Course CIE3330

Hydraulic Structures General

November 2009

Faculty of Civil Engineering and Geosciences

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PREFACE

Work on these lecture notes does not seem to stop. Is this caused by frequently changing educational circumstances, never ending perfectionism of the writer or enthusiasm for design of hydraulic structures? Although the answer does not matter much, let's hope it is the enthusiasm, generally driven by new developments in the hydraulic engineering field!
As always the comments of the reader, students in particular, will be appreciated, it will keep the writing engine running.

Delft, November 2009
Wilfred Molenaar

READER TO THESE LECTURE NOTES

These lecture notes "General" are part of the study material of the course 'Hydraulic Structures 1' (code CT3330), part of the Bachelor of Science and the Master of Science, the Hydraulic Engineering track, for civil engineering students at Delft University of Technology.

For the course "Hydraulic Structures 1" the following lecture notes are available:

1. "Hydraulic Structures 1 - General"
2. "Manual Hydraulic Structures",
3. "Locks"
4. "Caissons"
5. "Dome Island Beaufort Sea"

The use of the lecture notes, better said the intensity of use, differs from varies lecture note to lecture note, and from academic year to academic year. The focus in all these documents is on functional and structural design of hydraulic structures, however, "General" has a more introductory character, the "Manual" has a definite engineering calculations character. "Locks" and "Caissons" focus on design of a specific structure, whereas "Dome Island Beaufort Sea" puts the structure in perspective of a large project at a remote location in the worst imaginable climatic conditions.

Regarding the lecture notes "General":
Chapter 1, Introduction to Hydraulic Structures, is intended to briefly show a lot of structures, describe functions and construction methods where possible. Where Chapter 1 is answering the question "what", the "How" question is answered for design in Chapter 2 and in Chapters 3 through 5 for construction.

For Dutch BSc-students this is probably one of the first courses in the English language, so some very specific technical terms have been translated into Dutch (indicated between brackets and in italics).

THE COURSE HYDRAULIC STRUCTURES 1 – CT3330

Hydraulic structures are generally quite large, and they are found in, under, next to, above, anyways always near water; water like the water in the sea, rivers or canals, or groundwater. Hydraulic structures could be subdivided into earth works and structures. Earth works are works generally constructed using soil, clay, sand or other granular material; for instance rubble mound breakwaters, dikes and dams. In the course CT3330 the focus shall be on the hydraulic structures for which a lot of steel and concrete, sometimes wood is needed. However, there is no structure without a foundation, hence soil(s) will be treated as well.
A large structure in or near water is hard to construct. And of course construction circumstances (climate, availability of labour, materials and equipment, soil conditions) are different in every location. That's why "design for construction" is very important in every hydraulic engineering project, and in this course.

"Hydraulic Structures" in the Civil Engineering curriculum

The course "Hydraulic Structures 1" focuses on design and presents the student practical applications of other engineering and/or science disciplines. This will mainly involve more theoretical fields such as material sciences, structural engineering, soil and fluid mechanics, see Figure I.

That is why some of the material covered in the lecture notes and the manual will undoubtedly repeat material covered in other courses already completed by the student. These summaries have been included, mainly in the "Manual", in order to be able to quickly translate the considered theory into boundary conditions and loads important for the design from a hydraulic engineering point of view. Occasionally the material covered in this book will be more advanced than the theoretical subjects covered in the curriculum. In such cases no theoretical derivations will be given, the "Manual" will remain confined to stating the design rules and showing practical approximations.

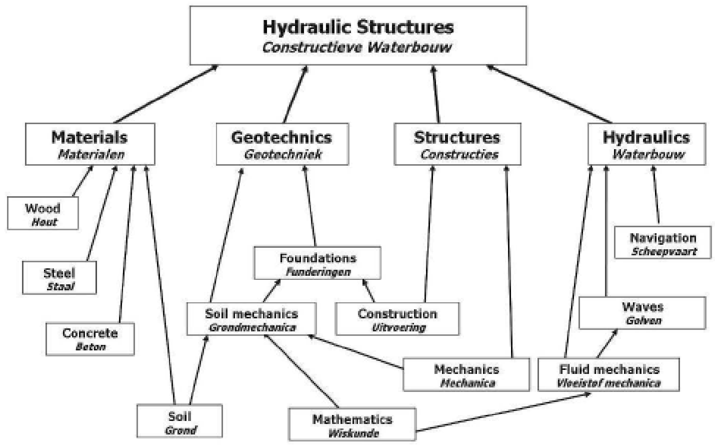


Figure I Relationships with other disciplines

Having written in short what precedes this course, it will be described shortly what follows. The course 'Hydraulic Structures 1' is part of a series of courses dealing with hydraulic engineering works and could be considered as the basic module for follow-up courses on "Water Power Engineering", "Bored and Immersed tunnels", "Weirs, Barriers and Port Infrastructure" and "Flood Defences" in the fifth year, see Figure II.

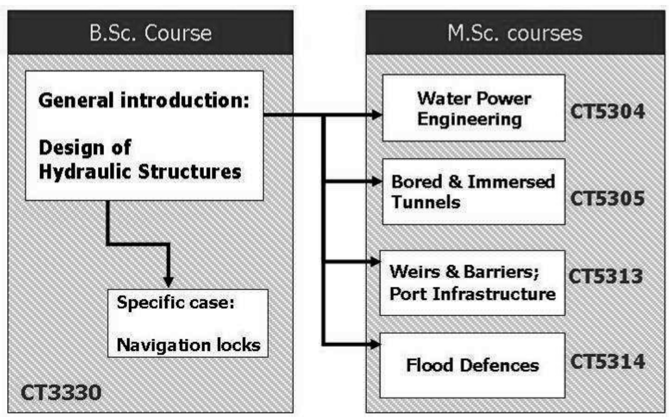


Figure II Modular structure of the courses Hydraulic Structures

1. Introduction to Hydraulic Structures

The following hydraulic structures will be briefly discussed in this chapter:

1. Piers;
2. Artificial islands;
3. Breakwaters;
4. Soil retaining structures;
5. Quay;
6. Jetty;
7. Breasting dolphin, mooring dolphin and guard wall;
8. Building pit, cofferdam and construction dock;
9. Dry dock and floating dock;
10. Road in continuous cut, approach or access road;
11. Tunnel, aqueduct and culvert;
12. Sluices/locks, siphon and pumping station;
13. Navigation lock, ship-lift and inclined plane;
14. Storm surge barrier;
15. Water-retaining structures: dam and weir;

The table below shows the above-mentioned structures and a summary of their primary functions.

Functions →
Structures ↓
X = usually
O = some times

	Structures	Vertical support	Horizontal support	Space for construction	Berthing & Mooring	Shelter for waves	Water retention	Soil retention	Horizontal translation of objects	Vertical translation of objects	Water discharge	Energy production	Crossing infrastructure
		1	2	3	4	5	6	7	8	9	10	11	12
1	Piers	X	O										
2	Artificial islands	X		X	X	O							
3	Breakwater					X							
4	Soil Retaining structures	X					X	X					
5	Quay		X		X		O	X					
6	Jetty		O		X								
7	Breasting & Mooring dolphin, Approach wall		X		X								
8	Building pit, cofferdam, construction dock			X			X	X	O				
9	Dry dock, floating dock						X	X	X				O
10	Road in continuous cut, Approach road						X	X	X	O			X
11	Tunnel, Aqueduct, Culvert						X			X			
12	Sluices/locks, siphon and pumping station						X			X			
13	Navigation lock, ship lift and inclined plane						X			X	X		
14	Storm surge barrier						X					X	
15	Water retaining structures: Dam, Weir						X			X			

Figure 1-1 Functions of Hydraulic Structures

Keep the function in mind when preparing the design for a structure: it is the reason for construction or for the project.

1.1 Piers

A pier is not a hydraulic-engineering structure in itself, but is a hydraulic-engineering element belonging to a civil work, for instance a bridge, barrier, weir or barrage. The primary functions of a pier are:

- being a part of the substructure, supporting the vertical load of the superstructure, for instance a bridge deck or a (drilling) platform;
- resisting horizontal loads as a part of water or soil retaining structures.

In general this means that the horizontal and vertical loads are transmitted from the superstructure to the foundation. As a consequence of the primary function, unwanted side effects, such as having to resist drag and inertia forces on the pier due to water flow and erosion of the soil (in rivers), may have to be taken into account.

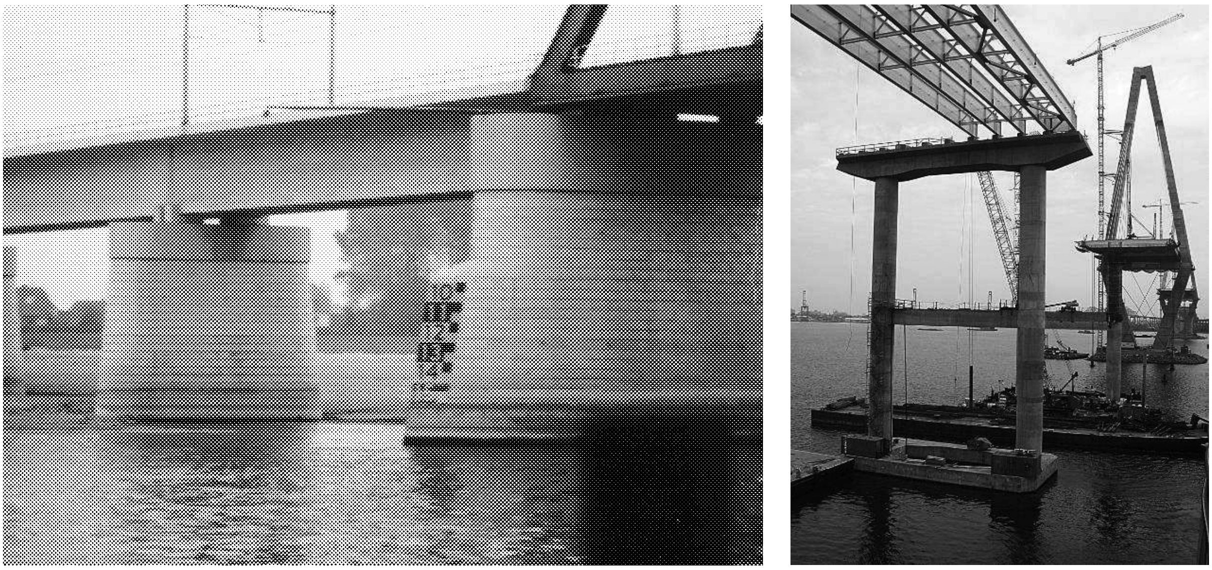


Figure 1-2 Bridge piers; Masonry pier (left), Concrete piles (right)

The most common pier is the bridge pier. These piers can be made of masonry, reinforced concrete or combinations. The type of foundation of these piers depends on the soil conditions. If the pier is provided with a shallow foundation, one will usually try to design the pier so that the entire surface area of the foundation is subjected to compression force. This generally results in piers of great weight. In order to be able to carry this weight, it is mostly necessary to improve the soil conditions before construction. This is especially important if the foundations are situated in the loose granular sediment of a river delta. A loose granular sediment bottom will also necessitate construction of scour protection.

Examples of piers used in water-retaining structures can be found in the Oosterschelde Storm-Surge Barrier and the Hartel Barrier.

The most important loads on a pier are:

1. Vertically:
    - load from the (main) superstructure: dead weight of the structure and variable loads like traffic loads
    - dead or selfweight of the pier;
2. Horizontally:
    - water pressures caused by: water flow, waves, hydrostatic pressure. Both direct and indirect pressure; the latter due to retaining elements such as gates;
    - wind: direct wind on the pier and indirect e.g. via the main structure and traffic;
    - ice load;
    - earthquake load.

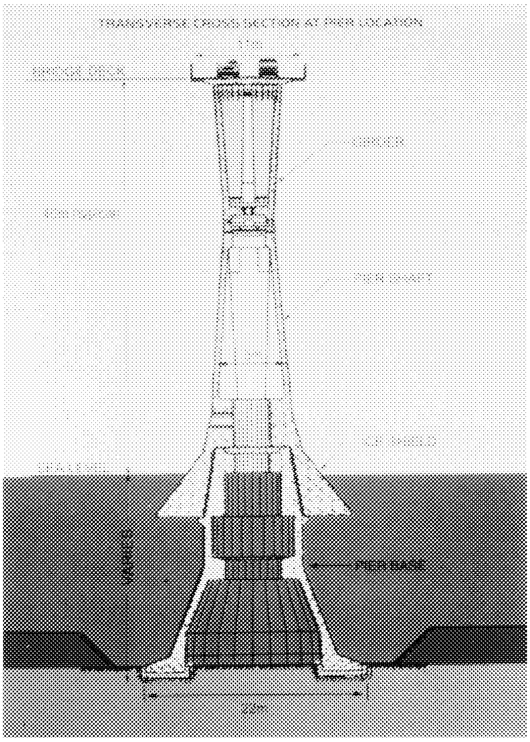


Figure 1-3 Precast pier with cone shaped ice braker

In many cases the shape of the pier is determined by its primary function. For instance the shape of a bridge pier will be different from a pier for a storm surge barrier (compare Figure 1-3 with Figure 1-5). The bridge piers mainly transfer vertical loads, dead weight is the governing load *(maatgevende belasting)*, while the piers in a water retaining structure, such as a storm surge barrier, although the dead weight is definitely not negligible, first of all have to be able to resist the water pressure loads in horizontal direction.

Considering loads in the horizontal direction, besides e.g. current and wave loads, ice can play an important role in the design of piers. A nice example of reducing the ice load using a specific shape for the pier is shown in Figure 1-4. The cone shape will force the ice to move upwards to break it. Its round shape prevents the ice from piling up against the structure.

Another example of using 'shape' is a pier in a river in which flow only occurs in one direction. Such a pier will be streamlined in order to reduce the loads on the pier caused by water flow.

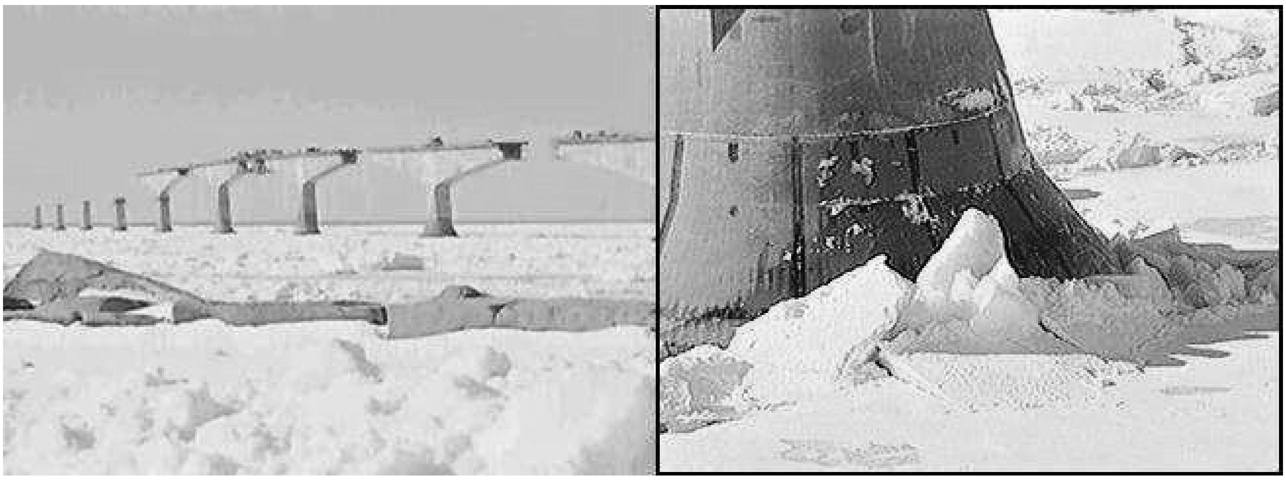


Figure 1-4 Prince Edward Island bridge - Canada; Ice load on the piers reduced by conic shape

For structures with a large number of piers it could be a good idea to prefabricate the piers in a building or construction dock *(bouwput of bouwdok)*, see Figure 1-5. The big advantage of prefabrication is that it reduces the number of (relatively inaccessible) building sites in the water, reducing the construction time. However, in order to place the piers in their final location special equipment will be required. This equipment usually purpose designed and constructed (see Figure 1-6).

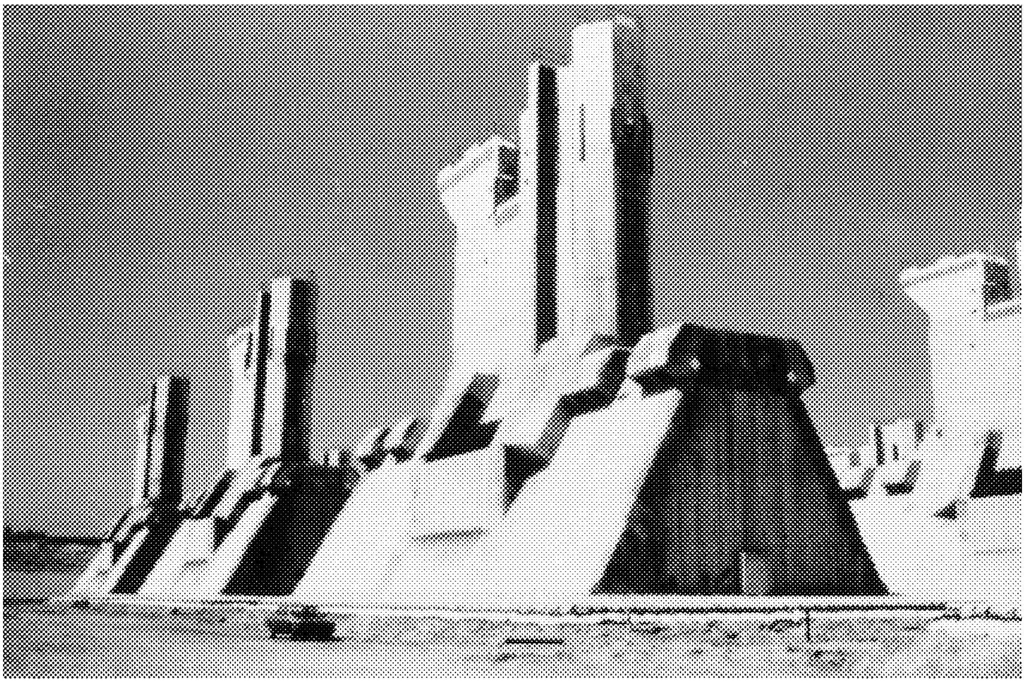


Figure 1-5 Precast piers for the Oosterschelde Storm Surge Barrier in the construction dock

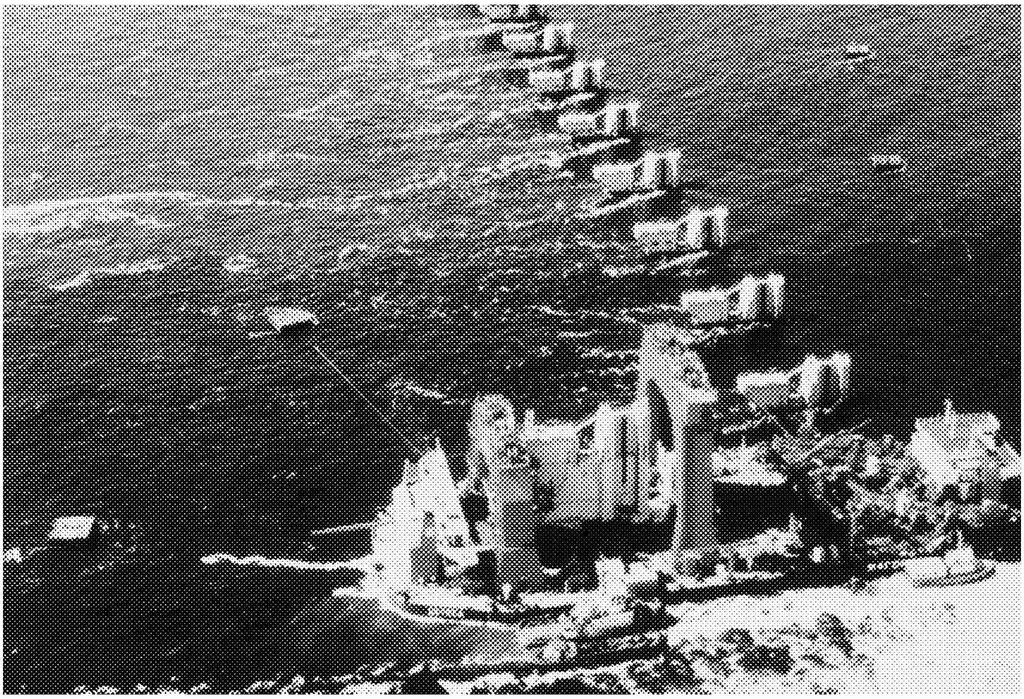


Figure 1-6 Positioning of piers for the Oosterschelde Storm Surge Barrier using the crane ship Ostrea

An important aspect to take into account designing piers is scour. The presence of the pier causes a local increase of turbulence and sometimes an increase in average flow speed as well (see the flow patterns in Figure 1-6). This will increase the erosion locally and the foot of the pier, the foundation may get exposed. Often it will be necessary to construct a scour or bottom protection to prevent undermining the whole structure.

1.2 Artificial islands

In fact artificial islands could be considered to be nothing more than very wide piers; they have the same function: create a working surface above the water and supporting vertical loads. For instance for supporting an oilrig or serving as a foundation of a lighthouse, windmill or radio mast. The largest artificial islands are used for harbor expansion or to create land for industrial zones.

Figure 1-7 Artificial islands – Chek Lap Kok near Hong Kong (left) – World at Dubai (right)

Artificial islands can be constructed using soil, structures or combinations. The choice which type will be used mainly depends of settlement sensitivity of the subsoil and the depth of the ground layer having sufficient bearing capacity below the water surface. To use soil, usually sand, for an island in very deep water requires huge quantities of soil to create the long slopes. This is why islands in deep water, created as earth works are generally financially unattractive. For artificial islands requiring a relatively small surface, the use of one or more pier like structures is more interesting. For larger islands a combination of both types of works is possible. The structure is used to bridge the height difference between the top of the island and the foundation level. Examples of this are a circular wall and the use of caissons filled with soil placed in a square. The most important functions of the structure at the outside of such an island are:

- to retain the soil;
- to retain water during high tide;
- to provide protection against erosion caused by wave attack and flow;
- and to provide quay, berthing and/or mooring facilities, if required

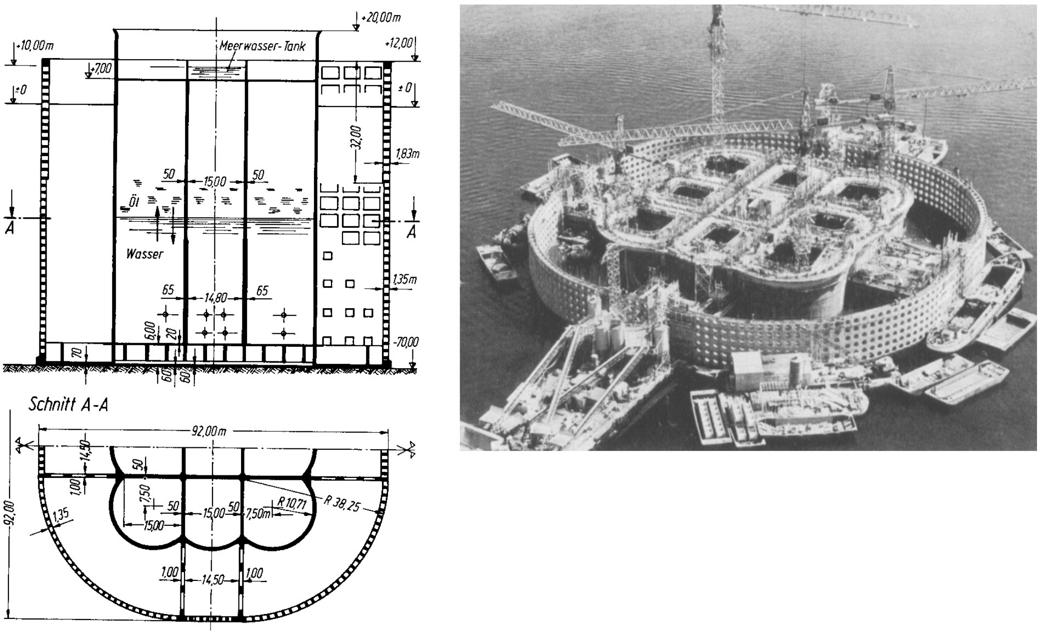


Figure 1-8 Artificial island – Ekofisk, constructed 1972-1973 using concrete caissons

1.3 Breakwaters

The main function of a breakwater is to:
- shelter (water) areas for wave action and some times for strong currents generally for creation of quiet mooring places and calm navigation water.

Additional functions are:
- reduction of sedimentation of the navigation channel;
- visual guidance of navigation;
- creation of space for recreation, see Figure 1-9, etc.

The functions are realised by:
- reflecting and breaking of waves;
- guiding and diverting cross currents.

Figure 1-9 A promenade on a breakwater

From a functional point of view, a breakwater should be a barrier for waves and/or flow. In order to be able to stop the waves and the flow effectively the breakwater has to be a more or less solid obstacle from the bottom or bed till above water level. To provide this obstacle there are three effective, however, fundamentally different solutions, namely:
- as a beach with very shallow slopes, using only soil as construction material; this would turn into construction of an island, see Section 2.2.
- as a dam made of soil, usually sand, in combination with stone or other single elements allowing steeper slopes and better resistance to wave attack (Rubble Mound Breakwater);
- as a vertical structure made of concrete or steel caissons, concrete walls, blocks or steel sheet piling.

Varying combinations of the three fundamental types are possible, known as composite breakwaters, see Figure 1-10.

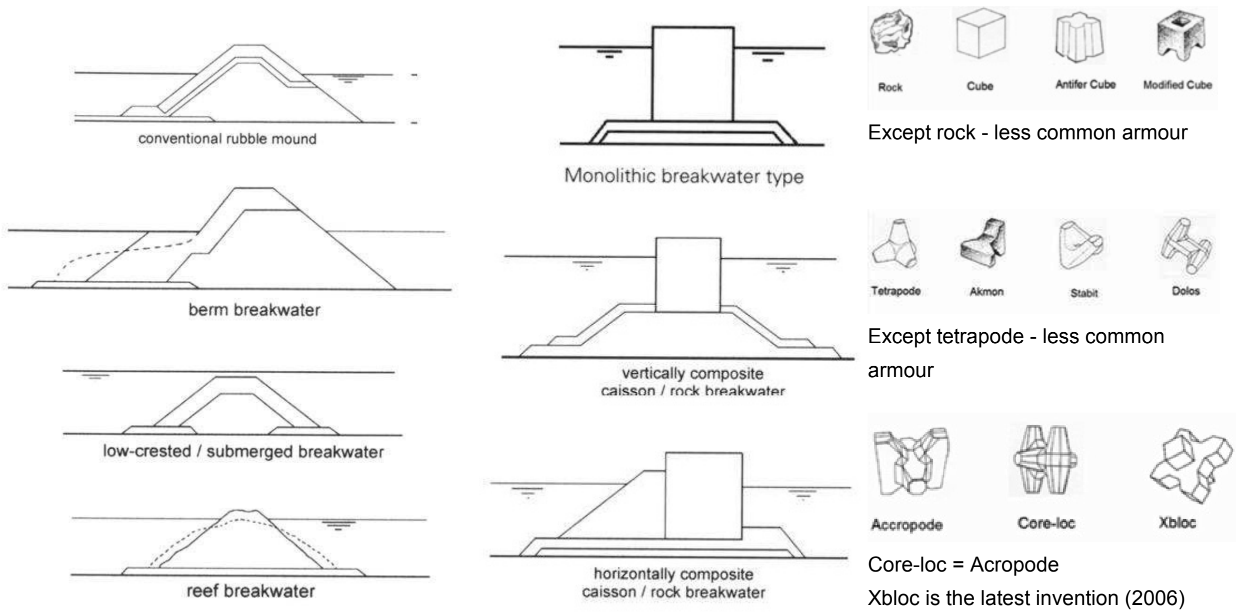


Figure 1-10 Breakwater types and concrete armour types

For further, elaborate information, reference is made to the course 'Breakwater and Closure Dams' (CT5308).

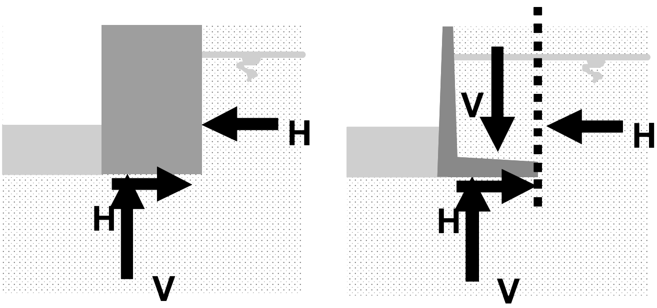
1.4 Soil retaining structures

Soil retaining structures generally not only retain soil, their primary function, they also retain water, i.e. resist water flow, due to water level differences over the wall, or wave attack. Often they are an indirect support for facilities or (other) structures at the surface of the retained soil, which results in extra loads to be resisted by the soil retaining structure. Usually the soil retention function results in the biggest (horizontal) load on the structure, determining to a far extent the dimensions of the structure, however, be aware of the exceptions to this rule. The following structure types can be used for soil retention:

- gravity structures;
- wall structures;
- composite structures.

1.4.1 Gravity structures

In general a gravity structure has a shallow foundation, in the Netherlands some times referred to as steel foundation. It retains soil and water transferring the loads to the foundation by means of a compression force and a friction force, both acting on the foundation surface, and both the result of gravity or dead weight of the structure. It requires the layers having sufficient bearing capacity to be right beneath the structure. Soil improvement of the top layer, by means of dredging or excavation, followed by filing the trench by coarse sand and/or stone filter layers, is quite common. The loads behind the structure are mainly supported by the soil and partly by the structure, which is the result of friction between the soil and the structure.
Some gravity structures have pile foundations providing horizontal and vertical support. The loads are transferred into deeper soil layers, hence the name deep foundation as alternative for pile foundation. Tension piles transfer their tension forces solely by means of friction along the pile shaft, whilst compression piles transfer compression forces directly to the subsoil under the pile toe and, generally to a lesser extent, by shaft friction as well.



Do not forget overturning moment M for stability

Figure 1-11 Gravity structures - shallow foundation

Examples of gravity structures are:

- caissons;
- pneumatic caissons;
- non-reinforced concrete or masonry walls;
- block walls;
- L-shaped walls;
- (cellular) cofferdams.

Caissons are used as pier, breakwater, quaywall, closing element for flood defenses etc.. Nowadays caissons are generally built in construction docks, floated, then transported using whatever waterway to the building site. Looking at the use for quaywall construction, once the caissons have been sunk, the ground level behind the line of caissons can be raised to the required port terminal level.
After being sunk and positioned, the caisson can be used to transport building materials 'over the head'. The sand fill for the caisson will be quite a large volume of the material to be transported. It is a matter of

cost whether floating (dredging) equipment or the overhead construction road will be used for caisson filling. Generally the superstructure, the top part of the caisson, most of the times a cover or roof slab, all the utilities, fenders and bollards for quays, is finished using the overhead construction road.

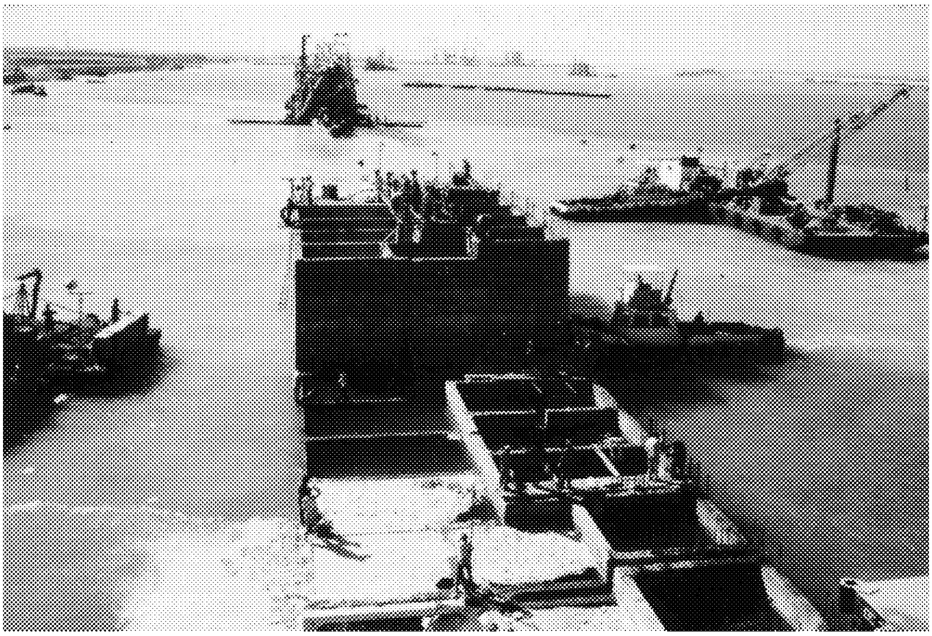


Figure 1-12 The sinking of caisson to be used for a quay wall

Pneumatic caissons are built on site at ground level in dry circumstances, and are sunk to the required depth by removing the soil below the structure, effectively undermining the caisson, the dead weight of the structure being the driving force. A pneumatic caisson is equipped with relatively sharp edges, to cut through the soil, and a chamber at the bottom of the caisson where the soil is being removed. To prevent the chamber being filled with (ground) water, an overpressure (air) will be provided.

Non-reinforced concrete and masonry walls *(metselwerk of (bak)steen muur)* can be used as retaining structure when the required height is not too high. Obviously the availability of materials plays an important roll in the decision to build such a structure. The dimensions of the structure are determined by the maximum allowable stresses in the material. In non-reinforced concrete and masonry, tensile stresses are unacceptable and the shear stress is rather limited. Hence, the dimensions of the wall increase significantly (square of the height) with increasing height.
Masonry walls and massive non-reinforced concrete walls should be constructed 'in the dry'. The larger the required retaining height, the more important the presence and the position of -if any- load bearing soil layers for the foundation. Differences in the settlement process will definitely result in cracks in the wall. Partly for this reason vertical expansion joints *(dilatatie voegen)* are constructed in the wall allowing a certain amount of differential settlement. The wall panel between the expansion joints is now able to rotate a little. Another good reason for the use of expansion joints is prevention of cracking due to temperature differences.

Block walls, blocks having the size of man or larger, are often used as quaywalls, however, use above ground for instance for soil storages is quite possible, see non-reinforced concrete and masonry walls. Construction of the underwater part of a block wall can start from dry land or simply from a pontoon, the latter is done more frequently. When starting on dry land, the remaining structure is built overhead, over the top of the completed part. The additional soil behind the quay can be placed from the land. Since a block wall has many joints, groundwater will be able to seep through it. To prevent erosion of the soil to be retained a granular filter or a filter cloth could be made behind the wall. In this case the block wall does not retain groundwater flow although a water level difference may exist or develop due to changing permeability. In case of complete impermeability the wall retains water as well and has to be able to resist maximum water level differences. The blocks are made of non-reinforced concrete or natural stone. The non-reinforced concrete blocks are often produced near the construction site or prefabricated at another site close to the main source of material supply, e.g. a mine. This may invoke transport problems.

A block wall constructed using the 'sloping bond' method should be considered as a special type of wall (see Figure 1-13 and Figure 1-14). To solve settlement problems due to weak soil layers the blocks are placed against each other at an angle in order to spread the load better through friction in the joints and to redistribute the load on the subsoil in case of locally larger settlement. However, the blocks are not only leaning diagonally sideways against each other, but backwards against each other as well. Once the wall is built, back filling of the soil behind the quay pushes the wall forwards into position.

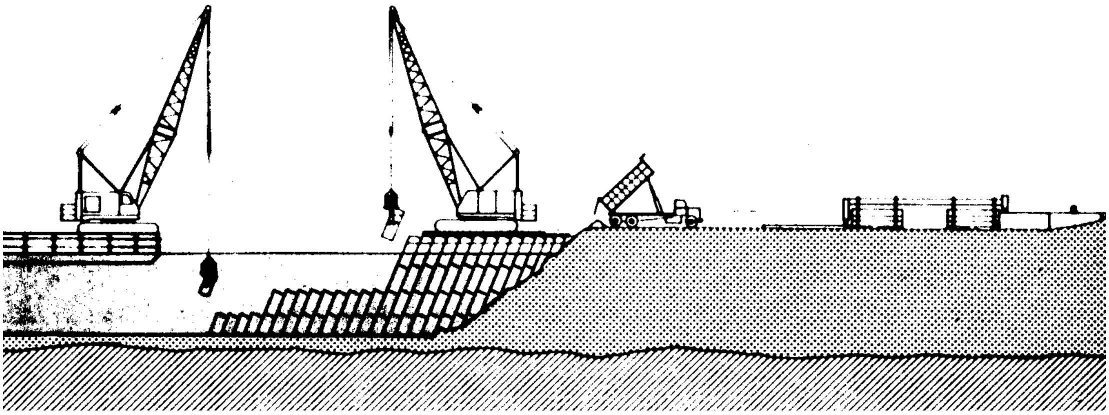


Figure 1-13 Sloping bond block wall

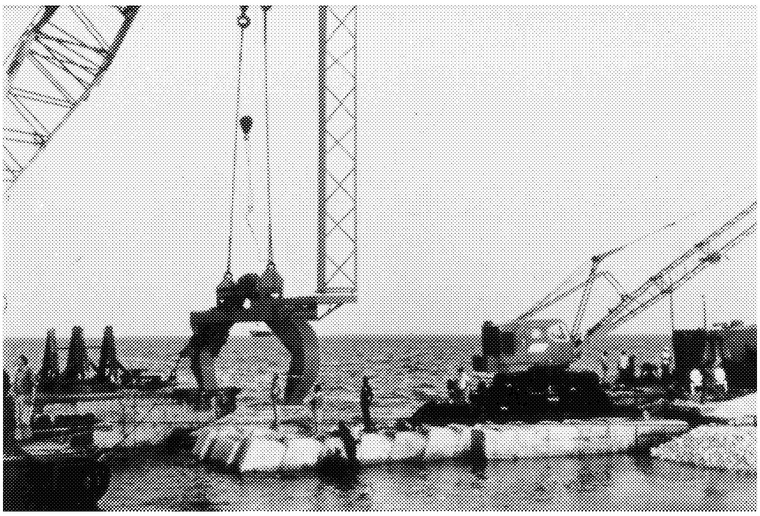


Figure 1-14 Construction of a sloping bond quay wall

L-shaped walls have a horizontal slab and a vertical wall, obviously piecing together the shape of an “L”. The slab or floor carries the weight of the soil above it and the weight of the wall. Add the dead weight of the slab to arrive at the total dead weight of the structure. The stability of the L-wall depends on the friction stress developed, which is linearly related to the total dead weight of the structure but limited to a maximum depending on the strength characteristics of the soil, beneath and adjacent to the bottom of the foundation, the slab. The wall and the connection of the wall to the slab are subjected to bending forces, requiring the use of reinforced concrete. L-shaped walls are always built in the dry. If the required height is not too high it is possible to use precast walls and transport these to the building site by ship, some times by rolling stock.

A single cellular cofferdam is a combination of steel sheet piling and soil, working together as a gravity structure, when the stability of the whole cell is being considered. The sheet piles are driven into the ground in a circle, see Figure 1-15. Once the circle is completely closed, the cell is filled with soil. Often bracing is used near the top of the sheets to maintain the circular shape. The load on the sheet pile wall due to the soil filling is transformed into a ring stress in the sheets, which introduces tensile stresses in the sheetpile wall and the joints. Zooming in on the steel sheet, the use of flat sheets is very efficient, because the tensile stress in the wall remains as tension stress, whilst the more commonly used U-shaped sheet piles, have to resist extra bending moments, which would also be introduced by the ring stress in the wall.

Note:
- The word cofferdam is also used for a multiple of these cells in a straight line or in a curve, as required. It is possible to construct a cofferdam without using cell shapes, e.g. simply as two parallel sheetpile walls connected by bracing at one or more levels.
- The cofferdams discussed here, are either above ground level, constructed in the dry, or above bottom level, when constructed in water *(kistdammen)*. Soil, some times stone, is used as filling.
  Another type of cofferdam, also referred to as braced excavation is discussed in Section 2.8. Soil is being excavated, as the second description implies.

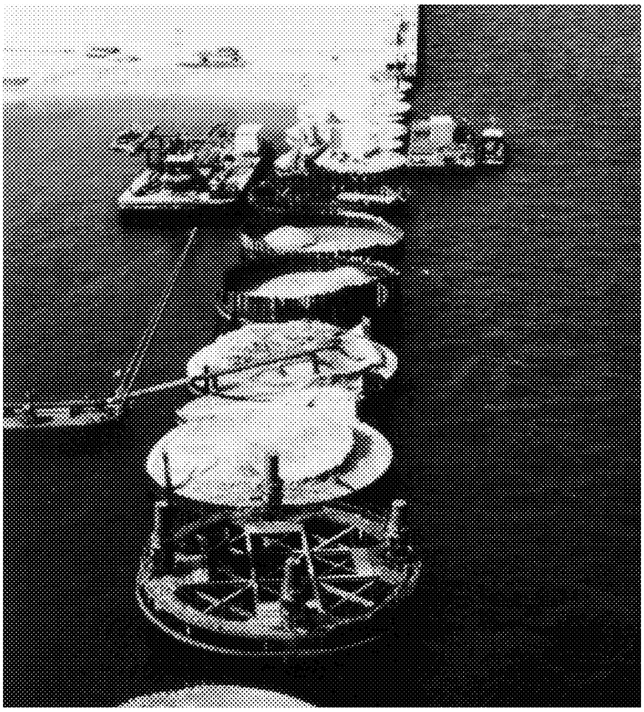


Figure 1-15 Cellular cofferdam under construction

When the soil layers with sufficient bearing capacity are positioned at a larger depth, it is usually economically not feasible to construct a gravity type of wall. It may not even be technically feasible. Depending on the depth of the load-bearing layer it may be possible to apply a soil improvement or use a pile foundation. In practice, a pile foundation is quite frequently used in combination with an L-shaped wall of reinforced concrete. In the past there are also examples of masonry walls on pile foundations. (On a number of occasions, for aesthetic reasons or to preserve history, masonry walls are constructed in front of the reinforced concrete walls).

1.4.2 Wall structures

In the case of wall structures, the horizontal load is partly resisted by friction in the soil, but mainly transferred by the wall and the anchorage, if present, into the deeper subsoil. The vertical load is partly carried by the wall (friction) but mainly transferred through the soil into deeper layers (vertical soil pressures).

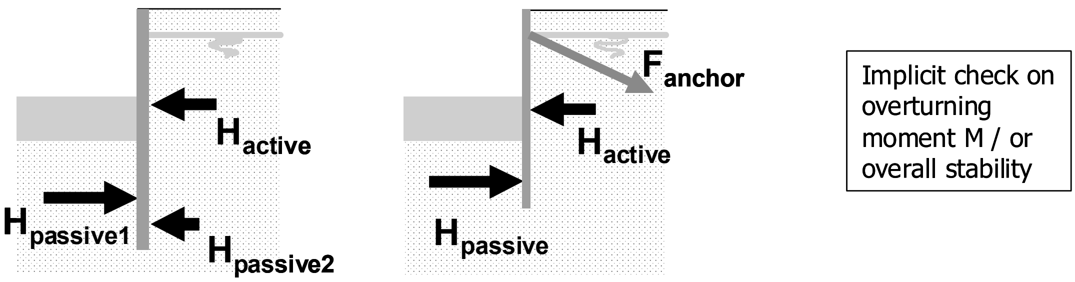


Figure 1-16 Wall structures; cantilever wall (left), anchored wall (right)

The wall structure has to resist bending moments, compression and shear forces. There are various types of wall available:

- Wooden piles and planks;
- Berliner wall;
- Steel sheet piling;
- Concrete sheet piling;
- Combiwall (steel);
- Diaphragm wall (concrete) *(Diepwand)*.

Cantilever walls relatively small retaining heights:

- Wooden piles and planks are the simplest soil retaining structures and are frequently used for ditches and ponds, i.e. only for small retaining heights.
- A Berliner wall is a wall consisting of H-shaped profiles, driven into the ground at a certain distance to each other. In-between wooden beams or planks are either horizontally or vertically put into position. They are slotted down between the flanges of the H-piles. Vertical wooden beams, if embedded in the soil deep enough, transfer a part of the load to the subsoil themselves, the horizontal ones transfer the load to the H-piles, which are doing all the work in that situation. Berliner walls are cheap and easy to build. Because this type of wall is not watertight and not suitable for the larger retaining heights they are seldom used in Dutch hydraulic engineering works.
- Steel sheet piling is generally used for temporary structures and for retaining heights which are not too great.
- Concrete sheet piling is often used for permanent works because of its durability. Sheet piling can be placed both from dry land and from a pontoon on the water.

Anchored walls for larger retaining heights:

- combiwalls consist of tubular piles at regular centre to centre distances, taking care of strength and stiffness of the wall, between them two or three sheets of sheet piling, preventing soil to slip away, see Figure 1-19 and inset.
- other composite walls are H-profile walls and box-pile walls, both with infill sheets.
- An alternative to the combiwall or composite wall is the reinforced concrete diaphragm wall *(diepwand)*. Sections of the diaphragm wall are sometimes provided with ribs to create a T-shape, which results in larger strength and stiffness compared to a flat wall. Reinforced concrete diaphragm walls are always constructed from ground level in the dry.

If sheet pile walls are used in a quay structure, the top should be provided with a heavy concrete coping beam or cap *(deksloof)*. The coping provides the space for fenders and bollards. The bottom of the coping can be extended to below the low tide mark to prevent corrosion; however, this will lead to additional demands during construction (watertight formwork).

The possibility of using wall structures for quay construction is greatly determined by the soil conditions and the height that needs to be retained. Especially, in case of is a large surcharge loads *(bovenbelastingen)*, the length of the wall can be a limiting factor.

1.4.3 Composite structures

Composite structures are combinations of the above-mentioned conceptual solutions. The combined surcharge loads and soil pressures being too large for a wall structure, a heavy structure on pile foundations combined with a wall can be a solution, see Figure 1-17.

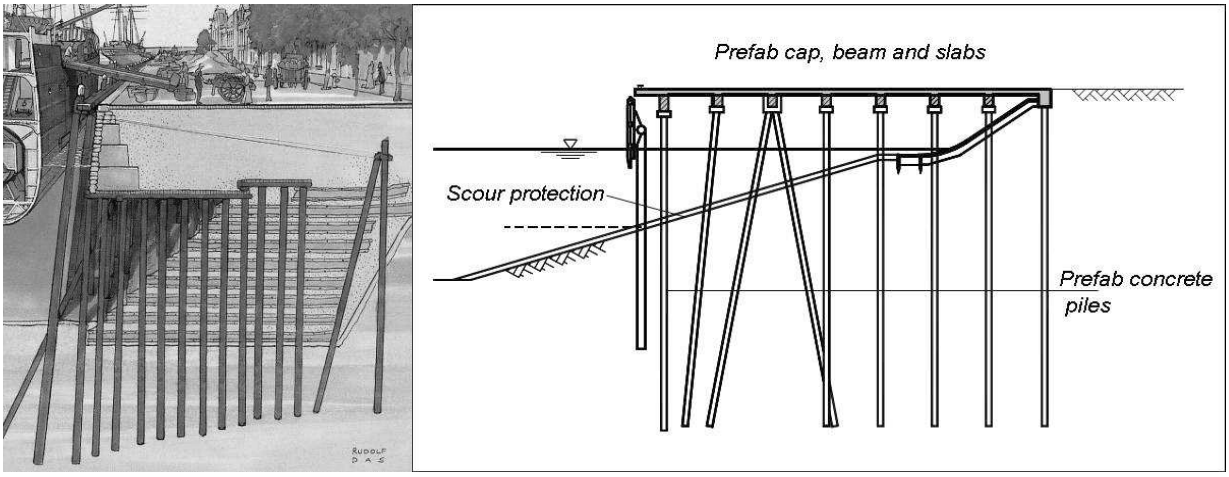


Figure 1-17 Composite soil retaining structures; Gravity wall and piles (left), platform, piles and slope (right)

A platform on pile foundation can be used for instance to carry top load and possible ship load a wall retaining the soil behind the platform. Below the platform a slope can be made so that the retaining height of the wall structure is reduced, see Figure 1-18. In this case there is a strict split of functions.

Figure 1-18 Precast platform under construction

Another commonly used composite quay structure is a heavy concrete coping structure on a pile foundation with a steel (pile) wall right below it. The heavy structure carries the top load and retains partly the soil while the wall can be used as a part of the pile foundation and as a soil-retaining element. In this case there is a less strict split of functions, see Figure 1-19.

1.5 Quays

The primary functions of a quay structure are:
- enabling berthing and mooring ships;
- support function for cranes and vehicles;
- support function for temporarily stored cargoes/goods.

The secondary functions of a quay structure are:
- retaining soil;
- water retention or preventing groundwater flow, if required;
- bank protection / erosion protection.

The demands on a quay structure follow the functions that need to be fulfilled. The function of mooring ships leads for instance to demands concerning:
- the draught in front of the structure;
- the facilities such as fenders, mooring bollards, Quick Release Hooks, ladders etc.;

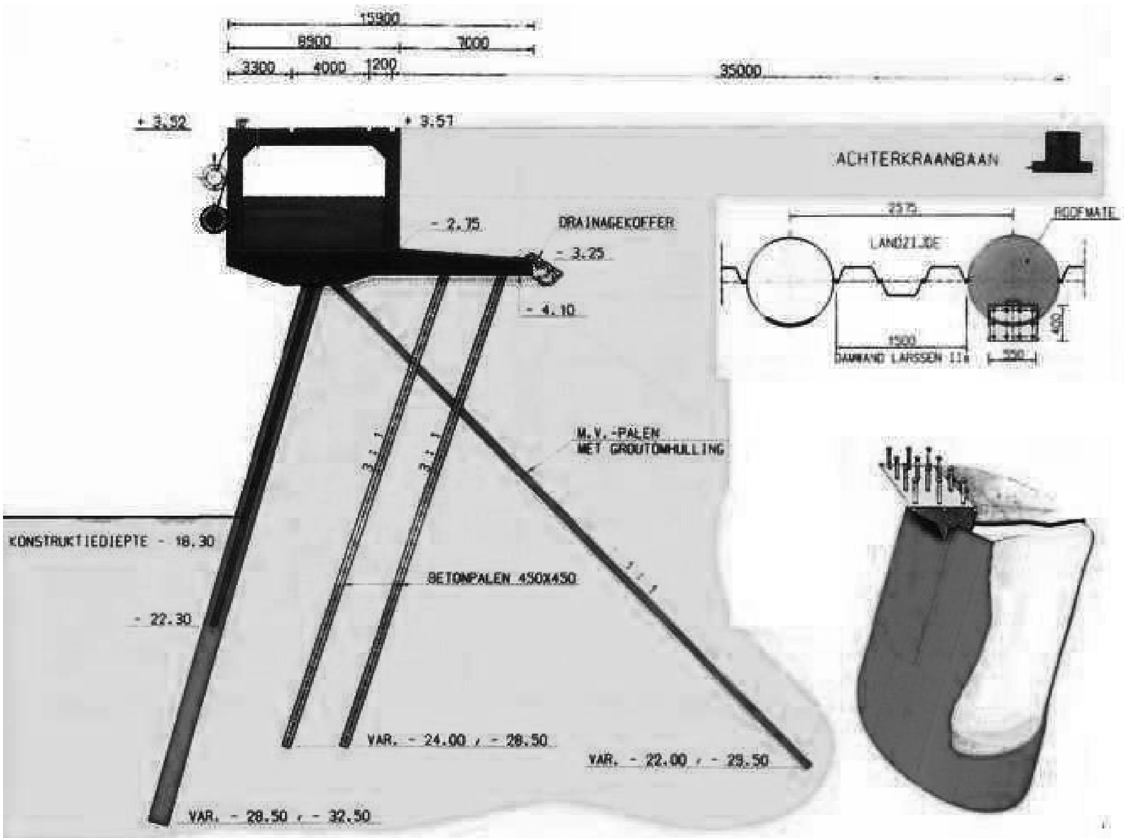


Figure 1-19 Composite quay wall at European Container Terminal (ECT) in Rotterdam

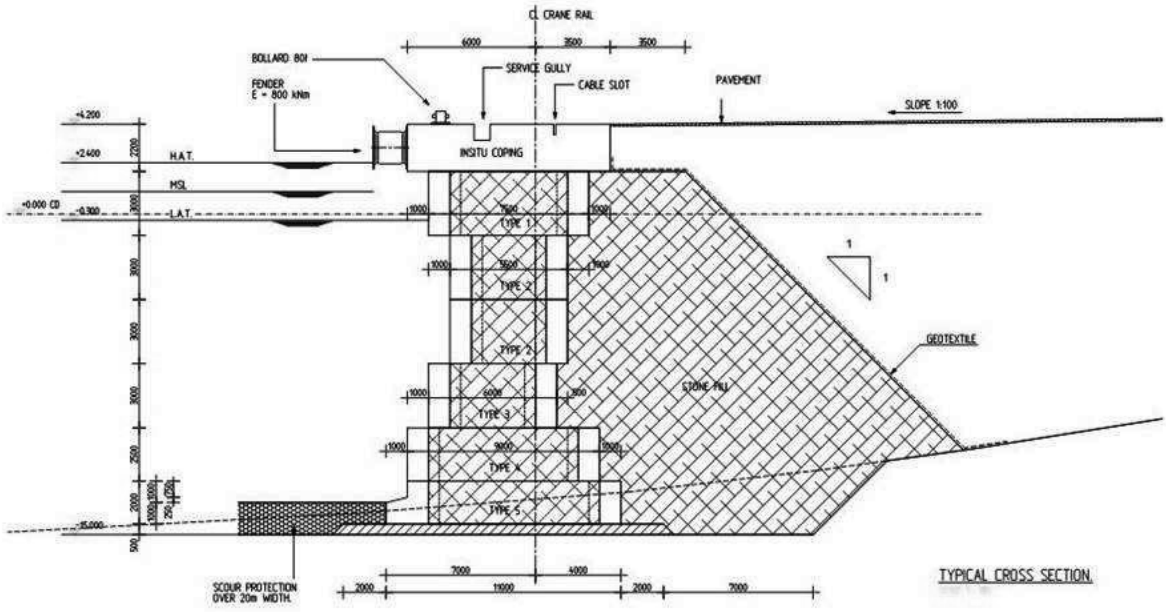


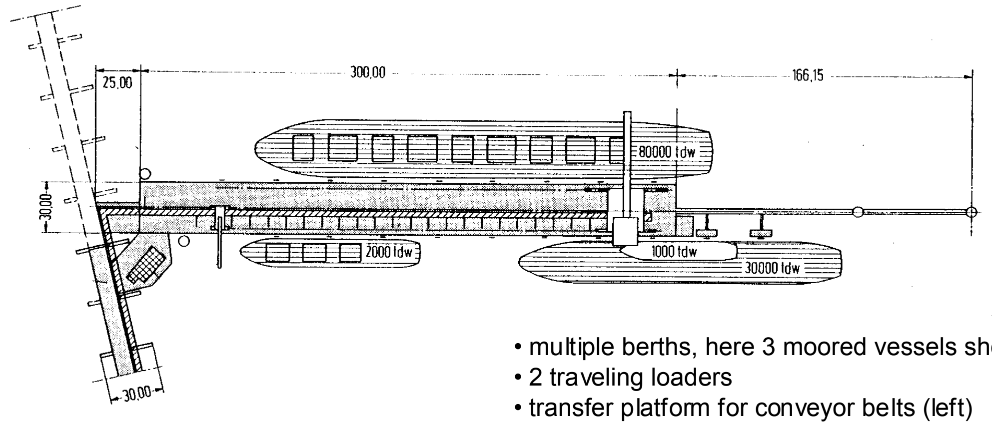
Figure 1-20 Block wall quay wall - Alternative design drawing

1.6 Jetties

The primary functions of a jetty are:
- enabling berthing and mooring of ships;
- support function for loading/unloading equipment, pipelines, conveyor belts and vehicles.

Jetties are typically light structures, only having to resist relatively small loads, in the absence of large loads due to the storage of goods and materials, or soil and water retention loads. As such, one of the most important advantages of a jetty is the possibility to construct it over a quite a distance into sea, to a spot where sufficient water depth is available for (deep) draught vessels, thus avoiding a lot of dredging. When designed and constructed for that reason or advantage, on both sides of the jetty, ships can be moored; a quay wall would have to be twice as long.
Looking at jetties often a high level of function separation can be observed. A berthing dolphin with its own fenders is used to absorb the kinetic energy of the ship, see the next Section on dolphins. Once the ship has been moored the forces on it caused by the wind and waves are transferred to mooring dolphins through hawsers. The mooring dolphins do not have fenders, only mooring bollard or quick release hooks to tie the hawsers to. The jetty structure carrying the loading/unloading equipment can be relatively light because the dolphins resist the larger horizontal berthing and mooring loads.

Some typical jetty types:
- Jetties e.g. for small and middle category crude or bulk carriers and for cruise ships are generally of the fingerpier or L-shape type, therefore have the same cross section along their entire length.
  - fingerpier jetty: generally extending perpendicularly into sea from the shore. This jetty has the same cross section over its entire length:
  - L-shape jetty: the trestle of an L-shape jetty covers the distance between the shore and the main part of the jetty in deep water. Usually the main jetty runs parallel to the shore, thus the trestle is perpendicular to shore and main jetty.
- Jetties for large tankers and bulk carriers are usually of the T-shape type.
  - T-shape jetty: generally with a loading/unloading platform or jetty part, which is connected to the shore by means of a trestle. The positioning of separate berthing and mooring dolphins around the main platform results in the typical T-shape.



• multiple berths, here 3 moored vessels shown
• 2 traveling loaders
• transfer platform for conveyor belts (left)
• existing trestle (left)
• future jetty/ trestle extension indicated

Figure 1-21 Top view on the main jetty or wharf of an L-type jetty

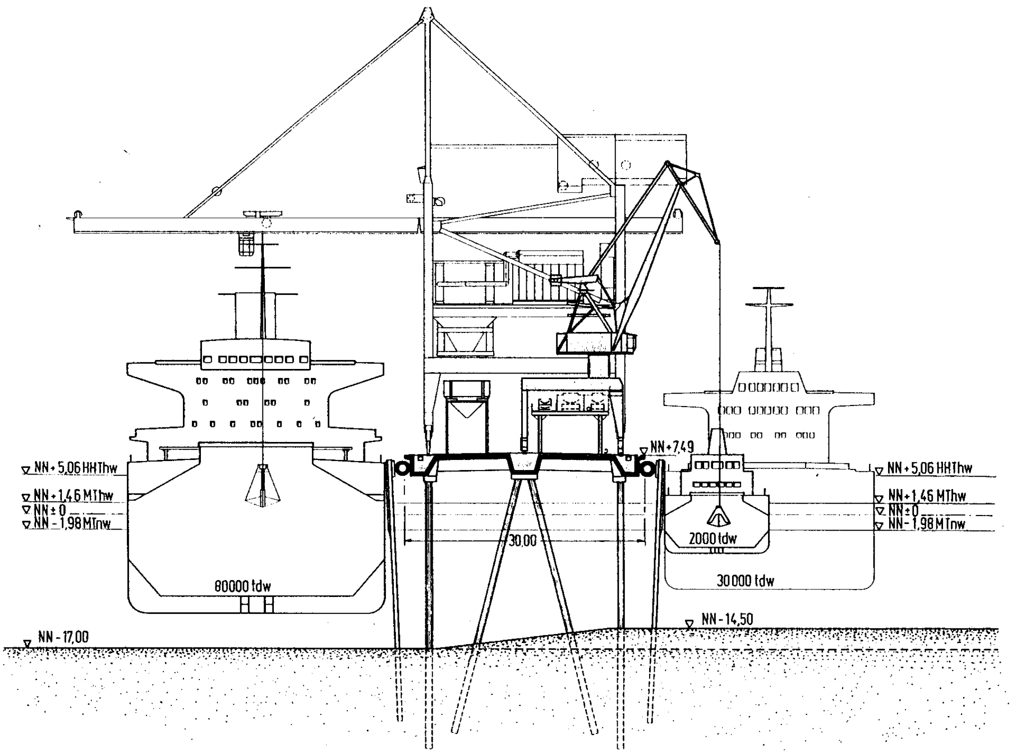


Figure 1-22 Cross section main jetty

Long trestles connecting the main jetty, which is in deep water, to the shore, may be relatively expensive. For this reason the trestle may be omitted by simply laying the pipes on the seabed. The crews of both the platform and the ships are transported to and from land by boat. An example of such a detached jetty is shown in Figure 1-23. Pay attention to the positioning of the mooring-posts. The berthing dolphins are placed a little bit forwards of the jetty and the mooring dolphins are placed further back to prevent collisions.

Figure 1-23 T-jetties; one main plus extension

Figure 1-24 Oil tanker at a jetty

1.7 Breasting & mooring dolphins, Approach walls

1.7.1 Breasting and Mooring dolphins

Breasting dolphins, known as berthing dolphins *(meerstoel of dukdalf)*, have the following function:

- resist the horizontal load caused by (stopping) the berthing ship.

The primary functions of a mooring dolphin *(bolderstoel)* are:

- provide space for bollards or Quick Release Hooks holding the ships' mooring lines;
- resist horizontal forces.

A secondary function of dolphins in general is:
- providing some space for either berthing and/or mooring equipment;

Breasting and mooring dolphins are relatively simple structures used for mooring a ship. The structures have no functions other than slowing down ships (berthing) and keep the ship in position when it is moored. Typically dolphins are placed in harbours and next to jetties
Breasting and mooring dolphins appear in various shapes. For instance, there is the simple single berthing or mooring pile, generally a steel pile with a large diameter, or dolphins consisting of a larger number of raked piles *(geschoorde palen)* that are connected together by a capping beam or block. The decision to use a single pile or multi-pile dolphin depends mainly on the expected loads and construction costs, some times on the available space for the structure.

The performance of a breasting dolphin depends on its ability to dissipate the ship's berthing energy.

Figure 1-25 Flexible dolphin in the Port of Rotterdam

Flexible dolphins are generally single, large diameter, vertical piles, see Figure 1-25. A relatively small fender, and opposite to that, a relatively large fender panel is assembled to the top of the pile. Note:
- by far the largest part of the pile is either under water or embedded in soil;
- chains are used to secure the fender panel in case of fender collapse.

The ship hits the fender panel, resulting in deformation of the fender, and a relatively large pile top displacement because the pile is loaded horizontally. The fender and pile return in their original shape or position. Due to all the initial and reversed elastic deformation and displacement, the surrounding water being pushed back and forth, the kinetic energy of the ship will be dissipated. Overloading, the general cause for failure of a flexible dolphin, may result in an irreparably damaged fender, yielding of steel sections in the pile or rupture of the ground, which is most severe and causes the pile to tumble over, rendering the main structure unprotected for berthing.

In order to berth large sea-going ships it is more common to use a rigid breasting dolphin, a number of piles with a (concrete) cap or coping on top of the piles, providing the overall rigidity. A large fender assembled to the pile cap should have sufficient deformation capacity to dissipate all the kinetic berthing energy. The piles of the dolphin are driven raked or inclined into the ground, constituting yokes *(jukken)* that are relatively stiff. The horizontal berthing load is mainly transferred into compression and tension forces in the piles, which is an advantage from steel use point of view. The weight of the concrete superstructure will reduce the tensile forces in the piles having to resist tension force, which is more important for reduction of the embedded length of the pile in the soil than for reduced use of steel. The rigid dolphin is far stiffer than a flexible dolphin, therefore a larger fender will be required because this fender has to dissipate nearly all the berthing energy alone.

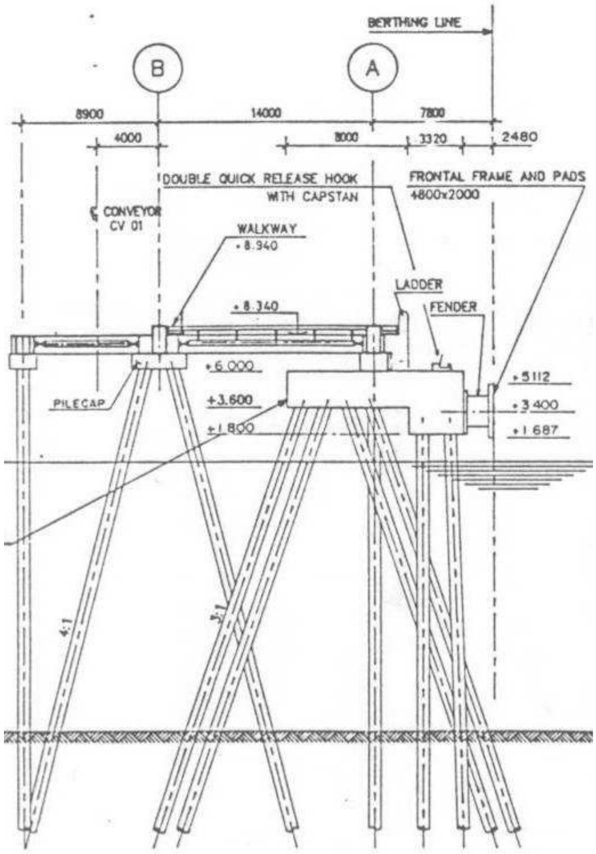


Figure 1-26 Rigid dolphin combining berthing and mooring function

Figure 1-27 Cellular cofferdams used as mooring dolphin

Although the idea of mooring is to keep the ship in a fixed position, the ship should not be tied rock solid to mooring dolphins, think e.g. about tidal variation or the loading/unloading. Usually quite a fair length of the mooring line will be synthetic, lines will be tended manually or automatic, hence flexibility is included in the mooring system. Contrary to the profitable use of flexibility for breasting dolphins, caused by considerable load reduction, little is gained by providing more flexibility to mooring dolphins.

Cellular cofferdams, see Figure 1-27, are a competitive alternative for other mooring dolphin structures. The relatively light and flexible sheet piling, convenient for construction, and the soil in the cell, combine to a stiff system. The stability of the circular cofferdam is determined mainly by the shearing friction of the soil in the cell and the supporting subsoil.

1.7.2 Approach walls

Approach walls are also known as guidance walls *(geleidewerken)* or guard walls *remmingwerken)*. The primary functions approach walls are:

- guiding ships in the desired direction to prevent collision with the hydraulic structure
- slowing down and, if needed, mooring ships.

Like single breasting and mooring dolphins, approach walls are relatively simple structures. Their main function is to guide ships into the desired direction and to slow them down, preventing collisions with the hydraulic structure (lock, pier, etc.). They are also used to create waiting positions (berths) for ships or barges that have to wait for a lock or for an empty loading/unloading berth. Approach walls are more or less linear structures, generally consisting of a row of piles or dolphins, connected by waling beams. If a ship collides with a fender system, the system responds like an elastic-supported beam, allowing the load to be spread across the various piles or dolphins.

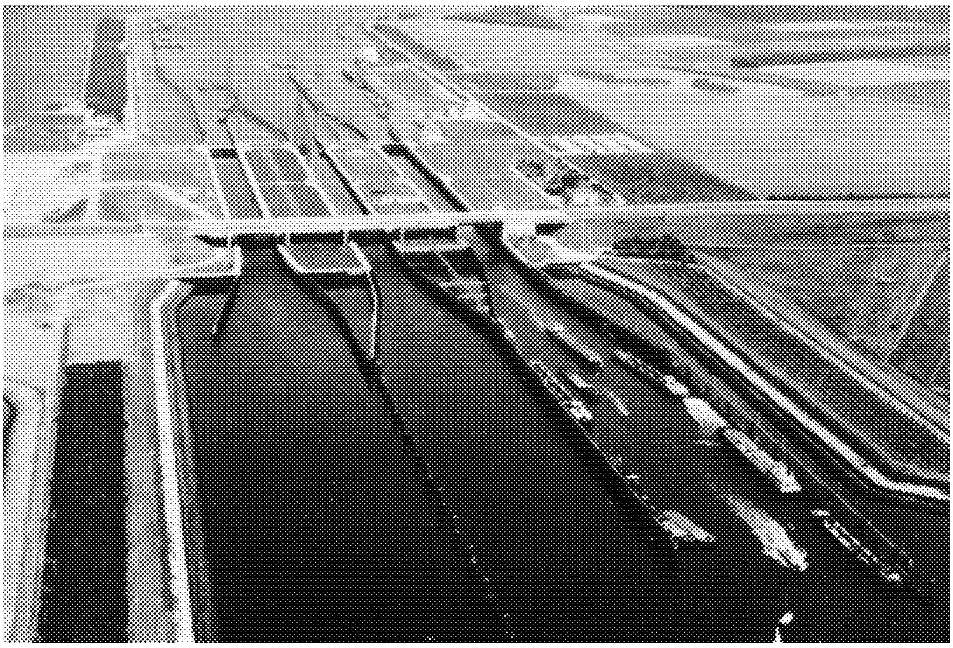


Figure 1-28 Approach walls near a lock

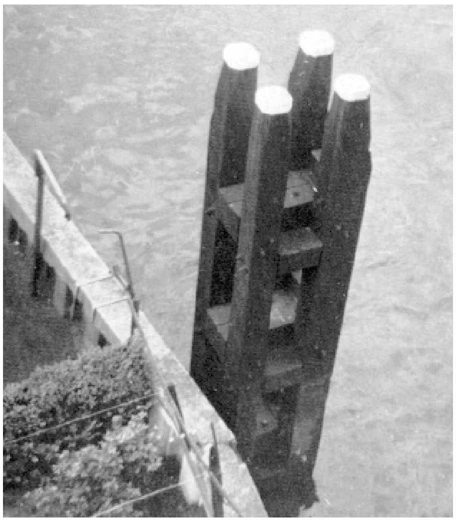


Figure 1-29 Dolphin with wooden piles

1.7.3 Fender

The fender's function is to absorb the kinetic energy of a mooring ship in such a way that the force on the structure and therefore the force on the ship remains below an acceptable limit.
In the previous section we considered the capacity to deform of a mooring structure. The amount of energy that needs to be absorbed is $E = \int_0^{u_{max}} F(u)\delta u$, in which $F(u)$ is the force and u_{max} is the entire displacement. This leads to the conclusion that for a given amount of energy, the force is determined by the total displacement and therefore the stiffness of the structure. In principle, the more flexible the structure is, the smaller the force is and the larger the displacement. The relationship between the force and the displacement is determined by the spring characteristics of the structure. The spring characteristics of a linearly elastic structure are linear. In the case of rubber fenders, the spring characteristics are generally not linear. An example for a rubber cell fender is given in Figure 1-30.

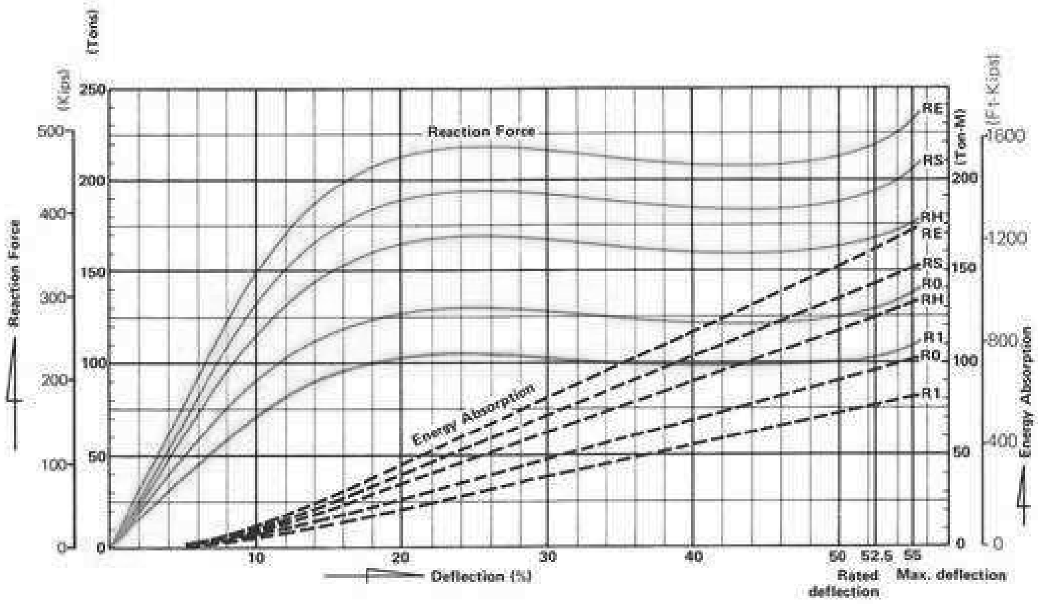


Figure 1-30 Spring characteristics of Cell-fenders with different rubber compound

In the spring characteristics one can observe a flat section in the middle in which the force provided by the fender is hardly affected by the compression. This flat section is the effective working range of the fender. The force of the fender on the structure is in this case more or less the same for any amount of collision energy within the effective working range. By selecting a suitable fender, the force of the berthing ship on the structure can be kept more or less constant.

Fender panels are used to spread the concentrated force of the fender over a larger part of the ship's hull. The distributed load on the ship should not exceed 400 kN/m^2 to prevent damage to the tankers. It is important to take into account the kind of ship and its cargo when selecting a fender type. For instance, at mooring places for Nafta tankers it is important to keep the forces on the ship small, resulting in soft fenders. A disadvantage of such soft fenders is that the moored ship will move more.
The following aspect plays an important role in the decision of the dimensions of structure:

- the kinetic energy must be absorbed by the fender, piles and ground;
- the pressure on the ship's plating must stay within an acceptable range;
- the deformation of the structure may not lead to any contact points with the ship other than at the fenders or fender panels.

1.8 Building pit, cofferdam and construction dock

The primary function of building pits, cofferdams and building docks is to retain soil and to prevent the flow of groundwater, in horizontal and vertical direction, in order to create dry space to work.

1.8.1 Building pit

A building pit is an excavation with slopes and in which if necessary the groundwater level has been lowered until below ground level of the building pit. A building pits is situated on definitive site of the structure. The purpose of a building pit is to create dry land at the foundation level of the structure. A building pit needs relatively a lot of space, because differences in height areas are held in place by earthworks. The pit can be kept dry using pumping systems. However, pumping does have some disadvantages. Often objections are heard, because pumping can dry out the surrounding area or make it brackish. Also the groundwater might be polluted and therefore it may not be discharged onto the surface water. Pumping can also cause settlement in the surrounding area. This settlement occurs due to a drop in water pressure in ground layers sensitive to settling. Using (airtight) return pumping can largely eliminate these problems. Nevertheless, in many cases pumping permits are only issued for a maximum of one year. This is not long enough for big construction projects.

Figure 1-31 Building pit; In the waterway (left), on land (right)

1.8.2 Cofferdam

In many cases there is no space for a traditional building pit or pumping is forbidden. In these cases a vertical walled building pit, a cofferdam can be used. Often the sheetpile walls have to be anchored, not always, frequently convenient use is made of the wall at the opposite side by putting struts in between the walls. Sometimes, it is also possible to integrate the cofferdam in the final construction. Examples are entry ramps to tunnels and in lock chambers in which the cofferdam walls form the walls of the final structure. This is an efficient use of space and materials.

A watertight floor is either present in the shape of impermeable ground layers (clay or peat) or by a layer of underwater concrete cast in-situ during construction. A floor made of underwater concrete will always be more expensive than using natural ground layers, assuming these are thick and heavy enough to prevent the floor from bursting up. The stability of an underwater concrete floor normally must be secured using tension piles or ground anchors. An underwater concrete floor can also be integrated within the final structure by providing the underwater concrete with reinforcement steel or steel fibers. This way the underwater floor can act as a full construction floor, however, this definitely not common procedure.

Figure 1-32 Cofferdam or braced excavation

1.8.3 Construction dock

When constructing tunnel sections for a submerged tunnel or big elements such as large precast piers, generally, a building site is used which can be flooded so that the elements can be transported by water once they are finished. This type of building pit is called a construction or building dock. A building dock is situated beneath the average water level minus the draught of the elements and it is kept dry by a pumping system and/or by sealing off the construction area by means of impermeable ground layers. The separation between building dock and surrounding water is maintained by a water retaining structure. This water retaining structure is (partly) removed when the elements are finished, in order to transport the elements to the definite site by water. Often a caisson or a rolling gate with a floater will be used as a closing element or gate of the construction dock.

Figure 1-33 Tunnel sections for the HSL railway line in a construction dock

The fact that the elements remain only temporarily on the construction site is the typical characteristics of a building dock. Generally the size of the elements in a building dock is quite large, obviously the size of the construction dock is even larger. This means a large amount of space has to be reserved for a building dock. This generally means great expenses. Especially if a large number of elements has to be realised, it may be more economical to search for alternatives.
Tunnel sections for instance can also be built on a slope or on a conveyer belt after which they are directly transported to their definite sites by land or by water. It is also possible to reuse a smaller building dock several times, to produce the necessary elements in phases.

1.9 Dry dock, floating dock

A dry dock or a floating dock allows ships to be repaired out of the water without putting the ship on land. A dock consists of a chamber with an open and a closed end. The open end has a gate that can be closed, sealing the dock watertight. By pumping the water out of the chamber, the dock will become dry. It is important that the walls and the floor are also watertight. The foundation floor of the dock should counter the upward force under the dry dock. This is generally achieved by using tension piles or anchors. A dry dock construction resembles a lot the one of a navigation lock, however with only one entrance.

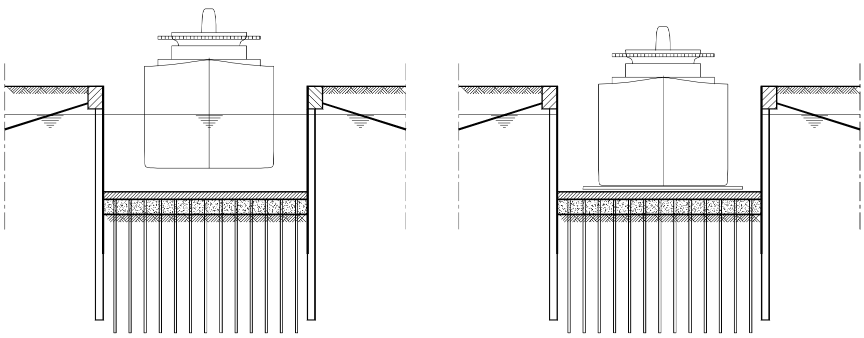


Figure 1-34 Dry dock

Another type of dock is the floating dock. The essential difference between a dry dock and a floating dock is that the latter will start to float when the water is pumped out, while the former will remain in place when being pumped dry. The closure mechanisms could consist of one of many types of gates used in navigation locks.

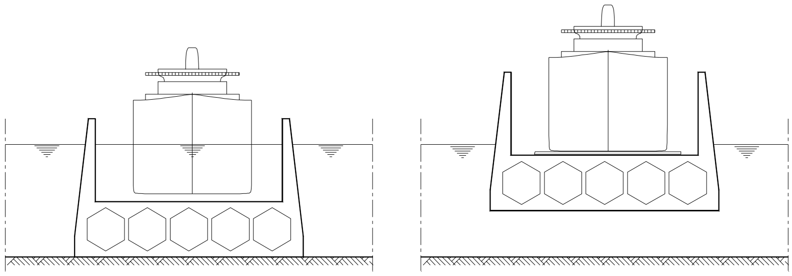


Figure 1-35 Floating dock

1.10 Road in continuous cut, approach road

The objective of constructing a road in a continuous cut is:

- to reduce discomfort caused by traffic (in residential areas);
- to realise free crossings.

Approaches to tunnels, rail and/or road tunnels, have a lot in common with roads in continuous cuts with the marked exception that over a certain distance, being as short as possible, the approach has to ‘bridge’ a level difference between top of the relevant water defense structure and bottom level of the tunnel. Approaches are sloping down or up. Approaches or entry ramps do not have roofs, see Figure 1-36, however, they may have some struts between the opposing walls in the deeper parts.

Figure 1-36 Prinses Margriet tunnel with approaches

Because the road is below ground level, it is necessary to create a watertight structure in order prevent flooding of the cut, or to pump it dry continuously.
Possible structures for roads in continuous cuts and approaches are a concrete ramp or a diaphragm wall with of without anchors. If necessary, a watertight floor can be constructed between the diaphragm walls. **Error! Reference source not found.** shows some examples. An advantage of a watertight structure is that a groundwater dewatering system is not needed, because only rainwater has to be pumped out.

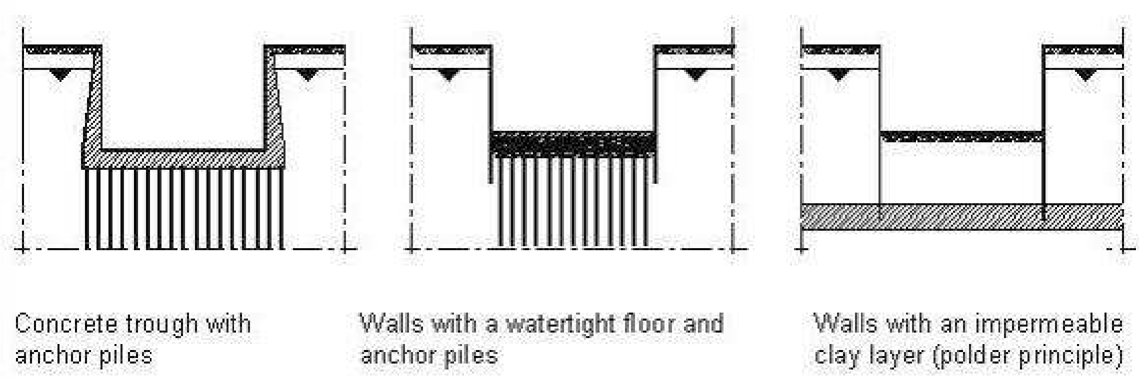


Figure 1-37 Rigid or inflexible structures

Figure 1-38 Underwater concrete and tension piles in the approach to a tunnel

Another method to achieve a watertight structure is using geotextiles (polder principle). The geotextile membrane is impermeable, but cannot absorb any load without large deformations or, worse, being ruptured. The soil being placed on top the membrane can ensure the stability of the membrane and prevent damage due to loads. Figure 1-39 shows some examples.

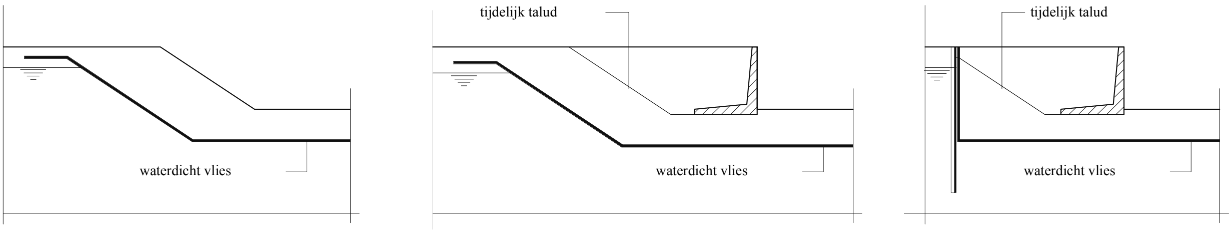


Figure 1-39 Possible membrane structures

1.11 Tunnel, aqueduct and culvert

The purpose of a tunnel is to allow traffic to cross unhindered by means of an underground or underwater connection between two points. A preliminary classification of tunnels can be made according to the type of transportation in the tunnel, such as:

- tunnel for slow moving traffic (pedestrian, cyclist or tractor);
- tunnel for other moving traffic (cars, trucks, etc.);
- tunnel for trains;
- tunnel for pipes and cables.

It is also possible to classify a tunnel in the method of the construction of the tunnel, as:

- tunnel constructed in situ from ground level (in a building pit or cofferdam)
- "cut and cover" tunnel (little space required, little disturbance)
- immersed tunnel
- bored tunnel

The list above runs from the cheapest to the most expensive type, as long as there are no big obstacles. In the case of rivers, in situ and “cut and cover” tunnels are practically impossible. Passing buildings or large cables and pipes under roads bored tunnels are often cheaper.

1.11.1 In situ tunnel

Examples of in situ tunnels are the Velser tunnel and the Schiphol tunnel. The Velser tunnel was built at a crossing between a road and a waterway. In order to let ships pass during the construction of it, the tunnel was built in phases, constantly leaving 50% of the width of the fairway open for navigation.

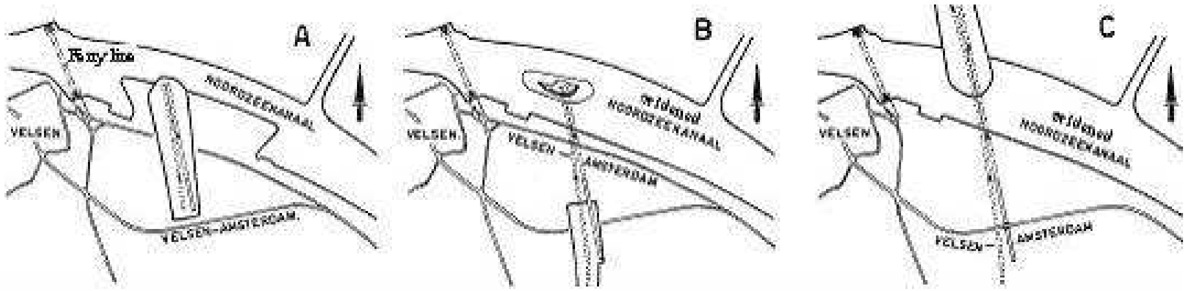


Figure 1-40 Phased construction of the Velser tunnel

During the construction of the tunnel under the Haarlemmermeer ring canal (A4 near Leiden) the entire ring canal was temporally diverted in order to build the tunnel.

1.11.2 Cut and cover tunnel

In the “cut and cover” method, the first action is to install the walls into the ground. These walls could be anchored or unanchored diaphragm walls or sheet piling. Being only excavation between walls, the all-over construction area will be smaller than a traditional building pit. Depending of the situation a watertight floor will be constructed after excavation. The walls being placed in an (almost) impermeable ground layer, the watertight floor can be omitted. In this case the tunnel is kept dry by pumping out the seepage water. The roof must be constructed, before a ground filling can be started.

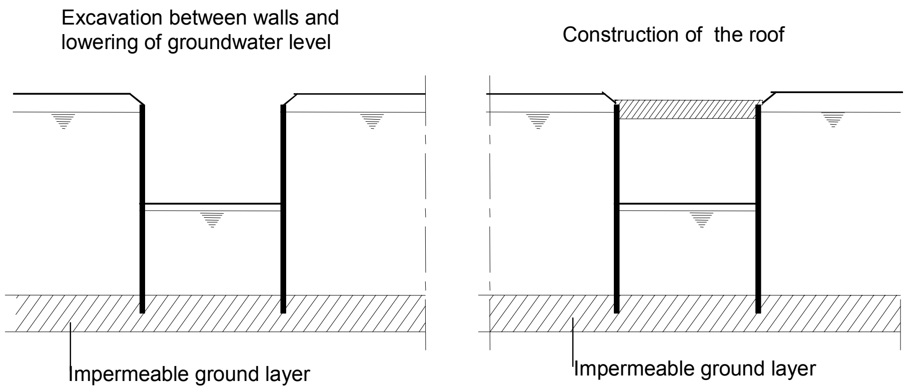


Figure 1-41 Cut and cover tunneling method

In urban areas the order of construction usually is changed to cause as little disturbance as possible to the surroundings. In such a case the walls are the first to be constructed, the roof is put on top of it, then excavation and construction of the tunnel floor take place under the roof. The biggest advantage of this method is that the activities at ground level are only disrupted for a short part of the construction period. A problem often encountered is the absence of sufficiently impermeable (horizontal) ground layers being between the walls. Solutions could be to pump the inflowing water out, often unacceptable in urban areas, to pressurise the construction area, dangerous work and expensive, or to seal the area with an arc of grout. The latter has proved not to be as successful as expected in a number of projects.

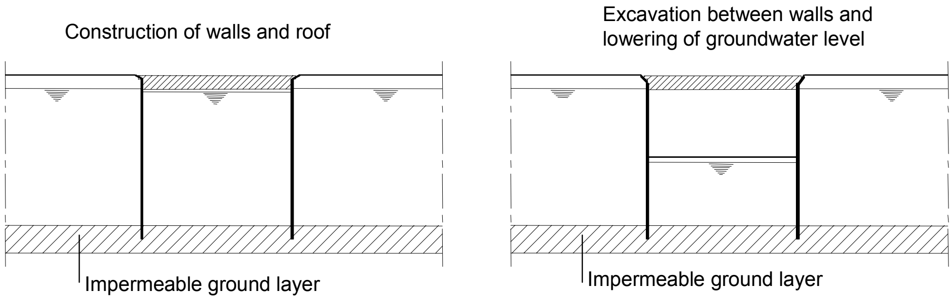


Figure 1-42 Alternative phasing for the cut and cover method

1.11.3 Immersed tunnel

At the crossing of a road and a waterway, the part of the tunnel that lying under the waterway can be sunk into place. In this case the tunnel is prefabricated in sections in a building dock and the inconvenience to the navigation is significantly reduced compared to a tunnel built in situ. The tunnel sections are equipped with a bulkhead, allowing them to remain afloat without ballast and they can be transported by water. The sections are transported by water to the location of the tunnel and sunk to the bottom of a trench dredged before.

Figure 1-43 Immersed tunnel sections at the Piet-Hein tunnel, Amsterdam

The sections are, at one end, provided with a rubber seal (Gina profile). During the sinking process the sections are placed together, with the rubber seal connecting the two sections. Pumping out the water between the two joined sections, the sections are pressed against each other by the water pressure on the other ends of the sections, creating a watertight seal between the sections. The vertical positioning of the tunnel sections is done using concrete slabs, which are placed beforehand. The sections being placed, sand is sprayed in beneath them and the trench is refilled. Occasionally it can be necessary for the tunnel to have pile foundations. In such a case a pile foundation with the slabs or beams will be made first and after that the tunnel sections will be sunk down on the foundation.

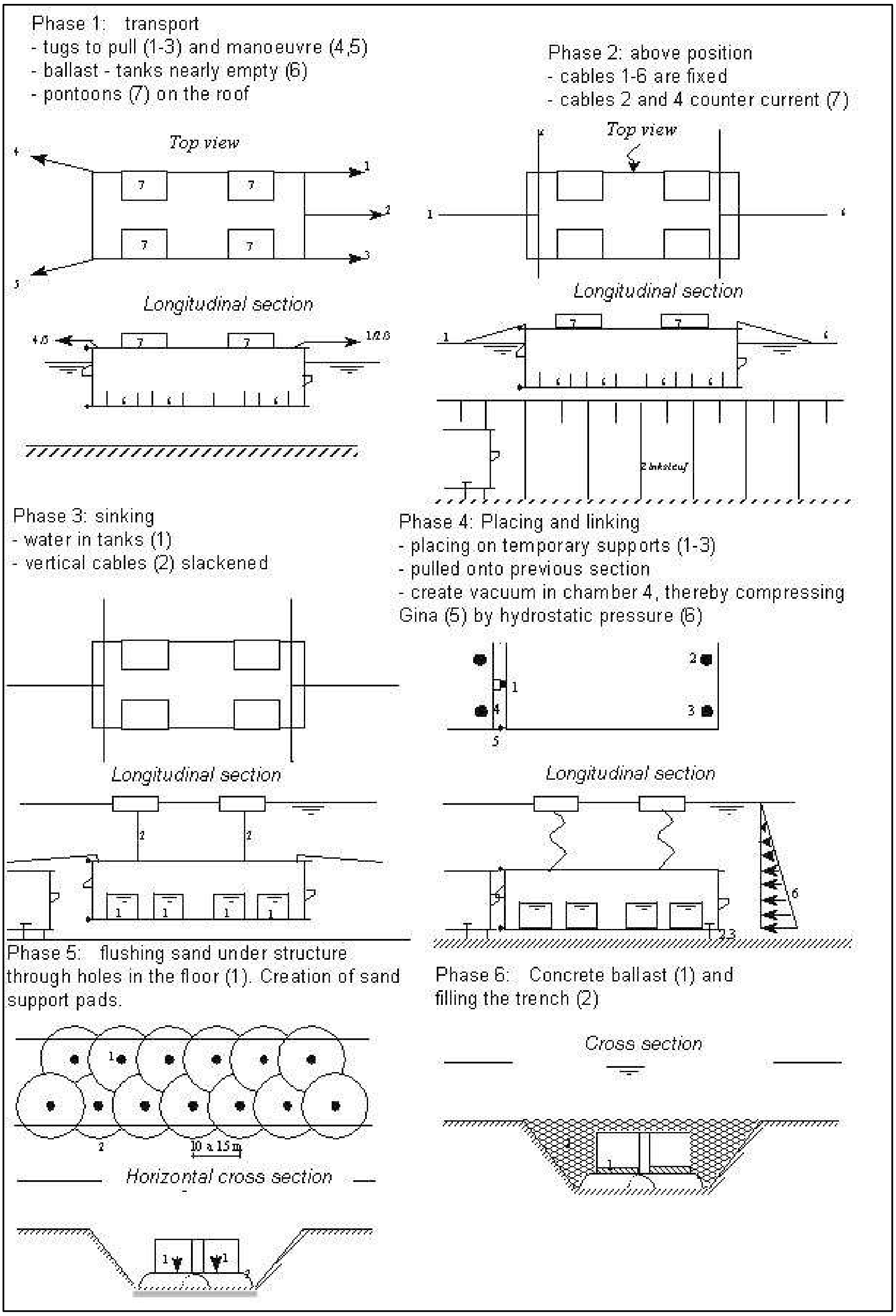


Figure 1-44 Transport and positioning of an immersed tunnel element

This method of tunnel construction cannot be applied in areas where the tunnel is above water level; in that case the tunnel has to be constructed in situ. The structure between the entry ramp and the sunken sections is called a transitional section. In the Netherlands this method of sinking is well tried and tested method and often used.

1.11.4 Bored tunnel

In the Netherlands another unusual method of construction, is boring a tunnel. Bored tunnels are widely used abroad in very cohesive grounds and in rock. In Japan, Germany, England and recently also in the Netherlands bored tunnels have been built in weak soil types (sand and clay). In the Netherlands the first bored tunnels of large diameter are the second Heinenoord tunnel, the Botlek railway tunnel and the Westeschelde tunnel.
The advantages of bored tunnels are:
- Little or no space is required for the building site;
- There is very little inconvenience at the site.

The disadvantages of bored tunnels are:
- If the drill breaks down, sometimes it will be necessary to repair it at ground level;
- At the start shaft of the tunnel more space and hinder may be expected;
- Great logistical problems (all the soil and the tunnel sections must be transported through one tube!);
- High costs.

There are several methods of boring techniques not being discussed further in this book. The student is advised to follow the courses ‘Bored and immersed tunnels’ and ‘Underground space technology’.

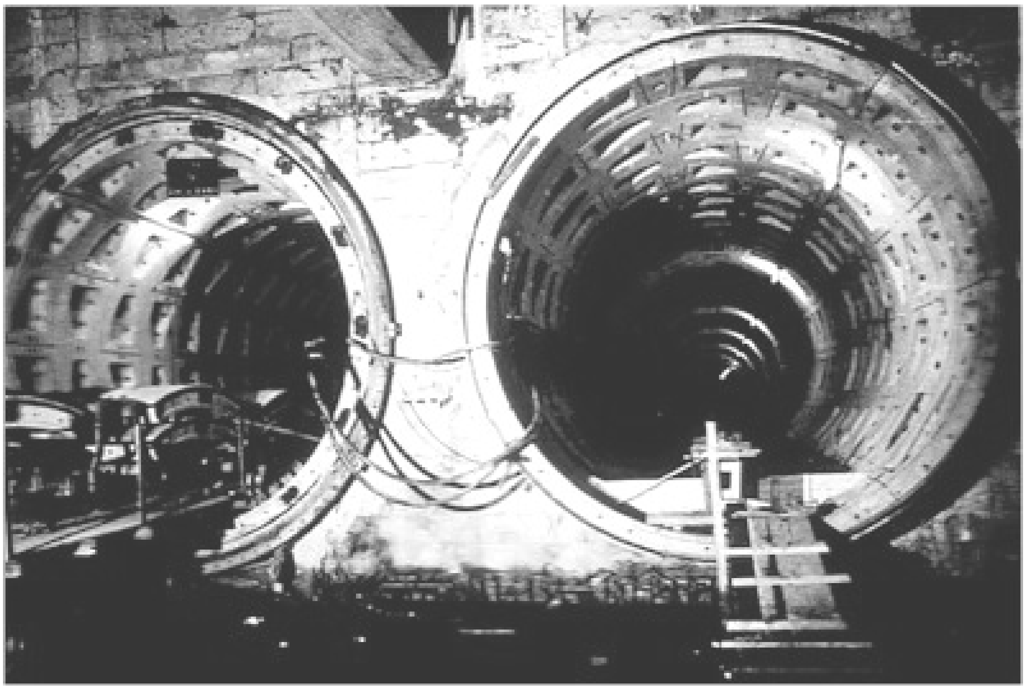


Figure 1-45 Beginning and end of a bored tunnel. The segments are clearly visible

1.11.5 Aqueduct

The function of an aqueduct is to support a canal. Aqueducts are used in:
- drinking water canals;
- irrigation canals;
- shipping canals.

The main difference between an aqueduct and a tunnel under a waterway is that in the case of an aqueduct the water is led into a trough over the other infrastructure. In the case of a tunnel the crossing infrastructure is led under the river in a tube. Of course aqueducts are also used to cross valleys.

If the waterway is less wide than the crossing road, an aqueduct could be preferred above a tunnel. In the Netherlands are some aqueducts. For example:

- the aqueduct near Leiden (the crossing between the Haarlemmer ring canal and the A4);
- the aqueduct near Zevenhuizen;
- the Gouwe aqueduct near Gouda.

Figure 1-46 Gouwe aqueduct

An example of an aqueduct crossing another waterway can be found at the Elbe crossing near Magdeburg in Germany.

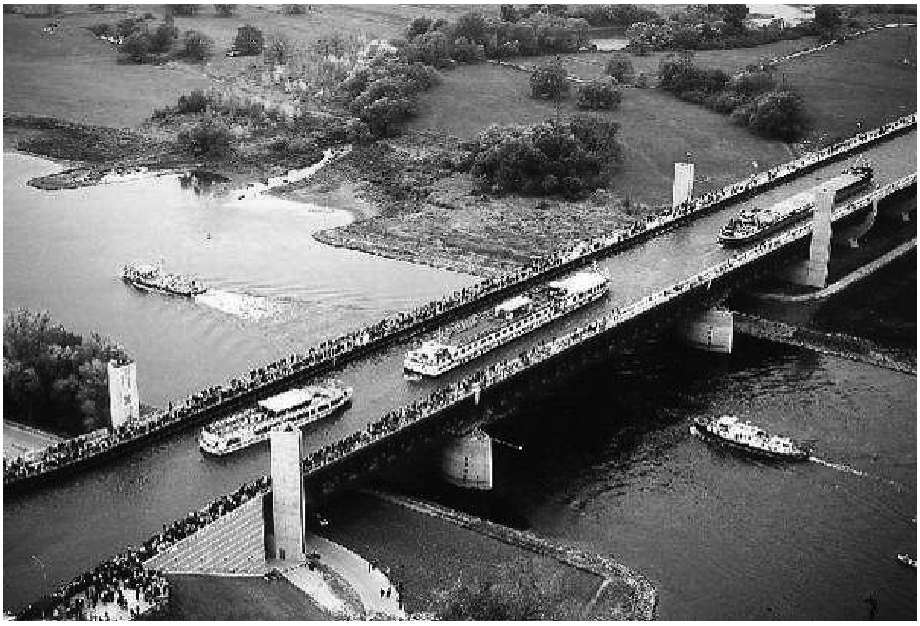
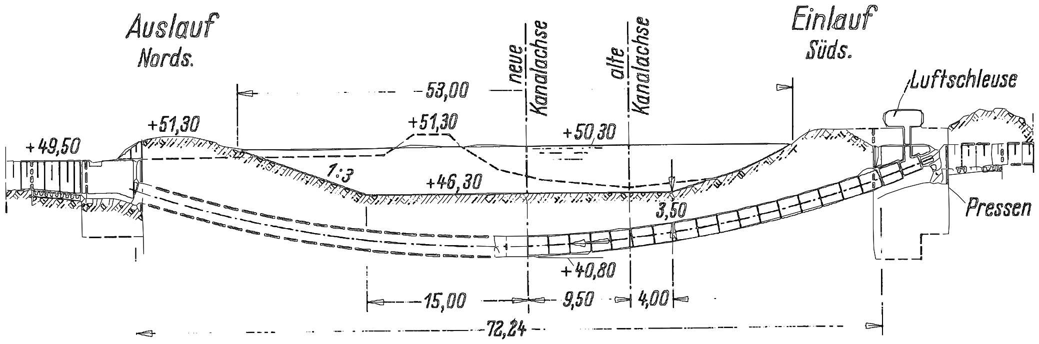


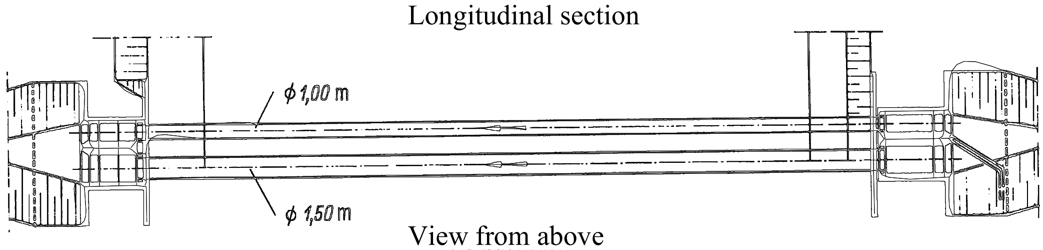
Figure 1-47 Aqueduct near Magdeburg - Germany

1.11.6 Culvert

In case of a tunnel and an aqueduct, the waterway is situated above the level of the crossing infrastructure. In many cases it can be more attractive to lead the waterway under the road. This can be achieved for instance by a bridge, fixed or moveable. It can however also be achieved with a culvert. A culvert is in fact a tunnel under the road through which the water can flow, thereby crossing the road. It is also possible that two waterways can cross each other by means of a culvert, see **Error! Reference source not found.**, gives an example of this.



Longitudinal section



View from above

Figure 1-48 Culvert crossing another waterway and a road

A culvert can also be used at the crossing of a waterway and pipes.

1.12 Dewatering gate, stop lock, siphons and pumping-stations

1.12.1 Dewatering gate

Dewatering gates are designed to discharge an excess of water in the polder reservoir into a river or the sea. They are part of the water defense system. Accordingly, the functions of a dewatering gate are:
to transport water through the water retaining or flood defense structure;
to retain water during high water level;
to separate fresh water from saltwater.

Water can be discharged through an open (channel) or closed tube (culvert). The water can be retained using various types of gates; in the past mitre gates generally have been used. Because of the difference in water pressure, the gates are kept shut during high water level and they open automatically when the water level drops. Nowadays it is more preferable to be able to control more accurate the water level in the polder. This requires controlled discharge of water. As a consequence, gates are chosen which can be opened and shut in a controlled procedure and retain water in both directions. Examples of such controlled dewatering gates are the Afsluitdijk near Den Oever and the Eider dam near Kornwerderzand.

This type of sluice can also be used to let water in when polder water levels are dropping or when water levels in the inner waterways are too low. In this case the structure is referred to as an intake sluice. In arid countries such intake sluices are quite common in irrigation systems.

Figure 1-49 Dewatering gate at or in the Eiderdam

1.12.2 Stop lock

A stop lock is designed to flush a waterway, canal system or a harbour. Flushing is done to remove sediment and contamination. The structures of a stop lock and dewatering gate are very similar. A stop lock is used to dam a reservoir in which rainfall from the surrounding area is collected. When the water level in the reservoir is high enough, the gates are opened to flush the downstream waterways. In principle a stop lock fulfills similar functions as those fulfilled by a dewatering gate, keeping in mind though that a stop lock is always opened under conditions of water pressure difference. For this reason mitre gates will not be used for stop locks. The Haringvliet barrier can be considered as a stop lock.

Figure 1-50 Haringvliet stop lock

1.12.3 Siphon

Another method to discharge water through a water retaining structure is to use a siphon. In this case, water is not transported through the water retaining structure, but over its top. A big advantage of this system is that it does not require a closure mechanism. Filling the siphon entirely with water induces the flow of water in the siphon. The piezo-metric height difference on either side of the water retaining structure will cause the start of a water-flow in the siphon. It is sufficient to let air into the siphon to stop the flowing process. For this reason, the openings of the siphon must be situated far below water level in order to prevent that free air can penetrate the siphon system. This method has been applied in Bergen op Zoom.

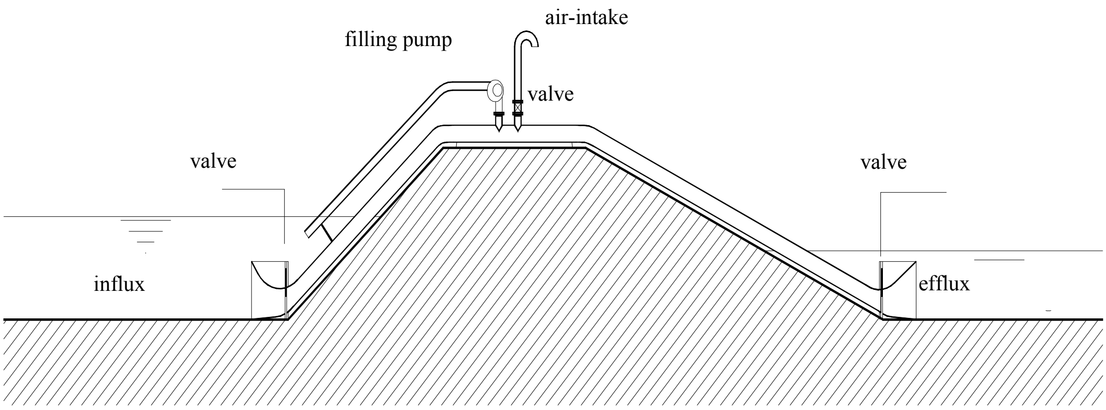


Figure 1-51 Schematic representation of a siphon

Both dewatering gates and siphons only work in the direction of the lower water level. If it is necessary to discharge water in the direction of the higher water level, sluices or siphons can not be used. In that case the use of a pumping station is the only option.

1.12.4 Pumping stations

Pumping stations are designed to transport water from lower level areas to higher areas. Because polders are situated below the water level of surrounding water, it is necessary to pump the surplus of rain and seepage water out of the polder using pumping stations. Figure 1-52 and Figure 1-53 show the renovated (1998) and enlarged Rozema pump station in Termunterzijl. The pump station discharges water from the polder through a culvert in the dike to the Waddensea. The four pumps have a total capacity of 2700 m^3 per minute. Construction of the pump station building alone took almost € 20 million. Changes to provide everything necessary for its hydraulic function took more than € 20 million (1998 prices).

Figure 1-52 Pump station Rozema near Termunterzijl

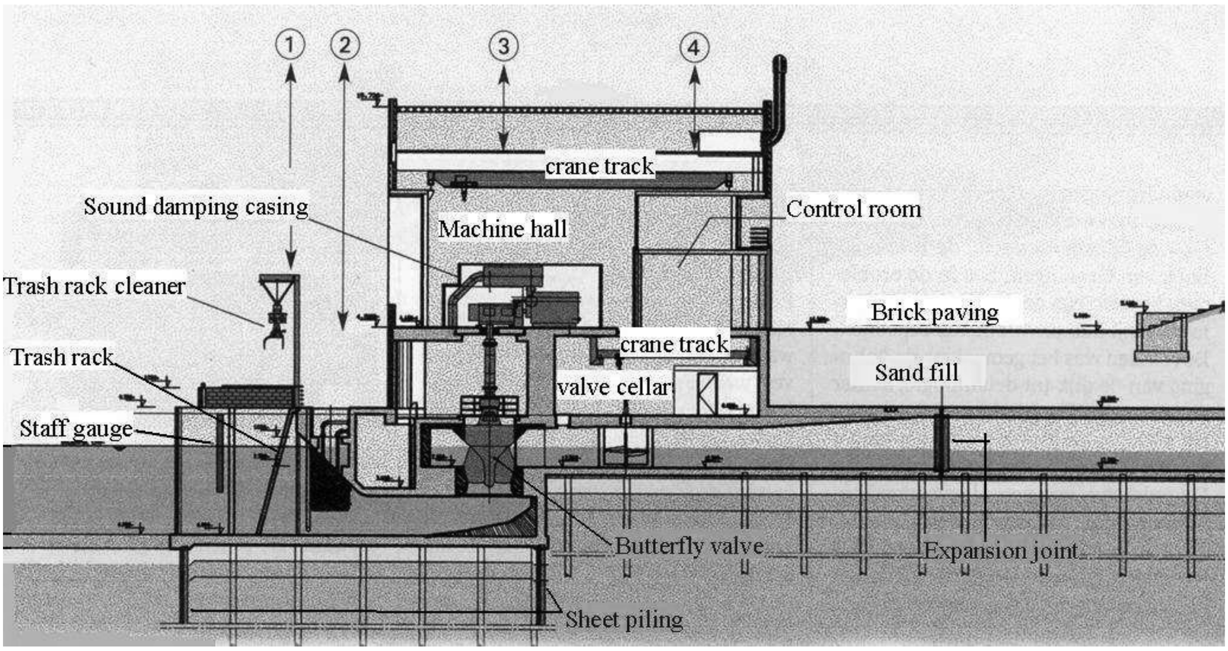


Figure 1-53 Cross section pump station Rozema and culvert through dike (partial)

1.13 Navigation lock, inclined plane and ship-lift

The purpose of lock facilities is to bridge a water-level difference between two water sections for navigation, whilst the difference in water levels remains. Lock facilities are therefore designed to:

- retain water;
- allow horizontal transport of ships;
- transport ships vertically.

The vertical and horizontal transport of ships can be achieved in two fundamentally different ways, viz.:

- by adjusting the water level in a closed chamber;
- by transporting the ship and its surrounding water together vertically in a closed chamber.

The first concept is used for (navigation) locks, the latter in a ship-lift or an inclined plane.

1.13.1 Navigation lock

Navigation locks are used for instance when shipping has to pass a weir or water retaining structure. Logical locations for locks are:

- in a river next to a weir;
- at the beginning and/or at the end of a canal with a regulated water level;
- near the coast as a passage way through the primary water retaining structure

A navigation lock consists of at least a lock chamber closed by closing elements located in the so-called lock heads. Different types of closing elements can be used, such as mitre gates, lift gates, rolling gates, radial gates, shutter gates and sector gates.

Raising or lowering the water level in the chamber can be achieved by letting in or out water by opening the gates or shutters in the gate. In more advanced locks this is achieved by transporting water through a system of by-pass pipes. Pipe systems in the floor and walls of locks are enabling a more controlled supply and discharge of water resulting in more convenient ship handling during emptying and filling the lock chamber.

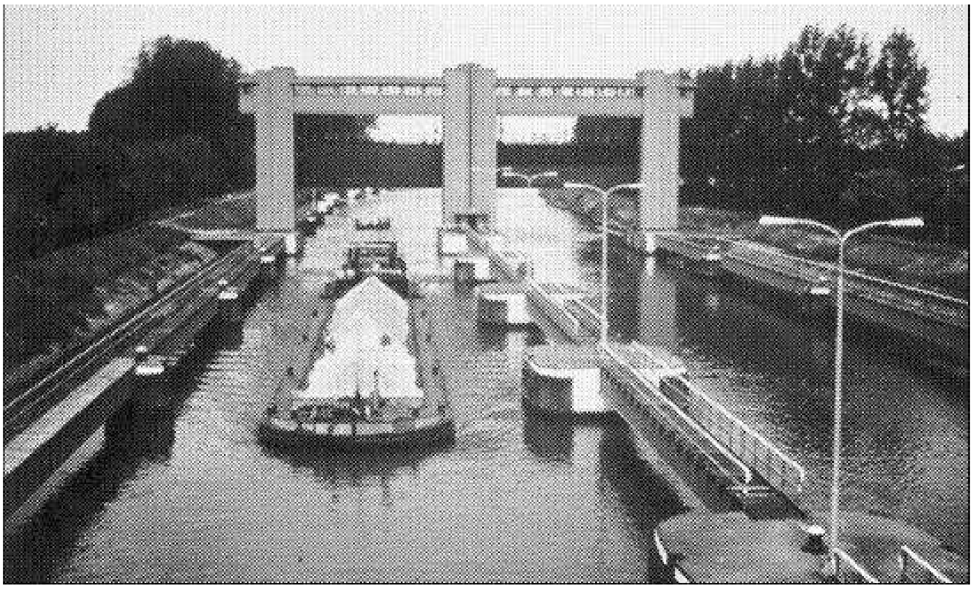


Figure 1-54 Navigation lock for inland navigation

During the locking cycle, water is transported from the upstream side to the downstream side. Sometimes it can be troublesome when, in every locking cycle, a large quantity of water is lost upstream or added downstream. In coastal areas salt water can infiltrate the river during locking cycle under the condition that sea level is higher than river level. In these cases it is possible to use storage basins adjacent to the lock, which are filled with water when the chamber is being emptied. This water can be used again to fill the lock chamber in the next cycle. The loss of water is reduced in this way. In the case of great water level difference a series of locks can be used to reduce the loss of water. An example of this are the seven locks between the Ottawa River and the Rideau Canal.

In the Netherlands navigation locks are used to lift and lower ships up to about 12 meters and are subjected to the condition that enough water is available to compensate the water loss caused by the locking process. Abroad, locks are used with still bigger water level differences. In these situations special storage basins usually are applied to reduce the loss of water.

The storage basins are situated at different levels. The water loss is equal to the amount of water between the downstream level and the water level (after filling) of the lowest storage basin.

A ship-lift and an inclined plane can raise or lower ships in case of big water level differences in one cycle with little or no loss of water; this goes together with little loss of navigation time.

1.13.2 Inclined plane

An inclined plane makes use of the slope of the surroundings. There are two types of inclined planes. In the case of the first type the water in a canal (with one or more ships) is pushed up and down along a steep slope by means of a watertight bulkhead. This type is called a “Pente d’eau” (water slope). In the case of the second type the ships and water are pushed up and down a slope in a chamber (“plan incliné”).
The chamber of a ship-lift or an inclined plane is sealed at both ends by closing elements. The adjoining canals are also equipped with closing elements.

Figure 1-55 Inclined plane in lateral direction

1.13.3 Ship-lift

In the case of ship-lifts the chamber is filled with water and a ship and then it is transported vertically by means of screw pumps, plungers or cables.
Applying plungers or floating bodies a lot of space is required beneath the structure, see Figure 1-56. The ship-lift is operated by letting air into or out of the plungers, causing the floating bodies to rise or to sink. Irregular settlement of the structure has to be avoided, because of the danger that the plungers will get stuck against the walls.
In the case of a ship-lift with cables, all moving parts are situated on the surface area. The weight of the chamber can be compensated by counterweights or by a second chamber. The latter is shown schematically in Figure 1-56.

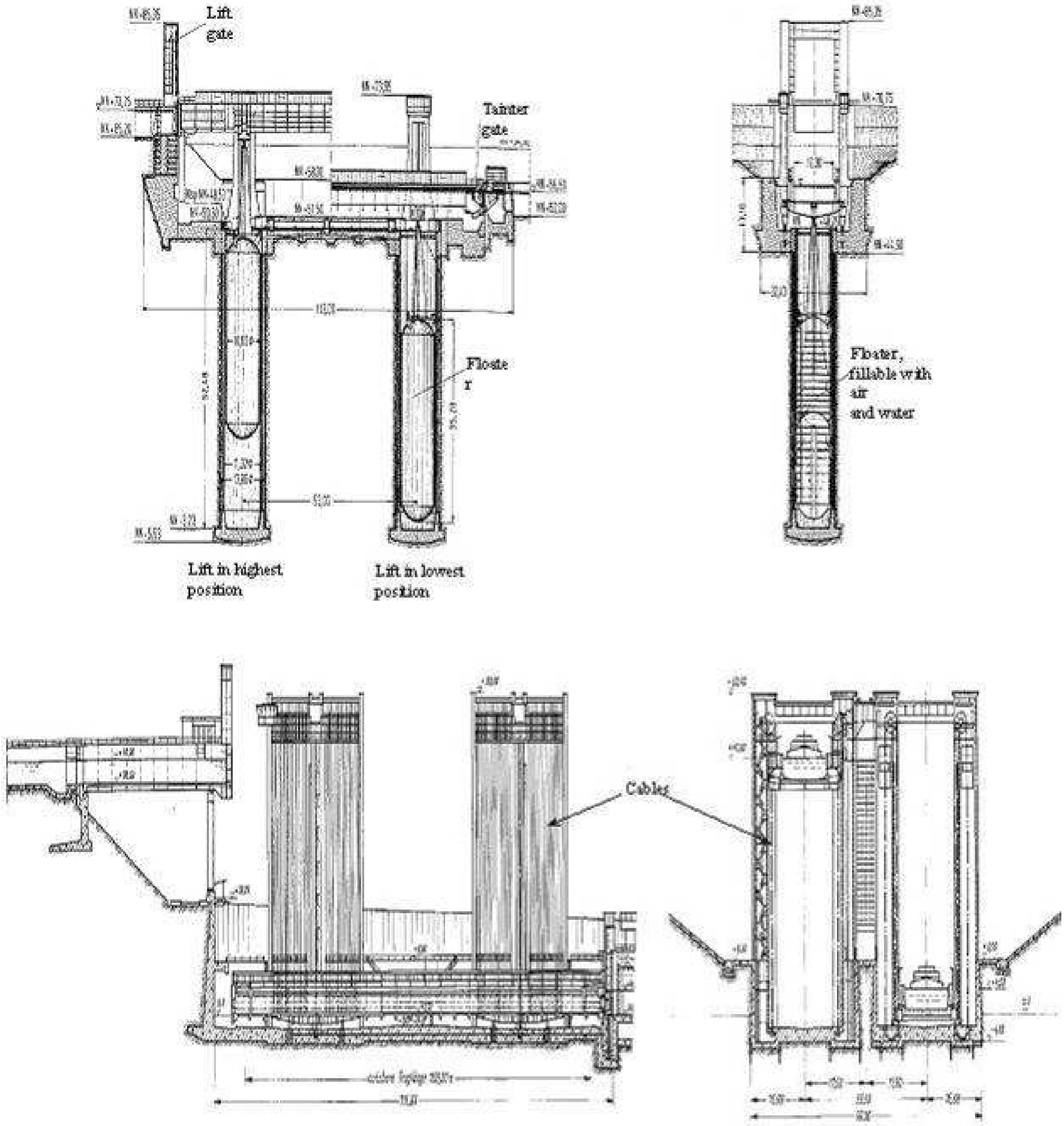


Figure 1-56 A ship-lift with plungers or floaters (upper halve) and a cable ship-lift (lower halve)

1.14 Storm-surge barriers

Storm-surge barriers have two opposing primary functions, namely retaining water and allowing passing of ships and/or water. The functions are generally not fulfilled at the same time.

A storm-surge barrier is only closed in times of extreme water levels. Examples of storm-surge barriers are the water retaining structure in the Weser near Hamburg, in the Hollandsche IJssel near Capelle aan de IJssel and in the Nieuwe Waterweg. Another interesting example of a storm-surge barrier is the barrier in the Oosterschelde. The need to maintain the tidal movement in the estuary for environmental reasons led to the construction of a storm-surge barrier instead of a closed dam.

Figure 1-57 Storm-surge barrier near Capelle aan de IJssel

In many places in the world politicians are proposing to build storm surge barriers to prevent floods. In November 1995, the prime minister of Thailand announced construction of a storm surge barrier in the Chao Phraya River. This was necessary because of the flooding of Bangkok caused by increased river discharges. However, it is obvious that a storm-surge barrier cannot provide defense against high river drainages. On the contrary, the situation would be worse if the river could no longer flow freely to the sea. It is therefore essential to analyse the danger with and without a storm surge barrier. Such a research should also lead to operational demands for the storm surge barrier system.

Figure 1-58 Storm-surge barrier in the Oosterschelde

Figure 1-59 Storm-surge barrier in the Nieuwe Waterweg (Maeslantkering)

1.15 Water retaining structures: dams and weirs

Dams and weirs are primarily designed to head up the water flow upstream of the dam or weir. There can be several reasons for heading up the water, for instance:

- ensuring sufficient water depth for shipping;
- creating a reservoir for an irrigation system;
- creating enough water-level difference to create hydroelectricity using turbines;
- protecting downstream areas from floods.

The difference between a dam and a weir is that a weir can be partially or entirely removed during periods with high river discharge, a dam has not this possibility.

A dam usually has a reservoir and causes a big difference in water level on both sides of the dam. The water pressure on the dam is spread through the dam to the sides and bottom. Taking into account geological conditions and the availability of building materials, a dam will be constructed of reinforced or non-reinforced concrete (an arch dam or a gravity dam) or as a dam of loose materials such as soil and rubble.
Generally in an area with compact rock formations with great compressive and shearing strength, a concrete arch dam will be designed, in which case its forces will be conducted to the sides and bottom by the arching mechanism. Such structures are relatively slender.
In an area with loose materials and with rocks unable to provide great shearing strength, a dam made of soil and rubble or a concrete gravity dam will be chosen, or possibly a buttress dam. An earth fill dam is made impermeable by adding a core made of clay or bentonite, which is extended below ground level in order to reduce seepage through and beneath the dam.
A dam is usually equipped with one or more spillways to control the water level. Such spillways can be equipped with movable closure devices, in order to regulate to a certain extent the water level in the reservoir.

Figure 1-60 Concrete arch dam and spillway

The drop in water level in the case of a weir is substantially smaller than in the case of a dam. The river is dammed up within the existing riverbed. A weir can have a fixed or adjustable crest, allowing the regulation of the upstream water level.
In the Netherlands there are several weirs along the Meuse and the Neder-Rhein. The original purpose of the weirs is to canalise the rivers in order to improve the navigability. Rising energy prices have made it economically interesting to use the pressure head over the weirs to generate hydroelectricity. Till now it was not interesting. However, from the point of view of the environmental and sustainable energy it is more interesting to use the fall over the weirs for water-power stations. This is why some hydroelectric stations were built next to some weirs afterwards.
The way and how the water levels are influenced by the introduction of a weir are very important to the functional and spatial design of a weir.

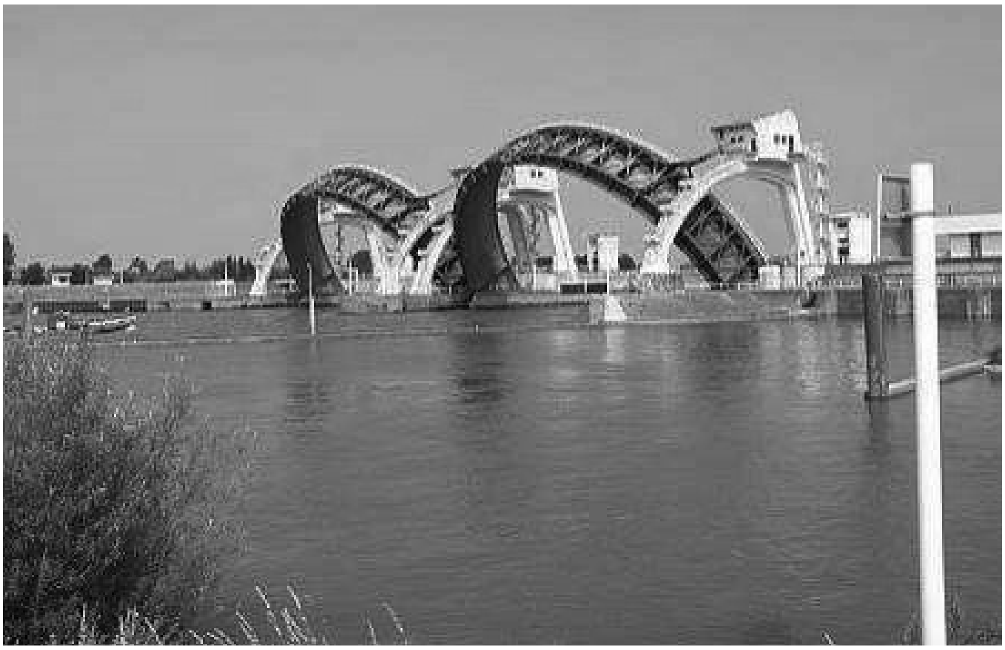


Figure 1-61 Visor weir in the Neder-Rijn

2. Design of hydraulic structures in general

2.1 Position of design in the whole life cycle

Within the life cycle of a hydraulic structure a number of successive life stages can be distinguished, see Figure 1-3. The number and name of the life cycle stages is a matter of definition. Note the contradiction between life cycle and the linear successive representation in the figure.

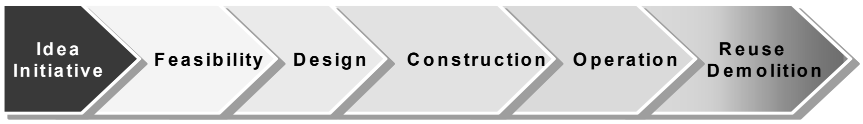


Figure 2-1 Life cycle stages

One can imagine the cyclic character best by realizing that in the 'Reuse & Demolition' stage, here depicted as the last part of a line, 'new' ideas and initiatives are developed, either for termination or, on the contrary, for a following service life. Hence the start of a new cycle and not the end of the line.

Lifecycle stages can be further subdivided. The table below is not intended to be exhaustive nor limitative.

Design	Construction	Operation	Reuse	Disposal
Idea / Sketch	Tender stage	Use	Renovation	Demolition
Conceptual design	Contract negotiations	Inspection	Downgrading	Partially
Final / Tender design	Construction	Evaluation	Upgrading	Complete
Detail design	Transfer	Maintenance	Other location	Other location

In reality the distinction between the life stages is not always clear-cut. Reuse and demolition have been grouped together in the above figure but could be represented separated as well.

Generally the design activities are not limited to one life cycle stage only, and there definitely is a period during the whole project life where design is predominant. During feasibility for sure a conceptual design, or at least a good sketch, of the structure will have to be produced in order to proceed into the next stage. And then the opposite of serial activities again; countless are the projects where construction has already begun and the structural engineer is still finishing the detailed design; i.e. design as a parallel or simultaneous activity.

2.2 The importance of a structured design approach

Throughout the life cycles the money spent on the structure, even if it exists only on paper, increases. Costs increase especially in the construction stage. If all is developing well, from 1st sketch to detailed design the impact of decisions is decreasing, a converging design process results in the final solution. Changing the design is most easy in the beginning, and vice versa, most difficult towards the end.

In situations where changes are made in later stages, e.g. construction stage, one could speak about divergence, there are serious consequences. Often the changes are forced upon the project, consider e.g. political decisions. Obviously design efficiency is reduced, an amount of work has to be redone, required time and cost increase but there may be other further reaching time and costs consequences as well. Think, for example, about initial governmental procedures (permits) or the costs of construction that had already started. It's clear that from the beginning to the end care should be taken to let the project or process converge instead of diverge.

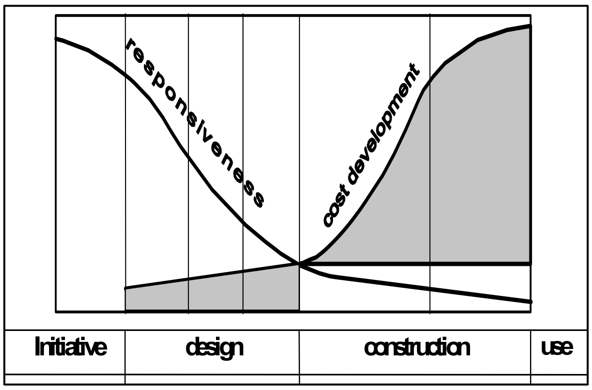


Figure 2-2 Design responsiveness and cost consequences

General scheme for structured design approach

To achieve convergence of the design process a structured design approach should be used. An example of a structured design process is shown in Figure 2-3. Before the analysis phase generally an initial definition of the problem and an initial set of objectives have been produced. In the analysis phase, the project or structure is analyzed on:

- Functions (*Functie analyse)*
- Operational aspects *(Proces analyse)*
- Requirements or boundary conditions *(Randvoorwaarden)*
- Starting-points or Assumptions *(Uitgangspunten)*

What was initially conceived as problem and objectives may be further defined after analysis.

Usually the analysis phase results in a List of Requirements or Specifications. In Dutch: "Programma van Eisen (PvE)"; in Anglo-Saxon literature "Basis of Design" is an expression frequently used. "Terms of Reference (ToR)" is often found in tender or contract documents.

After development of alternatives (synthesis), based on the Program of Requirements and elaboration (simulation) up to level where meaningful comparison is possible (evaluation), one alternative or solution will be selected for detail design. All going well, the process converges into a solution or design.

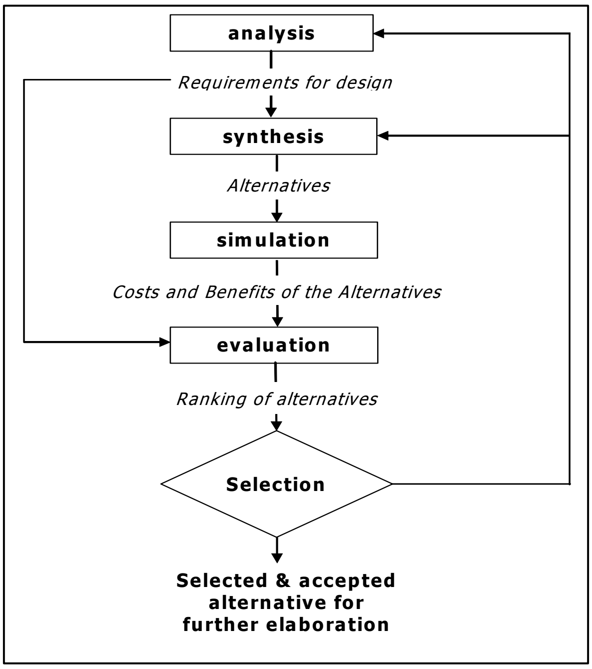


Figure 2-3 Flow chart structured design approach - basic design cycle

Note that at specific moments feedback will be necessary to remain on the right design track, thus the design procedure gets a cyclic character. Every life cycle stage may have its own design cycle, every cycle may have its subcycles, there will be more than one design phase, thus a whole cycling process originates.

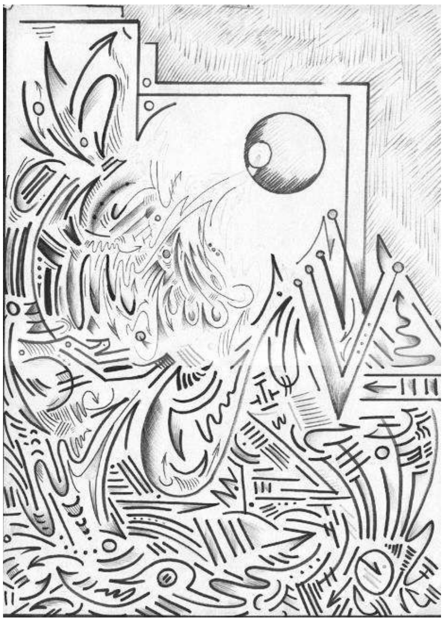


Figure 2-4 Chaos by painter Robert Schwendeman

How to obtain the best design?

However, it has to be kept in mind that a converging structured design process resulting in a “relative best” solution is by no means a guarantee for obtaining the “absolute best” solution or design.

The best solutions are often found after a flash of inspiration followed by hours of transpiration (Free interpretation of Albert Einstein’s original quote).

How to provoke this flash of inspiration? Isn’t this creative phenomenon completely in contradiction with the well argued structured approach? Isn’t creativity synonymous to chaos? Yes and no, the designer, certainly the ambitious one, has to deal with it, has to be able to switch between the creative mind for production of alternatives and the methodical approach for selection etc. in order to keep the design process going in the right direction.

2.3 Design as an iterative process

In the previous design has already been introduced as a cyclic and converging process, trying to find a satisfactory technical solution working from general, rough ideas into more and more detailed design.

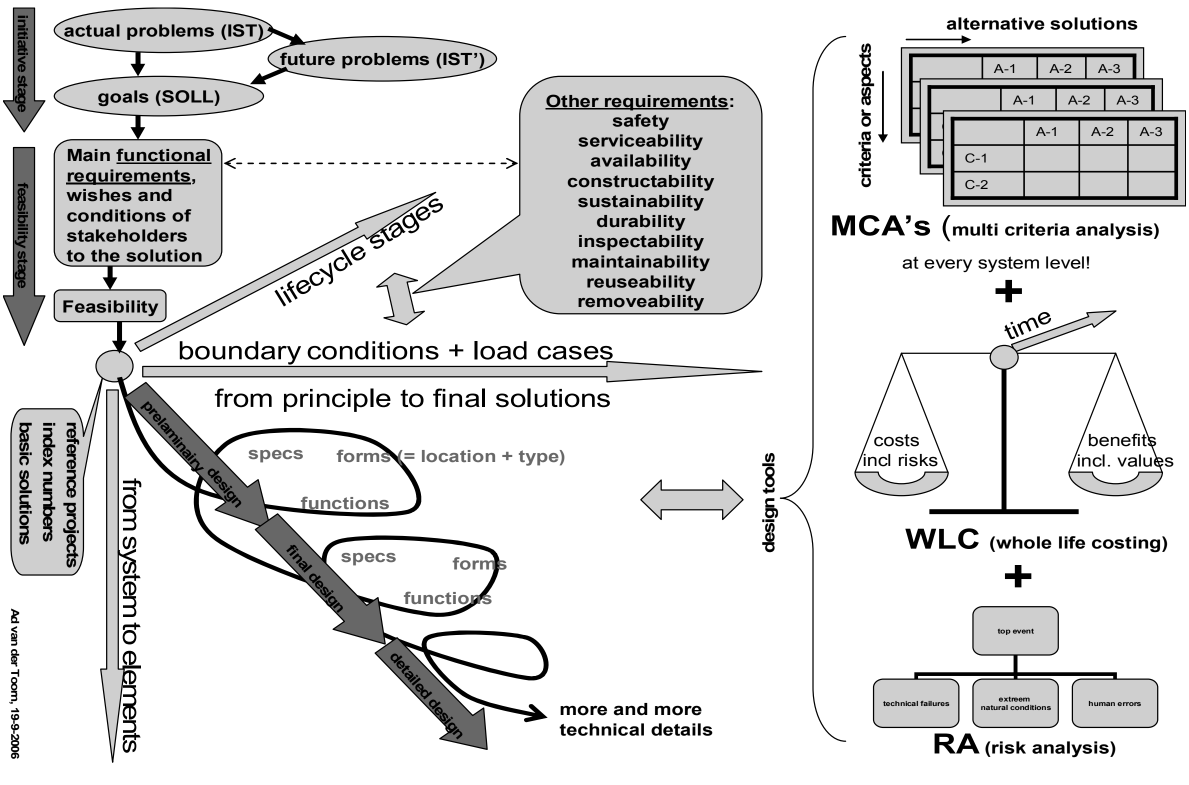


Figure 2-5 Cyclic design process in functional and technical plane

Actually, the start of a cyclic design process presents a real "chicken and egg" dilemma. There should already be a rough idea, a first plan, which includes a location and a type of structure, sufficing to most of the requirements to start the process at the system-level.

Although reference projects from elsewhere with similar solutions, together with the experience of the designer and his colleagues may shorten this iterative process, it should be clear that such a complex process will hardly ever immediately result in "the" optimal solution. First of all because for hydraulic structures every situation is rather unique (the situation in St-Petersburg is yet slightly different from the one in Rotterdam or Venice), secondly because there is no single optimal technical solution (The barrier in Rotterdam could be made in rolling gates too), in the third place because the solution, the structure, gives rise to other or even new criteria and the other way around, new criteria present technical challenges that may result in new innovative for the structure, for construction, maintenance, reuse, etc.

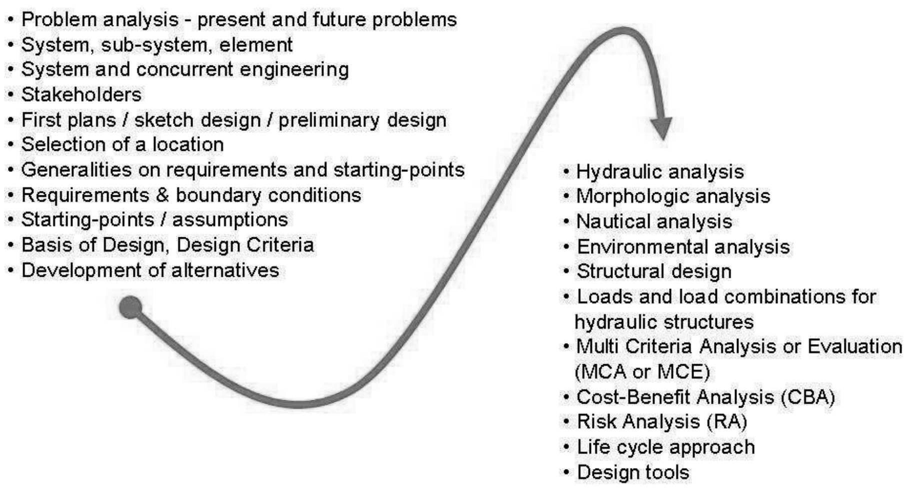
2.4 Subdividing the design stage

Notice that within the design process generally three main stages can be distinguished:

- In the first **initiative stage** there is an actual and/or future problem that triggers a stakeholder, the government (national or local), pressure group or sometimes an individual visionary to define objectives for improvement of the present and future situation at a macro system level.
- In the following **feasibility stage** research is undertaken to identify all the stakeholders that may be involved, their objectives, the functional requirements to possible solutions and whether or not there are solutions for which costs (incl. risks) and benefits (incl. other values) are in balance, not only for the near future but for the whole expected lifetime. The feasibility stage ends with a go/no go decision.
- In the **technical design stage** these first solutions or even better, sketch or conceptual designs, will be elaborated to lesser or greater extend (from preliminary design via final design to detailed design).

2.5 Design issues or aspects

Nowadays, thru all the three design stages, a number of issues have to be dealt with within a design process, viz.:



The issue may come up in any design stage, only once or more often. In the following subsections the above issues will be discussed in more detail one after the other although in real life design the order of appearance may be completely different.

2.5.1 Problem analysis - present and future problems:

To avoid putting a lot of energy and other resources in finding a solution that is not the answer to the actual problem, the first step in a structured design process should be aimed at accurate and exact definition of the existing problem or problematic situation (= IST; 'ist' is the German expression for 'as it is now'). However this should not be limited to the present situation only, the (near) future should be taken into consideration as well. Predict the future with the aid of trendwatching and/or modeling to identify possible future problems (IST') that will occur without extra (preventive) measures.
Right from the outset an idea will exist about the desired situation (SOLL; German to express 'what is strongly preferred'). No matter how vague or preliminary this idea or solution, it will help produce and direct the problem analysis and, later, the evaluation of solutions for the problem.

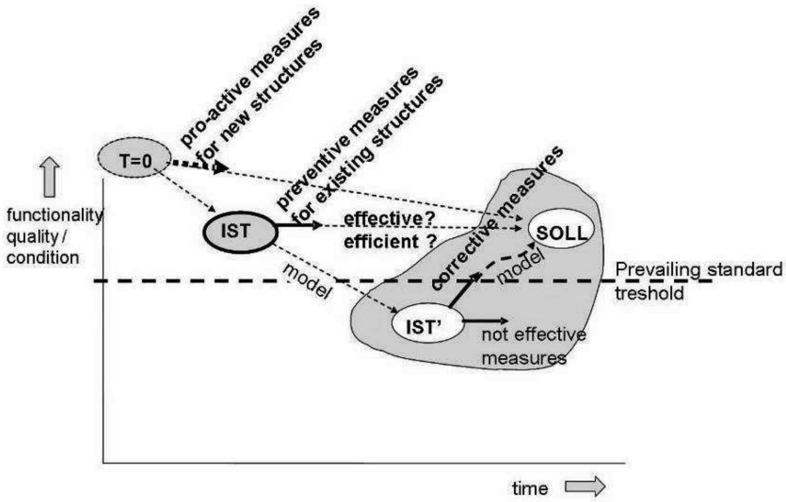


Figure 2-6 Between IST, IST' and SOLL

Hydraulic structures like movable weirs, coastal barriers and inland navigation locks have to fulfill hydraulic as well as nautical requirements. On the one hand they have to protect society against extreme low or high water levels, on the other hand shipping around the clock should be possible. It needs good hydraulical, nautical and not in the least structural analysis to combine different main functions in one structure. Where to start the analysis is not just a matter of taste of the designer, but also depends on the type of hydraulic structure:

- for weirs and barriers the (extreme) hydraulic requirements are mostly dominant
- for navigation locks the nautical requirements are more important, and generally governing for the main dimensions.
- for piers and quaywalls usually large horizontal loads are governing the design.

Whether or not the hydraulical and nautical requirements are interfering or even conflicting, depends on the specific local situation. If critical maximum water levels result in a high closing frequency of a flood protection and/or a long closing time in relation to frequencies of shipping or demands of water-regulation, these three functions of flood-protection, water-regulation and shipping may be separated from each other and may be serviced by different structures.

Examples:

- *the single structure the Maeslant Kering is primarily a stormsurge barrier, with a very low frequency of use (order 1 time per 10 years). Shipping and flood protection have their own functional requirements, but here it does not result in unacceptable non-availability of the waterway due to the stormsurge barrier.*
- *the mu;tiple structure sluice complex near IJmuiden is a combination of navigation locks plus flood protection on the one hand, and parallel, a drainage sluice / pumping station on the other hand.*

2.5.2 System, sub-system, element:

After analysing the cause of the initiating problems, it will be possible to define the size and boundaries of the considered **system** and **objective(s)**, and last but not least, to defining the desired situation (SOLL) or at least the general direction to move. Sometimes problem analyses show it is better to widen the scope of the system because there is a strong relation inside-out the initially defined or chosen boundary and for a better result a more integral approach and solution is preferred. Although it is a good habit to look further than just the system that has to be designed, most of the times the macro-system is like "a given hors". Seldom that what is beyond system can be changed, "looked into the mouth", by definition it is "out of the project scope".

Requirements of different stakeholders may be fulfilled by a technical system like a quaywall, weir or stormsurge barrier, but such a system is always a part of a much bigger macro-system like a port, a river or an estuary. An integral design takes into account that macro-system, not only as a source of the boundary conditions or for extra cost of connecting works, but most of all because the performance of the macro-system as a whole is important for the performance of the designed system as such (and opposite).

Examples:

- *just raising the crest of a local river dike or strengthening a sea defence work "to be safe", may not be the optimal solution if you look at the system as a whole.*
- *a stormsurge barrier does not prevent the hinterland from flooding if adjacent dikes in the same dikering are the weakest link.*
- *a quaywall can not accommodate ships with much more draught than the approach channel of the harbour*
- *a weir in a river could not regulate the river discharge sufficient and/or maintain enough waterdepth when in the upstream river reaches adequate measures have not been taken*

Going into more and more detail, so from macro-system, via system, sub-system to element (the so called technical decomposition), the objectives and requirements at macro-system level have to be decomposed in a consistent structured way (the so called functional decomposition). Here design tools like failure-trees and event-trees are of great help to clarify how subsystems or elements do or don't contribute to the performance of systems at a higher level.

2.5.3 Systems engineering and concurrent engineering:

Principally there is a lot of freedom for the designer to change elements or subsystems within a system, as long as the requirements at the system level are satisfactorily met. However to shorten the total design & construction period, to reduce risks and costs, and make the design process more transparent, design methods like systems engineering and concurrent engineering (copied from the car, airplane and computer industry) have been introduced.

Systems engineering

Systems engineering (SE) is an approach to organize the design process of a very large and/or complex project. How to get there, not what will be the result, is the question to be answered when using systems engineering techniques. The two main activities within SE are:

- splitting up the overall system into smaller, easier to design, sub-systems. Progress and quality of the design of the individual sub-system designs has to be kept in control.
- interactions at the boundaries between the different subsystems, so called interfaces, and accompanying requirements should be carefully defined, subsequently guarded and changes strictly managed.

SE offers a holistic perspective of the system as a whole and as such provides a good base for integration of all (technical) efforts ensuring that subsystems will work with one another.

Hydraulic structures like stormsurge barriers and weirs are fit for the approach, because there is a natural separation (physical, material and with regard to knowledge) between the movable gates, which are most of the time steel structures, the mechanical machinery, electronic control system, the concrete piers or upper structures, abutment or lower structures, sillbeams and foundation, bottom protection, etc.

SE used as an interdisciplinary approach for the development, realization and exploration of complex systems, can focus on customer needs early in the development cycle, documenting requirements, is able to cover the whole lifecycle, while considering environment, cost and benefits, design & development, operation and maintenance, etc.

Concurrent engineering

Concurrent engineering aims at design of different sub-systems or important elements, in different design teams, working separately but more or less parallel. Concurrent engineering can be considered as a special type of SE. The distinguishing factors are:

1. start and finish of design of the sub-systems are at the same moment; generally there are deadlines to be met that put design under (enormous) time pressure.
2. As with SE, care has to be taken that the individual sub-systems, i.e. the results, are integrated into an overall design having a better quality than just the sum of the constituent sub-designs. To control the interfaces the different design teams are often forced to work in literal close proximity to each other, e.g. on the same floor of an office, in the same building. Communication lines are kept as short as possible.

2.5.4 Stakeholders:

For hydraulic structures like river movable weirs, coastal storm surge barriers, inland navigation locks, quaywalls in ports, etc. there are many stakeholders, directly or indirectly involved, subjected to a lot or just a few of the effects of the decision to design, build, operate, maintain and reuse or remove the hydraulic structure. To mention some stakeholders: the public or private owner of the structure, users such as the maritime or inland navigation associations, living communities in the vicinity, operational parties as pilots and towing-services in ports, (sub)contractors, finance ministries/departments, maintenance managers, action groups and political parties, etc. Every stakeholder has different wishes and requirements, different for every life cycle as well.

In complex situations an extensive broader stakeholder analysis is needed to get the whole picture, i.e. to find all parties (stakeholders) directly involved and all (in)directly affected by the (initial) problem and by possible solutions, their specific objectives are and their commitment to different solutions. It is important to clearly define specific objectives, responsibilities, contributions and importance to the project. A so called stakeholder matrix is a useful tool.

Any development requires a promoter or Client from the public or private sector, who has an idea or concept in mind of what facility is required. The first stage of Planning and Design is initiated through the Clients Brief. A Clients Brief should include as a minimum:

- What he requires eg of a container terminal
- Where the facility is to be located
- When it is required to be commissioned - programme
- Planned performance of the facility- throughput
- Planned economic life and implementation of LCM
- Potential future use for the facility at the end of its economic life

- Likely external influences eg Planning consents
- The available budget

Normally a port structure will require some form of legal authority for its development and may well have been subject to a planning inquiry that will have led to certain caveats on what must be adhered to during the development phase; for example additional noise restrictions on piling, visual impact – crane heights, transport of construction materials etc.

2.5.5 First plans / sketch design / preliminary design:

Although the design process is still in a very early (feasibility) stage, there are only objectives in terms of a desired situation and a first set of functional requirements has been defined, there is already a strong need to 'see' the solution. The first plans or sketches or a preliminary design serve to:

- make things literally imaginable
- make a first assessment on the functional and technical feasibility of the sketch design
- allow a first rough cost-benefit calculation

If there is a positive answer to functional and technical feasibility and the cost-benefit calculation as well, then design can move from feasibility design stage into technical design.

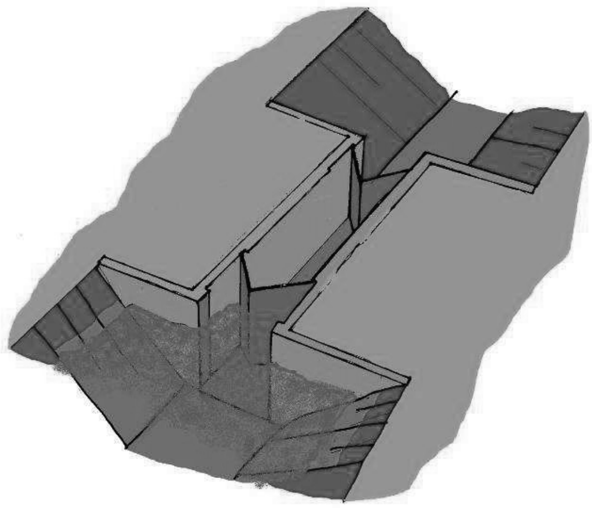


Figure 2-7 1st sketch of a small lock for pleasure navigation

The design used in feasibility stage can be based on scaling of other reference projects and index numbers (price and profit per unit of a typical dimension). Generally there is no technical solution worked out for the specific situation at hand.

2.5.6 Selection of a location:

The selection of a location presents a "chicken and egg" dilemma, because you need a rough design to select a location, but opposite you need a global location to make a first design. This is a typical loop in a cyclic design process.

First step is the selection of a few alternative building locations. The choice of the provisional location depends on a lot of aspects, which are more or less influenced by the location, such as:

- The possibilities for construction of a solid foundation of the structure.
- Exposure to environmental loadings (e.g. current, tide, wave attack)
- Safe and easy access for nautical traffic and other users
- Reduction of environmental impact
- Optimization the use of available space
- Approval by (local) authorities
- Easy access during construction.
- The necessary adjacent extra works (connecting waterways, roads, dams, etc.)
- Rough estimate of total lifecycle cost

Essential for the foundation of the structure is the availability of sufficient information on soil conditions. If the information is not available a soil investigation has to be carried out. This could be the major cost item in the feasibility stage.

In this design stage a general idea about the structure will do. Criteria used for the selection of the location or the construction site could be the consequences for the foundation, exposure to climatic or hydraulic loads, access, environment, etc.. The means to do the actual selection of the location would be the Multi Criteria Evaluation (MCE).

2.5.7 Generalities on requirements and starting-points:

There is a typical difference between requirements or boundary conditions and starting-points or assumptions. The first category is more or less forced upon the project, is quite clearly defined and can be expressed quantitatively in a straightforward approach. The latter category is much more open to discussion. Requirements can be associated with demands and from 'outside' the system, starting-points more with wishes and from 'within' the system. Much more important than a dogmatic argument is the fact that a clearly defined and quantified criterion ends up in the Basis of Design whether it originated as a requirement or starting-point.

Safety is a matter always subjected to ‘emotion’, thus lengthy discussion. Although from a technical point of view everything seems to be possible, the economic realities are quite different and this is a hard message to get across. It has to be decided what level of safety will be required or provided. Whether a safety level, or the counterpart risk level, is accepted or not also depends on the level of prosperity and culture, so may be different in other countries. Often there is a more subjective relation between the recent appearance of a (near) disaster and the political willingness (as voice of the society) to invest in risk reduction, because in normal times it doesn't attract votes to invest money in safety.

Example:

- *the level of accepted risk of the stormsurge defence in New Orleans was in the order of a 1:100 years storm, while in Holland this level is based on loads with a return period in the order of 1:10.000. Partly this was due to the fact that the boundary conditions in New Orleans were much more substantial in combination with budget restrictions and partly because more attention was given to evacuation measures in times of threats or real flooding.*

From an economic point of view, determining the required safety level is a matter of optimizing between reduction of remaining risks for all stakeholders (from direct users up to society as a whole) against the extra cost of these risk reducing measures, see the illustration below. Because the complete calculations inclusive damage to structures and risk of users and society may be very complicated and is very laborious, often simplifications are introduced, e.g. just calculating for the dominant risks, effects and costs.

There may be strong sentiments in society opposed to expressing safety in mere cost figures. Obviously here decision techniques based on Multi Criteria Analysis (MCA) are helpful to force a decision.

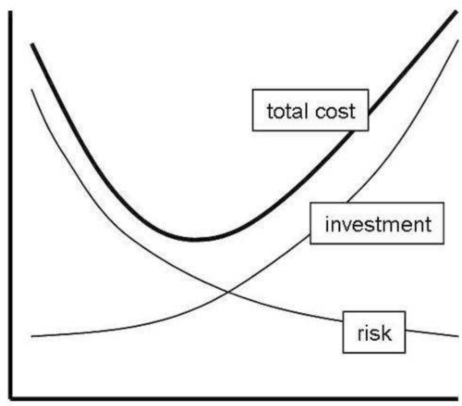


Figure 2-8 Reducing the total cost by balancing investment and risk

2.5.8 Requirements & Boundary conditions:

(Randvoorwaarden)

Requirements and boundary conditions are forced upon the project, they should be considered as matters that can not be influenced by the most directly involved stakeholders and certainly not by the design team or engineer. This is the main difference with starting-points or assumptions, see next paragraph.

Examples:

- *if the structure has a nautical function generally a design ship or governing vessel will be defined. For the owner or end user of the structure the design ship will be selected weighing the benefits of a larger ship against extra cost that may result and vice versa. For the owner the design ship is a starting point, especially in the initial and feasibility stage. However the designer has to accept the design ship as a requirement.*
  Note the governing ship may only exist on the drawing board; it is the future that has to be taken into account.
- *laws, licenses, workforces, budget restrictions, etc are typical requirements or boundary conditions. Note again that these requirements may change in time, however, now or in the future they will be forced upon the project.*

A lot of stakeholder objectives will be translated into requirements to the desired situation, including the structure. The requirements will be different or show overlap to greater or lesser extends, sometimes conflicting requirements will result. Especially parties involved with the operational use or maintenance of a structure will be able to define a wider set of requirements. Unfortunately it seems to be (bad) tradition to introduce these requirements rather late in the design process. The objectives of the (local) society generally result in requirements of a different or extra category, the category of environmental friendliness and esthetics.

Examples:

- *local society does not want high cranes at their skyline or noisy transshipment activities. The Environmental Impact Assessment (EIA), generally prescribed by law for large civil infrastructure projects, will be used by the locals to prevent they get something they don't want.*
- *unlimited air draught for ships sailing through (coastal) locks or barriers, which excludes lift gates and the accompanying high structures, coincides with the demands to avoid pollution of the horizon.*
- *people living near the boundary of a system want the same safety as others within the system, but do not want the enormous dike or other defence structure in their backyard (NIMBY – not in my backyard).*

Where the word 'requirements' seem to relate more to stakeholder objectives, the expression 'boundary conditions' is usually related to physical phenomena.

Examples:

- *At the waterside there are the typical dominant hydraulic boundary conditions like:*
  - *bathymetry of the dry and wet part of potential building location (by GIS)*
  - *wave heights, frequencies and directions*
  - *tides with heights and frequencies of exceeding*
  - *current velocities, directions and frequencies*
  - *water density (variable in space and time)*
  - *wind velocities, frequencies and directions*
  - *fog and rain density and frequencies*
  - *air and water temperature*
  - *river run-off in amount and frequencies of exceeding*
  - *morphology of the river, estuary or sea*
  - *ice occurrence, thickness, period and frequencies*

- *On the landside there are the geotechnical boundary conditions like:*
    - *origin, nature and building up of the (sub)soil*
    - *soil properties like density, bearing resistance, compressibility, permeability*
    - *earthquake forces and frequencies (also due to gas and oil exploration!)*

The presence of the structure influences its direct surroundings. Without further thinking one could say the hydraulic boundary conditions influence the solution, but in the opposite direction the boundary conditions are influenced by the structure. According to strict SE and mathematical definition the conditions at the boundary are not or can not be influenced by the presence of the hydraulic structure. Literally, the boundary should have been chosen at such a distance that it can not be influenced by the structure.

2.5.9 Starting-points / Assumptions:

(Uitgangspunten)

Compared with requirements starting-points are more open to discussion, i.e. the quantification is a matter of agreement between stakeholders and the design team or engineer. It is important to pursue clarity and agreement on starting-points because approval of- and support for the final design result depend on it.

Examples:

- *planned performance of the facility may remain open for discussion relatively long because the benefits of higher performance may outweigh extra costs. At certain moment a decision has to be made otherwise the design process comes to a hold. Think for instance of:*
    - *throughput of terminal*
    - *number of cars per day or per hour through a tunnel*
    - *the traffic intensity, on a river or canal, in the number of ships per hour, for both the upstream and the downstream direction*
- *planned economic lifetime and planned technical lifetime.*
- *for society environmental mitigation and compensation measures are a starting-point; for the project these measures are a requirement.*

When requirements and boundary conditions are prone to quite unpredictable changes in the course of time it is hard to determine the quantities for design. Obviously the design team has to discuss this with the owner or Client. For the specific project at hand the threshold, the criterion has to be defined quantitatively. The unquantified requirement changes into a quantified starting-point, the line between requirement and starting-point is crossed. What matters is that a well defined and quantified item ends up in the Basis of Design.

Examples:

- *sealevel rise because of melting icecaps (0.5 m, 1 or 2 m?) or increasing river runoff (e.g. maximum run-off of River Rhine from 15.000 to 18.000m^3/sec), both caused by global longer term climate changes that are hard to predict qualitatively and quantitatively.*
- *the coastline or course of a river may not be so fixed as it is in the Netherlands, so "fixed" abutments of hydraulic structures may become loose.*
- *the Eastern Scheldt stormsurge barrier tempers the in and outgoing tidal stream, so creates sandbanks in front of the barrier and so reduces the governing wave height.*

2.5.10 Basis of Design, Design Criteria:

The Contractor, Consultant or Engineer developing the project will prepare a Basis of Design to clarify, in engineering terms, the client's requirements. It is most important for the client to understand and approve the Basis of Design presented to him at this initial stage so that the completed work meets his expectations.

The biggest part of the Basis of design will be produced in the initial and feasibility stage of design. However, in the technical design stage additions will be made. Up to the moment that construction starts or even up to disposal, the Basis of Design may be expanded, which is in complete agreement with the cyclic nature of the whole design process. Obviously the most important elements will be decided upon in the beginning, the additions or changes in later stages will pertain to ever more detailed elements or matters.
If detailing the design proves to be not that easy as supposed at the global level and gives rise to fundamental hard to solve technical problems, than be aware that these results may lead, not only to another technical solution, but as a result to other MCA's and/or CBA's, thus to a completely different project (which includes no project) as well.

The Basis of Design should include as minimum:

- A recital of the Clients Brief
- Local and site specific physical conditions
- The design loadings to be adopted
- Impacts from external sources e.g. planning conditions
- The results of any investigations undertaken and their impacts
- A maintenance strategy
- Anticipated Re-use/removal of the structure at the end of its economic life

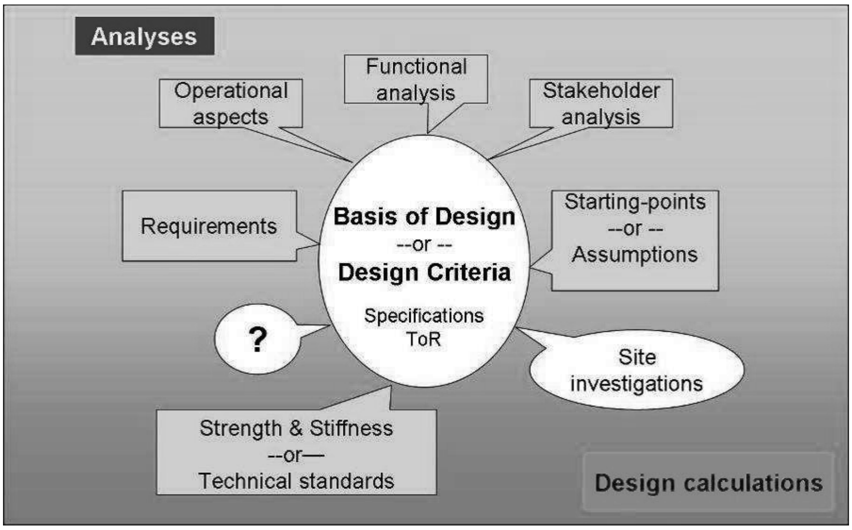


Figure 2-9 Contributions to the Basis of Design

Most important is clear unambiguous definition and quantification of every item in the Basis of Design.

2.5.11 Development of alternatives:

Having established a Basis of Design, using the first plans or sketch design, now the time has come to produce a more elaborate design and even alternatives to this design. The professional will use theory, his or her experience, reference projects, local design and construction custom and tradition to develop a range of solutions. In this way, to a greater or lesser extent, the solution will be based on proven technology.

Examples:

- *a quaywall may on the one hand have up to 7 different functions like a place for berthing ships, supporting cranes for the transshipment of goods, storage capacity, wave absorber, high tide barrier, etc. and on the other hand there are a lot of known basic solutions like the caissontype, gravity type blockwalls, the combiwall with concrete relieving platform, etc.. Most of the times one of these basic solutions will suffice to all the requirements and some scaling and detailing work has to be done to prepare the design for the specific situation at hand. Only in a few cases the requirements are extra ordinary and /or boundary conditions extreme, so a new, more creative solution has to be found. See the combined breakwater-quaywall-storage facility-garage in Monaco.*
- *a stormsurge barrier has mostly a few different functions such as the barrier-function in times of high tide, the discharge-function and the passage of ships in normal times. From comparable situations elsewhere there are already a lot of technical solutions with proven technology that fits the specific situation. There are common types like sector gates with vertical axles, lifting gates, drum gates, etc. Only in a few cases the requirements are extra ordinary and /or boundary conditions extreme, so a new, more creative solution has to be found. See buoyant retractable floodgates in the flood protection near Venice because of the valuable undisturbed view at Venice.*

The creative and or associative mind:

In situations where (a combination of) real new or extreme functional requirements and boundary conditions have come into play, a more creative process is needed to come up with (a combination of) innovative solutions. Tools like "design-tree" or "morphological chart" may be of help, because they structure the creative process by decomposing complex functions into elementary functions and, the other way round, combine elementary solutions into a new composition. Even in this case ideas are generally not completely new (so no eureka!) but most of the time an extrapolation beyond the field of experience. Use of new materials and/or equipment, transformation of ideas from other sciences or engineering fields and/or a smart combination of existing methods resulting in successful stepwise or incremental innovation.

Examples:

- *If a quaywall has to be constructed in much deeper water, there is search for floating or semi-floated and prestressed anchored solutions (see offshore oil platforms).*
- *If a stormsurge barrier has to be designed in a visual and/or historical more sensitive environment, there is a search for inflatable or retractable floodgates. (Ramspol, Venice).*
- *If waveimpact on the gate in a stormsurge barrier results in too high forces, there is a search for impact reducing measures such as perforation of the plates or use of tubular beams as in offshore platforms.*
- *If quaywalls or storm surge barriers are used under extreme salty conditions there is a search for materials such as high strength (and dense!) concrete already used for bridges or cathodic anodes already used for offshore platforms to protect them against corrosion.*
- *If leakage of water and sand through temporary or permanent retaining walls is a time and money absorbing problem, there will be a search for leakage detection devices (based on differences in electric conductance or temperature) and leakage stopping techniques (based on smart soils).*
- *If in situ construction of a quaywall or stormsurge barrier in a severe conditions, gives to much unworkable days, there is a search for prefab techniques with very large specific equipment like already used in the offshore (Balder & Hermod/Heerema, Svanen/Ballast Nedam, etc.).*

2.5.12 Technical analyses:

Hydraulic analysis:
For hydraulic structures like river movable weirs, coastal barriers and inland navigation locks which will influence more or less the existing hydraulic condition it is important to be sure that, from a hydraulic point of view, functional requirements are fulfilled.

For movable storm surge barriers near the coast on one hand the main function is to protect the hinterland against flooding caused by an extreme combination of high tide, wind upset and waves, and a barrier should be as closed as possible. But on the other hand during the much more frequent normal conditions the barrier should be as widely opened as possible, e.g. for preservation of a typical natural area. Due to piers and sill beams, which influence discharges, tide effects inside the estuary, etc., there are always changes to the initial situation without the structure.

Example:

- *one of the main requirements for the Eastern Scheldt stormsurge barrier in normal (open) conditions was to guarantee a tidal range of more than 2,70 m inside the Eastern Scheldt estuary near Ierseke, because of the local mussel culture. This leads to a minimum gate opening, while the stormconditions work in the opposite direction.*

Assume that after analysis and some initial design a first structure is produced on the drawing board (or on AutoCad screen). The next step is to check whether or not the hydraulic structure works as it should.

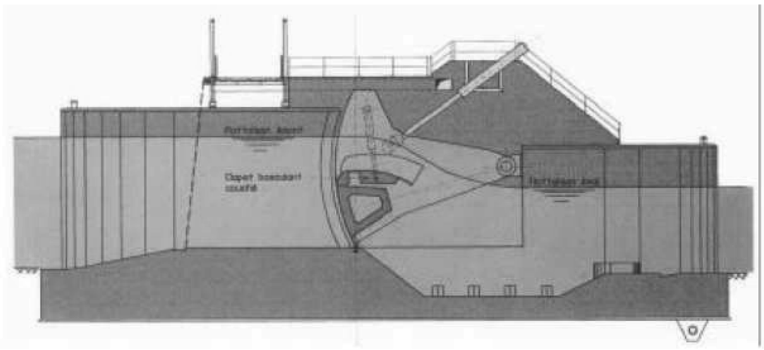


Figure 2-10 Complicated flow through a stop lock

Based on the simplicity of the hydraulic situation and structure, and the governing hydraulic conditions, the check will be done analytically (formulas/theory), using 2 or 3 dimensional numerical models (like DELFT-3D) or by physical model testing. Although the first aim is to check the hydraulics, it should work the way it is supposed to work, the second aim is to determine the hydraulic loads on the structure.

Morphological analysis:
Sedimentation and counterpart erosion are two aspects which should give special attention. Although primarily caused by hydraulics and shipping they may the other way around also influence hydraulics, nautical use, normal functionality of the structure and in extreme situations the structural behaviour and integrity.

Examples:

- *reduction of the undisturbed velocity by the presence of the hydraulic structure (open or closed) may result in overall and long-term extra sedimentation in the original river or seabed. Nonetheless, it may also initiate local erosion in the direct vicinity of the structure because of high pressure differences, standing waves and water jets. This may cause some leakage, backward erosion and in extreme* *situations instability of the foundation and structure as a whole. Hence the bottom protection of Eastern Scheldt, Hartelkering, etc..*

- *construction of a quaywall at the (very) sheltered lee side of the Nieuwe Maas gives less problems regarding current and the resulting mooring forces at the berth. On the other hand the sedimentation rate is that high that for every new ship arrival again sounding and dredging is needed.*
- *caisson-type quaywalls at the port of Rotterdam faced through out the years growing depth of berthingplaces. Till the toe of the caisson has such a few cover that erosion causes local wholes and unequal settlement of the different caissons.*

Nowadays there are a lot of hydraulic models like DELFT-3D to predict overall tides and velocities or even higher order models (based on nonlinear Navier-Stokes) to predict local turbulent velocities in the direct vicinity of the piers and gates. But the prediction of sedimentation and erosion by morphological models or special coupled modules is much more difficult, because stability of loose materials is very sensitive to local velocities (in theory stability is dependent on v^3). This is even worse when erosion is caused by local flow from main screws or side propellers of ships. So empirical parametric formulas based on physical scale models in combination with frequent inspection or monitoring of the real structure are still important to reduce and manage the risk of sedimentation or erosion.

Nautical analysis:
For hydraulic structures like movable weirs in rivers, coastal barriers, and inland and coastal navigation locks it is important to be sure that the nautical functional requirements are fulfilled. In the open situation sufficient water area should be available for the required ship manoeuvres, which should take into account governing conditions from a nautical point of view, i.e. for instance the maximum current and wind velocities for ship manoeuvres. (Extreme current and wind conditions may not be relevant for shipping, but all the more for loads on the structure).

After selection or definition of the governing or design ship, the extra width, depth and air height are determined using advice of nautical experts, design manuals like the PIANC-guidelines for approach channels, or ship simulation and physical model testing.

If structural design of a weir, for the governing closed condition, results in dimensions that are insufficient for ship manoeuvres in open condition and/or the resulting current velocities in the remaining water section are too high for navigation, then the decision must be made to provide a separate navigation lock. For this lock the usual design rules may be used to determine the required lock dimensions.

Figure 2-11 Inland barges navigating through opened visor gates, weir river Lek - Driel

Environmental analysis:
Since many construction projects have been delayed or even cancelled because of underestimating the environmental consequences, this is the last but not the least analysis of all. Every structure interferes with the original (natural or social) situation and the arguments in favour of the project have to be made very clear. Especially in projects near or interfering with so called natural habitats the consequences should be investigated profoundly. With or without this Environmental Impact Assessment (EIA) there may be (legal) claims for mitigation of and compensation for the loss of environmental values.

Examples:
- *construction of the Westerscheldt Container terminal was delayed, might be completely cancelled, because selection of that particular location was not motivated convincing enough.*
- *the project of Maasvlakte II was delayed for more than a year because the consequences of the disturbed sediment flow along the Dutch shore up to the Waddensea and the consequences for the flora and fauna were not investigated in satisfying detail.*

2.5.13 Structural design:

The following steps are taken within the structural design:

1. Take the preliminary or reference design and make a first check, (in)consistencies from a structural point of view.
   - What is the shape of the structure (3D), what are the dimensions and materials? Estimate the dimensions for the main structural elements by rules of thumb or by scaling reference structures.
   - A load vector diagram showing the main soliciting forces on the structure and the transfer to the foundation and further into the subsoil, thus the resisting forces, could and should be drawn for the check.
2. Establish all the loads and all the load combinations, not just the most governing, in the Serviceability Limit State (SLS) *(gebruikstoestand)* and the Ultimate Limit State *(bezwijktoestand)*
3. Prepare the design for the superstructure. The structural design is finished when all the checks on stability, strength and stiffness have shown satisfactory results. This should result in a more accurate estimate of e.g. the dead weight load on the substructures.
   3a
4. Prepare a design for the substructure or foundation. The main choice is whether to use a shallow foundation or a deep foundation (pile foundation) for the superstructure taking into account the opportunities offered by the soil conditions. For the foundation stability, strength and stiffness have to be checked satisfactorily as well as for the superstructure.
5. If necessary, e.g. when the footprint of the foundation is considerably larger than the footprint of the superstructure, an iteration between the previous steps has to be started and followed thru.
6. From a structural point of view (stability, strength and stiffness) the robustness of structure may have been proven, now the resulting draft final design has to be compared with the Basis of Design or the Design Criteria. At the end of the design process the designer has to verify safety, serviceability, durability, constructability, maintainability, reuse of all elements, subsystems and the system as a whole.

3a In some hydraulic structures special or “key” elements can be distinguished, for instance the gate of a stormsurge barrier or the water intake and outflow system in locks. Before starting with the super- or substructure, the design of the key element should be taken to a more detailed level.

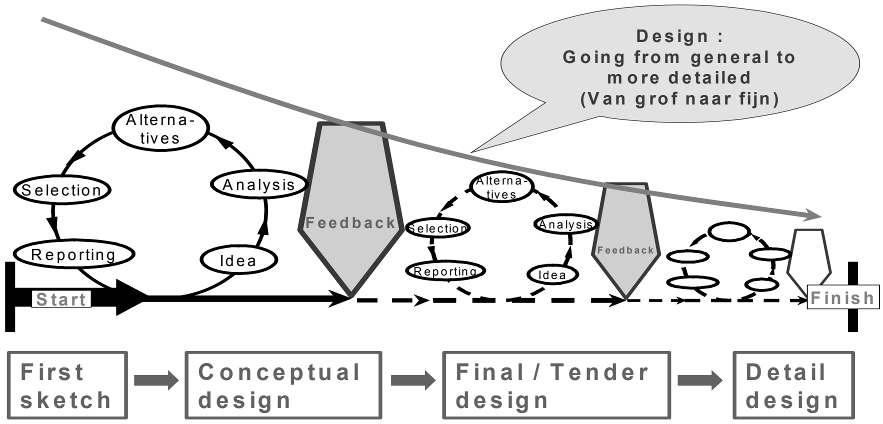


Figure 2-12 Design cycles

The level of design calculations and the level of detail of the design itself are going hand in hand from general to more and more detailed. The sequence and associated characteristics are as follows:

1. Hand calculations:
   The available sketch of the desired structure is schematized into beams and columns and supports, a statically determinate or indeterminate system. The mechanic formulas used are, e.g. $^{1}/_{10}$ - $^{1}/_{8}ql^{2}$ for bending moments or $^{5}/_{384}ql^{4}/EI$ for the displacement and so on, formulas every civil engineering student has to know by heart. The use of software would be limited to spreadsheets; always handy for fast computation of some alternatives. Full reference is made to the “Manual for Hydraulic Engineering Structures”.
2. 2D or plane frame calculations, simple software:
   Based on the hand calculations the obvious errors in the shape of the structure have been removed and basic shapes and rough dimensions of structural elements have been established. Generally the structure as a whole will be modeled and analysed using 2D or plane frame software. The analysis of the whole structure with the help of the computer is often much too advantageous compared with the very laborious, therefore prone to errors, work by hand calculations. Typical 2D structures are jetties, quaywalls, tunnel, dikes. For example caissons and piers are typical 3D structures.
3. Computer calculations based on 3D Finite Element Methods (FEM):
   The use of FEM based software (ANSYS, DIANA, PLAXIS, etc.), generally for 3D designs or situations, is especially meaningful in final design stages, in fact boils down to doing a last check on structure regarding stability, strength & stiffness. Use in earlier design stages should generally be avoided because the ‘specialist’ time needed to run the program, moreover the software and hardware, take a lot of money.

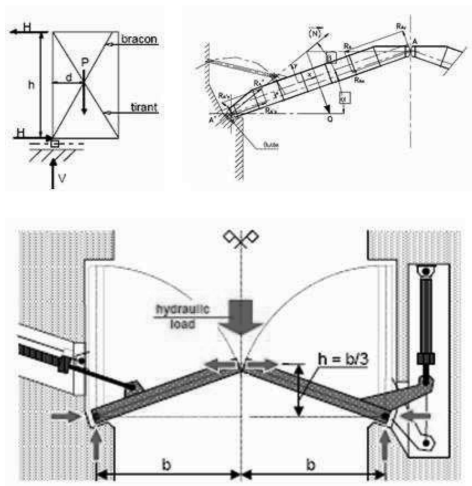


Figure 2-13 1st Schematisation mitre gates; hand or 2D software calculations

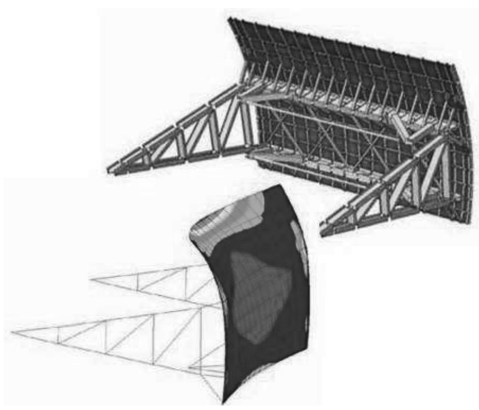


Figure 2-14 Finite Element Method (FEM) calculations - 3D

Often the adopted work method ‘from general into more detailed’ calculations prevents situations where fundamental changes to the design are required because the ‘details’ can not be solved, but not always.

Examples:

- *in a rather late design stage it had to be concluded that the wave impact forces on the gates of the Eastern Scheldt stormsurge barrier were much too high to be resisted by standard plate girders; which were selected based on the available experience in the design office. Providing perforations in the plates of the girders could not solve the problem, however, it took a long time to disqualify this solution. In the very final stage, under great time pressure, a complete new design, the idea originating from the "offshore ", using tubular beams and complex joints was selected and further developed into detailed design.*
- *Again in a rather late design stage, now during the Maeslant barrier project, dynamic forces resulting in alternating movements during the closure of the gates forced the design team into considerable extra physical model testing and, based on those test results, making fundamental changes to the design of the gates.*

Memorize the above with the following slogan that is often heart in design land:

THE DEVIL IS IN THE DETAILS

Note that in case detailing the design proves to be not that easy as assumed at the global level and gives rise to fundamental hard to solve technical problems, than be aware that these results may lead to other MCE and/or CBA's, so maybe to completely different main solutions or structures.

Loads and load combinations for hydraulic structures:

The following list gives an idea about the loads to be considered in the design of a hydraulic structure:

- Deadweight of the hydraulic structure
- Hydraulic loads (hydrostatic and dynamic) due to the tide, discharge, setup, sealevel rise, etc.
- Loads caused by primary users of the structure like ships (normal and accidental)
- Loads due to other use like traffic, or other incidental use
- Loads as a result of wind, ice, waves, etc. (if not already included in hydraulic loads)
- Extreme loads cause by earthquakes, tsunamis, seiches, etc.

The list is not meant to be complete or limitative; this might be the reason it looks pretty tight and short. However, when these loads are quantified the following will be taken into account:

- the direction of the load
- the serviceability-, the ultimate-, sometimes an accidental limit state; SLS, ULS (ALS)
- the distinguished life cycles, e.g. construction, operation (including inspection and maintenance), upgrading or re-use, and removal
- varying water levels (minimum-maximum)

So for instance for a wave load at least 4 figures have to be determined/calculated, and depending on the complexity of the project, the maximum may be 36. Wave from the North, from the West, for SLS, ULS and ALS, for 3 life cycle stages and 2 water levels.

Note that, apart of the dead weight and some of the life loads, most of the loads act in the horizontal direction. In general horizontal loads are governing the design of hydraulic structures. For housing, office and industrial buildings, generally the vertical loads are dominating the design, even though a wind or earthquake load may occasionally come into play.

Also note that, for hydraulic structures more life loads have to be taken into account in the design process than for housing or building design.

For housing, office and industrial buildings, loads and load combinations are prescribed in standards like the Dutch code NEN 6702, Eurocodes, or guidelines and handbooks accepted by a wide spread public. In these codes, standards and guidelines definitions are included on representative loads, partial safety factors, i.e. load and material factors. Construction volumes in housing and offices are far larger, which contributed to standardization in that field. Society has chosen a risk level, which is accepted implicitly by just using the code or standard. Obviously the implicit risk level will be on the safe side because the code has to cover a broad field of house and office buildings and conditions.

Besides the direction of forces (horizontal versus vertical) there is another factor that makes a big difference in the determination of loads. For hydraulic structures that are often part of a larger system the accepted probability of failure is smaller than for e.g. office buildings. This is best illustrated by the fault or failure tree in the figure below:

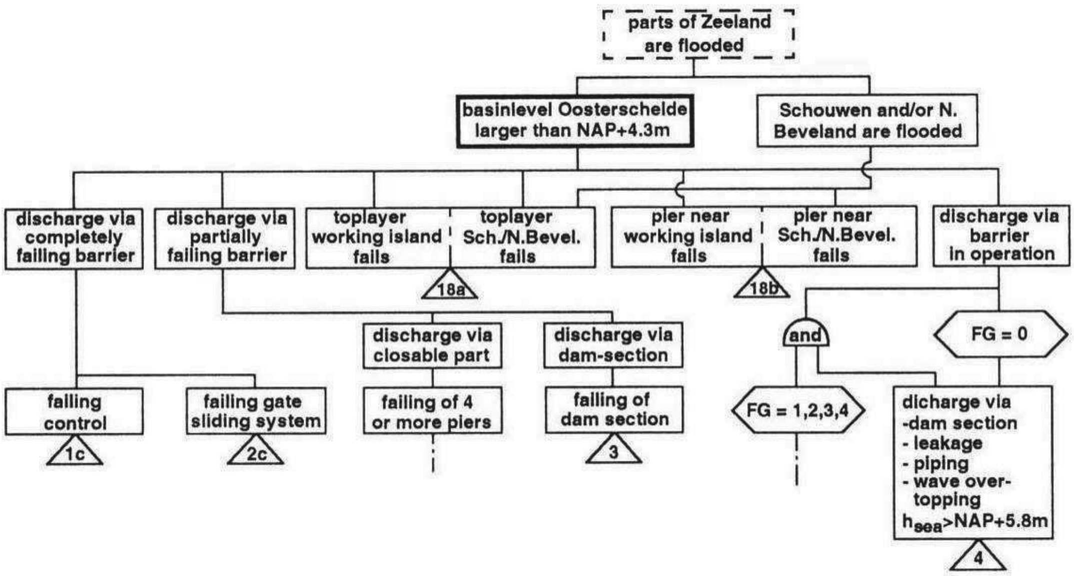


Figure 2-15 Fault or failure tree for flooding of Zeeland - the Netherlands

The probability of the top event 'parts of Zeeland flooded' is equal to the summation of the underlying probabilities of failure for the defined failure mechanisms or bottom events. Assuming the probability of the top event is fixed, the more failure mechanisms the smaller the probability related to the bottom event/fault should be.

As a consequence of the demanded smaller probability of failure for the hydraulic structure the probability of occurrence of the considered load is generally smaller than those of representative loads mentioned in codes and standards (like the NEN 6702). Moreover the probability of occurrence is generally different for every project because it varies with the number of bottom events.

In the above some typical aspects of loads on hydraulic structures have been highlighted, viz:

- considering a single load, the diversity in direction, limit states, etc.
- the large number of life loads
- governing life loads often act in the horizontal plane
- depending on the project, hence the system, the representative load has to be based on yet another probability of occurrence 'again!'

The overall conclusion is that determining the loads on hydraulic structures is laborious work. Generally a number of the life loads and the ever present dead load are working together and should be combined with wisdom.

Combination of loads:
The articles on combination of loads in most of the codes or standards, e.g. the NEN 6702, state that the dead or permanent load has to be combined with the relevant life loads. In fact it is left to the designer or structural engineer to decide and determine which life loads are taken into account and how many are combined with each other. The standards prescribe the (partial) load factors that should be used in the combinations; generally this based on the so called "Turkstra-rule". The maximum value of one variable load should be combined with the mean values of the other variable loads in that specific load combination, not with other extremes.

Every load being the result of a natural phenomenon has its own frequency of occurrence. Between these life loads there may be weak or strong correlated probability distributions. Consider, for example, storms and water levels, or wind setup and wave heights. Determining the right correlations is important either to prevent failure of the structure due to underdesign, or to prevent overdesign as a result of adding two extremes together. This may result in a lot of tedious work, however, it will be rewarding and/or profitable.

Example:

- *For the Eastern Scheldt stormsurge barrier probabilistic design, resulting in good description of the correlations, thus the right combination of boundary conditions resulted in a reduction of nearly 40% on the total load.*

2.5.14 Multi Criteria Analysis or Evaluation, MCA or MCE:

The Multi Criteria Analysis or Evaluation is a methodology by which the relative merit of alternatives can be compared using a range of quantitative and qualitative criteria. MCA/MCE is also referred to as multi-objective decision making, multi-objective decision support system, and multi-criteria decision aid.

For most projects there are many considerations which must be factored in by decision makers. Community awareness of social and environmental impacts is increasing while general expectations of financial and technical efficiency remain strong. However, these considerations are reflected in different ways. Criteria like costs and benefits are measured e.g. in dollars, whilst e.g. environmental impacts can only be measured in a relative way, which complicates comparison of the alternatives. Nonetheless the whole process should result in selection of only one, best alternative.

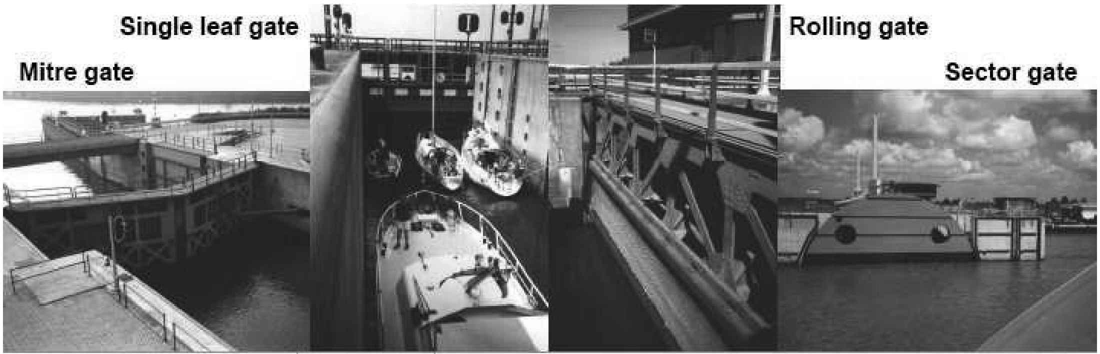
Briefly, the steps to be taken within a MCA are as follows:
1. Identify the alternatives to be compared;
2. Identify a set of criteria for comparing the alternatives;
3. Identify the relative importance of each criterion (weighting);
4. Score the alternatives against each criterion;
5. Multiply the score by the weighting for the criterion;
6. Add all the scores for a given alternative and rank the alternatives by their total score.

MCA is a systematic methodology, which can be replicated and opened up to public scrutiny.
Although MCA does not necessarily require quantitative or monetary data, the information requirements to compile the effects table and derive the weights can, nevertheless, be considerable.

Take good notice of the following:

- when the cyclic design-process has reached a more detailed and reliable level, the extra knowledge or information obtained, relevant for the earlier MCA (and CBA), should be used as feedback on the decisions based those earlier MCA's and CBA's.

- MCE-scores are in first instance often qualitative (++, +, 0 or -, --) or relative numbers (like 1 to 10) and sometimes real quantitative data, for which unit differs per criterion.
- Reality shows costs are always a very important criterion, thus there is a tendency to express or in second round translate every aspect to cost (or benefits). There are economic tools like "willingness to pay" or "willingness to accept" which are of help to translate values in to money.
- Sometimes one criterion dominates or overrules all the others. In MCA-terms the criterion has a very high weighting factor compared to the other criteria. For example budget restrictions, military constraints (second bridge in stormsurge barrier St.Petersburg), local constraints (Thames barrier in London), landscape constraints or prestige constraints may result in one criterion 'ruling' or governing the MCA. In these (extreme) situations in fact the MCA is of little help, it merely shows the unbalance in the weight of (decision) criteria used.



		Gate type			
Criterion / Option	Weighting factor	Mitre gates	Single leaf gates	Rolling gates	Sector gates
1. Total costs	0.40	8	9	6	6
2. Operation	0.35	9	8	8	7
3. Local constrains	0.10	8	7	8	7
4. Navigation	0.10	8	7	8	6
5. Environment	0.05	7	7	6	7
Total :	1.00	8.30	8.15	7.10	6.50

Figure 2-16 MCA use at element level - matrix for gate selection

2.5.15 Cost-Benefit Analysis, CBA:

The Cost-Benefit Analysis in financial terms is a technique enabling expenditures and revenues to be discounted over time and normalised to a common base year. As such it can be used to enable owners to appraise projects and assist them in making decisions about:
- different strategies for projects
- evaluate different projects competing for limited expenditure.

Provided the relevant cost figures and a few other parameters are known the technique is very flexible and can if desired incorporate many items such as:
- Initial capital cost
- Financial repayment options
- Revenue streams
- Maintenance costs
- Loss of revenue
- Demolition costs

and of a different order,
- Lifetime of the structure.

Although CBA can be extended to consider the environmental impacts of the whole construction process from raw material extraction to different end of life management scenarios for the structure, the application of non quantifiable costs may add an element of confusion and divert attention from a true financial comparison of the alternatives. For evaluation of qualitative issues the use of a MCA is more appropriate.

A CBA generally is important already in the beginning stages of a project because in many feasibility studies the main decision to be made is whether to proceed with or to cancel the project. Especially projects with the burden of a set complicated demands and wishes tend to end up with cost prohibitive prices with respect to the economical and/or social benefits. Obviously in this very early stage of the design process only very rough indications of cost (+ risks) and benefits (+ values) may be given based on reference projects and index numbers (= cost or benefits per unit, m, m^2 or m^3). A wide range of uncertainty, 20 - 40%, should be taken into account, even though there is often political pressure to reduce, or just the opposite, to increase the estimates. Pressure groups nearly always use uncertainties to manipulate the CBA, thus the outcome of the feasibility study in the desired direction.

For most projects, but surely for larger infrastructural projects, benefits will occur much later and longer than the investment costs. For all the costs and benefits the Net Present Value (NPV) has to be computed, with the help of the discounting factor, to allow fair and meaningful comparison of investment alternatives. The table below illustrates the use of a CBA in a much later design stage for the selection of the type of gate.

Example: Assessment for a new weir on the Meuse in Sambeek

Option Costs (€)	Vertical lift gate (a)	Sector gate (b)	Suspended flap gate (c)	Bottom flap gate (d)
Construction	36,000,000	37,000,000	34,000,000	32,000,000
Maintenance:				
- per year	340,000	447,000	365,000	421,000
- capitalized	7,596,000	9,987,000	8,155,000	9,406,000
Operation:				
- per year	246,000	246,000	246,000	246,000
- capitalized	5,496,000	5,496,000	5,496,000	5,496,000
Totally (€)	49,092,000	52,483,000	47,651,000	46,902,000

4th PIANC WG26 Meeting, Rotterdam, March 2004. 6

Figure 2-17 Results of NPV computations used for gate selection

Costs and risks:

Though construction costs are by far the biggest contributor to the whole life costs, there are other costs like the cost of obtaining or using the building area, cost of design, cost of environmental measures, cost of operation, cost inspection and maintenance, cost of demolition and/or extra costs for future reuse, etc.

The later these costs appear in the lifetime of the structure, the more uncertainty there is about the exact time of occurrence and the height of the amount. But these uncertainties will be less dominant when counted back to the NPV.

There are foreseen and unforeseen risks in every lifecycle stage. These risks have a certain probability of occurrence and certain estimated consequences. In the first round these risks are rough estimated but if they are proved to be relative dominant, probabilities and/or consequences have to be better estimated in second round.

Benefits and other values:
For ‘private’ objects like quaywalls and jetties there are direct benefits like quay dues and (part of the) harbour dues. The transshipment activities will also generate indirect income to society in terms of employment, the possibility of added value to the goods that are transshipped, etc..

For most public works like weirs, stormsurge barriers and public quaywalls, there are no direct benefits like payments for use, for protection, transshipment of goods or passage, but only indirect benefits for the society in terms of (extra) capacity for shipping so less waiting time and maybe more increased traffic or (extra) safety against or less risk of drowning or just economical damage, etc.
When people feel save behind the primary dikerings and people or goods are easy to transship and to transport, then the value of buildings is increasing and there is a positive attitude to (more) investments.

Evaluation model / evaluation instruction:
Generally hydraulic structures require considerable public or societal investment on one hand, on the other hand considerable, however rather diffuse, benefits for society are generated, see the above paragraph on ‘Benefits and other values’. In the Netherlands a specific method Onderzoek Economische Effecten Infrastructuur (OEEI; in English: Evaluation of Economic Effects of Infrastructure) has been developed to evaluate the economic cost consequences of these social impacts in a more standardized transparent way. The OEEI-method was developed by an expert group of ministry of transport and state public works, economic and financial departments. [www.minvenw.nl/oeei].

2.5.16 Risk analysis:

Taking care of risks during design; some notes:
Risk is the product of probability of events and consequences (material and/or immaterial). It goes without saying that risks should be reduced to an acceptable level, whether the project is on system level or at an element level, dealing with overall safety against flooding or design of a single structure, etc.. Risks may be of technical, human and/or natural origin, and shall be identified for every Life Cycle stage, including risk of mistakes made in the design stage. Risks that may be identified for every project system or structure, neglecting the probability part of the definition here, are for instance:

- budget overruns
- time delays
- mal- or disfunctioning systems or structures
- poor design
- construction mistakes
- technical failure, i.e. failure either on strength & stiffness and/or displacements
- poor maintenance resulting in functional or technical failure
- etc.

Generally it is the task of the project leader to reduce the overall risk of the project, which is most easily done in the design stage because the (cost) consequences of changes are relatively small. Often risk assessments or simply risk analysis are used in order to identify every possible risk to the project as early as possible, and subsequently deal with the risk. An effective and communicative tool is the "fault tree", with basic-events at the bottom leading to unwanted top-events like the structure being out of use or structural failure. Risk reducing measures that may be considered during design:

- QA&QC to reduce the number of mistakes during "overall" design and during the risk analysis in particular
- use of back-up systems
- incorporating robustness or redundancy into the (structural) design
- use of a statically determinate structure instead of indeterminate
- the use of parallel instead of serial systems
- etc.

A threat to overall risk assessment is to underestimate contributions from the design stage (caused by a "tunnel vision" of the designer), just focusing on risky construction stages and modifying the design just to solve all foreseen troubles from the construction or use and maintenance stage.

Examples:

- *the design of the Eastern Scheldt stormsurge barrier was done by the design office of State Public Works, an organisation which was not used to have external control. In the very end of the design there was a forced control because of tax(!) reason, which revealed some design faults caused by changing loadfactors for different piers. At that moment supporting steel structures were needed to mobilize enough steel cross section!*
- *the Eastern Scheldt stormsurge barrier is designed as a parallel system of more than 60 piers and gates, that still have the ability to defend Zeeland against high tide when 5 gates are out of use. In contrary to the Maeslantkering where the two floating sector gates should be seen as a rather serial system, meaning that the top-event "structure fails" occurs when only one of the gate fails!*
- *if a quaywall is designed as a combiwall, like a chain of interconnected sections of about 40 meter and each section can withstand the active ground pressure by more than two anchors, than there is a lot of robustness or redundancy in that structure against shipcollision, deterioration of anchor rods, local overloading by lifeloads, etc.*

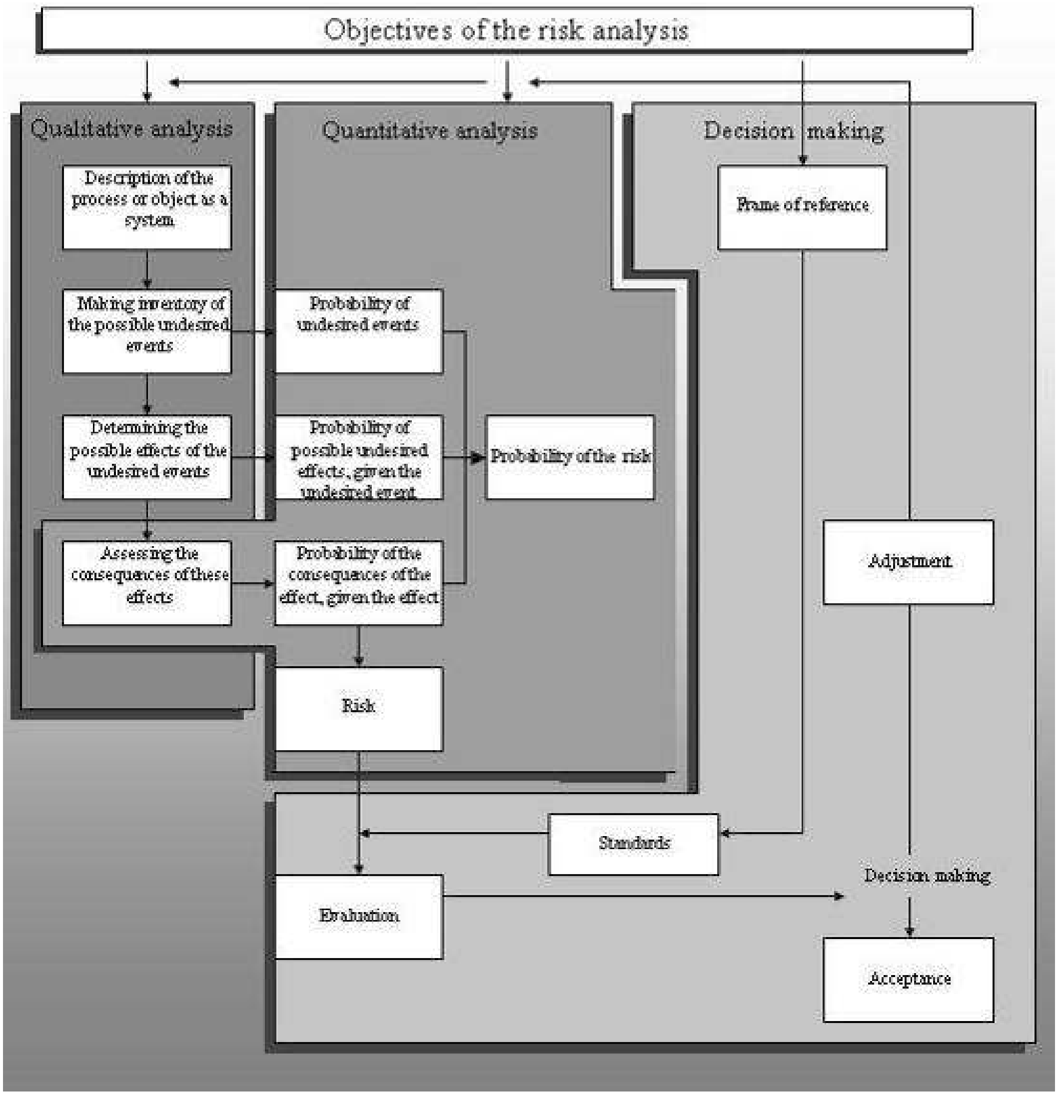


Figure 2-18 Components of a risk analysis according to CUR 190

Risk analysis and probabilistic design:
Risk analysis is a structured approach to identify and quantify probabilities and consequences of failure of a structure. The risk analysis points out whether the risk level of a system or the structural design achieves a desired risk standard. If the risk level of the design is insufficient, the analysis provides insight in how the design can best be altered to measure up to the desired standard. The risk analysis also enables a quantified comparison of different structural design alternatives.

Figure 2-18 distinguishes three main components in a risk analysis. A qualitative analysis consists of an analysis of the functions and components of the system. The qualitative analysis also forms an overview of the threats, failure modes, the consequences and the mutual connections among those. The second component, the quantitative analysis, calculates the probability of failure, quantifies the consequences and calculates the risks. In the third component of the risk analysis, decision-making, the risks are assessed by comparing the risks to the risk standards.

The qualitative analysis decomposes a system into subsystems, and components and elements. The failure modes and consequences of failure of these subsystems and components are analyzed. A failure mode is a chain of events leading to failure of a component, subsystem or the total system. The definition of failure depends on the functions of the structure.

CUR 190 discusses a number of methods that support system analysis:

- *FMEA (= Failure modes and effects analysis)* is a risk analysis which is based on the schematic approach in Figure 2-19. The main purpose of a FMEA is to give an as detailed view as possible of all the foreseen undesired events and consequences in a system or process.

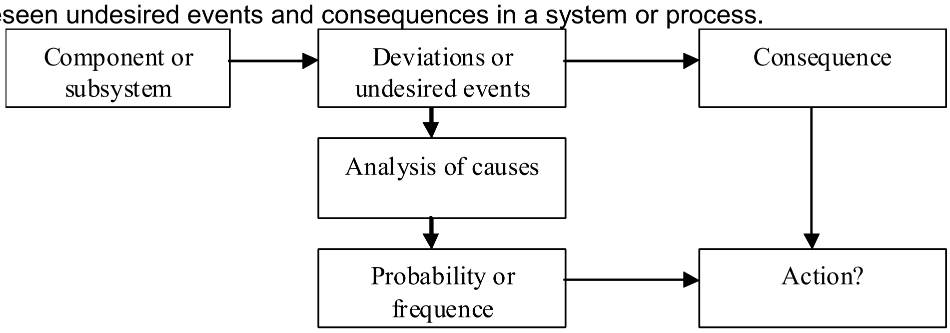


Figure 2-19 FMEA schematic approach (from CUR 190)

- *FMECA (= Failure modes, effects and criticality matrix)* is a FMEA with an additional criticality matrix. In this matrix the different failure modes and consequences are related and the consequences are ranked based on the severity of the consequence.
- *Tree of events* enables an analysis of the system response to one event. The tree of events relates in a logical way one ‘start event’ and all possible consequences. All possible events that can follow from the start event are inventoried and analyzed.
- *Fault tree* lists the logical chain of all events that lead to one undesired top event. This event is placed on top of the tree. Fault trees are especially suitable for displaying cause-consequence chains that lead to an undesired top event when one cause has two distinct consequences (yes or no, positive or negative, good or bad, failing or not failing, etc.). Only the negative consequences are listed in the fault tree.
- *Cause consequence chart* is a combination between the tree of events and the fault tree. The cause consequence chart makes the consequences of failure of an element or subsystem in an overall good functioning total system visible.

The limit state is the transition state between failure and non-failure of a structural system, a subsystem or component. Two types of limit states are distinguished according to the nature of the failure, namely:
- ultimate limit states

- serviceability limit states

The reliability of an element is the probability that the structure does not fail. The probability of failure is calculated by means of a limit state function:

$$Z = R - S \qquad (1)$$

In which R (Resistance) represents the strength of the element and S (Solicitation) summarizes the loading models affecting the element. Failure occurs if the loading, S, of the element exceeds the strength of the element, R. The limit state is defined as $Z = 0$. For example a concrete wall loaded by the hydraulic pressures of a rising water level. One of the failure modes is horizontal sliding of the concrete wall.

The strength is supplied by the resulting weight of the concrete wall, including the hydraulic uplift force, and the friction of the foundation with the subsoil. The variable load is the horizontal hydraulic pressure due to the rising water level. The volumetric weight of concrete, the dimensions of the concrete wall, the friction angle of the subsoil and the water level are examples of variables in the strength and loading models of the limit state function. The values of these variables are in reality almost always subject to variation rather than deterministic. In a probabilistic analysis this variation is acknowledged by representing each variable, x_i, by means of a distribution function. The probability of failure, P_f, is subsequently calculated from the cumulative distribution function of the strength $F_R(x)$ and the density function of the loading $f_S(x)$, with the following probability integral:

$$P_f = P\left(Z \le 0\right) = \int_{-\infty}^{+\infty} F_R\left(x\right) f_S\left(x\right) dx \qquad (2)$$

Analytical solutions are usually not available for the probability integral. There are three levels of probabilistic calculation methods to approximate the integral.

The Monte Carlo method is an example of a level III method. The method samples from the distribution functions of the variables, x_i, and calculates whether $Z \le 0$. The samples are drawn a large number of times. The amount of times that the structure fails is divided by the total number of samples to approximate the probability of failure.

Level II methods are based on the linearization of the limit state function and the expression of this function in the normal space. These methods make the computations relatively fast compared to level III methods. However, the approximations are subject to some inaccuracy. More details about this method are given in other courses.

The Level I method is most widely used in structural design and is based on safety factors:

$$R_d \ge S_d \quad \Leftrightarrow \quad \frac{R_{rep}}{\gamma_R} \ge \gamma_S S_{rep} \qquad (3)$$

in which:

R_{rep}	=	representative value for the strength
S_{rep}	=	representative value for the load
γ_R	=	partial safety factor for the strength (material factor) = γ_m
γ_S	=	partial safety factor for the load (load factor) = γ_g, γ_q. γ_R
R_d	=	design value of the strength
S_d	=	design value of the load

When determining the dimensions of the design, a strength will be selected that has a design value larger than the design value of the load.

The idea is that, by assuming 95% of the upper limit of the load and by multiplying this with a load factor a design value is acquired with a small probability of exceedance. If the 95% lower limit of the strength divided by a material factor (giving the design value of the strength) exceeds the design value of the load, the area overlapped by both curves is extremely small (see Figure 2-20). This overlapping area is a measure of the failure probability, but it is not the actual mathematical probability of failure!

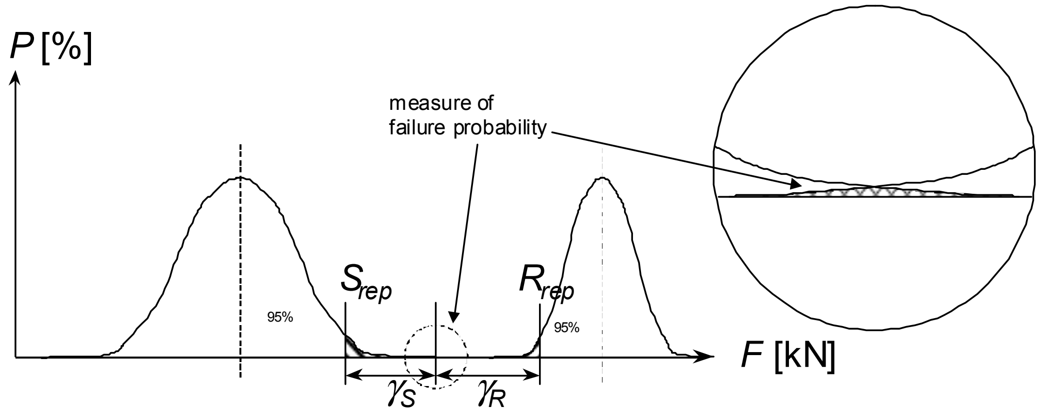


Figure 2-20 Design values for load and strength related to (selected) probability of failure

Design standards for steel, concrete or geotechnical structures specify the design factors, i.e. load and material, some times model factors, for a variety of design situations. However, it is hard to capture all of the design situations that may occur in practice in the design standards. Especially hydraulic structures are often subject to site specific hydraulic loading conditions requiring a site specific design approach, think of for instance barriers or breakwaters. The design factors can be collected from design standards for other structures or need to be established specifically for the hydraulic structure.

Example determination of design wave heights and related load factors:
The definition of safety factors is often based on a safety standard defined for a structure in an Ultimate Limit State or Serviceability Limit State. The safety standard of a structure can be expressed in terms of a reliability index. The reliability index β relates to the probability of failure P_f by a cumulative standard normal distribution Φ:

$$P_f = P(R \leq S) = \Phi(-\beta) \quad (4)$$

The cumulative standard normal distribution Φ is defined as follows:

$$\Phi(U) = P(U \leq U) = \int_{-\infty}^{U} \frac{1}{\sqrt{2\pi}} \exp\left\{-\frac{X^2}{2}\right\} dX \quad (5)$$

Every variable with a standard normal distribution can be transformed to a variable with a cumulative normal distribution function F_N by:

$$F_N = \Phi\left(\frac{X-\mu_X}{\sigma_X}\right) \quad (6)$$

In the following example the derivation of a safety factor is illustrated. Figure 2-21 shows a crown wall loaded by wave impact. The safety standard for structures in safety class 2 is given in NEN 6700:

$$\beta = 3.2 \quad (7)$$

Whereby safety class 2 is characterized by a low probability of life endangerment, a low probability of economic damage and other loading types than wind loading in the ultimate limit state. The aim of the crown wall illustration below is to establish the probability of exceedance of the Rayleigh distributed significant wave height H_s corresponding with a safety factor of γ_S =1.5.

The crown wall can fail in several different ways. This example focuses on breaking of the reinforced concrete crown wall by the bending moments introduced by the wave impact. The reinforced concrete wall is able to withstand a maximum bending moment, formulated by equations (5) and (6) below. The maximum tensile force in the steel reinforcement bars is a function of the area of the bars and the yield stress. The arm in the bending moment is the distance between the reinforcement bars and the resulting force in the pressure zone, see Figure 2-21. A rule of thumb for this distance is $0.8 \cdot d_s$, in which d_s is the distance between the reinforcement bars and the boundary of the concrete section. The strength model is given by:

$$M_u \cong N_s 0.8 \cdot d_s \quad (8)$$

$$N_s = A_s f_y \quad (9)$$

in which M_u is the bending moment capacity due to the reinforcement bars, N_s is the tensile force in the reinforcement bars, A_s is the total area of the reinforcement and f_y is the yield stress of the steel in the reinforcement bars.

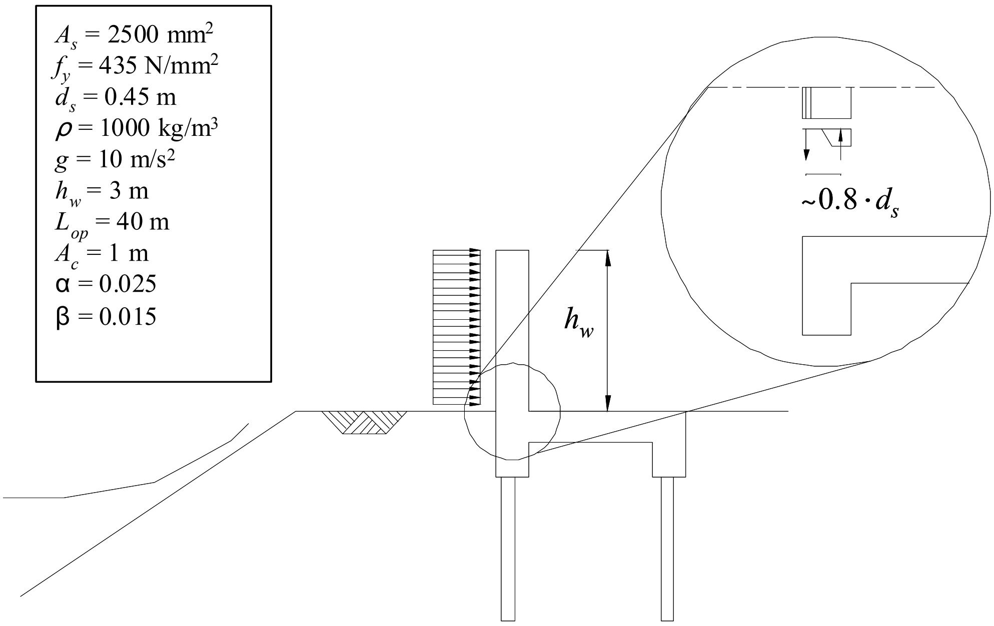


Figure 2-21 Wave impact on a crown wall (from Rock Manual)

The bending moment introduced by the wave impact force on the crown wall is given below.

$$F_{H,0.1\%} = \rho g h_w L_{op} \left(\alpha + \beta \frac{H_s}{A_c} \right) \quad (10)$$

$$M_H = F_{H,0.1\%} \cdot \tfrac{1}{2} h_w \quad (11)$$

in which M_H is the bending moment introduced by the wave impact on the crown wall, ρ is the density of the water α and β are coefficients related to the shape of the vertical wall, g is the gravitational constant, h_w is the height of the loaded wall, A_c is the distance between the water level and the crown of the slope and L_{op} is the wave length. The strength model R and the loading model S are given by:

$$R = M_u$$

$$S = M_H$$

Figure 2-21 also shows the deterministic values of the variables in the numerical example below. Based on the deterministic values, the value of the strength model is as follows:

$$R = 2500 \cdot 435 \cdot 0.8 \cdot 0.45 = 391500Nm \qquad (12)$$

The strength model *R* is subsequently assumed to be normally distributed with a variation coefficient of 0.1. The calculated value in (10) is considered as the mean value μ_R, the standard deviation σ_R is derived from the expression for the variation coefficient:

$$V = \frac{\sigma_R}{\mu_R} \rightarrow \sigma_R = 0.1 * 391500 = 39150Nm \qquad (13)$$

The loading model *S* is a function of deterministic values for all the variables except the significant wave height H_s. The variation in H_s is captured by the safety factor γ_S.

$$S = 1000 \cdot 10 \cdot 3 \cdot 40 \cdot \left(0.025 + 0.015 \cdot \frac{\gamma_S H_s}{1} \right) \cdot \tfrac{1}{2} \cdot 3 = 45 \cdot 10^3 + 27 \cdot 10^3 \cdot 1.5 \cdot H_s \qquad (14)$$

The loading model *S* is in this case deterministic. The aim is to find the H_s corresponding with the defined safety standard of β = 3.2. The normally distributed strength *R* with mean μ_R and standard deviation σ_R is combined with the deterministic loading model *S* from (14) as follows:

$$P(R \le S) = F_N(S) \qquad (15)$$

$$F_N(S) = \Phi\left(\frac{S - \mu_R}{\sigma_R}\right) = \Phi(-\beta) \qquad (16)$$

The deterministic value of H_s is derived from (16) as follows:

$$\frac{S - \mu_R}{\sigma_R} = -\beta \qquad (17)$$

$$\frac{45 \cdot 10^3 + 27 \cdot 10^3 \cdot 1.5 \cdot H_s - 391500}{39150} = -3.2 \qquad (18)$$

$$H_s = 5.5$$

The safety level of β=3.2 and a safety factor of 1.5 applied to the significant wave height therefore corresponds with a significant wave height of H_s =5.5m. A Rayleigh distribution is applied to model maximum significant wave heights H_s according to (19) below with m_0 = 2.0. H_s=5.5m then has a probability of exceedance of $5.2 \cdot 10^{-4}$.

$$F_{Ray}(x) = 1 - \exp\left\{ -\left(\frac{x}{\sqrt{2 \cdot m_0}} \right) \right\} \qquad (19)$$

According to this calculation, the structure is safe for a choice of deterministic wave heights lower than 5.5m.

Without application of a safety factor, i.e. γ_S =1.0, the wave height amounts to H_s=8.2m with a probability of exceedance of $5.0 \cdot 10^{-8}$. The use of the safety factor therefore leads to a safer design of the crown wall for H_s=8.2m with a much lower probability of exceedance. In fact a choice of wave height lower than 8.2m achieves the required safety standard.

2.5.17 Life cycle approach:

The main functional requirements come from the primary user as the main stakeholder during the expected long period of normal use. The primary user is mostly present in the initiative stage, but for sure in the feasibility stage because he is expected to bring the benefits into the project. But there are much more lifecycle stages in which different stakeholders will be active, who may have extra requirements to the hydraulic structure.

Example(s):

- *the main user of a quaywall is the transshipment company, which has functional requirements for the depth of the expected ships, the loads of the cranes and height of the storage.*
- *the owner may have extra functional requirements for future upgradability or re-usability or extra quality requirements in terms of sustainability or breakability.*
- *the local environment may have extra requirements for the esthetics of the structure (old fashioned outlook, kneeling cranes ?).*
- *the maintenance manager may have extra requirements for the inspectability or maintainability, etc.*

So on the one hand anticipating at future lifecycle stages and/or other stakeholders demands may lead to extra functional or quality requirements and surely to extra investments, but on the other hand it may probably lead to cost savings in the near and far future.

In case of much uncertainty about future (re)use this may lead to an extreme concept "the throw away quaywall" and opposite in the case of less uncertainty about future (re)use this will lead to a very flexible and durable solution.

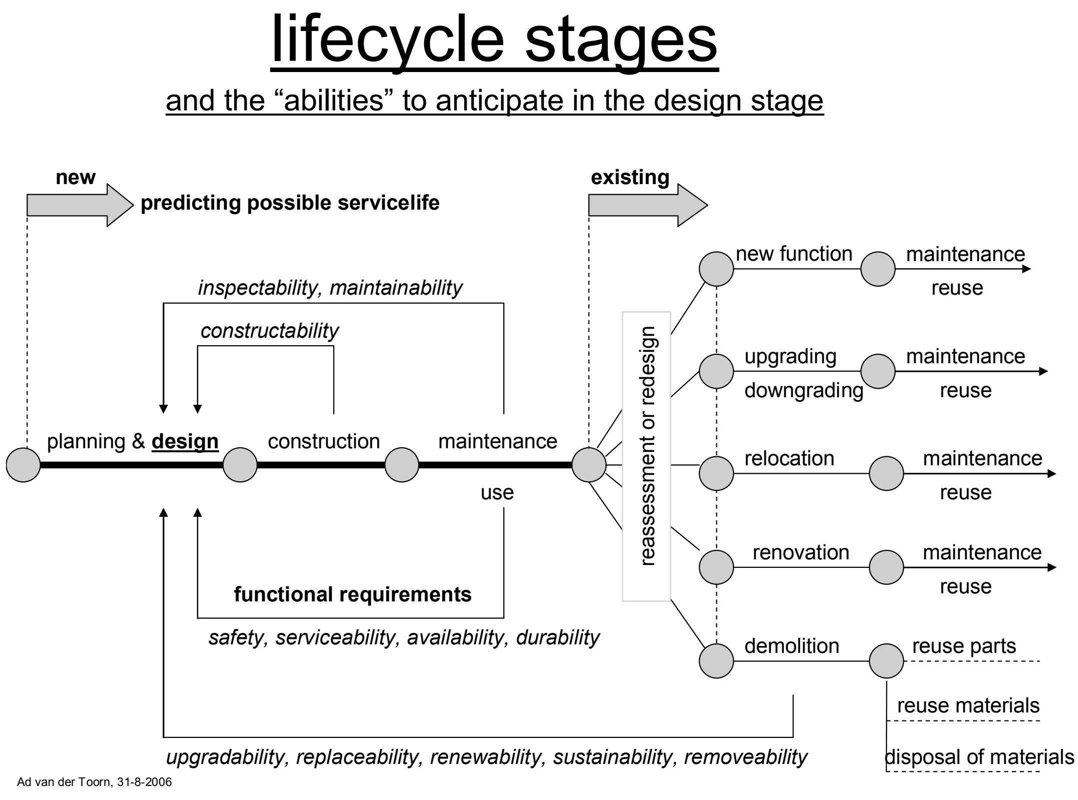


Figure 2-22 Life cycle stages and performance criteria

2.5.18 Design tools:

During the different stages of the design process there are a lot of design tools to support a designer. Sometimes he even has to use them more then ones at different levels and in different stages. For example the use of a Multi criteria analysis.

In the **initiative stage** there are:

Diagrams: to define the actual problem and possible undesired consequences.
Models: to predict growth of use, boundary conditions, loadings, loss of strength by ageing mechanisms to predict possible future problems.
Design cubicle: set of questions useful to analyse the situation or project.
Brain storms: to generate solutions for the project at hand in a more creative way. Especially useful when traditional solutions or extrapolation does not seem to result in an acceptable solution.

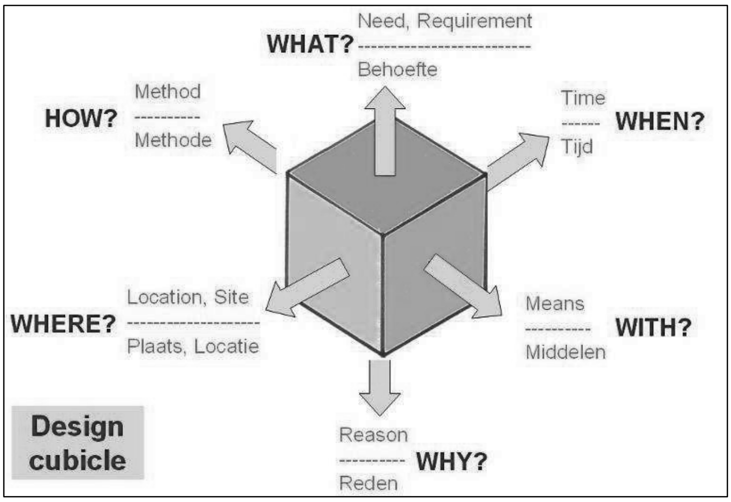


Figure 2-23 Design cubicle - useful for analyses

In the **feasibility stage** there are:

Stakeholder-analysis: to determine which parties are involved in the problem, their interests, their preferred solution and their importance for alternative solutions.
Reference projects: to get a rough idea of possible solutions.
Index numbers: to get a global idea about costs and benefits.
Risk-assessment: to get a rough overview of possible risks with respect to politics, finance, economics, environment, techniques.
Value-analysis: to have a better understanding of the real values for the direct users, owner, local environment, society, etc.
Cost-benefit-analysis: including risks and added values to get a total financial figure.
Multi-criteria-analysis: to let decision makers make a choice between alternative solutions and give the green light to the more technical design stage.

In the **technical design stage** there are:

System-engineering, functional and technical decompositions: to get a bright view at the macro system, system as such, subsystems and elements.
Brainstorming or Delphi method: to generate and evaluate alternative solutions.
Design-trees and morphological chart: to generate alternative (partly) solutions at a more structured approach.
Life cycle approach: to think about extra requirements from other life cycle stages.
Whole life costing and Net Present Value to get a good insight into all relevant costs and benefits.
Risk analysis to get a balanced idea about possible risks involved with the design, construction or use of the hydraulic structure and possible measures to reduce the risks.
Design calculations: at all kind of system levels and all kind of accuracy to come up with the right dimensions of structural parts.
Probabilistic design methods: to get a better idea about the consequences of uncertainties in the combined action of loads and strength at the design calculations.
Multi criteria analysis: to evaluate alternative solutions at the system, subsystem or element level, and make the right choice between them on the bases of relevant aspects.

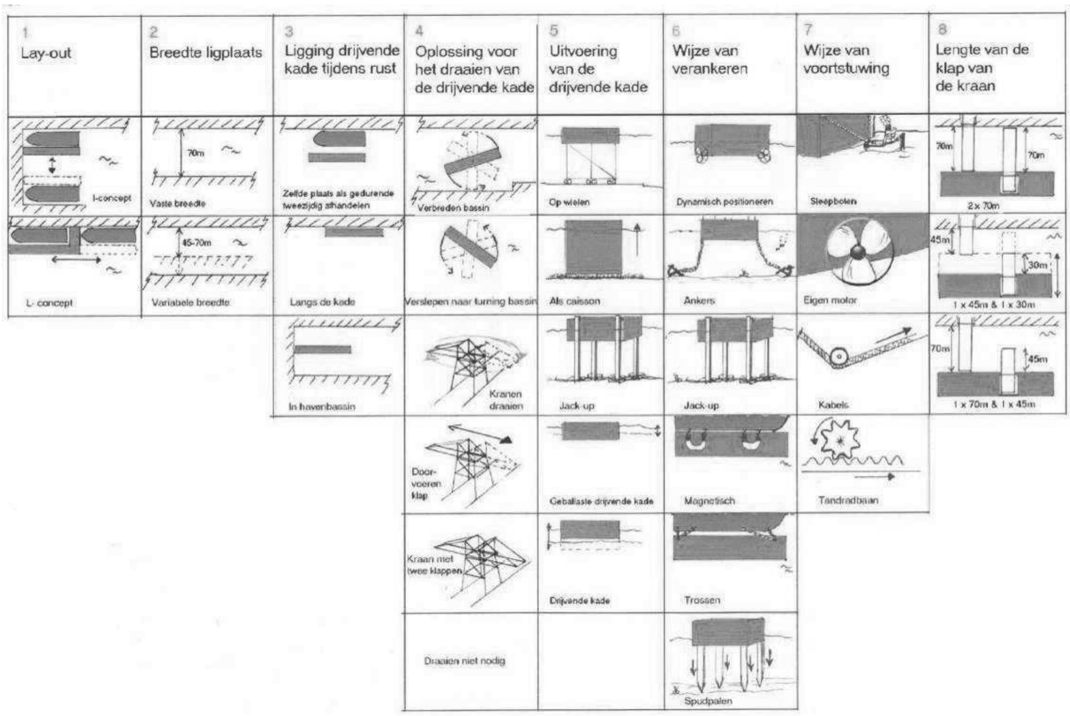


Figure 2-24 Morphologic chart for design of a floating quay – Graduate student Christian Paus 2004

3. Water, Structures, Design & Construction

3.1 Design for construction

Hydraulic engineering structures are situated in, on, under, adjacent and above water bodies. Here the concept ‘water body’ is very broadly defined. It can include groundwater, ditches, canals, harbour basins, streams, rivers, generally with fresh water, and estuaries, seas and straits, usually salt water. It is almost impossible to construct good quality concrete, masonry, wood and steel structures in water without making a lot of costs. Whenever possible the aim is to build on dry sites and to construct under water only when this is absolutely necessary. This means that the construction process is a greater determinant for the design of hydraulic structures than it is for many other kinds of civil engineering structures. The work method can influence the structure, selection of the location and orientation of the structure at the location. One of the most important motto's in hydraulic engineering is therefore:

"Don’t build in water (but if you must prefabricate as much as possible)"

To create the conditions to construct in-the dry, thus under prepared conditions, the following measures can be considered:

- In situ construction:
  - above the water.
  - outside the watercourse and diverting or widening the watercourse later.
  - in the watercourse within in a temporary retaining structure (cofferdam).
- Prefabrication:
  - The construction of large sections off site, after which they are floated to the site and positioned (large-scale prefabrication).
- Combinations:
  - Partly constructed in situ and partly prefabricated.

These work methods will be described in following (sub)sections. The choice of the construction or work method has great influence on design. Design and construction cannot not be considered separately.

Note that the time spent on designing the permanent structure may be exceeded by the time needed to design the temporary structures required for construction of hydraulic structures.

3.2 In-situ construction methods in and around water

3.2.1 Construction above water

Construction above water is only possible when the main structure is positioned above water, for instance structures constructed on piles such as jetties, berthing and mooring dolphins and bridges.

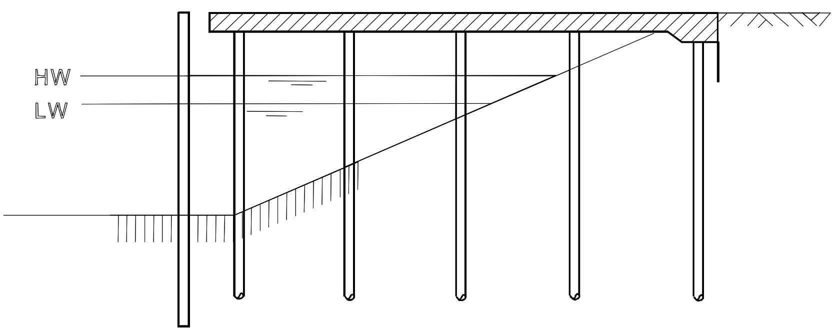


Figure 3-1 Construction above water - Principle of a jetty or open berth quay

For a jetty the (prefab) piles can be driven in position using a crane and flying hammer working from a barge or pontoon and some piles can also be driven using land based equipment working from the shore. The deck is cast on a formwork that is attached to the piles or is part or entirely constructed from prefabricated sections. The level of the deck is determined by the required end situation (inundation frequency, limitation of upward wave impact), but the formwork also requires a minimum space above the high water levels during the construction period. The high-water level chosen for this depends on:

- the exceedence frequency (and thus also the season),
- the duration of these levels,
- the duration of the construction period,
- the damage (down time, wash outs of concrete that didn't harden yet etc.) and
- other cost aspects.

3.2.2 Construction besides the watercourse

In most cases it is less expensive to build a hydraulic structure in a construction pit than to mess with water. Sometimes the structure can be built in a construction pit outside the watercourse. After completion of the work the watercourse can be redirected in such a way that the structure is included in or connected to it. The use of a construction pit requires quite some area. If there are objections to this (space required, dewatering damage), alternatives can be used for the construction pit. It is also possible to divert the entire watercourse or part of it before starting the work, then the structure can be built in the original waterbed. After this the watercourse will be returned to its previous position and the water can be led over, through or under the structure. The latter method was already used 4000 years ago for construction of a tunnel under the Tigris in Babylon Figure 3-2 and Figure 3-3 give some random examples of diversion/widening after completion of the work.

For construction of a weir and lock complex in a river, see Figure 3-2, a construction pit in a short-cut between 2 river bends can be used, this would combine construction of the structures in-the-dry with river straightening. The old disconnect river bend may be used for recreational or other purposes.

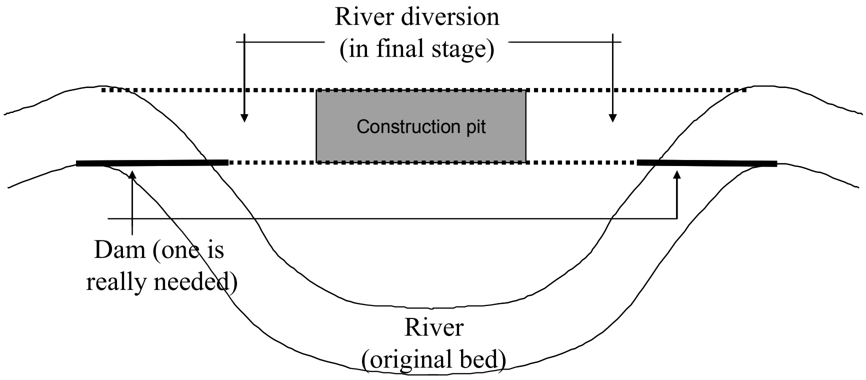


Figure 3-2 Pit outside the watercourse and (later) diversion to connect the lock

For the construction of the quay wall of Figure 1-3 only a limited construction depth was required and a limited reduction of the groundwater level. From the bottom of the construction pit the sheet pile walls and piles are driven, after which the L-wall is built. Next the landside is back-filled in and the fore bank is excavated, for example by dredging. In this situation a different type of quay wall, for example an in-situ gravity wall, would require a deeper construction pit, more space and more dewatering work.

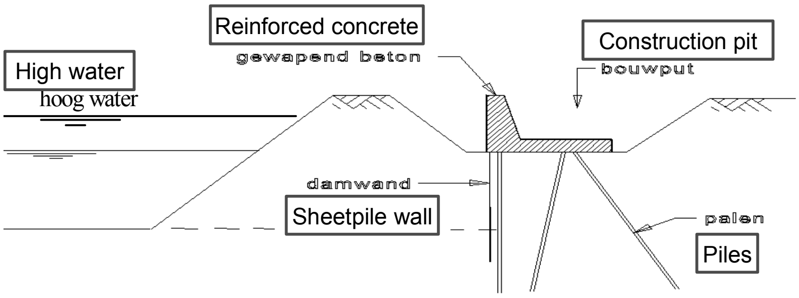


Figure 3-3 Construction pit for a quay wall

It must be emphasised that the L- wall and the gravity wall are only two of the many types of quay wall structures.

The above examples show that the possibilities to divert or widen the watercourse after or before construction of the structure, is one of the factors that determine the position of the structure.

Other issues to consider with regard to this method are:

- the amount of soil to be excavated for the construction pit, and the distance this volume has to be moved, to create the diversion or widening of the watercourse. Sometimes during excavation or dredging contaminated soil is encountered, which requires extra measures.
- the space that is required. Sometimes the presence of buildings, agriculture or a nature area does not permit the diversion of a river for the dam, weir or lock. Sometimes the positioning of a quay would result in the loss of too much storage space or industrial area.
- the possible consequences of the temporary reduction of the groundwater level.

The above is based on existing waterways. In the case of new canals (or of harbour basins that are to be dredged) there is no question of diversion or widening. Here too, structures can be built in constructions pits or variants of these, often in advance of the excavation of the canal or harbour basin. Odd as it may be to construct a quay or lock surrounded by land this is often necessary because the construction of bigger structures takes a long time. Excavation of the canal or basin itself generally takes less time. Every effort is made to ensure that the works on the critical path will be ready as soon as possible. The remaining works may be finished before but not later than that.

3.2.3 Construction in the watercourse within a temporary retaining structure

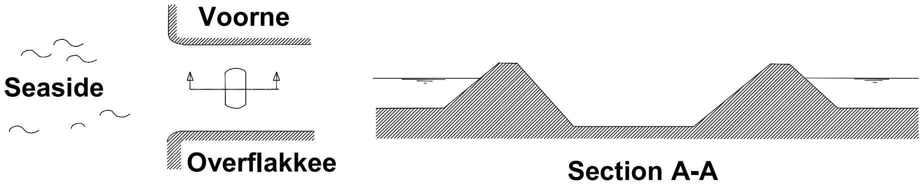
The construction site ‘in’ the watercourse has to be surrounded by a water retaining structure and could be considered to be an island or a peninsula. In the latter case the construction site can be accessed from the landward end. Depending on the situation (including the available space) and the cost, the temporary retaining structure may take various forms: a dike or cofferdam. A cofferdam is either sheet piling driven into the ground or a dam out of sheetpile walls built above the groundlevel. Cofferdams in a waterway are vulnerable to damage caused by ship collision and are usually surrounded by separate anti-collision structures on the waterside, for example dolphins, berthing beams or other fender structures.

In order to be able to work in the dry, the groundwater level is lowered below bottom level of the permanent structure, alternatively a bed seal, for example an impermeable soil layer (clay or peat) or underwater concrete, is used. Underwater concrete is usually used for sealing off cofferdams, i.e. to prevent water from flowing in from the sides and from under.
Three examples are given below:

- construction pit / island surrounded by a dike
- cofferdam / island using sheet pile walls
- peninsula cofferdam using sheet pile walls

Example: Island surrounded by a dike

It was decided to construct a dam for flood defense purposes between the islands Voorne-Putten and Goeree-Overflakkee in the province Zeeland – the Netherlands. However, the watercourse between the islands, the Haringvliet, is of paramount importance for discharge of river water into the North Sea. Therefore a part of the dam (to be) constructed is an outlet structure or dewatering gate.



The construction pit is in fact an (artificial) polder in the middle of the Haringvliet

Figure 3-4 Construction in the waterway: outlet sluice in the Haringvliet

A ring dike was constructed in a shallow part of the Haringvliet to create a polder where the dewatering gate could be constructed in-the-dry. In order to limit the need for earthworks after completion of the structure, the bottom of the pit was dredged to almost the construction level of the outlet sluice. Much of the dredged material that was excavated was immediately used for construction of the ring dike. On the Haringvliet side a harbour mole was built to provide protection for the berths that were used by ships transporting the required personnel, equipment and materials.

While the construction pit existed the channels deepened, especially north of the island. After completion of the structure channels were dredged to provide access to the outlet sluice in the shallow parts and the ring dike was removed. As a result of the increased wet cross section the current velocities declined. This was favourable for construction of the closure dam to the north and south of the outlet sluice. In the southern section of the dam there is a navigation lock that was also built within a ring dike.

Very short after start of construction of the outlet sluice started it emerged that most of the construction labour lived on the south bank and that too much time was being lost in transporting them by ship. It was cost effective to build a temporary fixed connection (partly a Bailey-bridge and partly part of the future Haringvliet dam) between the construction pits of the outlet sluice and the Overflakkee lock. This link was primarily used by the workforce and to transport light equipment.

Figure 3-5 Construction pit (period 1958-1968) - left,. Overview Haringvlietdam (2005) - right

Example: cofferdam / island using sheet pile walls

Many piers of bridges have been and will be constructed in cofferdams in rivers or sea straits.

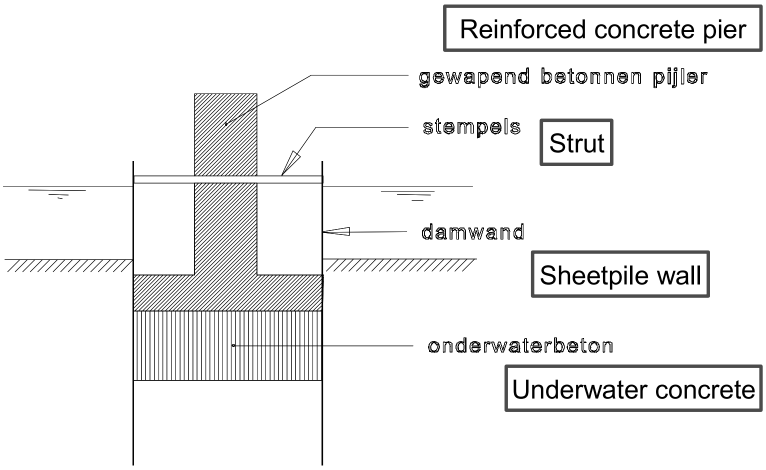


Figure 3-6 Cofferdam for a pier in a river

The sequence of construction is: driving the sheet pile walls from a barge with a crane on top of it, placing the struts, excavation, pouring the underwater concrete floor (If necessary this can be anchored to the previously driven sheetpiles to prevent heave resulting from the water pressure), pumping out the cofferdam, construction of the pier, filling the cofferdam with water, removal of the struts and sheet pile walls. The sheet pile walls can then be cut off just above the pier foundations by divers, if case an anchor between (underwater) concrete and the sheets was used, otherwise the whole sheet pile is retrieved.

The sheet pile wall is vulnerable to collisions. In this regard, the safety of the people who are working within the cofferdam is very important. Fendering (protection against collisions) is therefore necessary. Transport to and from the cofferdam is done by ships and for this purpose a berthing and mooring facility is needed close to the cofferdam. For larger structures (such as the piers of the storm flood barrier in the Thames) a temporary access bridge may be cost effective. The access bridge to the cofferdams used for the piers of the Thames flood barrier was combined with work platforms round the cofferdams.

Example: peninsula cofferdam using sheet pile walls

In Phase 1 the first half of the tunnel is constructed in a cofferdam that extends just beyond the middle of the watercourse, see Figure 3-7. The right end of the tunnel tube is closed by a temporary watertight bulkhead before the end of this phase. The construction sequence is broadly similar to that used for the cofferdam of the river pier. Thus a little less than half of the waterway is available for shipping and the possible water current. After completion the sheet pile walls are removed (cut off) and the remaining trench is back-filled up to the level of the original bottom, or slope the waterway or to the original dike/landscape.

In the cofferdam of Phase 2 the right end of the tunnel that was completed during the first phase is within the cofferdam, so that the following part of the tunnel can be built on to it. The sheet pile wall on the left end of the second cofferdam must fit like a collar round the part of the tunnel that has already been completed. This is not a simple matter, but it will not be considered here.

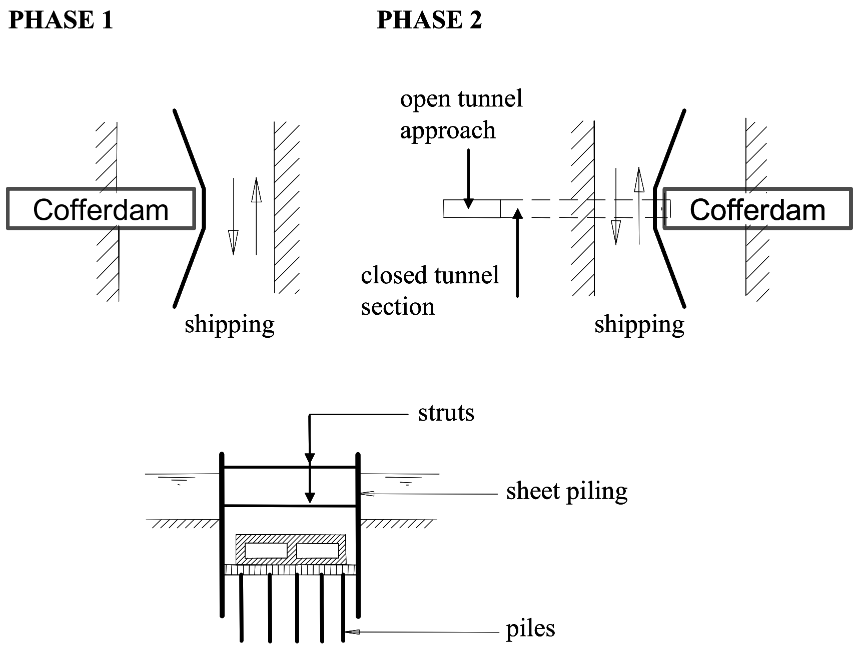


Figure 3-7 Cofferdam constructed in-situ for a tunnel

The narrow passage increases the chance of ship collision with the cofferdam. Guiding or fender beams (for example vertical steel piles with continuous horizontal fender beams) are used as an aid to ship navigation and to prevent collisions with the cofferdams.

In narrow canals consideration can be given to (temporarily) widening the navigation channel at the opposite side of the cofferdam. This has been done at the Gouwe-aqueduct in the A12 and in some other places, but it does lead to a closed tunnel section that is longer than would have been necessary for crossing the waterway with its original width.

3.2.4 Some issues regarding pits and cofferdams in and around water

For all the temporary retaining structures that have been mentioned the question arises what retaining height should be used in the design considering varying water levels and waves. This is a question of the duration of the period the retaining structure is used and of the consequences of possible collapse or overtopping. For the temporary retaining structure (construction phase) a larger probability of failure and thus a lower design height for the water levels (including waves) is acceptable than for the permanent structure itself (operational phase generally much longer). Thus for example, the ring dike of the construction pit in the Haringvliet was designed for sea levels with a probability of exceedence of 0.02 per year (construction phase), whilst for the outlet sluice in the Haringvlietdam a probability of 2.5×10^{-4} per year was used (operational phase). It is not possible to give a general rule for this and each case must be individually investigated.

Other issues coming into play regarding construction within temporary retaining structures in a watercourse are:
- the costs of construction and removing the temporary retaining structure.
- the obstructions to shipping and the flow of water (impounding upstream, currents that are too strong where this is undesirable, etc.) caused by the structures. For example, higher flow velocities can result in undesirable erosion of sand banks and subsequent loss of environmental and ecological values.
- the isolated position of construction pit or cofferdam solutions. Transport per ship results in higher costs for the transport of personnel, equipment, and materials. As previously mentioned, for large

projects the alternative of a temporary bridge is often chosen. However for small hydraulic structures such an investment will not pay off.
- the work must be organised and/or planned better than for works "on land", e.g. the absence of a (small) part can lead to hours of forced work stoppage for many people and expensive equipment.
- working within the cofferdams, which is usually experienced as confined space by the workforce, often reduces the production per worker. For the Gouwe-aqueduct, constructed in a cofferdam, production amounted to only 75% of what it would have been if the same tunnel had been constructed in a construction pit. In fact a peninsula-solution was used, in which the accessibility from the land was considerably better than it would have been in a real island look alike situation.

3.3 Large-scale prefabrication

Let's state the obvious first: "There is no prefabrication without in-situ construction". But there are enough projects were prefabrication was/is predominant.

The idea about large scale prefabrication is to construct large parts of the structure (or the entire structure) elsewhere in controlled conditions (in-the-dry), after which these parts are transported, not seldom floating transport, to the construction site, installed in the required position and connected to the each other or to other structures. The big advantage is that the best location can be found (even in other countries) to manufacture prefabricated sections/parts. The disadvantage is that there are additional transport, assembling and foundation problems. These factors must be weighed against each other.

One of the best known examples of large-scale prefabrication is immersed tunneling. Since the tunnel sections are manufactured outside the watercourse, immersed tunneling causes less impediment to shipping and the water regime than the method ‘peninsula surrounded by a sheet pile wall’.

3.4 Combinations of in-situ and prefab construction methods

Two examples will suffice to illustrate how in-situ construction and prefabrication were combined in such a way that neither of them was predominant.

Example: Thames flood barrier
As mentioned previously, the piers of the storm flood barrier in the Thames were built within cofferdams. Initially the use of a cofferdam was also considered for the sills between the piers. Finally the alternative proposed by the consortium of contractors "large-scale prefabrication” was used. The sills (in fact large hollow concrete beams) were made elsewhere, floated into position and immersed between the piers.

Example: Zeeburger tunnel
For the Zeeburger tunnel, running under the Buiten-IJ – east of Amsterdam, originally the "peninsulas surrounded by sheet pile walls" method was considered. Finally the proposal of the contractor was chosen. This was to construct a larger middle part of the tunnel, in the shipping channel, using the immersion method (large-scale prefabrication), while the side parts of the tunnel (where the water was less deep and there was little shipping traffic) were constructed in cofferdams with a bottom seal of underwater concrete and tension piles (in-situ, peninsula-method).
On the southern river bank the cofferdam was used as a construction dock for the tunnel sections that were to be immersed. After the last tunnel section had been floated out and immersed, the remaining part of the tunnel was built in this cofferdam. By using this construction method (first using the cofferdam for prefabrication then for in-situ construction) the total construction period was longer, but it saved construction of a new dock. Existing construction docks were/are west of Amsterdam, which implies that tunnel elements would have to pass the Oranjesluizen, a lock too narrow for the width of the required tunnel elements. A new dock east of Amsterdam could have been constructed. However, there were environmental objections to the construction of a new eastern dock. Irrespective of the dock’s location, the

Buiten-IJ would have to be deepened at several places for the transport from a new dock to the immersion trench (the navigation channel was also too shallow), which was also objected for environmental reasons. All these problems were avoided by the double use of the cofferdam at the southern bank.

3.5 Selection of a construction method

Whether a construction method is suitable for the project at hand depends every time, and again, on the project and/or situation.

Consider construction pits for lock or weir construction:

- the construction pit for the dam and outlet sluice situated in a cut-off that is to be made later, is only suitable if the river has a very meandering course
- the diverted watercourse should suit the water and sediment transport function of the river, and allow development of navigation
- in addition there is the question of whether the existing buildings, agriculture and natural areas permit such an intervention.

Compare two large closures, the Haringvliet-dam and the Eastern Scheldt barrier, fairly similar natural conditions, still completely different construction methods were used for large parts of the works:

The island like construction pit for the Haringvliet outlet sluices could only be constructed because the estuary was wide enough. Sufficient wet cross section remained for the discharge of the river Rhine water, the in and out going tidal currents, as well as for the shipping. Due to its location on the shallows (circa N.A.P -6 m), the cost of the temporary ring dike was relatively low. To a certain extend it could be stated that the shallows determined the position of the outlet sluices.

In narrow waterways ring dikes with slopes will require too much space and use of a cofferdam with vertical walls is more suitable, provided there is enough space for this alternative solution. Although there may be enough space in larger or wider water areas, still the ring dike option is not always the preferred solution. The Oosterschelde storm surge barrier was entirely prefabricated and placed in the deep tidal channels, while the dam sections were built on the sandbanks.

The prime reason for closure of the Oosterschelde was safety for floods, which could have been provided by construction of a dam. Environmental concerns resulted in a compromise where a part of the barrier is a dam and other parts are an open barrier structure, i.e. opened most of the time but definitely closed during storm surges.
The primary functional, at times conflicting, requirements for the barrier were:

- flood defence thus water retention
- to maintain, in so far possible, specific natural values. Amongst other things the latter meant that: the Oosterschelde basin should remain saline, the reduction of the tidal amplitude should be as minimal as possible (this determines the flow through wet cross section of the storm flood defence) and maintaining the sandbank areas.

The latter was an important factor for selection of the construction method.

Looking at the ring dike solution for the Haringvliet, for the Oosterschelde barrier there would have been two options: islands situated on the sand banks or islands in the channels.
The first option would have led to cheap ring dikes, but would have been in conflict with the primary requirement: preservation of the sandbanks. After all when the storm flood defence had been completed it would have been necessary to dredge access channels through the sandbanks, which would not only have resulted in extensive dredging, but in unacceptable reduction and damage to nature as well.

For the second option, ring dikes in the channels, construction of the dikes would have been very expensive and would have resulted in the loss of the sand banks: the tidal range resulting in large ebb and flood water volumes, in combination with the 'closed' parts of the Oosterschelde would force large currents over the sandbanks, which would completely erode them.
Largely for these reasons the use of large-scale prefabrication provided the obvious solution.

Did such consideration play any part in the Haringvliet? No, the primary functional requirements were different: reducing the danger of flooding (increasing safety), controlled discharge of Rhine water and the creation of a freshwater basin on the inner side. In other words: a radical change in the environment (salt water to fresh water). In this changed situation the retention of the shallows (through which access channels were dredged to the outlet sluices) played no part.
Thus preservation of the natural environment was not an issue and by no means an obstruction for the use of a large construction pit that would definitely reshape its surroundings. At the time that decisions had to be made for the Haringvliet, the nineteen-fifties, prefabrication was not as well developed as later. For these circumstances, in-situ construction, the use of a large island was the logical solution. Had it been later, then for construction of the outlet sluices, large-scale prefabrication would have been considered because the technique of prefabrication improved and might have been more cost effective.

As criteria for selection of a construction method, assuming that the situation permits freedom of choice, the following can be mentioned:

- Costs, which includes:
  - construction costs
  - costs made by the client such as design, purchase of land, indemnifications
  - costs of environmental measures, mitigating or compensating changes or loss of environmental values
- Required construction time, in so far as this is not already taken into account costs
- The influence on the society and natural environment, in so far as this is not taken into account in costs.

Generally stated this means that it is necessary to strive to ensure that “all important aspects are given a price ticket”. This makes the financial comparison reasonably objective. In addition to the financial side there are always other matters that require consideration (human life, appearance, environment, quality of life, nuisance, etc.). It must be remembered that money is not an aim, only a means. Indeed: well-being comes before prosperity. Evaluating the material and intangible aspects against each other is a societal matter and therefore often a political affair.

4. Construction method: in-situ

For construction the most important task is to make ‘dry’ what otherwise would be ‘wet’’. In this chapter the following in-situ construction methods serving that objective are considered:
- Construction pit
- Cofferdam
- Geotextile / Membrane
- Cut and cover method
- Pneumatic caisson
- Drilled tunnel

All these cases involve methods in which two functions are fulfilled:
1. Relative water tightness
   This function can be satisfied passively with horizontal and vertical walls (for example: underwater concrete slab and sheet pile walls) or actively by pumping. Pumps can be used temporary or permanently.
2. Stability
   This concerns both vertical stability and horizontal stability: vertical stability in relation to the heave of the structure or the soil layers or a possible underwater concrete slab: horizontal stability in relation to the soil retaining structure (for example concrete wall, sheet pile wall or slope, etc.)

4.1 Construction pit

Introduction

The difference between a construction pit and a construction cofferdam is simple: In a construction pit the excavation is bounded by slopes, while in a cofferdam the excavation is enclosed by walls (usually sheet pile walls or combiwalls).
In a construction pit the groundwater level is lowered to beneath the bottom of the pit during the construction period with the aid of a dewatering system.
In a cofferdam the groundwater level can be lowered by dewatering, but it is also possible to make a bottom seal or to use a sealing bottom layer. It is also sometimes possible to do this in a construction pit.

A construction pit is used to create a space in which a structure can be built on a dry site, and thus under controlled conditions. With a cofferdam it is possible to include the wall and/or the bottom seal in the structure itself.
The choice between a construction pit and a cofferdam is determined by the situation, the type of structure and the requirements set by the surroundings. If there is sufficient space and dewatering is permitted (and will not lead to serious damage), a construction pit is usually less expensive than a cofferdam. After all, with the latter method it is necessary to construct retaining walls which cost more than extra earthworks. This is a general rule to which, of course, there are many exceptions.
Because the costs are relatively low construction pits are more often used. For the method of “construction outside the watercourse” this is an obvious option, while for "construction in the watercourse", for example the working area for the outlet sluices in the Haringvliet, can be considered to be a construction pit.

A construction dock for the prefabrication of sections is often built in the form of a construction pit. Because of the great importance of the construction pit, a number of aspects of construction pits are considered below (dimensions, slope stability, dewatering and possible environmental problems). the alternative option for the construction pit, the cofferdam will be discussed as well.

Construction sequence

The order of work for a construction pit is as follows:

- Installation of the dewatering system, see Figure 4-1.
- Excavation.
- If it is not possible to use a spread foundation, the installation of piles from the bottom of the construction pit (not shown in the Figure).
- If necessary, the installation of screens to prevent piping.
- Installation of a concrete blinding that is 6 to 10 cm thick.
- Making the concrete structure (in the figure a basin profile, for example a lock or a tunnel exit, but also possibly a tube profile or a multi-storey building such as a pumping station).
- Backfilling of the structure, preferably with the soil excavated earlier from the pit.
- Stopping the dewatering process.

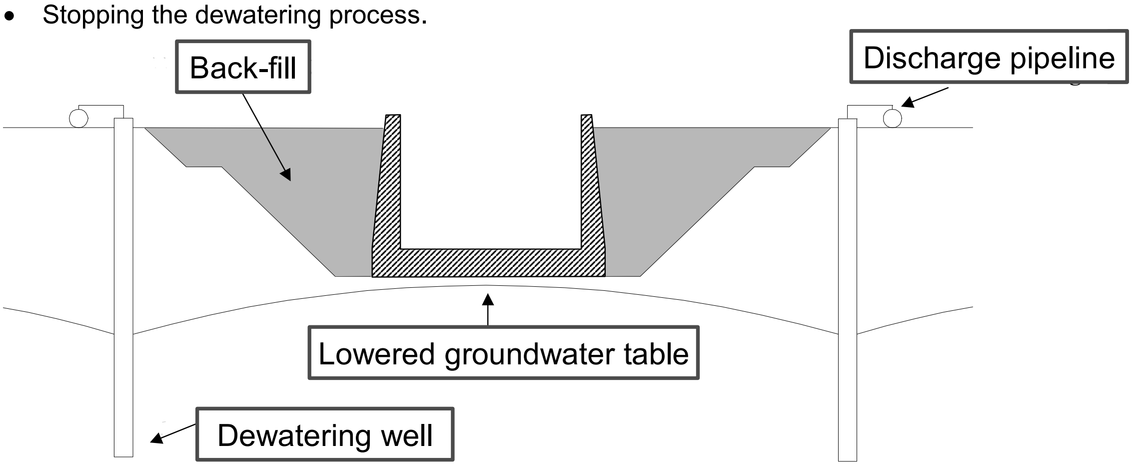


Figure 4-1 Construction pit

The excavation of the construction pit usually takes place on a dry site, though it is also possible to excavate a construction pit by dredging. The dredgers then make an access channel from the open water to the site of the construction pit. After the dredging of the pit the dredging equipment is removed via the channel and this is closed, either dammed or completely back-filled, so that the pit can be dewatered. Alternatively demountable cutter dredgers could be used avoiding the extra work of dredging the access channel and the subsequent restoration works.

Amongst other things, the choice between wet and dry excavation work is determined by:

- The accessibility of the construction pit to floating or demountable dredging equipment (which is transported by road). If the construction pit is adjacent to open water it is only necessary to dredge a short access channel.
- The availability of sufficient room for depots and sedimentation basins in the immediate area. Often dredging depots cannot be made as high as those constructed of dry materials, at least if the work must be completed in a short period and therefore requires more space for the necessary sedimentation basins than those excavated in dry conditions.
- The available construction time. With a dry excavation process it is possible to start building the structure when still only part of the pit has been completed. However, with a dredged pit, after removal of the dredging equipment, it is necessary to construct a closure dam. Only after the dewatering of the entire pit can the construction start. A quicker start translates into lower interim interest and overhead costs.
- The dewatering time. With a dry excavation the dewatering of the slopes is more gradual and more time is available for this, since the excavation takes place layer by layer and the dewatering starts before the pit has reached its full depth. In a dredged pit, however, the water level is only lowered after the slopes have been completed over the full height. If the water in the pit is pumped away quickly and the groundwater levels (or water pressures) under and behind the slope do not react to this quickly enough, but do react quickly to the dewatering pumps that have been put into operation, this can lead

to the collapse of the slopes. This is especially true for poorly permeable soils. Naturally this can be compensated by timely starting of the dewatering and pumping the water out of the pit slowly, but compared to the dry excavation, with its inherent gradual dewatering and timely dewatering of the slopes, this can lead to extension of the construction time.

- The combination of time and costs. Certainly for large construction pits the rate of excavation will be slower for dredging than for dry excavation, but that must be set against the fact that the price per m³ is also lower.
- For cofferdams with tension piles and underwater concrete there is a rather different story. In this case much expenditure can be saved by dredging and only lowering the water level after the installation of the underwater concrete. The sheet pile walls can then be lighter and shorter because the underwater concrete functions as a support that thus reduces the forces on the wall.

Dimensions

With regard to the depth and dimensions of the bottom of the construction pit the following should be noted.
The bottom lies at the level of the underside of the structures that is to be built. The pit does not have to be the same depth over its entire surface; the slabs of the piers of outlet sluices, for example, must be thicker than those of the chamber. Moreover it may be necessary to make the bottom of the construction pit slope, for example for a tunnel exit. In a dredged pit the smoothness and the height of the bottom will not be as accurate as is required for the finished structure. It is therefore not unusual for the pit to be made les deep than required and to remove the last layer of soil by other means when the pit is dry. In this way a greater degree of accuracy can be achieved.

The horizontal dimension of the bottom of the construction pit is the sum of the external dimensions of the finished structure, the space required for the formwork with supports, the space for work roads (if required), crane tracks and storage space.
The slopes will be made as steep as the stability permits. This is done to limit the amount of earthmoving that is needed: first the excavation, later back-filling.

Supply - access to the site & cranes for lifting

Supplies to the site include the whole volume of construction material needed, in a wider interpretation all the equipment and the workforce (every day) could also be considered as supply and delivery items. It's on thing to provide access to the site by means of (temporary) roads and slopes or ramps, but the next thing is to transport everything to the right spot on site, especially the materials. Cranes are instrumental to this; the radius or reach and the corresponding maximum lift load determine the number of cranes given the dimensions of the construction pit. An example of the layout of the construction pit for a lock is given in Figure 4-2. This is only one example, there are other methods.

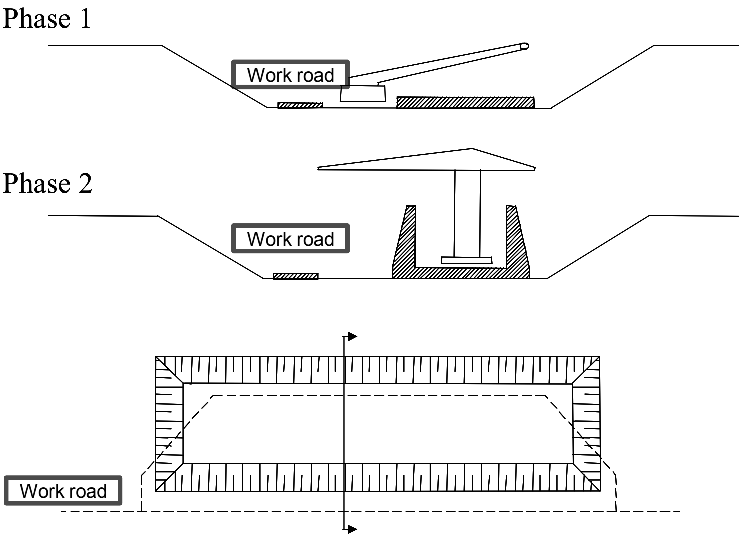


Figure 4-2 Construction in a construction pit

The work road runs along the slope from ground level to the pit bottom. The line of the road is such that freight wagons leaving the pit an continue to travel in the same direction. This promotes the through flow in busy periods, especially that of the concrete carrying vehicles during the casting process.

In Phase 1 the materials for the slab are transported to the site by a mobile crane or from the road by freight truck. In Phase 2 the walls are made with the aid of a travelling tower crane, the required rail is assembled on the ready floor slab. The phases, thus floor and wall construction, overlap each other.

In addition to the cranes shown, other equipment can be used, including concrete pumps and piling frames. The execution of the work benefits from ample work and storage space and work roads. However this leads to extra dredging and purchase or renting of land. For each situation the total costs must be optimised.

Slope stability

In order to limit the amount of dredging and acquisition of land the slopes are designed to be as steep as possible. The more quickly, steeper and higher a slope is constructed, the greater the chance of heave or sliding of the slope. The slope stability depends on the type of soil, the groundwater level, the depth of the excavation and whether or not there are banquettes.
In Figure 4-3 the basic principle of slope stability is shown schematically. The calculation method is considered in detail in the Manual. (Part 2, "Material, Soil, strength").

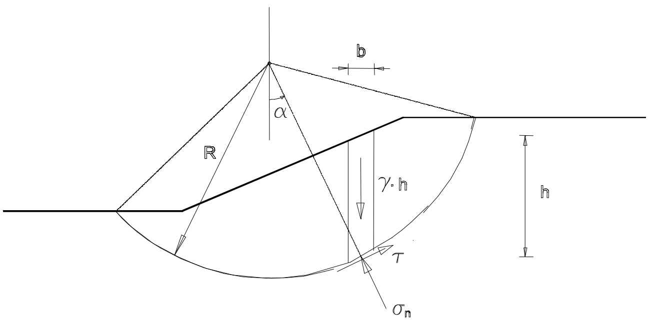


Figure 4-3 The principle of a sliding plane calculation

Dewatering

Nowadays it is increasingly often impossible to use dewatering installations because of certain disadvantages of this process. These disadvantages are:

- Dewatering permits are usually given for one year, which is too short for most large hydraulic engineering projects.
- Sometimes there is contaminated groundwater. This may not be simply discharged.
- Dewatering may cause settling of the ground, which leads to problems for existing foundations.
- If there are wooden foundations in adjacent buildings they may rot.

A good example of the consequences of soil settlement is shown in the figure below.

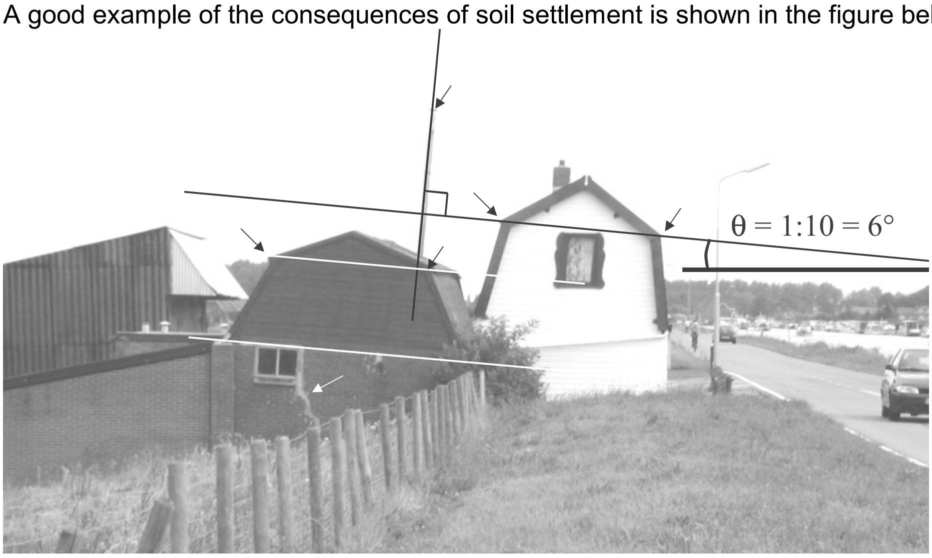


Figure 4-4 Settlement of a house (rotated) due to dewatering a construction pit

During the construction of the aqueduct for the crossing of the A4 by Haarlemmermeer Ringvaart (ring canal) the canal was tempoararily diverted and the aqueduct was built in-situ. The dewatering of the construction pit (near the row of trees on the horizon in, see Figure 4-4) the dike of the ring canal subsided as a result of which settlement occurred. The rotation of 1 op 10 is very much bigger that the standard that now applies (1:300).

If such drainage problems arise consideration can be given to the use of:

- Return drainage
- impermeable screens

With return injection the water that is pumped out is returned into the ground further away to maintain the groundwater level there. For this purpose the drainage pipelines are connected to wells with filters, via which the water that has been pumped out is injected into the ground at some distance from the construction site. It should be noted that return water always requires stronger pumping. The shorter the distance between the dewatering pumps and the return pumps, the greater the pumping capacity that is required.

It is also possible to make impermeable screens, for example cement-bentonite screens or simple steel sheet pile walls. These must then extend to a natural poorly permeable bed.

In principle there are two sorts of groundwater: Phreatic water and pressure water. In the former situation there is a free water level in a permeable bed or soil layer; in the second there is a water-saturated permeable bed, or series of water pockets trapped between two poorly permeable layers. In this case the hydrostatic pressure head of the groundwater in the saturated bed extends to above the underside of the upper impermeable layer.

Depending on the soil and groundwater conditions at the site where the construction pit is made, it is possible to choose between two types of dewatering system:

- Phreatic (surface) drainage: the lowering of the phreatic level to circa 0.50 m beneath the pit bottom in order to build the structure on a dry site. The circa 0.50 m extra lowering is necessary to ensure that no part of the work site is softened (frost heave can also play a role), to allow for heavy precipitation and to ensure that if one of the pumps breaks down the pit is not immediately flooded (under water).
- Water pressure dewatering: the prevention of heave of the pit bottom when there is a poorly permeable layer underneath it. If the hydraulic pressure head in an aquifer beneath the pit is not sufficiently lowered by the water pressure dewatering (which concerns pressure water), the bottom of the pit will be forced up because, as a result of the excavation, the top load has been reduced.

In some bottom profiles, to some extent depending on the depth of the construction pit, it may be necessary to lower the phreatic level beneath the level of the construction pit bottom and also to apply pressure dewatering under a poorly permeable layer, the upper side of which is at some depth beneath the bottom of the pit.
To prevent slope instability the dewatering process must ensure that the hydraulic pressure head and the water pressures close to the slopes are reduced.

To ensure operational reliability it is usually preferable to use a number of small infiltration wells (in the order of 10 m^3/hour) rather than a few larger ones (in the order of around 60 m^3/hour). The design, installation and operation of the system must be carefully carried out. For example, when the system is being aerated, iron flocculation may occur, causing blockage of the filters. Infiltration filters may also become blocked by the accumulation of gas (owing to the reduced pressure methane gas that is dissolved in groundwater forms gas bubbles when pumped up).

Alternatives to the construction pit

The principle reasons for using a cofferdam or another construction method rather than a construction pit are:

- Construction pits take up much space. This is especially important when construction is to take place in urban and industrial areas, nature areas or when adjacent to an existing structure (for example when the capacity of a lock or outlet sluice has become too small and it is necessary to construct a second lock or sluice).
- The first mentioned drawbacks or a fall in the groundwater level. This can lead to a system up in which dewatering is entirely or partly avoided or the consequences of this are limited (return injection).

Depending on the situation, in addition to these two points, more requirements may a part during the construction, such as, for example:

- Maintaining traffic flow: This can be done for example by temporary diversions around the construction site, but this is not always possible. It may then be necessary to construct temporary bridges over the construction site, or to choose a construction method that disturbs the ground level little or not at all.
- Limitation of noise and vibration nuisance in the areas (caused by example driving piles for sheet pile walls); using diaphragm walls instead of sheet pile walls is one of the possible options.
- Limiting deformation of the subsoil that may have detrimental effects on the bearing capacity or the foundations of adjacent structures. This can lead the need for heavier sheet pile walls for cofferdams than the strength calculations had demanded (a heavier sheet pile wall means less bending out and therefore less deformation of the soil under the foundations of neighbouring properties). Consideration can also be given to the use of diaphragm walls or augered grout pile walls, which also have a higher bending stiffness than steel sheet pile walls.

4.2 Cofferdam

One of the most frequently used alternatives for the construction pit is the cofferdam. This consists of vertical walls and a horizontal closure. This horizontal closure can be a sealing soil layer or a man-made layer (underwater concrete slab, possibly anchored by tension sections, an injection layer or a membrane). It is always necessary to take into consideration that the sealing or impermeable layer may start to leak and thus to install emergency generators, pumps and over pressure valves.
The advantages of a cofferdam are the small space required and the saving on dewatering system. The disadvantage is that the cost is higher than for a construction pit.

Sealing layer

Fortunately in the Netherlands and elsewhere sometimes there is a good sealing clay or peat layer beneath bottom of the pit.

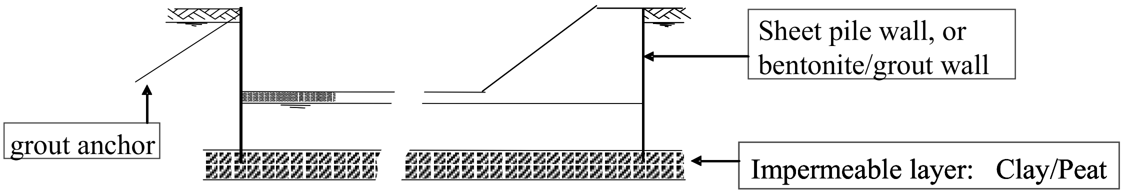


Figure 4-5 The use of a natural sealing layer (Cofferdam - left; Construction pit - right)

In that case the options marked A and B can be used when making a construction pit. With B a lighter sheet pile wall profile (or a clay-cement screen, example, 0.08 m thick) is sufficient and no grout anchors (or struts) are necessary. Against this must be set the disadvantage the area required for the site. The weight of the clay layer and of the remaining soil layer above this must be greater than the upward water pressure against the clay layer. Often a safely factor of 1.1 to 1.2 (heave criterion) is taken.
Some water drawbacks will remain because the clay layer is never absolutely watertight and moreover interlock leakage of the sheet pile wall must be taken into account, as well as seepage through the clay-cement-screen. Where the need for more room plays no part, option B is often used as the final structure for roads in cuttings and tunnel entrances (Drechttunnel) by constructing the road on the bottom of the excavation. In this case - as the only exception this section – the construction method is less expensive than that for a construction pit: after all it is not necessary to build a reinforced concrete structure anchored on tension piles.

In principal a poorly permeable layer can also be artificially created by using injection lances or the installation of a horizontal injected layer (thickness 1 to 1.50 m) between the undersides of the sheet pile wall screens.
Coarse alluvial soils can be injected with stable mixtures based on clay and cement.
For mid-fine sand (k = 10^{-4} to10^{-5} m/s) gels based on water glass are used.
Because of the relatively high cost of injection, efforts are made to limit the extent of the area and seek solutions like those shown in Figure 4-5A, such that the injected layer ends just to the right of the sheet pile wall and the layer is relatively thinner than the clay bed shown in the figure. Even when there is an injection layer it is necessary to take seepage into consideration.

Underwater concrete slab

If there is no sealing layer, usually blinding or an underwater concrete blinding is anchored to tension piles as shown in Figure 4-6.

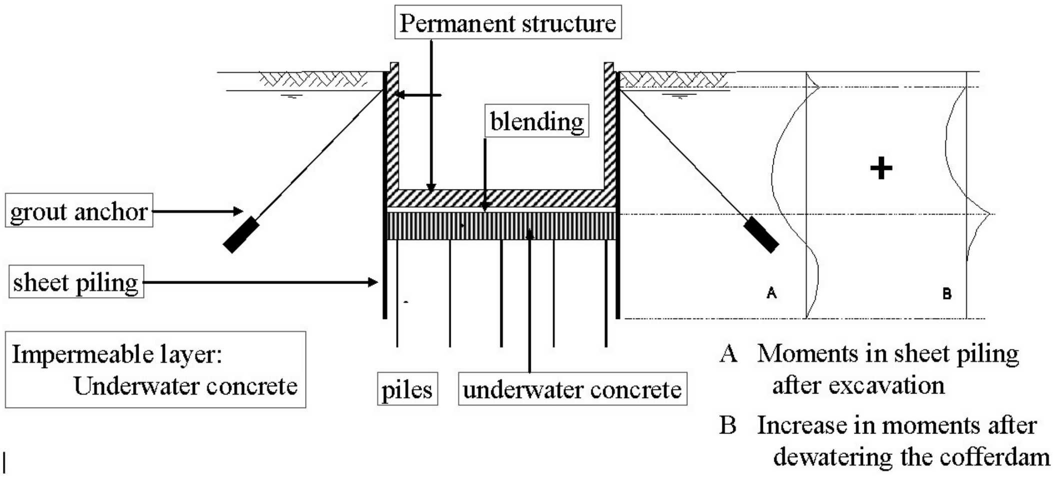


Figure 4-6 Cofferdam with underwater concrete slab

Sequence of construction

The order of work:

- Hammering down the piles of the sheet pile wall.
- If grout anchors are to be used: small excavation.
- Installation of grout anchors or struts.
- Excavation of the soil between the walls (the groundwater level in the cofferdam is maintained).
- Driving of permanent foundation piles from a frame above the water.
- Casting of the underwater concrete layer.
- Pumping the water out of the cofferdam.
- Construction of the permanent reinforced concrete structure.

The cofferdam consists of various structural sections that are described briefly below. See the "Manual – (Temporary) Structures" for the engineering calculations required to determine the dimensions of the cofferdam and it's elements.

Vertical wall (sheet pile wall):

The vertical wall usually consists of sheet pile walls but sometimes of combi-walls or diaphragm walls. During the excavation the sheet pile wall is loaded only by the horizontal grain stress. On the upper side it is supported by the struts (or grout anchors) and on the underside by the ground. After the pumping out the plane of moments that thus forms must be added to the plane caused by the water pressure (see the right side of Figure 4-6).

For the water pressure the sheet pile is supported by the strut and top of the underwater concrete (about 0.20 m under top of concrete). The degree to which a fixed support or pin support should be assumed depends on the passive earth pressure below this turning point, but 2/3 of the full fixed support value is a reasonable first assumption to calculate the dimensions for the sheetpile wall.

In addition to the tension piles sometimes the sheet pile walls considered to be sections to take up the tension forces caused by the upward water pressures against the underwater concrete blinding (the number of tension piles can then be reduced). For this purpose reinforcement is welded to the sheet pile walls, reinforcement that will be cast into the underwater concrete, to transfer the vertical forces from the underwater concrete (shear force in the reinforcement).

Anchors:

Grout anchors are usually inserted in a shallow excavation just above the groundwater level, anchor installation can be done in the dry. Because of the requirement to anchor outside the active soil wedge the anchor has a certain length and the end of the anchor with grout body generally is well below groundwater.

Struts:

In most cases struts are used. These are suspended loosely between the waling. Struts are cheaper than anchors and can be reused, but they may well form an impediment during construction.
Besides being used above the water level struts and anchors can be positioned at lower levels so that multiple supported sheet pile walls are created that demand a higher profile. The lower lying strut frames are then installed as the water in the cofferdam basin is lowered. This system can be attractive for deep cofferdams (like those of the Willemsspoor tunnel).

Piles:

Usually prestressed prefabricated concrete piles are used, which for better transmission of transverse forces have corrugations, say small ribs or ripples, over the distance to which they are embedded in the underwater concrete.
Furthermore vibro-combination piles are often used. These piles are made by drilling a hole, suspending a prefabricated reinforced concrete core in this hole and filling the space between the prefabricated core and the soil. The rough wall of the drill hole ensures that there is a good friction transfer (skin friction).

In the construction pits on and close to the Potsdamer Platz in Berlin small diameter auger grout piles were used because of the great length required in a type of soil which made pile driving extremely difficult. Auger grout piles are also often used in the Netherlands, especially to prevent nuisance deriving from pile driving.

It is not only necessary to dimension the piles for the tension forces during the construction phase, but also for the loads that work on the finished concrete structure during the user phase. With regard to this, it should be noted that the underwater concrete slab is certainly not watertight (as a result of imperfections and possibly cracks in the non reinforced slab) and it must be taken into account that directly under the blinding there is full upward water pressure. Therefore it is stipulated that the concrete must be removed (stripped) from the pile heads and that the pile head reinforcement is incorporated in the concrete blinding when it is poured. In this way a tension a connection that is resistant to tensile strain is formed between the pile and the main structure.

Horizontal sealing:

The easiest way to obtain a horizontal seal is to use an impermeable soil later (clay or peat). The leakage water is then remove by a temporary of permanent pumping out system. Examples of permanent structures with dewatering are the NS Station Rijswijk, the parking garage in Alkmaar and the access roads of the tunnel between Dordrecht and Zwijndrecht (16 m difference in water level!). If there is no impermeable soil layer, or permanent dewatering leads to problems an underwater concrete slab is one of the possible options.

Underwater concrete slab:

The thickness of the underwater concrete slab is often in the order of 1.0 m, to accommodate unevenness of the excavated bottom and to take into account uncertainties about the correct position of its upper surface. The entire casting process takes place under water, therefore less ideal conditions (vibration and finishing are impossible). After pumping dry the cofferdam the highest points on the underwater concrete surface are cut off and the entire floor is smoothed off with fill concrete before the permanent floor is made. The concrete is removed from the pile heads that project through the fill concrete, so the pile reinforcement is incorporated in the blinding. When there are high vertical forces tension anchors are used instead of tension piles.

Combining the temporary and permanent structure

In essence the soil and water retaining structure is constructed twice, once in the form of a cofferdam and then as the final structure. This is done because the cofferdam is usually not considered adequate to function as a permanent durable structure.

These factors also mean that the advantages (limited spatial requirement, no dewatering) must be set against considerably higher costs. Thus the extra costs for the construction within such a cofferdam against those of a construction pit (slopes, dewatering) for the open exit of a 2 or 3-lane road tunnel (28 m between the walls) is in the order of 25%. The method is also suitable for many types of structure: locks, closed tunnel profiles, pumping stations etc.
In some cases costs can be reduced by removing the sheet piles after completion of construction. This is not always possible, e.g. if the sheet pile walls have been used as tension elements against the upward water pressure under the concrete floor. To disconnect the welded-on steel transverse reinforcement is cost prohibitive.

Nowadays, increasingly efforts are made to include the temporary structure in the completed structure. This applies, for example, to concrete sheet pile walls and diaphragm walls. In addition, sometimes efforts are made to reinforce the underwater concrete slab which can then be incorporated in the final structure so that it is no necessary to make a blinding later. To date this has only been done for structures with relatively shallow foundations (low upwards water pressures). That this system has not yet been used for larger structures relates to doubts about its quality. It is difficult to construct a high quality structure under water. Think, for example of silt inclusions, uniformly good concrete covering of the reinforcement, etc.

Instead of a steel sheet pile wall it is possible to use a diaphragm wall or an augered grout pile wall. It is then no longer necessary to make a structural concrete wall. However it is necessary to construct a 'decorative' wall to please the eye of the end users because diaphragm walls and an auger pile walls are not smooth (only diaphragm walls with prefabricated panels are relatively smooth). The combination of a stiff diaphragm wall with weak grout anchors is not favourable: with increasing forces in the strut (excavate and pump water out of the cofferdam) the anchors will be relatively highly deformed and the diaphragm wall in the position of the partial clamping near the floor will undergo too great an angular deformation and break. A combination with stiffer struts is better. In the final phase the strut function must be taken over by a roof or floor (not possible with an open basin construction such as that in a lock, but it is possible with tubular profiles or for example a pump house).

Cofferdam alternatives - no tension piles

Unanchored underwater concrete slab

In the above it was assumed that the underwater concrete slab would be anchored by tension piles. If the use of tension piles could be avoided this would save (considerable) costs. It is possible to use an unanchored concrete slab, however, the slab must be heavy enough to prevent heave.

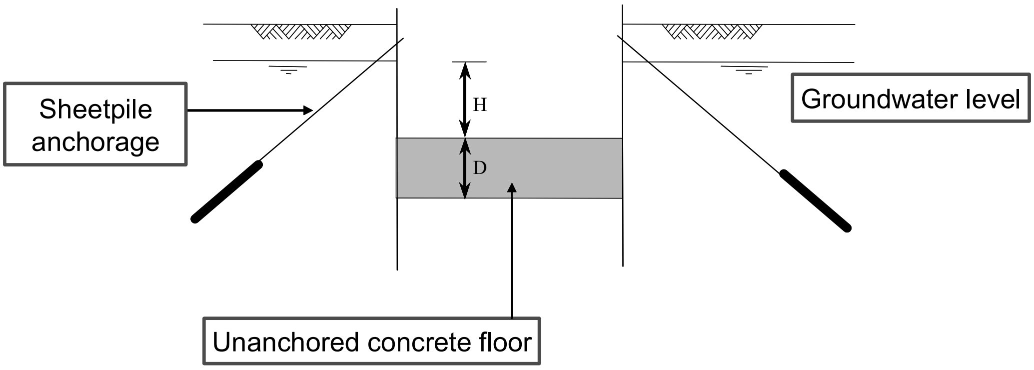


Figure 4-7 Cofferdam with unanchored concrete slab (gravity slab)

If the underside of the completed structure is projected to a distance *H* under the groundwater level, the thickness of the underwater concrete slab *D* can be determined from the vertical equilibrium as follows:

$$n\,\gamma_w\left(H+D\right)=\gamma_b\,D$$

in which:	n	=	safety coefficient	[-]
	γ_w	=	volumetric weight water (10)	[kN/m^3]
	γ_b	=	volumetric weight concrete (about 22)	[kN/m^3]

If a value of 1.1 is taken for *n,* from this follows a thickness of *D* = *H*.

An unanchored slab might well be more expensive than one anchored by tension piles, certainly for structures where the bottom of the permanent structure lies at a greater depth. The higher costs derive from the thicker underwater slab, the deeper excavation and in consequence of this, the longer and heavier sheet pile wall profiles. For these reasons an underwater concrete slab is almost always anchored, even for relatively shallow pits. Moreover it is necessary to ensure that the structure itself will not float up since it is not anchored by tension piles. A remedy for this could be to anchor the permanent structure to the (thick) underwater concrete slab.

Injection layer

The choice between the option with underwater concrete and that with an injection layer usually depends on the costs:

- the injection layer may be either less expensive or more expensive than the underwater concrete slab + piles, depending on the injection material used
- both the length and the profile of the sheet pile wall are less favourable when used with the injection layer than when the underwater concrete slab is chosen because an injection layer is assumed to result in less effective strut action.

Because of this in most cases it is more expensive to use an injection layer.

To counter the above-mentioned unfavourable factors the grout arch has been developed. This is based on a grout-injection layer, which is given an arch form and can thus exert a force in the strut, which permits a reduction in both the length and weight of the sheet pile wall. As yet projects with the grout arch can be considered less successful (Tramtunnel Den Haag) but this concept does deserve further consideration

4.3 Membrane construction

Another method to seal the permanent structure of for groundwater is the use of a membrane, e.g. for roads in cuttings and tunnel exits.

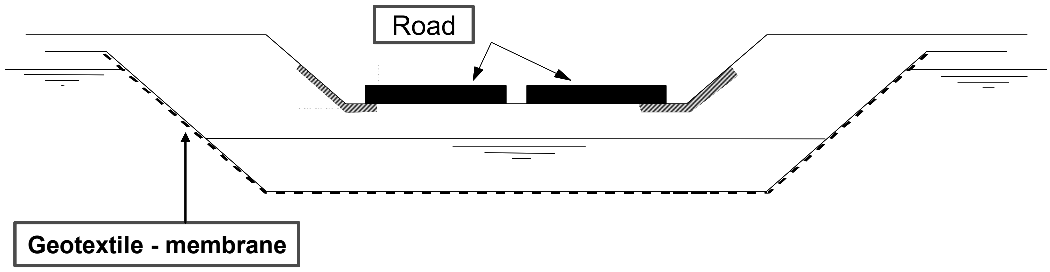


Figure 4-8 The use of a membrane for construction

In a cutting a PVC or polyester membrane (1.0 mm for example) is laid and then covered by a layer of sand on which the road is laid. A drainage system ensures that the water level above the membrane (precipitation and possibly leak water) remains a good 1 m beneath the road. The membrane can be laid in dry conditions, but if, with a view to the surroundings, dewatering is not permitted it can also be laid under water. The cutting is than dredged so that the water level in the cutting is maintained at the same as the ground level. After the membrane has sunk onto the bottom and the slopes the soil fill is introduced. After this the groundwater level above the membranes can be lowered. Here too the soil cover on the membranes must be heavy enough to resist the upward water pressure. (Safety 1.1 to 1.2).

4.4 Cut-and-cover method

This method was developed for construction of metro lines in towns. (Milan). With the cut-and-cover method diaphragm walls are made, working from a shallow trench. The diaphragm walls are covered by a reinforced concrete roof plate that is cast in situ.
After this the street level can be restored and taken into use. Under the roof excavation can continue until the required depth is reached, after which a concrete floor is cast.

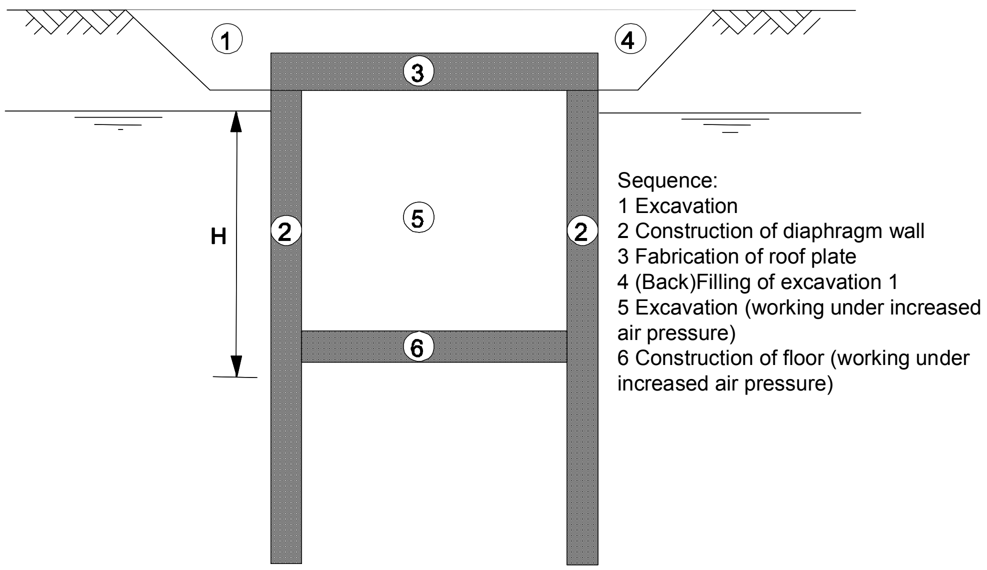


Figure 4-9 Cut-and-cover method

During the construction phase the groundwater level can be lowered in two ways:
1. Walls in a sealing soil layer (if present)
2. Using compressed air

If there is an impermeable soil layer the use of this is strongly advised, since in this case it is not necessary to use compressed air. This method was use for the closed parts of the Drecht tunnel that were built into the bank. If there is no impermeable soil layer it is possible to work under the roof plate by using compressed air ($p_{overpressure} = p_{groundwater} = \gamma_w H \approx 10H$, see Figure 4-9).

Personnel, materials and equipment must enter and leave the working area via an airlock. This, and strict regulations for working under overpressure, significantly reduce productivity; hence increase construction time and costs.

The advantages of the cut-and-cover method include not only the avoidance of dewatering and the minimal need for space, but also the very short time before the area above the tunnel can be returned to use.

4.5 Pneumatic caisson

Pneumatic caissons are also used to build structures without the need for dewatering and which require little space. This method is being used for the metro line in Amsterdam and its use is also under consideration for one of the variants of the storm flood barrier in the Nieuwe Waterweg, in this case as foundations for the hinges for the proposed sector in the banks. It has also been used for the small lock at Almere and it has been proposed for the lock bays that have not yet been built.

Even if there is sufficient room, a caisson may be chosen, for example if the bottom of the structure is so deep that dewatering or an underwater concrete slab with tension piles is no longer a real alternative. For the entrance shaft of the Westerschelde tunnel a 20 m high caisson of 20.000 ton was chosen.

A caisson is built at ground level. Excavating beneath it causes it to gradually subside until it reaches the desired depth. The excavation takes place from inside the work chamber. This work chamber is kept at an overpressure such that the groundwater cannot enter. The work chamber is accessible via an airlock that is not shown on the diagram. To limit soil friction during the sinking the side edges of the work chamber project slightly outside the profiles of the structure.

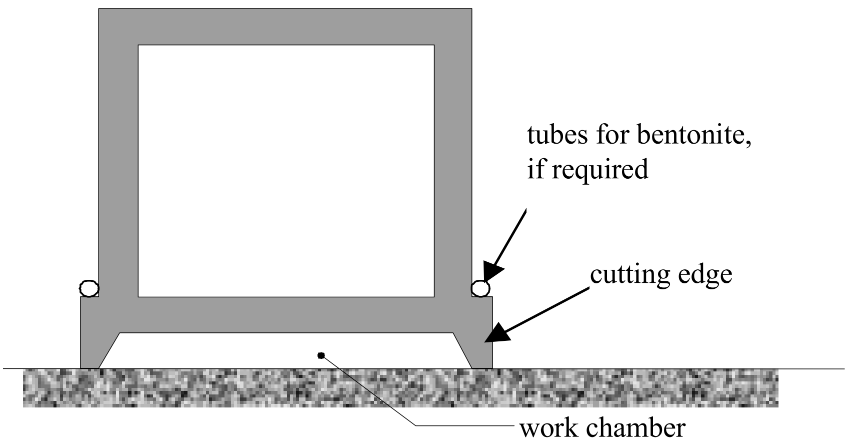


Figure 4-10 Pneumatic caisson

In order to reduce the friction still further a bentonite flushing system with perforated pipes is sometimes added. The weight of the caisson must be higher than the upward water pressure against the underside of the air bell in the work chamber. Once the desired depth has been reached the work chamber is filled with concrete to provide a good foundation.

Figure 4-11 Entrance shaft of the Westerschelde tunnel constructed as a caisson at ground level

4.6 Drilled tunnel

General

The great advantage of a drilled tunnel is that it does not take up any space above ground, except at the sites of the beginning and end shafts. Moreover, the groundwater level does not have to be lowered. However, it is necessary to take into account settlement of the ground near the tunnel.

Drilled tunnels are suitable for both urban tunnels and cross channel connections. Initially this method was considered less suitable for Dutch conditions. Since the successful completion of the Tweede Heinenoord tunnel (small traffic only), this opinion has been entirely changed.
At present (2001) many drilled tunnels are under construction (Botlek tunnel, Sophia tunnel, tunnel under the Pannerdens Canal, Westerschelde tunnel and the Groene Hart tunnel). Only the later has a single tube, the others all have two tubes.

It is remarkable that, with the exception of the Westerschelde tunnel, these are all railway tunnels; the round section fits reasonably well with the profile or free space required by this form of transport. This is much less the case for road traffic tunnels and thus an inefficient use of space.

In addition to the projects under construction that are mentioned above, preparations are underway for other projects, such as the North-South Line in Amsterdam and the OLS, the Underground Logistic System/Ondergronds Logistiek Systeem, provide a direct link between the flower market and Schiphol Airport.

In most cases immersed tunnels are a much cheaper option for river crossings for road traffic. Immersed tunnels are also more attractive for passing buildings (North-South line, Amsterdam) or large industrial cables and pipeline alleys (Botlek tunnel). With a drilled tunnel it is necessary to realise that the measures to prevent calamities during the work at great depths are extremely expensive and time consuming. Recent problems in the Channel tunnel, the tunnel under the Great Belt and the Second/Tweede Heinenoord tunnel are very instructive indeed!

During the construction of the Tweede Heinenoord tunnel much research was carried out into all aspects that relate to drilling. The results are reported in detail in “Monitoring bij de Tweede Heinenoord tunnel” a publication of COB (Centre for Underground Construction - Centrum Ondergronds Bouwen).

There are various drilling methods:
- Fluid shield (hydro shield or slurry shield) (BS)
- Earth Pressure Balance (EPB)
- Pipe jacking
- Other

Fluid shield

The principle of the fluid shield method is that there is a supporting fluid shield between the drill face and the tunnel-drilling machine. This supporting fluid consists of water with bentonite (clay). Within this an excavation wheel, which scrapes thin layers from the drill face. As a result, increasing amounts of soil mix with the supporting fluid, which must therefore be constantly cleaned. For this very long delivery and discharge pipelines running to the sand-bentonite separating installation are needed.
The fluid pressure *q* must be considerably higher that the groundwater pressure *p*: ($q > p + 30\ \mathrm{kPa}$) to ensure that the drilling face does not collapse (blow-in), because in that case the excavation wheel will become stuck and the ground above the tunnel will cave in. But the fluid pressure may not exceed the vertical ground/soil pressure ($q < \sigma_v$), because then the ground above the tunnel will blow out and the support pressure will be lost. In that case a balance/equilibrium will return unless, stubbornly the increase in the support pressure is continued.
During the excavation the machine pushes against the already constructed tunnel with the aid of jacks. When the jacks are far enough apart, a new tunnel ring can be built up from prefabricated tunnel sections that are transported to the required position by rail.

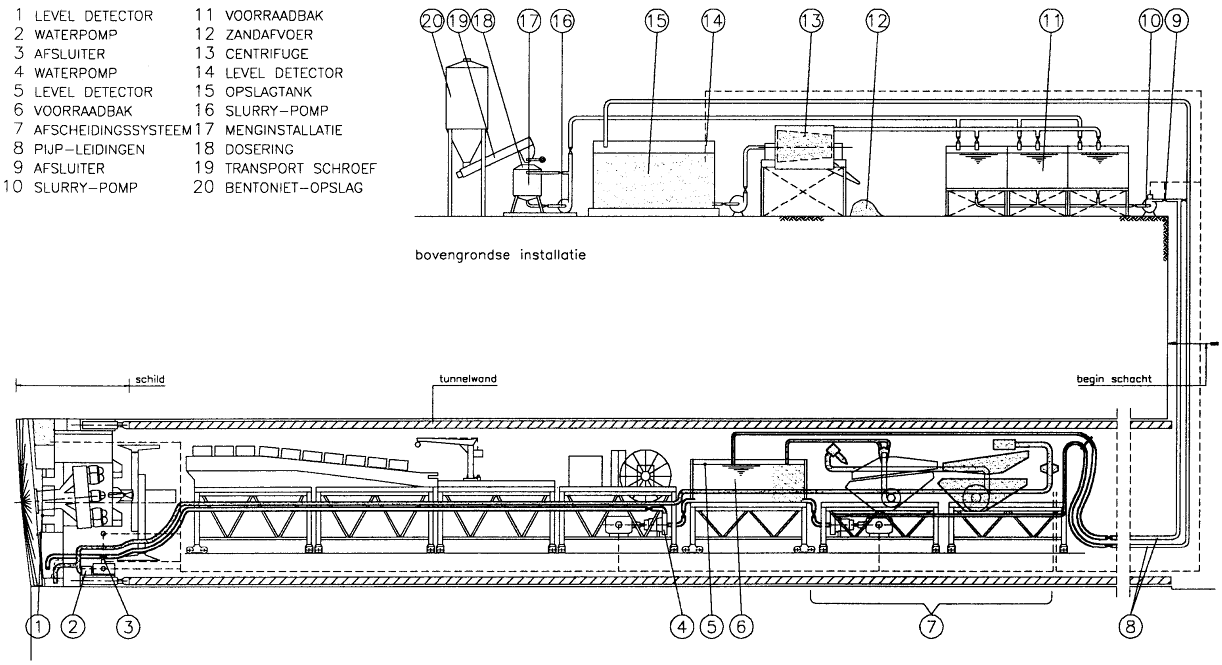


Figure 4-12 Fluid shield tunnel drilling machine

Earth pressure balance shield

The earth pressure balance soil shield method is very similar to the fluid shield method, only it is now not the drill face that is supported by fluid but the almost closed excavation wheel itself. The soil that is scraped off moves via holes in the excavation wheel into the excavation chamber.

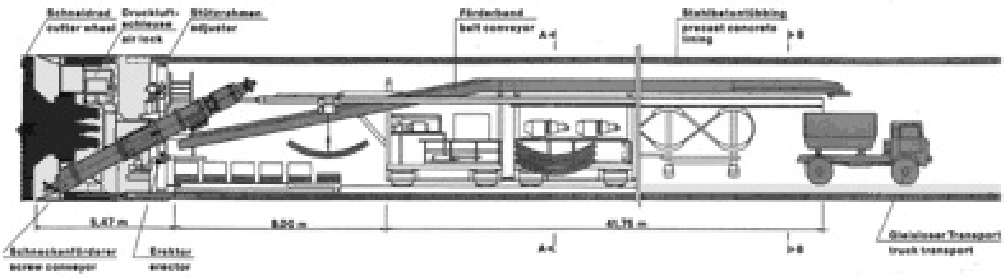


Figure 4-13 Earth shield tunnel drilling machine with diagonal auger

It is difficult to remove the soil out of the excavation chamber with a worm wheel (auger) without too much groundwater entering the tunnel-drilling machine via the worm wheel. For sand layers in particular, this is a problem for which foaming agents are often used in an attempt to find a solution. These foaming agents must be biodegradable, because otherwise the soil will becomes seriously contaminated.
The advantage of this method is that no sand-bentonite separating installation is required and that a conveyor belt can be used to transport the soil out of the tunnel instead of pipelines. The disadvantage is that there are more and greater fluctuations in the soil pressures. For this reason the tunnel drilling equipment must be heavier and thus more expensive. An earth pressure shield is volume controlled and a fluid shield is pressure controlled.

Pipe jacking

This is a method that bears some resemblance to the fluid shield method in which the entire underground work involves forcing the tunnel section forward. With this method the entire tunnel profile, in the form of rings (and not built up from segments), is forced forward. This method is often used for pipelines and cable ducts. No further consideration is given to it here.

Other methods

There are also many other drilling methods. Some are only suitable for drilling above the groundwater level, so that the drill face needs much less support or even no support. These methods are sometimes used for drilling below groundwater level. The groundwater water must then be kept out by over pressure in the tunnel drilling machine. In this case an airlock is necessary. Working in compressed air is dangerous and expensive (loss of time during entering and leaving). For this reason at present methods with a closed partition such as the fluid shield or soil pressure shield are employed.

5. Construction method: large-scale prefabrication

5.1 Prefabricated structures

In case of large-scale prefabrication, parts of the structure (or even the entire structure) are prefabricated elsewhere in controlled conditions (on a dry site), after which these parts are transported, often floated, to the construction site, installed and connected to each other or to other structures.
The prefabrication can, for example, take place in a construction dock, an existing ship's dry dock, on a construction or ship slipway or on a building site close to the bank of the watercourse. A construction dock is a temporary construction pit on or close to the course where the sections of the structure are made, after which they are floated to the construction site. A construction dock is financially attractive when little area is available at the construction site or if lowering the water table causes damage to the surroundings.

For floating transport use is made of buoyancy of the sections. When the floating capacity of the sections is not sufficient extra buoyancy can be obtained by using pontoons or barges. Furthermore it is possible to transport the sections on submersible or non-submersible steel barges or transport vessels.

Installation, for example the positioning of self floating sections on the bottom, can be carried out by the addition of ballast. Often water, which can be quickly admitted, is used for this purpose. This shortens the time needed and reduces hindrance, for example to shipping traffic. The temporary ballast is later replaced by concrete or gravel. If extra buoyancy has been obtained during the transport operation by using barges or pontoons, consideration can be given to winching the section (that is not entirely self buoyant) into position from the pontoons.

Connecting the sections to other structures under the water may involve parts that have previously been built into the bank, thus not prefabricated (for example the landward parts and open exits of tunnels). The most famous example of large-scale prefabrication is the immersed tunnel.

The immersed tunnel is a structure that entirely disappears under water. There are also structures of which a part remains above water, such as the piers of the storm flood defence in the Oosterschelde and the piers of the Willemsbridge in Rotterdam.

Large scale prefabrication is often used, for example for:
- immersed tunnels
- quay walls built up from caissons.
- permeable caissons and closed caissons to close off tidal waters
- caissons for the construction of harbour moles (including the harbours in Normandy for the invasion in 1944).
- piles, storm flood defences and bridges.
- other parts of storm flood defences, such as the sills of the storm flood barrier in the Thames and the sill basins of the Oosterschelde.
- structures for offshore oil and gas exploration and extraction.
- bridges (Zeeland bridge, Bahrain, Great Belt)

In addition large-scale prefabrication is possible for:
- tunnels and underwater bridges.
- turbine-houses and outlet sluices for tidal power stations.
- the extension of existing locks.
- dams.
- etc.

Immersed tunnel

A typical Dutch method of tunnel construction is based on immersion of tunnel sections. The tunnel sections are prefabricated in a construction dock, often a construction pit. Each section is a long section of the tunnel (for example 100 to 250 m long), both ends of which are closed by temporary watertight bulkheads when is finished. These sections form large hollow boxes, which are dimensioned in such a way that after dewatering process has been stopped and the construction dock has been filled with water they can be floated to the intended site, preferably with a small freeboard, so that later little ballast is required.
Before they can be moved the closure dam must be removed by dredging, or if the dock is equipped with one, the dock gate can be opened.

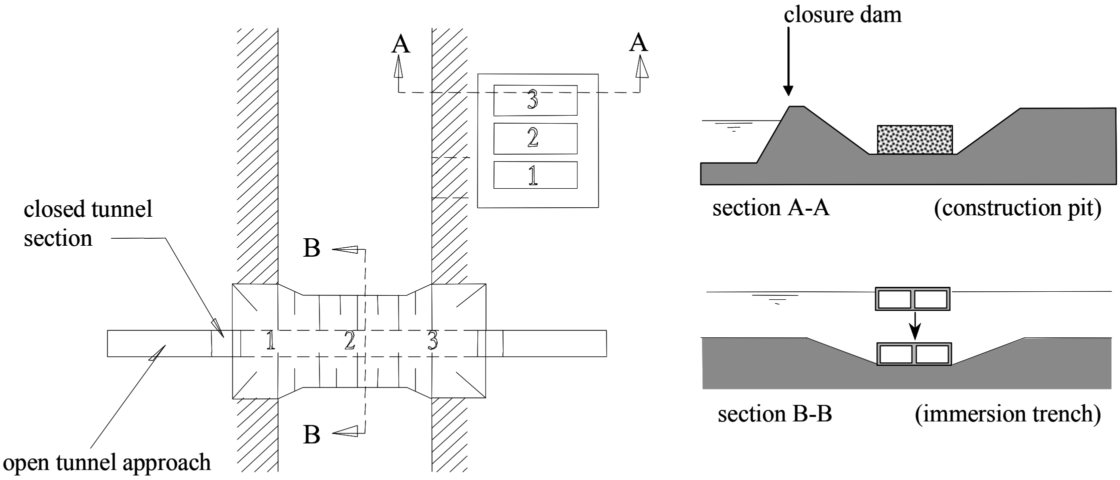


Figure 5-1 Construction site for an immersed tunnel

At the same time as the floating sections, the land sections (open entrance plus connected closed tunnel) are built on each bank on dry sites. Depending on the situation, this can be done in construction pits or within cofferdams. This construction phase is not shown in the above figure, although the next phase, in which the trench across the waterway between the landward ends has been dredged, is shown.
Tugs transport sections successively from the construction dock to the required position. Here the sections are ballasted (water in tanks which are inside the section), as a result of which they sink to the bottom. They are carefully positioned on their foundation and connected to the closed part of the tunnel that extends from the land or to the previous section. The immersion must be controlled and accurate. The sections are held in place in the horizontal plane by a number of cables that are equipped with winches and extend in different directions. After being ballasted (so that its weight is greater than its buoyancy capacity) the section sinks vertically, suspended on four vertical cables which are controlled by winches on the pontoons that remain floating.

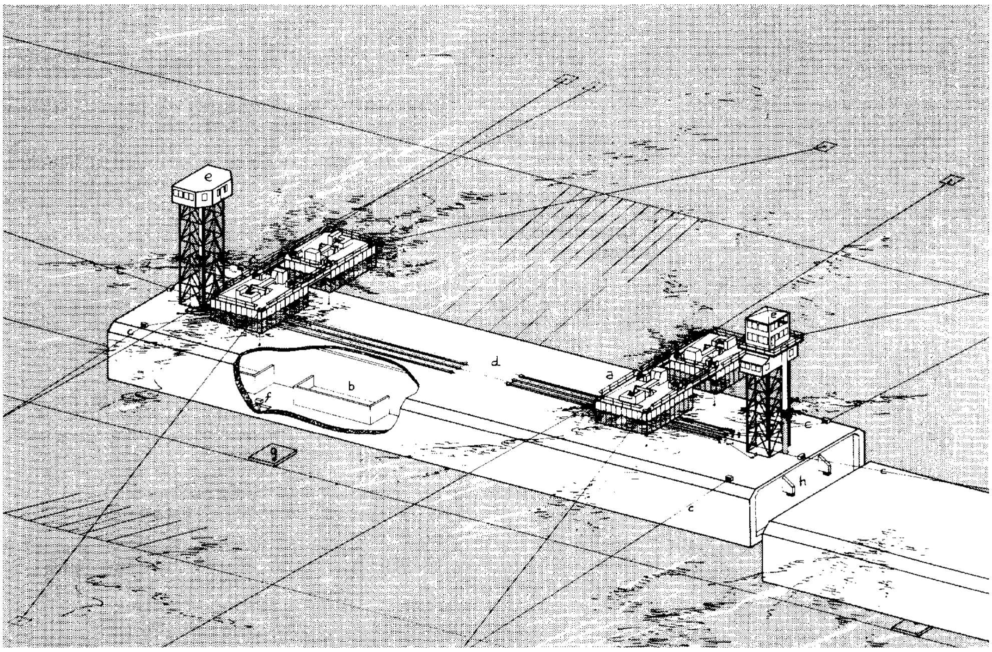


Figure 5-2 Sinking with the aid of pontoons and cables

After the sections have been positioned the temporary ballast (the water in the tanks) is replaced by the permanent ballast: for example a layer of un-reinforced concrete laid on the floor of the tunnel tube. The road surface or rails are laid on the ballast concrete. It is also possible to lay the rails and sleepers on a hardcore ballast bed such as that used for the rail bed outside the tunnel.

The bulkheads are removed, while the trench is in filled until the watercourse is restored to its original depth. This can be done, for example by the deposition of the dredged material that was earlier removed from the trench and stored in a temporary depot above or below water level.

In Figure 5-1 the dock shown is adjacent to the tunnel location. The cost of the floating transport is reduced by this, but often there is no room available for the dock or the dewatering of the construction dock may cause too much damage. In such cases the dock can be planned at a greater distance from the site of the tunnel. However it is necessary for the access route to be deep enough for the tunnel sections, with their preferably small freeboard (and thus deep draught). The distances between the piers or any bridges and the width of locks must also be sufficient to permit the passage of the sections. When the sailing depth is limited other options may be possible. One of these is to place the roof of the tunnel only after the tunnel has passed a particular obstacle as was done for the A15 tunnel under the Noord.

Extra requirements

For large-scale prefabrication in particular, the method of implementation exerts a very big influence on the design. Large-scale prefabrication imposes many extra requirements:

- transport
  - adequate buoyancy
  - stability during the transport
  - means of connecting to other sections
- foundation
  - connection to spread foundations or to piles on prefabricated construction
  - must take up horizontal forces and tension forces
- construction dock
  - availability of space for a construction dock
  - the necessary time for the procedures to obtain the permits to build a construction dock

- availability of docks that have been used for previous projects (for example the construction dock at Barendrecht has been used for the prefabrication of seven different tunnel sections) or of ship dry docks.
- Water depth of the sailing route between the dock and the construction site. (As earlier stated there are possible variants to compensate for a limited depth. Sections cannot be entirely completed, buoyancy aid is required from pontoons, steel barges or by other means).
- costs of the transport (floating transport, sinking and positioning), foundation and construction dock

5.2 Transport

The transport can be divided into two types:
- Partially self propelled transport
- Transport by a floating crane vessel

Self-propelled floating transport

Weight and buoyancy capacity

Already a distinction has been drawn between floating and non floating sections as well as between sections which, after ballasting, will be entirely under water and those which, when completed, will extend partly above water. Below the relation between the weight and the internal hollow area will be determined for a tunnel element that will be floated in first and then immersed entirely into final position. The immersed stage is shown in Figure 3-3. The tunnel is at the bottom of the trench and is founded on injected fill sand.

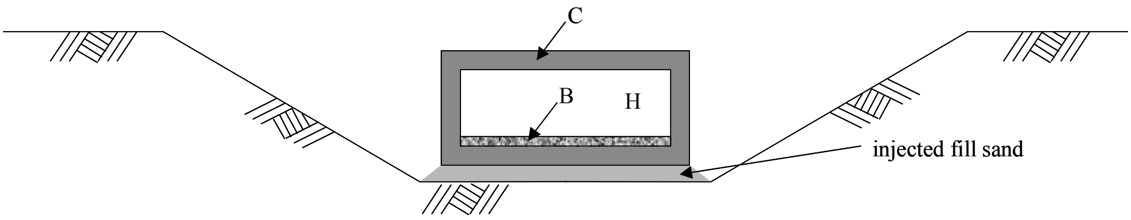


Figure 5-3 Immersed tunnel

The starting point for the design is the necessary profile of free space that has a surface area of H m^2. Around this is the reinforced concrete structure (walls, floor a roof; with bigger tunnels and also often, the partitioning walls, not shown here) with a surface area of C m^2. The section is floated to the site and then immersed with the aid of water that is admitted into the ballast tanks inside the tunnel section. After the tunnel has been founded on the layer of fill sand the layer the temporary ballast is replaced by the permanent ballast: a layer of non reinforced concrete with an area of B m^2.
If the weight of the temporary watertight partitions on the ends of the tunnel section as well as that of the further immersion equipment (including that of the ballast tanks) is discounted, the following equation per running meter of tunnel can be derived:

<u>During the floating transport:</u>
When an immersed tunnel section is transported via an inland waterway the designer will usually strive for a small freeboard, say 1% of the external height of the tunnel, so that later as little ballast as possible will be required. This means that the weight corresponds to 99% of the maximum buoyancy. If the volumetric weight of reinforced concrete is 24 kN/m^3 and that of water 10, then:

$$24C = 0,99 \cdot 10 \cdot (B + C + H)$$

On the bottom (before the filling of the trench):
The weight must now be greater than the buoyancy, say 8% more. If the volumetric weight of un-reinforced concrete is 23 kN/m^3, then:

$$24C + 23B = 1{,}08 \cdot 10 \cdot (B + C + H)$$

Solving these two equations gives:

C = 0.714xH m^2
B = 0.0752xH m^2

In other words: from the necessary hollow or free area, the chosen construction method and the limits set by these (freeboard, over weight), follow the amounts of construction and ballast concrete required. In any case the equations are only indicative; actually it is necessary to take into consideration the weight of the immersion equipment, the distribution by volumetric weight of concrete and of water (fresh, salt, brackish), dimensional stability of the concrete structure, etc.

In the following round of the design cycle an investigation is carried out to determine whether the area of the construction concrete is sufficient to take up the loads. This is determined by the end situation when the trench is filled and the normative high water level is taken into consideration. Often it will be necessary to have a different distribution of the construction concrete from that shown in the cross-section in Figure 5-3, by for example adding bevelling to the roof and floor close to the walls in order to improve the absorption of the transverse forces and moments. This additional material close to the corners must be removed from elsewhere because the concrete area in the cross section may not be bigger. After all too much material would also make transport by floating impossible.

In the Dutch situation (depth of the fairways, width of the locks) it appears that, as a rule of thumb, with the above calculated area of construction concrete it is possible to make traffic tunnels of reinforced concrete that can easily take up the loads (earth and water pressures). In these cases only low quality concrete with little reinforcement is required.
In fact there are exceptions. With very deep water and/or wide traffic tunnels the transverse forces are too high, especially on the roof and floor.

In such cases the following solutions can be considered:
- partly or entirely prestress the transverse section .
- use light concrete rather than concrete with a gravel aggregate.
- make the space (H) bigger than that deriving from the functional requirements (the required profile of free space).
- do not make the tunnel free floating (for example use barges with extra buoyancy capacity).
- combinations of the above.

Immersed tunnel versus other prefabricated structures

Tunnels differ from many other large prefabricated structures (such as or piers) because:
- the immersed tunnel disappears entirely under water and thus the displacement of water does not increase after the roof passes below the water line. The controlled immersion takes place with the aid of vertical cables which are let down by winches. The winches are mounted on pontoons that remain afloat. For structures which extend partially above the waterline when finished increasing amounts of ballast must be added continuously during the immersion stage.
- immediately after positioning on the bottom and in the final stage the immersed tunnel bears few or no horizontal loads. Thus the overweight in relation to the buoyancy capacity may be low. For this only 8% (for that matter an arbitrary value) is taken, principally with a view to uncertainties (deviation in

volumetric weight) and for example the time lag in the recovery of the groundwater level under the section in relation to the surface water level. Thus during the falling water levels in tidal rivers it is conceivable that the groundwater in the in-fill sand has a somewhat higher hydraulic pressure head than the water on the roof of the tunnel.

In many structure large horizontal loads can occur. Thus after the placing of the piers of the storm flood defence in the Oosterschelde it was necessary to take wave loads into account. This was also the case for the caisson piers of the Great Belt bridge and for offshore platforms. For these types of structure it is necessary ensure that the structure possesses the greatest possible stability against horizontal loads as soon as possible after it has been positioned and before the entire structure has been finished. This stability requirement should be based on waves with a larger frequency occurrence than those for which the finished structure is dimensioned.

For this type of prefabricated structure a much higher horizontal load must be taken into account. For immersed tunnels the overall principal dimensions are largely determined from the vertical stability during each successive construction phase (after which the monitoring of strength takes place). For a quay wall caisson (gravity structure) on the other hand, the overall dimensions are largely determined by the stability criteria:

- Sliding ($H_{\max} \leq V \tan\delta$)
- Resultant of *H* and *V* within the core of the basal area of the caisson.
- Foundation (grain) pressure under the basal area of the caisson not higher that the permissible value.

It should be observed that for the last two criteria this concerns a quick preliminary approach: greater insight can be obtained by study of the sliding planes.

The next step in the design cycle is to investigate whether the caisson designed for the end stage can indeed be floated to the site; for this the stability during the floating transport must be checked.
It is also necessary to investigate whether the water depth of the transport route is sufficient. If one or other of these is not adequate the main dimensions for the competed are adjusted or other measures are taken (for example stability pontoons).

Next the thickness of the concrete (floor, walls, etc.) is checked. In addition it may occur that the load is much higher immediately after the installation than at the end state. This happens for example when little ballast is needed in the inside the caisson before it is positioned on the bottom. A situation then arises in which the floor and the underside of the walls is heavily loaded by the water pressure on the outside and there is very little supporting pressure from the thin layer of ballast in the inside. The ballast for the positioning (submersion) of the caisson may consist of water (that can be admitted quickly as a result of which the sinking process is short) or of sand or gravel.

In the final stage the caisson is entirely filled by sand or gravel and water and perhaps partially by concrete if the stability demands this. If, from the strength calculation, it appears that the floor and/or walls are too thin modification is necessary (increased thickness; extra partition walls that reduce the excess pressure of the floor and outer walls) and the design cycle must be entirely or partly revised.
In this design cycle for a quay wall caisson (successively check whether the stability in the end phase, the floating transport and the concrete thickness) no attention has been paid to the previously mentioned stability with regard to wave and current pressures immediately after positioning.
For quay walls this is usually not necessary (apart from those that are directly built on the coast), but for other structures such as piers for storm flood defences and offshore platforms this is an essential part of the design and/or execution cycle.

Transport by floating crane

The possibilities of using floating cane equipment have undergone a spectacular development. In recent years lifting capacities up to 100.000 kN were used during the construction of the fixed links in Denmark (Great Belt and Öresund) and Canada. By using these lifting capacities the prefabrication can take place on relatively simple building sites.
This development is a continuation of what was accomplished for the Zeeland Bridge (sixties) and the Bahrain Causeway (eighties), with maximum weights of 6.000 and 15.000 kN respectively.

In such a way transport is possible over great distances. The parts of the bridge over the Great Belt were made in Portugal because it is easier to obtain the sand and gravel needed for the concrete there than in Denmark. Moreover labour costs were lower in Portugal. In any case it was necessary to build a transport vessel because in situ construction of the bridge piers was forbidden by the client.

Figure 5-4 and Figure 5-5 show the floating crane and its arrival in Sweden with a complete bridge span.

Figure 5-4 Floating crane Svanen (Ballast Nedam) with bridge girder for the Westbridge over the Great Belt, Denmark

Figure 5-5 Svanen at the prefab-yard, Nyborg, Denmark

Figure 5-6 Floating crane Ostrea transporting a pier of the Oosterschelde barrier

5.3 Foundation

Difference between in-situ and prefab construction with regard to the a foundation

When construction in situ two types of foundation are possible:
- On a spread foundation (shallow foundation, directly on the subsoil)
- On piles

With a spread foundation first a concrete blinding is laid on the bottom, and on this first the bottom slab is cast, followed by the walls etc. The underside is thus a contra-mould of the bottom, so that a good transfer of forces is ensured. With a pile foundation, first the piles are driven and then the parts of the piles that project above the blinding (often only the reinforcement after removal of the pile heads) are cast into the concrete floor. Here too there is a good transfer of forces.

Compared to the in-situ and in the dry constructed structure the foundation of prefab structures is slightly different. For a spread foundation for large prefabricated sections, the relatively smooth underside is placed on a less smooth bottom. This may be in the natural subsurface, a dredged surface (e.g. the bottom of a tunnel trench) or a built-up rip-rap bottom protection or stone bed (e.g. for caissons for breakwaters which are placed on a rubble bed). None of these are of the same order of accuracy (smoothness) as the bottom surface of the prefabricated structure.
The use of pile foundations for prefabricated elements requires special solutions for the (underwater) connection between the piles and the structure.

Many of the requirements set for in situ foundations are also applicable for the foundation of prefab sections. The requirements are as follows:
- a good transfer of forces to the subsoil, thus sufficient contact area between the bed and bottom of structure
- the slope of the bed where the prefab section has to be positioned should suffice to what is required, see Figure 5-7
- settlements may not lead to unacceptable deformation of the finished structure unless adjustment options are available.

Example: Prefab Caisson

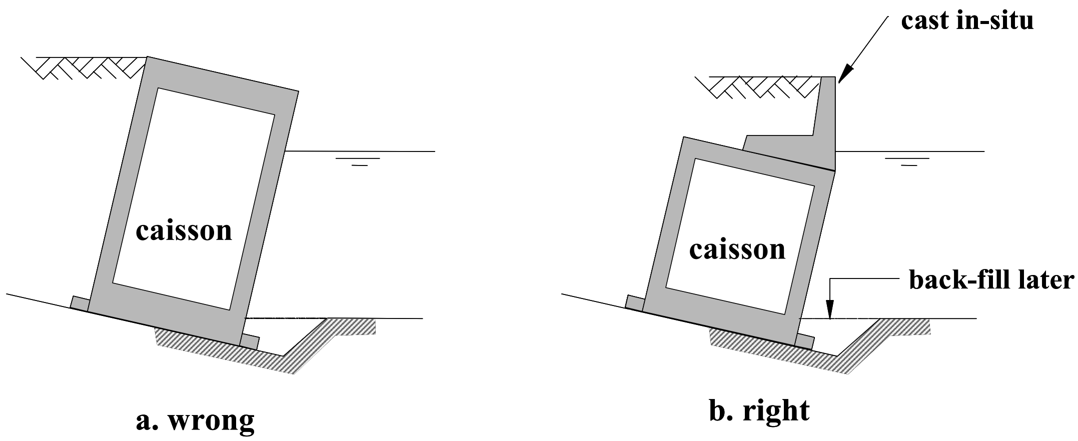


Figure 5-7 construction error in the positioning of a quay wall-caisson (exaggerated)

It is not realistic to assume that underwater slopes are always in the shape they should be, generally these slopes are constructed with a certain construction tolerance. Figure 5-7A, where things are exaggerated, shows the consequences of a misshaped underwater slope, in this case the slope is too steep. Unfortunately as a result the deviation in slope angle may translate into a larger deviation in the required position of (parts of) the final structure, here a quay wall constructed from caissons. Due to the steeper

slope of the bottom the quay apron (top surface of the caisson) is misaligned and the quay front is not vertical but has an angle to the vertical axis. Corrective measures will be necessary evening out the apron and providing a vertical berth face for safe mooring of ships. Figure 5-7B shows an option that is often used to correct the wrong slope or angle of a caisson. After positioning less high caissons a reinforced concrete L-wall is cast above the waterline; naturally this must be well anchored to the caissons. The front face of the L-wall is vertical, so that ships can moor alongside. The error in the slope of the bed can be counteracted in this way.
Even in case of a bottom bed of the right slope, large caissons cannot always be accurately positioned in the horizontal plane, e.g. due to unfavourable weather conditions during the sinking procedure. Their front faces could end up not being in the same plane, unless preventative measures were taken such as the use of "shear keys". The resulting protruding angles could result in damage to berthing ships. Here as well, the quay face can be made smooth by constructing an L-wall on top of the caissons after positioning.

Note:
- Caissons placed directly on the bottom have the disadvantage that erosion, for example caused by ships screws, leads to undermining of the caisson. The bottom level of the caisson could be chosen as deep as the bottom of the anticipated scour hole, which depends on the erosion load and the bottom material. Alternatively bottom protection material can be used.
- Figure 5-7B shows that the bottom slab of the caisson extends both at the front and back of the caisson. The extension on the front is intended to increase the foundation area and by means of this the resultant of the vertical loads is kept within the core of the structure’s cross section. The extension on the back is used to mobilize the vertical weight of the fill at the back, besides the negative active soil pressure, in order to satisfy the stability criteria (ΣH, ΣM, ΣV).

Issues regarding shallow foundations of prefab structures and alternatives

Issues to be taken into account and alternatives to be considered include:
- The bottom must be as smooth as possible and the results of any deviations which occur in spite of this must be compensated (corrected) or accepted.
- The area of the foundation must be made as small as possible.
- Levelling (with grout injection).
- The structure must be placed on a limited number of fixed points or ribs.
- The structure must be placed on temporary jacks

Important factors to be considered are the loads that must be transferred, the required degree of accuracy, whether or not correction is possible and of course the total cost. It is possible to design and construct pile foundations for prefab structures, but this option is not considered here.

Make bottom as smooth as possible

It is very difficult to obtain a smooth finish to the bottom, especially when large areas are involved. In some cases an attempt is made to do this by applying a layer of suitable material and smoothing it off. For the quay wall-caissons, commonly, a layer of gravel or rubble/riprap is fed in by a pipe and thus discharged onto the bottom in a controlled manner. It is then smoothed off by a levelling beam that is pulled over guide beams. The guide beams are part of a frame that is placed on the bottom, the upper side of which projects above water. The guide beams are kept as close as possible to the horizontal and at a set height. The discharge pipe is also incorporated in the frame. Because the frame has only very limited dimensions it must be frequently moved by the floating cranes in order to provide the following parts of the site with a sand or gravel bed. The surface will never become entirely smooth. See caisson example in the above. For the transfer of forces to the subsoil it must be assumed that this will not optimal. In some places there will be no sand or gravel on the bottom surface of the caisson, while in other places the gravel that is too high will exert more force (although the forces pressing down on the gravel ‘peaks’ will result is some levelling). The floor of the caisson must be dimensioned to accommodate these locally higher pressures, although it is not known in advance where these will occur. In other words compared to that of a structure built in situ, the floor of the prefab caisson-quay may have to be thicker over the entire surface.

Make the foundation area as small as possible

This method is used, for example, for American immersed tunnels.

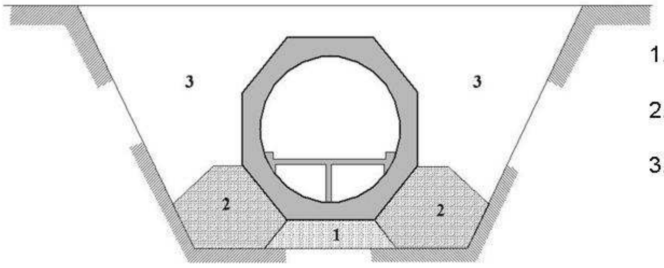


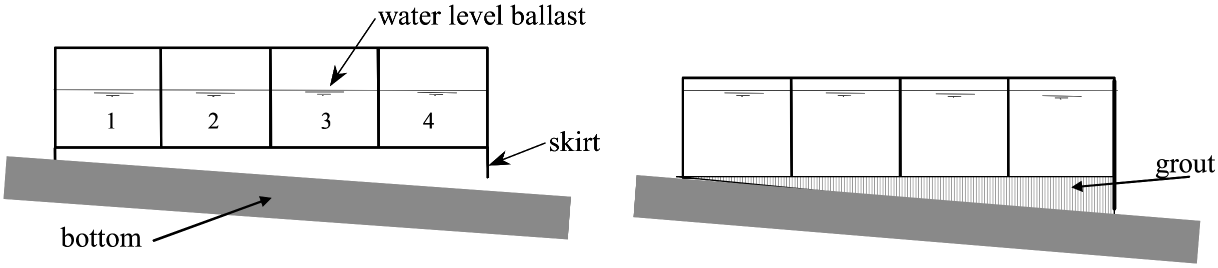
Figure 5-8 American immersed tunnel

In Figure 5-8 the slopes of the trench are too steep (at least for Dutch soil conditions). The tunnel sections consist of steel plates and concrete and are prefabricated on a slipway. Much of the concrete is poured during the floating stage. Before immersion of the sections a gravel bed is placed on the bottom of the trench (no. 1). The top of this is flattened by means of a levelling beam that is pulled forward over a frame that is placed on the bottom. Relatively high accuracy is required, not only for the transfer of forces, but also for the connection with the previous section. After installation the trench is further back-filled with gravel (volumes no. 2) using a fall-pipe from the surface of the water. By cutting off the corners of the hexagonal bed at an angle of 45^0, the gravel connects reasonably well against the sloping surfaces. After this the remaining trench volume (no. 3) will be back-filled. For this volume it is possible to use materials of lower quality, for example the material that has been dredged from the trench. A bottom protection may have to be constructed at the top. In America this is sound tradition to protect the tunnel against falling and dragging anchors, in the Netherlands the concrete tunnel roof is usually constructed thicker and avoid the need for a bottom protection.
Bed 1 forms the foundation, 2 the side connection (for example in case the dumping of the material for bed 3 occurs in large unequal amounts – thus with big internal differences in height on each side of the section - and is less expensive to carry out), layer 3 the fill of the tunnel trench.

A small foundation area is the result of a deliberate choice and thus results in a small area for gravel bed no. 1. The smaller this area is, the lower the cost of a relatively accurate finishing process. It should be noted that a tunnel needs to transfer only small vertical forces and scarcely any horizontal forces, apart from those that result from the previously mentioned uneven filling.
For structures that must transfer bigger forces (such as quay walls) is this not a reasonable solution, but the opposite: the width of the foundation area (and therefore of the structure) is determined by the stability in the end state (no slip, resultant within core) and not by the mode of execution.

Levelling
This method is primarily used for offshore structures. The levelling process as shown in Figure 5-9 is simplified and drawn on an exaggerated scale. During levelling the structure is surrounded by sheet pile walls (skirts) that are driven vertically into the bottom, after which the space within the skirts is filled with grout.
Left in the picture the skirt of the section touches one point on the bottom the section. Here it is assumed that the bottom slopes evenly in one direction, but actually the bottom may have a much more uneven

profile. If ballasting with water continues to be equally distributed in all the compartments the section will then tilt. The angular displacement is measured on the steel deck that projects above the waterline. Admitting more water into the compartment furthest left (no. 1) presses the section into the bottom in that position.



a) Element touches bottom
b) Element in final position (grouted underneath)

Figure 5-9 The principle of levelling

As more ballast is added (also in the other compartments), finally the situation shown in the right hand figure (see b) is reached in which the skirts are forced to penetrate into the ground along the entire circumference of the box . The entire process of horizontal submergence is achieved by measuring and ballasting in these compartments, where this is necessary for the levelling (horizontal position) and for the penetration of the skirt into the bottom. A great deal depends on the inaccurately known position of the bottom. The length of the skirts is calculated from the expected irregularities in the position of the bottom plus a certain reserve.
After sufficient penetration of the skirts the space under the bottom of the area that is within the skirts is filled with grout via pipelines that are cast in place. There must also be pipelines to expel water during the injection of the grout. This space cannot be filled in a single operation so it is sub divided into smaller compartments.

The skirts serve the following functions:
- making levelling possible.
- as formwork during the under-grouting .
- protecting the concrete bottom plate from locally excessive pressures. If the section is placed directly on the bottom without a skirt, irregularities in the bottom could exert very high pressures against the bottom plate at the moment when the section is fully loaded to ensure its stability against horizontal wave loads.
- to obtain some protection to prevent undermining by bed erosion by tidal currents or other effects. Investigations must to carried out to ascertain whether it is necessary to add a bottom protection.
- To transfer the sliding plane of the underside of the section to the underside of the skirts. In this way irregularities in the surface, possibly with lower ϕ-values are avoided. Moreover extra weight is provided for stability under wave and current loads (no sliding, resultant within the core of the structure's cross section) by the grout and soil mass between the skirts (weight, minus the water displacement, multiplied by the volumetric weight of water).

Example: oil extraction platform

A good example of levelling is provided by the Dunlin A (Andoc) oil extraction platform, in which the principle - somewhat distorted, is shown in Figure 5-10.
This platform, a gravity structure, is located on the Norwegian continental shelf, in water that 150 m deep. The height of the design wave is 30 m. Where wave attack is the highest (close to the waterline) the four columns have been made as thin as possible. The underside of the deck on which the installations and accommodation is located, is well above the wave crests (1.5 times the wave amplitude), in order to avoid high upward wave impacts. Pipelines leading to and from the seabed run through the columns. The box structure (105x105x32

m^3) transfers the loads to the seabed and provides sufficient weight. The box is compartmented by partition walls. In the compartments there is ballast material and oil (oil storage). The box is positioned at great depth, so despite the large area the wave loads are small.
Parts A and B in the figure are concrete, C is a steel structure, while the skirts required for the leveling are steel sheet pile screens.

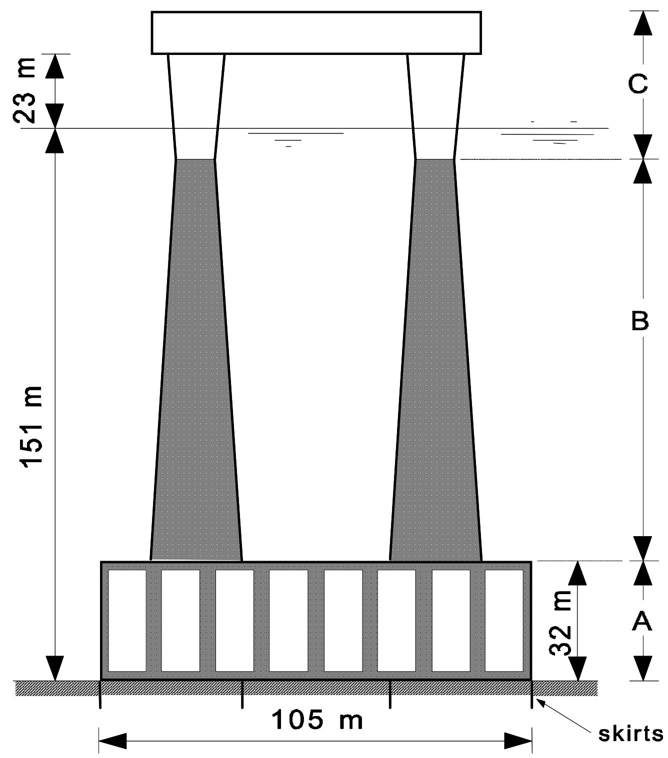


Figure 5-10 The Andoc platform

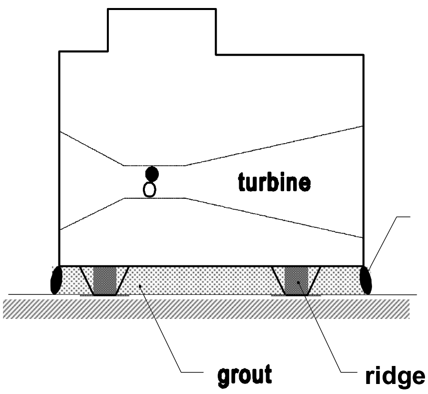
The construction sequence was as follows:
1. The skirts and part A, apart for some of the partitions and the roof of A, were fabricated in Europoort in a construction pit with a dewatering facility. So that the construction dock would not need to be so expensive Part A was not entirely finished.
2. After the filling of the dock and dredging of the access channel through the dam between the dock and open water, the section was towed out. Next part A was completed while it was floating in the Europoort area and sliding formwork was used to construct the four columns of part B.
3. The section was then towed to a deep fjord in Norway. During this transport the upper section of part A was well above the water level. The large area of the waterline area induced a high inertial moment and by thus sufficient metacentre height (buoyancy stability).
4. In the fjord, where the water was deep enough, the section was ballasted until the four columns projected only a short distance above the water. The assembling of part C began with the placing of the four steel parts of the columns with the aid of floating cranes. After this, the section was sunk further (the steel columns projected just above the waterline) and, with the aid of pontoons, the steel deck of C was towed above the columns and mounted.
5. After the water ballast had been pumped out the section came to lie a littlie higher (waterline some distance below the plane separating parts B and C) and the platform was towed to its working location. The waterline area during this floating transport was considerably lower that that mentioned under point 3: only 4 circles, although some distance from each other, in place of the 105x105 m area of the box. An important reason why the metacentre height was still sufficient was that heavy ballast (including iron ore) was put on the floor of the box structure, so that the centre of gravity G came to lie relatively low. This permanent ballast was installed while still in the fjord.
6. Once above the working location the platform was held in position by tugs and lowered by adding water as ballast. Next it was correctly position vertically by following the leveling method and a grout foundation was injected.

Finally a few observations on this method:

- The last phase of the approach to the seabed must be very slow. Owing to the large bottom area (at leas of the Andoc-platform) much water must escape sideways through an increasingly narrow gap. If velocity during the sinking of the section is too high the water velocities will be so high that undesirable erosion will occur.
- If there is a chance leaning of the section or with a bottom that is not horizontal, the water pressures that are created under the section can cause a sudden horizontal movement of the section. This movement can sometimes be controlled by the tugs.
- To prevent the occurrence of this mishap the Andoc-platform had a steel tube on each corner. As the platform approached the bottom these four tubes projected 2.20 m below the skirts. They made the first contact with the seabed and as ballasting proceeded penetrated, thus preventing undesirable horizontal movement. During the transport from Europoort the tubes were drawn up to restrict the draught and when lowered they were fixed in position by explosion bolts, so that they maintained their position in relation to the section during the penetration of the seabed.

Positioning on a limited number of supports

The smaller the area of the bottom surface is, the easier it is to satisfy the requirements for accuracy and the less expensive this is. This relates only to the dimensioning and how this affects the superstructure. This alone is not sufficient to ensure a good foundation; extra measures must be taken. Figure 5-11 gives an example of this.



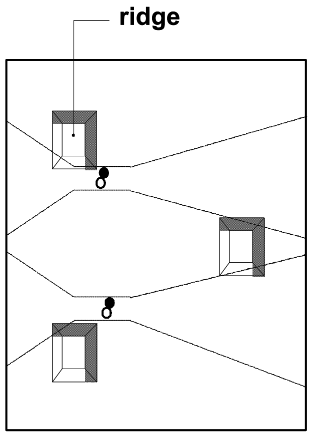


Figure 5-11 Turbine caisson

This example is a turbine-caisson (which houses 2 turbines) for a tidal power generator. The underside of the caisson is fitted with <u>3 ridges or supports</u>, used to position the caisson on the gravel bed, after ballasting. The gravel bed should be prepared or constructed to the required level by scraping off the excess material. The whole ensures the correct positioning, vertical and horizontal. For the permanent or final foundation grout is injected under the entire area (sand-cement-water) via pipelines which are cast into the concrete structure of the caisson. Grout injected under pressure without the use of side seals would not fill the space under the caisson, it would flow away sideways. The side seals can consist of a layer of synthetic material that is attached to the caisson and after placing is pumped full of grout. It is also possible to use a gravel-filled tube, as described below for the Oosterschelde storm flood defence. Depending on the horizontal area covered, compartmenting may be necessary (division of the entire area into smaller separate areas).
The injection of grout into a closed space is only possible if the water inside can escape, therefore there are multiple injection points (which can be independently opened and closed in order to get a good

distribution) in the bottom area. Injection takes place at one point while water escapes through one or more other selected points. When instead of water grout escapes from one or more of these points, the first injection point is closed and the injection of grout continues from these points.

A structure for which a support on ribs instead of on ‘’point’ supports or ridges is the Oosterschelde storm surge barrier (Figure 5-12). A detailed description of this follows.

Example: Oosterschelde storm surge barrier

The storm surge barrier consists of 64 openings with adjustable sluice gates that are separated by piers at 45 m centre to centre (c.t.c.) distance. The openings are opened and closed by lifting gates that are powered by hydraulic jacks and moved between guide rails in the sides of the piers. The traffic bridge, the lower sill and the upper sill of the sluice opening span the distance between two piers. The lower sill is embedded in rubble/riprap. When the gates are closed water can flow through the rubble. Due to the size of the Oosterschelde-basin the resulting water level rise is acceptable, i.e. the dikes around the basin are high enough to prevent flooding.

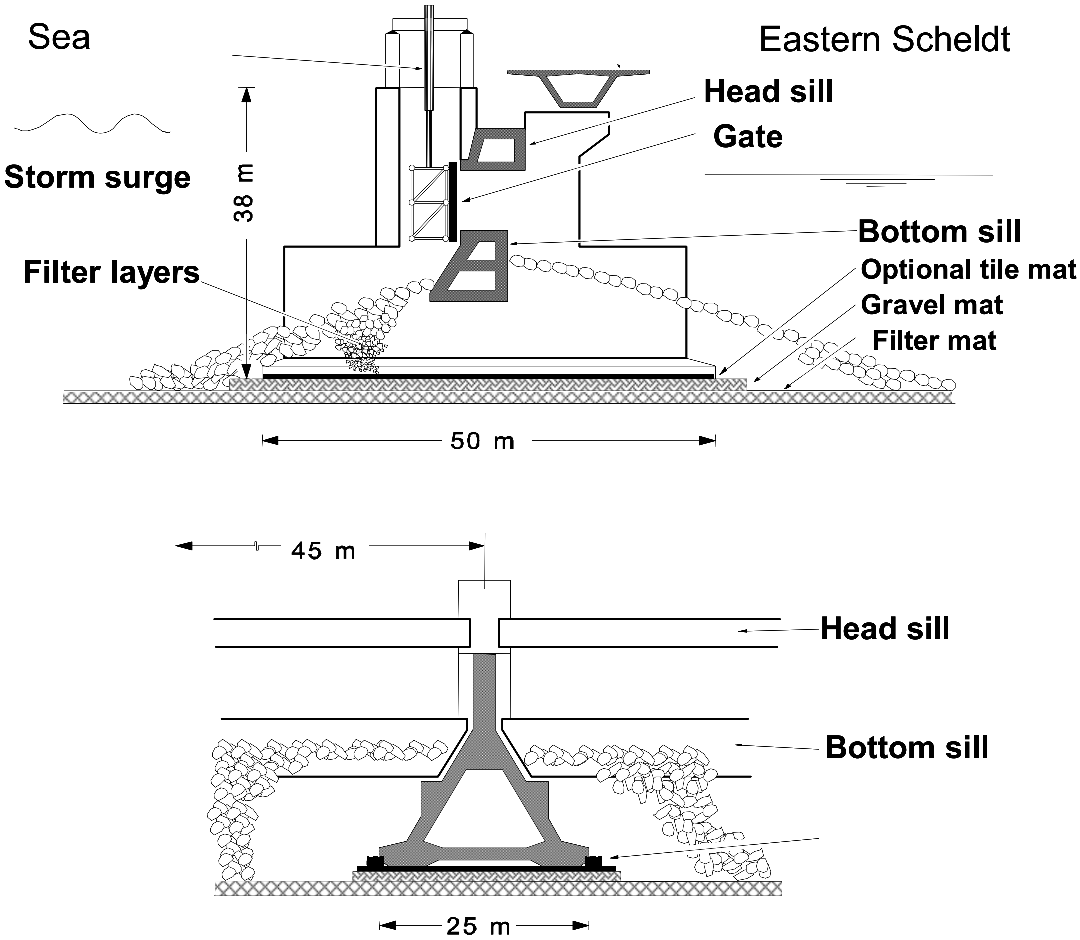


Figure 5-12 Storm surge barrier in the Oosterschelde

Many sections of the storm surge barrier were prefabricated. After the bottom had been dredged as accurately as possible and locally in-filled and consolidated by mechanical vibration (rüttelen) and other means to obtain better packing and thus better bearing capacity, prefabricated filter mats were rolled out (from floating drums), at right angles to the barrier over the entire bottom area. Each mat was about 42 m wide and about 200 m long.

The mats (a type of mattress) were ca. 0.35 m thick and the outer layers were made of a synthetic fabric. Between the two layers of fabric from the bottom to the top there were layers of fine and coarse sand, and gravel, so that a filter structure was obtained between the fine bottom material and the sill out of rubble material. The sill itself was constructed as a filter: from finer material at the bottom to coarse material at the top, and covered by a layer of heavy stone to withstand waves and currents.

The piers, with a foot area of 25 x 50 m², to protect the filter mat (during the placing of the piers) a gravel mat 31 m wide, 60 m long and 0.35 m thick, was rolled out.
Before each pier was positioned the height of the gravel mat was measured. If this fell outside the stipulated tolerances block mats were placed under the ribs of the pier foot to adjust it to the required height.
These mats consisted of concrete blocks connected by cables. The concrete blocks were dimensioned such that once laid on the gravel mats their upper surfaces would again form a horizontal surface; actually two parallel strips or a horizontal area on which the ribs of the piers could be placed.
This meant that the concrete blocks were contra moulded to the deviations of the upper surface of the gravel mat in relation to the desired foundation area (note: if the gravel mats were too high this method of correction could not be used, while if they were too low or irregular its use was possible). In practice very few block mattresses were needed because the desired degree of accuracy was usually attained.
The piers were then moved into position with the aid of a catamaran crane ship that also positioned them with the required degree of accuracy. After that a part of the sill was installed between the piers. Next the prefabricated sill beams, the bridge, the sluice gates and mechanisms were installed and the sill was finished partly by dumping and partly stone for stone for the top layers close to the under sill to avoid damage to the sill beam.

Apart from the stones, all the sections described in the above paragraph were installed with the aid of heavy floating cranes. The under and upper sills had to fit accurately between the guide trails on the sides of the piers. Positioning these in full sea from a crane ship (that was itself kept in position by cables and winches) led to horizontal deviations from the desired position (too close to the sea or too close to the Oosterschelde, not only in relation to the long axis of the retaining structure or turned the horizontal plane). Therefore the piers were surveyed after they had been installed. The adjustments were made on the outer ends of each beam. In order to lose as little time as possible the beams were already prefabricated, except for the ends. After the surveying of the position of the piers the ends of the beams were cast to size. The same applied to the steel gates the end parts of which were only made to fit after the measurement of the piers in their positions.

As in the case of many large-scale prefabricated structures, sand and silt had to be removed by dustpan dredgers and other equipment before a section could be placed. This had to be done each time a section was positioned: before the filter mat was rolled out, before the laying of the gravel, before the positioning of the piers etc. If siltation is not removed the foundation is poorer and the chance of settlement caused by compression under load or later erosion of the silt or sand increases. In first case the pier, with its ribs, which extend ca. 0.70 m below the foot plate, was placed on the gravel mat (or on the block mat, if the upper surface of the gravel mat was not sufficiently accurately positioned). Later the space under the foot area was injected with grout to provide a good and big enough foundation area. The side seals for the grouting process consisted of a gravel-filled hose. During the floating transport the hose, which entirely surrounded the foot of the pier was supported by cables so that it could not be damaged during the installation of the pier was After positioning the cables were loosened and the hose sank and resting on the bottom, formed the seals for the space under the pier, and thus the ‘formwork’ for the grout fill. The space inside the pier was filled with sand in order give the structure sufficient weight (and stability) to withstand the water pressures and wave loads when in the retaining position (closed gates as shown in Figure 5-12).

Temporary support on jacks

This method is used for the immersed tunnel shown in Figure 5-13. The tunnel is constructed entirely from reinforced or prestressed concrete in accordance with the European principles, and not as a composite structure composed of steel plates and concrete as is an American immersed tunnel.

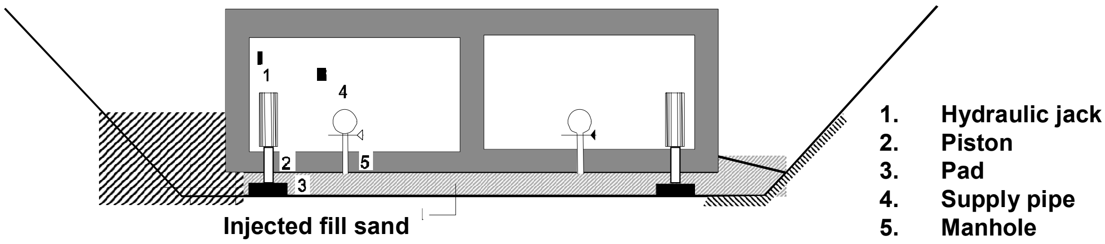


Figure 5-13 Positioning and foundation for European tunnels

The section is placed on 3 support points; on one end via an aligning support on a console that is attached to the preceding section and at the other end via two jack pins on two tiles (see Figure).
The aligning support on the previous section ensures very accurate positioning of the section is necessary in relation to the watertight joint that has to be made later. On the other end the jacks can be used to accurately adjust the height. The "tiles" (order 6xx6 m^2) are place on the bottom of the trench in advance by floating cranes. The jacks are inside the tunnel and assembled to either the bottom slab or the tunnel walls.
The 3-point support is a temporary one to permit the most accurate possible positioning of the section. The actual foundation consists of fill-sand that is supplied in the form of a sand water mixture via temporary pipeline on the inside or outside of the tunnel. In fact this is a form of levelling but with sand instead of grout.

The sand-water mixture can be pumped through the pipelines from one of the land ends and then via pipelines in the sections that have already been installed be transported to one of the injection points (the opening shown in the figure). The mixture flows under the section, the sand settles and the water escapes. The sand mixture forms a circular sand fill between the tunnel and the bottom around the first injection point, which, depending on the type of sand, the sand concentration of the mixture and the pumping pressure, can reach a diameter of 12 to 15 m. After this the injection point is closed and the following one is opened. From this a second circular fill is formed. Actually this is cannot be quite circular because it closes up against the previous one from the first injection point. By carefully choice of the injection pattern (in the transverse and longitudinal directions) it is possible to fill the space under section completely or almost completely, so that a foundation of reasonable quality is created. Unlike the grouting method this does not require side seals, but exactly the opposite: a side seal would make it impossible for the sand to settle regularly (the water must be able to flow out of the sand-water mixture). As the figure shows, the sand extends beyond the area covered by the section. After the injection of the sand layer the jacks are removed. Usually the section settles a little further (around 5 to 8 mm) because the packing of the sand is not optimal and because the fill-sand is not evenly distributed against the bottom of the section. Finally the trench is filled with the material previously dredged from it.

To date this method has only been used for immersion tunnels for which very great positioning accuracy is demanded (connection to the preceding section), while only small loads have to be transferred to the subsoil. The latter is also true immediately after immersion (with the ballast water inside it is possible to control the weight accurately; a great weight is not necessary for the stability because there is almost no horizontal load), so the jacks used do not have to be very heavy and expensive as when the final foundation is made (the fill sand). There too there are not usually heavy loads that have to be transferred to the subsoil and it is not necessary to set such high requirements on the quality of the fill sand.

Comments:

- The thickness of the fill layer that is jetted or injected under the caisson is determined by the accuracy to which the dredging of the bottom can be carried out.
- With dredging tolerances of + or - 0.1 m a layer thickness of 0.5 m is adequate.
- Silt deposits must be removed. The trench that cuts directly across a river or estuary bottom causes local deepening, as a result of which the velocity of the currents decreases, so sand and/or silt can settle on the bottom. This may make it necessary to clean the bottom with a dustpan dredger as short a time as possible before the placing of the fill. Silt inclusions can also lead to undesirable settling.
- This is also true for the bottom directly under the blocks/tiles. In exceptional situations (great sedimentation/silt deposition) even if the silt under an section that has been placed before the start of the injection of the sand has been removed or measures must be taken to prevent this sedimentation, (for example by installing reinforced synthetic cloth along the side of the section to close the split/gap).
- Apart from jetting there are other methods that can be used to install the sand layer under the section.