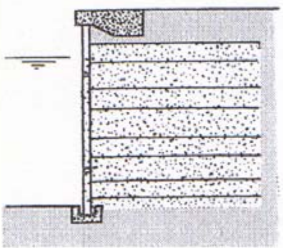
	Large Diameter Cylinders with Relieving Platform	Cylinders and Platform are of Prefabricated or Monolithic Construction . Material: Reinforced Concrete.
	Platform Supported on Deep Caissons	Steel or Concrete Caissons, Monolithical or Prefabricated Reinforced Concrete Platform
	Platform Supported on Piles with Increased Carrying Capacity	Steel or Concrete Screw Piles or Piles with Local Widenings. Monolithical or Prefabricated Reinforced Concrete Platform
	Reinforced Earth Wall	Prefabricated Concrete Elements and Metal Anchor Strips

Fig. 10

4.2 Gravity-type structures

Gravity-type structures are those that develop their resistance to soil pressure and miscellaneous loads primarily form their own weight. This type of construction is typically used where the foundation material does not permit pile driving or where heavy ice, waves, or other environmental forces can be dangerous to the piled structures. Various kinds of gravity-type structure are used. The gravity-type quay wall may be built in the form of mass concrete walls or walls composed from heavy prefabricated concrete blocks or elements, as floating-in-caissons, cantilever or internally anchored structures, sheet-pile cells, and others.

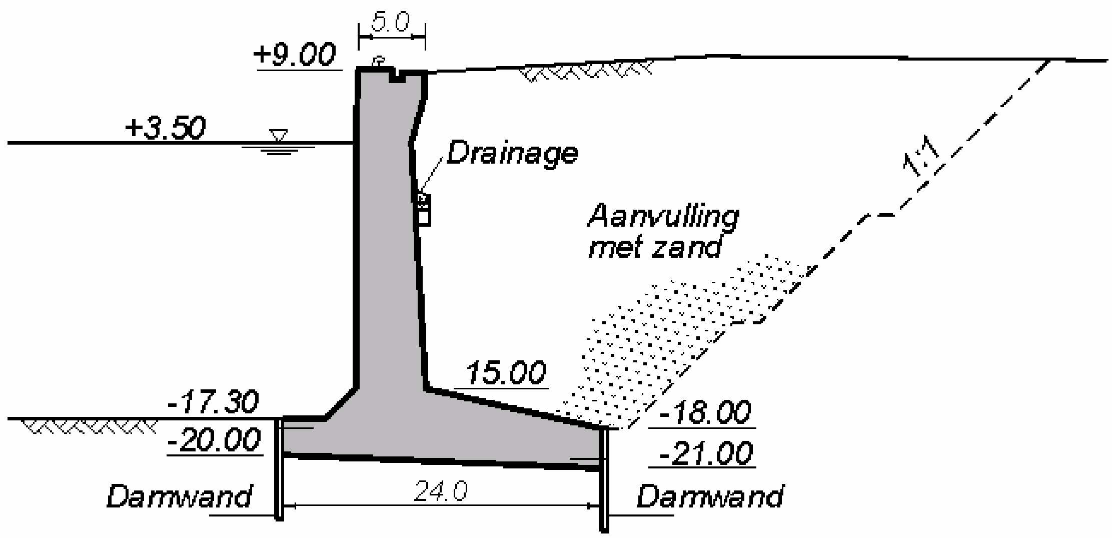


Fig. 11 Gravity-wall

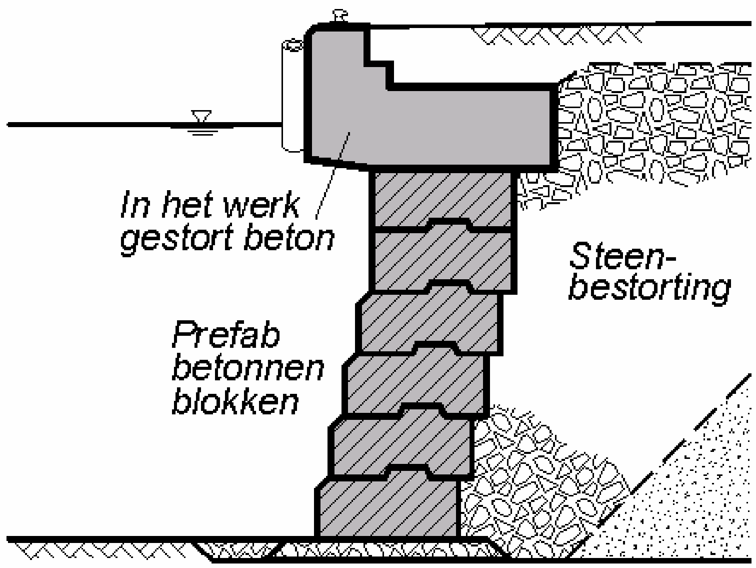


Fig. 12 Block-wall

4.3 Sheet-pile type structures

Sheet-pile bulkheads are structures formed from flexible sheeting restrained by an anchor system and by penetration of sheeting below the dredge line. Sheet-pile anchorages may be provided in a variety of ways, such as by tie backs secured to anchor walls of different constructions, by anchor piles, or ground or rock anchors. Anchored sheet-pile bulkheads may have just a single row of anchors, or be multianchored. Sheet piles of different shapes and materials are used for sheet-pile bulkhead construction. The type of sheeting and anchorage typically depends on the height of the structure, the kind of foundation material, and live loading.

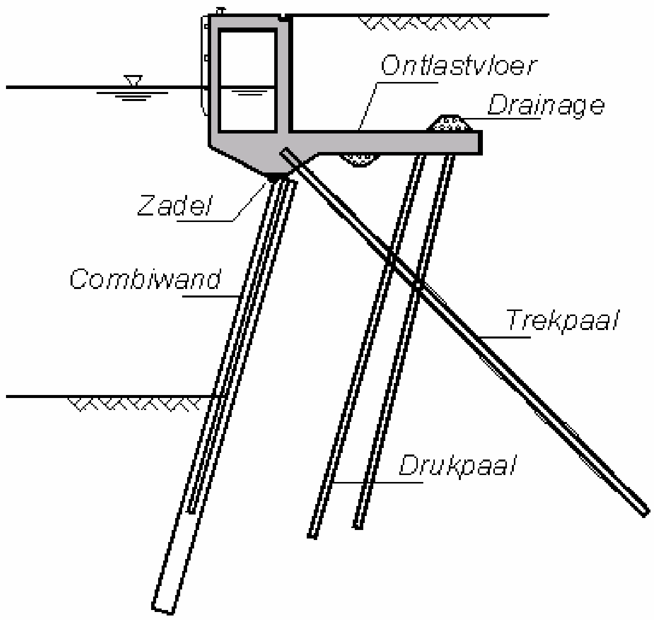


Fig. 13 Quaywall with relieving floor

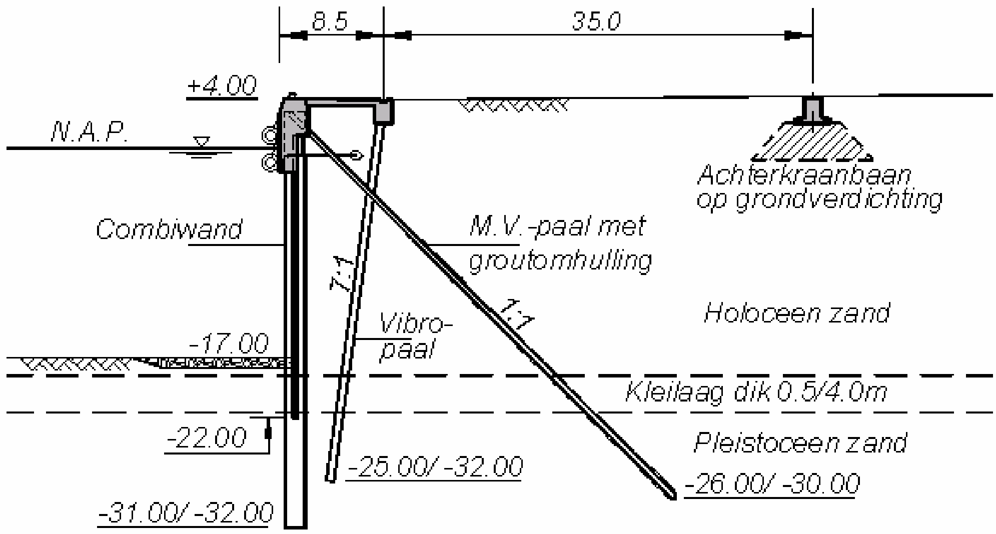


Fig. 14 Quaywall with small relieving floor

4.4 Jetties

Piled structures are those whose stability depend on pile bearing and lateral load-carrying capacity.
Piles are usually designed to carry vertical and lateral loads due to the structure deadweight and live load, and miscellaneous sources of lateral loads such as mooring forces, soil lateral pressure, and others. Pile cross section and material primarily depends on pile length, foundation material, kind of performance whether the pile is basically designed to carry vertical, lateral, or combined load), and pile-driving techniques.

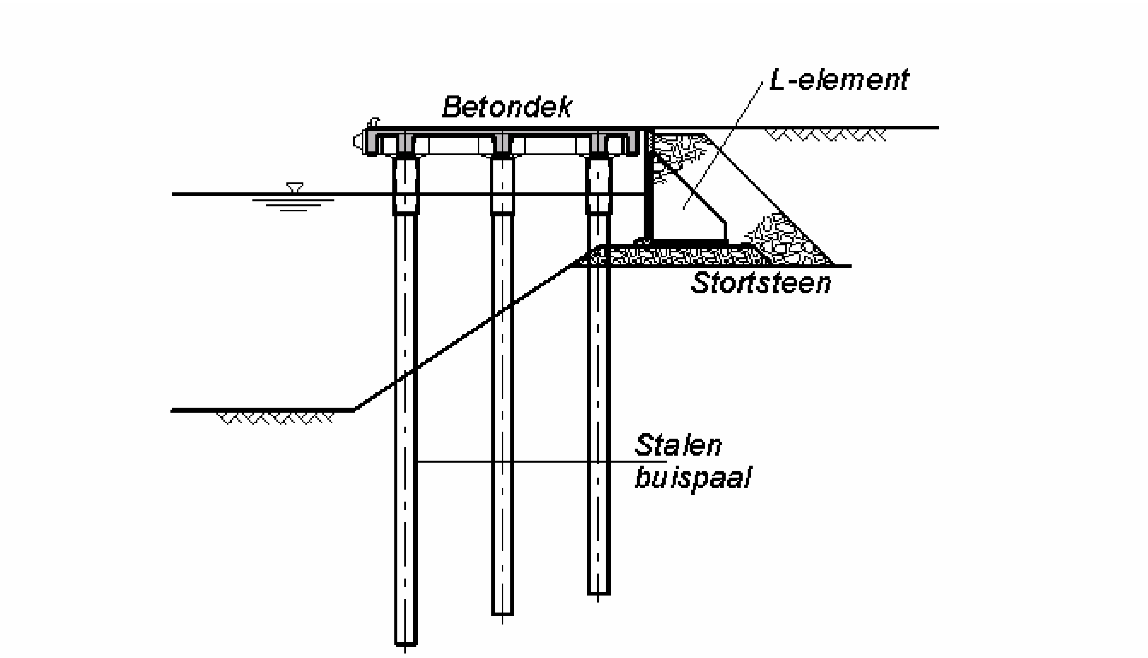


Fig. 15 Jetty on vertical piles

4.5 Ro-ro facilities

Ro-ro facilities structures are special designed structures fitting for ro-ro traffic.
These structures are mostly constructed like jetties with special related parts inclusive fender beam, movable deck.
Some typical examples of Ro-ro facilities are displayed in figures 16 and 17.

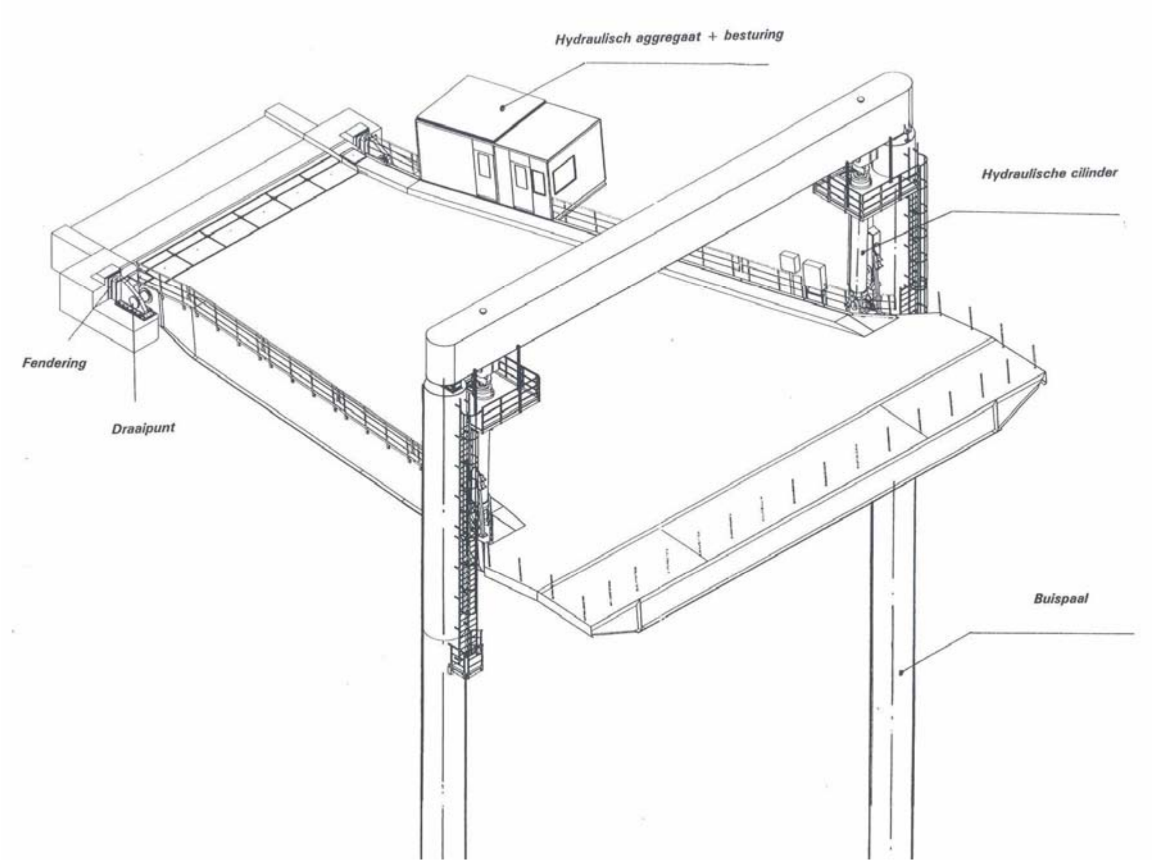


Fig. 16 Ro/ro terminal with pipe piles.

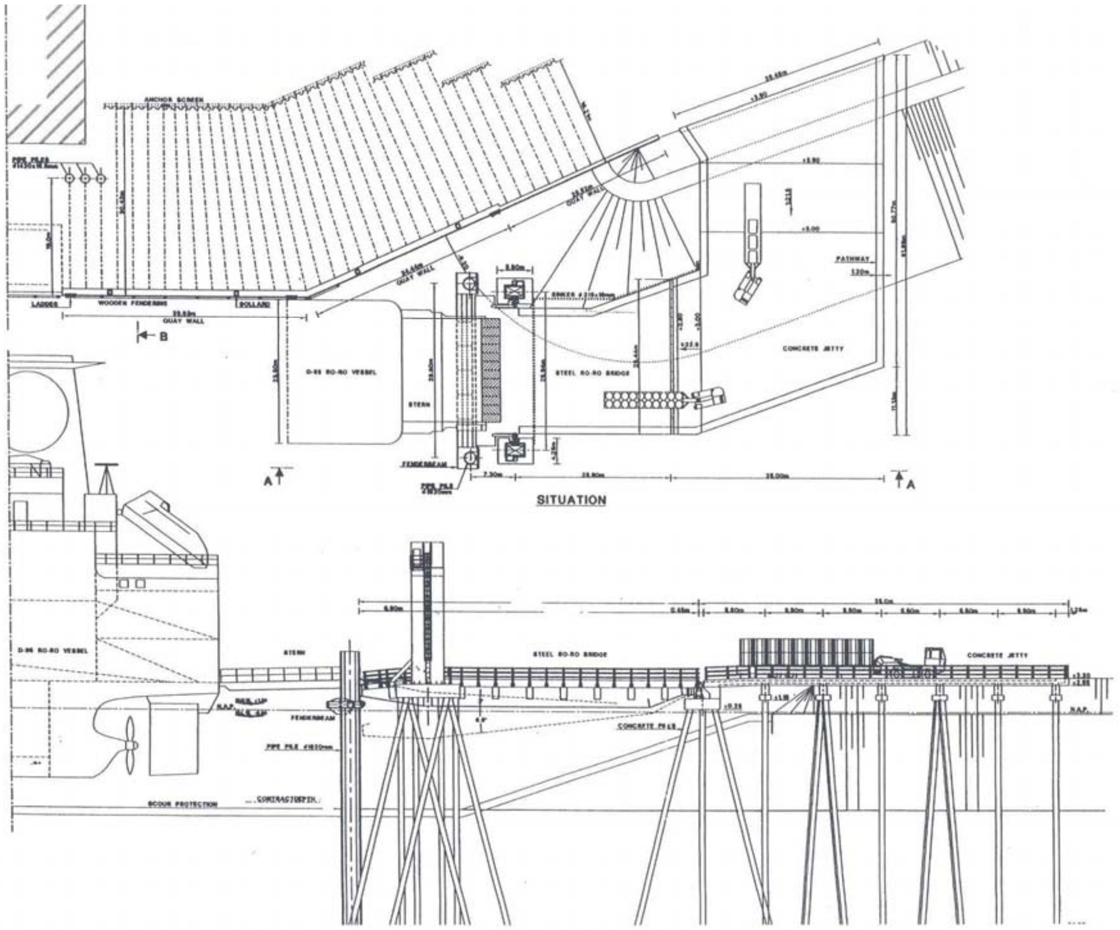


Fig. 17 Ro/ro terminal concrete piles.

4.6 Guiding structures/dolphins

Guiding structures are primarily used to guide the ship while mooring. These structures protect concrete structures from vessel impact.
Guiding structures are used with jetty-like structures.

Dolphins are used for mooring ships. They may consist of a single pile or combination of more piles.

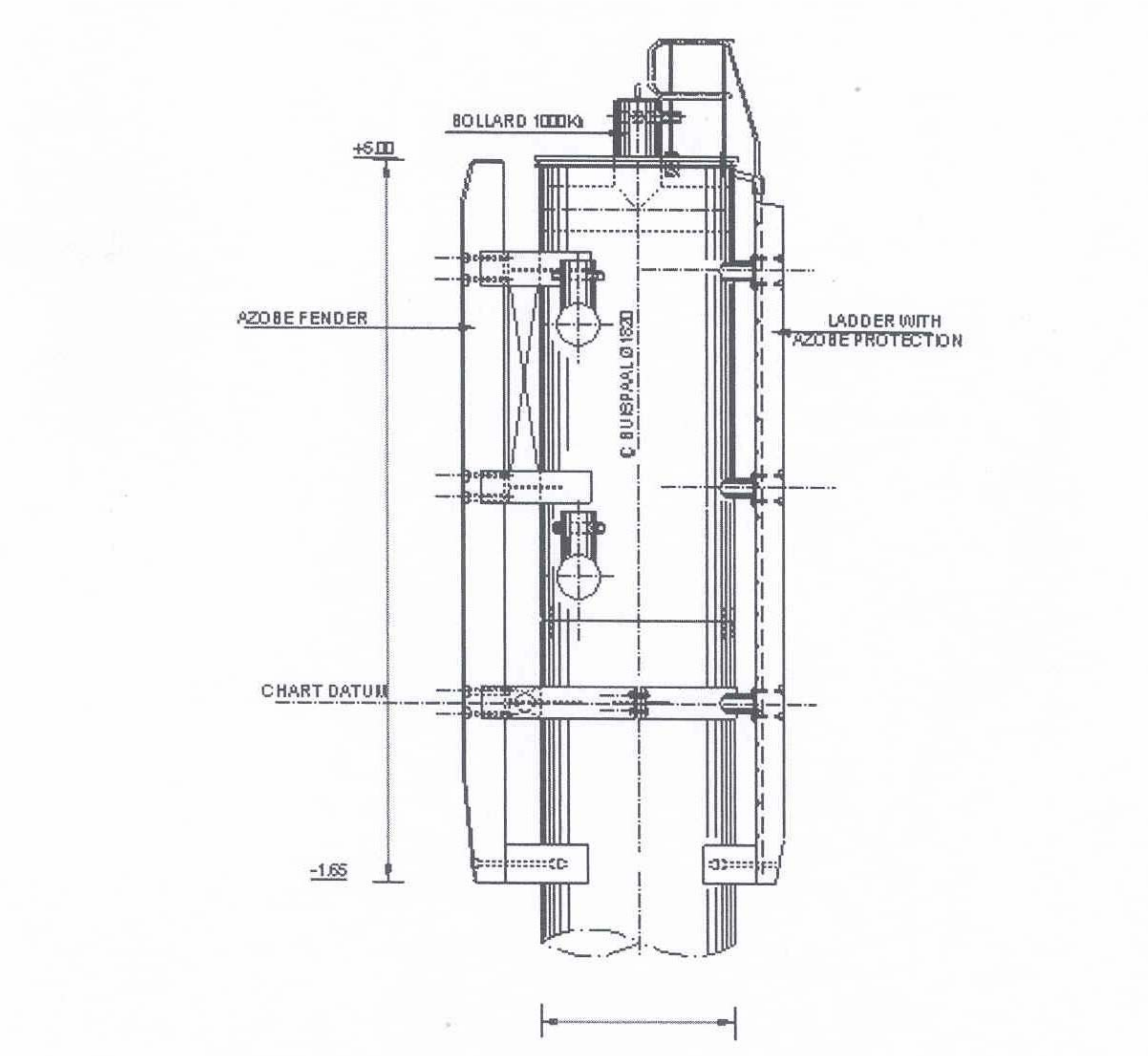


Fig. 18 Dolphin head.

5. Design of Port Infrastructure

5.1 General

Safety plays a very important role in the design process. Although until recently the attitude to the safety of a structure was based on a deterministic approach, at present, under the influence of national and European regulations, a fundamental or probabilistic approach is taken.

The deterministic approach to safety, which however is still often taken in the designing of quay structures, allows a set margin between the characteristic values of loads and the characteristic value of the strength that must be maintained in order to ensure the safety of the structure. This is reached by using safety factors that are based on a stochastic distribution of the loads and the strength, but these are largely based on experience.

The probabilistic approach to safety is based on the principle that a design that is being developed within defined maximum probability of failure. For this, all calculation variables are considered stochastic. In stipulation of the maximum acceptable probability of failure is based on the principle that: the greater the consequences of failure, the smaller the acceptable probability of failure. The failure of the structure can be caused by various mechanisms, each with its own probability of failure. Together these mechanisms determine the total probability of failure. The objective of the methodology is that, assuming a stipulated maximum probability of failure for the structural system, probability of failure margins are allocated to the various mechanisms. By means probabilistic analyses for each mechanism it is possible to verify whether the chosen dimensions of the structure satisfy the safety requirements.

However, to fulfil meeting the safety requirements only is not sufficient to ensure that a structure will satisfy the stipulated requirements throughout its entire lifetime. The national and international regulations add supplementary requirements. For example, verifications of quality must me made in the design phase and during the construction to ensure that the basic format and the requirements on which the design and construction are based satisfied. During the period of use, the maintenance of the original objectives of the design must also be guaranteed. This can be provided for by means of a management and maintenance plan based on the Programme of Requirements which lays down the necessary maintenance and the inspections.

The current Dutch regulations stipulate that a probabilistic approach should be taken to safety during the design process. Reference is made to the most relevant NEN-standards for the design and of quay structures: *NEN 6700 TGB 1990, General Basic Requirements; NEN 6702, Loads and Deformation, NEN 6740 TGB 1990 Geotechnology, Basic Requirements and loads and NEN 6743 Geotechnology, Calculation method for foundations on piles, bearing piles.*
In *NEN 6700*, three safety classes based on the consequences of failure are defined. For each safety class, the maximum probability of failure of the structure during the construction phase and during the phase of use is stipulated for the limit states under consideration. For the calculation, variables are considered stochastic variables. However, a problem arises in that the knowledge about the static distribution of many variables is limited.

This is especially true for loads and load combination.
To maintain the practicality of the design method and to avoid probabilistic analyses, the NEN-standards are based on a semi-probabilistic approach to safety. Because the Dutch regulations do not treat sheet pile walls and quay structures in detail the need arose for a more specific approach that fits in with the probabilistic safety philosophy. For this reason in 1993 *Cur Report 166, Sheet pile structures*, was published [6.1]. This handbook provides sufficient points of departure for the design and construction of simple quay structures, in which the sheet pile wall is the most important member. The structure of a quay with a relieving platform differs so greatly from that of a sheet pile wall structure that it demands a much more specific approach.

Besides this, for many years work has been in progress to develop and issue an European Directive for Structures. The regulations are also based on a semi-probabilistic approach to safety. The European programme, much of which has already been implemented, is set down in a number of codes that cover the various areas of design. Furthermore, each member state can add a national appendix in which specific parameters can be stipulated within the degree of freedom indicated in the code. These are called national stipulate parameters. The national Dutch appendix has not yet been developed and in due course, together with the set of Eurocodes, it will replace the current NEN-standards.

The following Eurocodes are being developed:

- Eurocode EN 1990:2002 Basis of Structural Design
- Eurocode 1 EN 1991:2002 Loads on structures
- Eurocode 2 NVN-ENV 1992:1995 Design of concrete structures
- Eurocode 3 NVN-ENV 1993:1995 Design of steel structures
- Eurocode 4 NVN-ENV 1994:1995 Design of composite steel and concrete structures
- Eurocode 5 NVN-ENV 1995:1995 Design of timer structures
- Eurocode 6 NVN-ENV 1996:1995 Design of masonry structures
- Eurocode 7 NVN-ENV 1997:1995 Geotechnical design
- Eurocode 8 NEN-ENV 1998-1995 Design of earthquake-resistant structures
- Eurocode 9 NVN-ENV 1999:1995 Design of aluminium structures.

It is necessary to consider whether to use the existing and proven design methods or take the fundamental approach to safety of which there is still limited experience in the design of quay structures. For the first category design recommendations specifically directed towards port structures EAU (Empfehlungen des Arbeidsausschusses Ufereinfassungen) are available. These recommendations, which are issued by the German Port Construction Association (Duitse Hafenbau Technisches Gesellschaft), include the fruits of many years practical experience of the design, construction and use of port structures. From time to time, the EAUs that are extensively used in internationally practice are revised. The last two versions are: the EAU 1990 and the EAU 1996.

All in all the following design methods and associated safety philosophies are appropriate:
- Design approach according the EAU 1990
- Design approach according the EAU 1996
- Design approach followed in the *'Handbook on Sheet Pile Structures'* based on the NEN-standards
- Design approach according the Eurocode 7.

Before a choice is made it is useful to explain the basic assumptions and characteristics of the design methods that have been mentioned. For example, the way in which the design values of the earth pressures on the sheet pile constructions are determined is crucial. For most design methods, the design values of the soil properties are obtained by dividing the representative values by a partial material factor that is greater than one. In some cases, these partial factors are dependent on the chosen safety class. With the design values of the soil properties as basis the design values of the earth pressures, the distribution of forces of the sheet pile wall is calculated. Such an approach has the disadvantage that the calculated distribution of forces does not reflect the actual behaviour of the structure in the limit state that is under consideration. An approach in which the design values of the soil properties are equated with the representative values does not have this disadvantage. When the structures are being dimensioned, the calculated distribution of forces is increased by using partial safety factors in order to attain the desired safety level.

Therefore it is recommended to use for the soil properties a factor 1.0 which means that the characteristics values are representative values.

Choosing a design approach, safety philosophy and regulations
A choice is based on the following basic assumptions:
- As close as possible to the current regulations
- Taking a semi-probabilistic design approach
- Using what has been previously developed
- Taking into consideration the specific characteristics of the type of quay wall with relieving structure
- Aspiring to a simple and effective design approach.

A semi-probabilistic design method has been developed for quay wall with a relieving floor.

For gravity type structures and jetties no such analysis has been made to date. This means that the design is carried out with overall safety factors.

However it is necessary to develop for pile infrastructure also for other structures as semi-probabilistic design approach.

As a basis the semi-probabilistic design method as developed in the 'Handbook Kademuren' for the cantilever floor structure can serve as basis.

To compare structures it is essential that the same design procedure is adopted.

5.2 Design aspects of quay walls with sheet piling and relieving floor

5.2.1 General

On first sight, when quay walls concepts are being developed design seems to have a high degree of freedom. The many possible types of structure seem to confirm this. It must be realised that this diversity can partly be explained by differences in local conditions, such as the location of the quay, substrata, climatic, hydraulic and geo-hydrological aspects. Nautical aspects such as the types of ships that are expected also play a parting this. Moreover, the quay concept depends to a high degree on whether the possible implementation options determined by the location permit construction in a dry pit. Construction in a dry pit implies that the structure is built up from a surface level that is created either by making excavation or by landfill in which the groundwater-level may possibly be artificially lowered. Otherwise, the quay is constructed in or over open water. Furthermore, the designer must realise that the building of quay walls is always accompanied by a variety of problems. New problems are always experienced when the limits are extended or new concepts are being built. To illustrate this, the experience gained in the use of a new combined sheet pile system and the use of new foundation members can be used. In addition, problems arise that relate to the behaviour and use of the quay structure are considered. If this experience is considered the conclusion that to reach feasible and effective concepts the designer must use this freedom wisely. In the development of quay concepts, a balance must be found between competing aspects such as construction costs, durability and robustness. The feasibility and the accompanying construction risks play a very dominant part in the evaluation of the quay concept. To reach a balanced and effective design it is recommended that sufficient freedom be left in the design process to organise the development of several concepts and to analyse them.

In those cases where heavy demands are set, the deformation behaviour of the structure can also play an important role in the evaluation.

The final choice can then be based on the results of this comparative analysis.

The following aspects are taken into consideration:
- construction costs and construction time
- costs of management and maintenance
- implementation risks
- robustness, susceptibility to catastrophe and overloading
- clear understanding of the interaction of forces
- durability.

5.2.2 Structural system

In this section, the most important design aspects of parts of quay structure are considered from the structural point of view. The quay concept on which this chapter is based consists largely of a superstructure that also functions as a relieving structure for the retaining sheet piles. the superstructure is supported by a bearing sheet pile wall on the water side and a system of tension and bearing piles. In addition to supporting the superstructure, this foundation system must also provide stability to the quay wall. The various loads that arise during the serviceability stage act on the superstructure and are carried by this to the foundation. The soil retaining function of the structure is provided by the sheet piles. Where there is a deep lying relieving platform the superstructure takes over a large part of the soil retaining function. Usually the aim is to place the bearing sheet piles directly under or close to the crane track on the water side.

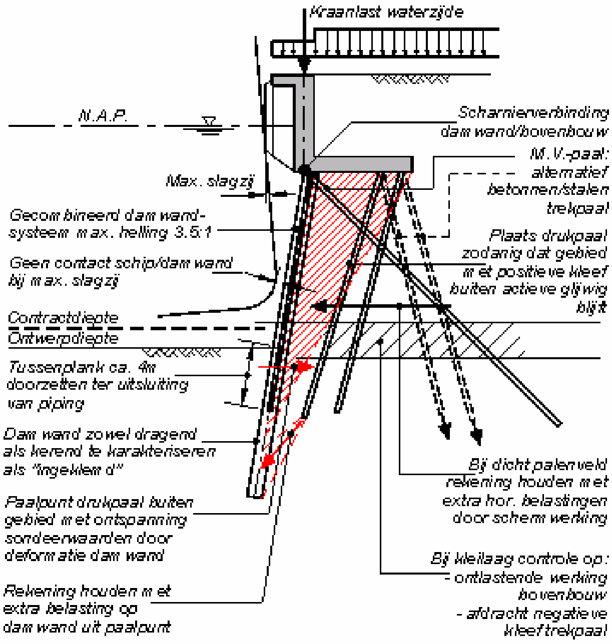


Fig. 19 Aspects to be considered of quaywall with relieving floor.

5.2.3 Design aspects relieving structures

The concept of the relieving structure is the essence of this part of structure of the quay wall; the relieving of the earth pressures that work on the retaining sheet piles. The aspects that play a role in the design and construction of relieving structures are discussed below. These mainly relate to the determination of the width and construction depth of the relieving structure. Cost and feasibility are the most important criteria on which the choice is made.

The construction width is determined from an optimisation in which various aspects are considered including; the relieving of the sheet pile, the effects on the pile foundation and the dimensioning of the relieving platform. In addition, the minimum width that derives from the design of the foundation must be taken into consideration. Thus, the foundation members such as the sheet pile and the pile trestle system must be given a place under the relieving structure. The minimum distance between these members, that is required for technical reasons and the implementation of the design determines the minimum width of the relieving structure.

In a specific situation various motives influence the choice of the construction depth including:

- saving on the sheet pile by the reducing moments and pile depth
- shortening the length of the sheet piles to limit the installation risks
- restricting the length of foundation members such as tension and bearing piles in relation to availability and feasibility
- saving on the number of tension members in the pile trestle system by increasing the vertical load component with soil.

Reduction of active earth pressure on the sheet pile

The use of a relieving structure primarily reduces the active earth pressure on the uppermost part of the sheet pile wall. The most important effects are the saving on the cost of the sheet pile through a reduction of moment and pile depth. When the relieving platform is situated of just under the level of the quay area, the reduction is more or less limited to the top load consisting of yard and traffic loads. When deeper relieving structures are made the earth pressure reducing effects are much greater. Figure 20 shows the principles on which the determination of working of a relieving platform is based. The influence of the lop load, working on the level of the underside of the relieving structure on the vertical effective in the position of the axis of the sheet pile is shown.

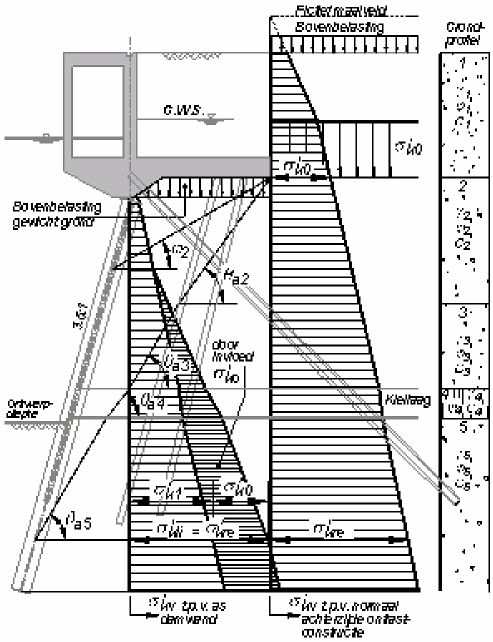


Fig. 20 Principal of quaywall with relieving floor.

The area of influence begins where the line cuts the axis of the sheet pile at the angle of internal friction. The full influence is valid when the composite line, composed of the various active sliding planes, does this. The sliding plane angle a depends on the angle of internal friction, the wall friction angle δ, the slope of the ground surface β and the slope of the sheet pile δ.

The expression for the case in which δ, β and α are not zero is:

For a further indication of the variables and the drawing conventions reference may be made to figure 21.

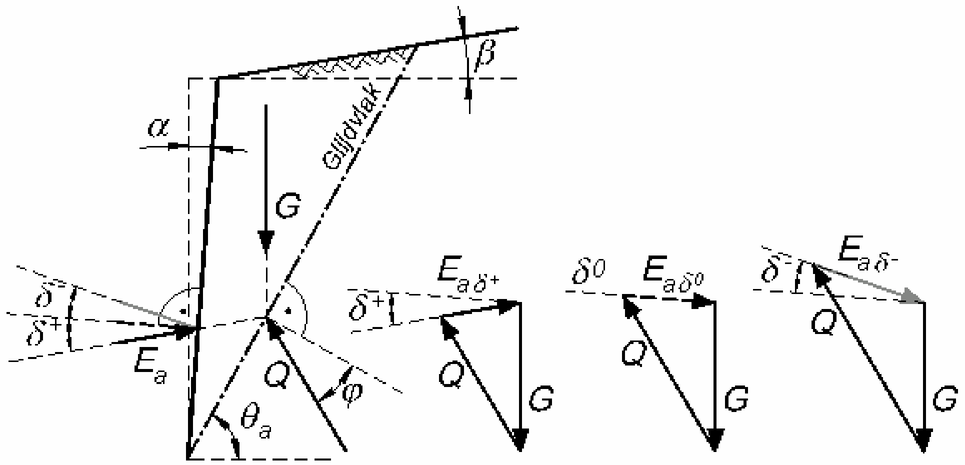


Fig. 21 Sign convention.

Likewise it is to important to ascertain that the shear stresses that develop in the soil mass under relieving structure development, are high enough to ensure that the relieving work comes into play. If a weak clay layer is present or if stiff sheet pile is used the level of the shear stresses in the clay layer may be insufficient. This situation can also be caused by water over pressures.

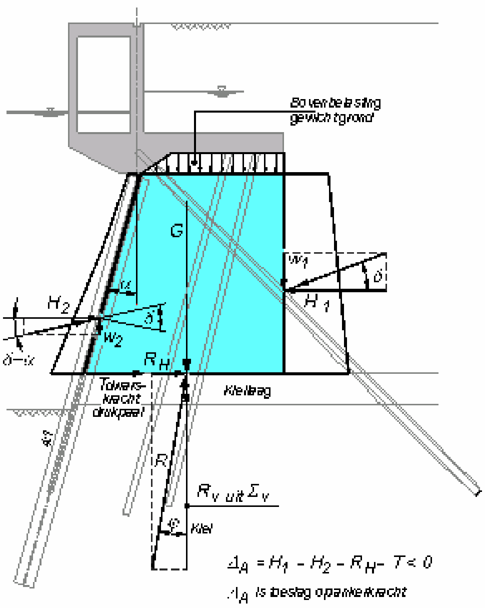


Fig. 22 Verification of the relieving load of a relieving platform in the presence of a clay layer.

The verification implies that the horizontal balance of the soil mass above the clay that is located between the sheet pile and the vertical behind the relieving platform is considered. The magnitude of the stabilising shear forces is derived from consideration of the properties of the clay layer on the limit surface.
The following horizontal forces play a part in this:

- working to the left: the resultant of the horizontal earth pressure H1 working on the vertical behind the relieving platform
- working to the right: the resultant of the horizontal reaction forces H2 exerted on the soil mass by the sheet pile (equal and opposite to the earth pressure on the sheet pile)
- working to the right: the horizontal component RH of the reaction force R.

The vertical reaction force RV is derived from a consideration of the vertical balance of the soil mass based on effective stresses. The horizontal component is determined from the expression: $R_H = R_V \tan \varphi_{ger}$, in which φ_{ger} is a reduced angle of friction that corresponds with the deformation that occurs in the clay layer and assuming the appropriate stress-strain curve.
If necessary, water stresses must also be taken into consideration. If H2 + RH $\geq$ H1 the relieving working of the relieving structure can be assumed to be as shown in figure.
If H2 + RH < H1 this cannot be entirely accepted and a higher horizontal load on the sheet pile must be taken into consideration.

It is clear that in the hypothetical case in which the friction resistance of a deep lying layer is equal to zero, the relieving working up to the layers above is rendered impossible. For some large quay projects, this phenomenon is taken into account by means of a supplement
ΔA on the anchor force. In this, the lateral force capacity of the piles, T in kN/m, is taken into account. The expression for the supplement to the anchor force in that case is:
ΔA = H1 - H2 - RH - T. In figure only bearing piles are found in the interface plan. It should be noted that the check can also be carried out with the aid of a geotechnical computer program that is based on the finite element method, f.e. PLAXIS.

5.2.4 Design aspects of sheet pile

Bearing function and the position of sheet piles
The basis of this is that in addition to a soil retaining function, the sheet pile also has a bearing function so therefore positioning the bearing sheet piles on the water side directly under the crane track a good option. This principle is very suitable for cases when the distance between the water side crane track and the front of the quay wall is not too big. For a relatively big distance, the position of the sheet pile wall must be such that the optimum foundation system can be constructed.
For both technical and economic reasons it is not advisable to use sheet pile on the rear side of the relieving structure since all the earth pressures act on the sheet piles causing high anchor forces and requiring the use of relatively heavy sheet piles. When there is an underwater slope under the relieving platform the anchor force and the system length of the sheet pile can be slightly reduced. However, this influence is very slight because the passive supporting pressures that can be provided by an underwater slope are of very limited extent. Even so there are situations for which this solution is chosen, especially in those cases where an existing quay wall must be adapted to accommodate ships with a deeper draught.

Inclined sheet piles

Because the planned crane track is usually some distance from the front of the quay wall, it is advisable to use the available space and drive the sheet piles at an angle. When doing this it must be ensured that a ship cannot come into contact with the sheet piles during berthing or when moored. It is necessary to maintain sufficient space, this being determined by the maximum list of the ship, the maximum deviation from position and deflection of the sheet pile. Consideration may be given to allowing extra play so that the introduction of ships with a deeper draught can be accommodated. If the crane track on the water side is designed to be further inland, bearing in mind the maximum inclination, the positioning of the sheet pile, can be adjusted. It should be noted that in practice inclinations up to 3.5:1. If the inclination of the sheet piles a quay wall must continued round an angular bend, a transition structure with splined connecting piles is used. In principle, the slope of the sheet piles is reduced in stages until the sheet pile at the corner reaches a vertical position. Because this is a vulnerable part of the quay structure, great attention must be paid to the detailing of the transition structure.

If a quay wall is built at a site where recent land reclamation of infilling in has taken place the inclination of the piles is limited. During the installation of the sheet piles and the foundation members, the loosely packed sand fill is considerably compacted and sheet piles inclined towards the land side will be subjected to significant deformation.

The most important reason for inclining the sheet pile system is that as a bearing foundation member, the sheet pile system makes a considerable contribution to the stability of the quay wall and thus relieves the other members of the foundation system. An important additional effect is that the inclined position of the sheet pile creates space for the feet of the bearing piles in the pile trestle system. This space has a favourable influence on the required width of the relieving structure. Besides the positive effects, there are also negative effects. The inclination reduces the active earth pressures, but the reduction is even greater for the passive earth pressures.

When making a choice it is advisable to analyse the effects of the inclination of the sheet piles on the distribution of the forces of the sheet piles. Although the positive effects on the total design of the quay wall usually dominate, the inclination must not be exaggerated. A very effective solution is reached by combining the inclined bearing sheet pile wall with a tension pile driven at an angle of ca. 45 degrees. The component of sheet pile that is at an angle of the tension pile is then taken up by the sheet pile as an axial pressure load. For the bigger quay walls, the bearing capacity of the sheet piles can usually be adapted without incurring much extra cost. The above does not apply in situations with weak deep subsoil and must be adapted to the conditions encountered.

Static sheet pile system

As previously stated, the main function of the sheet pile wall is to retain the soil, making possible the handling of ships at the berths. The sheet pile wall is considered as a beam that is loaded by soil and water pressures. On the upper side, the sheet pile wall is anchored via the superstructure by an anchorage. At the foot, the sheet pile is supported by the passive soil resistance of the layers under the bed of the harbour. Within certain limits, the pile depth of the sheet pile wall can be varied. With a minimal pile depth, the soil layer providing resistance is just able to ensure the stability of the sheet piles. Greater pile depths lead to restrained/fixed end moments. The degree of restraint of the sheet pile wall depends on a number of factors such as the extra sheet pile length in relation to of the minimal length, the stiffness of the resistance-providing soil layers and the flexural rigidity of the sheet pile. The various calculation methods used give different results.

According to the Blum method that is based on the failure condition of the soil, when minimum active and maximum passive earth pressures occur, with adequate pile depth the sheet pile is fully restrained. By using calculations according to the principle of the sprung supporting beam, the soil is schematised by a system elasto-plastic springs. Only with sufficient deformation of the sheet pile wall the plastic branch of the soil elasticity is reached and does active earth pressures or passive soil resistances occur. With this approach to calculation, as in the case of the method of Blum - even for piles exceeding the minimum length - no fully restrained sheet pile wall is found.
The static system for sheet pile walls has the following extremes of schematisation:
- sheet piles supported at both ends;
- restrained sheet piles.

Between these extremes, a variety of intermediate forms, termed partly restrained pile systems, is possible. For the following reasons the type of quay wall with a relieving structure that is considered in this handbook is based on retrained sheet piles:
- minimising of the chance of loss of stability caused by inadequate from passive supporting pressure;
- the creation of extra redistribution capacity in very extreme load situations;
- bearing function appears to be often normative for the determination of the length of sheet piles, which goes a long way in the direction of restrained sheet piles;
- the ultimate bending/deflection moment is reduced, the material required for the solution with restrained sheet pile is usually more favourable;
- with the restrained sheet pile solution the horizontal anchor force is reduced.

Only if there are hard layers in the subsoil, which cause risks that are too high for driving the sheet pile members, the choice of a fully restrained sheet pile wall is considered.

Retaining function of the sheet pile wall
When calculating sheet pile walls that also have a bearing function in the quay concept that is under consideration, a favourable working wall friction angle δ, is often assumed for the determination of the horizontal active and passive soil pressures on the sheet piles.
A precondition for this is that the soil friction on the sheet pile wall is downward on the active side and upward on the passive side. With adequate deformation, the active earth pressure takes on the minimum value and the passive earth pressure the maximum value.
When the sheet pile wall settles as a result of high axial loads, the wall friction between the sheet pile and soil mass can change direction causing increased active earth pressure. For axially loaded sheet pile wall it is necessary to ensure that after the settling of the foot of the sheet pile wall that is caused by axial loads adequate wall friction can still be assumed. To illustrate this, based on an equilibrium assumption of the active sliding wedge for various directions of the wall friction angle δ, the effect on the magnitude of the resultant of the active earth pressure Ea is given in Figure 21. In this equilibrium assumption the reaction force of the wall on the active soil wedge is used instead of the resultant of the active earth pressure Ea. The approximation assumes a fixed value for the angle of the sliding plane θ a, despite the dependence of €. It is clear that the resultant of the active earth pressure Ea with a maximum positive value of δ assumes a minimal value and increases when the value of δ decreases.

If the sheet pile is loaded by an axial tensile force, the wall friction is reduced and from, a certain magnitude of the tensile force, the direction of the wall friction will be reversed. The effect of this is that the maximum passive earth pressures are also reduced. Actually, it is more or less out of the question that such a loading situation can occur for the types of quay with relieving structure.

It is recommended that when calculating is made for the sheet pile wall an extra resisting shear force that develops as a result of the axial load on the sheet pile caused by displacement of the sheet pile foot should be taken into consideration. This principle is illustrated by Figure 23. The maximum value of this shear force can amount to the product of the normal force on the foot and the tangent of the angle of friction of the foundation of the sheet pile wall.

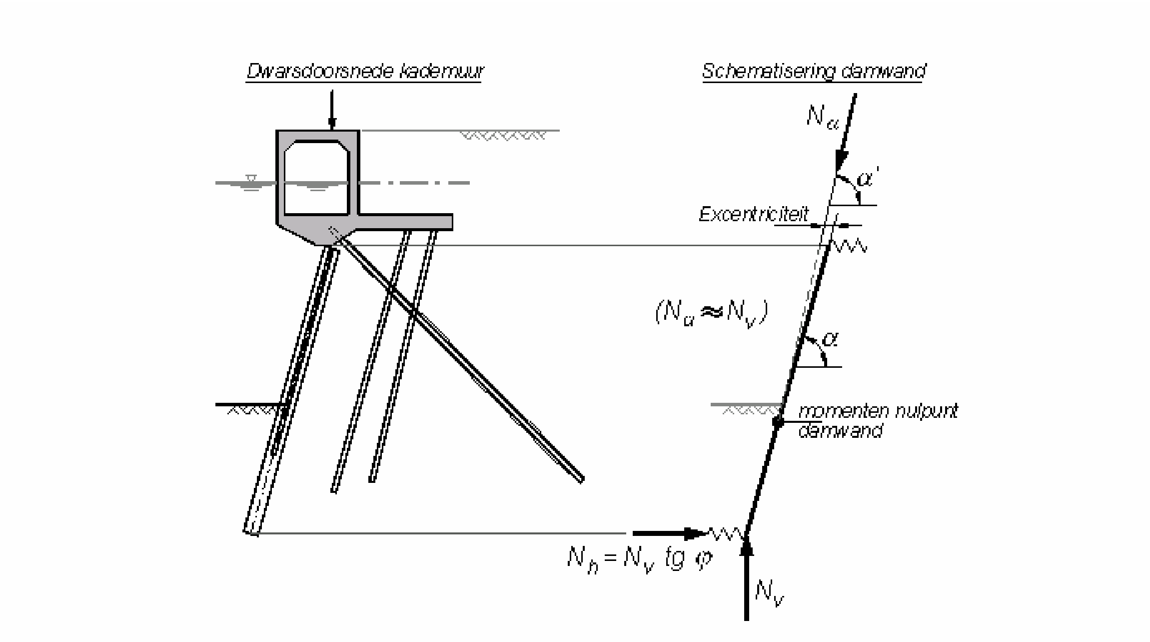


Fig. 23 Extra shear strength at the tip of an axially loaded sheet pile wall.

Sheet pile wall system

For the type of quay wall under consideration, usually a composite steel sheet pile system is chosen. This system consists of heavy main members that are placed at a fixed distance from each other, and light secondary members that close the gaps between the main members of the sheet pile wall and seal it. The most frequently used system is the combi-system composed of tubular piles as the main members with triple sheet piles between them.
The construction height of the main members is determined from a sheet pile wall system based on the principle of the restrained sheet pile or, if that is normative, on the required bearing capacity. The dimensioning of the main members is based on the distribution of forces found in the sheet pile calculations. The sheet piles must be strong enough to transfer the earth and water pressures to the main members. To ensure that the sheet piles can be safely installed in a controlled way, in addition to the strength, stipulations are made in relation to the stiffness and cross section. In principle, it is assumed that the sheet piles will be installed to the required depth by means of vibration combined with jetting.

In Fig. 24 an overview of the main steel sheet pile systems is presented.

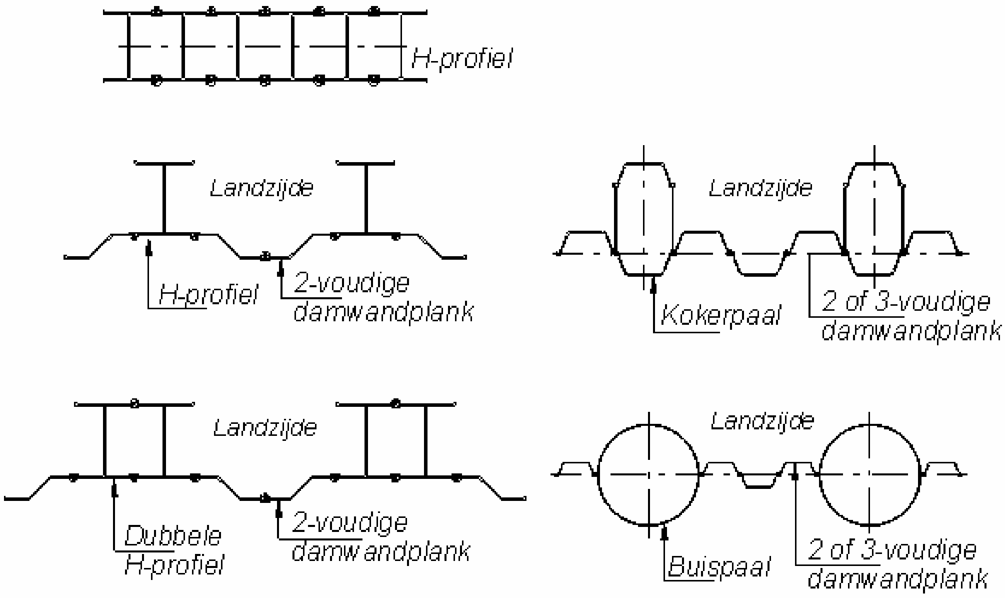


Fig. 24 Sheet pile systems

The determination of the length of the intermediate sheet pile in a composite sheet pile-system is based on the following:

- the level of the underside of the intermediate sheet pile must be at least equal to the load zero point;
- verification of piping and hydraulic soil failure is carried out for two extreme load situations;
- to allow for possible faults during the implementation, a extra margin of 0.5 m is added to the design level of the underside of the sheet pile.

In addition, in association with the TGB 1990, the following management activities should be undertaken in the construction phase and service phase to ensure the reliability of the soil consolidation/compaction of the sheet pile wall:

- intensive monitoring of interlock openings during the implementations, the fitting of interlock sensors and monitoring of interlock opening when dredging to expose the sheet pile wall;
- investigation of increased depth close to the quay structure by taking periodical soundings;
- with great depths: inspection of the exposed area of the sheet pile for interlock opening, repair of interlock opening, supplementation and if necessary the placing of bed protection.

Connection of superstructure and sheet pile system

The connection to the relieving structure can be accomplished in various ways. A fixed moment connection is possible, but requires good detailing of the connection. A disadvantage is that the construction system of the quay wall is much less clear. This is because the distribution of forces is strongly dependent on the deformation of the quay system. It is difficult to calculate this deformation. Moreover, it should be noted that the anchor forces in a fixed moment connection are considerably bigger than those occurring in a hinged connection.

A hinged connection is also one of the options and has big advantages. For the bigger quay walls in Rotterdam the connection is achieved with the aid of a cast iron steel saddle. The saddle creates a hinge between both parts of the quay wall, so a more statically determined construction system is created and the distribution of forces in the sheet pile wall, the foundation and the superstructure becomes more clear. Moreover, placing the saddle on the front flange of the sheet piles creates a favourable exocentric moment on the upper side of the sheet pile. This reduces the field moment and the associated reduction of the deep restraining moment remains restricted.

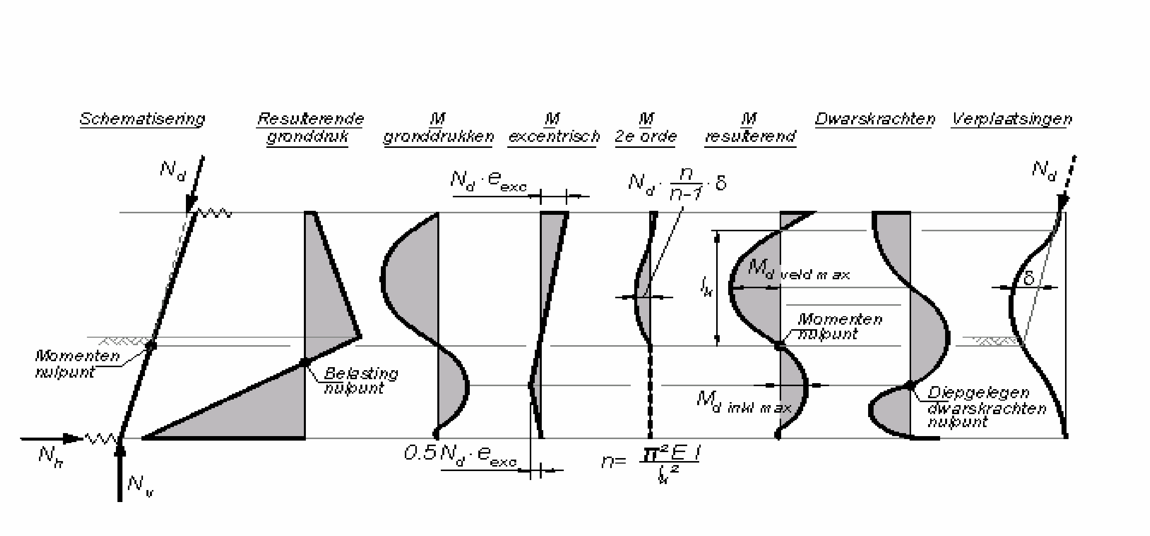


Fig. 25 Effect of inclination of wall and hinge construction on moment distribution.

Length of sheet piles in the sheet pile system

Usually intermediate sheet piles can be kept considerably shorter than the main members. The intermediate sheet pile can be minimally set to the level of the load zero point, the point where the earth pressures of the active and passive sides are equal. A normative condition for the determination of the length of the intermediate sheet pile is that the sheet pile wall must be soil-tight and that in no circumstances internal erosion can occur although in spite of this, it frequently does occur in quay and sheet pile structures. There are various reasons for this, such as intermediate sheet piles that are too short, the occurrence of local increases in the water depth caused by the effects of propeller jets and the presence of interlock openings or combinations of these.

An important mechanism that plays a role in non-cohesive soil layers is internal erosion (piping) of hydraulic soil failure. Parallel with the tidal movement and groundwater level that reacts to this, there is transport of soil to the front of the sheet pile wall. If not noticed, this can eventually lead to the formation of large hollow spaces behind the sheet pile wall and under the relieving platform and ultimately to the local failure of quay members.

Corrosion

Corrosion plays a very important role in the dimensioning of the sheet piles. Corrosion occurs mainly on the water side of the sheet pile. If there is no exchange of oxygen-rich water corrosion is usually negligible on the side in contact with the soil. However, in the position of the gravel coffers/caissons/glabions grindkoffers corrosion is a real threat. For specific information on this reference may be made to Chapter 8. The various types of corrosion, especially in seawater, can lead to considerable corrosion of the steel sheet pile. Local environmental conditions or conditions of use, such as the influence of contaminants and the effect of propeller jets, have an important influence on the rate of corrosion. A dominant factor is the vertical positioning of the sheet pile in relation to the high and low water levels. In the literature 4 to 6 zones with significant different rates of corrosion are distinguished. Depending on the type of corrosion, adverse corrosion rates are found in the splash, tidal and low water zones.

The highest rate of corrosion often occurs in the oxygen-rich area just below the low water zone. In designing quays, the corrosion problem can be severely restricted by choosing a construction level for the steel sheet pile wall with some margin below the water level.

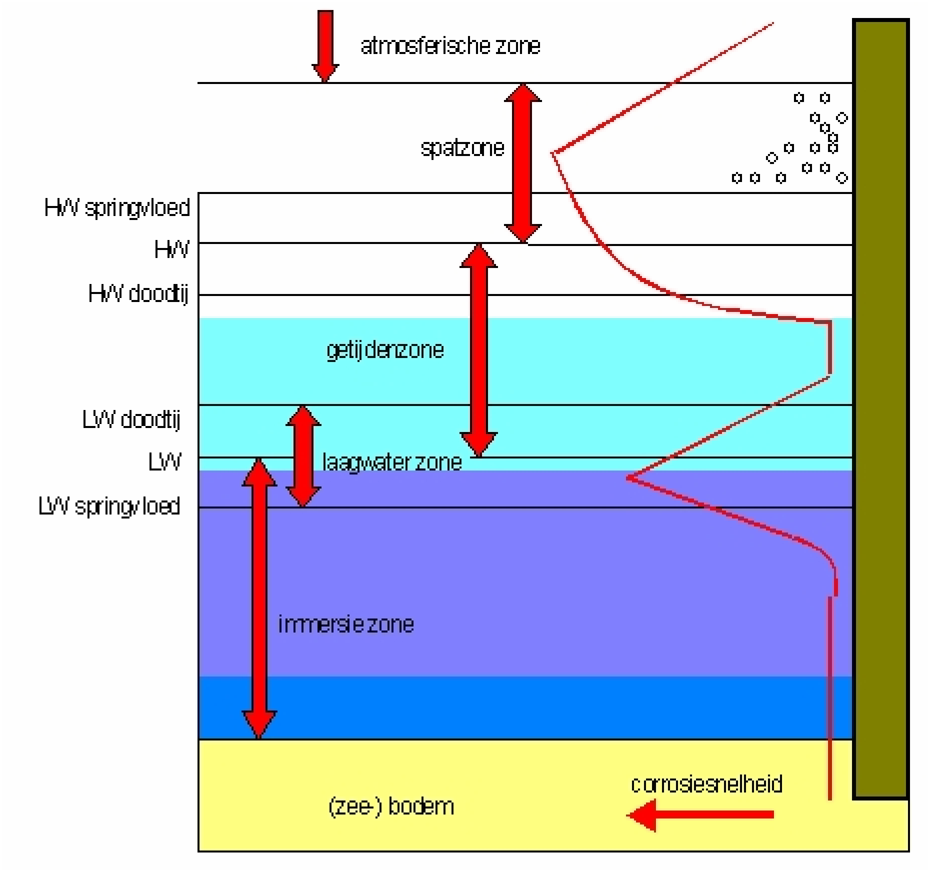


Fig. 26 Corrosion zones.

It is clear that when dimensioning the sheet pile wall great attention must also be paid to the phenomenon of corrosion during the lifetime so that the quay continued to conform with the requirements relating to safety and usability. In this, it is important to distinguish between general and local corrosion. General corrosion relates to average corrosion and reasonable estimates of this rate of corrosion can be made. The data on general corrosion can be used to assess the strength and stiffness requirements for the sheet pile wall and mainly concern the main members. Local corrosion relates to corrosion that may be concentrated in inconvenient positions and in which very high rates of corrosion can occur. It is very difficult to predict the rate of type of corrosion. Corrosion measurements on sheet pile walls in comparable situation may provide important information. The information on local corrosion rates can be used in the assessment of the durability and functionality of the main members and of the intermediate sheet piles. The appearance of gaps in the main members and in the intermediate sheet piles, resulting in the density of the soil being at issue is unacceptable.

Based on data relating to local and general corrosion rates and the chosen lifetime, in consultation with advising the client, a choice must be made between possible measures to protect against corrosion, including:

- the use of a corrosion supplement, extra thickness of the steel;
- the application of a coating;
- the use of active or passive cathodic protection;
- combinations of methods.

It should be noted that cathodic protection only works for parts of the sheet pile wall that are under water, which are then fully protected. In the tidal-zone, cathodic protection works only for part of the time and often gives inadequate protection against corrosion. In which case a coating can be applied in combination with a corrosion supplement. In unfavourable conditions with such a combination, the lifetime is often limited to 25 years. Other combinations can also be used, such as a corrosion supplement in combination with equipment for cathodic protection that can be activated later. In order to monitor the actual corrosion process it is advisable to take supplementary management measures in the form of periodic inspections and measurements.

5.2.5 Design aspects foundation system

Functions

The foundation of a quay wall must ensure both the horizontal and the vertical stability of the quay wall. Naturally, this applies throughout the entire lifetime of the quay structure. Sustained effects, such as the effects of cyclic loads, incidental loads or overloading may not lead to unsafe situations or unacceptable behaviour.

In principle, the foundation system is formed by a bearing sheet pile wall, in combination with a system of tension and bearing piles. The most suitable solutions are:

- a system with an inclined sheet pile wall and inclined prefabricated concrete tension and bearing piles. Such a system is only suitable for lower retaining heights;
- a system with inclined sheet pile wall, MV-tension piles, at an angle of ca 45° beside the sheet pile wall and inclined prefabricated concrete bearing piles. Usually the MV-pile is positioned close to that of the sheet pile wall. The vertical component of the tensile force in the MV-pile is taken up by the main members of the sheet pile system.

It is necessary to prevent the foundation design from turning into such a dense pile field that it becomes difficult to drive the piles and that the entire structure starts to behave like an extra sheet pile wall screen so that the relieving working of the structure becomes an issue. In such a case, consideration can be given to using heavy piles that are cast in situ instead of prefabricated concrete piles. The inclinations are then also limited.

In unusual situations, a system with inclined sheet pile wall, horizontal anchoring and inclined prefabricated concrete bearing piles can be used. Such a solution can be interesting if the use of tension piles present a problem in relation to technique or cost. A few options are shown in Fig.

Loads

The foundation system is loaded by the resulting horizontal and vertical loads.

Horizontal:

- horizontal loads caused by earth and water pressure on the superstructure, and the horizontal loads caused by the use of the superstructure;
- anchor forces from the sheet pile system;
- supplement to anchor force if the relieving working of the relieving platform is restricted by a cohesive layer;
- extra calculated horizontal loads resulting from screening effects of foundation members in the area between the sheet pile and the rear of the relieving platform.

Vertical:

- vertical loads from the superstructure door the self-weight and the use of the quay wall;
- earth on relieving structure, including soil friction forces;
- vertical component from the anchoring.

Foundation members

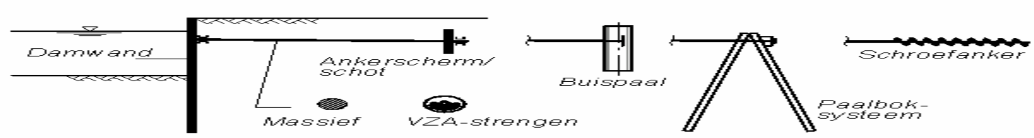
When designing the foundation members it is important to take into account a number of matters that have led to problems in the past.

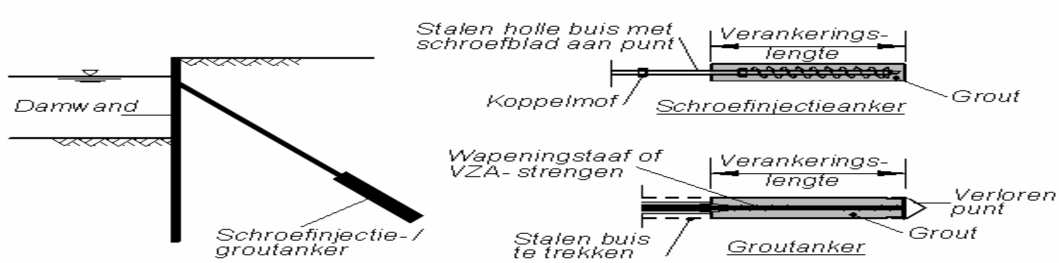
- Deformation of the sheet piles on the active side of the sheet pile wall can lead to relaxation of the strong bearing force layers. In the design of the foundation system, the feet of the bearing piles must be at sufficient depth and a safe distance from this area. Moreover, the bearing piles must be placed in such a way that the area expected to contribute to the bearing capacity of the bearing pile via positive friction remains outside the active sliding area.
- When inclined tension piles are used above a compressible soil layer it is necessary to investigate whether settling of the compressible layer under the tension piles can be expected as a result of high loads on the area. With settling, the part of the maximum possible negative friction that exceeds the pile tensile force that is supplied as external loads on the pile trestle system is shed. This can lead to overloading of the bearing piles and to settlement. In such a case, to prevent this effect the foundation of the tension piles should lie in a deeper non-compressible layer. The extra negative friction that occurs can then be transmitted directly to the subsoil.

5.2.6 Design aspects of anchorages

A discussion of some general design aspects that influence the design of an anchorage follows. Two types of quay in which the stability is provided by means of an anchorage system are distinguished:

- single anchored sheet pile
- quay walls with relieving structure and anchoring.





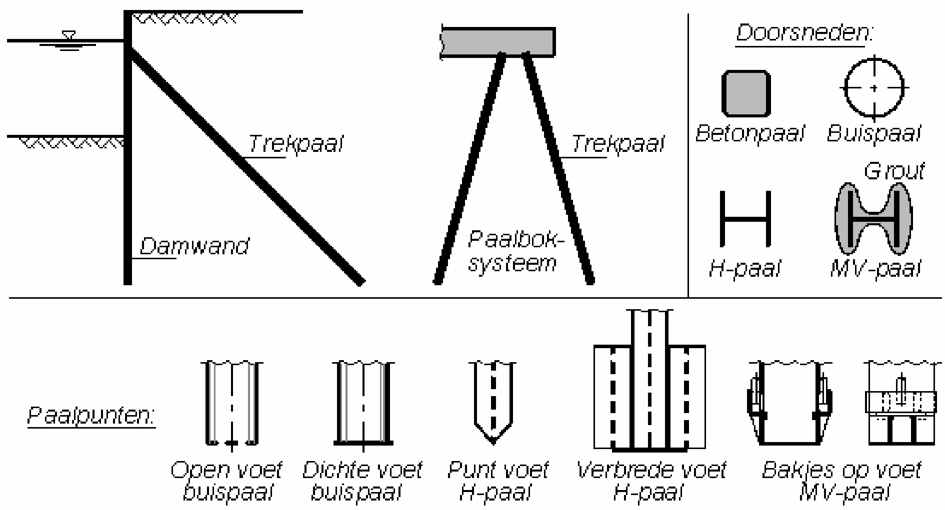


Fig. 27 Various anchor systems

Single anchored sheet pile wall

With this type of quay wall, the use of vertical sheet piles is assumed. The stability is provided by an anchorage. The choice of the anchoring level is determined by the economy of the design. The optimum distribution of moment of the sheet pile wall is chosen so that the cost of the material for the sheet pile wall, which governs the choice, is minimised. The costs of implementation also play a role in this, for example, when an anchoring level is chosen this means that drainage must be used. The members of the anchorage are dimensioned in accordance with the calculation of the anchor force that derives from the calculation of the sheet pile wall. The anchorage consists of horizontal anchors in the form of bars or cable and an anchor plate.

The anchoring can also be achieved by means of members that are placed at an angle and are anchored in bearing sand layer, such as prefabricated concrete piles, smooth steel piles, MV-piles, grout anchors and auger injection anchors. The chosen angle is circa 45°. In special conditions it may be necessary to place the anchor members at a steeper angle, for example owing to lack of space or because other objects are in the way. For relatively high sand layers it is also possible to place screw injection anchors or grout anchors at an angle of 20 to 30 degrees. In such a concept, the horizontal components of the tensile force of the anchors are taken up to be the sheet pile wall.

Quay wall with relieving structure and an anchorage

Under some conditions, the use of a foundation system that consists of a bearing sheet pile wall and a pile trestle system composed of tension and bearing piles. In this case, instead of tension piles, horizontal anchoring is used. The level of the anchorage is largely determined by the geometry of the superstructure of the quay. The anchoring is dimensioned based on horizontal loads working on the quay system reduced by the part that is taken up by the inclined sheet pile and the inclined bearing piles.

Anchorage
The dimensions of the anchorage (anchor plate, anchor and anchor length) are largely determined by the safety in relation to the stability. When determining the safety both high and deeply positioned sliding planes are taken into consideration. It is also necessary to consider deformation of the soil mass before the anchor screen starts to work. It is recommended that in view of the usually stiff structure of a quay wall, the anchorage should be pre-tensioned. With high quality steel anchors, the need for pre-tensioning is still greater because of the stretching of the anchors that takes place when loaded. From experience, it appears that because of the relaxation of the soil, the pre-tensioning process must be carried out in phases in order to get an even distribution of the pre-tensioning in the anchors. Phased implementation is also often needed to prevent overloading of the quay structure and quay members, horizontal beam and anchor plate.

5.2.7 Deformation behaviour and the deformation of quay structures

Requirements
Often operators set special requirements in relation to the deformation of a quay wall because from experience they know that with a safe design, there will remain within acceptable limits and the functional uses will not be negatively influenced. The deformation of the sheet pile wall as part of a quay structure with relieving platform, also appears to have no influence on the behaviour of the quay wall and is not of interest to the operator. An exception to this is formed by tolerances that the transhipment companies and crane suppliers set on the position of the crane tracks. To obtain an impression of this *see NEN 2019 Cranes, the metal framework*. In this standard, the tolerances of the supporting structure are defined. In addition, the transhipment companies and the crane suppliers can agree to different tolerances. The tolerances concern the horizontal and vertical positioning of the rail profile. The order of magnitude is often expressed in millimetres. The most important tolerances relating to the design of crane beams concern:
- rail gauge
- height
- slope.

For simple quay structures, consisting of a single anchored sheet piles, as shown in figure, the water side and land side crane tracks are not integrated with the sheet pile structure and have separate foundations. Depending on the bearing capacity of the subsoil, the foundation of the crane beams is a shallow footing or a piled foundation. When the water side crane beam is situated close to or within the active sliding surface of the sheet pile, even if the subsoil is good bearing ground a pile foundation may be chosen. For quay walls with a relieving structure the water side crane beam is usually integrated in the superstructure. Because of the relatively wide gauge of modern harbour crane rails, the land side crane beam often has a separate foundation. The quality of the subsoil determines the type of foundation.

Deformation behaviour of quay structures
The deformation behaviour of quay structures is largely determined by the retaining height and the quality of the subsoil. Furthermore, the structure and more especially the stiffness of the pile trestle system of the anchoring structure determine its behaviour. For each type of design, a simple calculation can be used to determine the horizontal spring stiffness. From a comparison of the results, it appears that the stiffest behaviour is found in pile trestles with steep inclinations.

Thus, the horizontal stiffness of the pile trestle consisting of the inclined sheet piles and an MV-pile at a 45 degree slope is considerably greater than that of a pile trestle with inclined piles at a gradient of 3:1. A solution with a horizontal anchor built up from pre-tensioned cables and an anchor plate often appears less stiff. If stringent stipulations are made with regard to the tolerance, the choice of a structure with horizontal stiffness seems obvious. It should be noted that no reliable deformation calculations could be based on horizontal spring stiffness. The best approximation is found with calculations using the finite elements method, for example with the PLAXIS program. Owing to deviations in the schematisation and the modelling and the many assumptions about the soil properties, the margin of accuracy of the calculations with PLAXIS is in the order of ± 30%.

Deformation caused by dredging free

These are in the order of centimetres and usually exceed the tolerances set for crane tracks. The deformation caused by dredging free is determined by two mechanisms:

- reduction of vertical effective stress in the soil layer in front of the quay wall;
- the initiation of the retaining function of the quay wall.

The first mechanism involves the elastic rising of the bed of the harbour causing the sheet pile wall to rise and the rotation of the quay, the front of the quay rising higher than the rear. The magnitude of the effect is determined by the extent of the reduction of the grain stress. The design of the quay wall has scarcely any influence on this. The biggest deformation is found with the highest retaining heights and when construction takes place on a dray site. The effect of the second mechanism, the occurrence of horizontal deformation, is primarily dependent on the retaining height, but can be influenced by choosing a quay structure with stiff deformation behaviour. The conclusion can be drawn that the deformations caused by the dredging free of the quay that mainly work through onto the water side crane beam, exceed the tolerances. It is advisable to install the crane rails only after dredging free has been completed.

Deformation caused by use

From deformation measurements on quay walls with a relieving structure, it appears that some time after the dredging free extra horizontal deformations arise in the direction of the harbour basin. Depending on the deformation behaviour of the quay structure this varies from a minimum of a few centimetres to sometimes more than ten centimetres. The time dependent behaviour of cohesive soil layers, such as the creep, relaxation and setting behaviour plays a role in this. Under some conditions, the deformations can rise to several decimetres. In addition effects such as wedge formation caused by dynamic or cyclic loads, the appearance of much higher loads than anticipated in the Programme of Requirements because of injudicious use or by emergency storage. It must be remembered that a quay structure that is exposed to an unfavourable load situation will not fully return to its original state.

This is plausible because the deformation does not give rise to any open spaces behind the quay wall and the earth retains contact with it. During the lifetime of the quay structure this gives rise to the development of a situation in which, because of previous unfavourable load situations, the superstructure is more or less attached to the soil.

5.3 Design models and calculation methods

5.3.1 General

In this section only the specific mathematical models and calculation methods that are used in the design of quay structures or quay members are included. The following subjects are considered:

- sheet pile calculations;
- the calculation of ground and structural models with the finite element method;
- the calculation and verification of the stability of the quay system and quay members;
- the calculation of the bearing capacity of foundation members.

The first two subjects are treated in detail in the CUR Report 166. Here only the broad lines and principles are considered. The calculation and verification of the stability problems occurring with quay walls are considered in detail. For the calculation methods to determine the bearing capacity of foundation members, the available regulations and the results of some recently published CUR reports are used. For each subject attention is paid to issues specific to quay structures. Familiarity with structural mathematical models is assumed and not explained here.

5.3.2 Sheet pile calculations general

For the calculation and dimensioning of the sheet piles in a quay structure, because of their simplicity and user-friendly nature two calculation methods are considered suitable. These are the standard calculation methods of Blum and the third method of calculation. It is based on a model in which the properties of both the ground and the structure are introduced. With this method of calculation, stresses and deformations of the ground and the structural members can be calculated in a fundamental way. The method is primarily used in those cases where reliable estimates of deformations are required. The method is rather complicated and is less suitable for the dimensioning of the sheet piles and the quay wall.

Calculation of sheet pile according to the Blum method

This method assumes a failure situation in the ground in which the deformations are so large that maximum shear stresses can develop. This means with the Blum method, calculations made by using minimum active and maximum passive earth pressures. The magnitudes of the earth pressures are therefore fixed and the sheet pile calculation can be carried out as a simple beam calculation. With this method, various types of sheet pile structures can be calculated including unanchored, single and multiple anchored, freely supported and retrained and restrained sheet pile.

Because of its simplicity, the Blum design method is still frequently used in the development of draft designs. This involves the first investigation of the minimum length of the sheet piles, the pile length at which restraining is achieved and the determination of the moments and anchor forces. In practice both graphical and approximation methods of approach are available. The graphical method provides good insight into the working of the sheet pile, but it is not practical because calculations must be carried out manually. In practice, only computer calculations based on an analytical method are used.

A disadvantage of the Blum method is that the actual earth pressures on site may differ considerably from the minimum active and maximum passive values. This is the case, for example, in the embedding area of restrained sheet piles, where the deformations are too small for the development of maximum passive ground resistances.

The result is that with this method the retraining moments found are too large and the site moments are too small. Moreover, this results in anchor forces that are rather low. The Blum method cannot be used for the calculation of quay walls with very high flexural rigidity/bending stiffness, such as those of diaphragm walls. The deformations that occur usually remain so limited that no minimum active and maximum passive earth pressures can develop.

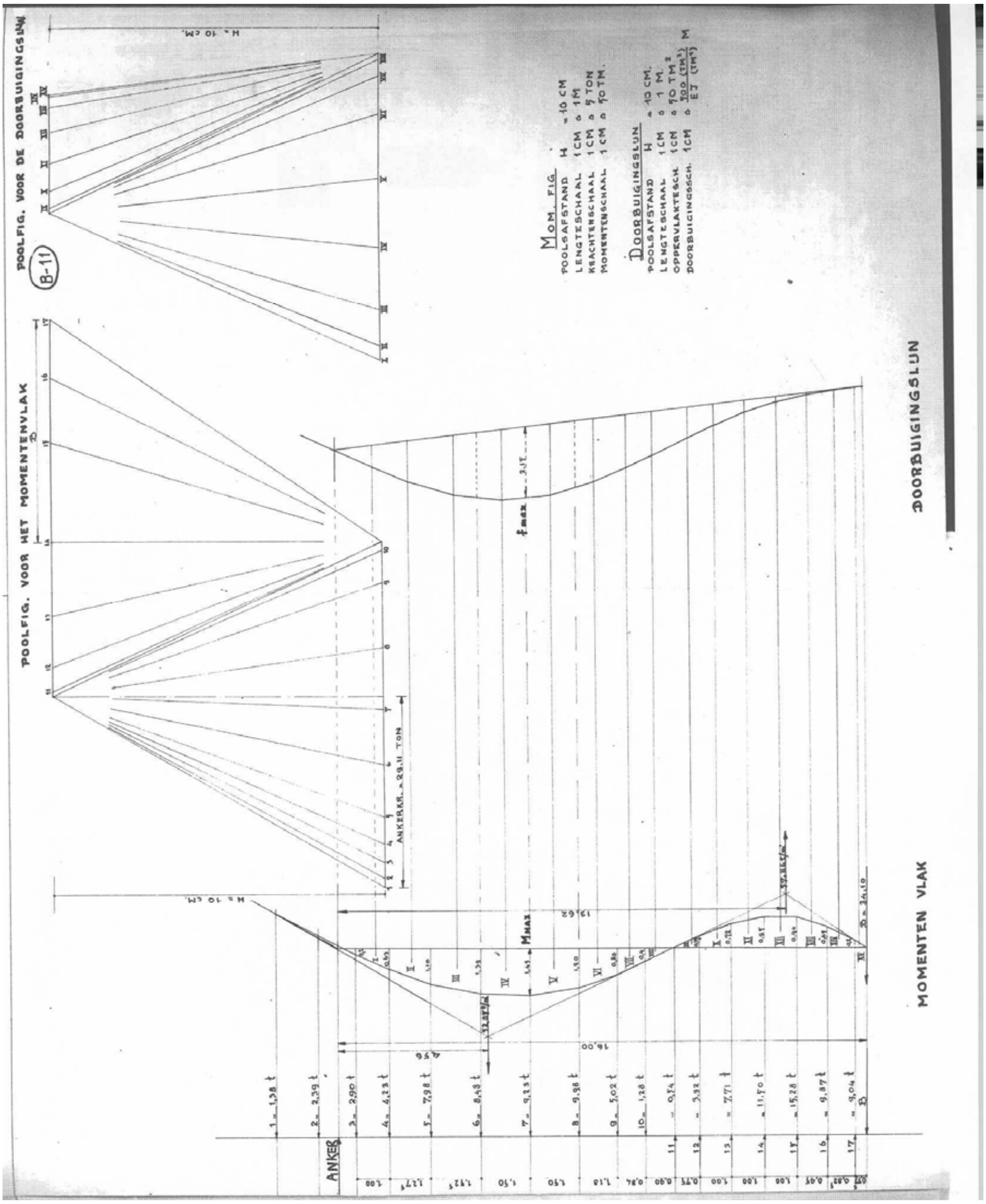
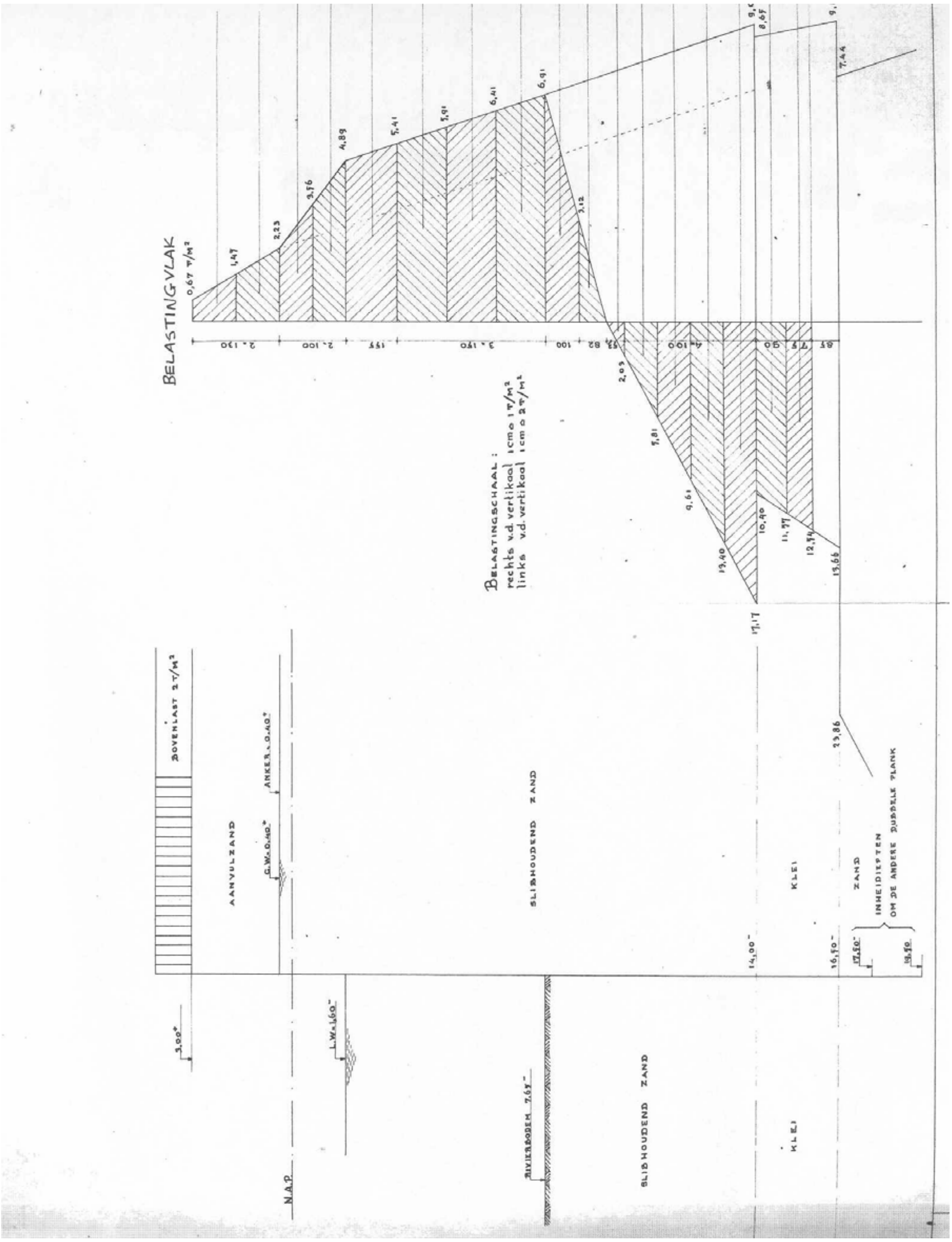


Fig. 28 Example of Blum calculation


BELASTINGVLAK
Belastingschaal :
rechts v.d. vertikaal 1cm ≙ 1T/M²
links v.d. vertikaal 1cm ≙ 2T/M²
BOVENLAST 2T/M²
AANVULZAND
SLIBHOUDEND ZAND
KLEI
ZAND
INHEIDIEPTEN OM DE ANDERE DUBBELE PLANK
N.A.P.

Calculation method for spring supported beam

In this design model the ground is schematised as a set of elasto-plastic springs. Only with adequate deformation of the sheet pile the plastic branch of the ground spring is reached and do minimum active earth pressures or maximum passive ground resistance develop. If there is no displacement the earth pressure is neutral. Because the earth pressures depend on the deformation of the sheet pile wall the calculation follows an iterative process. After each calculation step a verification of whether the calculated earth pressures correspond with the displacements is made. The calculation process ends when the results have converged. The available computer programs are based on uncoupled springs. This means that the effect of arch working of the ground, which causes an important reduction of the field moment in non-cohesive ground is not taken into account.

For the principle of the determination of the calculation parameters, see Section 6.6. Because of the high distribution capacity of the superstructure in the types quay walls with relieving structure, the following calculation parameters of the average values are used in this handbook:

- coefficients of sub-grade reaction kh of the ground;
- stiffness parameters of the sheet pile;
- spring stiffness of the stabilising pile system of the anchorage.

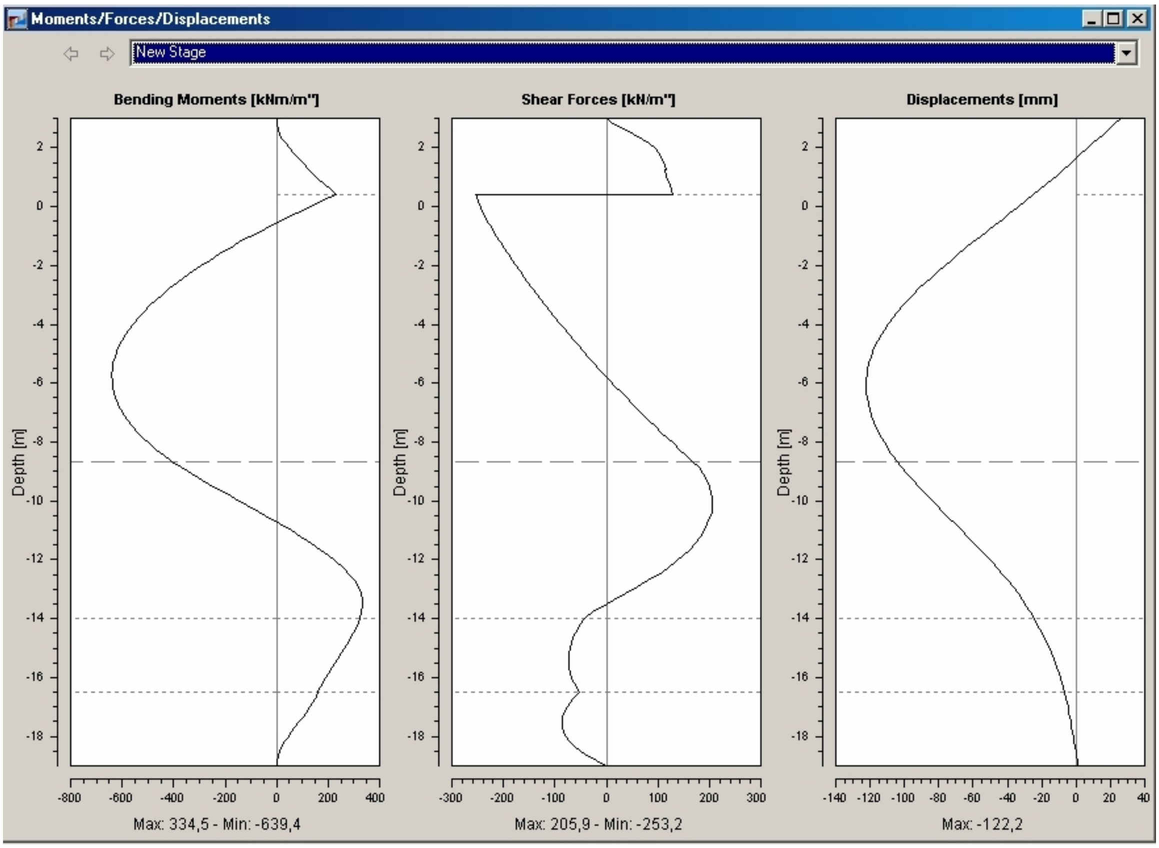
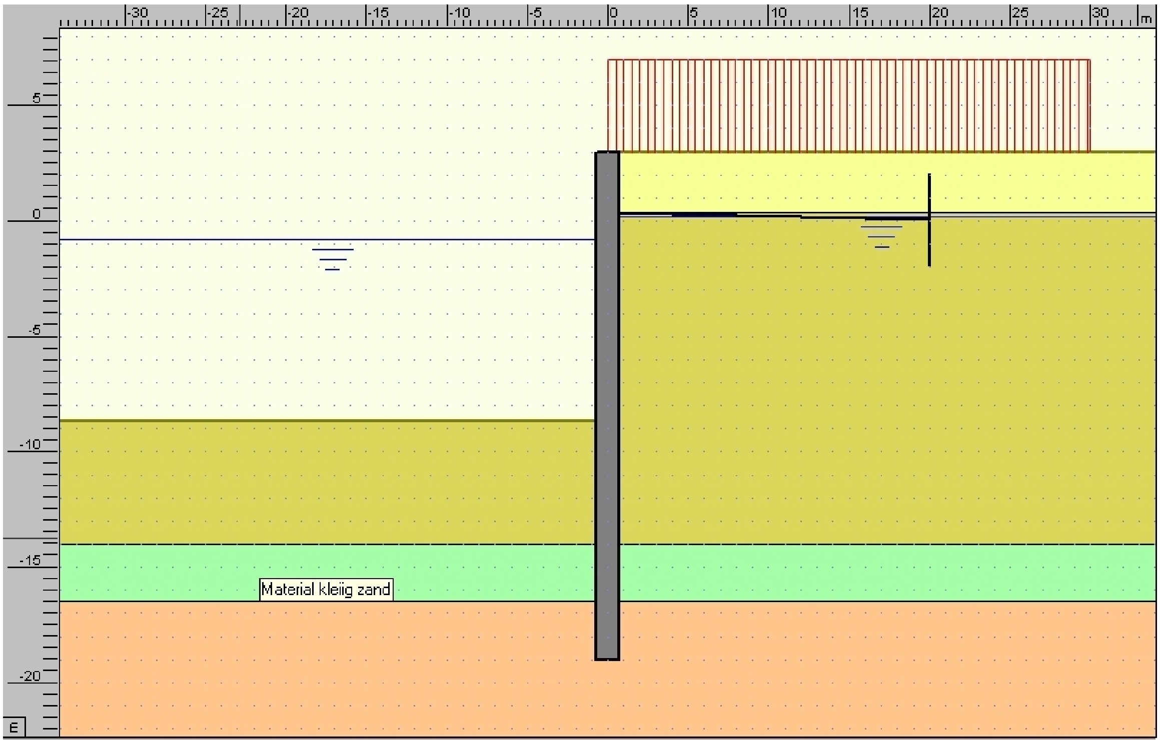


Fig. 29 Example of Spring supported beam calculation.



With this calculation method a sequence of phases in which the stress history of the sheet pile is carried to the following phase can be calculated. Changes in the spring characteristics of the ground caused by fill or excavation operations can also be entered. In the same way, the pre-stressing of the anchors can be included in the phasing. The reliability of phasing calculations must be greeted with some scepticism because the schematising of the behaviour of the ground is far from being perfect. It has been found that when an unfavourable load situation occurs during an earlier phase the distribution of forces in the sheet pile after relieving decreases much less than might be anticipated. Thus, the unfavourable moments that are developed in the sheet piles with relatively high tension of the anchoring continue to dominate in subsequent phases. This does not agree with the reality, in which the pre-tensioning and the effect of this on the sheet pile seems to decrease overtime as a result of creep. This can be explained by the fact that the effect of the pre-stressing on the sheet pile is determined not only by the behaviours of the ground in the area of influence close to the sheet pile, but also the soil mass between the sheet piles and the ground mass between the sheet piles and anchorage. One must ask oneself whether this phased calculation approach is suitable for the assessment of safety in the ultimate limit state.

In the calculation of the sheet pile as part of a quay structure the following aspects are considered:

- effect of the inclination on the active and passive earth pressures;
- effect of the axial loading by the superstructure on the distribution of forces on the sheet pile;
- effect of an eccentrically placed saddle on the upper side of the sheet pile on the distribution of forces;
- effect of the transfer of the axial loading of the foot of the sheet pile on the sheet pile system.

Most programs do not allow for the entry of data on inclined sheet piles. The effects of the inclination on the horizontal earth pressures are therefore taken into account by using adjusted earth pressure coefficients. The programs do not usually cater for axial loads either.

The effect on the distribution of forces of the sheet pile, such as the second order effect, must be calculated separately. The eccentricity moment that acts as a result of the eccentric position of the saddles on the top of the sheet piles can be entered as an external load into most programs. For the principle of the structure of the distribution of moments in the sheet piles. For the calculation of the sheet pile it is recommended that account should be taken of an extra resistance-providing shear force that arises as a result of the axial loads on the sheet piles with displacement of the feet of the sheet piles, see Figure 23. The maximum value of this shear force can rise up to the product of the normal force on the foot and the tangent of the angle of friction of the foundation of the sheet pile.

5.3.3 Finite element method

The finite element method is based on a model in which the behavior of the ground and the structure are integrated. The properties of the ground are introduced by means of stress deformation relations. With this method fundamental calculations of stresses and deformation of earth and structural members can be made. The method can be used to verify the global stability of the quay wall and to verify deformations. The finite element method can also be used to analyse other more fundamental problems that play a role in the design of quay walls such as:

- horizontal deformations in the position of the foundation members, to determine bending moments in the piles and extra horizontal loads on the superstructure;
- deformation of the superstructure in various phases, the results of which are used to verify the deformation of the crane track;
- vertical arch working of the ground on the active side of the sheet pile, the result: reduced moments and a higher anchor force can be taken into account in the dimensioning of the sheet pile and the quay wall;
- verification of the relieving work of the superstructure in the presence of a weak cohesive soil layer.

The finite element method can also be used for three-dimensional problems, for example for the investigation of the distributions of the earth pressures over the main members and intermediate piles in a combined sheet pile system. The software programs that can be used to analyze and calculate geotechnical structures include: PLAXIS (of the PLAXIS Foundation) and DIANA (of TNO).

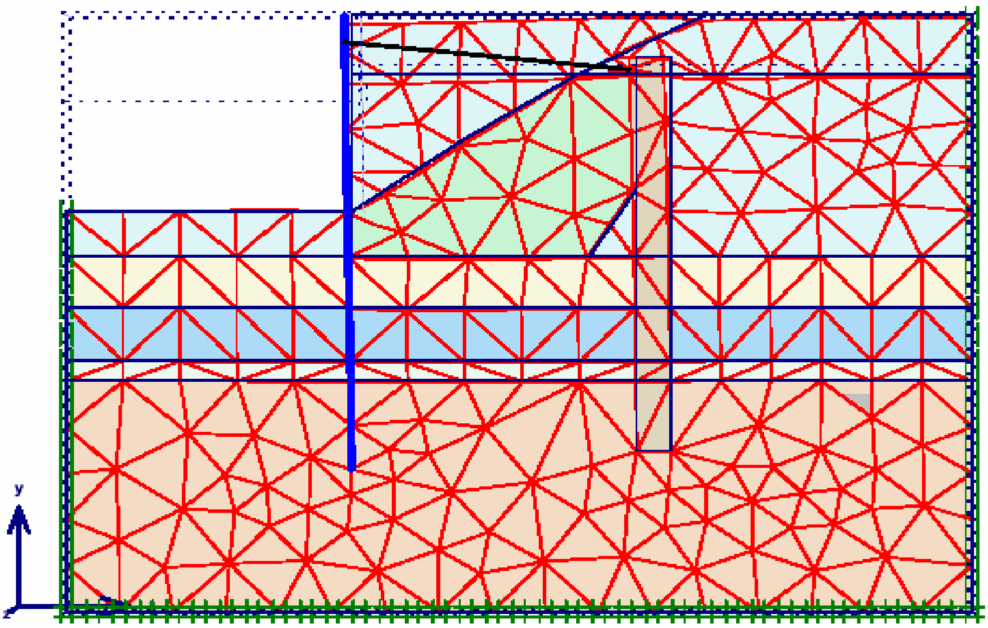


Fig. 30 Example of PLAXIS analysis

5.3.4 Verification of stability analyses

The verification calculations of the stability of the quay wall include possible unfavourable and deep sliding planes. The verification assesses whether the quay wall under consideration is sufficiently safe against a normative rotational sliding plane. The available verification methods are based on arcuate/circular or multiple angled sliding planes.

Method Bishop

A much used calculation method is that of Bishop, which is based on a circular shape. The specification for stability is that the design value of the driving moment M_{ad} is smaller or equal to the design value of the resisting moment M_{rd}. For the determination of these moments the ground mass is divided into a number of vertical lamellae.
Summation of the moments round the midpoint of the rotational sliding plane caused by the driving or resisting resultants per lamella yields the driving or resisting moments.

Two situations can be considered:

- a situation with undrained soil properties in the cohesive layers. For this the shear strength of design value *fundr is taken*. The angle of friction f_{undr} is set at zero;
- a situation with drained soil properties and that is possibly taking into account excess pore pressures in the cohesive ground layers caused by incidentally occurring high site loads.

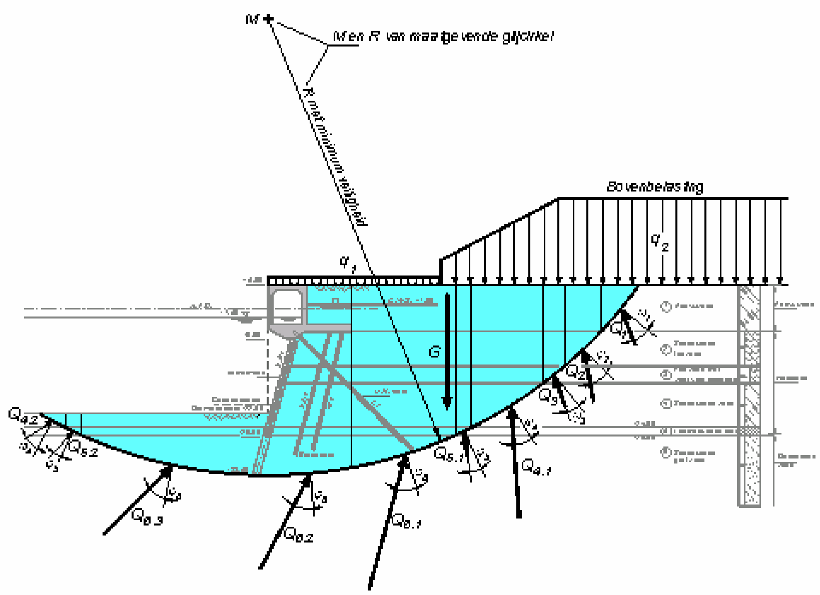


Fig. 31 Verification of total stability of a quay wall according to Bishop.

The vertical site loads and the weights of the ground to the right of the midpoint of the assumed sliding circle form the driving forces. The resistance forces Q and the weights of the ground left of the midpoint of the assumed sliding circle are stabilizing.
The verification of the total stability is calculated for the ultimate limit state 1A. For the calculation of quay walls the partial factors for the soil properties are taken as 1.0 and representative values are used to calculate the resistances forces and loads. The relation between resistance-providing forces and driving forces for the limit state 1A must be a minimum rate of 1.3.

Method Kranz

With the Kranz method, the stability of the anchorage is verified on the basis of a deep straight sliding plane. When tension piles are used, this sliding plane runs from the deeply positioned lateral force zero point of the sheet piles to the centre of the anchorage area of the tension member. If an anchorage with an anchor plate is used, the sliding plane of the deeply positioned lateral force zero point of the sheet pile runs to the underside of the anchor plate. The ground mass that lies between the sheet piles and the vertical through the centre of the anchorage area is acted on by the design value of the tensile force of the tension member of the anchorage must be in equilibrium. For this the resistance-providing shear force is based on the representative values of the soil properties.

The equilibrium is verified for a width of 1 m. Here too, as for the verification of the total stability of the quay wall, both undrained and drained situations are considered.

The representative value of the anchor capacity *Fkr;rep*, from the assumption of the equilibrium of the ground segment, must satisfy:

Fkr;rep $\geq$ 1.50 *Fa;max;gr;d;* in which:

Fkr;rep	the representative value of the anchor capacity per m, determined by using the representative values of the soil properties;
Fa;max;gr;d	the design value of the tensile force per m' of the tension member of the anchor.

In Figure on the basis of the basic assumptions and the assumption in the Figures the verification of the stability of the tension pile according to the Kranz method is explained. Two situations are analysed:

- with short MV-pile (I);
- with long MV-pile, (II).

By comparing the two situations, the big influence of the pile length on the design value of the anchor capacity becomes clear.

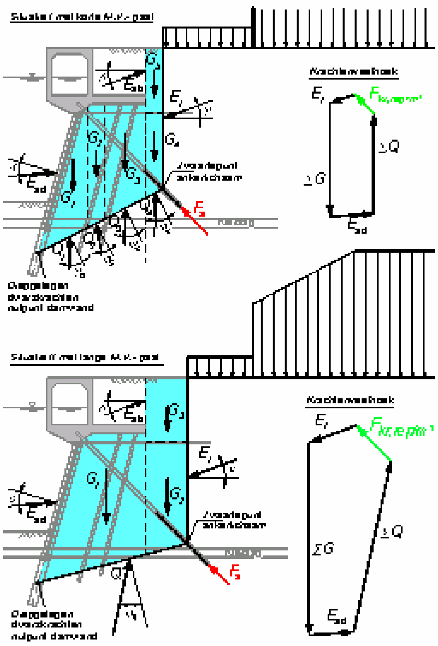


Fig. 32 Example of the verification of the stability of the anchorage according to the Krantz method.

If a considerable groundwater flow accurse in the ground segment under consideration, to determine the anchor capacity *Fkr;rep*, it is necessary to take into account unfavourable excess pore pressure. This is taken into account from the lateral zero point to the level of the groundwater, see Figure 33.

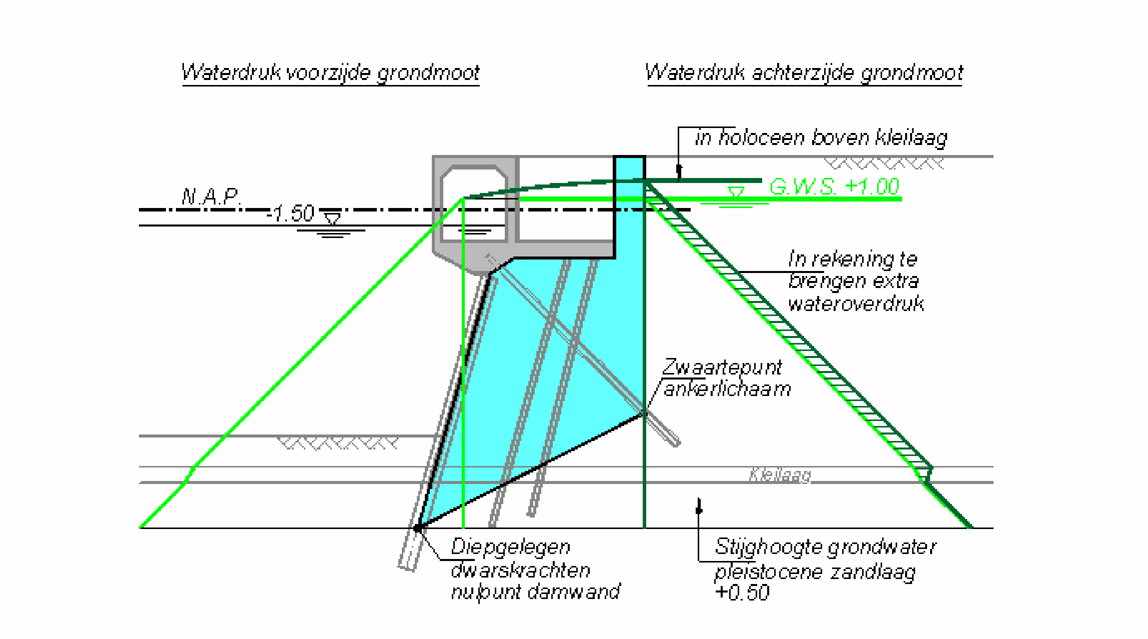


Fig. 33 Water pressure difference in a segment of soil by verification of the Kranz stability.

Verification of heave

In tidal areas in which, relative to the depth of the bottom of the harbour, where is a shallow lying semi-permeable ground layer, there is a danger that as a result of the occurrence of excess pore pressure during dredging of the harbour or the dredging free of the quay wall will heave. Heave greatly reduces the capacity of resistance-providing ground layer on the passive side of the sheet piles, which is undesirable. Consequently, it is necessary to verify whether there is adequate protection against this phenomenon. Verification should be carried out during the construction phase to determine whether there is excess pore pressure that has not adapted, which could lead to heave in the layer beneath the semi-permeable layer. A. normative situation arises with a low outside water level immediately after the dredging free of the quay. In addition, the situation in the service phase should be verified. An extreme situation occurs if, after a period with relatively high water levels during which the layer under the semi-permeable ground layer has adapted to the high water levels, an extremely low outside water level occurs.

The verification consists of an inspection of the vertical equilibrium of the semi-permeable ground layer in which the vertical forces that act on the underside of the semi-permeable layer are measured. These are the upward directed design value of the vertical excess pore pressure Wd and downward directed design value of the effective weight Gd of the upper layers.

The following equilibrium values must be verified:
Wd $\leq$ *Gd;*

In which *Wd* = ϒ *fW Wrep* and *Gd* = ϒ *fG Grep*

in which:

Wd	design value of the vertical upward directed excess pore pressure
Gd	design value of the effective weight of the upper layers
Wrep	representative value of the excess pore pressure
Grep	representative value of the effective weight of the upper layers

The representative values of the excess pore pressure must be determined on the basis of hydrological observations during the construction phase and during the service phase. Software that can be used to calculate level steady state groundwater flows in permeable layers in a ground mass is available in the market. For the partial loading factors of the excess pore pressure $_{fw}$ and the effective self-weight fG the following values are taken:

- construction phase: ϒ fw = 1.30 and ϒ fG = 0.90;
- service phase: ϒ fw = 1.50 and ϒ fg = 0.90.

For example, the excess pore pressure situation in the service phase can be verified. This is shown in Figure 6.30 and is in agreement with what is shown in Figure 6.24. This is based on the assumption/beschouwing of the vertical equilibrium of the underside of the clay layer to the level of NAP -25.50 m.

The verification of heave in the service situation is based on:

- sand layer 2.0 m thick; ϒ wet = 20 kN/m^3;
- clay layer 1.5 m thick; ϒ wet = 16 kN/m^3
- representative value of the excess pore pressure based on the representative excess pore pressure from the assumption of Figure 6.24 of 1.5 m.

Verification of heave:
Wd $\leq$ Gd

Wd = ϒ fw Wrep = 1.50   15 = 22.50 kN/m²

Gd = ϒ fG Gd = 0.90 (2   10 + 1.5   6) = 26.10 kN/m²
This consequently also satisfies the safety requirement in relation to heave.

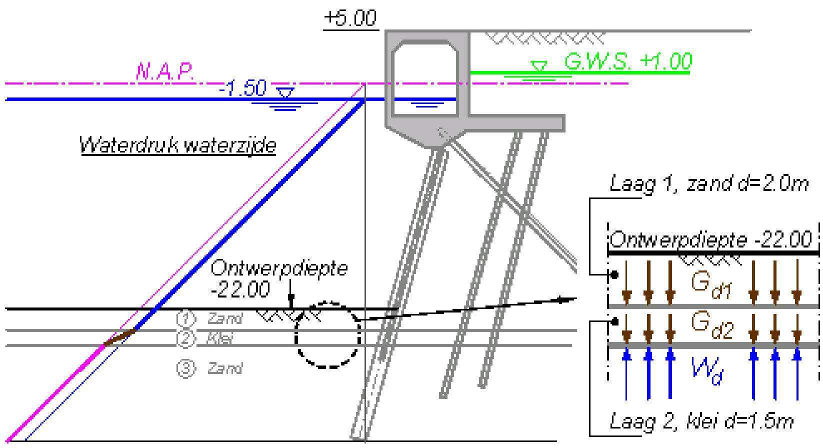


Fig. 34 Example of verification of heave.

Principle and verification of piping

In fine-grained permeable soils it is possible that an upward directed ground water flow, caused by a difference between outside and phreatic levels, may initiate internal scour on the passive side of the sheet piles. A condition for this is that there is a free water level on the high side of the retaining structure. For the type of quay wall under consideration it is assumed that this condition is satisfied because settlement of the ground under the relieving platform causes the formation of a continuous longitudinal split.

The phenomenon is found in quay walls in which, in consequence of tidal activity, unfavourable excess pore pressure situations arise. Internal scour occurs when the upward flow pressures are greater than the vertical grain pressures from the self-weight in the passive area between the harbour bottom and the underside of the sheet piles; in other words, when the critical flow velocity is exceeded. The result is that soil particles are washed out of the passive ground layers close to the bottom of the harbour. This scour process is retrogressive in character, so beginning on the bed of the harbour small channels are formed that extend via the underside of the sheet piles to create links with the open water.

Method Terzaghi

The method of Terzaghi is used to determine the vertical equilibrium of a soil segment ABCD. For this the friction forces on the vertical planes AC and BD are neglected. The flow pressure of the groundwater on the plane CD can be determined form model calculations of the groundwater flows on the basis of the Darcy's law: v = k i, in which:

v velocity of the groundwater flow
k permeability coefficient
i hydraulic gradient.

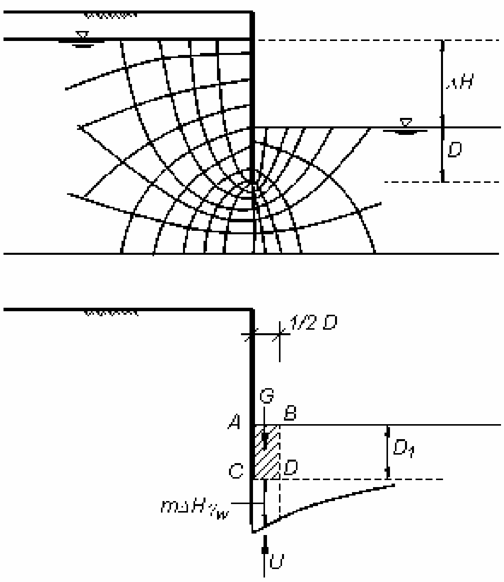


Fig. 35 Calculation of piping according to Terzaghi.

The model is based on the design values of the calculation parameters. The calculation is made with the aid of a rectilinear network consisting of flow lines and equipotential lines. Software is also available with which two-dimensional plane steady state ground water flows can be calculated.

The upward flow pressure found from the calculation is: m ΔH γw.

There is equilibrium when: D (Υ sat - Υ w) $\geq$ m ΔH Υ w., in which:

γ sat saturated volumetric weight of the ground
m a factor for the potential difference ΔH working on the plane CD determined from the rectilinear network
ΔH potential difference in metres equivalent to the difference between the phreatic level and the outside water level
D depth of the considered layer under the bottom of the harbour in metres

A critical situation arises at ΔH that satisfies: D (Υ sat - Υ w) = ΔHkrit Υ w.

the critical hydraulic gradient here is: $ikrit = \dfrac{m\ \Delta Hkrit}{D} = \dfrac{(\Upsilon\ sat - \Upsilon\ w)}{\Upsilon\ w}$

Verification of piping, basis and calculation variables
The verification of piping is based on NEN 6740. A potential calculation must be used to show that the design value of the normative hydraulic gradient id;crit it sand does not exceed: id $\leq$ id; crit = 0.5. For quay structures with a relieving structure, a composite sheet pile-system is often used. The length of the intermediate pile determines this. When the harbour is deepened, various cases of internal scour have occurred that have been caused by intermediate piles that were too short or undetected interlock openings, Besides, in this consideration/assessment/ beschouwing it is assumed that no bottom protection has been used. For verifications of piping the underside of the intermediate pile is considered to the underside of the sheet pile wall. When determining the design value for the level of the underside of the intermediate pile a reduction of 0.5 m in the design level of the bottom of the intermediate pile is applied to allow for possible deviations in execution. The two following unusual situations are considered:
- an unfavorable low outside water level based on a probability of within range frequency of 5% over the reference period and an unfavorable logical high phreatic level combined with an extreme scour of 2.0 m caused by propeller wash;
- an unfavorable low outside water level based on a probability of within range frequency of 5% over the reference period and a unfavorable high phreatic level based on a non-working drainage system.

For each situation the design value of the potential difference ΔH is equal to the differences in the design values of the groundwater and outside water level. For the determination of the design values of the geometrical variables such as bottom depth, groundwater and outside water level.

Verification of piping
To illustrate the calculations for the verification of piping with an intermediate pile in the situation described above, see Figure 36.

The basic assumptions are as follows:
- situation 1 with scour of 2.0 m, combined with an intermediate pile that is 0,5 m too short: potential difference ΔH w;d = 2.50 m;
- situation 2 with a non-working drainage system, combined with an intermediate pile that is too short by 0.5 m: potential difference ΔH w;d = 3.50 m.

The determination of the development of the excess pore pressures is carried out according to Figure. In this, the sheet pile is primarily located in sand and piping may occur. The maximum water pressure on the passive side occurs at the bottom of the intermediate pile, the rise in relation to the hydrostatic water pressure amount to 0.50 m in situation 1 and to 0.70 m in situation 2. The design value of the pressure gradient *i* amounts to 0.33 in situation 1 and to 0.20 in situation 2. This is smaller than the critical pressure gradient of sand of 0.50 and thus acceptable.

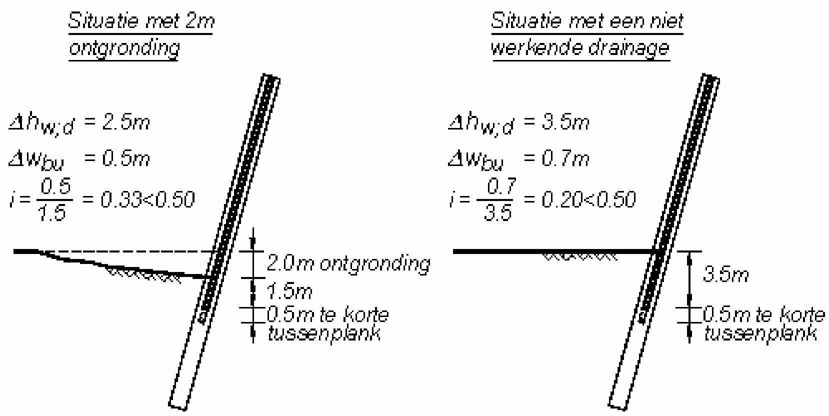


Fig. 36 Verification of piping in combined sheet pile wall system.

Verification of hydraulic soil fracture

Following on from this, in connection with piping it is also important to carry out a verification of hydraulic soil fracture. For this, as in the case of piping, the vertical equilibrium of a soil body on the passive side of the sheet piles is verified. The top of this soil mass/body is bounded by the bottom of the harbour and the bottom by underside to the intermediate pile. The width is assumed to be 50% of the depth to which the intermediate pile penetrates the bottom of the harbour. The design value of the upward flow pressure Wd must be in equilibrium with the design values of the effective weight of the soil mass under consideration Gd. The design values of the flow pressures are determined in the same way as given under the heading **Piping**.

The equilibrium condition is ϒ fsW Wd $\leq$ ϒ fG Gd.
The partial loading factor for the flow pressure ϒ fSW = 1.50. The factor for the effective weight ϒ fG = 0.90.

5.3.5 Calculation of the bearing capacity of foundation members

This section gives the essence of the calculation of the bearing capacities of the types of foundation members is discussed:

- bearing piles;
- tension piles;
- open tubular piles as bearing or tension piles.

Usually the calculation of the bearing capacity is based on penetration values that are associated with the stress condition of the subsoil. If the piles are not driven without vibration, for over-consolidated sand and gravel layers the penetration values must be reduced.
In addition, the reducing effects caused by excavation or dredging free after the penetration tests must be added to the penetration values. The timing of the installation of the foundation member (before or after the excavation) has an effect on the penetration values that must be taken into account.
It should be noted that influence of the technique used to create and install on the bearing capacity via a factor €, must be included in the calculations.
In principe, the calculations of the bearing capacity are made in the ultimate limit state 1A. An assessment is made to determine whether the design value of the bearing force is equal or greater than the design value of the pile:

$F_{r;max;d} \geq F_{s;d}$,	in which:
$F_{r;max;d}$	design value of the maximum bearing force
$F_{s;d}$	design value of the pile load.

In principle, the deformation behaviour of foundation piles should be assessed in the limit states 1B and 2. In view of the character of the structure of the superstructure of quay walls, which in the lateral direction are statically determined and with a heavy distributive function in the longitudinal directions, usually scarcely any requirements are set with regard to deformation. However, for bearing piles, which are loaded by negative friction and bearing capacity of which is determined on basis of the verification of limit state 1A according to NEN 6743, in the ultimate limit state very large deformations will occur. In fact, the final deformation is only reached when the settlement is so big that the defection in which the friction between the soil and the pile in the areas where negative friction occurs has reversed. In view of the unpredictability of the deformation behaviour, this is considered undesirable.

Therefore, in contrast to *NEN 6743*, for the verification of the ultimate limit state 1A the design value of the negative friction is taken into account as a load. In this way, the verification of the ultimate limit state 1B is translated into a verification of the ultimate bearing capacity in limit state 1A. The verification of bearing piles loaded by negative friction in the ultimate limit state 1A is: $F_{r;max;d} \geq F_{s;d} + F_{s;nk;d}$, in which:

$F_{r;max;d}$	design value of the maximum bearing force
$F_{s;d}$	design value of the pile loading
$F_{s;nk;d}$	*design value of the negative friction*

The calculation of the bearing capacity of bearing piles in a pile group can be based on a single pile since the effect of the other piles on the bearing capacity can usually be considered negligible.

When calculating the bearing capacity of tension piles, a distinction is made between a single tension pile and tension piles in a pile group.

Determination of the bearing capacity of the piles.
For the construction type discussed one may distinguish for the determination of the bearing capacity ether compression or tension the following items:
- sheet pile wall - compression capacity
- 1 pile in combi-wall-system bearing capacity
- tension piles, tension capacity
- foundation piles which support the relieving floor.

The computation is the bearing capacity is carried in according with the Dutch NEN-codes. In this code the several computation techniques are indicator for the specific used pile type.
In the Netherlands the computation of bearing capacity (tension, compression) is based on the C(one) Penetration T(esting) method. This implies that computation is done in relation to the measured cone resistance.

An other method which is worldwide used is based on the Stand Penetration Test method.
A useful relationship between the CPT and SPT results depending on the soil composition is given in Fig. 8

5.4 Gravity-type quay walls

General

Gravity-type structures are those which rely primarily on their weight and grip on the foundations to resist any of the possible adverse load combinations.

Gravity-type quay walls may be used at wharves receiving ships of any size and type; from small general cargo vessels to the largest container ships, and very large bulk carriers and super tankers. These walls are particularly useful and durable under severe marine environmental conditions, such as salt water, hot and cold temperatures, large waves, and heavy ice loads.

In the past and at present, in a great many cases gravity-type structures are used where local foundation conditions preclude pile or sheet-pile driving.

In general, because of their heavy weight and the character of the load distribution at the base, the gravity-type structures require reasonable foundation condition.

The structures can be built in dry or wet conditions. The underwater portion of the structure can be inspected by divers or with the help of special equipment. Repair of the underwater portion is usually a quite difficult and costly undertaking. This is basically the reason why in a great many cases different structural materials are used for construction of the underwater portion of the structure and its superstructure. As the cost of the underwater portion is usually high, it is customary to limit its height and place it at about 0.5 m above the assumed construction water level. In case of timber cribworks, the underwater portion of the structure is usually placed at about 0.5 m below the design minimum water level. As stated earlier, the type of the underwater portion of a gravity quay wall is dependent on local geotechnical, environmental and operational conditions.

The performance of the gravity-type wall depends heavily on its foundation. Usually a certain amount of wall movement is expected. The magnitude of gravity wall movements is basically a function of the type of wall, the quality of the foundation and backfill materials, as well as the nature and sequence of application of miscellaneous loads.

Basic structural arrangements

The following basic types of gravity quay wall are normally considered in modern marine engineering practice:
1. concrete blockwork structures;
2. structures composed of floated-in concrete caissons;
3. structures composed of large-diameter concrete and steel cylinders;
4. steel sheet-pile cellular bulkheads;
5. prefabricated concrete L-shaped walls;
6. timber and concrete cribwork with concrete superstructures of miscellaneous designs;
7. innovative designs.

Block-wall structure

The blockwork walls are by no means a new type of construction. This type of quay wall construction has been used in the past and is still considered by marine structure designers where local conditions are suitable and where walls of blockwork construction represent an economical alternative solution to the problem. It is generally believed that these walls are relatively immune to the various forms of serious deterioration affecting the more sophisticated thin-walled concrete structures such as alkali-aggregate reaction, carbonation, and chloride penetration. They can also be dismantled relatively cheaply if they should ever become redundant or obsolete.

Blockwork walls are typically built on competent foundation soils, or soils whose bearing capacity can be enhanced if required to meet design requirements.

The success of blockwork walls in the past and their continued use at present is based on certain advantages, which may be stated as follows:

1. excellent durability and reliability attributed to the robust nature of the marine concrete blocks;
2. relatively simple construction technique required;
3. use of basically readily available material;
4. good quality control achieved by the reproduction process of manufacturing pre-cast concrete blocks;
5. good response to major accidental impact by vessels.

Although there have been many variations on the basic design of blockwork walls, they are generally classified as follows:

1. bonded construction using solid concrete blocks;
2. walls formed with hollow or special concrete blocks.

With due consideration given to soil friction on concrete walls, the soil pressures on the frontal panel of counter fort walls as determined by standard methods can be reduced by 20-30%; the minimum reduction is used when the distance between adjacent counter forts is about 4 m, and maximum when this distance is about 2 m or less.

The horizontal component of the mooring force normal to the face of the quay is typically distributed along the capping superstructure. The distance to which this force is distributed depends on the type of bollard foundation and the type of capping superstructure.

Basic static principles

It should be noted that, in general, the design of the gravity earth-retaining structures has not undergone significant changes in the past 20-30 years. The existing practice has been considered by many marine structures designers as, although conservative, satisfactory.
This, however, is relevant mostly for the design of the gravity walls constructed on competent foundation soil. On the other hand, the stability of gravity walls to be built on soft foundation strata must be thoroughly examined, and necessary redundancy to the wall structure must be provided.

Because all types of gravity quay wall are, in general, considered as a solid block, the basis static principles are similar for all of them. The design process normally starts with a tentative dimensioning of the wall, which is followed by an analysis for stability and structural requirements; this is basically a trial process during which several alternative solutions are analyzed in an attempt to obtain the most economical and at the same time reliable solution to the problem. As a rule of thumb, the width of the base of a gravity-type wall basically depends on the type of wall construction, kind of the backfill material, and the properties of foundation soil; the width is typically (0.5 to 0.8)H, where H is the height of the wall.
In general, the design process can be programmed easily for the computer which may help to find the most economical solution to the specific problem.

The gravity-type wall is routinely analyzed for the following conditions.

1. Sliding stability at the base level.
2. Sliding stability of the wall-mattress system at the interface between the mattress and the foundation soil.
3. Acceptability of bearing stresses at the base level and at the interface between the mattress and the foundation soil.
4. Overturning stability.
5. General or global stability.
6. Settlement when constructed on compressible foundation.
7. Piping aspects.

Normally, all the above analyses are conducted on a typical cross section of the wall.
Several interactions of these analyses usually produce a balanced design with respect to economy and safety level. If potential problems are identified with respect to any one of the above design conditions, then several options are usually considered that include: (1) increase the base of the structure, (2) replacement of local bottom soil by good quality granular material, (3) relocation of the structure to an alternative site, or (4) performance of additional soil investigation to justify possible reassessment of soil parameters used in analyses.

In the following sections, the analytical procedures used for design of the gravity-type quay walls are described.

5.4.1 Design of gravity quay walls

5.4.1.1 Basic design principles

As stated earlier in this chapter, the blockwork walls are the oldest known type of structure used for quay wall and breakwater constructions. Because a substantial part of these structures is usually built underwater by divers, as well as the need to use very heavy marine construction equipment, sometimes makes blockwork quay walls quite expensive. However, where site conditions are favourable (e.g., long quay wall to be founded on a hard, competent foundation strata), a scarcity of skilled workers and foreign exchange and the availability of cheap labor to cast a large number of concrete blocks the blockwork wall can be very successful and present a competitive construction alternative.

The size of individual blocks is usually determined by wall stability requirements and by the capacity of the available block handling equipment both onshore and offshore. The weight of concrete blocks may vary from 5 to more than 200 tonnes. When local conditions so demand, the blocks can be cast with large pockets or voids to suit available handling equipment. If these blocks are too light to resist the design loads, the pockets or voids can be filled with concrete or iron ore. Sometimes if the occasion so demands, the interconnecting block reinforcement is provided in order to turn the wall into a monolithic structure. This type of block anchoring is particularly beneficial where frequent and severe stress reversal is expected (e.g., in the case of a quay wall used as a breakwater). If practical, individual blocks should cover the entire width of the wall; they have to be keyed to provide for continuity along the wall. The blocks are to be shaped and placed in a way to provide for the least horizontal soil thrust and for the best distribution of bearing stresses at the wall base. This may be archived through balancing the vertical and horizontal forces acting on the structure.

The concrete used for fabrication of blocks should be dense and resistant to effects of local water conditions. Typically, for better interaction, blocks are placed in a way to overlap each other.
In some cases they are placed in the form of individual columns.

As stated earlier, blockwork walls produce substantial pressure on foundation soils and normally the most favourable condition for this type of construction exists where blocks can be laid directly on rock levelled with stone bedding or with in-situ concrete footing; where bedrock does not exist, the blockwork wall must be founded on a carefully graded, well-compacted, and thoroughly screed rubble mattress. When the underlying foundation soil is not a good quality rock, the minimum thickness of a rubble mattress should be no less than 1.0 m. The surface of a mattress must be carefully cleaned from any sediments before the first course of blocks is placed, so that the interface between blocks and mattress does not become a sliding failure plane.

When fine-grained granular material underlies the rubble mattress, then to prevent the mattress from settling under the heavy load, the voids in rubble must be filled with suitably graded granular material. Otherwise, the gravel filter must be placed between the foundation soil and the mattress.

The quay wall design is usually based on static analysis conducted on the basis of at-site geotechnical, hydraulic, wind, and ice conditions as well as dock operation criteria. The latter typically includes complete information on a design vessel, cargo handling and hauling equipment, and miscellaneous live loads associated with dock operation.

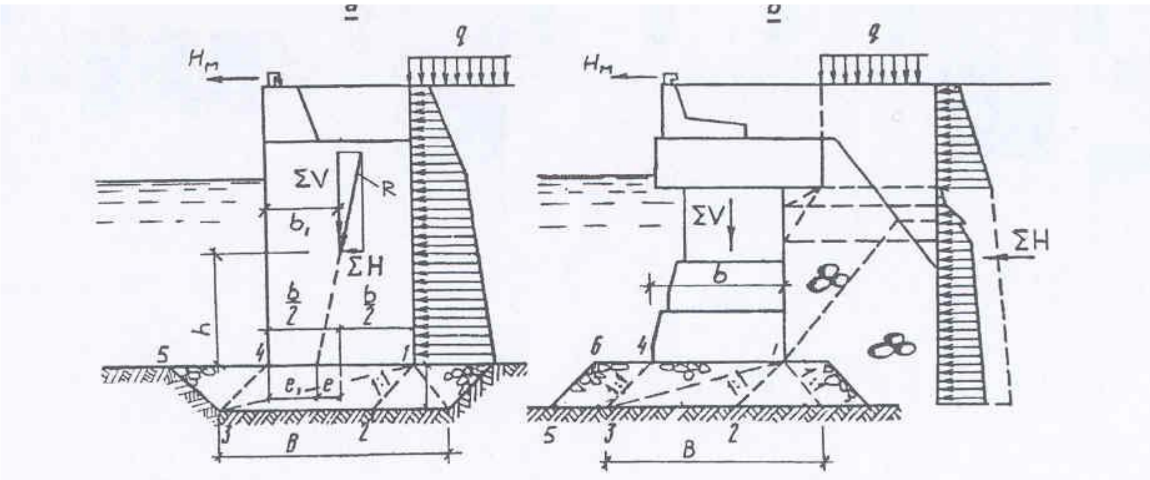


figure 37 the typical loading configuration of a gravity wall.

5.4.2 Stability against sliding failure

The wall must provide adequate stability against the sliding mode of failure which is determined by the following ratio.

$$F_{sl} = \frac{\text{Sum of resisting forces}}{\text{Sum of driving forces}}$$

where F_{sl} is the factor of safety against sliding.

The unusually low and rapidly developed water level in front of a quay wall that can be the result of a full-moon tide, wind effect, or severe storm waves, and the groundwater level behind the wall remains substantially higher produce unbalanced hydrostatic pressure on the wall, which should be taken into account.

The effect of an unbalanced hydrostatic pressure is less pronounced when the wall is placed on permeable rubble bedding and is backfilled by coarse granular material such as rubble or gravel. This enables free flow of water in and out, thus minimizing the possibility of unbalanced hydrostatic pressures.

5.4.3 Wall horizontal sliding

$$F_{sl} = \frac{\left(\Sigma V - \Sigma U\right) f}{\Sigma H}$$

where

F_{sl} = safety factor; F_{sl} should not be less than 1.5 for normal loading and no less than 1.25 for an extreme load combination.
ΣV = sum of all vertical loads acting on wall base; ΣV includes a vertical component of the lateral soil thrust.
ΣU = uplift (buoyant) force.
ΣH = sum of all horizontal driving forces; ΣH basically includes horizontal component of soil lateral thrust, unbalanced hydrostatic load, and mooring force.
f = coefficient of friction. For a concrete structure placed on a rock fill mattress, the coefficient of friction f of 0.5-0.65 is usually used. The friction coefficient between rubble mattress and foundation soil is usually assumed to be tg$\left(\frac{2}{3}\phi\right)$, where ϕ is the angle of internal friction of material used for the wall bedding or foundation soil, whichever produces the smaller value of f. The upper limit of the coefficient of friction f = tgϕ.

Note that $\left(\Sigma V - \Sigma U\right)$ represents the effective weight of the structure.
It should also be noted that in most cases the passive pressure is not included in the calculation of resisting forces.

5.4.4 Stability against overturning

This is determined from the following ratio:

$$\mathrm{F}_o = \frac{\text{Sum of moments to resist overturning}}{\text{Sum of overturning moments}}$$
$$= \frac{\mathrm{M_r}}{\mathrm{M}_o}$$

where

F_o = factor of safety against overturning; for normal loading, F_o is usually taken as equal to 2.0, and for extreme loading it is reduced to 1.5.

$\mathrm{M_r}$ and M_o = resisting and overturning moments, respectively, about the wall toe

The evaluation of the wall stability against overturning mode of failure is usually not required where the resultant vertical force is within the middle third of the wall base. Where e is the eccentricity and b is the width of the wall base.

5.4.5 Contact pressure

Contact stresses at the wall base and at the interface between the mattress and the foundation soil as well as at any critical horizontal section in the wall structure σ are determined from the following:

$$\sigma_{\max/\min} = \frac{\Sigma V - \Sigma U}{A} \pm \frac{M_o}{W}$$

where

ΣV = sum of all vertical loads acting on wall base; ΣV includes a vertical component of the lateral soil thrust.
ΣU = uplift (buoyant) force.
A = base (section) area
M_o = moment of all loads about the geometrical centre of the wall base or any critical section
W = base (section) sectional modulus about its longitudinal axis.

Subsequently, A = b per linear meter of wall; $M_o = (\Sigma V - \Sigma U)\ e$; W = b^2/6 and

$$\sigma_{\max/\min} = \frac{\Sigma V - \Sigma U}{b}\left(1 \pm \frac{6e}{b}\right)$$

where e is the eccentricity of the resulting load $(\Sigma V - \Sigma U)$ with respect to geometrical centre of the wall base or any critical section. For the rectangular base of the wall, e = 0.5b - e_1, where e_1 is the distance from the wall toe (front edge of the section) to the point where the vertical component of the resultant force acts at the wall base (critical section):

$$e_1 = \frac{M}{\Sigma V - \Sigma U}$$

where M is the moment of all loads about the wall toe (or edge of the critical section).

Relationship is used when $e \le b/6$; in other words, when the vertical component of the resulting force is acting within the middle one-third of the base (or middle one-third of the wall critical section). In the latter case, both $\sigma_{\max}$ and $\sigma_{\min}$ cannot be negative.
When $e \le b/6, \sigma_{\min}$ will have a negative value, and because the soil cannot take tensile stresses, the 'effective' bearing area per linear meter (ln. m) of wall will be somewhat smaller than b x (1.0 m) m². In conventional analysis, this effect is accounted for by reducing the bearing area of the footing according to empirical guidelines. According to the latter, the maximum contact stress can be determined from the following formulation:

$$\sigma_{\mathbf{max}} = \frac{2}{3}\frac{\Sigma V - \Sigma U}{e_1}$$

Normally, the $e > b/6$ condition would be acceptable for the extreme load combinations or in the cases where the wall is built on sound bedrock foundation or on foundations which include dense granular materials. In the case of bedrock foundations, the value of e should not exceed 0.25b, and in the latter case it should not exceed 0.2b.

Naturally, the maximum contact stress should not exceed the allowable value of the bearing stress, σ_f . The contact stresses at interface between the stone bedding and foundation soil, σ^1, under the $e \leq b/6$ condition are determined with due consideration given to the stress distribution through the mattress material at 45°,

$$\sigma^1_{\max/\min} = \sigma_{\max/\min} \frac{b}{b + 2h_m} + \gamma_r h_m \leq \sigma_f$$

where

γ_r = buoyant weight of the mattress material

$\sigma_{\max/\min}$ = bearing stress level at the wall base

h_m = mattress thickness

σ_f = allowable stress on foundation soil; for preliminary design, the data presented in table 1 may be used.

Accordingly, the minimum thickness of the stone bedding $h_{\min}$, can be obtained from

$$h_{m(\min)} \geq \frac{2\sigma_f - \gamma_r b}{4\gamma_r}$$

$$-\left[\left[\frac{2\sigma_f - \gamma_r b}{4\gamma_r}\right]^2 - \frac{b(\sigma_{\max} - \sigma_f)}{2\gamma_r}\right]^{0,5}$$

In more sophisticated analyses, the foundation is treated as an elastic half-space in which the foundation stiffness characteristics are represented by replacing the soil mass with linear spring and dashpot elements. In recent years finite element analyses are more used f.e. PLAXIS.

Table 1

Allowable pressure on foundation soils (σ_f) for preliminary design (kPa)

Type of Foundation	Normal Loading	Extreme Loading
Sound rock	One-seventh of ultimate strength in water bearing condition	One-fifth of ultimate strength in water bearing condition
Weathered rock	600 - 1.500	900 - 2.100
Marl and heavy chalky clay	250 - 750	350 - 1.000
Well-compacted crushed stone dense gravel bedding	600	850
Dense sand		
Coarse-grained with gravel	350 - 450	500 - 650
Medium	250 - 350	400 - 500
Fine	150 - 250	200 - 350
Sandy clays		
Void ratio 0.5	250 - 300	330 - 400
Void ratio 0,7	150 - 250	200 - 330
Void ratio 1.0	100 - 150	150 - 200
Dense clay	300 - 500	400 - 700
Soft clay	50 - 200	100 - 300

5.4.5.1 Ultimate load on foundation (bearing capacity)

In foundation engineering, it is generally assumed that the failure of a loaded shallow foundation would occur when the peak ultimate load is reached simultaneously with the appearance of slip lines at the ground surface, which is followed by foundation collapse and a considerable bulging of the soil mass on the side of the footing.

In general, computation of the ultimate load represents a problem of elastic-plastic equilibrium which can be solved in plane-strain and axisymmetric geometries. In the case of gravity-type quay wall the resulting force is inclined in relation to the wall base and is eccentric. Therefore, failure can occur either by sliding along the base or by general shear of the underlying soil. At the verge of sliding, the total (ultimate) load horizontal component ΣH is basically related to the vertical component ΣV by

$$\Sigma H_{\max C} = \Sigma V \tan \phi \, \mathrm{A'c_a}$$

where

A' = effective bearing area of the wall
c_a and ϕ = the adhesion and the angle of internal friction between the soil and wall base, respectively

The quay wall, which is essentially a retaining wall, is subjected to moments and shears in addition to vertical load. These forces are usually replaced by an eccentric-inclined load action at the wall base. As indicated earlier, the usual practice in the design of such walls is to resolve the eccentric-inclined load into two parts, namely: (1) an eccentric vertical load and (2) a central oblique load. Unless a heavy gravity wall is placed on a competent bedrock foundation the bearing capacity of the footing must be evaluated.

In conventional practice the bearing capacity of the footing is obtained by analyzing the problem in two separate parts: (1) the bearing capacity of footing subjected to the central vertical load and (2) the bearing capacity of footing subjected to the central oblique load. The two values are superimposed to get the bearing capacity of footing subjected to the eccentrical load.

There are several proposed methods for analysis of the bearing capacity of eccentrically loaded footings. The following are recommendations on the determination of foundation soil ultimate bearing capacity.

The bearing capacity equations provided are limited to shallow foundations and are based on the assumption that the soil is a rigid, perfectly plastic material that obeys the Mohr-Coulomb yield criterion. Note that, by general definition, shallow foundations are those for which the depth of embedment is less than the minimum lateral dimension of the foundation element.
The method is strictly applicable to idealized conditions of uniform soil strength. Where use of these equations is not justified, a more refined analysis should be considered.

Undrained bearing capacity

If loading occurs rapidly enough so that not drainage and hence no dissipation of excess pore pressure occurs, then an 'un-drained' (or 'short-term') analysis is performed. In this case, the soil is treated as if its friction angle ϕ equals 0, such that the stability of the foundation is controlled by an appropriate undrained shear strength, c.

The maximum gross vertical load V_u which a foundation can support under undrained conditions is computed from

$$V_u = (cN_cK_c + \gamma D)A^1$$

where

V_u = maximum vertical load at failure
c = undrained strength of soil
N_c = a dimensionless constant; N_c = 5.14 for undrained conditions (ϕ = 0)
γ = total unit weight of soil
D = depth of embedment of foundation
A = effective area of the foundation depending on load eccentricity
K_c = correction factor which accounts for load inclination foundation shape, depth of embedment, inclination of the base, and inclination of the ground surface; this factor is discussed below.

Drained bearing capacity

If the rate of loading is slow enough such that no excess pore pressures are developed (i.e., complete drainage under the applied stresses) and sufficient time has elapsed since any previous application of stresses such that all excess pore pressures have been dissipated, a 'drained analysis' is performed. The stability of the foundation is controlled by the drained shear strength of the soil. The drained shear strength is determined from the Mohr-Coulomb effective stress failure envelope (i.e., the cohesion intercept c^1 and friction and ϕ^1).

The maximum net vertical load V_u^1 which a foundation can support under drained conditions is computed from

$$V_u^1 = \left(c^1 N_c K_c + q N_q K_q + 0{,}5\gamma^1 b N_\gamma K_\gamma\right) A^1$$

where

V_u^1	= maximum net vertical load at failure
c^1	= effective cohesion intercept of the Mohr envelope
N_q	= $\left[\exp(\pi \tan\phi^1)\right] \left[\tan^2 (45^\circ + \phi^1)\, 2\right]$, a dimensionless function of ϕ^1
N_c	= $\left(N_q - 1\right)\cot\phi^1$, a dimensionless function of ϕ^1
N_γ	= empirical dimensionless function of ϕ^1 that can be approximated by $2\left(N_q + 1\right)\tan\phi^1$; Kumbhojkar (1992) evaluated the magnitude of N_γ and suggested some corrections to this factor that are based on the shape of the base, the depth and tilt of the footings, as well as the rigidity and layering of the soil below footing
ϕ^1	= effective friction angle of the Mohr envelope
γ_1	= effective unit weight
q	= $\gamma^1 D$, where D is the depth of the embedment of the foundation
b	= minimum lateral foundation dimension
A^1	= effective area of the foundation depending on the load eccentricity
K_c, K_q, K_γ	= correction factors which account for load inclination, footing shape, depth of embedment, inclination of base, and inclination of the ground surface, respectively; the subscripts *c*, *q*, and γ refer to the particular term in the equation; these correction factors are discussed below.

Effective area $\left(A^1\right)$

Load eccentricity decreases the ultimate vertical load that a footing can withstand. This effect is accounted for in bearing capacity analysis by reducing the effective area of the footing as discussed earlier or according to other available empirical guidelines.

Correction factors

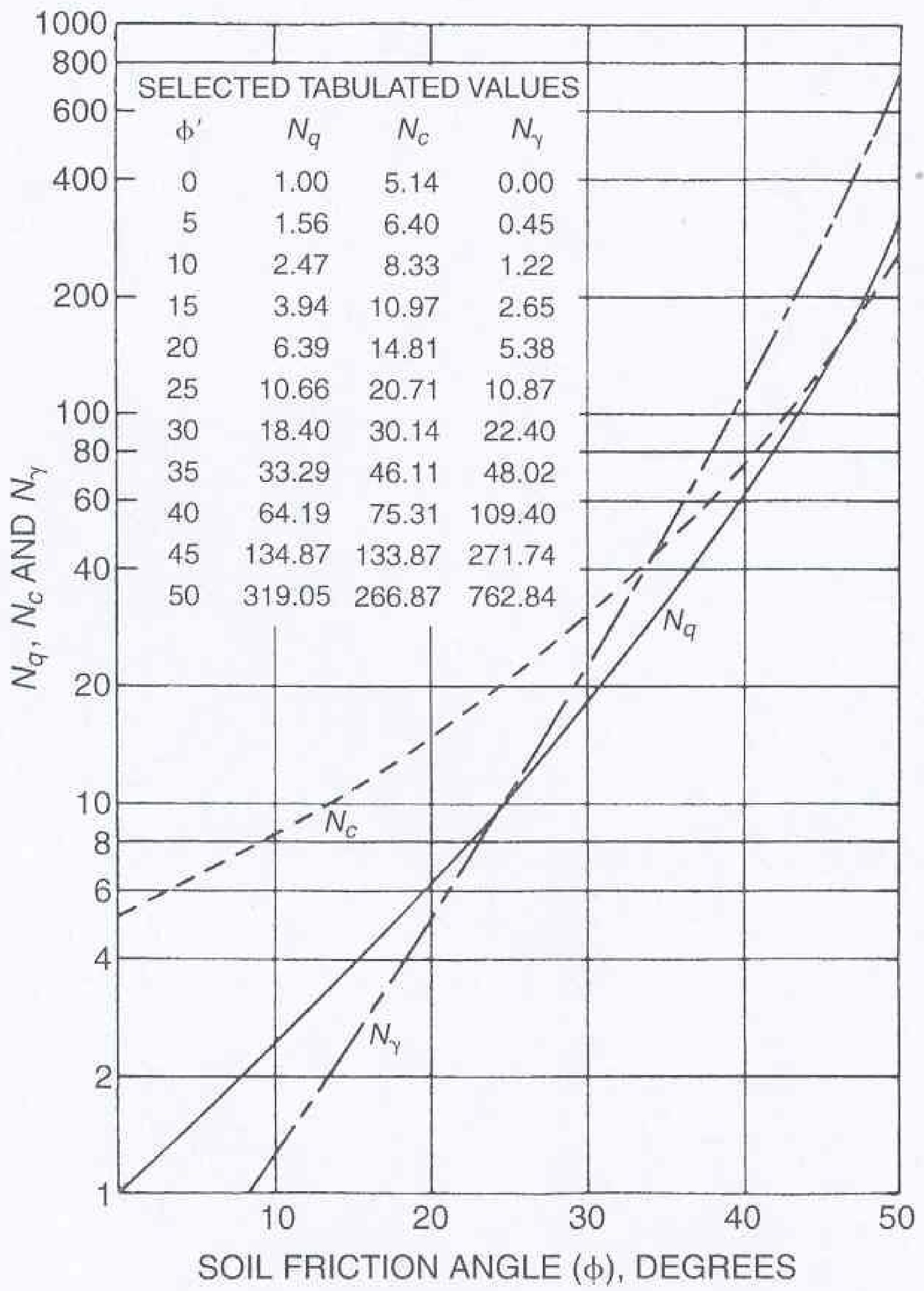
The correction factors K_c, K_q and K_γ are usually written

$$K_c = i_c s_c d_c b_c g_c$$

$$K_q = i_q s_q d_q b_q g_q$$

$$K_y = i_y s_y d_y b_y g_y$$

where *i*, *s*, *d*, *b*, and *g* are individual correction factors related to load inclination, foundation shape, embedment depth, base inclination, and ground surface inclination, respectively. The subscripts *c*, *q*, and γ identify the factor N_c, N_q, or N_γ with which the correction term is associated.



ϕ'	N_q	N_c	N_γ
0	1.00	5.14	0.00
5	1.56	6.40	0.45
10	2.47	8.33	1.22
15	3.94	10.97	2.65
20	6.39	14.81	5.38
25	10.66	20.71	10.87
30	18.40	30.14	22.40
35	33.29	46.11	48.02
40	64.19	75.31	109.40
45	134.87	133.87	271.74
50	319.05	266.87	762.84

Figure 5–62. Values of N_c, N_q, and N_γ as a function of the soil internal friction angle. [From Det Norske Veritas (1977).]

Fig. 38 Bearing capacity factors as function friction angle of soil.

Base and ground surface inclination factors

For a horizontal base and a horizontal ground surface, inclination factors are usually taken as equal to zero.
Because in most practical cases of quay wall construction the base of the wall is horizontal, the base inclination factors $b_q = b_c = 1.0$. For sloped ground surfaces, inclination factors are computed from the following expressions:

$$g_q = g_\gamma = (1 - \tan\beta)^2 \text{ when } \phi \geq 0$$

$$g_c = g_q - \frac{1 - g_q}{N_{c \tan\phi}} \qquad \text{when } \phi \geq 0$$

$$g_c = 1 - \frac{2\beta}{N_c} \qquad \text{when } \phi = 0$$

where β is the ground inclination angle in radians.
Numerical values of $N_c, N_q, \text{and } N_\gamma$ are tabulated in fig. 39.

Safety factor

The quay wall foundation should have an adequate margin of safety against bearing failure. Hence, values of ultimate bearing capacity determined by the described method are normally reduced by a factor of safety of 2.0 for normal loading and may be reduced by 1.5 for extreme load conditions. These values should be used after cyclic loading effects (if any) have been taken into account. Where geotechnical data are sparse or site conditions are particularly uncertain, an increase in values of factor of safety may be warranted.

Settlement and Tilt

Appreciable settlement of a gravity wall built on nonbedrock foundations may be expected; therefore, analyses to predict immediate, long-term, and differential settlement (wall tilt) are conducted when required to assure that the wall displacements are within tolerable limits for the overall satisfactory performance of the structure.
Walls built on dense granular soils would undergo most of the expected settlement by the time its construction and backfilling are completed; there, the long-term settlement is negligible because settlement immediate due to the rapid dissipation of pore pressures. In the case of cohesive soils with consolidation potential, the wall will continue to settle for some time after the completion of construction, because excess pore pressure dissipates very slowly in highly impermeable soils.

When long-term settlement is expected, then in order to keep the settlement relatively uniform, the resultant force, *R*, should be kept within the middle of the wall base.
A reliable prediction of the various settlements requires a thorough knowledge of the soil properties and subsurface variations along the wall. Comprehensive laboratory testing of quality samples is required to better understand the stress history, time-rate consolidation characteristics, Young's modulus, and the effects of cyclic loadings on the engineering properties of the foundation soils.
Where the foundation soil varies greatly along the wall, a differential settlement may result. In this case, the wall performance can be improved by replacing, compaction, or stabilizing the relevant soils along the wall or by reducing the contact stresses on the wall base and, therefore, on the foundation. The latter can be achieved by increasing the width of the wall base and/or by increasing the depth of the stone bedding (mattress).
A certain amount of wall tilt should be expected when it settles.

Table 2 Allowable displacements for gravity quay walls

	Mode of Displacement (cm)					
	Uniform Settlement (cm)		Tilt (radian)		Horizontal (cm)	
Type of Wall	Gurevich	Brum et. al.	Gurevich	Brum et. al.	Gurevich	Brum et. al.
Blockwork	-	10 15	-	0,01	-	-
Floating-in caisson	12 - 15	15 - 20	0,005 - 0,008	0,015	5 - 8	-
L-shaped with internal anchorage	10 - 12	-	0,005 - 0,008	-	4 - 6	-
L-shaped with external anchorage	10 - 12	-	0,005	-	4 - 6	-

When wall tilt is not associated with a foundation failure, its rotation toward and away from the fill is most likely to occur. The problem with wall method the effect of shear force existing between adjacent segments is not taken into account, this method, in general, is treated as conservative. Therefore, a factor of safety equal to 1.1.

The overall stability is normally investigated with a Bishop analysis.

In conclusion of this section it is worth mentioning that wall movements to the greater or lesser degree have always been experienced and should be expected unless it is founded on hard rock. To date, no many failure cases have been reported, and in nearly every case of those reported, the cause can be traced to a bad foundation. Almost all of them indicate either neglect in obtaining sufficient site geotechnical data or name appreciating the significance of some soil characteristics.

5.5 Loads on jetties

Jetties can be saver loaded f.e. by wind, wave ships.
In figure 39 these loads are indicated.

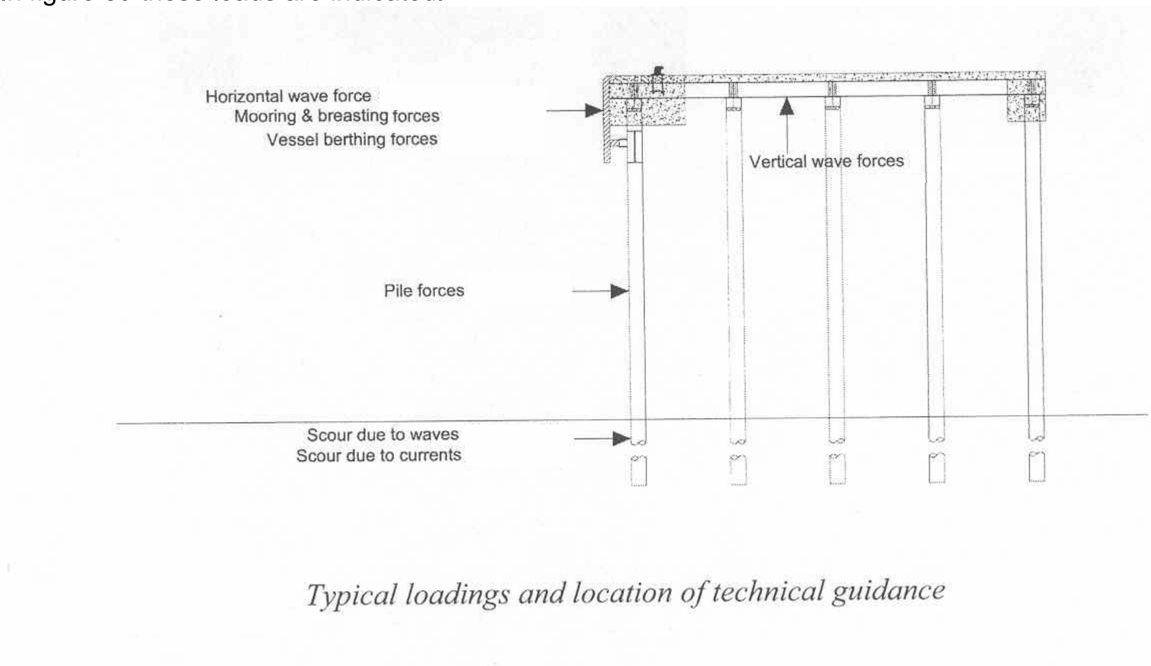


Fig. 39 Loads on jetties.

5.5.1 Hydraulic and related loads

Hydraulic loadings to be considered vary significantly between different structure types. The main hydraulic design issues to be considered are summarised in the following sub-sections. Their relative importance for the various types of structure is summarised in Table 5.

Table 5 Structure types and important design aspects

	(a) Solid, vertical wall	**(b) Rubble mound**	**(c) Open piled**	**(d) Marginal quay**
Wave/current drag or inertia wave loads (quasi-static)	Usually design case, resisted by weight	Act on armour units, armour sizing	Frequent but moderate significance	Rare and moderate
Wave overtopping loads	Possibly important but often ignored	Important and usually predicted	Usually ignored but can be very dangerous	Usually ignored but can be very dangerous
Uplift forces	Methods available, but often ignored	Methods available and easily predicted	Seldom predicted, no reliable methods	Seldom predicted, no reliable methods
Wave slam or impact forces	New methods now available	Methods available	Not well predicted	Not well predicted
Vessel mooring loads	Moderate	Not important	Significant	Frequent and significant
Vessel impact loads	Not significant design case for the jetty	Not significant design case for the jetty	May by critical	May be important
Local bed scour	May be severe	Less than for (a) but may be significant	Only local and for limited cases	Can be an issue in estuaries due to high currents
Shoreline morphology changes	Potentially significant	Potentially significant	Usually very small	Seldom significant
Future sea level rise	Important	Important	Important	Important

Note: this list is for general guidance only. The designer should identify the issues of concern for any particular scenario, where the relative importance of various issues may differ from that identified here.

5.5.2 Quasi-Static Wave loads

Quasi-static loads are slowly varying wave-induced forces, where the duration of the loading is typically of the order of 0.25 to 0.5 times the incident wave period. The magnitude of quasi-static forces is generally a function of the incident wave height.

Wave drag and forces on piles

To determine wave forces for piles and related elements, Morison's equations are used with appropriate coefficients. Relatively little information is available for the particular configurations of exposed jetties, but methods to apply Morison's method for a single extreme wave are well established. Little guidance is available for random waves, nor on phasing of loads along the length of multiple pile structures. The effects of wave obliquity, short-crestedness and reflected waves on wave loads are not well defined and the designer should consider whether detailed physical modelling or other studies are required to fully address these issues.

Quasi-static wave (momentum) loads on vertical walls and related elements

For vertical or composite walls, quasi-static wave loads may be estimated using methods based on Goda's equations originally developed for caisson breakwaters (see Goda (1985), CIRIA/CUR (1991), BS 6349 Pt 1). Guidance for evaluating the effects of obliquity and short-crestedness is given by Battjes (1982).

5.5.3 Wave overtopping loads

<u>Wave overtopping induced impact loads acting downwards on decks</u>

The loads induced on a deck by the impact of overtopping water have not been predicted by any established method. Research studies are being undertaken as part of the VOW's ('Violent Overtopping of Waves at Seawalls') project by Edinburgh, Sheffield and Manchester Metropolitan Universities which have identified example loads due to severe overtopping events, but no generic methods are known.

5.5.4 Wave uplift forces

<u>Quasi-static wave pressures acting upwards on walls or other submerged elements</u>

Quasi-static wave pressures acting upwards on submerged elements may be predicted using simple wave theories, or Goda's method originally developed for caisson breakwaters on rubble foundations (see Goda (1985), CIRIA/CUR (1991), BS 6349 Pt 1 (2000)). Vertical wave forces on the underside of decks are discussed in Chapter 5, based on the results of new model tests.

<u>Impact or slam loads acting upwards on decks or other elements</u>

Impact of slam loads acting upwards on the deck or other elements cannot be predicted by any generic method.

5.5.5 Wave slam forces

Wave impact/slam forces acting in the direction of wave travel on fenders, beams or other projecting elements

Wave impact/slam forces acting in the direction of wave travel on fenders, beams or other projecting elements can be estimated (with low reliability) using adaptations of methods by Goda (1985), Blackmore and Hewson (1984) or Müller and Walkden (1998), or by calculations using slam coefficients in Morison's equations.

5.5.6 Current forces

Loads imposed on exposed structures by tidal or fluvial currents can be classified as:

(a) drag (or in line) forces parallel to the flow direction;

(b) cross flow forces, transverse to the flow direction.

Current drag forces are principally steady and the oscillatory component is only significant when its frequency approaches a natural frequency of the structure. Cross-flow forces are entirely oscillatory for bodies symmetrical to the flow. Further guidance and methods of force calculation are given in BS 6349 Part 1, BSI (2000) and Sumer and Fredsøe (1997).

5.5.7 Vessel induced loads

Vessel berthing loads on jetty

Vessel berthing forces are taken by the jetty and/of independent berthing structures, usually through fenders, as vessels come to rest at the berth. Structural design methods are generally based on vessel energies and on the characteristics of the fender systems. Dynamic conditions are usually simplified to static equivalent loads. These forces can be assessed using guidance given in BS6349 Pt 4 (1994) PIANC.

Vessel mooring and breasting forces

- Mooring forces are due tot wind, wave and current forces pushing the vessel *off* or along berth and the load being transferred to the structure through mooring lines.
- Breasting forces are due to wind, wave and current forces pushing the vessel *onto* or along berth and the load being transferred to the structure through fenders.

There are uncertainties in the load transfer from vessel to structure and modelling is often undertaken to assess loads.

5.5.8 Bed scour or morphological change

Lowering of sea bed at or close to pile or wall

Lowering of the sea bed at or close to piles or walls due to waves and currents can be estimated by methods suggested by Whitehouse (1998). These are described in more detail in Chapter 7. Where underkeel clearances are small, propeller and bow/stern thruster scour may be an issue. For further guidance on propeller scour, see PIANC (1997), EAU (1996) and Römisch and Hering (2002).

These jetties are constructed with open piles to minimise interruption of waves, currents and sediment movement along the coastline, and to minimise wave forces onto the structure. A typical jetty is shown in Figure 2.6. For oil or gas cargoes where cargo transfer is by flexible hoses or marine loading arms with swivel joints, these types of jetties are typically designed to be so high that there is always an 'air gap' between the crest of the extreme design wave and the underside of the jetty deck. The air gap is provided to eliminate the occurrence of wave loads on the jetty deck and protect topside equipment. The jetty deck elevation may, however, be dictated by berth operations and vessel draught and freeboard, to ensure efficient design and operation of loading arms. Where the air gap is not sufficient and loading occurs on the underside of the jetty deck, deck elements may be damaged by wave action.

5.5.9 Typical quay built over slope

These structures share features of the other three structure types. Generally a piled deck used for cargo handling is constructed over a marginal slope, which is armoured, usually by rock (Figure 2.9). The vertical face is required for berthing against while the rock armour slope assists in dissipation of wave energy. The quay structure may also accommodate vessel mooring loads through fenders.

The deck level for these quays is generally set much lower than for open-piled structures, often driven by the levels of surrounding paved areas and access roadways. As a result, extreme storm conditions can generate wave slam forces on structural elements, and can cause overtopping impacts onto the upper deck. Wave shoaling and run-up on the armoured slope may also generate significant uplift forces on the deck, which can cause damage of the deck, as illustrated in Figure 2.10. The detail at the top of the armoured slope is also particularly vulnerable, as wave energy can be concentrated in this location causing armour damage.

5.5.10 Typical locations

Exposed jetties are constructed worldwide. Some structures (at their most exposed outer ends) can be remote from the land in deep water, where shallow water effects are small. Some sites or structures may still be exposed to large waves, such as marginal quays in regions subject to cyclones or hurricanes. At these sites, shallower water may allow breaking wave effects to become more significant.

Whilst exposed jetties are constructed worldwide, they tend to provide a more economical solution where dominant wave conditions are relatively calm and more severe wave conditions and storms are relatively rare and/of limited to a short period of the year (as for locations subject to monsoon conditions).

5.6 Flexible dolphins and berthing beams

<u>General</u>

Flexible dolphins are vertical or near vertical piles cantilevered from the river or sea-bed which absorb the berthing energy by deflection of the pile heads horizontally under the berthing impact. Dolphins may be formed of a single pile or of a group piles acting together.

Berthing beams are formed of a row of flexible piles covered by one or more horizontal girders which are equipped with panels of rubbing material e.g. wood, polyethylene, rubber, etc. Both structures can be equipped with rubber fenders in order to enhance the energy absorption capacity.

Application

Flexible dolphins are commonly used at jetties where unloading takes place at dedicated places, e.g. for liquid bulk, gas, oil, etc.

In front of the loading platforms often berthing beams are used, especially when small ships have to be also accommodated and/or large berthing angles are likely to occur. Berthing beams are also often used as guiding structures for lock and bridgepiers.

Considerations for using flexible dolphins or berthing beams instead of fenders mounted on the jetties are:

- separation of functions: avoiding fenderloads on operational structures, such as loading platforms in order to reduce movements and vibrations;
- safety aspects: in the case of overload due to calamities etc. the operations structures can be kept intact.

General technical aspects

In order to provide a safety margin in case of accidental extreme berthing, it is recommended to increase the distance between the face of the fender panel (fenderline) and the structure, e.g. a distance of twice the maximum elastic deformation of the pile enables a possible (plastic) energy absorption of over 3 times the ‘elastic’ design-energy (provided that the pile has enough yielding capacity).

The clearance between fender face and pile has to be enough to prevent the pile from being touched by the berthing ship. In addition to the maximum deflection, the heel and any belting of the ship has to be taken into account.

The energy capacity of a flexible pile is proportional to the square of the steel stress and linear to the applied wall thickness.

Hence the use of high tensile steel and a large wall thickness is effective for high energy absorption.

When selecting the design level of the seabed bottom, the effect of scour around the pile has to be taken into account.

Loading and load factors

Flexible dolphins should be designed to resist the following forces:

a. berthing impact;

b. hawser forces where the dolphins are also used for mooring purposes;

c. wind, wave and current effects on the ship.

The following load factors for the limit state design method are advised:

Load factor: depending on the pile capacity to resist overloads by plastic yielding.

- no yielding possible: γ = 1.25

- yielding possible until a displacement of at least two times the maximum elastic displacement: γ = 1.0.

Soil parameters: the factors as indicated in geotechnical specifications should be used. Material factor on steel: normally a factor of 1.0 can be adopted.

In the case of not predominantly static loading, the decrease in the fatigue strength with reference to the static strength has to be observed (especially in welds).

When the Working Stress Design is used, the allowable stress in the design standard of each country should be used.

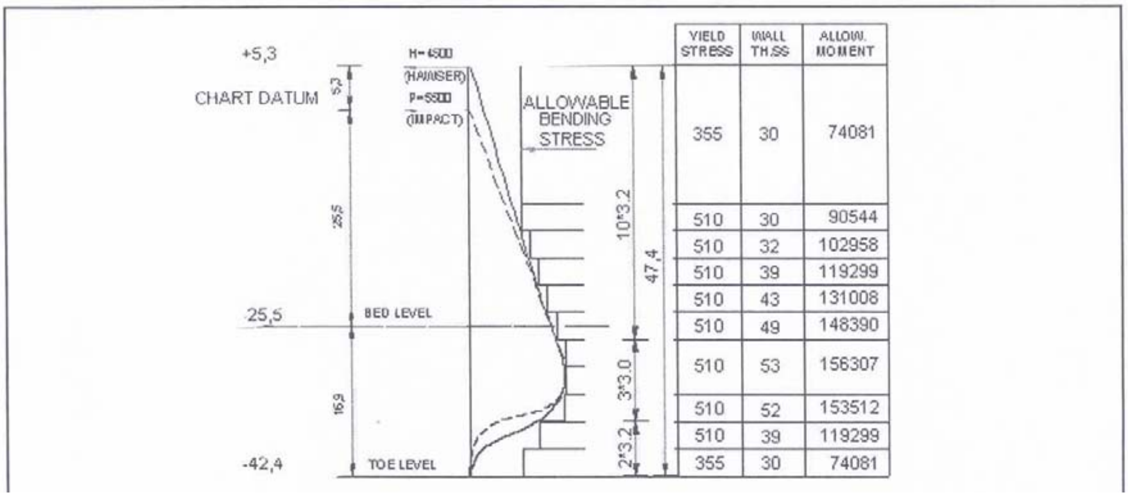
Geotechnical considerations

Suitability of flexible dolphins is dependent on soil conditions capable of resisting the horizontal loads exerted by the embedded length of the pile during impact of the vessel and returning the pile to its original position when berthing or other applied forces have ceased to act.

For the pile analysis, four methods are mentioned:

A. The methods based on the earth pressure theory under ultimate equilibrium condition of the soil, e.g. BLUM's method (EAU, ref. 1), Brinch Hansen's method.
B. An elastic approach (subgrade reaction), as proposed by Matlock and Reese (OTC 1204 & 2312, 1970 & 1975 respectively) and conform the API standards using p-y curves.
C. The PHRI method in which the soil is regarded as non-linear, as proposed by the Port and Harbour Research Institute (see references).
D. The best method to describe the soil – pile interaction is a three dimensional finite element model that takes plastic deformation into account. However this approach is elaborate and requires specific soil data.

When adopting the design values for soil parameters, toe level, etc. it is important to keep in mind that both stiff and soft behaviour of the subsoil and minimum and maximum toe level should be considered. Stiff soil and high toe level with impact on a low level are important as they effect the dimensions of the cross-section of the pile.



YIELD STRESS	WALL TH.SS	ALLOW. MOMENT
355	30	74081
510	30	90544
510	32	102958
510	39	119299
510	43	131008
510	49	148390
510	53	156307
510	52	153512
510	39	119299
355	30	74081

Bending moments allowable moments and pile makeup

Fig. 40 Example of allowable moments of dolphin.

<u>Materials</u>
If dolphins requiring a high energy absorption capacity are required, it is practical and economical to construct them of higher-strenght, weldable fine-grained structural steels. Steel qualities with yield stresses within the range of 355 to 690 N/mm^2 are used.

Piles are mostly of circular shape and built up of sections with variable wall thickness. The upper sections should be of easy weldable steel to facilitate welding on site of the upper section, deck or other fittings. It is recommended to select wall thickness large enough to enable some plastic deformation before local buckling of the pile shell occurs. Another way to reduce the local buckling problem is fill the pile up to a height of 6-10 m above the bottom level with a mixture of sand and gravel or concrete.

Special attention should be given to the horizontal welds in cases of severe corrosion attack combined with fatigue effects. In these cases a lower strength steel quality is advised.

<u>Equipment and details of breasting dolphins</u>
The contact area between ship and breasting dolphin is mostly formed by a panel of hard wood or a steel panel with ultra high molecular weight polyethylene (UHMW) pads.

For a better distribution of the contact pressure on the vessels hull, the panel may be designed to be able to rotate. In Figure 42 an example is given.

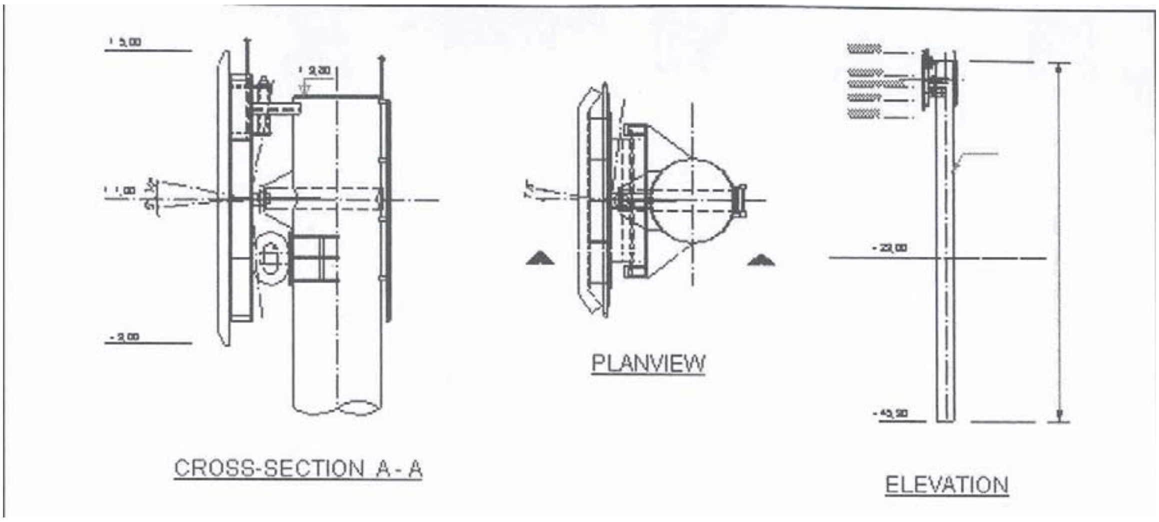


Fig. 41 Example of flexible panel.

5.7 Ro-Ro-facilities

Ro-Ro-constructions are structures which give the possibility to load and unload the ship by vehicles i.e. cars, trains.
Therefore it is necessary that a temporary approach is available to make the transport of cars or trains possible.
For this type of transport especially designs are available that enable them to manage their unloading/loading procedures. Concerning the ships different types of ramps are present today:
- side ramp;
- port quarter ramp;
- starboard quarter ramp;
- stern slewing ramp.

These constructions pose their loads directly on the quay structure. The facilities for this type of ramp on the quay structure are limited. The part of the quay which is used for these activities must be flat and should not have bollards in the direct vicinity.

The loads exposed at the quay structure are generally limited up to 40-50 kN/m^2. These loads are controlled from the ship.

For bow ramp and straight stern ramp mooring facilities more extended facilities are required. Three possible solutions are available:
- moving facility;
- fixed facility;
- floating facility.

- Moving facility
  Two extreme situations can be considered as indicated in Fig. 43 if the ship to shore connection is made i.e. high tide with unloaded ship and low tide with loaded ship.
  The hinge point being S.

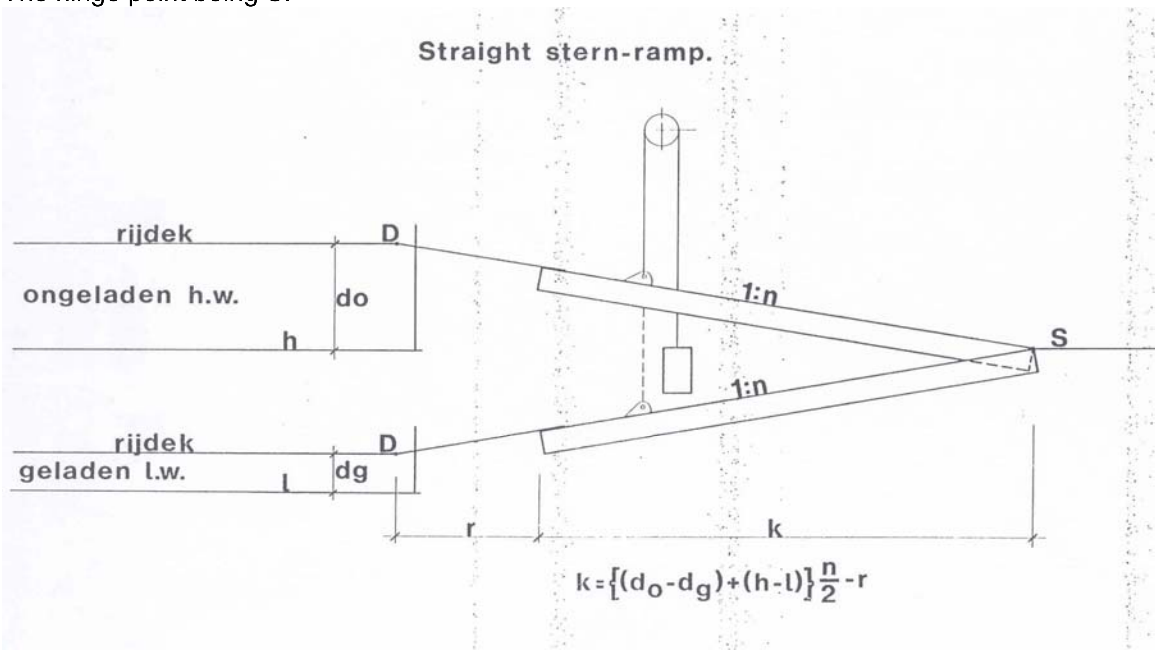


Fig. 42 Design expects of moving ramp.

- The length of the ramp is determined by $k = \{(do - dg) + (h - l)\}\dfrac{n}{2} - v$ in which

h	=	high water level (m)
l	=	low water level (m)
do	=	distance between hinge *D* of ship ramp and water level of unloaded ship (m)
dg	=	same distance however with situation of unloaded ship (m)
v, k	=	horizontal distance of ships ramp and Ro-Ro-ramp respectively
l/n	=	maximum acceptable slope.

Besides these considerations one has to realize that also guiding structure and/or mooring dolphins are necessary.
In the Baltisea Ro-Ro-traffic is very common. There are constructions were the ships more in a harbour dock.
This also means that very high flow velocity may occur that require measures to prevent scour.

6. Constructing quay walls

6.1 General

In this paragraph several practical aspects of constructing quay wall are discussed.
The items that are discussed are i.e., driving aspects, material aspects, dredging aspects.
For special construction types additionally information is provided and discussed.

6.2 Effects of dredging

The bearing capacity of the piles is calculated on the basis of the cone penetration test results. It is important, therefore, to know the reduction of the cone resistance after completion of the construction, excavation of the building pit and dredging of the harbour basin.

Today, three methods are available to predict the effect of reduction of the cone resistance:
- Begemann method;
- Broug method;
- Brooker and Ireland method.

An overview and comparison of these methods can be found in Gijt, J.G. de and Brassinga, H.E. (3).
Figure 44 gives the measured cone resistance (q_{cv}) before and after dredging (q_{cm}) of the sea quay-wall for the Euroterminal methods. This figure shows that the cone resistance after dredging can be adequately predicted using the Broug method or the Brooker and Ireland method.

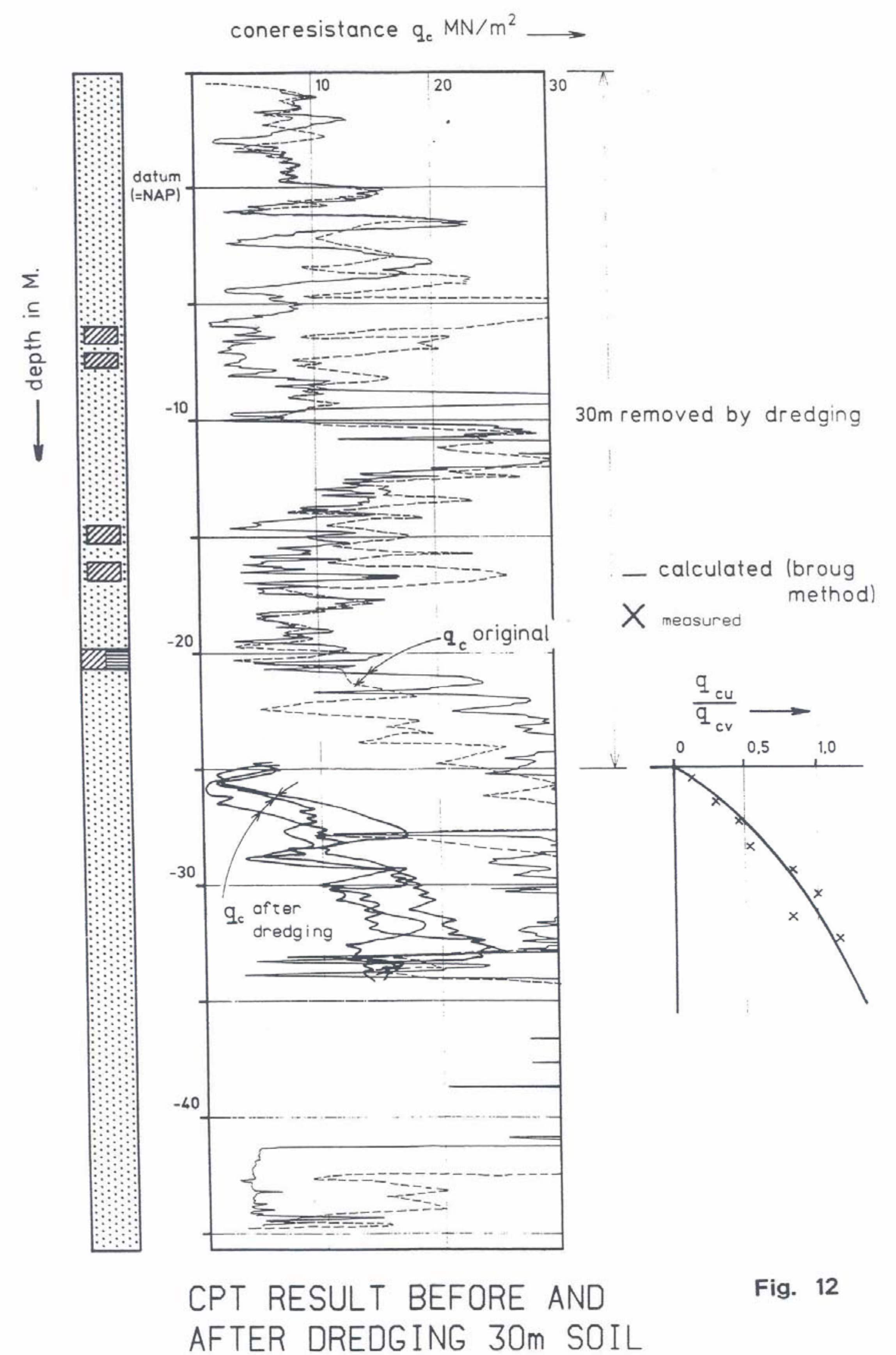


Fig. 43 Effects of dredging or CPT-results.

6.3 Pile bearing capacity

The effect of the reduction of cone resistance or the bearing capacity should taken into account.

6.4 Drivability analysis

By performing drivability analysis it is possible to anticipate on installation problems during the construction phase.

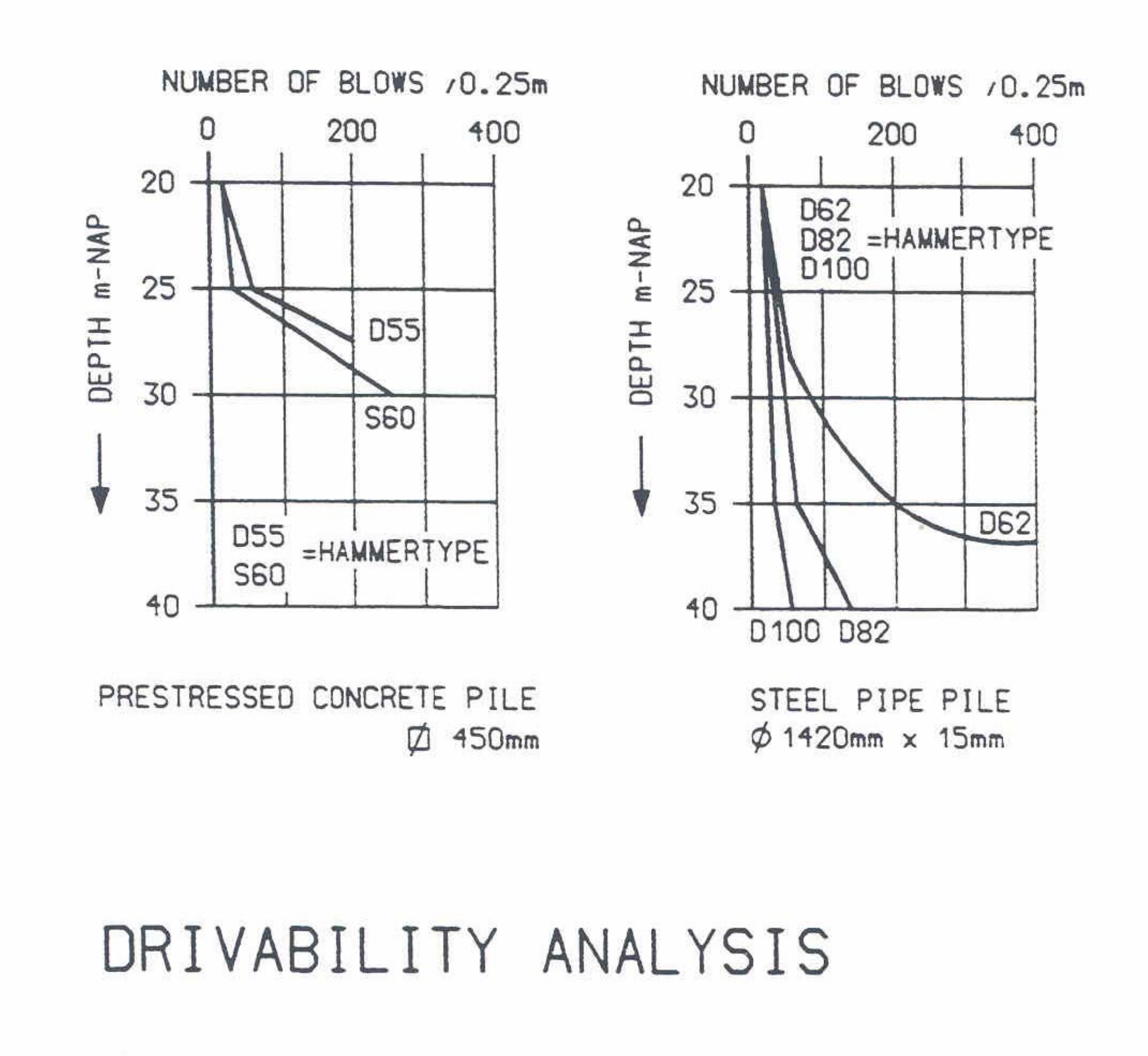


Figure 44 An example of drivability analysis is shown for both steel and prefab concrete piles for different hammer capacities.

6.5 Structural aspects

In this paragraph some of the new construction parts of the quay-wall are described. Since the first use of sheet wall elements, this system has evolved as indicated in *Figure 24.* The present combiwall consists of pipe piles with both a retaining and bearing function and sheetwall elements in between.
In Figure 46 the cast iron saddle construction is shown. The eccentric position of the saddle on the pile can clearly be seen. This saddle is functioning as a hinge.

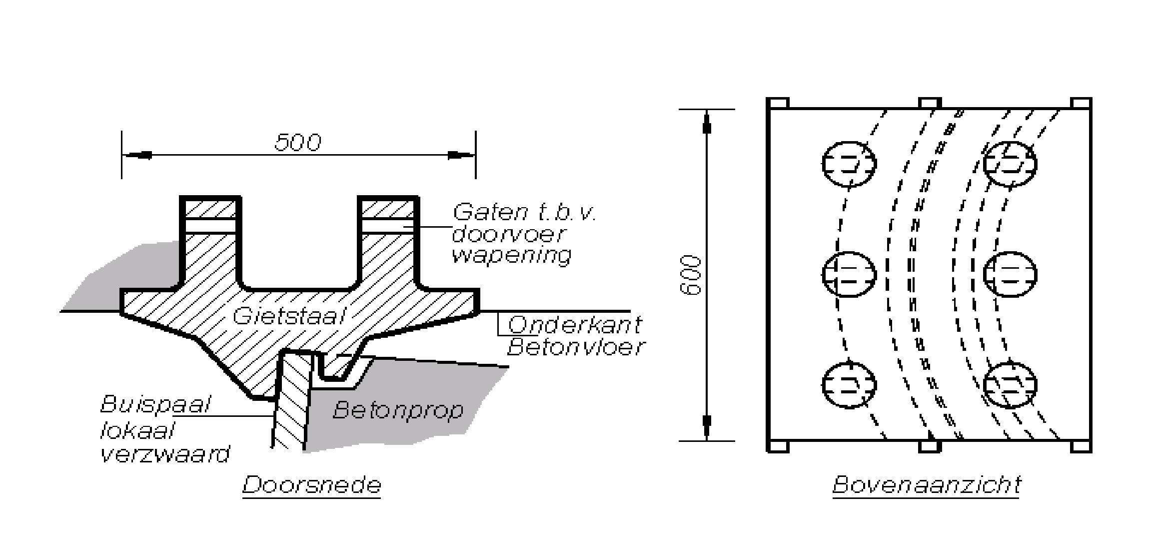


Fig. 45 Cast iron saddle.

The M.V.-pile (M.V. stands for Müller Verfahren), *Figure 47,* as used in Rotterdam is a modification of the originally used form.

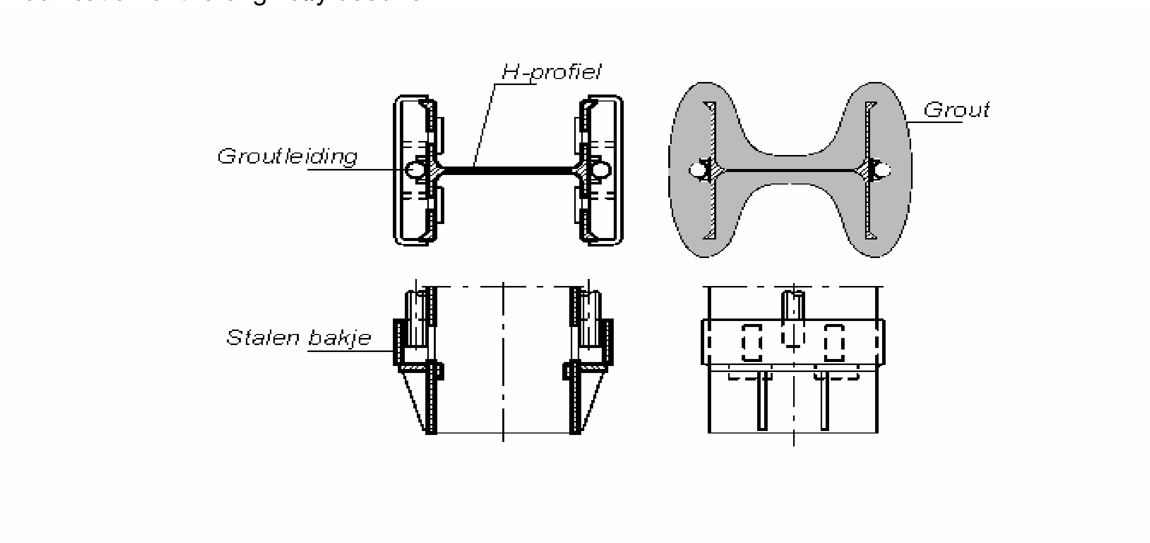


Fig. 46 M.V.-pile.

This modification comprises a reduction of the tip area, facilitating pile-driving and reducing grout consumption. The pile-drivability is 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 and an enlarged tip.

6.6 Installation aspects

6.6.1 Installation of the combiwall

The following procedure is used for the installation of a combiwall. The open steel pipe piles, Ø 904 - 1,420 mm, length up 30 to 35 m when very dense layers are encountered, are installed by vibration (RBH 160) down to the pleistocene sand layer. The piles are then driven into the pleistocene sand layer to the final depth with D62 of S200 hammer. This installation procedure is adopted for reasons of practicality, time and effort, and to obtain adequate bearing capacity.
A guiding frame is used during the installation of the piles, as illustrated in *Figure 47.* Using this frame, the positioning of the piles can be secured.

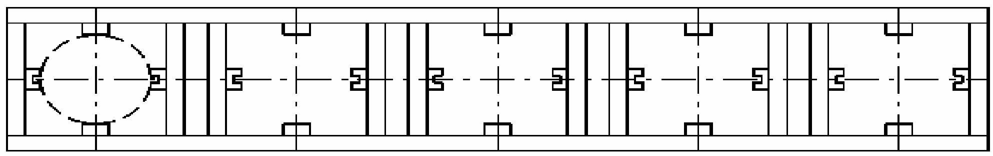


Fig. 47 Guiding frame for the installation of combiwall.

After the installation of the open steel pipe piles, the sheet wall elements are installed. The combiwall installation and, in particular, the sheet wall elements Larssen IIIS or equivalent with lengths up to 24 m, require a high degree of skill from the installation contractor in order to prevent the occurrence of interlock openings.

The following installation procedure is therefore prescribed:
1. vibration with waterjetting
2. vibration.

In exceptional situations, driving is permitted.

When the soil conditions show that the strength is relatively low or that the installation must take place in a slope or close to adjacent structures, preference is given to installing the combiwall by driving, to avoid settlement and flow slides and minimize generation of excess pore pressure.

From the driving and vibration behaviour of the combiwall, it is very difficult to obtain data which indicate if interlock openings do occur. This means that interlock openings are only traced during dredging operations when the harbour basin in front of the quay-wall is constructed.

Repair of interlock openings is a very costly operation and should therefore be minimized. Therefore, detection methods were developed and tested.
In *Figure 48,* three of these methods are displayed. The approach switch method is the only one which gives a continuous record.

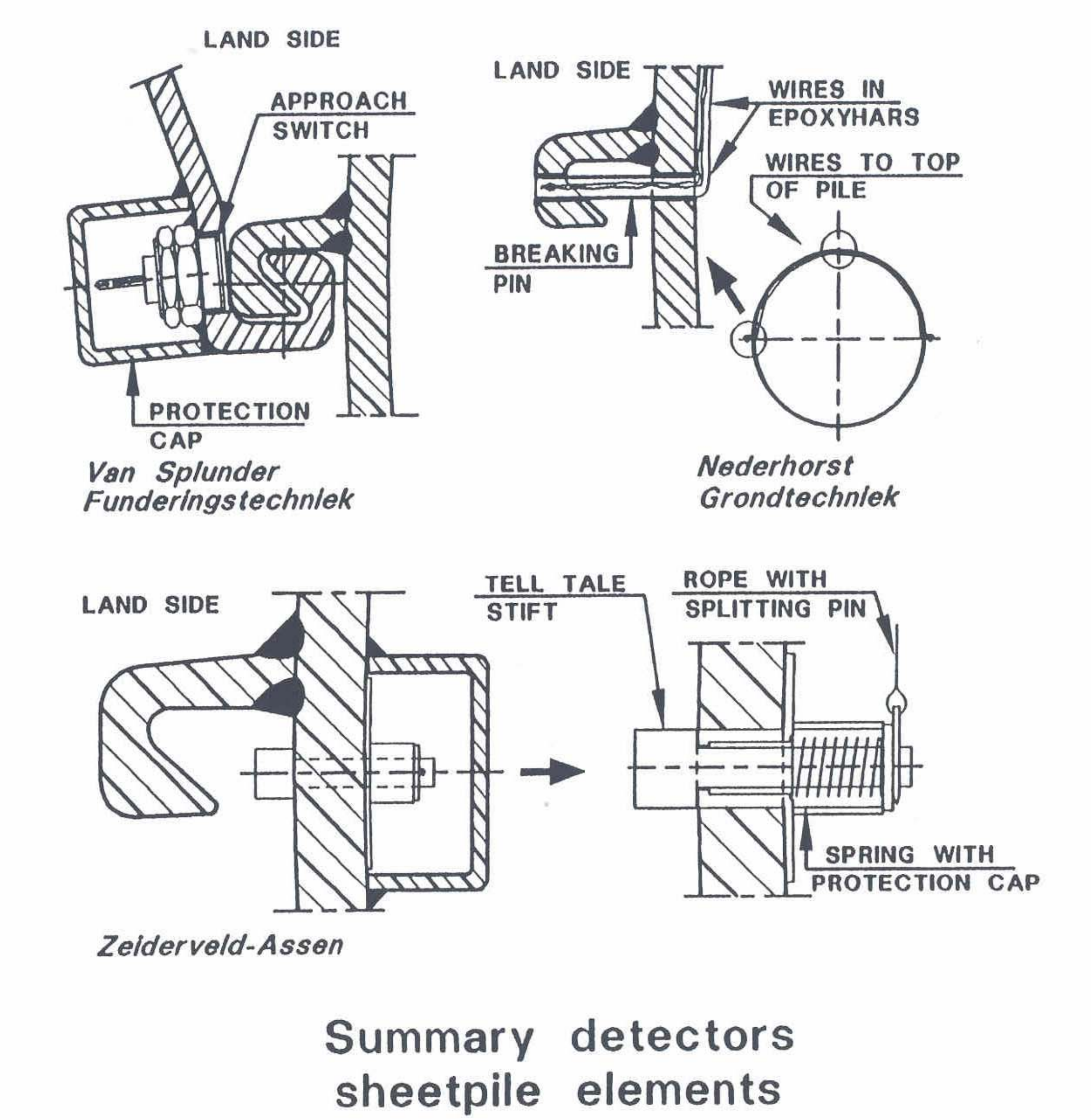


Fig. 48 Interlock detectors

6.6.2 Installation of concrete piles

The prefab prestressed concrete piles have diameters of 450 and 500 mm, and vary in length from 22 m to 30 m.

Experience with previous projects in the Maasvlakte area has shown that it is necessary to use waterjetting pressures ranging from 5 to 10 bar to install the pile up to the top of the pleistocene sand layer. Using this installation method, adhesion is reduced so that pile driving in the pleistocene sand layer can be continued longer without damaging the pile head. The jetting arrangement is shown in *Figure 50.*

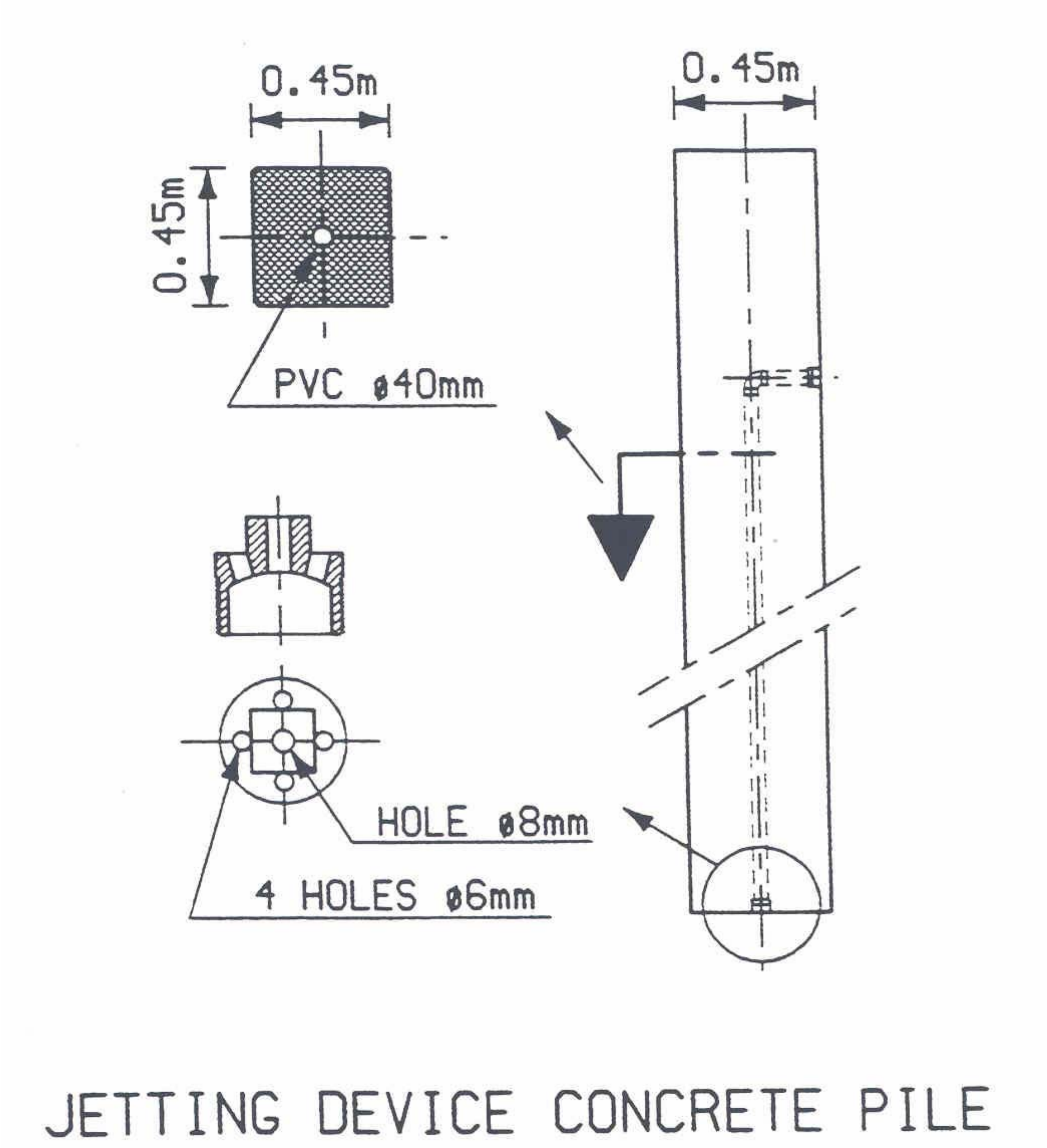


Fig. 49 Jetting device for precast concrete piles.

Consequently, it is then possible to achieve the desired penetration in the pleistocene sand layer, approximately 4 to 7 m (by driving D55 or D62 diesel hammer). An additional advantage of this installation method is that the negative skin-friction is reduced due to the waterjetting, which loosens the soil around the pile in this area and, thus, improves the bearing capacity.

Driving of concrete piles
Concrete piles must have an age of at least 8 weeks to prevent damage during driving. The stress development in concrete follows for tensile and compression stresses a different pattern as displayed in *Figure 50*.

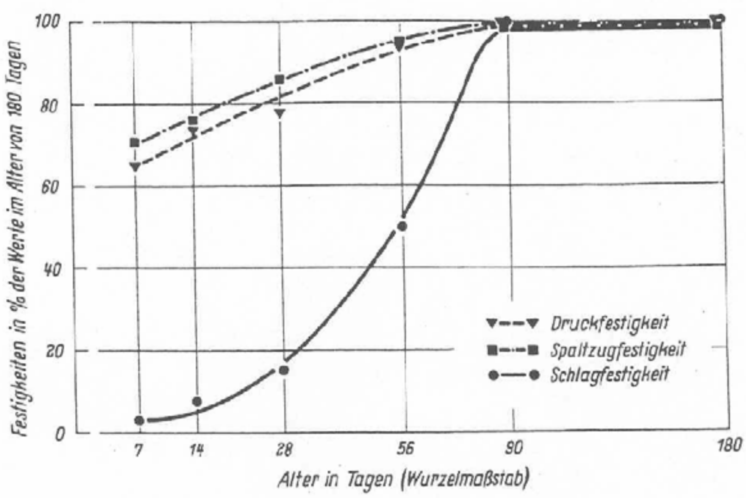


Fig. 50 Development of tensile and compressive strength of concrete

Of course this may be a little compensated by prestressing the pile. However experience shows a lot of damage on the pile driving head due to the young age of the concrete. Generally extra reinforcement is placed in the pile head to prevent pile driving damage.

6.6.3 Installation of M.V.-piles

The pile lengths vary between 30 m and 45 m.
During pile-driving, a space is formed above the enlarged tip, which is immediately filled with grout. It is essential that the grouting process is not interrupted during pile-driving and that the grout pressure is kept as high as possible by ensuring that the grout level stays at the ground surface level. The M.V.-pile is driven with an S70 or S90 hydraulic hammer. The M.V.-pile has a working load of 3,000 kN. This force is transferred to the concrete construction by means of 80 dowels of approx. 16 mm. In view of the importance of the M.V.-pile for the stability of the quay-wall, the piles are subjected to a static tensile load test. In *Figure 51* the loading frame is shown.

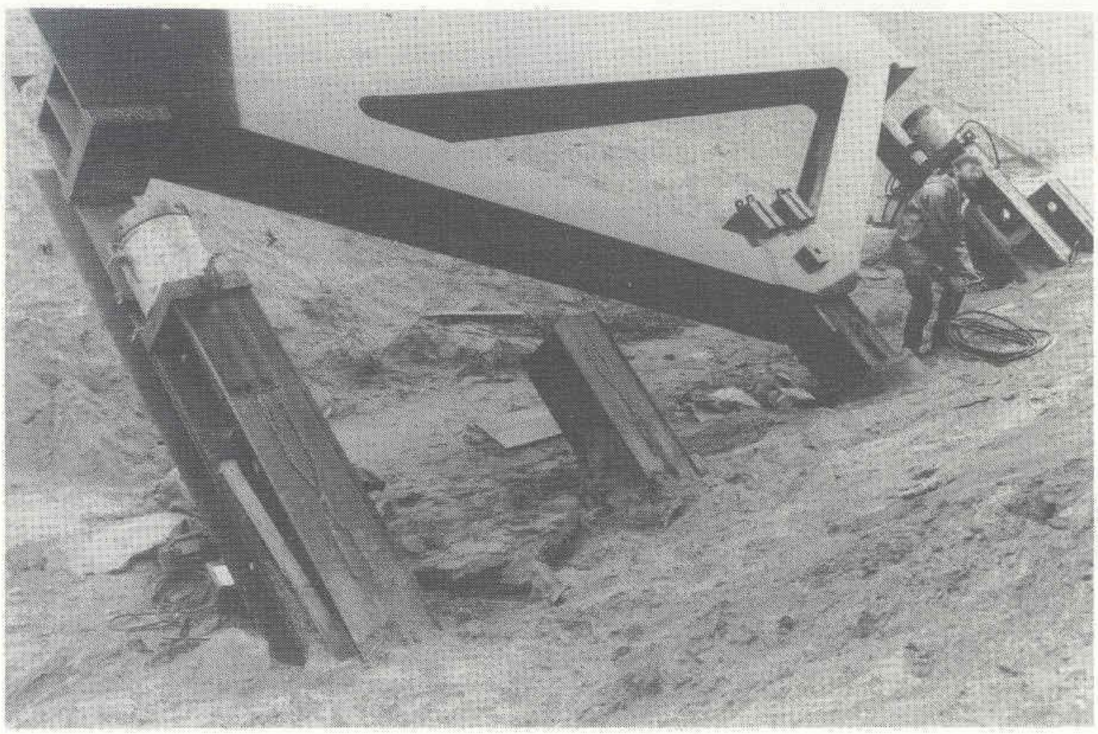
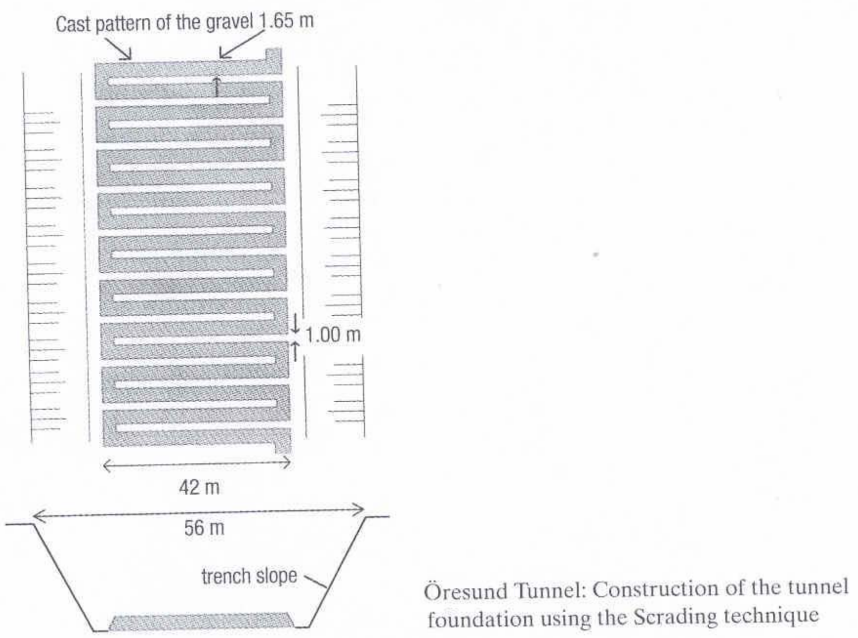


Fig. 51 Example of static load test of M.V.-pile

6.6.4 Flatness foundation bed

For gravity type quay walls it is essential that the foundation bed is flat.
In the past the control of the flatness was checked by a wooden frame like presented in *Figure 52*.



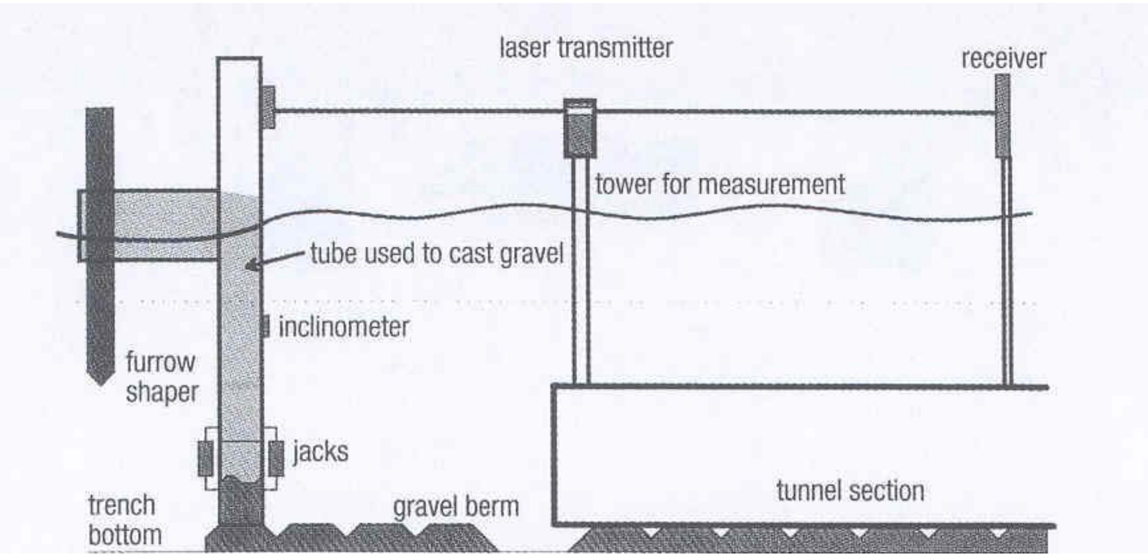


Fig.52 Assessing the required flatness of foundation bed with gravity structure.

At present however a very sophisticated system has been developed by Boskalis. That system has been used successfully with the Stϕrebeld Bridge Construction in Denmark.

Problems with blockwalls arise due to inadequate friction capacity between the blocks and overloading the bottom block.
These problems may overcome by friction connectors between the blocks.
The overstressing of the bottom block may overcome by applying reinforcement or to enlarge this block.

Dewatering aspects
Depending of the amount of required lowering of the water table and or relieving the pressure underneath the building pit bottom several dewatering techniques are available.

In *Figure 53* these several techniques are displayed in relation to the soil composition and achieving lowering of the water table.

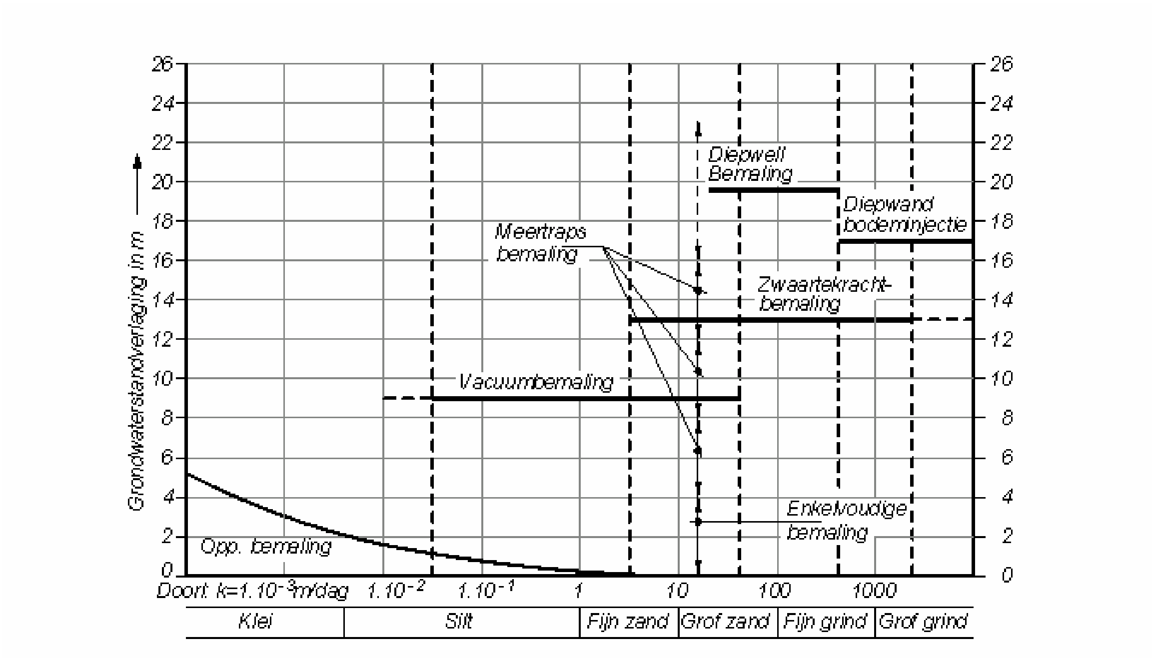


Fig. 53 Dewatering systems

Learning effect with concreting
With relatively large constructions, greet length with a considerable amount of concrete work a cut in construction time can be achieved due to the so called learning effect.
This effect is composed of intelligent support of the concrete, logistic estimation and reduction of handling time due to repetition.

An example of learning effect is displayed in figure 54.

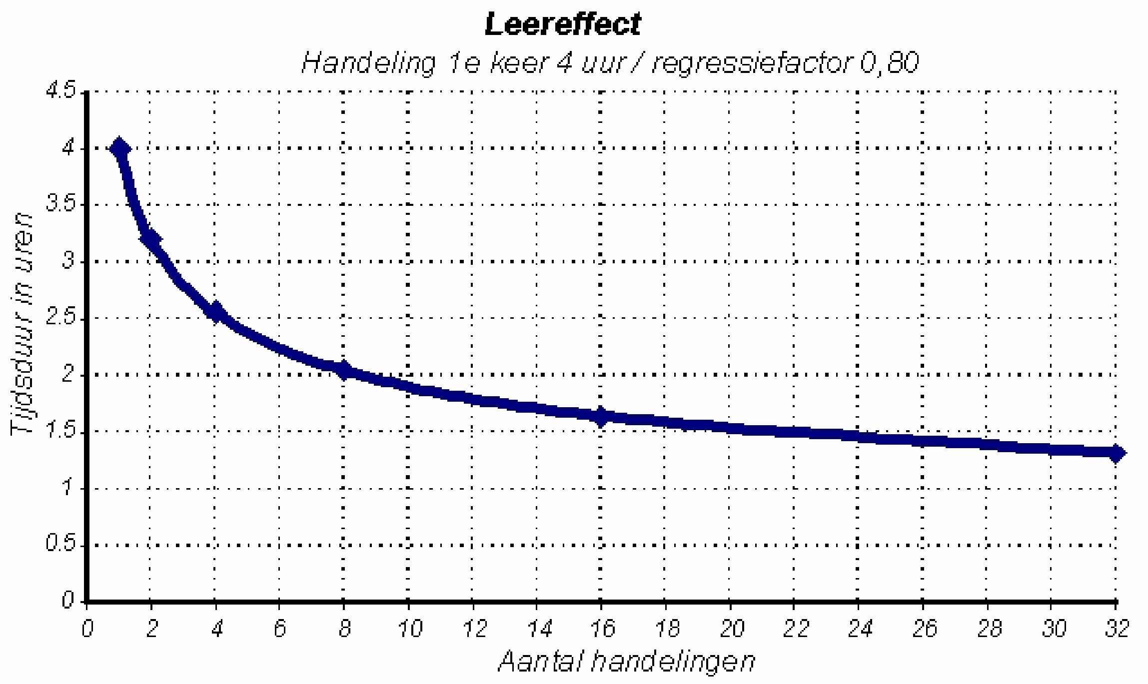


Fig. 54 Learning effect

7. Multicriteria analysis, Risk analysis and Costs

7.1 Multicriteria analysis

The multicriteria analysis is used to weigh the several designed structures on aspects like f.e.:
- construction time;
- maintenance;
- implementation of risks
- robustness, susceptibility to overloading or accidents
- durability

SELECTION OF ALTERNATIVE DESIGNS

Selection criteria	weighting factor	Design alternatives 1	2	3	4	5	6
1 Costs per m'	15	8,7	8,3	6,2	6,5	7,2	7,6
2 Durability	3	4	4	10	10	9	7
3 Maintenance	4	3	3	10	10	6	8
4 Construction aspects	6	5	5	7	6	6	6
5a Vulnerability of collision	3	3	3	9	9	6	9
5b Vulnerability of overloading	2	3	3	10	10	10	8
6 Soil tightness	2	6	6	6	6	6	6
7 Environmental/legal aspects	3	9	9	2	4	4	6
8 Deformation behaviour	1	6	7	9	9	8	6
9 Project requirements	1	5	5	10	8	1	8
10 Scour aspects	1	2	2	4	4	4	4
11 Multi-functionality	2	3	3	5	5	5	5
12 Construction time	2	7	7	2	4	4	6
Total score	45	6,0	5,9	6,8	7,0	6,4	7,0

Rating : 0 - 10, very poor - excellent
Costs : [(Pmax-P) / (Pmax-Pmin)]×10
Total score : (sum of numbers × weighting factor) / 45
Pmax = maximum construction costs
Pmin = minimum construction costs
P = costs of alternative

Fig. 55 A multicriteria matrix is presented to compare the different alternatives.

7.2 Risk analysis

In modern design a disk analysis is carried both for technical aspects as well as other aspects such as risk acceptation client, contractor, engineer, financial risks.
It is very important to do such a risk analysis to avoid later misunderstanding, bad communication and discussions about acceptability and money.

7.3 Costs

The selection for a structure to be built is not only based on technical aspects but also the costs are important.
The costs items are:
- building costs
- maintenance costs
- demolition costs.

For the determinations of the building costs the so called CROW approach is adopted. Depending in the phase of the project the uncertainty and margins must be selected.

The maintenance costs are determined based upon experience progress although is being made to access these costs in the initial phase. The same is valid for including the demolishing costs. By considering the afore mentioned costs the so called life cycle approach can be used.

As first the following indicative for quay wall costs per metre can be used.

Retaining height/m[1]	Costs retaining height/m
5 - 10	300 - 600
10 - 20	600 - 900
20 - 30	900 - 1.200

The costs of piers vary between € 750,00 - 1.250,00/m². These costs are exclusive fendering and scour protection measures.

Further a comparison made of different constructing forms a concrete support system together with the man hour shows dramatic decrease in costs.

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

8. Fender Design

<u>Introduction</u>
The deterministic method is the oldest and so far most commonly used method for fender design. The method is described in this section. The designer should carefully consider whether this method is indeed suitable for the specific situation. Especially in cases where external forces may have an impact on the berthing energies, more sophisticated methods may be required.

The following section concerns a vessel in the process of berthing.

<u>Energy equation for a vessel in the process of berthing</u>
The kinetic energy of a moving vessel may be calculated as:

$$E = \frac{1}{2} * M * v^2$$

where:

E	= kinetic energy of the vessel itself (in kNm)
M	= mass of the vessel (= water displacement) (in tonnes)
v	= speed of the approaching vessel perpendicular to the berth (in m/s).

The design energy that has to be absorbed by the fender can be calculated as:

$$Ed = \frac{1}{2} \underline{M} * v^2 * Ce * Cm * Cs * Cc$$

where:

Ed	= design energy (under normal conditions) to be absorbed by fender system (in kNm)
<u>M</u>	= mass of design vessel (displacement in tonnes), at chosen confidence level usually 95% confidence level (Refer to Appendix C for values)
V	= approach velocity of the vessel perpendicular to the berth (in m/s) (use 50% confidence level)
Ce	= eccentricity factor
Cm	= virtual mass factor
Cs	= softness factor
Cc	= berth configuration factor or cushion factor.

Based on the manufacturer's performance curve for a selected fender, a fender reaction force can be defined for the calculated kinetic energy of the vessel. This force is a characteristic load, which should be used as specified in the code used for design of the quay structure. Berthing mode may affect the choice of vessel approach speed and the safety factor for abnormal conditions.

Mass of the design vessel (M)
Generally the size of cargo carrying vessels is expressed in Dead Weight Tonnage (DWT).

The size of passenger vessels, cruise vessels or car ferries is generally expressed in Gross Registered Tonnage (GRT).

DWT is the cargo carrying capacity of a vessel including bunkers (fuel, water, etc.).

GRT is the internal capacity of a vessel measured in 100 ft^3 (100 ft^3 = 2.83 m^3).

For the energy calculation the displacement of a vessel is required. The displacement tonnage (M) of a vessel is the total mass of the vessel and can be calculated from the volume of water displaced multiplied by the water density. In most case the vessel's fully loaded displacement is used in the fender design.

Approach velocity (v)
The approach velocity v is the most influential variable in the calculation of the berthing energy. The approach velocity is defined as the vessel speed at initial berthing contact, measured perpendicular to the berth.

The actual approach velocity is influenced by a large number of factors such as:
- prevailing physical boundary conditions: the influence of waves, wind and current should be considered;
- ease of navigation: is the approach to the berth easy or difficult?
- method of berthing: are berthing aids used, is berthing always parallel, when is the forward motion of the vessel stopped, etc.;
- type of vessel: is the vessel equipped with powerful engines, quick reacting engines, bow thrusters, etc.;
- use of tugs: are tug boats used, how many and of sufficient capacity;
- frequency of berthing: at berths with a high berthing frequency, generally higher berthing velocities are experienced;
- size of vessel: the approach velocity of larger vessels is generally less than the approach velocity of smaller vessels; range of vessels expected at the berth must be considered;
- berth appearance: ship masters will berth more careful when approaching a desolate berth instead of a new, modern berth;
- type of cargo: a vessel with hazardous cargo will generally berth under better controlled circumstances, the use of berthing aids for example;
- windage area of the vessel: a vessel with a large windage area is considerably more susceptible to wind;
- human factor: a most important factor, this may concern the level of experience, etc.

Designers must consider that the design values for the approach velocity should be close to the expected actual berthing speeds. It is the task of the designer to obtain data on the local conditions and seek out vessel operators, port engineers, ship owners, etc. in order to gain insight into the applicable conditions and to decide on he most likely and/or appropriate approach velocity.

The British Standard on Fenders (BS 6349 part 4) has adopted the design approach velocity as recommended by Brolsma et al. in 1977 (see Figure 4.2.1). In line with Baker (1953) Brolsma distinguishes five navigation conditions but does not elaborate on those conditions except that all vessels berthed with tug assistance. However, to date no more pertinent or accurate data has been found.

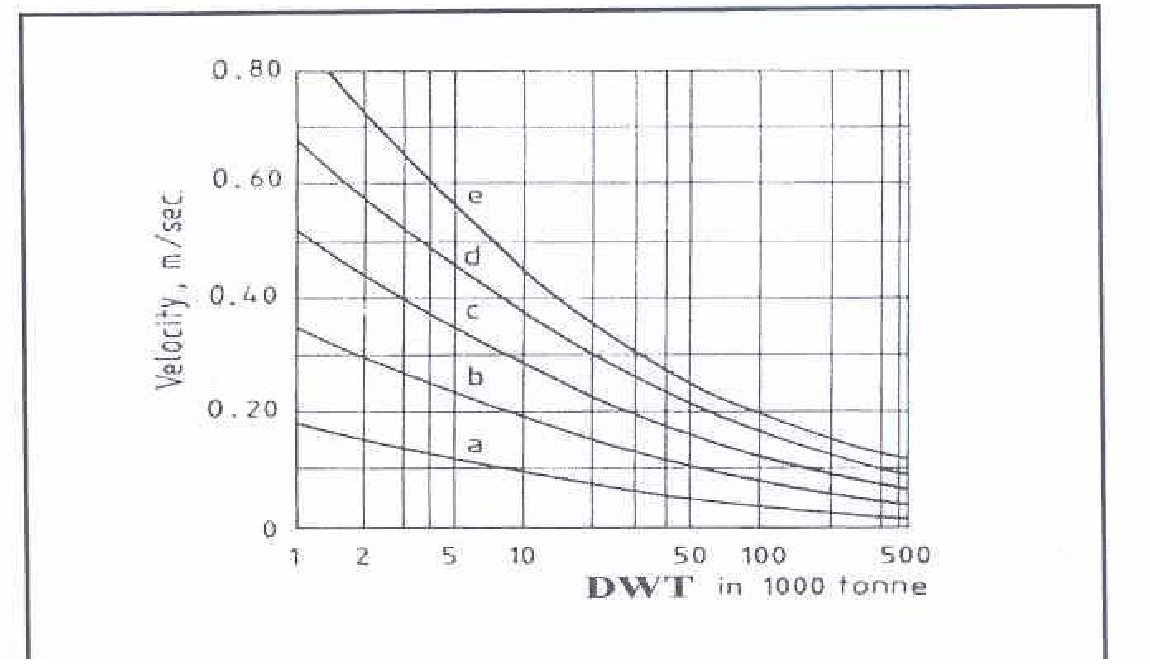


Fig. 56 Design berthing velocity (mean value) as function of navigation conditions and size of vessel (Brolsma et al. 1977)

a. Good berthing conditions, sheltered
b. Difficult berthing conditions, sheltered
c. Easy berthing conditions, exposed
d.* Good berthing conditions, exposed
e.* Navigation conditions difficult, exposed

* These figures should be used with caution as they are considered to be too high.

Mean value is taken to be equivalent to the 50% confidence level.

For the majority of cases it is considered sufficiently accurate to distinguish the above conditions. It is assumed that the environmental conditions are closely related to the degree of exposure of the berth (exposed, partly exposed or sheltered). In absence of more accurate figures, the following practical values may be adopted for the approach velocity v (in m/s).

Suggested Approach Velocity (Mean Values) m/s (Taken from the Spanish ROM Standard 0.2-90)			
Vessel displacement in tonnes	**Favourable conditions**	**Moderate conditions**	**Unfavourable conditions**
Under 10,000	0.2 - 0.16	0.45 - 0.30	0.6 - 0.40
10,000 - 50,000	0.12 - 0.8	0.3 - 0.15	0.45 - 0.22
50,000 - 100,000	0.08	0.15	0.20
over 100,000	0.08	0.15	0.20

Mean value is taken to be equivalent to the 50% confidence level. The figures given above are indicative, with tug assistance. The full graphs are set out in the ROM standard.

In case the berthing manoeuvre takes place without tug boat assistance, the above figures will be increased considerably.

Special attention is to be paid to berths used by smaller vessel, e.g. a tug boat jetty, as these smaller vessels tend to berth at relatively high speeds.

In recent decades more and more berths, especially tanker/chemical berths, have been equipped with berth approach detection systems. Information from these systems, if available, may be used to establish design approach velocities for specific facilities.

<u>Eccentricity factor (Ce)</u>
For the eccentricity factor two different scenarios have to be distinguished:
- a berth with continuous fendering;
- a berth with breasting dolphins (or island berth).

An important role in the determination of this factor is the berthing angle.

The berthing angle is also of importance for the determination of the reduction in energy absorption capacity of fenders, as a result of angular compression resulting in non-uniform deflections.

<u>Berthing angle</u>
Measurements in Japan have shown that for vessels larger than 50,000 DWT the berthing angles are generally less than 5 degrees with only occasionally an angle of 6 degrees. It is therefore suggested that 6 degrees must be used as a maximum approach angle for these vessels.

For smaller vessels, and especially for vessels which berth without tug boat assistance, the berthing angle may be larger, say 10 - 15 degrees (e.g. feeders/coasters 8 -10 degrees and barges 15 degrees).

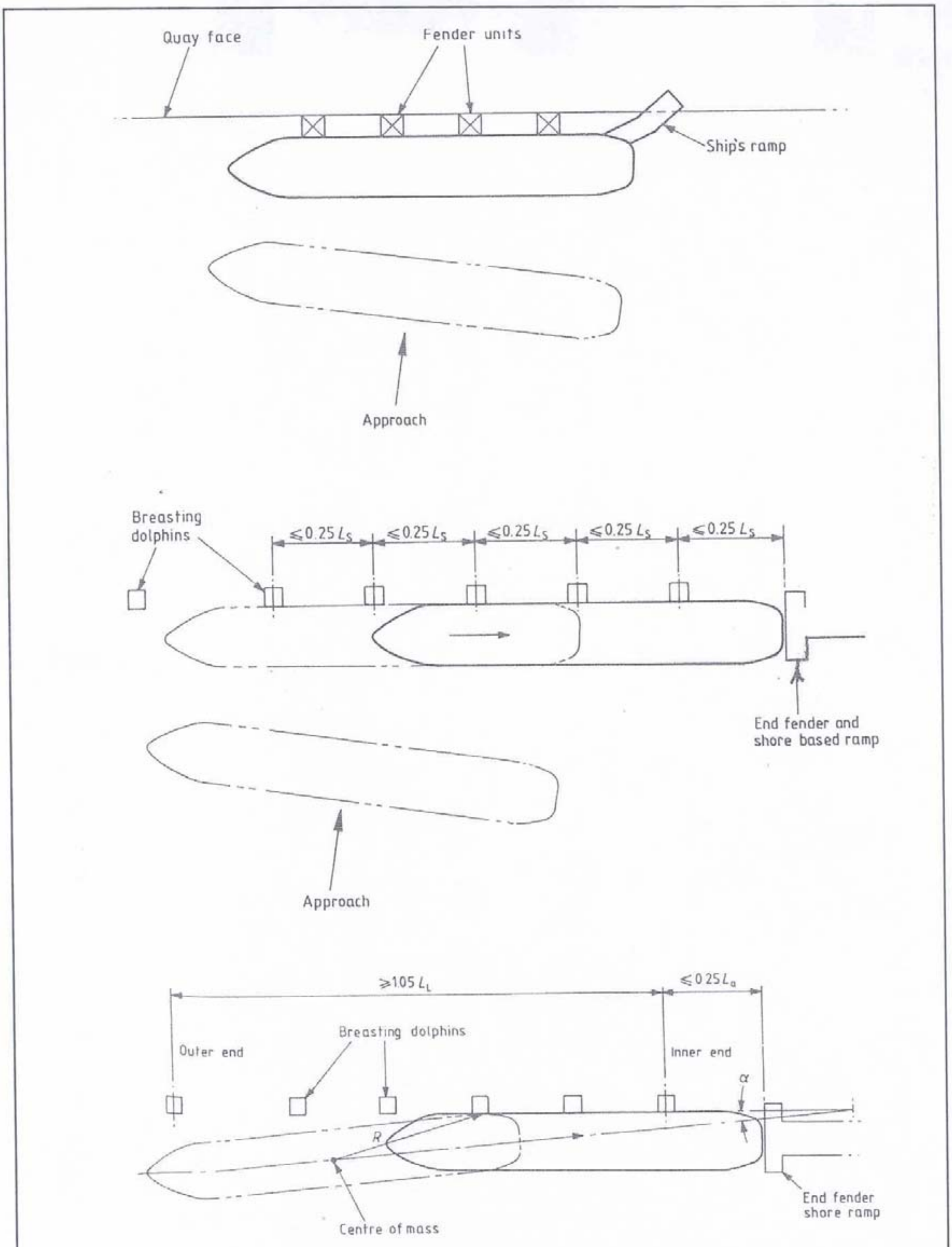


Figure 6.2.1 Ferry and Ro/Ro Vessel Berthing

Fig. 57 Berthing Model

<u>Eccentricity factor Ce</u>

The eccentricity factor can be calculated with the following formula:

$$C_e = \frac{K^2 + K^2 * \cos^2 \varphi}{K^2 + R^2}$$

or simplified, assuming ϕ is 90 degrees:

$$C_e = \frac{K^2}{K^2 + R^2}$$

K = radius of gyration of the vessel (depending on block coefficient, see below) (in m)
R = distance of point of contact to the centre of the mass (measured parallel to the wharf) (in m)
ϕ = angle between velocity vector and the line between the point of contact and the centre of mass.

$$K = (0.19C_b + 0.11) * L \text{ and } \mathrm{C_b} = \frac{M}{L * B * D * \rho}$$

where:
C_b = block coefficient (usually between 0.5-0.9, see below)
M = mass of the vessel (displacement in tonnes)
L = length of vessel (in m)
B = breadth of vessel (in m)
D = draft of vessel (in m)
ρ = density of water (about 1.025 ton/m^3 for sea water)

Lacking other data, the following may be adopted for the block coefficient:

Block coefficient	
For container vessels:	0.6 - 0.8
For general cargo vessels and bulk carriers:	0.72 - 0.85
For tankers:	0.85
For ferries:	0.55 - 0.65
For Ro/Ro-vessels:	0.7 - 0.8

For large tankers, K can be taken as approximately 0.25 L.

In the case where there is no accurate data or, in case only a quick assessment is made, the following numbers may be used:

<u>for a continuous berth</u>:
quarter point berthing, the berthing point of the vessel is some 25% of the vessels length from the bow: C_e = 0.5

<u>for a berthing dolphin</u>:
the berthing point of the vessel is some 35% of the vessels length from the bow: C_e = 0.7.

For Ro/Ro-vessels the C_e factor is taken as 1.0 for the end fenders.

Virtual mass factor (Cm)

For the virtual mass factor (also referred to as ‘added mass factor’ or ‘hydrodynamic mass factor’) several formulae are in use (Stelson, Malvis, Ueda, B.F. Saurin, Rupert, Grim, Vasco Costa, Giraudet) and much research work has been done:

Cm is generally defined as:

$$C_m \frac{M + M_v}{M}$$

M = mass of the vessel (displacement in tonnes)
M_v = virtual mass (in tonnes)

Some of the formulae used to obtain input for the calculation of Cm are given below.

Shigeru Ueda

$$M_v = \rho L D^2 * \frac{\pi}{2}$$

where:
ρ = density of water (about 1.025 ton/m^3 for sea water)
L = length of vessel (in m)
D = draft of vessel as used for calculation of mass of design vessel (in m)

The formula of Shigeru Ueda originates from 1981 and is based on model experiments and field observations.

The formula can be transformed into:

$$C_m = 1 + \frac{\pi * D}{2C_b * B}$$

where:
C_b = block coefficient (see sub section 4.2.4)
B = breadth of the vessel (in m)

Vasco Costa

$$C_m = 1 + \frac{2D}{B}$$

This formula was published in 1964 and is also used by the British Standards Institute. It is valid under the following circumstances:
1) the keel clearance shall be more than 0.1 * D; and
2) the vessels velocity shall be more than 0.08 m/s.

Unless the designer has good reasons to apply other values, it is recommended:

for very large keel clearances ie (0.5 * D): then use Cm = 1.5
for small keel clearances ie (0.1 * D): then use Cm = 1.8
for keel clearances in between 0.1 * D and 0.5 * D: use linear interpolation.

The above Cm values are valid for transverse approaches. A Cm value of 1.1 is recommended for longitudinal approaches.

Softness factor (Cs)
This factor is determined by the ratio between the elasticity of the fender system and that of the vessel's hull. Part of the kinetic energy of the berthing vessel will be absorbed by elastic deformation of the vessel's hull. Cs expresses the kinetic energy portion of the berthing vessel onto the fender.

The following values are often used:
- for soft fenders and for smaller vessels Cs is generally taken as 1.0;
- for hard fenders and larger vessels Cs lies between 0.9 and 1.0 (e.g. for VLCC Cs = 0.9).

The British Standard Code of Practice for Maritime Structures (BS 6349) suggests in Part 4 on the 'Design of fendering and mooring systems' that a hard fendering system can be considered as one where the deflections of the fenders under impact from ships for which the fenders are designed, are less than 0.15 m.

In most cases the contribution of the vessel's hull to the energy absorption is only limited. It can therefore be concluded that there appears little merit in maintaining the distinction between soft and hard fenders. This results in a general value of Cs = 1.0.

Hull Pressure Guide

Type of vessel	**Hull Pressure kN/m²**
Container vessels 1st and 2nd generation	<400
3rd generation (Panamax)	<300
4th generation	<250
5th and 6th generation (Superpost Panamax)	<200
General cargo vessels	
=/< 20,000 DWT	400-700
> 20,000 DWT 40	<400
Oil tankers	
=/< 60,000 DWT	<300
> 60,000 DWT	<350
VLCC	150-200
Gas carriers (LNG/LPG)	<200
Bulk carriers	<200
SWATH	)
Ro-Ro vessels	) these vessels are usually belted
Passenger vessels	)

It should be noted that ships with belting produce a line load on the fenders and can be considerably higher than the hull pressure quoted below.

<u>Hull structure</u>
The ships hull structure is generally comprised of three components:
1. side plating, thickness 15-20 mm;
2. longitudinal stiffeners, mostly spaced at approximately 0.86 m - 0.90 m;
3. transverse frames.

The dimensions of all three components may vary with type and age of ship and shipbuilder.
New tendencies are:
1. the use of steel with higher strength;
2. increasing of distance between transverse frames, e.g. 6.28 m for 5th and 6th generation container vessels and 3.14 m for earlier generation vessels;
3. berthing energies are increasing and allowable hull pressures decreasing.

Where hull pressures may be critical, naval architect or vessel owners should be consulted for specific requirements.

These figures include the factors of safety normally used by Classification Societies.

However, if the side plating, longitudinal stiffeners and side traverses data is given, the permissible hull pressure can be calculated.

For large vessels as a rule of thumb the permissible pressure on hull impact is at least equal to the maximum hydrostatic pressure (vessel fully laden/at maximum draft) which can act on the vessels hull.

Special attention should be paid to the positions of the horizontal chains on a fender panel. When chains are installed below the fender, the rotation of the fender panel, due to the vessel’s flare, can be restricted. Line loads may occur which exceed the permissible hull pressure.

Fendering aspects container ships and barges
This section addresses the fendering requirements for vessels which are dedicated to the transportation of containerised cargo. Containerisation of cargo is generally based on 20 foot equivalent unit (TEU) of forty foot equivalent unit (FEU) formats. Containers are manufactured in a range of types which include dry box, refrigerated freight and liquids in tanks.

Container vessels range in size from small feeder vessels which may carry 70 TEU, or less, up to vessels which at present carry in excess of 6,000 TEU. Large container vessels may soon exceed 8,000 or even 12,000 TEU.

Many small vessels have self loading and unloading capability or Ro-Ro capacity. Larger vessels are restricted to berths in terminals where cranage has sufficient outreach.
Dedicated refrigerated cargo vessels, ‘Reefers’, often have container capacity and may be geared for self loading and unloading. Many vessels are capable of carrying a mix of refrigerated and non-refrigerated containers. Containers are stored in the holds of vessels and on hatch covers at deck level. Some classes of vessels are hatchless and many use cellular guides. Combination vessels are capable of transporting a mix of containers and bulk or break bulk cargoes. Typically these vessels are fitted with vessel’s gear and are not totally dependent on quayside cargo handling equipment to load and offload containers. The particular problem associated with these vessels is the need to keep fender panels below the quay level to permit landing of the quarter ramps which are fitted to some vessels.

Container barges are configured to transport a range of containerised cargoes. The vessels are used either for transhipment from the vessel to shore or for shipment on rivers or canals. Therefore these barges come in a broad range of sizes, lengths and freeboards. Not all barges can be assumed to be fitted with belting fenders. Generally barges are unlikely to be fitted with their own cargo handling equipment. However, there are local variations where transhipment barges do have carnage.

Particular aspects to be considered

Fendering for container vessels has to consider the following specific issues.

Increased efficiency in cargo handling has significantly reduced the time a vessel spends alongside a berth. Reduced turn around times translate into a higher frequency of berthing.

Berths at major terminals will be expected to operate year round in a range of weather conditions.

Large container vessels rely upon shore cargo handling equipment therefore the crane outreach is of critical importance. In general, the horizontal distance from wharf face to fender face should be kept to a practicable minimum, in order to reduce the required crane outreach. There must be sufficient clearance to reduce the chance of the flare of the vessel hitting a crane leg at the edge of the quay, see figure.

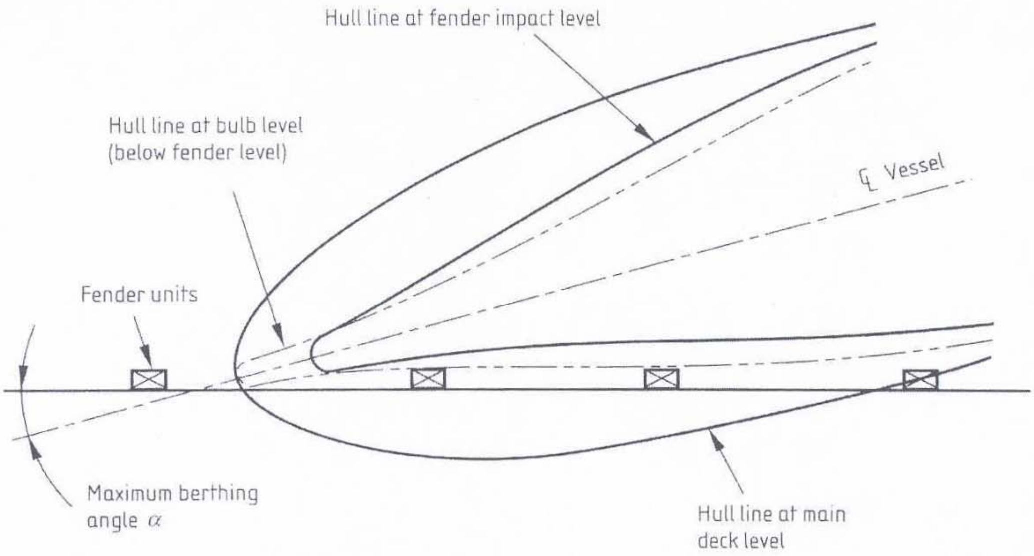


Figure 2.3.3 GEOMETRY OF VESSEL WITH BULBOUS BOW

Fig. 58 Flare of containership

Container vessel hulls, like most other vessels, are not designed to take high external loads from fenders.
Consideration must always be given to the spreading of fendering system loads evenly along the side of the vessel. Certain classes of vessels are constructed with large amounts of high tensile steel. In the more structurally sensitive areas of these vessels damage will generally command a premium cost to repair.

Container berths and their fendering systems will generally be required to cater for a wide range in vessel sizes and configurations, including barges. The types of port will generally dictate the type of service and therefore the vessels which will berth.

Rolling motion of a vessel, especially during self unloading, can be reduced by selecting a high friction fender.

The prevention of catching or hanging up and consequential damage of vessel sides and structures with the lower or higher edges of the fendering system must be considered. The topside flare, both fore and aft, of all modern seagoing container vessels is considerable, but as each vessel or class of vessel is unique, guidance can not be given on the expected flare.

<u>Design of fenders for container vessels</u>

Fender spacing and layout
Fendering systems should be designed to spread the berth loads evenly along as much of the vessel side as is possible. Many vessels have considerable amounts of topside flare forward and aft, below main deck level. In the bow the amount of curvature in the main deck, in plan view, will also vary. As a result the parallel or flat areas of the side of a vessel may be reduced by as much as one third, or more, of its overall length.

Tidal range and the shape of the vessel will dictate the vertical dimension of the fendering.
The lower edge of the fendering will need to be positioned so as to prevent the possibility of the fendering catching on low freeboard vessels at low states of the tide. The upper edge of the fendering will need to be configured so as to prevent or accommodate contact being made with vessels with considerable amounts of flare.

Certain lengths of wharf edge may be required, additionally to permit use of quarter or vessel side ramps. It may be necessary to make provision to enable certain sections of fendering to be removed or substituted for different types of vessels or means of cargo handling at container quays, even though the declared traffic is initially containers.

Fendering aspects Ro-Ro vessels including ferries

This section considers all roll on/roll off vessels carrying freight, either unaccompanied, that is on road, rail or vessels' trailers or cassettes without tractor units or drivers, or accompanied, this is with individual tractor units and road trailers with drivers and trade cars. The criteria for freight vessels and passenger ferries have considerable overlap with the distinction that passenger ferries may have more frequent berthings and faster turnarounds. Passenger ferries will generally have a greater number of openings such as windows and doors as well as protuberances such as lifeboats for which due consideration must be given.

Ro-Ro vessels can be divided into two main categories as follows:

a) Vessels with bow and/or stern ramps which require a shore ramp structure at the bow or stern.
b) Vessels with side or quarter ramps which can be landed on the quay. Side and quarter ramps are particularly appropriate where the tidal range is small and a fixed conventional quay is suitable for landing ramps. Many vessels however, have the capability to cope with a range of levels, for example, by having long quarter ramps or side ramps which can operate off different decks within the vessel. Where the tidal range is large, it may be necessary to provide a floating pontoon to land the vessel's ramp.

<u>Particular aspects to be considered</u>
The fendering for Ro-Ro vessels when compared with other vessels has to take into account the following factors:

a) Ro-Ro vessels usually require a short turn around time in port - consequently the vessels are more likely to berth at higher approach speeds. The tight schedules that Ro-Ro vessels usually operate often result in them having to berth in unfavourable weather conditions. Additionally, berths are likely to be subject to more berthings per year and delays due to damage to the fendering system are less acceptable when compared to other cargo berths. It is particularly important that fendering systems for Ro-Ro berths are designed to be robust and easily maintained.

b) Ro-Ro vessels are usually fitted with a belting strip, or multiple belting strips, which projects from the hull. This belting is usually located at the level of the main trailer deck and is typically 250 mm high and 300 mm wide. This results in the vessels applying a line load to the fenders.

   To overcome this, the fendering usually is provided with a suitably stiff facing panel. Such facing panels may result in double contact between the vessel and the fender with the second contact either at the top or bottom of the panel depending on the level of contact. This needs to be checked and if it is considered unacceptable then one of the following may be required:
   - a long lever arm pivot fender for example using a fender pile to ensure that the fender face does not tilt excessively;
   - parallel movement fender system for example based on torsion bar.

It is important to ensure that the facing panel extends sufficiently far vertically so that the belting cannot ride over the top or get caught underneath the panel whilst the vessel is on the berth. The panel should be designed to cater for a tidal range plus an allowance for weather variations and for operational variations in vessel draught and trim during loading/unloading.

In order to satisfy this requirement, the fender panels often have to extend above the quay level. The effect that this may have on the mooring lines should be checked to ensure that the vessel can be moored safely. This can cause problems where vessels with side or quarter ramps are to be accommodated and where the berth is also used for other cargo vessels. In these cases, it may be not acceptable to extend the fender panel above the quay in which case the top of the fender panel should be sloped to prevent the belting sitting on the fender panel. This results in significant downward vertical forces on the fender which should be allowed for in the design.

c) Ro-Ro vessels often have a large windage area relative to their displacement. This combined with the requirement to operate in all weather conditions increases the likelihood of a heavy berthing.

   It should be noted however, that Ro-Ro vessels are usually very manoeuvrable with bow or stern thrusters and/or other similar equipment fitted.

<u>Ship berthing manoeuvres</u>

Transverse ship approach to the berth
Berthing alongside transverse to the berth may be considered in the following cases.

a) For vessels with side and quarter ramps.
b) For vessels with bow and/or stern ramps, when the Ro-Ro vessels make a transverse approach to the berth. The vessels then move along the quay often under mooring winch control, using the side fenders for guidance until they are the appropriate distance from the shore ramp structure.

   For new dedicated Ro-Ro facilities the berth will usually have independent breasting fenders with no continuous quay. However, where the berth caters for both Ro-Ro and conventional cargo or uses an existing berth, there may be a continuous quay.

Head on vessel approach to the berth
Berthing head on is normally only practised by regularly scheduled Ro-Ro ferries on shorter routes. For this berthing manoeuvre, side breasting dolphins are provided as a guide to the vessel but the vessel berths directly against the shore ramp structure or a separated end fender.

In principle, this method of berthing is not preferred as any berthing accident may damage the shore structure at the most critical and complex part of the Ro-Ro shore facilities.

One version of nesting fenders is where fenders guide the vessel into and hold the vessel in the correct transverse location. This system is generally only used on short scheduled Ro-Ro ferry routes with dedicated vessels. The fendering would be designed to suit a particular vessel and if an alternative vessel is used, it may be necessary to modify the vessel so as to present the same profile to the fendering. Large transoms may have to be allowed for in the design of any side fendering.

Design of fendering

Side fenders
Side fenders should be designed taking into account the higher berthing speeds and frequency of use.

The fender spacing should be assessed taking into account the vessel characteristics and berthing procedure but should not normally be greater than 0.25L, where L is the length of the smallest vessel using the berth. Where a continuous quay is provided, the spacing will usually be closer than this with similar requirements for checking bow radius, flare and quay overhang as for other vessels.

Attention is drawn to the effect of the belting on the height of the fender panel, and if the panel has to extend above he quay level, the effect on mooring lines and side and quarter ramps will need to be checked.

The lead in fender should usually be designed for a midpoint berthing. At some berths this fender may also need to allow for vessels to be turned.

In ice conditions, ferries often have to berth by sliding along the berth and leaning against the fender panel. This is done to push ice blocks off the berth, which may cause extra transverse loads to fenders.

The method of departing from the berth should also be considered. Ro-Ro vessels often leave the berth by moving along the berth on the main engines while using a bow thruster to move the bow out and then depart. This may require the fender adjacent to the shore ramp to be designed for extra forces from the vessel during turning.

For modern vessels (last generation Ro-Ro), it is necessary to consider the flare angle during the fender selection process. The hull geometry, over the impact area should be considered in both horizontal and vertical planes.

When determining the eccentricity factor (Ce), account should be taken that the values of the block coefficient may be lower for Ro-Ro vessels (Cb = 0.7 - 0.8) than for normal cargo vessels. It should be noted that the centre of gravity of Ro-Ro vessels does not lie in the centre of the vessel length, but is further towards the stern.

End fenders
In some instances, end fenders are provided not only where vessels berth end on but also where vessels berth transversely, usually to prevent the vessel accidentally striking the shore ramp. When this should be provided is a matter of judgement on how far the vessel has to moor from the shore end ramp (this depends on the length of the vessel's ramp), the wind, wave and current conditions at the berth, the manoeuvrability of the vessels and any manoeuvring restrictions at the berth.

Operational expertise of the vessels should be considered.

Some older Ro-Ro vessels have blunt ended ramps without finger flaps. The ramp is landed into a recess in the shore ramp. In these circumstances, end fendering has to be provided to ensure that excessive forces are not applied to the shore ramp from the blunt end of the vessel's ramp.

End fenders can be installed in one of three positions as follows:

1. At the vessels end of the shore ramp.
   In this case the vessel strikes the fender directly and the force is usually transmitted to the abutment via the shore ramp. The large reaction force needs to be taken into account in the design and maintenance of the hinge bearings.

2. At the shore end of the shore ramp.
   A fender is installed between the shore ramp and its abutment. The vessel strikes the shore ramp which transmits the impact into the fender. The considerable horizontal movement of the shore ramp needs to be taken into account in the design and construction of the ramp supporting system and as a result this arrangement tends to be more suitable for buoyant or semi-buoyant shore ramps rather than lift systems. It can however be used for the latter.

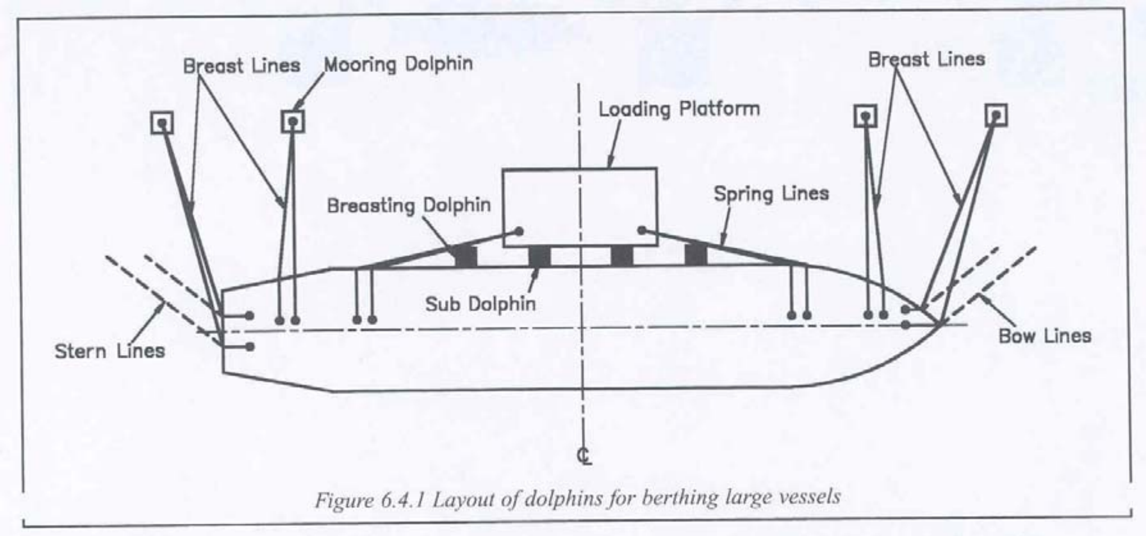


Figure 59 Layout of dolphins for berthing large vessels

In case the vessel length is smaller than the nominated berth, sub-dolphins should be positioned if necessary. Figure 63 shows the general layout for each dolphin. Bow and stern lines will effect the vessel motion when alongside the berth.

As the tension in the mooring lines effect the compression of the various fenders, this will have a marked effect on the fendering system, as a whole.

<u>Fender system for the large vessel</u>
Fender systems should be determined considering not only the absorption of the external load by the vessels berthing and mooring, but also the hull pressure of the vessel and the reaction load to the structure to ensure protection of them both.

It is general for piled dolphins with vertical piles that the energy absorption by deformation of fender and dolphin is taken into consideration. On the other hand, for the dolphin with coupled, battered piles, only the fender is to absorb the energy, because the reaction load to deform the dolphin is so high that it might damage the vessel's hull and possibly the dolphin structure.

Fenders should be selected taking into consideration the characteristics under angular compression as the large vessel usually berth with angle. It should be also considered that all fenders in the face line should not work under such an angular approaching condition.

9. Scour in front of quay walls

Introduction

The beds of rivers, canals, seas can be eroded by the action of tides, river flow, the wash from jet propellers, etc.
A combination of these factors can also be a decisive factor in designing protection for these beds.

The extent of erosion (deepening, amount) depends on current velocity, turbulence, how long scouring has been taking place, resistance of the bottom to scouring and the supply of new sediment to the scour hole.

Bottom scour is occurring at various locations in the harbours of Rotterdam and in parts of the Hartel Canal. Current velocities of about 2.5 m per second may occur here.

Bottom scour occurs often near permanent berths, which are mainly the points where Ro-Ro vessels tie up. These vessels usually propel themselves to and from their berths of need minimum assistance from tugs.
Occasionally bottom scour occurs where oil and gas tankers tie up. The consequences of bottom scour may include:

a. a loss of stability in banks and dams;
b. the collapse of retaining walls and quay walls;
c. soil fluxions behind and beneath retaining walls and caisson quay walls;
d. sand flowing away as a result of undetected openings in the interlock system of retaining walls;
e. the collapse of dolphins because they are not firmly embedded in the harbour bottom;
f. damage caused by ships' anchors to pipelines and cables beneath the harbour bed, due to inadequate cover.

In port engineering in the vicinity of the ships' berths, the action of the ships' screws is a prime eroding element, with speeds of up to 4 to 8 m/s possible near the harbour bottom. This contrasts with the current speeds of natural river or drift currents and backflow currents caused by the ships of only 1 to 2 m/s.

Scour Caused by Ships

Scour Caused by Jet Formation by the Stern Screw

The jet velocity caused by the rotating screw, so-called induced jet speed (occurs directly behind the screw), can be calculated with:

$$V_0 = 1.6 \, . \, \eta \, . \, D \, . \sqrt{k_T} \tag{1.1}$$

η = speed of the screw [l/s]
D = diameter of the propeller [m]
k_T = thrust coefficient of the screw, k_T = 0.25 … 0.50.

The simplified formula for a mean value of the thrust coefficient is:

$$V_0 = 0{,}95 \, . \, \eta \, . \, D. \tag{1.2}$$

If the output of the screw *P* is known instead of the speed, the induced jet speed can be calculated according to the following assumption:

$$V_o = C_p \left[\frac{p}{\rho_0 \, . \, D^2} \right]^{1/3} \tag{1.3}$$

P = screw output [kW]
ρ_0 = density of the water [t/m³]
C_p = 1.48 for frees crew (without nozzle) [1]
= 1.17 for screw in a nozzle [1].

As it progresses further, the jet expands cone-shaped from the turbulent exchange and mixing processes loses speed with increasing length, the maximum speed occurring near the bottom which is essentially responsible for scouring, can be calculated as follows:

$$\frac{\max V_{bottom}}{V_0} = E \, . \left(\frac{h_p}{D} \right)^a \tag{2.1}$$

Integration of equation (1.2) then produces:

$$\max V_{bottom} = 0{,}95\,.\,\eta\,.\,D\,.\,\mathrm{E}\left(\frac{h_p}{D}\right)^{a} \tag{2.2}$$

E = 0,71 for single-screw vessels with central rudder [170]
= 0,42 for single-screw vessels without central rudder
= 0,42 for twin-screw vessels with middle rudder [170]
= 0,52 for twin-screw vessels with twin rudders located after the screws
a = -1,00 for single-screw vessels
= -0,28 for twin-screw vessels
h_p = height of the screw shaft over bottom [m]
= $z + (h - T)$
z = $\left(\frac{D}{2}\right) + 0.10\,...\,0.15$
h = water depth [m]
T = draft.

The speed of the screw which is relevant to water jet velocity depends on the power plant output used for berthing and departing. Practical experience has shown that this machine output for port manoeuvres lies between
- approx. 30% of the rated speed for ‘slow ahead’ and
- approx. 65-80% of the rated speed for ‘half-speed ahead’.

A speed corresponding to 75% of the rated speed should be selected for designing the bottom safeguard systems.
The rated speed or, where applicable, increased speeds at maximum power plant output must be assumed for particularly critical local conditions (high wind and current loads for the ship, nautically unfavourable channels), or for basin trials in shipyards. These conditions must be clarified with the future operator of the port facilities, particularly with the port authority.

Water Jet Generation by the Bow Thruster

The bow thruster consists of a screw which works in a pipe and is located cross-wise to the longitudinal angle of the ship. It is used for manoeuvring out of a standing position and is therefore installed at the bow - more rarely at the stern. When the bow thruster is used near to the quay, the generated water jet hits the quay wall directly and is diverted to all sides from there. The critical element for the quay wall is the part of the water jet directed at the harbour bottom, which causes scour in the immediate vicinity of the wall on hitting the bottom, see Fig. 61.

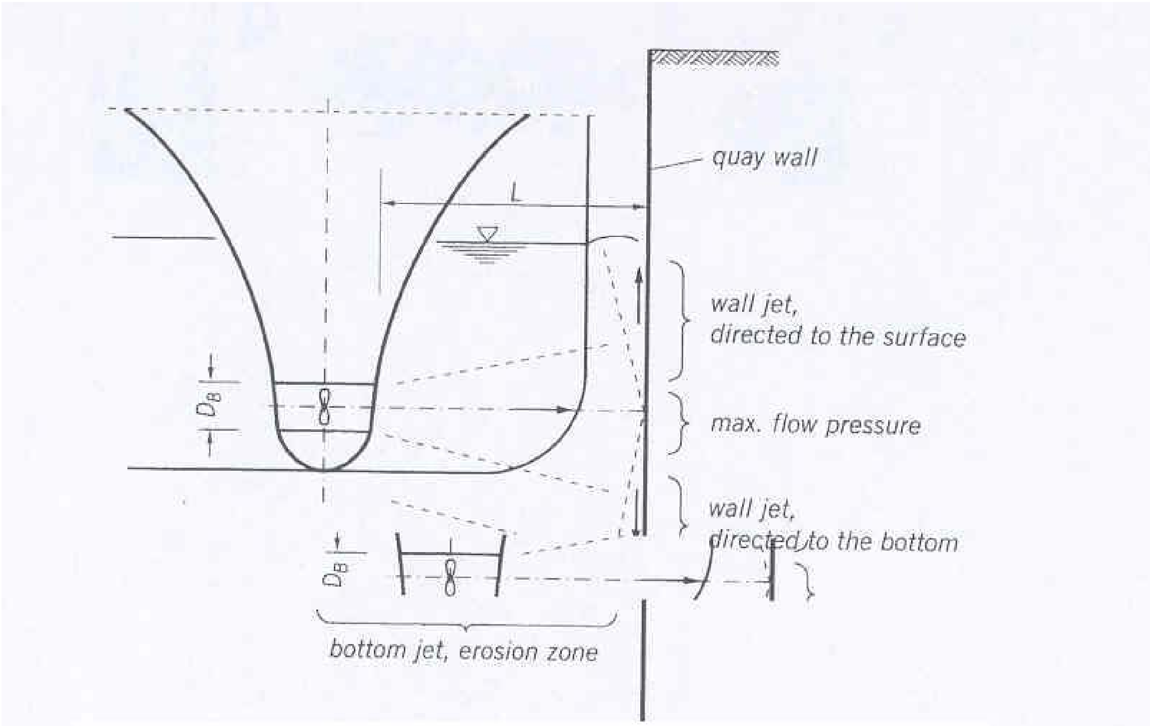


Fig. 60 Distance between opening of bow thrusters and quaywall.

The velocity at the bow thruster outlet $V_{o,B}$ can be calculated with equation (3):

$$V_{o,b} = 1.04 \left(\frac{p}{_{po} \, . \, D_B^2} \right)^{1/3} \tag{3}$$

P = output of the bow thruster [kW]
D^B = inner diameter of the bow thruster opening [m]
ρ_o = density of the water [t/m^3]

Jet velocities of 6.5 to 7.0 m/s must be expected for the bow thrusters of large container ships (*P* = 2.500 kW and $D_B \approx$ 3.00 m).

The velocity of the part of the water jet hitting the bottom max V_{bottom}, which is responsible for erosion, is calculated as follows:

$$\max V_{bottom} = 2.0 \left(\frac{L}{D_B} \right)^{-1.0} . \; V_{0,B} \tag{4}$$

L = distance between opening of the bow thruster and quaywall.

The bow thruster usually operates at full load.

Scour Protection

The following measures can be considered for averting the dangers to waterfront structures from scour:

(1) scour surcharge to the structures;
(2) covering the bottom with a stone fill in loose or as a grouting;
(3) covering the bottom with flexible composite systems;
(4) monolithic concrete slabs, e.g. in ferry beds.

To 1):
In this case, a protective layer is not applied and the scour formation is accepted. The structure is secured by calculating the theoretical foundation bottom at a depth which takes account of the corresponding scouring depth (scour surcharge).
This procedure creates problems because it is hard to even approximately estimate the anticipated scour depth and thus accurately calculate the scour surcharge. When this method is used, the bottom should be checked continuously by soundings in the area at risk form scour so that it is possible to react at once when the tolerances are exceeded.

To 2:
A loose stone cover constitutes one of the most frequently used protection systems. The following requirements must be met:

- stability when exposed to screw action;
- installation of the stone cover so as to cover the bottom reliably. This means installation in 2 to 3 layers;
- installation on a grain or textile filter rated for the relevant subsoil;
- connection to the waterfront structure underneath the current to be safe from erosion.

Verification of current stability is provided according to the following assumption:

$$d_{req} \geq \frac{V_{bottom}^2}{B^2 \, . \, g \, . \, \Delta^1}, \qquad (5)$$

d_{req} = required diameter of the stones [m],
V_{bottom} = bottom velocity as per equation (2.2) or (4) [m/s],
B = stability coefficient [1] according to [170],
= 0.90 ship without central rudder (stern screw),
= 1.25 ship with central rudder (stern screw),
= 1.20 bow thruster,
g = 9.81 (earth acceleration) [m/s^2],
Δ^1 = relative density of the stone material under uplift [1],
= $((\rho_s - \rho_o)/\rho_o$,
ρ_s, ρ_o = density of the stone material respectively water [t/m^3].

For single-screw vessels with central rudder (e.g. container ships), stone fills using broken rocks with ρ_s = 2.65 t/m^3 create problems for bottom speeds of 4 to 5 m/s, as the corresponding diameters with d_{req} = 0.7-1.0 m are so large that they cannot be easily handled.

Stone fills as a grouting are stable up to very high speeds ($V_{\text{bottom}} \approx$ 7 m/s) as a result of the cramping effect. Materials which adhere well and can be used underwater such as bitumen or colloidal mortar are suitable as grouting material.

The following aspects must be taken into consideration when installing such a solution:

- The grouting should not cover the whole area. A minimum pore volume (continuous from the bottom to the top surface of the fill) of 15-20% is necessary for pressure compensation.
- The grouting stone fill forms a stable but rigid unit. Scours frequently occurring at the edges cause underwashing and in some cases damage because the grouting cannot react flexibly to these phenomena. A combination with flexible elements at the edges can be beneficial to avoid such damage.
- The grouting thickness, positioning of filters and connection to waterfront structures are rated according to the same aspects as for loose stone fill.

<u>To 3)</u>:

Composite systems consists of systems in which various basic elements are combined to create a sheet-type safety mat. The most important principle to be observed here is that the elements are to be connected in a flexible way for good adjustment to and stabilisation of edge scouring. The following technical solutions are known:

- concrete elements connected by ropes or chains,
- mesh containers filled with broken rocks ('gabions'),
- geo-textile mats filled with mortar.

These systems provide excellent stability properties when rated with adequate dimensions.

A generalised, current-mechanics design assumption is only available for special cases because of the individual variety of available systems, so that these systems are frequently dimensioned according to the manufacturer's experience.

When the system is flexible enough, it shows good edge scouring behaviour, i.e. occurring edge scour is stabilised automatically by the system, thus preventing regressive erosion. The disadvantage of wire rubble mats ('gabions') with equally good stability and edge scour properties is that the wire mesh is liable to corrosion, sand wear and mechanical damage, and the current stability is lost when the wire mesh is destroyed.

<u>To 4)</u>:

An underwater concrete bottom offers ideal erosion protection because its depth can be produced with far greater precision than a stone fill.

There is no need to fear that individual stones can be dislodged from the system by an anchor or screw action; the thrust force transferred locally to the bottom by the screw is distributed across a wide surface.

The rigid underwater concrete bottom cannot follow uneven settlement, so that special solutions are required for the edges. Underwater concrete bottoms are installed in thicknesses from 0.30 to 1.00 m depending on the installation method. The installation of concrete under water is a technologically complicated, very costly process, which has to take account of diving operations, poor visibility underwater, floating equipment, underwater formwork, special types of concrete, suction of mud on the harbour bottom. A rigid concrete bottom as erosion protection becomes competitive again when it can be installed in the dry, e.g. with the protection of a catchment dam. The necessary spatial expansion of this type of consolidation depends very much on local conditions and should be rated so that the water jet velocity is essentially reduced at the edges to rule out any risk of the system being underwashed by resulting scour.
The minimum spacing of the system perpendicular to the quay must also be selected so that the area of the passive earth pressure wedge at the foot of the wall is not reduced by scour.
An initial approximation:

- for single-screw vessels:

• perpendicular to the quay:	L_N	= 3 …4 . D,
	D	= screw diameter,
• lengthways to the quay:	$L_{L,H,1}$	= 6 … 8 . D,
	$L_{L,H,2}$	= 3 . D,
	$L_{L,B}$	= 3 … 4 . D_B.

for twin-screw vessels:
The above values for L_N and $L_{L,H}$ are to be doubled.

The total expansion of the consolidation layer lengthways to the quay including the intermediate length L_Z depends on the possible variations in the berth position.

The quay wall can be pulled back to create a water cushion between front edge of the quay and ships wall, where necessary in combination with jet deflectors, offering efficient possibilities for minimising load on the harbour bottom.

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