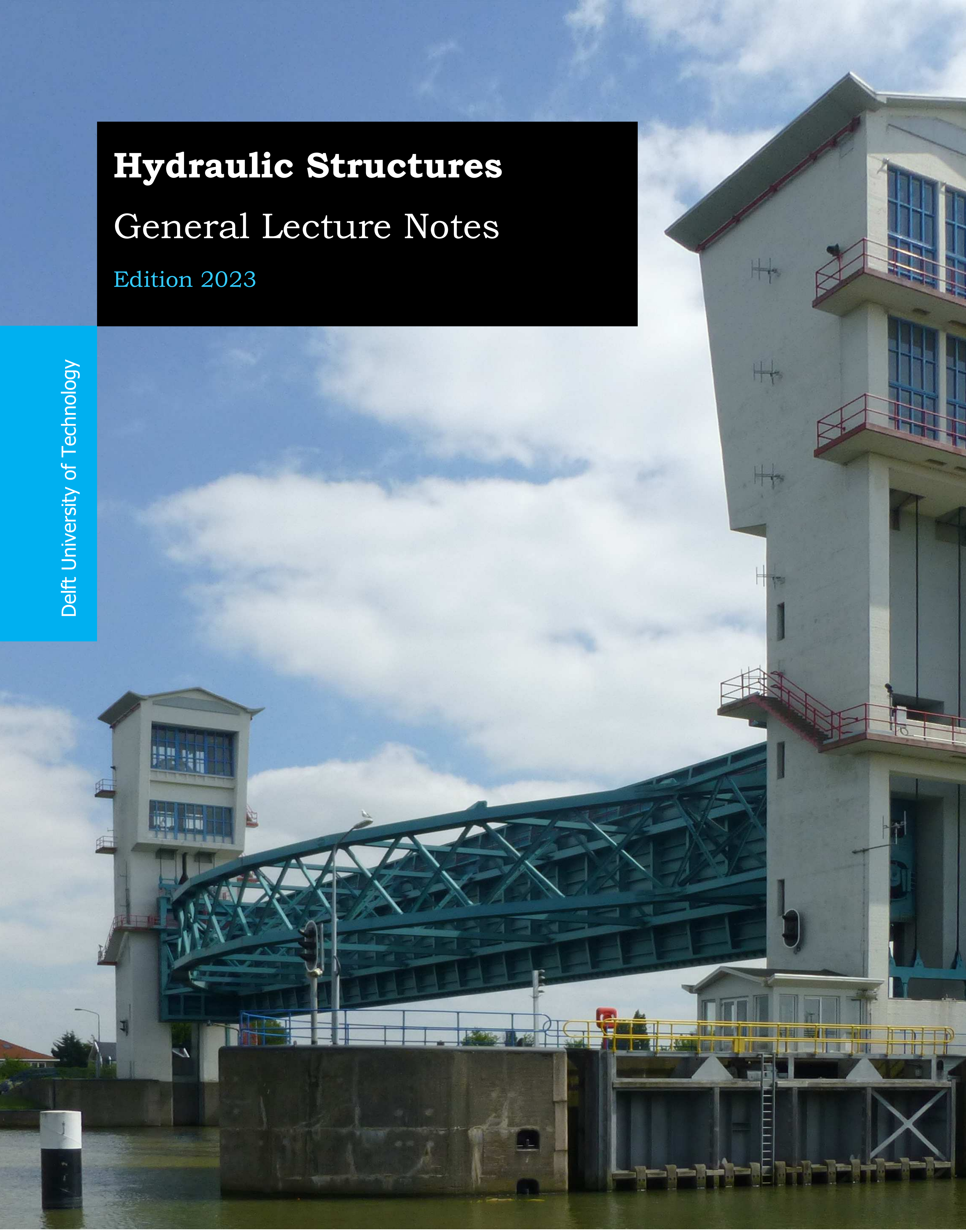
Hydraulic Structures

General Lecture Notes

Edition 2023

Delft University of Technology



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Challenge the future

HYDRAULIC STRUCTURES
GENERAL LECTURE NOTES

Edition 2023

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PREFACE

These General Lecture Notes belong to the educational material of the elective course "Hydraulic Structures," part of the Bachelor of Science Civil Engineering at Delft University of Technology.

The course "Hydraulic Structures" focuses on engineering design, for which the knowledge and methods of other engineering and/or science disciplines come together in a practical application. This mainly involves theoretical fields, such as material sciences, structural engineering, soil and fluid mechanics, see Figure 1.

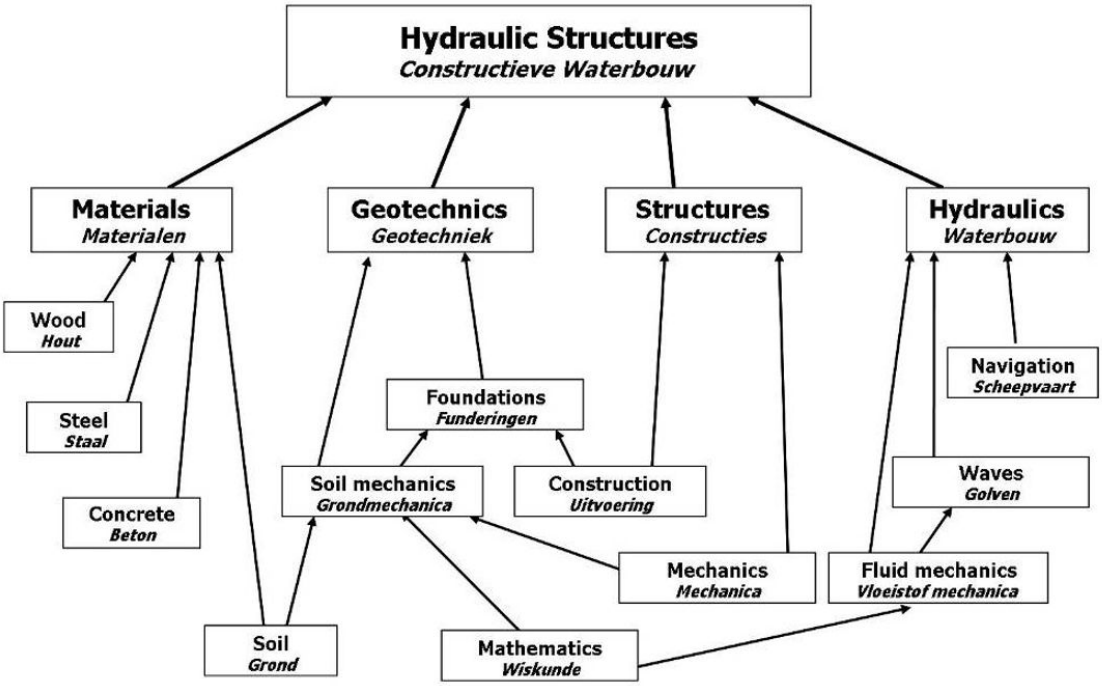


Figure 1: Relations with other disciplines

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

1. General Lecture Notes
2. Manual Hydraulic Structures, including an Exercise Bundle
3. Locks
4. Caissons

The focus in these documents is on the functional and structural design of hydraulic structures. The General Lecture Notes explain the basics of the general civil engineering design approach, the functional design of hydraulic structures and construction aspects. The Manual Hydraulic Structures restricts to the methods and techniques needed for a structural design. The lecture notes on "Locks" and "Caissons" focus on design of a specific type of structure.

The lecture notes partly repeat material covered in other courses that have already been completed by the student. These summaries have been included, mainly in the “Manual Hydraulic Structures”, to be able to quickly apply the considered theory to a design from a hydraulic engineering point of view. Occasionally, the material covered in the Manual book is more advanced than the theoretical subjects covered elsewhere in the curriculum. In such cases, no theoretical derivations are given, as the Manual remains confined to stating the design rules and showing practical approximations.

The course "Hydraulic Structures" 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 "Hydropower Engineering", "Bored and Immersed Tunnels," "Hydraulic Structures 2" (weirs, barriers and port Infrastructure)" and "Flood Defences" in the MSc. education, see Figure 2.

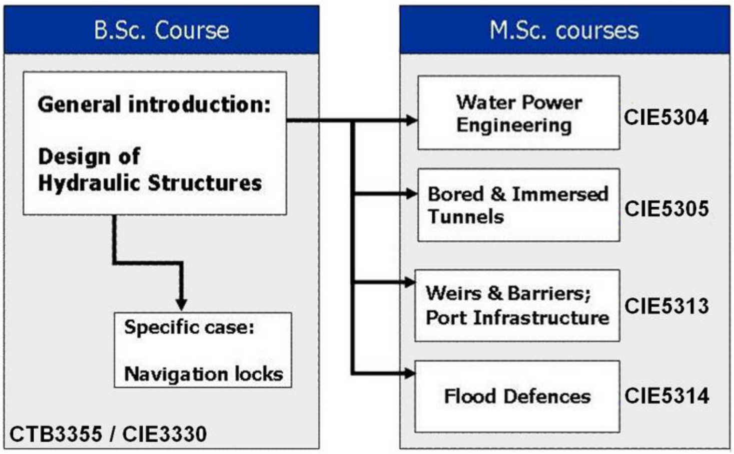


Figure 2: Modular structure of the courses Hydraulic Structures at TU Delft

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

Wilfred Molenaar
Delft, November 2009

The contents of these General Lecture Notes origin from older course material, mainly developed by ir. K.G. Bezuyen, ir. W. Colenbrander, ir. H.K.T. Kuijper, ir. C. Spaargaren, prof.drs.ir. J.K. Vrijling and dr.ir. S. van Baars.

In the 2018 edition of the General Lecture Notes, many pictures have been improved and the layout has been slightly changed. The hydraulic structures in Chapter 1 have been grouped into categories and their descriptions have been updated. The contents of the other chapters have mainly remained unchanged. The pictures with source 'TU Delft" have been made by the most recent reviser of these lecture notes. The 2019 edition was quite similar to the 2018 version.

In the edition of 2020, the overview of hydraulic structures in Chapter 1, was slightly expanded. The description of the engineering design process in Chapter 2 was reorganised: For clarity, the design activities were grouped per design phase and several design activities were added, for instance the process and function analyses. In addition, the explanation of the multi-criteria analysis was improved. Chapters 3, 4 and 5 have not been changed. An appendix has been added with an example of a design calculation. In 2022, a second example of a design calculation was added.

For the 2023 edition, the chapter about structural safety was added and extended (coming from the Manual Hydraulic Structures 2022). The establishment of the basis water levels by the Delta Committee was included in a new appendix (based on Appendix B of my dissertation). Meanwhile, the original MS Word document was transferred to Latex.

Mark Voorendt
Delft, February 2023

Contents

1

Overview of hydraulic structures

1.1 Definitions and categorisation

A **structure** is an arrangement and organization of interrelated elements in a material object or system. A **hydraulic structure** (*waterbouwkundige constructie*) can be defined as any civil feature that is intended to divert, restrict, stop or otherwise manage the natural flow of water, or facilitate shipping. A **hydraulic engineering work** (*waterbouwkundig kunstwerk*) is a hydraulic structure that is part of a flood defence (*waterkering*), which enables the crossing with (other) infrastructures. These crossings concern shipping, road traffic, water management, or passage of fauna. A **flood defence** is a hydraulic structure intended to protect land from being covered by water.

The primary function of a hydraulic structure aims at fulfilling a societal need, like protecting against floods, or enabling shipping. Hydraulic structures are often part of more comprising hydraulic systems. For example, a quay wall is a part of a harbour and a dewatering sluice is a part of a polder system.

This chapter briefly describes the following hydraulic structures[1]:

Shipping
1. Inland waterways (*binnenvaarwegen*)
2. Navigation locks (*schutsluizen*), ship-lifts (*scheepsliften*) and inclined planes (*hellende vlakken*)
3. Weirs (*stuwen*)
4. Quay walls (*kademuren*)
5. Jetties (*steigers*)
6. Breasting dolphins (*meerstoelen, dukdalven*), mooring dolphins (*bolderstoelen*)
7. Approach walls (*geleidewerken*) and fenders (*stootkussens*)
8. Breakwaters (*golfbrekers, havenhoofden*) and river groynes (*kribben*)

Flood protection
9. Dikes
10. Closure dams (*afsluitdammen*)
11. Storm surge barriers (*stormvloedkeringen*)
12. Discharge regulating structures (*regelwerken voor rivierwaterafvoer*)

Water control in polders
13. Pumping stations (*gemalen*)
14. Dewatering sluices (*uitwateringssluizen*)
15. Discharge sluices (*spuisluizen*)
16. Flushing sluices (*spoelsluizen*)
17. Inundation locks (*inundatiesluizen*)

[1] It has been attempted to categorise hydraulic structures by their use, rather than by their shape or location. However, several structures can be used for multiple purposes, so the arrangement in the proposed categories is not unambiguous.

Irrigation and energy
18. Reservoir dams (*stuwdammen*) including intake structures (*inlaten*) and spillways (*overlaten*)

Structures for crossing infrastructures
19. Piers (*pijlers*)
20. Tunnels
21. Aqueducts
22. Naviducts
23. Culverts (*duikers*), syphons and inverted syphons (*grondduikers*)

(Temporary) structures for construction
24. Dry docks (*droogdokken*) and floating docks (*drijvende dokken*)
25. Building pits (*bouwputten*), cofferdams (*bouwkuipen*) and construction docks (*bouwdokken*)

Miscellaneous
26. Artificial islands (*kunstmatige eilanden*)
27. Soil retaining structures (*grondkerende constructies*)

1.2 Inland waterways

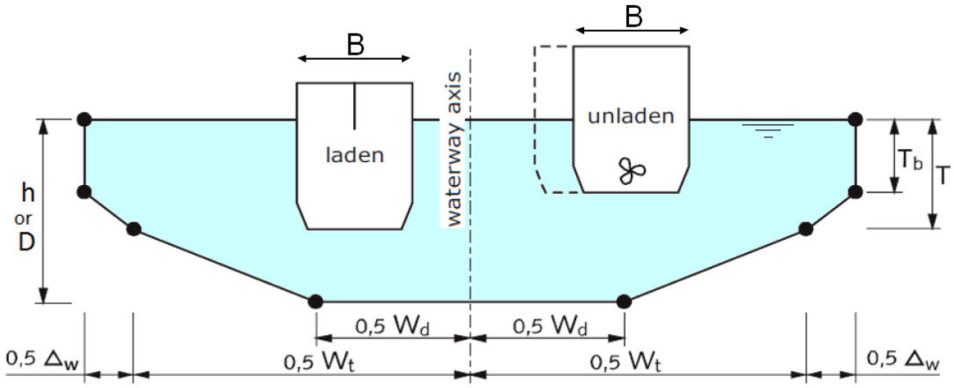
Waterways form the infrastructure that facilitate transport of freight or people over water. They have to provide sufficient water depth and width to allow the desired quantity of shipping. This not only requires a minimum cross-sectional area, but also facilities to reduce sedimentation or erosion, for berthing and mooring, for loading/unloading of freight, embarking and disembarking people and for overcoming water level differences in different (parts of) waterways. The following sections treat several of these facilities. This section concentrates on the cross-sectional area, required for shipping.

The starting point for the dimensioning of inland waterways is the size of the vessels and the intensity of shipping. Many countries have a classification for inland waterways, based on vessel types. The European Classification System of inland waterways (CEMT/ITF, 1992) is the basis for all European Guidelines. Rijkswaterstaat updated the CEMT classification in 2011, which was adopted in PIANC report 141, see Table 1.1.

	Motor vessel (m)			Pushed convoy (m)			Coupled units (m)		
Class	Beam	Length	Draught	Beam	Length	Draught	Beam	Length	Draught
I	5.05	38.5	2.5	5.2	55	1.9	5.05	77-80	2.5
II	6.6	50-55	2.6	6.6	60-70	2.6			
III	7.2-8.2	55-85	2.6-2.7	7.5-8.2	80-85	2.6-2.7			
IVa	9.5	80-105	2.9-3.0	9.5	85-105	3.0			
IVb							9.5	170-185	3.0
Va	11.4	110-135	3.5	11.4	95-135	3.5-4.0			
Vb				11.4	170-190	3.5-4.0	11.4	170-190	3.5-4.0
VIa	13.5-17.0	110-135	4.0	22.8	95-145	3.5-4.0	19.0-22.8	85-110	3.0-4.0
VIb				22.8	185-195	3.5-4.0	22.8	185	3.5-4.0
VIc				22.8	270	3.5-4.0			
VIIa				34.2	195	3.5-4.0			

Table 1.1: Dutch classification of vessels (PIANC report 141, 2019)

The design process of waterways starts with defining the preferred class. If the waterway is used by different types of vessels, a reference vessel is to be defined for each type. In Europe, there are motor vessels, pushed convoys and coupled units. Figure 1.1 shows the standard minimum canal width dimensions in the Netherlands, as an example. The channel depth is 1,4 times the laden draught for the normal profile and 1,3 times for narrow and single-lane profiles compared to the reference low water level (not exceeded at 99% of the time), see Table1.2.



Dutch Profile		
Profile	W_d	W_t
Normal	2*B	4*B
Narrow	2*B	3*B
Single-lane	B	2*B

Figure 1.1: Dutch canal width dimensions (B = the beam (width) of the reference vessel, see Table 1.1) (PIANC report 141, 2019)

For more information, for example on the dimensions of bends and rivers, reference is made to PIANC report 141 (2019) 'Guidelines for inland waterway dimensions.'

Class	Channel depth (m)			Wind Increments [m]			
				normal		narrow	
	normal	narrow	single	inland	coast	inland	coast
I	3.5	3.3	3.3	2	4	3	5
II	3.6	3.4	3.4	3	6	4	7
III	3.8	3.5	3.5	4	8	5	10
IV	4.2	3.9	3.9	5	11	7	15
Va	4.9	4.6	4.6	7	14	7	15
Vb	5.6	5.2	5.2	9	18	12	24
VIa	5.6	-	-	6	14	-	-
VIb	5.6	-	-	9	19	-	-

Table 1.2: Channel depth and side wind increments for Dutch canals (PIANC report 141, 2019)

1.3 Navigation locks, inclined planes and ship-lifts

The purpose of lock facilities is to overcome a water-level difference between two water stretches for shipping, whilst maintaining the difference in water levels. Lock facilities are therefore designed to transport ships vertically. Meanwhile, it should be prevented that water flows from the high level side to the low level side in too high quantities in an uncontrolled way.

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 vertically transporting the ship and its surrounding water together in a closed chamber.

The first concept is used for (navigation) locks, the latter in ship-lifts and inclined planes. Ship-lifts and inclined planes 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.

1.3.1 Navigation locks

Navigation locks are used for instance if ships have to pass a weir or a 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 flood defence (Figure 1.2)

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 enable a more controlled supply and discharge of water, resulting in more convenient ship handling during emptying and filling the lock chamber.

Figure 1.2: Navigation lock with mitre gates in the Haringvlietdam (TU Delft, 2014)

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 the locking cycle, if the sea level is higher than the 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, reducing the loss of water.

In the case of large water level difference, a series of locks can be used to reduce the loss of water. Examples of this are the seven locks between the Ottawa River and the Rideau Canal in Canada, the staircase locks of Fonserannes in the Canal du Midi near Béziers, France (Figure 1.3) and the locks next to the Three Gorges Dam in China.

Navigation locks in the Netherlands lift and lower ships up to about 12 meters and are subjected to the condition that a sufficient volume of water is available to compensate the intrusion of salt water caused by the locking process. Abroad, locks are used with still bigger water level differences, in which cases usually 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. The newest Gatún lock in Panama has three lock chambers with three large storage basins each.

Lock gates can have an additional function of controlling the water level in the upper canal section (Figure 1.4). In that case, it more or less functions as a weir. This combination is not common in the Netherlands.

For the functional design of navigation locks (main dimensions, gate and levelling systems), as well as the

Figure 1.3: Staircase locks of Fonserannes in the Canal du Midi, France (TU Delft, 2015)

Figure 1.4: Gates of a lock can be used to regulate the water level in the upper canal section. Cambrai, France (TU Deflt, 2015)

structural design (constructability, stability, strength), see the lecture notes CTB3355 'Locks'.

1.3.2 Inclined planes

An inclined plane makes use of the natural slope of the surroundings. There are two types of inclined planes:

1. 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).
2. The ships and water are pushed up and down a slope in a chamber (Figure 1.5). The chamber is sealed at both ends by closing elements. The adjoining canals are also equipped with closing elements.

1.3.3 Ship-lifts

The chamber of a ship-lift is usually filled with water and it transports ships vertically by means of screw pumps, plungers or cables (Figure 1.6). Ship-lifts can be operated by letting air into or out of the plungers (plunjers, zuigers), causing the floating bodies to rise or to sink. In the case of a ship-lift with cables, all moving parts are situated on the surface area. Plungers and floating bodies, however, require much space beneath the structure. Irregular settlement of the structure has to be avoided, because of the danger that the plungers get stuck against the walls. The weight of the chamber can be compensated by counterweights or by a second chamber.

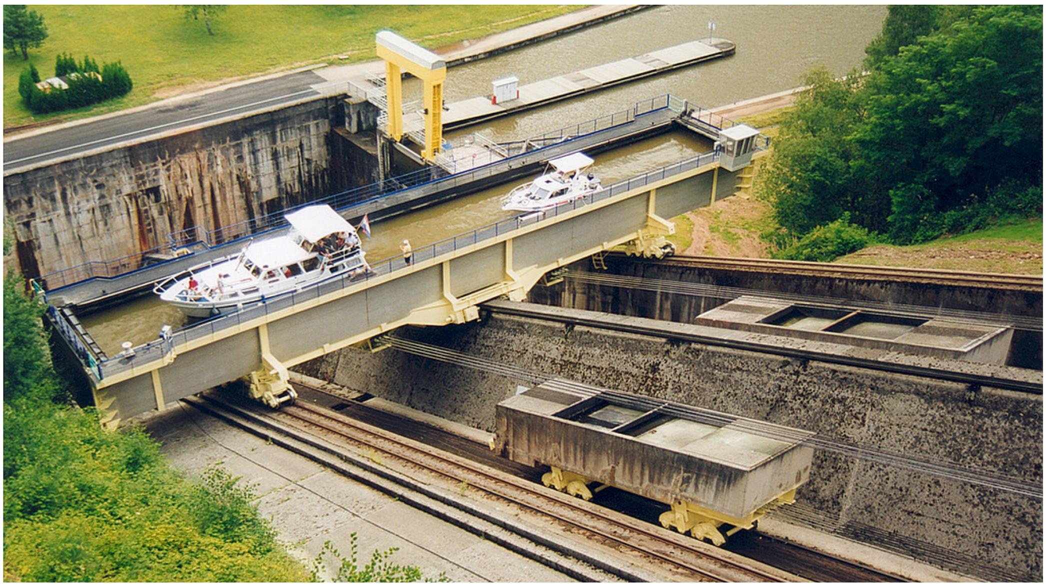


Figure 1.5: Inclined plane in lateral direction, Marne-Rhine Canal, France (Wikimedia Commons, 2006)

The ship lift near the Three Gorges Dam in China, the biggest in the world, has a vertical lift of 113 m. The chamber dimensions are 280 x 35 x 5 metres and its cycle time is 30 to 40 minutes. The Falkirk Wheel in Scotland is well-known because of its architecture. Its vertical lift is 24 metres, its chamber dimensions are 21 x 6 x 1,5 metres and its cycle time is only 4 minutes.

Figure 1.6: Cable ship lift of Strépy-Thieu, Belgium, with a vertical lift of 75 m (Wikimedia Commons, 2005)

1.4 Weirs

Weirs in rivers head up the water flow upstream of the structure. This can be done for multiple reasons:

- to ensure sufficient water depth for shipping (upstream of the weir)
- to protect against floods (downstream of the wear)
- to provide water for the irrigation of land for agriculture (upstream of the weir)
- to (partly) divert the river discharge (see also Section 1.13)

The two main types of weirs are:

- fixed weirs, that are not adjustable (Figure 1.7)
- weirs with moveable gates (Figure 1.8)

Weirs that head up the upstream water level for electricity production are usually called reservoir dams (*stuwdammen*), see Section 1.19.

Figure 1.7: Fixed weir in the Someşul Mic in Cluj Napoca, Romania (TU Delft, 2013)

In the Netherlands, there are several fixed weirs along the Meuse and the Lower Rhine. The original purpose of the weirs is to canalise the rivers to improve their navigability. Rising energy prices have made it economically interesting to use the pressure head over the weirs to generate hydro-electricity. Until now, the efficiency was very low. 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 several hydro-electric stations were afterwards built next to the weirs. The way in which the water levels are influenced by weirs is very important for the functional and spatial design of a weir.

Figure 1.8: Visor weir in the Lek near Hagestein (TU Delft, 2016)

1.5 Quay walls

The primary functions of a quay structure are:

- enabling berthing and mooring ships;
- supporting cranes and vehicles;
- supporting temporarily stored cargoes/goods.

Structural functions of a quay structure are providing sufficient stability and strength to resist the forces:

- retaining soil;
- resisting mooring and berthing forces
- resisting loads on top of the quay
- water retention or preventing groundwater flow, if required;
- bank protection / erosion protection.

There are four main types of quay walls:

1. Gravity walls, including block walls (Figure 1.9), L-walls, caisson walls and cellular walls;
2. Sheet pile walls, freestanding or anchored, single or combined, incl. diaphragm walls (*diepwanden*) and cofferdam walls (*kistdammen*);
3. Structures with relieving platforms (*ontlastvloeren*) (Figure 1.10);
4. Open berth quays (jetty-like structures with a deck on piles that extends over a slope).

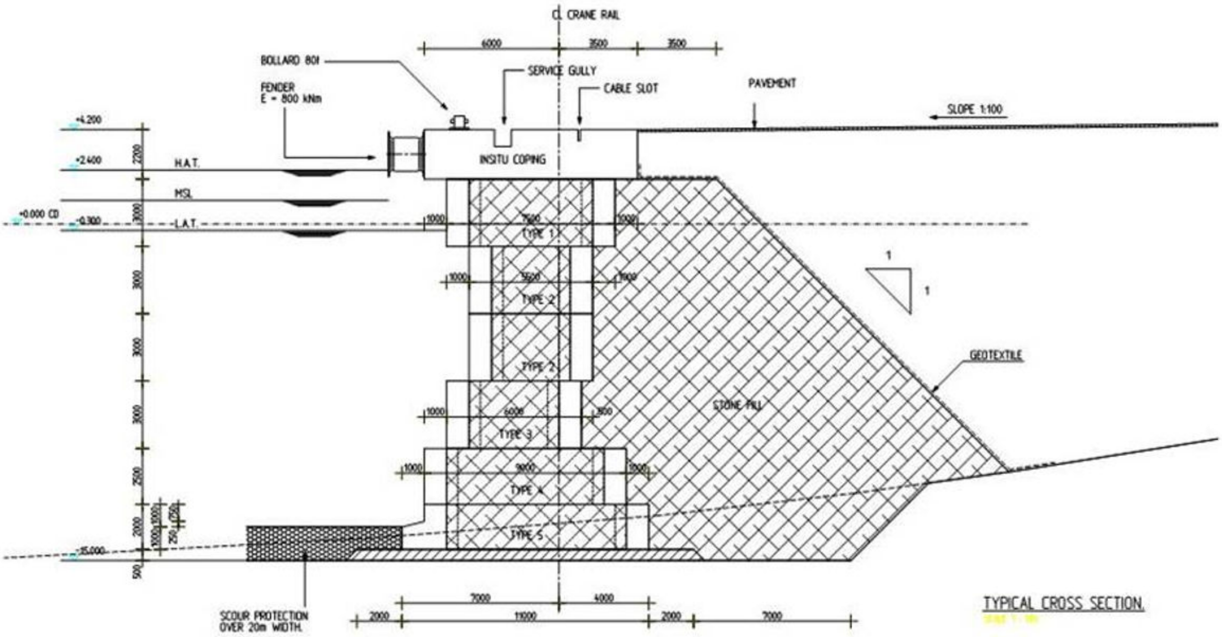


Figure 1.9: Block wall quay wall cross-section

Figure 1.11 shows the components that determine the design depth in front of a quay. As a rule of thumb, the required keel clearance can be estimated at 10% of the loaded draught of the vessel. The guaranteed nautical depth thus is equal to LLWS - 1,10 x the governing draught.

The sounding accuracy depends on the sounding equipment, but usually is about a few centimetres. The maintenance margin is 0,50 m if there is scour protection in front of the quay wall and otherwise it is 1,00 m. The dredging tolerance depends on the used dredging equipment and usually varies between 0,20 m for a water depth of 5 m to 0,70 m for a water depth of 25 m (Quay Walls, 2014).

The top of the quay level can be determined on the basis of wave overtopping requirements or by other usability considerations (for instance regarding loading/unloading).

Vessels are attached to quays by means of hawsers (*trossen*). The forces on the hawsers, amongst others, depend on the mooring configuration (Figure 1.12). The bollards, that are connected to the quay (see Figure 1.14 for an example), have to resist these forces.

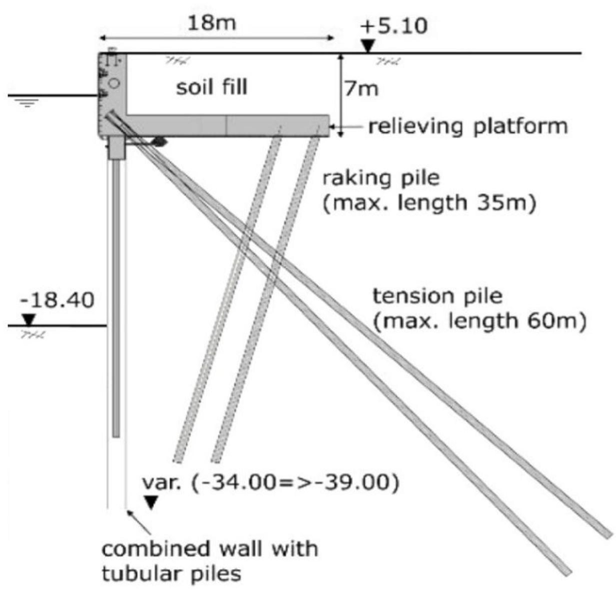


Figure 1.10: Composite quay wall at an offshore terminal in Maasvlakte 2, Rotterdam (left: Port of Rotterdam, right: TU Delft, 2016)

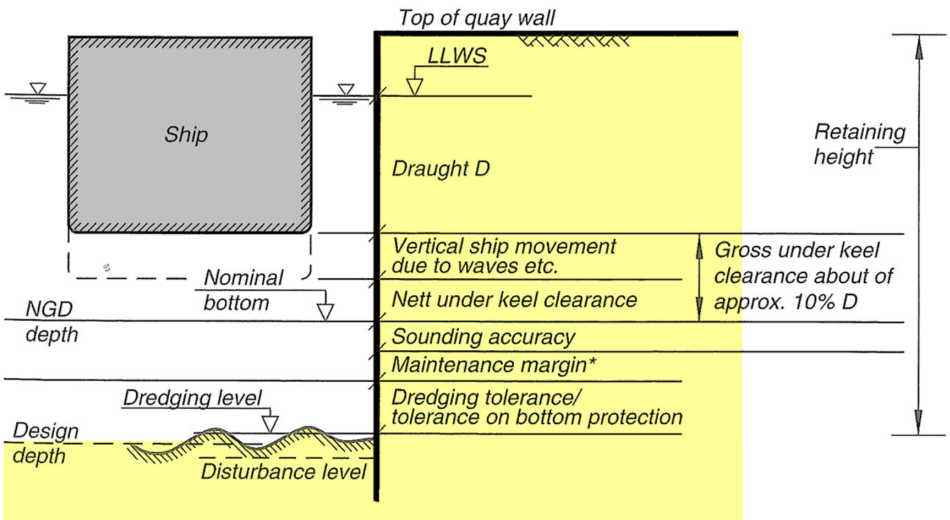


Figure 1.11: Determination of the nautical guaranteed depth (NGD) and the design depth (Quay Walls, 2014)

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 level of the quay surface relative to the water level;
- facilities such as fenders, bollards (bolders), quick release hooks (*sliphaken*), ladders etc.

An example of a quay layout facilitating these primary functions is shown in Figure 1.13).

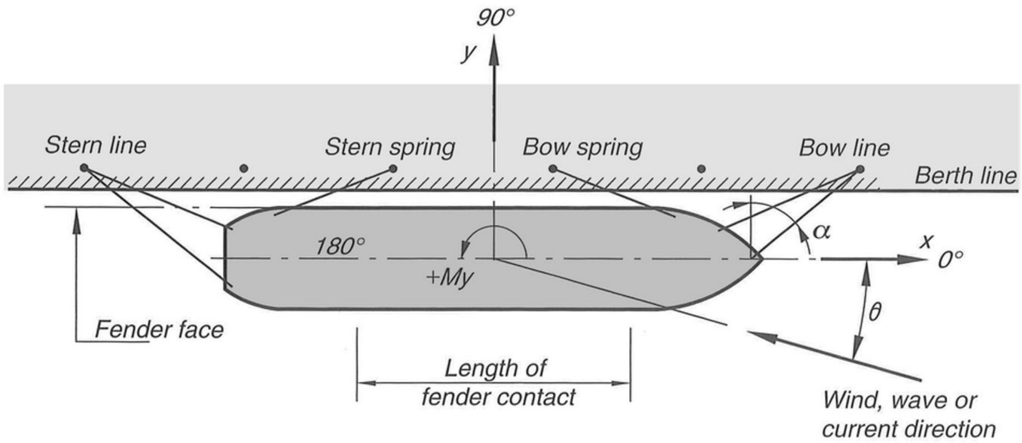


Figure 1.12: Example of a mooring configuration (Quay Walls, 2014)

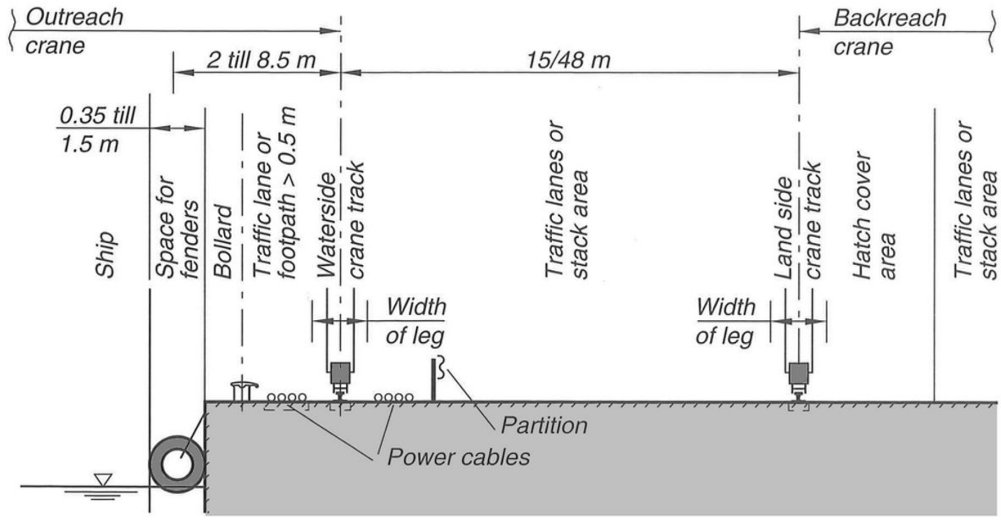


Figure 1.13: Example of the layout of the transshipment area of a container terminal (Quay Walls, 2014)

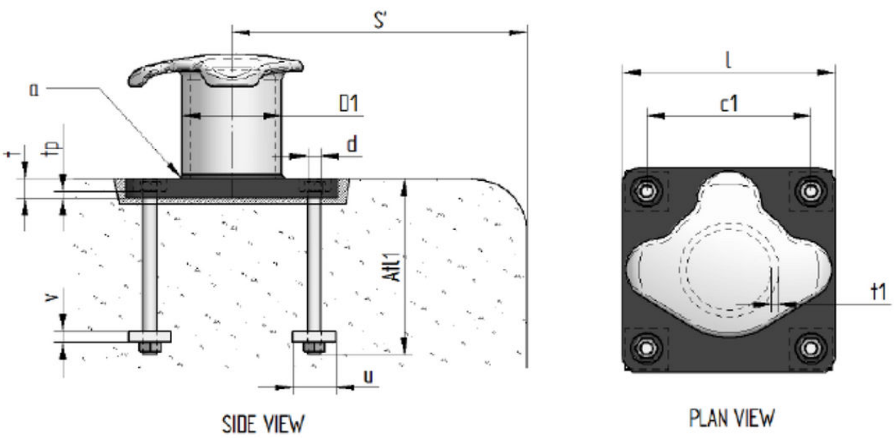


Figure 1.14: Example of the connection of a bollard to a concrete gravity wall (Havenbedrijf Rotterdam)

1.6 Jetties

The primary functions of a jetty (*steiger*) are:

- enabling berthing and mooring of ships;
- facilitating the loading/unloading of 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 considerable distance into sea, to a spot where sufficient water depth is available for deep vessels, thus avoiding much dredging. If designed and constructed for that reason or advantage, ships can be moored on both sides of the jetty. In that case, a quay wall would have to be twice as long.

Often, a high level of function separation can be observed. Berthing dolphins that have own fenders absorb the kinetic energy of the ship. Once the ship has been moored, the forces on it caused by the wind and waves are transferred to mooring dolphins through hawsers (*trossen*). 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.

Several 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 the 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 (*loopbrug*) 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, see Figure (1.15).
- 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, see Figure (1.16).
- Detached jetties: Long trestles (*loopbruggen*) 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 installing 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 1.17). Special attention should be paid to the positioning of the mooring-posts. The berthing dolphins are usually placed slightly forwards of the jetty and the mooring dolphins are placed further back to prevent collisions.

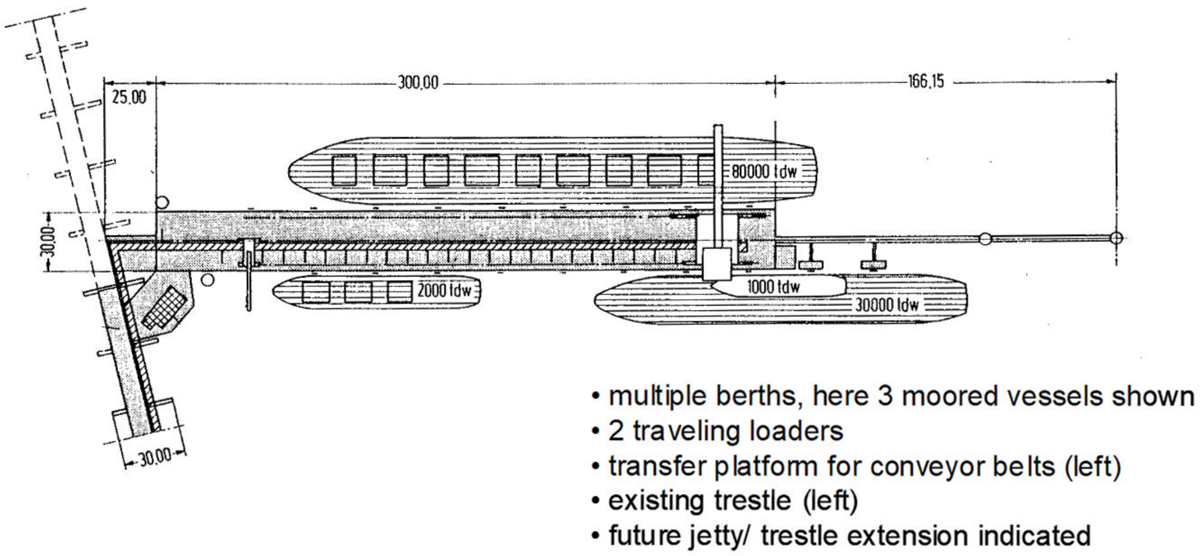


Figure 1.15: Top view on the main jetty or wharf of an L-type jetty

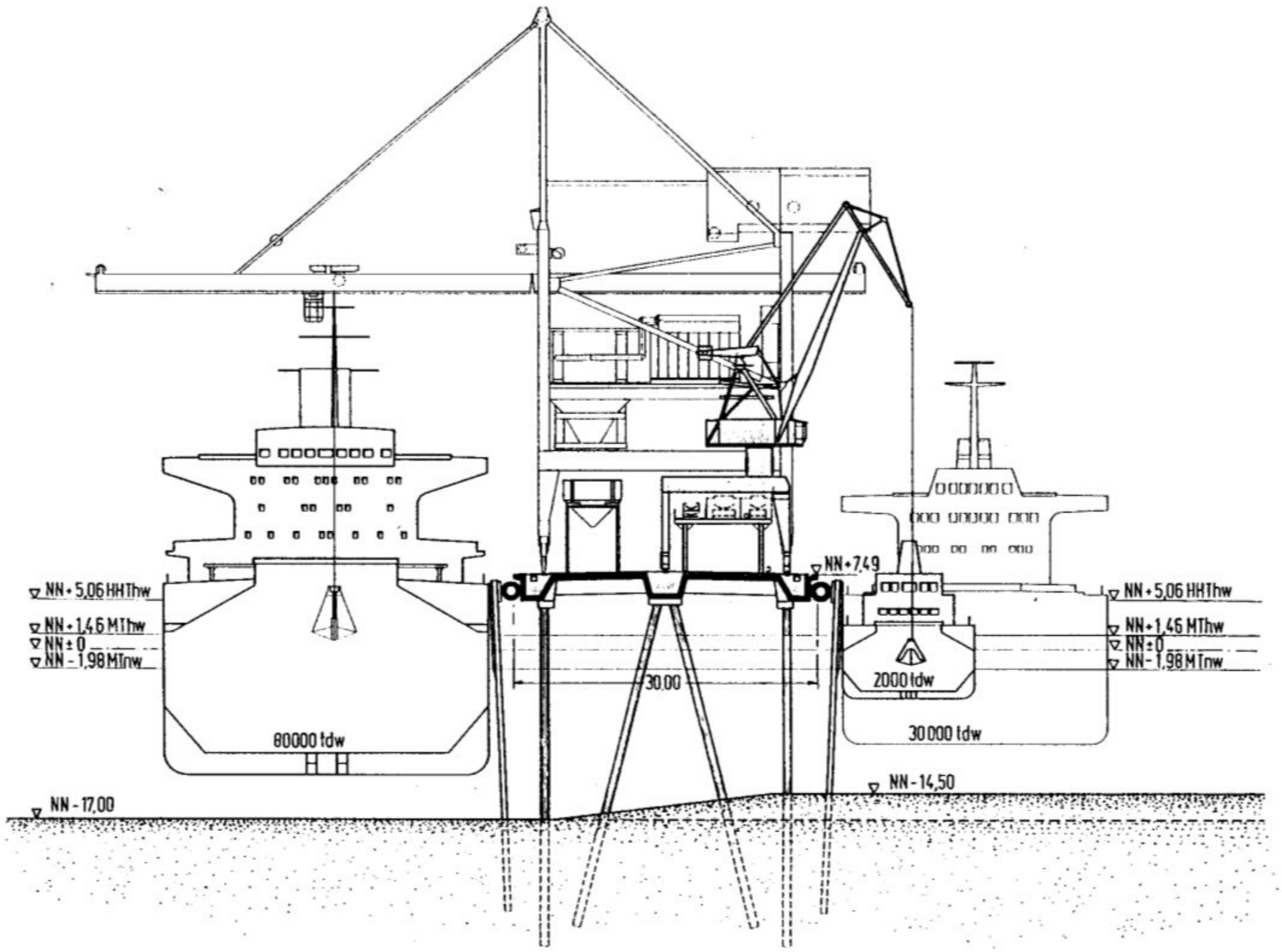


Figure 1.16: Cross-section of a main jetty

Figure 1.18 shows an example of a berthing structure. Vessels that have to wait to pass through the naviduct near Enkhuizen can wait here for green light.

Figure 1.17: Detached jetty in Malaysia (http://www.citech.co.uk)

Figure 1.18: Berthing structure near the Naviduct of Enkhuizen (TU Delft, 2015)

1.7 Breasting and mooring dolphins

Breasting dolphins, also known as berthing dolphins (*meerstoel, 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 (*sliphaken*) holding the ships' mooring lines;
- resist horizontal forces.

A secondary function of dolphins in general is:

- provide space for either berthing and/or mooring equipment.

Breasting and mooring dolphins are relatively simple structures used for mooring vessels. The structures have no functions other than slowing down ships (berthing) and keeping the ship in position when moored. Dolphins are typically 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 raking piles (*geschoorde palen*) that are mutually connected 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, sometimes on the available space for the structure. The performance of a breasting dolphin depends on its ability to dissipate the ship's berthing energy.

Flexible dolphins are generally single, large diameter, vertical piles, see Figure 1.19). 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.

Figure 1.19: Flexible dolphin in the Port of Rotterdam

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.

A ship hitting the fender panel results in deformation of the fender and a relatively large pile top displacement because the pile is loaded horizontally. After the initial deformation, the fender and the 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.

For large sea-going ships it is more common to use rigid breasting dolphins. They consist of 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 of steel. The weight of the concrete superstructure reduces the tensile forces in the piles, 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, for a rigid dolphin, a larger fender will be required because this fender has to dissipate nearly all the berthing energy alone. See Figure 1.20 for an example of a rigid dolphin.

Although the idea of mooring is to keep the ship in a fixed position, the ship should not be tied completely solid to mooring dolphins to be able to cope with 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.

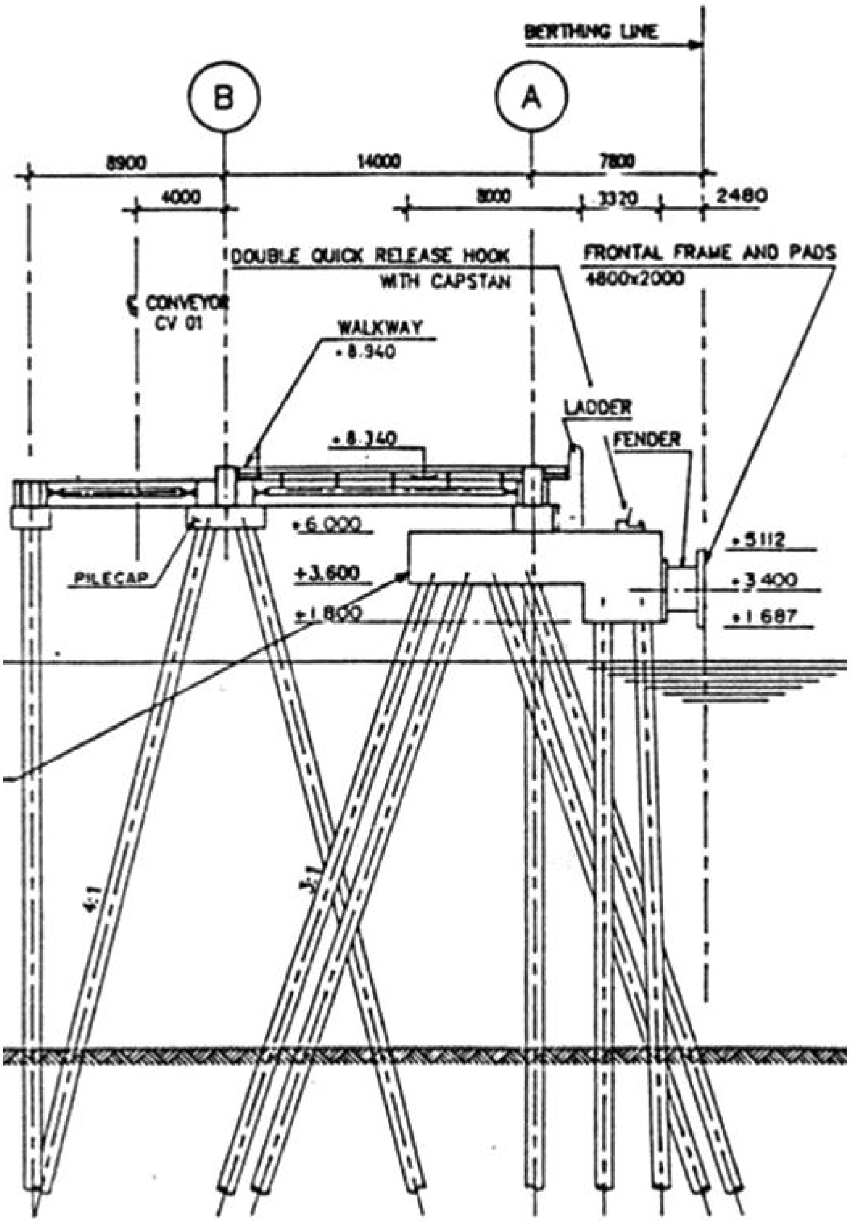


Figure 1.20: Rigid dolphin combining berthing and mooring function

1.8 Approach walls and fenders

1.8.1 Approach walls

Approach walls (Figure 1.21) 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 (*gordingen*). 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 1.21: Approach wall near the navigation lock of Born (TU Delft, 2014)

1.8.2 Fenders

The function of a fender (*stootkussen*) 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 remain below an acceptable limit. Fenders as such are not hydraulic structures, but usually a part of quay walls or lock chambers. Three types of fenders are shown in Figure 1.22.

Figure 1.22: Three types of fenders (TU Delft, 2014, 2015)

The following characteristics determine 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.

1.8.3 Collision structures

Collision structures are structures that prevent damage to another structure in case of a ship collision. They are placed next to or in front of the structure that has to be protected. The collision structure dissipates the energy of the colliding vessel and prevents that the protected structure is affected by the impact. The acting loads are transferred to the subsoil through the foundation of the collision structure. See Figure 1.23 for examples.

(a) Steel collision structure in front of a bridge pier in the Oder near Sczcecin, Poland (TU Delft, 2017)

(b) Concrete collision structure in front of the lift-bridge over the Garonne near Bordeaux, France (TU Delft, 2019)

Figure 1.23: Examples of collision structures

1.9 Breakwaters and groynes

The main functions of a 'breakwater' (*golfbreker*) or a 'mole' (*havenhoofd*), see Figure 1.24, are:

- create quiet mooring places and calm navigation water by reflecting and breaking of waves;
- improve the manoeuvrability of vessels by guiding and diverting cross currents;
- prevention of sedimentation by restricting the width of the navigation channel.

Additional functions can be:

- visual guidance of navigation;
- provision of dock or quay facilities;
- creation of space for recreation.

A groyne (*strekdam, strandhoofd*) is a type of breakwater that has the primary function of preventing sedimentation of navigation channels or to divert currents. River groynes (*kribben*) are situated every few 100 metres at both sides of the summer beds of rivers. They cause higher flow velocities in the narrowed summer beds and reduce meandering of the river and increase the navigational depth. Groynes can be beneficial for shipping, but can also be used to prevent erosion of beaches.

Figure 1.24: Breakwater in Palermo, Italy (TU Delft, 2015)

A breakwater functions as a barrier for waves and/or currents. To stop the waves and the currents effectively, the breakwater has to be a more or less solid obstacle from the bottom or bed up to 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;
- 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.

Combinations of the three fundamental types are possible as well, known as 'composite breakwaters', see Figure 1.25.

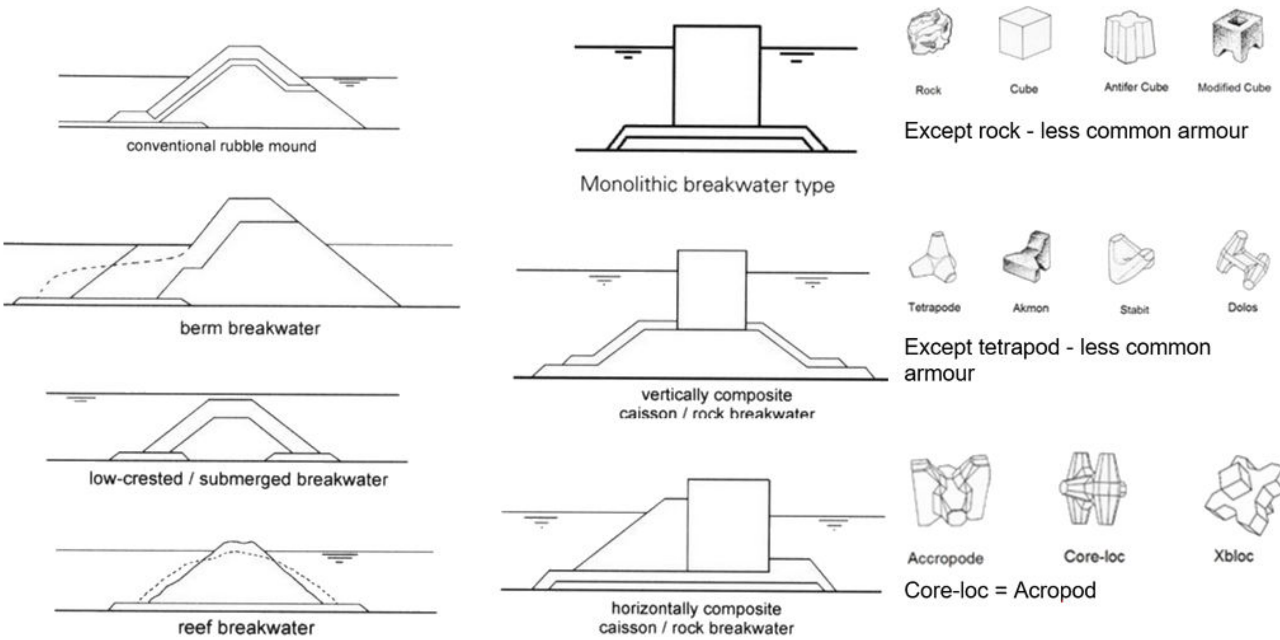


Figure 1.25: Breakwater types and concrete armour types

Reference is made to the course 'Breakwater and Closure Dams' (CIE5308) for more information.

1.10 Dikes

Dikes are flood defences, which protect land from being flooded by water coming from seas, rivers, lakes, canals and other 'surface waters'. They mainly consist of an embankment (soil body) with sufficient retaining height to withstand design water levels and accompanying wave heights. Figure 1.26 shows the most important components of dikes. The revetment (bekleding) on the outer slope (the slope on the water side) can consist of grass, concrete elements, stone or asphalt and should be able to withstand wave impact, and currents in case of water-adjoining dikes (schaardijken). This is a protective measure against erosion. Sea and lake dikes can be exposed to high wave impact. An outer berm can be applied to break the waves and thus reduce overtopping discharges. The level of the outer berm usually is at storm surge level. Inner berms are more common for river dikes. They can provide extra slope stability (stability berm), because it counter-acts the driving moment of a circular slip body (macro-instability). The same effect can be reached by a gentle inner slope (less than 1:3). Another form of piping berms are the inner berms and provide a longer seepage path, thus reducing the probability of piping. Piping berms are usually lower and wider than stability berms. The slope angle influences wave run-up (and successive overtopping) as well as dike stability.

The soil composition of a dike is related to the properties of the distinct soil types. No soil type has all material properties that are needed for a dike structure: sand and gravel are stable, but they are permeable. Peat (*veen*) is quite impermeable, but it is weak and instable and settles when it becomes dry. Clay is ony to a small degree impermeable, but it can easily deform when it is saturated. Many old dikes completely consist of clay, but modern dikes usually have a sand core and are covered by a 0,5 to 1,0 m thick clay layer. Revetment (*dijkbekleding*) of rock, concrete elements or asphalt is used to provide better protection against waves.

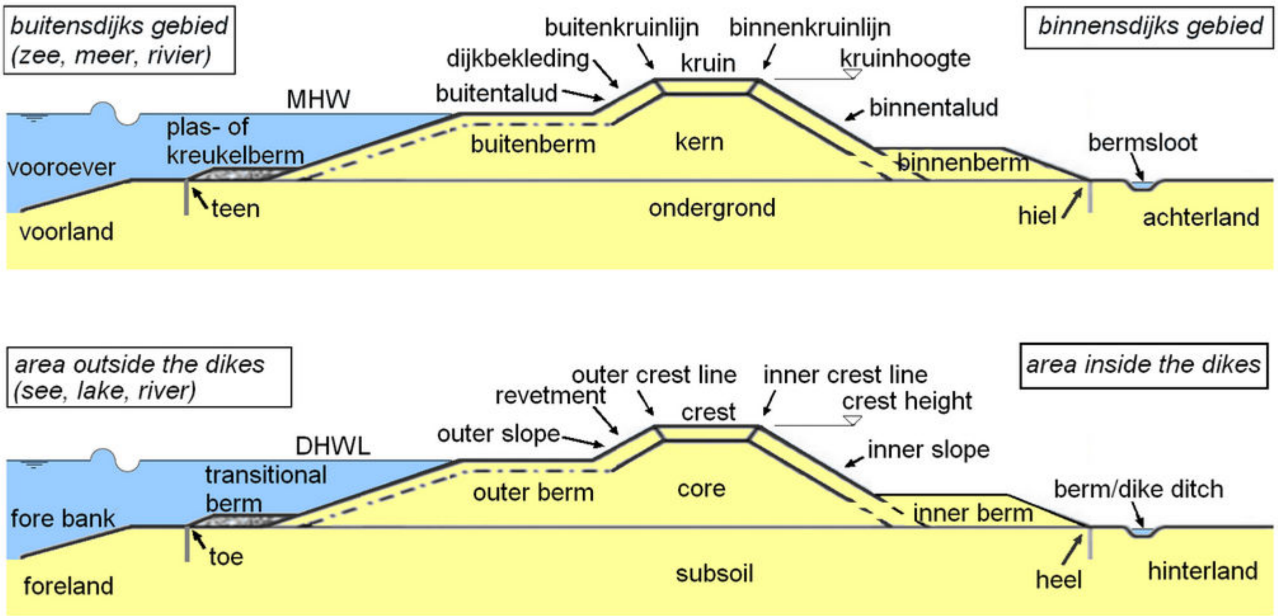


Figure 1.26: The main components of a fictitious dike

1.11 Closure dams

Closure dams are mainly used to close-off tidal inlets, estuaries etc. They can have the following functions:

- reclamation of land (*landaanwinning*);
- shortening the length of sea defence;
- creation of fresh water reservoirs;
- creation of tidal energy basins;
- creation of fixed-level harbour approaches.

The 'Afsluitdijk', for example, closes-off the IJsselmeer, a former sea branch (Figure 1.27). The dam created a non-tidal lake in which storm surges from sea cannot protrude any more. This enabled the creation of land reclamation works like the Noordoostpolder and the Flevopolder. Most estuaries in Zeeland were closed in the last half of the twentieth century, to shorten the coast line. The advantage was that the length of the sea defence was shortened as well, saving costs and providing better protection against storm surges. Fresh-water reservoirs and tidal energy basins are mostly created outside the Netherlands. An example of a closure dam for creating a fixed level for a harbour approach is the Oesterdam, which ensures sufficient depth for shipping between Rotterdam and Antwerp.

Figure 1.27: The 'Afsluitdijk', a closure dam, closing-off the IJsselmeer (TU Delft, 2014)

1.12 Storm-surge barriers

Storm-surge barriers have two opposing primary functions, namely retaining water and allowing passing of ships and/or water (tidal flow, or river discharge). 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 north of Bremen, in the Hollandse IJssel near Capelle aan den IJssel (see picture on the cover of these lecture notes) and in the Nieuwe Waterweg (Figure 1.28). Another interesting example of a storm-surge barrier is the barrier in the Oosterschelde (Figure 1.29). The need to maintain the tidal movement in the Eastern Scheldt estuary for environmental reasons has led to the construction of a storm-surge barrier instead of a closed dam.

Figure 1.28: Storm-surge barrier in the Nieuwe Waterweg (Maeslantkering) (RWS)

Politicians in many places in the world propose building 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 considered necessary, because of the flooding of Bangkok caused by increased river discharges. However, it is obvious that a storm-surge barrier cannot provide defence against high river discharges. On the contrary, the situation would have become worse, if the river could no longer flow freely to the sea. It is therefore essential to analyse the threat with and without a storm surge barrier. Such a study should also lead to operational demands for the storm surge barrier system.

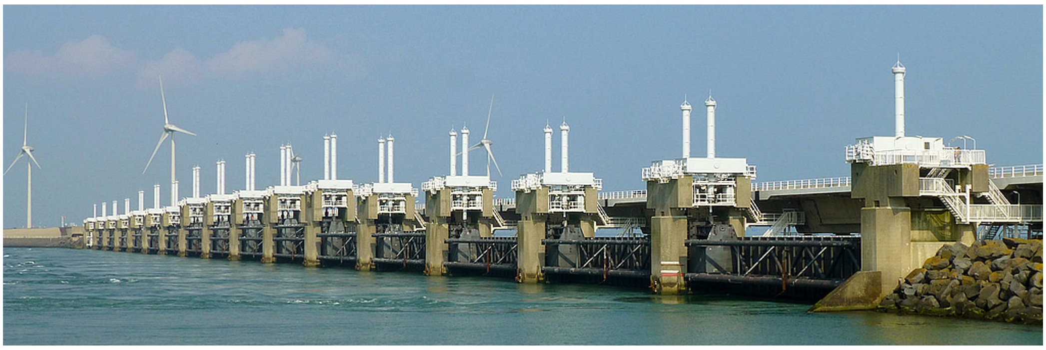


Figure 1.29: Storm-surge barrier in the Oosterschelde (TU Delft, 2014)

1.13 Discharge control structures / compound weirs

The function of discharge control structures, also known as compound weirs or fllexible spillways (*regelwerken*), is flood protection along a river branch from which the water is diverted into another branch where the flood threat is less severe. The discharge control structures are usually built in the forelands (uiterwaarden) of the rivers and their openings can be partly or entirely closed with help of beams (balken) or bulkheads (schotten).

In the Netherlands, there are two such structures (Figure 1.30). The first one, near Hondsbroekse Pleij (Arnhem), was built to regulate the extreme discharge distribution towards the Nederrijn and the (Gelderse) IJssel. The other structure, near Pannerden, regulates the discharge through the Pannerdensch Canal and the Waal river (Figure 1.31). These structures are only functional during high river discharges, when the forelands are flooded. Under normal circumstances, about 1/3 of the Rhine water flows through the Pannerdensch Canal and 2/3 through the Waal. The discharge through the Pannerdensch canal is distributed over the IJssel (1/3) and the Nederrijn (2/3).

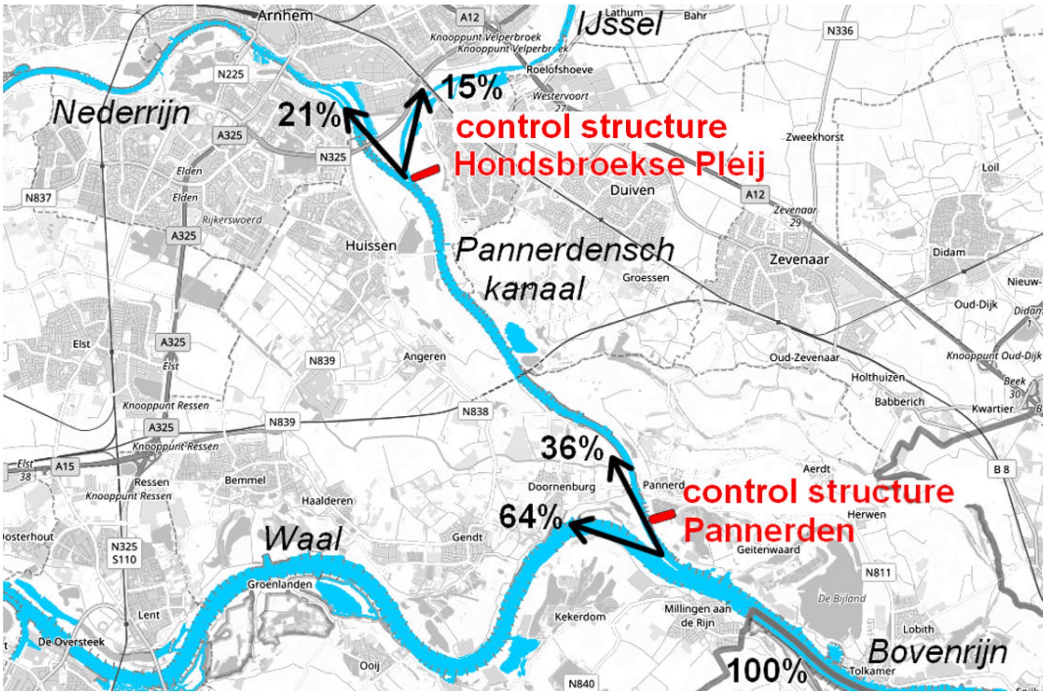


Figure 1.30: River discharge distribution of the Rhine water in the east of the Netherlands under normal circumstances (modified from OpenStreetMap)

Figure 1.31: River discharge control structure near Pannerden (TU Delft, 2015)

1.14 Pumping stations

Pumping stations (*gemalen*) are designed to move 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. Figures 1.32 and 1.33 show the renovated (1998) and enlarged Rozema pumping station in Termunterzijl. The pumping station discharges water from the polder through a culvert in the dike to the Wadden Sea. The four pumps have a total capacity of 2700 m^3/min. Construction of the pump station building alone cost almost € 20 million and the mechanical parts and pumps cost an additional € 20 million (1998 prices).

Figure 1.32: Pumping station Rozema near Termunterzijl

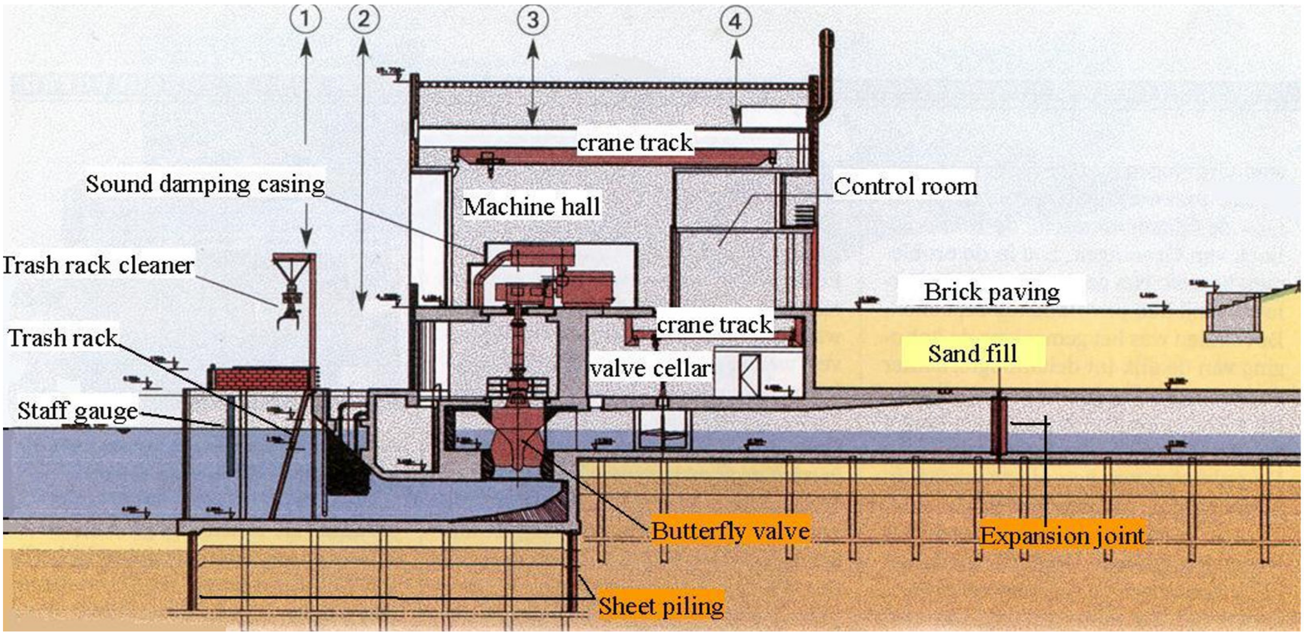


Figure 1.33: Cross-section pumping station Rozema and culvert through the dike

1.15 Dewatering sluices

Dewatering sluices (*uitwateringssluizen*) are designed to discharge an excess of water in the polder reservoir or belt canal (*boezemkanaal*) into a river or the sea. They are part of the water defence system. Accordingly, the functions of a dewatering sluice are:

- to discharge water through the water retaining or flood defence structure;
- to retain water during high outer water level;
- to separate fresh water from salt water.

Water can be discharged through open channels or a closed tubes (culverts, *duikers* in Dutch). The water can be retained by various types of gates. In the past, mitre gates have generally been used in the Netherlands. 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 more accurately control the water level in the polder. This requires controlled discharge of water. As a consequence, gates are chosen that can be opened and closed in a controlled procedure and can retain water in both directions.

A simplified example of a culvert-type dewatering sluice with mitre gates in a dike is presented in Figure 1.34.

Dewatering sluices 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 that case, the structure is called an 'intake sluice' (*inlaatsluis*). They are quite common in irrigation systems in arid countries.

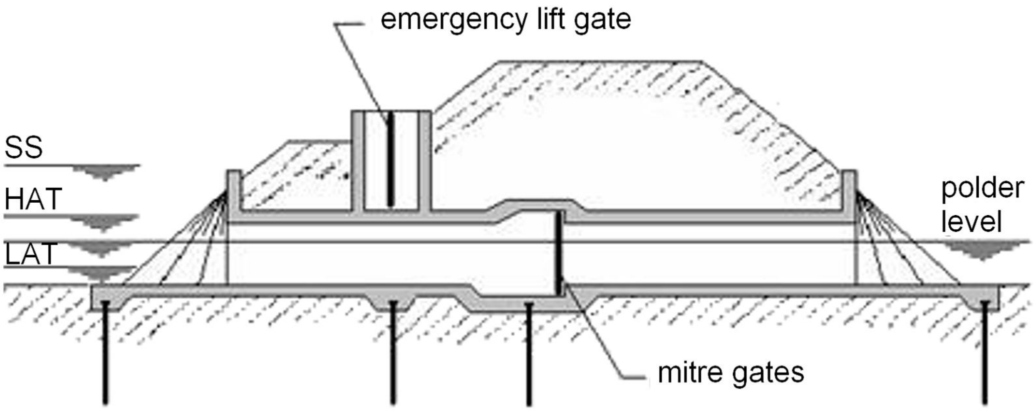


Figure 1.34: Cross-section of a culvert-type dewatering sluice (infra-wiki.nl)

1.16 Discharge sluices

Discharge sluices (*spuisluizen*) are hydraulic structures as part of a dam in a river mouth to allow discharge under natural flow and in meanwhile retain outer water. Gates are closed if the outer water level becomes higher than the inner water level, to prevent water flowing in the wrong direction. In this way, it protects against flooding and salt intrusion. Examples of such controlled discharge gates can be found in the Afsluitdijk near Den Oever and near Kornwerderzand, in the dam of the Lauwersmeer and in the Haringvlietdam (Figure 1.35).

Figure 1.35: Haringvliet discharge sluices (TU Delft, 2014)

1.17 Flushing sluices

A flushing sluice, or scouring sluice (*spoelsluis*), is designed to flush a waterway, canal system or a harbour to remove sediment or contamination. They were often used to flush the canals in old cities. Flushing sluices can be used in reservoirs dams as well, 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.

1.18 Inundation sluices

Inundation sluices (*inundatiesluizen*) are intended to deliberately flood an area of land. To make this successful, the inundatal land area is enclosed by dikes or other flood defences. This can be useful in military operations in the defence of land to prevent a hostile advance of ground troops. Inundated areas are not well accessible on foot or by riding vehicles, nor by boats. Many defensive inundation sluices in the Netherlands can be found in the *'Stelling van Amsterdam'* and in the *'Nieuwe Hollandse Waterlinie'*. The purpose of inundation can also be offensive, in which case the enemy is driven out of the inundated area. Another purpose of inundation sluices is to temporarily enlarge the size of a retention area. This will lower the water level in an adjacent river and, in this way, prevent the breaching of dikes during high river discharges. Finally, inundation sluices can be used in agriculture to cover land once per year during six to eight weeks, to fight eelworms (*aaltjes*), fungi (*schimmels*) or weed (*onkruid*). It reduces the use of pesticides.

Inundation sluices can be operated with help of lifting gates (*hefdeuren*) or mitre gates (*puntdeuren*), but sometimes also with vane gates (*waaierdeuren*) that can be opened with only the use of hydrostatic pressure, in the direction of the highest water level (Figure 1.36).

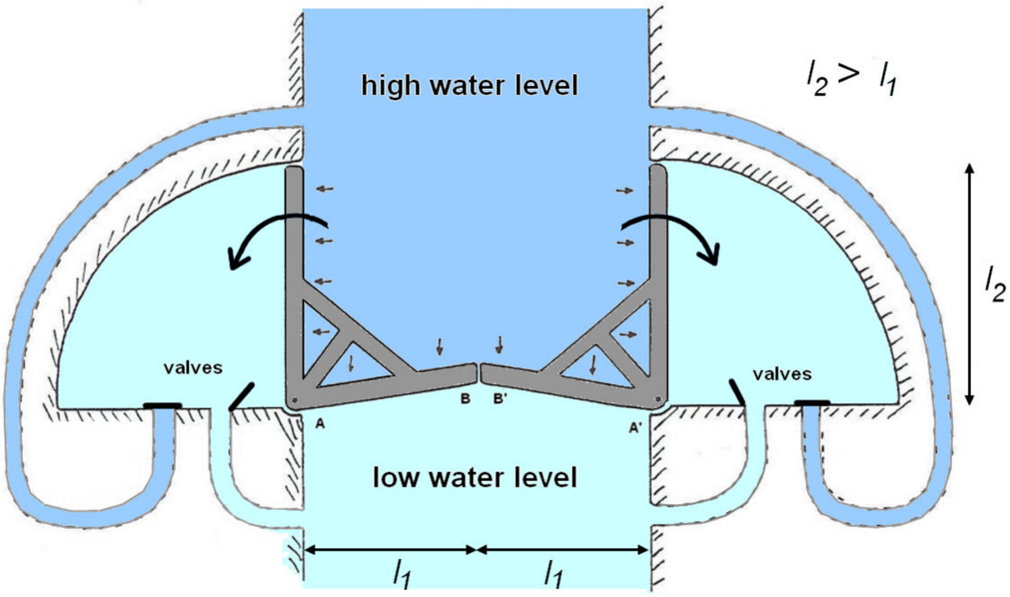


Figure 1.36: Principle of vane gates (modified from website Historisch Diest)

Related to inundation sluices are de-inundation sluices. They have a function opposed to inundation sluices: they are used to dewater an area of land when it is not necessary any more to have it inundated. They are usually located at the lower side of the area, so the water can flow out freely, without the need of pumping stations.

1.19 Reservoir dams

The function of reservoir dams (*stuwdammen*) is to head up the water level in a river, to create a basin for the purpose of irrigation or electricity generation. The dam causes a difference in water level on both sides of the dam (= 'water head difference'). The water pressure acting on the dam is diverted 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 (Figure 1.37).

In an area with loose materials and with rocks unable to provide great shearing strength, a dam made of soil and rubble (Figure 1.38) or a concrete gravity dam will be chosen, or possibly a buttress dam. An earth fill dam is made impermeable by a core of clay or bentonite, which is extended below ground level to reduce seepage or piping through and beneath the dam. The outer slope is very gentle and the embankment is drained through a drainage pipe system inside the dam.

Figure 1.37: 'El Atazar' Concrete arch dam near Madrid, Spain (Wikimedia Commons, 2014)

Figure 1.38: Gravity dam near Tsonevo, Bulgaria (TU Delft, 2011)

Intake towers

Intake towers (also called outlet towers) (*inlaten*) are used to capture water from reservoirs and convey it further to a hydroelectric or water-treatment plant (Figure 1.39). They usually consist of a vertical tubular structure with one or more openings. Intake towers are intended for the reservoir's regular operation, conveying clean, debris-free water.

Figure 1.39: Intake tower of the Tsonevo reservoir dam (TU Delft, 2011)

Spillways

A reservoir dam is usually equipped with one or more spillways (*afvoerkanalen*) to control the water level in the reservoir, usually to prevent overloading of the dam during emergency situations (Figure 1.40). The reservoir lake can be seen in the upper left corner of the picture. Spillways can be equipped with movable closure devices, to regulate to a certain extent the water level in the reservoir.

Figure 1.40: Spillway near the Tsonevo reservoir dam in Bulgaria, during the dry season (TU Delft, 2011)

1.20 Piers

A pier (*pijler*) is not a hydraulic-engineering structure in itself, but is a hydraulic-engineering element belonging to a civil engineering work, for instance a bridge, barrier, weir or barrier. 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.

This means that the horizontal and vertical loads are transmitted by the pier 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.

The most common pier is the bridge pier (Figure 1.41). 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 base of the foundation is subjected to compression force. This generally results in piers of large weight. To be able to bear this weight, it is usually 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 flood defence structures can be found in the Oosterschelde Storm-surge Barrier and the Hartel Barrier.

The most significant loads on a pier are:

1. Vertically:
   - load from the (main) superstructure: dead weight of the structure and variable loads like traffic
   - dead or self-weight 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.

The shape of the pier is in many cases determined by its primary function. For instance, the shape of a bridge

(a) Brooklyn bridge (Wikimedia Commons) (b) HSL Moerdijk bridge (Beeldbank RWS)

Figure 1.41: Bridge piers; masonry pier (left), concrete piers (right)

pier will be different from a pier of a storm surge barrier (compare Figure 1.42 with Figure 1.43). The bridge piers mainly transfer vertical loads: self-weight is the governing load (*maatgevende belasting*), while the piers in a water retaining structure, such as a storm surge barrier, although the self-weight is definitely not negligible, first of all have to be able to resist the water pressure in horizontal direction.

Figure 1.42: Confederation bridge - Canada (CBC news)

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 1.42. The cone shape forces the ice to move upwards to make it break. 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 occurs in only one direction. Such a pier will be streamlined to reduce the loads on the pier caused by water flow.

For structures with a large number of piers, it could be a good idea to prefabricate the piers in a building dock or construction dock (*bouwput, bouwdok*), see Figure 1.43. The big advantage of prefabrication is that it reduces the number of (relatively inaccessible) building sites in the water, reducing the construction time. However, special equipment is required to place prefabricated piers in their final location (Figure 1.44).

Scour is an important aspect to take into account for the design of piers. The presence of a pier causes a local increase of turbulence and sometimes an increase in average flow speed as well. This will increase the erosion locally and the foot of the pier, the foundation, may get exposed. It will often be necessary to construct a scour

Figure 1.43: Precast pier for the Oosterschelde Storm Surge Barrier in the construction dock (Beeldbank RWS)

or bottom protection to prevent undermining of the entire structure.

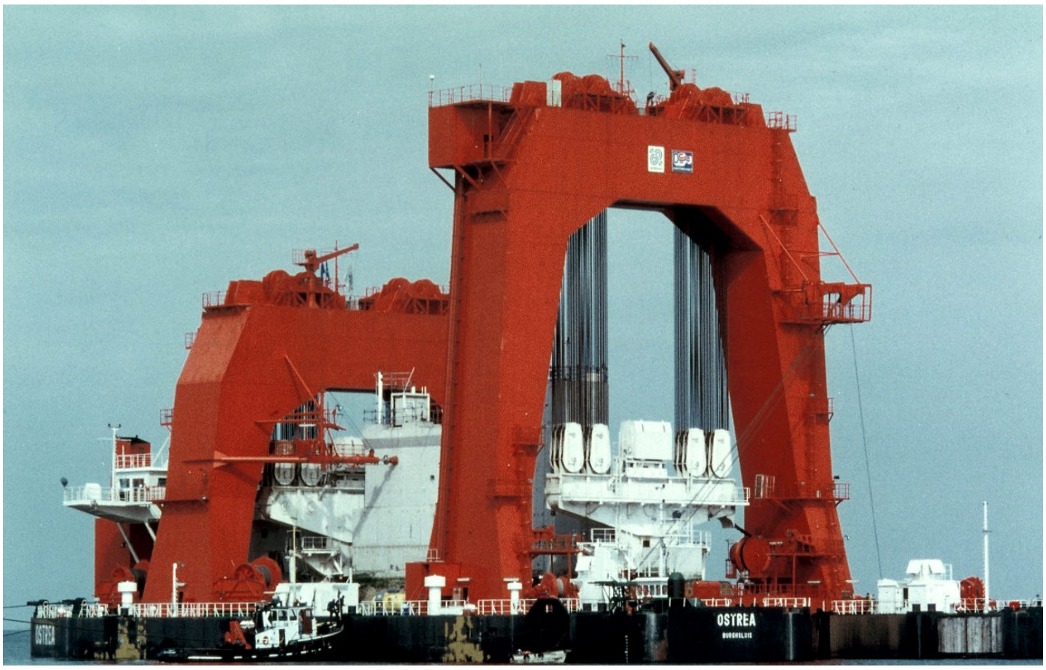


Figure 1.44: Positioning of piers for the Oosterschelde Storm Surge Barrier using the crane ship Ostrea (Beeldbank RWS)

1.21 Tunnels

1.21.1 Purpose and classification

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 road traffic (cars, trucks, etc.);
- tunnel for trains;
- tunnel for ships;
- tunnel for water transport;
- tunnel for pipes and cables.

It is as well possible to classify a tunnel by the method of the construction of the tunnel, as:

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

This list goes 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. For passing buildings or large cables and pipes under roads, bored tunnels are often less expensive.

1.21.2 In situ tunnels

In situ tunnels can be sub-divided into two categories: tunnels constructed in building pits with natural slopes and tunnels in cofferdams, which have walls. In the cofferdam method, the first action is to install the wall elements into the ground. These walls can be anchored or unanchored sheet pile walls, of diaphragm walls. Being only excavation between walls, the all-over construction area will be smaller than a traditional building pit with natural slopes. Depending of the situation, a watertight floor will be constructed after excavation. If the walls are driven 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 completed, before a ground filling can be started (Figure 1.45).

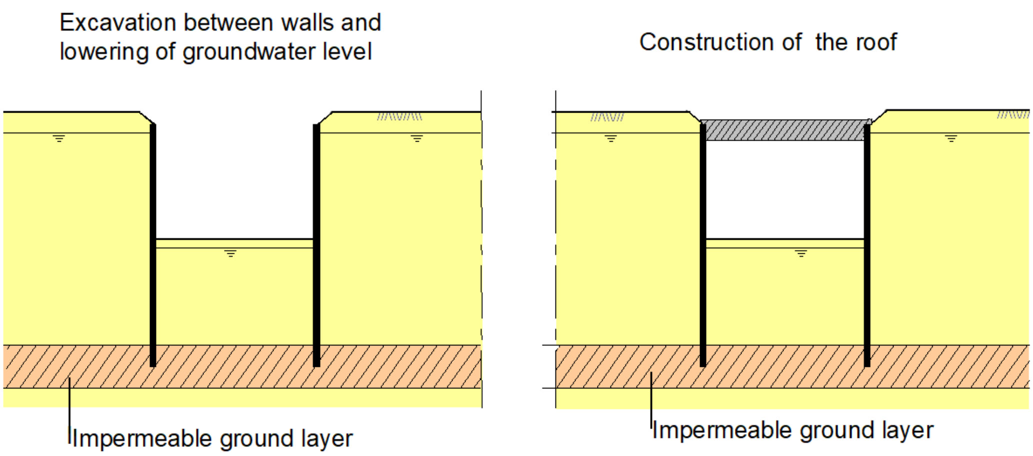


Figure 1.45: Cofferdam tunnelling method

Examples of in situ tunnels are the Velser tunnel and the Schiphol tunnel. The Velser tunnel was built at a crossing of a road and a waterway (Figure 1.46). To let vessels pass during construction, the tunnel was built in phases, constantly leaving 50% of the width of the fairway open for shipping.

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

1.21.3 Cut-and-cover tunnels

The order of construction in urban areas usually is adapted, to cause as little disturbance as possible for 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 (Figure 1.47). The biggest advantage of this

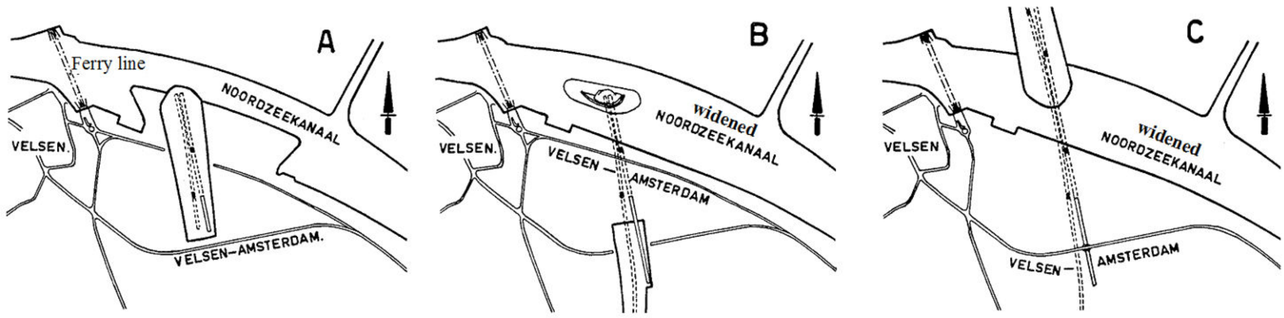


Figure 1.46: Phased construction of the Velser tunnel

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 air in the construction area, dangerous work and expensive, or to seal the area with an arch of grout. The latter has proved not to be as successful as expected in a number of projects.

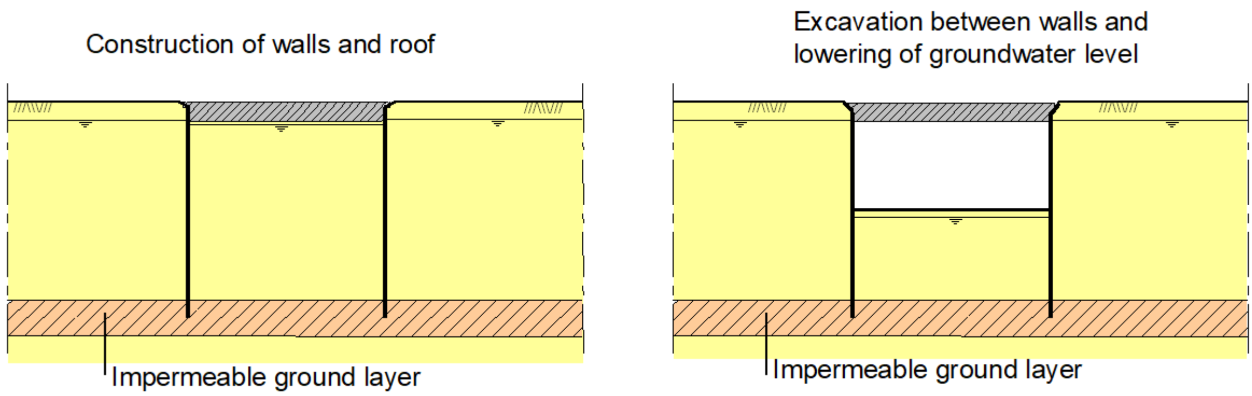


Figure 1.47: Cut and cover tunneling method

1.21.4 Immersed tunnels

At the crossing of a road and a waterway, the part of the tunnel under the waterway can be immersed into place. The tunnel is prefabricated in elements in a building dock and the inconvenience to shipping is significantly reduced compared to a tunnel built in situ. The tunnel elements are equipped with a bulkhead, allowing them to remain afloat without ballast and they can be transported by water. The elements are transported over water to the location of the tunnel, carefully positioned (Figure 1.48) and vertically immersed to the bottom of a trench dredged before.

The elements are, at one end, provided with a rubber seal (Gina profile). During the immersion process the elements are placed together, with the rubber seal connecting the two elements. While pumping out the water between the two joined elements, they are pressed against each other by the water pressure on the other ends of the elements, creating a watertight seal between the elements. Omega profiles are applied to create a second seal between the tunnel elements for water-tightness.

The vertical positioning of the tunnel elements is done using concrete slabs, which are placed beforehand. When the elements are being placed, sand is sprayed under 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 elements will be immersed down on the foundation.

This method of tunnel construction cannot be applied in areas where the tunnel is located above water level; in that case the tunnel has to be constructed in situ. The structure between the entry ramp and the immersed

Figure 1.48: Positioning of a tunnel element for the Coatzacoalcos Tunnel Mexico (Volker Wessels, 2013)

elements is called a transitional element. In the Netherlands, this method of immersing is a well tried and tested method and often applied. For the main stages of transport and immersion, see Figure 1.49.

1.21.5 Bored tunnels

Bored tunnels are widely used in very cohesive grounds and in rock. In Japan, Germany, England and recently also in the Netherlands, bored tunnels have also 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 Westerschelde tunnel.

The advantages of bored tunnels are:

- little or no space is required for the building site;
- very little inconvenience at the site.

The disadvantages of bored tunnels are:

- if the bore machine 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;
- large logistical problems (all the soil and the tunnel elements 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’.

1.21.6 Subsided tunnels (pneumatic caisson technology)

Subsided tunnels are tunnels created with help of caissons that have a work chamber at the bottom side in which the soil is excavated from within (Figure 1.51). The soil under the edges of the caissons collapses because of an inbalance of weight and bearing capacity. In this way, by continuous excavation, the caissons subside ever deeper into the soil. Compressed air in the work chamber keeps out the ground water to enable excavation in dry work conditions. Working under pressurised air can cause health problems, for which reason restrictions apply to the periods during which people can work under these pressures, and gradual decompression has to be applied in decompression tanks to let the people slowly adjust to atmospheric pressure, thereby avoiding severe injuries. For depths of about 35 m or more, human beings should not be exposed to the high air pressures and automated excavation is preferred.

A row of subsided caissons should be connected to each other by making the connections water-tight and by removing the head walls of each caisson. More explanation is given in the lecture notes of caissons (TU Delft, BSc-course CTB3355 ‘Hydraulic Structures’).

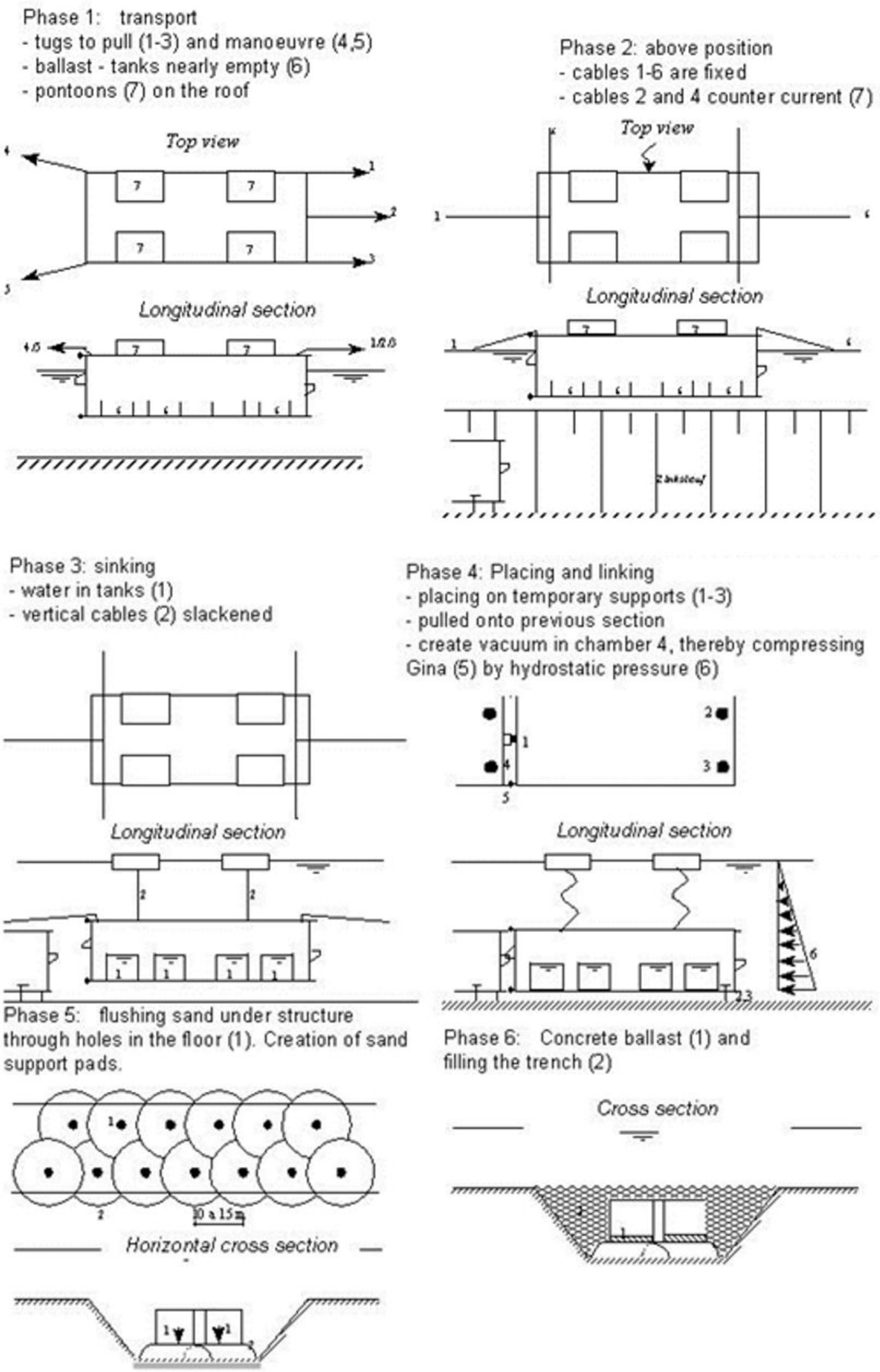


Figure 1.49: Transport and positioning of an immersed tunnel element

Figure 1.50: Break-through of a TBM in a head wall for a train tunnel, East London (www.ianvisits.co.uk)

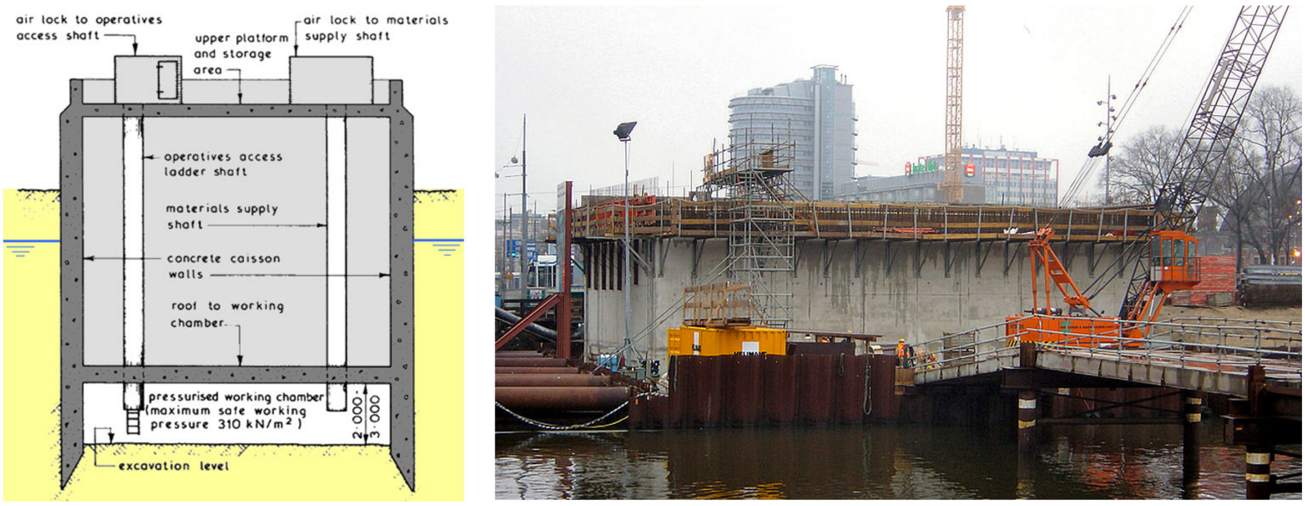


Figure 1.51: Schematic cross-section of a pneumatic caisson (left) and a picture of such a caisson before subsiding (right; Amsterdam – picture: TU Delft, 2008)

1.22 Aqueducts

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 aqueducts lead the water in a trough over the other infrastructure. In the case of a tunnel the crossing infrastructure is led under the river in a tube. See for example Figure 1.52. Of course, aqueducts are also used to cross valleys for water supply or shipping.

If the waterway is less wide than the crossing road, an aqueduct could be preferred over a tunnel. Examples of aqueducts in the Netherlands are:

- aqueduct near Leiden (the crossing between the Haarlemmer ring canal and the A4 highroad);
- aqueduct near Zevenhuizen;
- Gouwe aqueduct near Gouda (crossing of the A12 highroad).

Figure 1.52: Gouwe aqueduct (RSK1992, flickr.com)

1.23 Naviducts

A naviduct is a special case of an aqueduct, only here a navigation lock rather than just a canal crosses a road. An example of an aqueduct can be found near Enkhuizen, where a navigation lock crosses a road as part of the dam between the IJsselmeer and the Markermeer (the Houtribdijk) (Figure 1.53).

The Enkhuizen naviduct crosses a specially designed polder, having an artificially managed ground water table at NAP - 12,0 m. The polder has a length of 500 m and a width of 160 m and the crest level of the surrounding dikes is at NAP + 6,0 m. The road under the naviduct reaches a depth of NAP - 10,80 m. Both lock chambers are designed for CEMT class Va vessels and have a length of 125 m, and a width of 12,5 m.

Figure 1.53: Naviduct near Enkhuizen (TU Delft, 2015)

1.24 Culverts and syphons

In case of a tunnel and an aqueduct, the waterway is situated above the level of the crossing infrastructure. In many cases, however, it can be more attractive to lead the waterway under the road. This can be achieved for instance by a fixed or moveable bridge, but can also be achieved with a culvert. A culvert is in fact a tunnel under a road through which the water can flow, thereby crossing the road.

Another method to discharge water through a water retaining structure is to use a syphon. In this case, water is not transported through the water-retaining structure, but over its top (Figure 1.54). A big advantage of this system is that it does not require a closure mechanism. Filling the syphon 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 syphon must be situated far below water level in order to prevent that free air can penetrate the syphon system. This method has, for example, been applied in Bergen op Zoom.

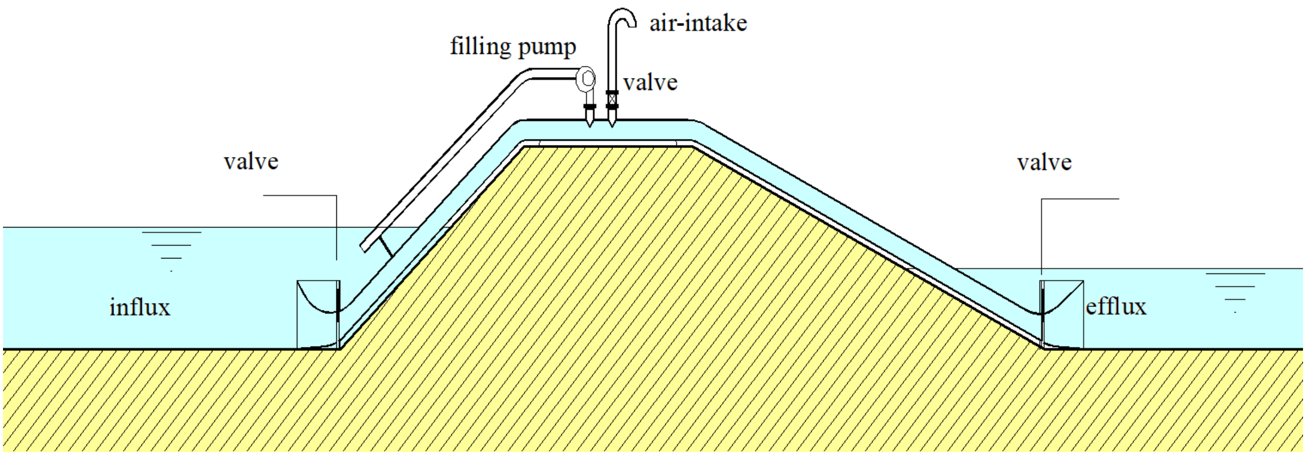


Figure 1.54: Schematic representation of a syphon (source unknown)

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 cannot be used. In that case, the use of a pumping station is the only option.

It is also possible that one waterway crosses another waterway, or pipes, by going beneath it through kind of a culvert (Figure 1.55). Such a structure is called an inverted syphon (*grondduiker*).

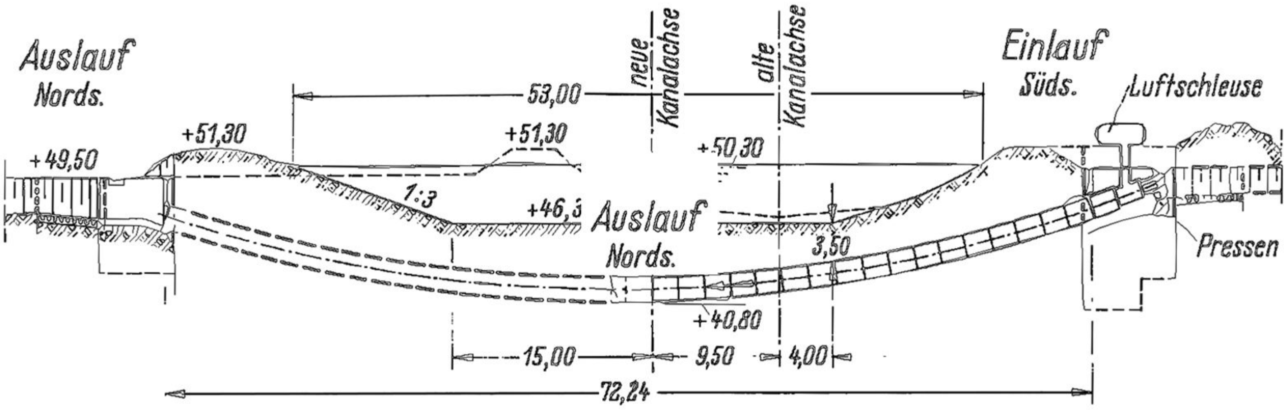


Figure 1.55: Longitudinal section of an inverted syphon crossing another waterway and a road (source unknown)

1.25 Dry docks and floating docks

Dry docks and floating docks allow ships to be repaired 'in the dry' without putting the ship on land (Figure 1.56). A dry 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. 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 very much resembles a navigation lock, however, it has only one entrance.

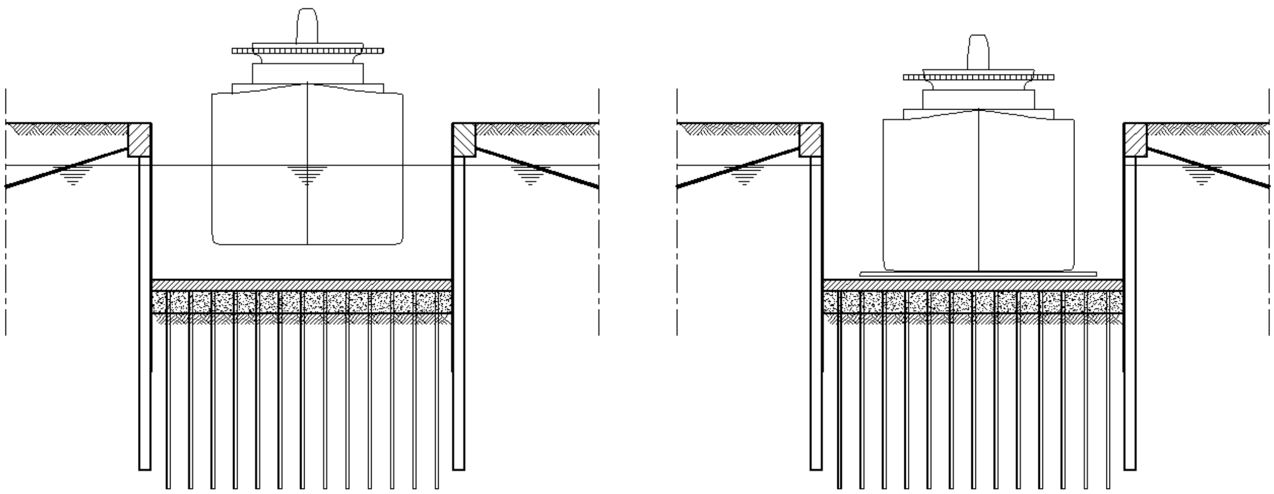


Figure 1.56: Dry dock (source unknown)

Another type of dock is the floating dock (Figures 1.57 and 1.58). 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 consist of one of many types of gates used in navigation locks.

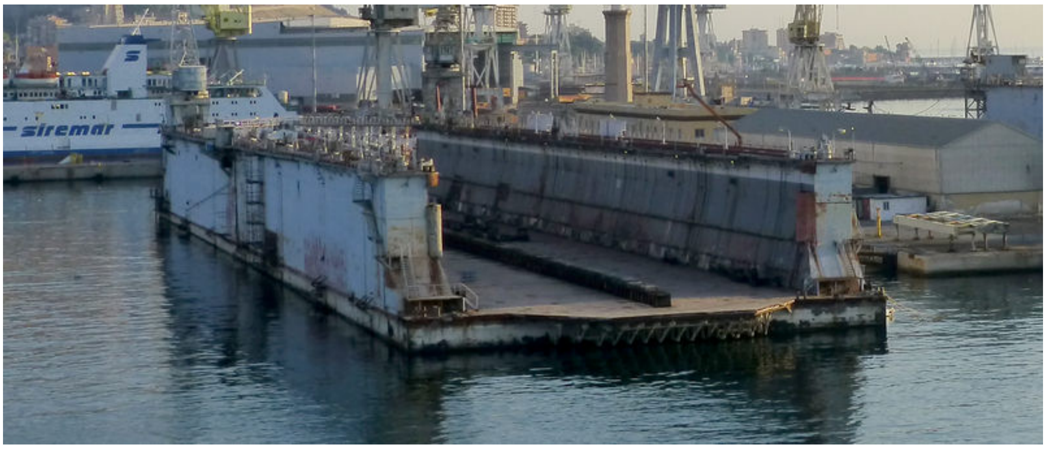


Figure 1.57: Floating dock in the port of Palermo (TU Delft, 2015)

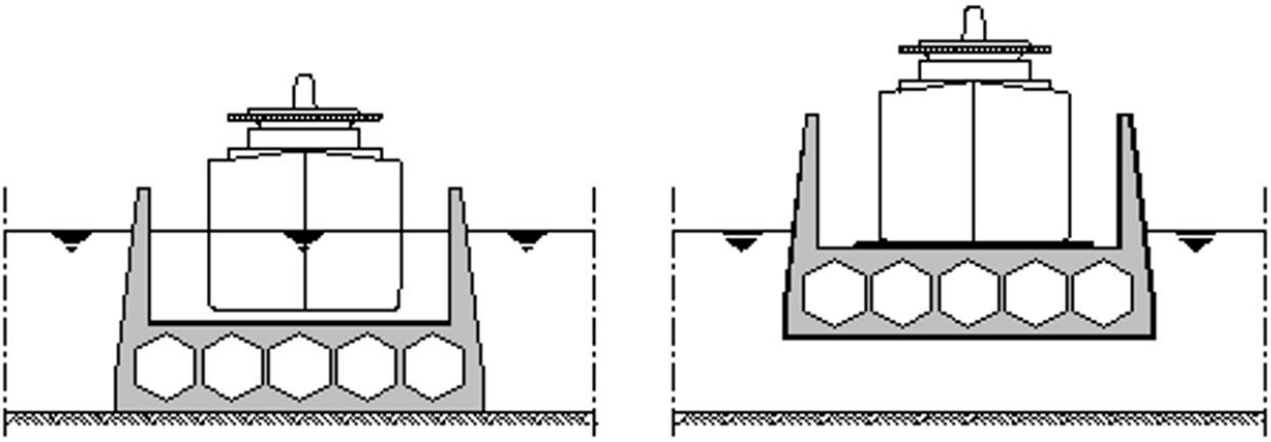


Figure 1.58: Floating dock cross-section(source unknown)

1.26 Buidling pits, cofferdams and construction docks

The primary function of building pits, cofferdams and building docks is to provide a dry workspace for in situ construction of prefabrication of structures or structural elements. This is usually achieved by retaining soil, groundwater and/or surface water, in horizontal and vertical direction, to create dry space to work.

1.26.1 Building pits

A building pit (*bouwput*) is an excavation with natural slopes, in which, if necessary, the groundwater level is lowered until below the ground level of the building pit. A building pits is situated on the definitive site of the structure (Figure 1.59). The purpose of a building pit is to create dry land at the foundation level of the structure. A building pit needs relatively much 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. In addition, 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. 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.

1.26.2 Cofferdams

In many cases there is no space for a traditional building pit or pumping is not allowed. In these cases, a vertical walled building pit, a cofferdam (= braced excavation pit) (*bouwkuip*) can be used. Often, the walls

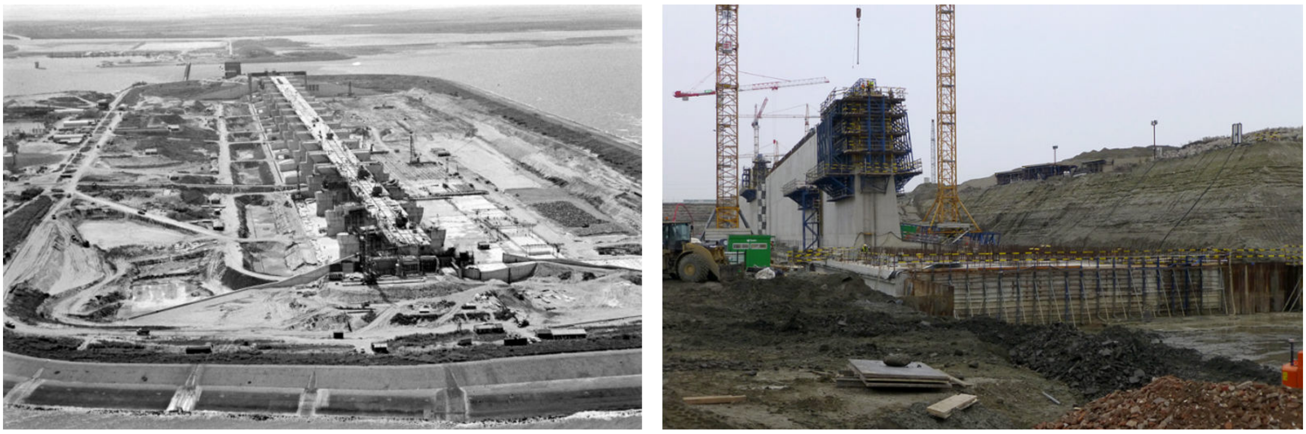


Figure 1.59: Building pits in the waterway (Haringvliet sluices) and on land (Deurganckdok lock) (ANP, 1964 - TU Delft, 2013)

have to be horizontally supported. This can often be done by applying a framework of wales (*gordingen*) and struts (*stempels*) in between the walls. Alternatively, the walls can be anchored in the soil behind the walls facing each other, leaving the space in the cofferdam free for construction activities. Sometimes, it is possible to integrate the cofferdam in the final structure. 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 an impermeable ground layer (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 sufficiently thick and heavy 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 fibres. In this way, the underwater floor can act as a full construction floor (like for the Julianasluis near Gouda), however, this definitely is not common practice.

'Travelling platforms' or 'traverses' can be used to drive piles before casting the underwater concrete floor, while there is still water in the cofferdam. In later construction phases, traverses can be used for lifting operations (Figure 1.60), although this is often done by cranes form the sides of the cofferdam.

Figure 1.60: Cofferdam with a framework of wales and struts, and a traverse for the road tunnel in Maastricht (Avenue2)

1.26.3 Construction docks

For the construction of elements for an immersed 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 over water once they are finished. This type of building pit is called a ‘construction dock’ or ‘building dock’ (*bouwdok*). See Figure 1.61 for an example.

Figure 1.61: Tunnel elements in the construction dock near Barendrecht (Bas Romeijn, 2018)

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 is used as a closing element or gate of the construction dock.

It is typical for building docks that the elements remain only temporarily on the construction site. 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 that a large amount of space has to be reserved for a building dock, which usually goes with large expenses. Especially if a large number of elements has to be built, it may be more economical to search for alternatives. Tunnel elements, for instance, can also be built on a slope or on a conveyer belt after which they can directly be 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.

1.26.4 Roads in continuous cut, approach roads

The objective of constructing a road in a continuous cut (*verdiepte bak*) is:

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

Approaches to tunnels, rail and/or road tunnels have much 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 the top of the relevant flood defence structure and the bottom level of the tunnel. Approaches are sloping down or up. Approaches or entry ramps do not have roofs, see Figure 1.62, however, they may have several permanent struts (*stempels*) between the opposing walls in the deeper parts.

Because the road is below ground level, it is necessary to create a watertight structure to 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 or without anchors. If necessary, a watertight floor can be constructed between the diaphragm walls. Figure 1.63 shows several 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 1.64 shows an underwater concrete floor after dewatering of the building pit. The pile heads were driven before casting the underwater concrete and have been jacked for integration in the structural concrete.

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.

Figure 1.62: Approach to the Kiltunnel, Dordrecht (Wikimedia Commons, 2016)

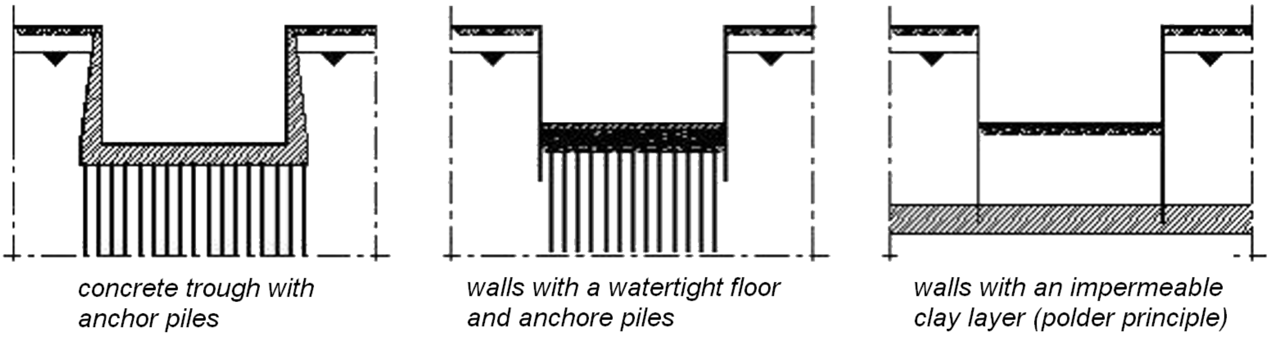


Figure 1.63: Rigid or inflexible structures (source unknown)

Figure 1.64: Underwater concrete and tension piles in the approach to the railroad tunnel near Kampen (Hanzelijn) (TU Delft, 2007)

The soil being placed on top the membrane can ensure the stability of the membrane and prevent damage due to loads. Figure 1.65 shows three examples.

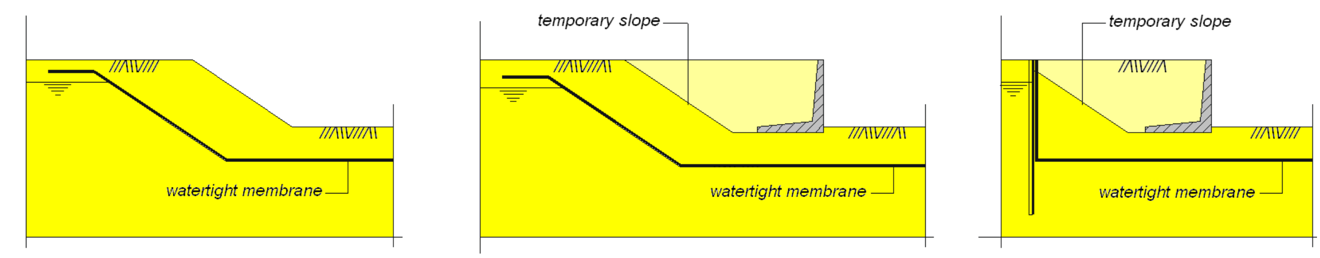


Figure 1.65: Various applications of membrane structures (TU Delft)

1.27 Artificial islands

Artificial islands create a surface above the water for, amongst others, supporting an oilrig or serving as a foundation of a lighthouse, windmill or radio mast. The largest artificial islands are used for harbour expansion or to create land for industrial zones, airports, recreation (Figure 1.66).

Figure 1.66: Artificial islands: Chek Lap Kok near Hong Kong (left) – World at Dubai (right) (sources unknown)

Artificial islands can be constructed using soil, structures or combinations. The choice mainly depends on settlement sensitivity of the subsoil and the depth of the ground layer having sufficient bearing capacity below the water surface. Huge quantities of soil are required for islands in deep water, to create long and gentle slopes that are sufficiently stable. 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 bridges the height difference between the top of the island and the foundation level. Examples are circular walls and caissons ballasted with soil, placed in a square. The most important functions of the circumfering structure (*omringende constructie*) are:

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

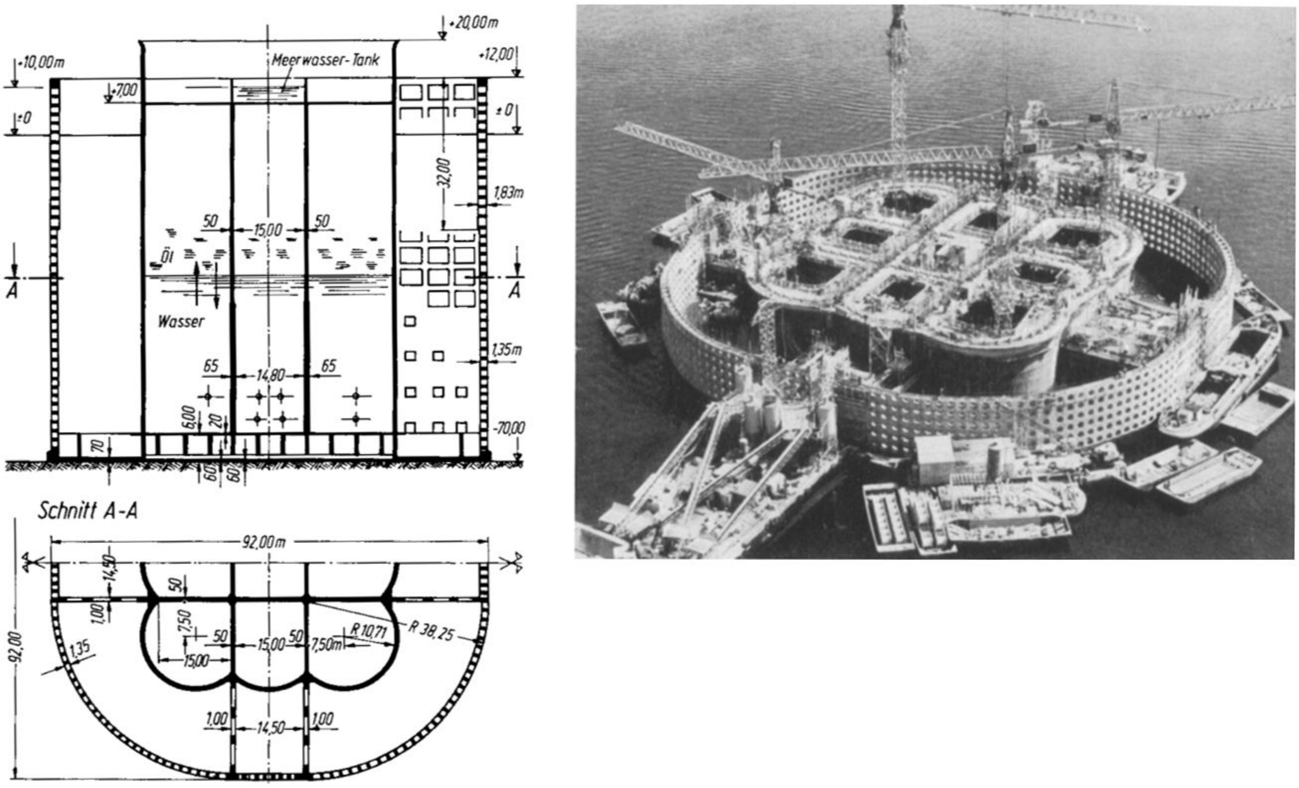


Figure 1.67: Artificial island – Ekofisk, constructed 1972-1973 using concrete caissons (source unknown)

1.28 Soil retaining structures

Soil retaining structures as such are often not hydraulic structures, but they only form a part of hydraulic structures, like the lock chamber walls that are part of a navigation lock. They generally retain water as well, i.e. resist water flow caused by 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;
- piled wall structures;
- composite structures.

1.28.1 Gravity structures

In general, a gravity structure has a shallow foundation (*fundering op staal*). It retains soil and water and transfers the loads to the foundation by means of a compression force and a friction force, both acting on the foundation surface, and both are the result of gravity or self-weight of the structure, and possibly ballast. It requires sufficient bearing capacity of the soil beneath the structure. It is quite common to improve the soil of a weak top layer, by means of dredging or excavation, followed by filing the trench by coarse sand and/or stone filter layers. The loads behind the structure are mainly supported by the soil and partly by the structure, making use of the friction between the soil and the structure.

Several gravity structures have pile foundations for the provision of the 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.

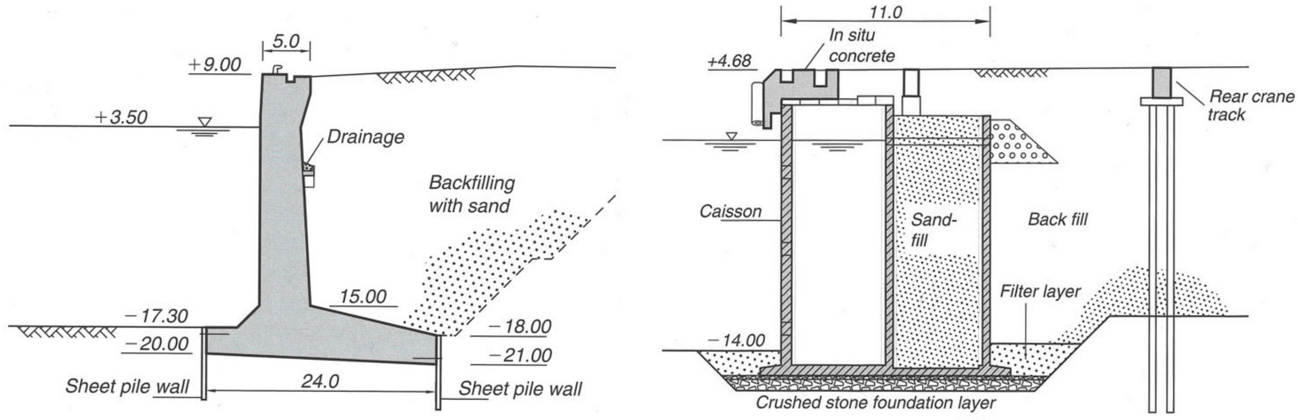


Figure 1.68: Two examples of gravity walls - shallow foundation (Quay walls, 2014)

Examples of gravity structures are:

- standard (floating) caissons;
- pneumatic caissons;
- non-reinforced concrete or masonry walls;
- block walls;
- L-shaped walls;
- cofferdams (*kistdammen*).

Standard caissons can be used as piers, breakwaters, quay walls, closing elements for flood defences, etc. (Figure 1.69). In addition, caissons can be used as tunnel elements (see Section 1.21.4). Caissons are generally pre-fabricated in construction docks, floated, and then transported over water to the final site and immersed to the water bed by ballasting. For quay wall construction, once the caissons have been immersed, the ground level behind the line of caissons can be raised to the required port terminal level.

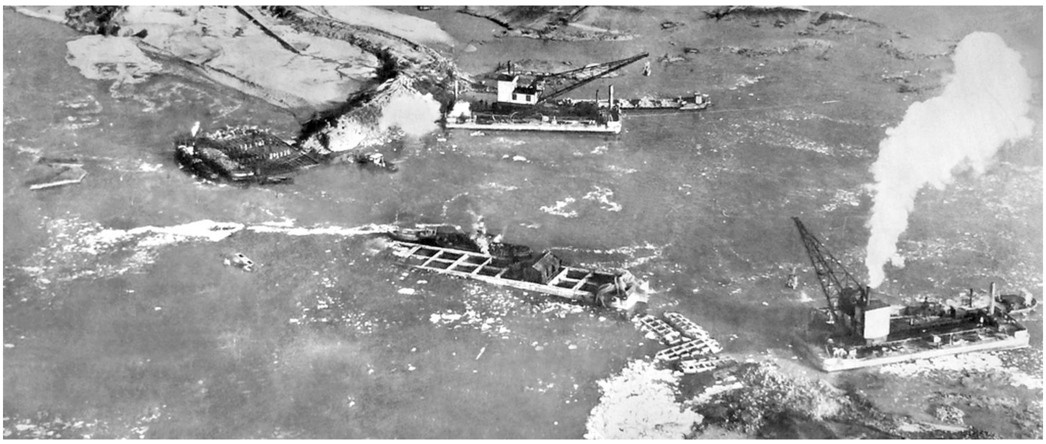


Figure 1.69: An immersed caisson (in the middle of the picture) to close a breach in the sea dike near Vlissingen, Zeeland (RAF, 1944)

Caissons used for the construction of closure dams, can be used to transport building materials 'over the head', after being immersed and positioned. The sand ballast 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 consisting of a cover or roof slab, all the utilities, fenders and bollards for quays, is finished using the overhead construction road.

Pneumatic caissons are built on site at ground level in dry circumstances, and are immersed to the required depth by removing the soil below the structure, effectively undermining the caisson, the self-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. Over-pressurised air prevents the intrusion of ground water in the work chamber.

Non-reinforced concrete and masonry walls (ongewapend betonnen en metselwerk muren) can be used as retaining structures, if the required height is not too high. Obviously, the availability of materials plays an important role 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 with an increasing the height (proportional to the square of the height).

Masonry walls and massive non-reinforced concrete walls are 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 definitely result in cracks in the wall. Partly for this reason vertical expansion joints (*dilatatievoegen*) are constructed in the wall allowing a certain amount of differential settlement. The wall panel between the expansion joints is then able to rotate a little. In addition, expansion joints prevent cracking caused by temperature differences.

Block walls, containing blocks having the size of man or larger, are often used as quay walls. Use above the ground, for instance for soil storages, is quite possible, see for instance 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 end of the completed part. The additional soil behind the quay can be dumped from the land side. Since a block wall has many joints, groundwater will be able to seep through it.

Erosion of the retained soil can be prevented by a granular filter or a filter cloth behind the wall. In this case, the block wall does not retain groundwater flow, but only prevents extrusion of sand particles, although a water level difference may exist or develop due to changing permeability. The wall retains water as well in case of complete impermeability, 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 cause transport problems.

A block wall constructed using the 'sloping bond' method should be considered as a special type of wall (Figure 1.70). The blocks are placed against each other at an angle to spread the load better through friction in the joints and to redistribute the load on the subsoil in case of locally larger settlement. This solves settlement problems due to weak soil layers. 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 forward into position.

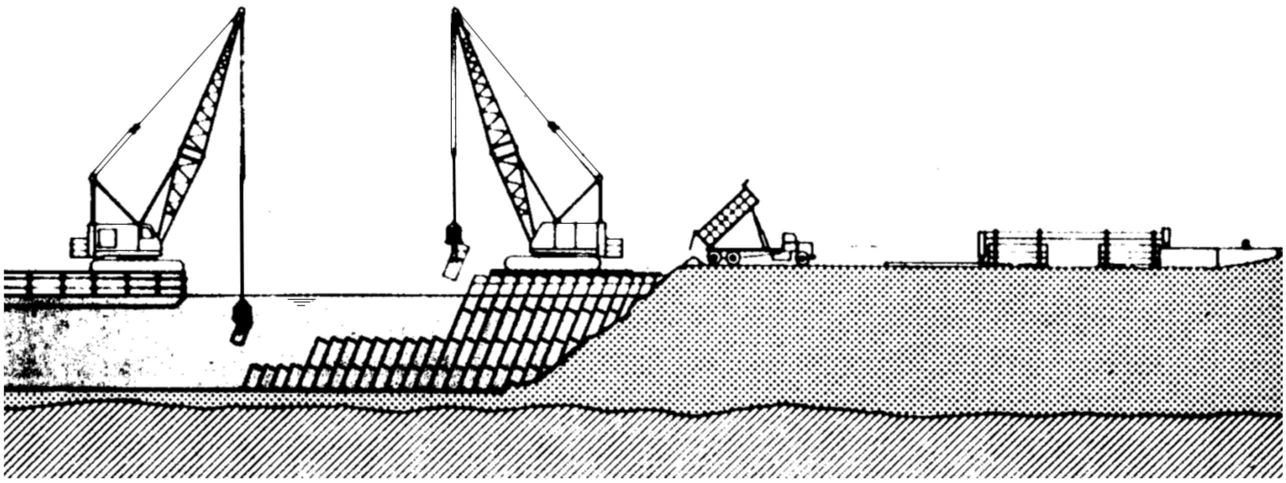


Figure 1.70: Sloping bond block wall (source unknown)

L-shaped walls have a horizontal slab and a vertical wall, in 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 self-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, sometimes by rolling stock.

Cofferdam walls (*kistdamwanden*) are a combination of steel sheet piling and soil, working together as a gravity structure. Two parallel rows of sheet piles are driven into the ground and connected to each other with anchor rods (*ankerstangen*) and then filled with soil. Figure 1.71 shows an example of a cofferdam in a dike, elevating the flood defence.

Figure 1.71: Cofferdam wall in a dike in Zeeland to increase the flood retaining height (RWS, 1968)

The cofferdams discussed here are either above ground level, constructed in the dry, or above bottom level, when constructed in water. Soil, sometimes stone, is used as filling. Another type of cofferdam, also referred to as braced excavation is discussed in Section 1.26.2.

If soil layers with sufficient bearing capacity are only present 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 to 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, masonry walls were constructed 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.

1.28.2 Piled wall structures

In the case of piled 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).

The wall structure has to resist bending moments, compression and shear forces. There are various types of walls, which are explained below.

Cantilever walls have 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 these profiles, 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 large.
- 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.

Figure 1.72: Combiwall for the start shaft of the tunnel bore machine for the Rotterdamse Baan, Den Haag (TU Delft, 2018)

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 (Figure 1.72).
- 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.

Sheet pile walls in quay structures are often provided with a heavy concrete coping beam or cap (*deksloof*) on the top of the wall. 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 mainly determined by the soil conditions and the height that needs to be retained. Especially, in case of large surcharge loads (bovenbelastingen), the length of the wall can be a limiting factor.

1.28.3 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 1.73.

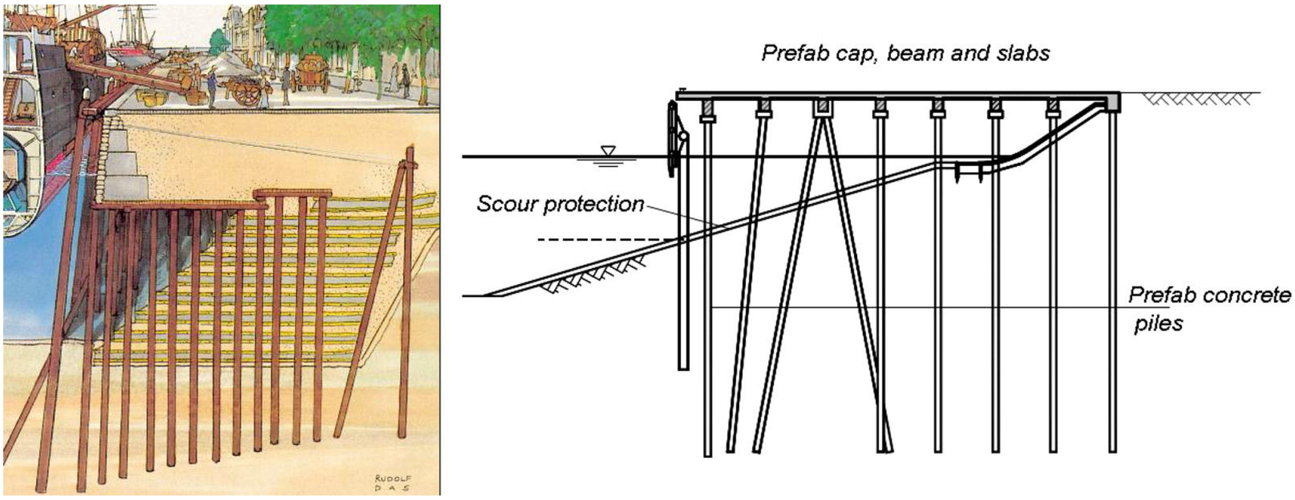


Figure 1.73: Composite soil retaining structures; Gravity wall and piles (left), platform, piles and slope (right) (source unknown)

A platform on pile foundation can be used for instance to carry top load and possible ship load, while a wall retains the soil behind the platform. Under the platform a slope can be made to reduce the retaining height of the wall structure, see Figure 1.74. In this case there is a strict split of functions.

Another common 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 partly retains 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 1.10.

Figure 1.74: Precast platform in Singapore under construction (hsl.com.sg)

2

THE CIVIL ENGINEERING DESIGN METHOD

This chapter describes the basics of the engineering design method that is commonly used in hydraulic engineering. Specific tools and methods that are applied in spatial design (like landscape architecture and urbanism) are not treated in this course, and the same applies to the organisation and management of a design process.

2.1 General hydraulic engineering design principles

2.1.1 The position of the design in the entire life cycle of a structure

A number of successive life stages can be distinguished within the life cycle of a hydraulic structure (Figure 2.1). The number and name of the life cycle stages is a matter of definition.

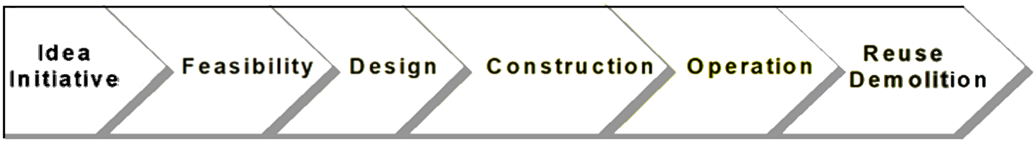


Figure 2.1: Life cycle stages of systems, products or structures

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, see Table 2.1, which 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

Table 2.1: Life cycle stages of systems, products or structures

In reality, the distinction between the life stages is not always clear-cut. Reuse and demolition have been grouped together in Figure 2.1, 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. A conceptual design, or at least a good sketch of the structure will have to be produced during a feasibility study to proceed to the next stage. Many activities take place simultaneously; countless are the projects where construction was already been started while the structural engineer was still finishing the detailed design.

2.1.2 The need for a structured design approach

The money spent on the structure increases throughout the life cycles, even when the project only exists on paper. Costs increase especially in the construction stage (Figure 2.2). If all is developing well, from first sketch to detailed design, the impact of decisions is decreasing, and 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.

If changes are made in later stages, e.g. during the construction stage, one could speak about divergence of the process, which can have severe consequences. The changes are often imposed upon the project, consider e.g. political decisions, or initial governmental procedures (permits) that are still going on when construction has already started. Obviously, design efficiency is reduced by these changes, because an amount of work has to be redone, required time and cost increase (Figure 2.2). It is clear that from the beginning to the end, care should be taken to let the project or process converge instead of diverge.

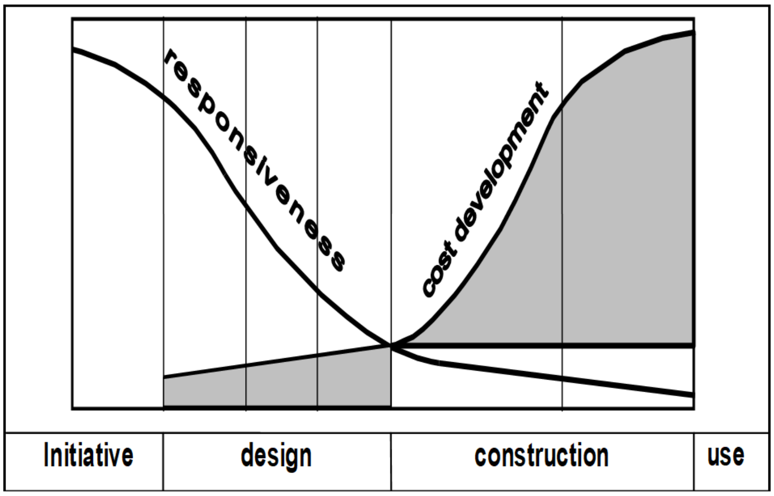


Figure 2.2: Design responsiveness and cost consequences

At the start of the project, usually an initial definition of the problem and an initial set of objectives are formulated, but generally, the problem will have to be explored further for better insight. During the first phase, the project or structure is analysed in several ways. What was initially conceived as problem and objectives may be further defined after these analyses. Functional requirements can be derived from the design objective and should be complemented with requirements regarding structural safety, constructibility, maintainability and integration in the environment. This results in a List of Requirements or Specifications (*Programma van Eisen, PvE*); in Anglo-Saxon literature “Basis of Design” is a frequently used expression. “Terms of Reference (ToR)” is often found in tender or contract documents.

Concepts are generated in a creative way, using tools like brainstorming and morphological charts, (also called the ’synthesis’ phase), but they have to be verified with help of the programme of requirements, to check whether the concepts are realistic (also called the ’simulation’ phase). If not, the concepts can be adapted, or rejected. Alternatives, which are realistic concepts, can be evaluated with help of criteria there are derived from interests of secondary stakeholders (*belanghebbenden*), or wishes of primary stakeholders. These criteria enable a meaningful comparison, which is often carried out with help of a multi-criteria analysis. Costs should be included in the comparison of alternatives by dividing the total score per alternative by the costs per alternative. The alternative with the most favourable value-cost ratio will be selected for a more detailed design per subsystem. Initial governmental procedures (permits) or the costs of construction that had already started. All going well, the process converges into a solution or design.

At specific moments, feedback will be necessary to remain on the right design track. Every life cycle stage may have its own design cycle, every cycle may have its sub-cycles, there will be more than one design phase, thus a whole cycling process originates.

Furthermore, a structured design enables subdividing the process into several clearly separated phases, which increases the transparency and clearness of the design process. In addition, it enables the division tasks amongst several (groups of) designers and it makes the entire process manageable.

A systematic and structured approach facilitates working from a general to a detailed level. The design process is repeated for each level of detail, becoming ever more definite. The process starts with first ideas at system level. The same design process is then done for all sub-systems and then again for all sub-sub-systems per sub-system, etc. (Figure 2.3). After the integration of all the (sub)sub-systems, the system as a whole will be meanwhile designed in a detailed way as well.

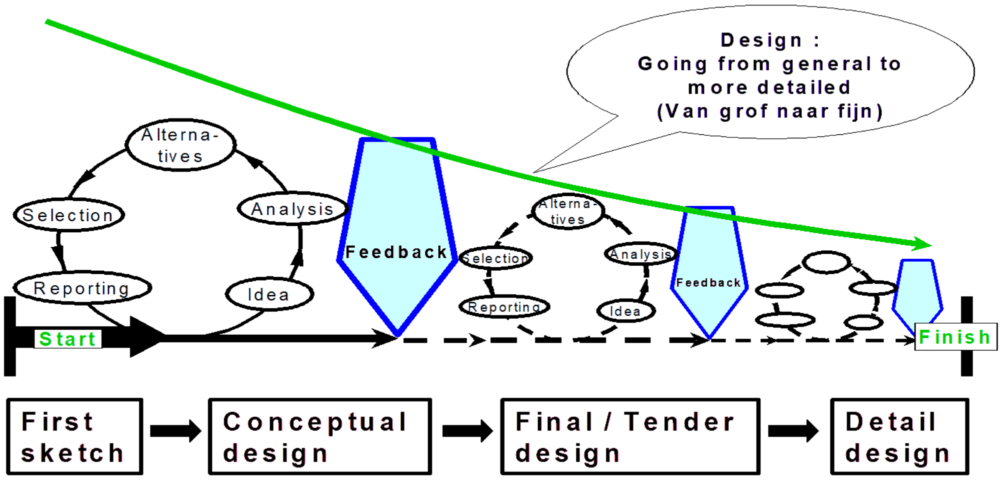


Figure 2.3: Design cycles from over-all to more detailed level

2.1.3 Systems engineering and concurrent engineering

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

Systems Engineering

Systems engineering 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 Systems Engineering 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.

Systems Engineering 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 storm surge 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, the mechanical machinery, electronic control system, the concrete piers or upper structures, abutment or lower structures, sill beams and foundation, bottom protection, etc.

Systems Engineering used as an interdisciplinary approach for the development, realisation and exploration of complex systems, can focus on customer needs early in the development cycle, documenting requirements. It 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 Systems Engineering. The distinguishing factors are:

- The start and the 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.
- As with Systems Engineering, 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.

2.1.4 Spatial design

An important aspect of a design is the spatial quality, especially for design loops at a large-scale system level. Spatial design is the domain of landscape architects, who seek to integrate a new system or structure into its surroundings, rather than to make it 'look beautiful'. More specifically, landscape architecture employs the "principles of art and the physical and social sciences to the processes of environmental planning, design and conservation, which serve to ensure the long-lasting improvement, sustainability and harmony of natural and cultural systems or landscape parts thereof, as well as the design of outdoor spaces with consideration of their aesthetic, functional and ecological aspects" (Nijhuis, 2015).

Spatial design should be considered in all design phases, for instance by doing landscape analyses in the phase of Problem Exploration. Related spatial requirements or criteria should be formulated in the Basis of the Design; spatial quality requirements should be checked in the Verification phase and not assessable requirements can be included in the Evaluation, next to stakeholders' interests regarding spatial quality.

To actually incorporate spatial quality in a design, it should be specified what exactly is comprised in that quite broad term of 'spatial quality'. It can be made tangible by expressing it in terms like 'landscape value' (for instance: coherence of elements and patterns in the landscape, or readability of the natural system), 'natural value' (characterisation by ecosystems, or substitutability of species), 'cultural history' (rarity of elements at different scale levels, characterisation of the structure for the surroundings) and 'socio-economic functions' (accessibility of agricultural fields, possibilities for recreation, impact for traffic).

Spatial quality, however, is not the core of this course, so the way of incorporating spatial quality in a design is not described here. It is a separate profession that can better be carried out by landscape architects (preferably in interdisciplinary design teams). The following sections therefore lack further description of the incorporation of spatial quality in the design process. It should be kept in mind that in reality, these elements should be included.

2.1.5 The civil engineering design method

The civil engineering method is a typical Systems Engineering approach. The rationale stems from a need to solve a societal issue, starting with an investigation of the problem, followed by the formulation of the design objective to solve the problem. This objective is formulated in an abstract way, namely as the fulfilment of a function. During the design process, this objective is transformed into specific shapes and materials.

Basic characteristics of the engineering method are:

- the problem is analysed;
- a project objective is defined as well as the (main) functions of the desired system;

- requirements are defined and an inventory is made of the boundary conditions;
- functions are transformed into specific systems or structures. This transformation starts with generating provisional shapes that are not necessarily realistic or fulfilling all requirements, but during the process, these initial concepts are transformed into verified and evaluated concrete and detailed solutions. This kind of reasoning is also denominated as innovative abduction (Eekels & Roozenburg, 1995);
- multiple concepts are developed that are optimised, systematically evaluated and compared to each other;
- provisional design solutions are optimised by iterations, or recursions, when more knowledge and insight has been acquired. Finding the right system definition, for instance, is often an iterative process;
- series of design sequences are cyclically repeated, adding more detail to the design.

A distinction is made between iterative and cyclic moves in a design process: 'iterative' means that steps are repeated or re-done with more knowledge. 'Cyclic' means that the same steps are done at a more detailed system level. The first cycle starts at the level of a complete system (e.g., a complete marina). It should then be repeated for the subsystems (the jetties, boat house and canteen belonging to the marina) and the elements of the sub-subsystems (the walls, floor and roof of the boat house).

The distinction of several design steps is essential for applying phasing of the design process and for organizing the activities that are needed to come to a working solution. In 1980, prof.ir. Jan Stuip, at that time lecturer at Delft University of Technology, already distinguished the phases of defining the problem (finding the desired functions of the future system), structuring (finding ways to fulfil the functions), shaping (determining the main dimensions) and dimensioning (structural design), to be elaborated in several design cycles at different levels of detail, starting with the over-all system level and ending with a very detailed component level. The organisation of a design is not treated in these lecture notes.

The levels of detailing result in different products: A conceptual design (*schetsontwerp*) is the least detailed level, followed by a tentative design (*voorontwerp, VO*) and a final design (*definitief ontwerp, DO*). This implies that after the initial cycle, there are multiple design processes, carried out by different people at different design levels. This requires a good organisation of the entire design process and appropriate reporting of the results after every design phase.

The result of an engineering design process usually is a set of reports that includes technical drawings and material specifications, but it can also be a software model or a prototype model. The first products of a conceptual design are mostly quite general and broad, but should be good enough to make a reliable cost estimate for the entire project, as the first overall designs are used to decide whether a tender proposal should be made. If it seems technically possible to realise the project for a reasonable price, a tender can be put out. It requires much experience and knowledge to judge whether a tentative design suffices to make a good cost estimate.

Presently, the engineering design process for civil engineering projects at TU Delft is given shape in accordance to Norbert Roozenburg and Johannes Eekels, who were professors at the faculty of Industrial Design at Delft University of technology (Roozenburg & Eekels, 1995). With several adjustments, the method has been adopted for the civil engineering practice in the TU Delft lecture notes 'Integraal Ontwerp en Beheer' (BSc course code CTB1220). The design phases are indicated in Figure 2 4 and are explained later in this chapter. In this figure, the engineering design process might look like a systematic and chronological activity, but in reality, it is less straightforward and less rational than suggested by the presented flow diagram. The figure shows the different design activities and indicates how they are related to each other. In an ideal case, the actual design process is according to the diagram.

In practice, design phase 1 is often subdivided into two parts: Firstly, a concise problem analysis is carried out to define the problem and formulate the design objective . Secondly, a more elaborate system analysis is done to explore the present situation, identify the interests of the stakeholders, do a function- and process analysis, etc., to be able to formulate a correct and complete basis of the design.

2.1.6 Engineering design as an iterative process

The design process has a learning and experimental character. The order of activities is in practice not strictly sequential, because earlier made assumptions or decisions may have to be re-considered at many moments, because of newly acquired information or insight. Hence the iterative jumps indicated in Figure 2.4.

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 one and only 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 have been made of rolling gates as well), in the third place because the solution, the structure, gives rise to other or even new criteria and, the other way around, new criteria leading to technical challenges that may result in new innovative methods or solutions for the structure, for construction, maintenance, re-use, etc.

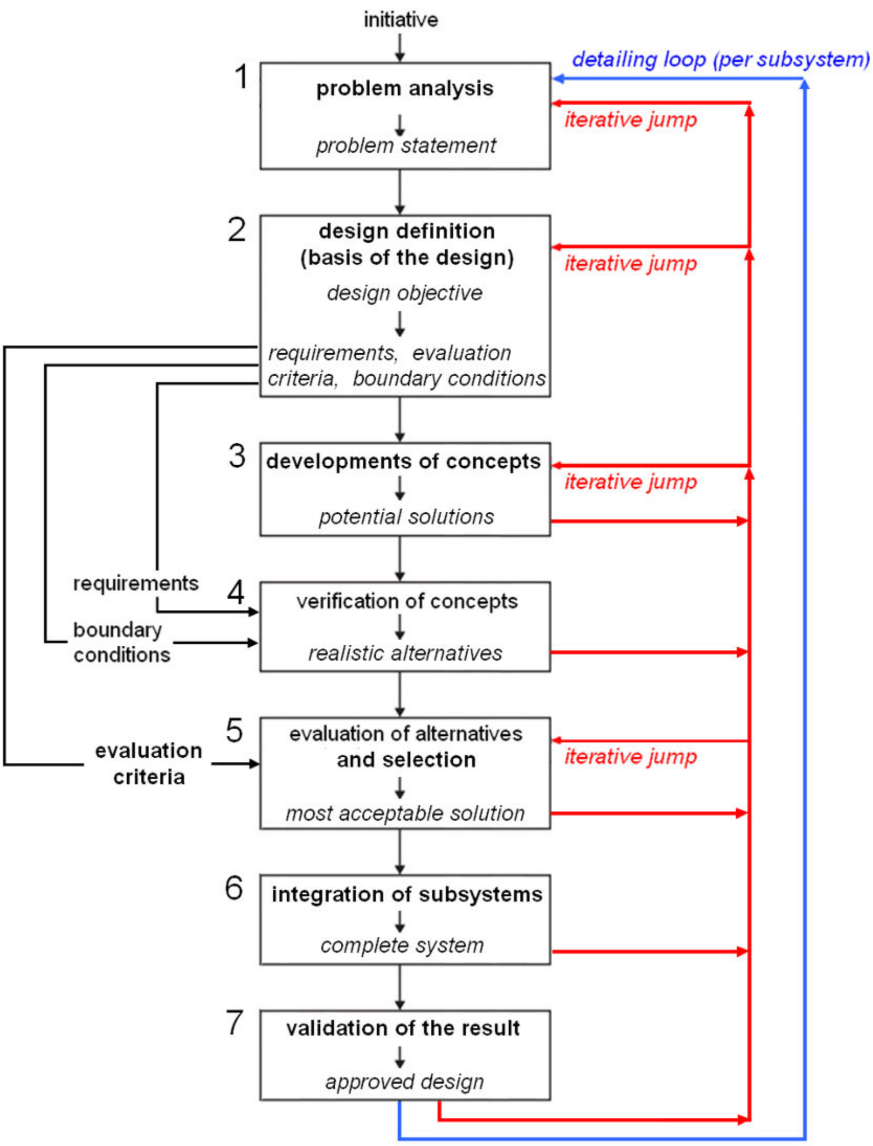


Figure 2.4: The basic engineering design cycle as applied in civil engineering

2.1.7 Design tools

During the stages of the design process there many design tools to support an engineer, which not seldomly have to used more than once at multiple 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 potential 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 (Figure 2.5).
- Brain storm sessions: to generate potential 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.

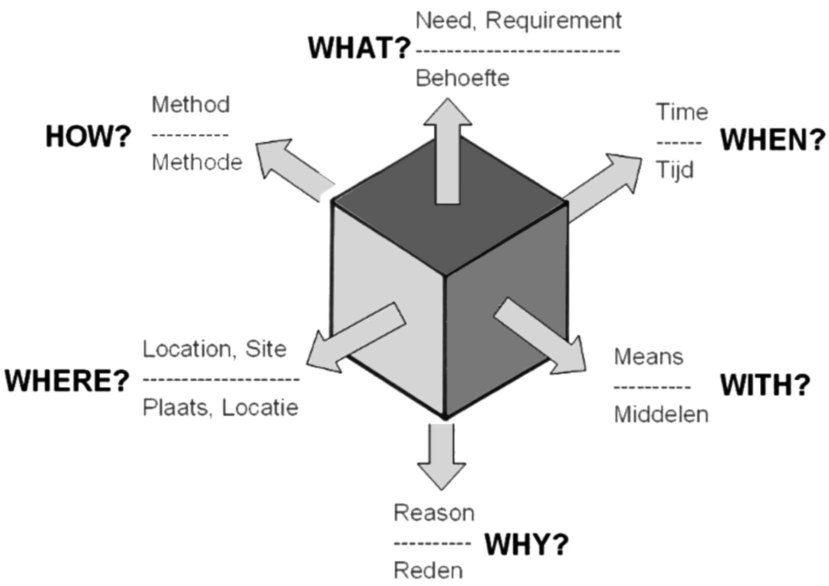


Figure 2.5: Design cubicle, useful for analyses

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 (MCA): 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, the following tools are available for the engineer:

- 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 systematically and transparently evaluate alternative solutions at the system, subsystem or element level, and make the right choice between them on the bases of relevant aspects.

2.2 Design phase 1: Problem analysis

2.2.1 Overview of the problem analysis phase

The aim of the first phase in the engineering design process, the problem analysis phase, is to explore the problem. The Analysis phase usually starts with describing the initiative or motivation (*aanleiding*) of the project and by exploring the backgrounds. An inventory of involved stakeholders sheds light on the involved interests. Process and function analyses give insight in the desired performance of the system or structure that has to be realised. An investigation of the environment and boundary conditions gives information on the restrictions of the solution space. A problem statement briefly summarises the core of the problem and enables the formulation of a design objective, which is part of the following design phase. A programme of functional requirements can then be derived from the design objective and criteria for comparison of concepts (in the Evaluation phase) can be derived from the stakeholders' interests. A project planning can then be drawn up and an inventory can be made of risks that could threaten the success of the project.

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 aims 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. The future should be predicted with the aid of trend watching and/or modelling 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).

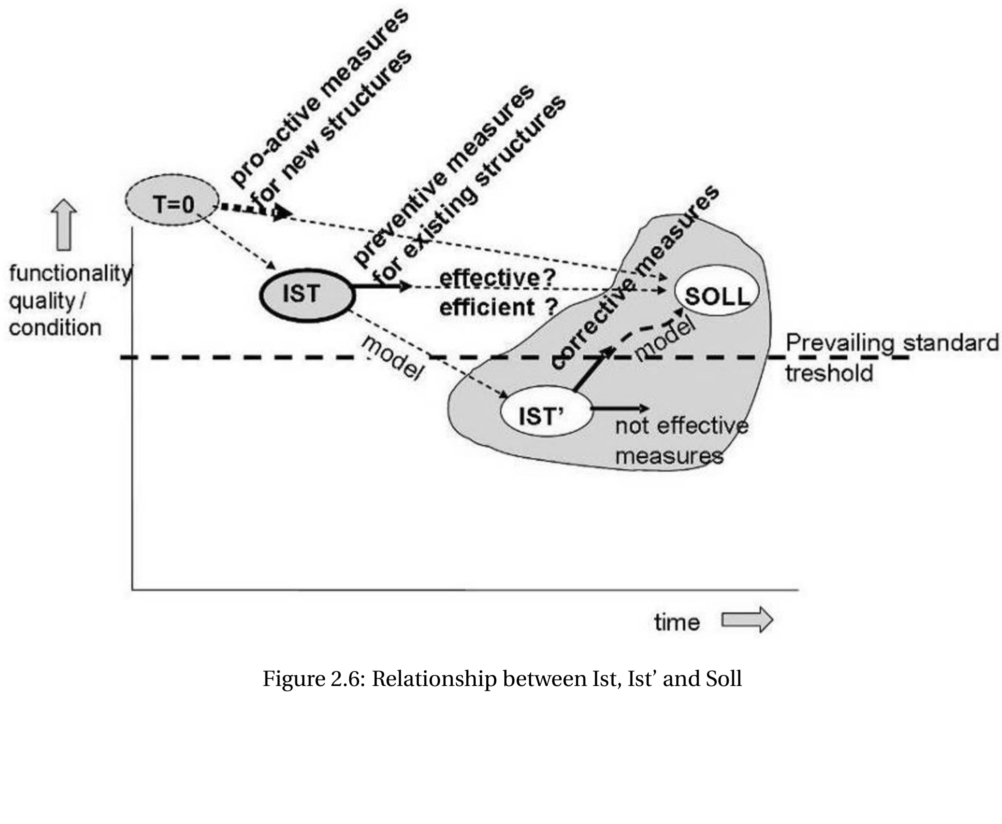


Figure 2.6: Relationship between Ist, Ist' and Soll

2.2.2 Inventory of stakeholders

In hydraulic structure projects, there are many stakeholders, directly or indirectly involved, subjected to many or just a few of the effects of the decision to design, build, operate, maintain and reuse or remove the hydraulic structure. To mention several 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, which can differ 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 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.

Stakeholders can be divided into four categories, ranked by their influence and involvement:

- Public Service Providers. This category consists of the most important stakeholder(s): the client(s) of the project. The client is the initiator of the whole area development project. Their influence on the project is very high and they are internally involved. These stakeholders monitor the developments very closely, set the requirements and make the final decisions in case of complexities. Typical public service providers for civil engineering projects are municipalities, provinces and Rijkswaterstaat.
- Private Service Providers. They are responsible for the design and construction of the structure and have high influence and involvement in the process. Which specific actors are involved and what their role is, is highly dependent on the type of cooperation between the public and private actors. The ownership of the project is a determining factor here. If a DBFM-contract (Design, Build, Finance and Maintain) is used, the investors and contractors play an important role, as they are not only responsible for the construction but also have ownership of the project. This will mean that investors and contractors are the most responsible stakeholders. Stakeholders in this category include suppliers, (sub)contractors, project management teams and investors.
- Core stakeholders, who do not have a large influence on the project itself but are highly involved with it. These are the users' and community representatives and secondary stakeholders. They must be informed well and should have the possibility of any form of participation in the process to ensure that their interests are served. Typical core stakeholders are mobility parties (like ProRail, ANWB, Fietsersbond, Rover, internationale spoorsector) and environmental organisations (such as Milieudefensie, Natuur- en Milieufederatie, Greenpeace).
- Periphery stakeholders. This group represents the wider groups of future users or citizens that are impacted by the construction and area transformation. They usually have a low influence on the project itself and are externally involved. However, their impact of the project should be carefully taken into account. Political and social organisations are included in this category.

Other ways of characterizing stakeholders are to make a distinction between Internal & External Stakeholders, Primary & Secondary Stakeholders or Direct & Indirect Stakeholders.

2.2.3 Process and function analysis

The process and function analyses are tools that can be used to get insight in the desired use and behaviour of the future system or structure.

A process analysis is a step-by-step breakdown of the phases of a process. A process is a sequence of activities that can be represented by a flow diagram. A process analysis can be used to inventory the inputs, outputs, and operations that take place during each phase. A process analysis improves the understanding of how the process operates. It aims at getting clear what should happen in the future system or structure by analysing the desired course and result of the processes for which the system or structure is designed. It can be helpful to obtain insight in the functions and related subsystems that are required for a well-going process. The sub-systems can be transformed into components when developing design concepts (Phase 3 of the design process).

Mainly three types of processes can be distinguished:

- Use processes: the activities of the individual users (for instance, vessels sailing in and out of a lock and the work of a maintenance team);
- System processes: the activities of the exploiting system (including management and maintenance);
- Natural processes: the activities of the natural environment (like rainfall, oxidation, scour of sandy river beds).

It should be noticed that a system is always part of a bigger system. The extent of the total process of the system should therefore be known, otherwise there will be a risk that the problem of the client won't fit in the surroundings or network. In addition, the influence of the processes on the surroundings should be made clear (like noise pollution for neighbouring residents).

A function analysis is a systematic breakdown of desired performances, aiming at a complete functional overview of the entire system. The needs of the client are translated into solution-free functions, like a 'providing a traffic connection between two sides of a river', rather than a 'tunnel', 'bridge' or 'ferry'. The function analysis obtains an abstract description of the future system in functional terms, but it is not yet a design but a model.

A function analysis is a detailed examination of the systematic breakdown of the main desired capability of a system, structure or product. A function consists of at least a noun and a verb, for instance, 'allowing the passage of vessels' or 'retaining soil'. A function description can thus have one of the following shapes:

- offer possibilities to ...
- create conditions for ...
- satisfy the need for ...
- etc.

The (sub)functions and their relations can clearly be represented in a tree structure, where the functions are split into sub-functions. Each level should contain functions of the same order and together lead to the function of one level higher. The main function can be derived from the reason of existence of the system, as indicated in the design objective. The results of the process analysis can be used for deriving the sub-functions, by linking the sub-functions to the sub-processes that they support.

A function analysis creates understanding for the stakeholders' interests and can be helpful in the communication between the design team and the stakeholders. In this way, it creates support for the various design choices. Moreover, a function analysis can lead to new insights and stimulates the creativity of the designer, as there are not yet specific solutions in this phase of the design.

A possible main breakdown of functions consists of distinguishing:

- a principal function, which comes from the motivation (*aanleiding*) to create the system;
- preserving functions, which originates from the fact that the system created to fulfil the main function can interfere with other functions. The new system should therefore 'inherit' these functions from the other systems;
- additional functions, that do not originate from the motivation for creating a system or preserving other systems, but that are extra opportunities created by the intervention.

This is illustrated with the example of a navigation lock in a canal:

- principal function:
  - enable ship passage between stretches of water of different levels
- preserving functions, to maintain present systems:
  - maintaining the water level difference between both canal stretches
  - protect the hinterland against flooding (if it is a part of a flood defence system)
  - preserve ecology
  - reduce salt water intrusion in the fresh water stretch
  - water management: prevent too big water losses
  - sediment and debris discharge
- additional function:
  - enable new road traffic from one side to the other side of canal.

Hydraulic structures often have to fulfil both hydraulic and nautical functions. 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 hydraulic, nautical and 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 usually governing;
- for navigation locks the nautical requirements are more important, and generally governing for the main dimensions;
- for piers and quay walls, usually large horizontal loads are governing the design.

It depends on the local situation whether or not the hydraulic and nautical functions are interfering or even conflicting. If critical maximum water levels result in a high closing frequency of a flood gate 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 of the Maeslant barrier is primarily a storm surge 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 storm surge barrier.
- The multiple 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.

2.2.4 Problem statement

After having analysed the cause of the initiating problems, it will be possible to define the extent and boundaries of the considered system. The problem can now be formulated in a concise way. The 'problem statement' (*probleemstelling*) is the conclusion of the problem analysis. The problem statement should indicate the discrepancy between the present or future situation (without intervention) and the desired situation (after intervention). It has to be formulated in terms of functions, not of solutions (so not 'there is no bridge over river X', but 'there is no road infrastructure crossing river X', or even better: 'there is no means of transporting people and goods across river X').

Problem analyses sometimes show that it is better to widen the scope of the system, if 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. The design objective may be fulfilled by a technical system like a quay wall, weir or storm surge barrier, but such a system is always a part of a much bigger macro-system like a port, a river or an estuary. An integrated design takes the macro-system into account, 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 storm surge barrier does not prevent the hinterland from flooding if adjacent dikes in the same dike ring are the weakest link.
- a quay wall cannot accommodate ships with much more draught than the approach channel of the harbour.
- a weir in a river cannot regulate the river discharge sufficiently and/or maintain enough water depth, if adequate measures have not been taken in the upstream river reaches.

2.2.5 The design objective

After having formulated the problem statement, the design objective (*ontwerpdoelstelling*) can be determined. However, it should be kept in mind that next to the main function, all major sub-functions are included. This is essential, because the functional requirements are derived from the design objective. If the objective is not complete, it can easily occur that several requirements will not be formulated, resulting in an insufficient design (this will probably only be detected in the 'Validation' design phase, but that is quite late in the process).

2.3 Design phase 2: Defining the Basis of the Design

2.3.1 Overview of the Basis of the Design phase

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

The biggest part of the Basis of the Design will be produced in the initial and feasibility stage of design. However, stage additions will be made in the technical design. Up to the moment that construction starts or even up to disposal, the Basis of the 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 appears to be as 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 Multi Criteria Analyses and/or Cost-Benefit Analyses, thus to a completely different project (which includes no project) as well.

The Basis of the Design typically consists of:

- A programme of requirements and evaluation criteria;
- Boundary conditions (*randvoorwaarden*);
- Starting points (*uitgangspunten*)

It is crucial that every item in the Basis of Design is defined in a clear, unambiguous and preferably quantitative way.

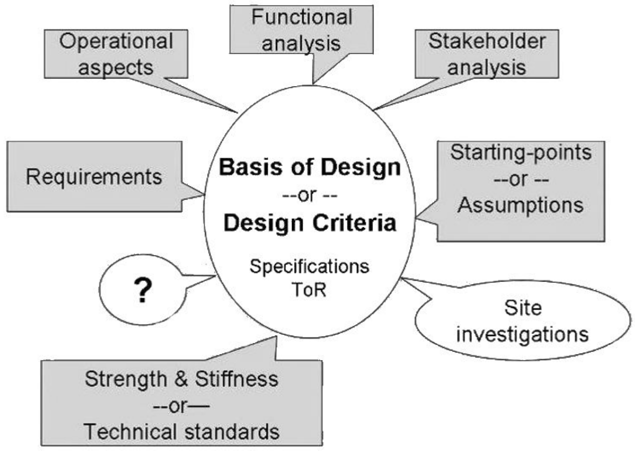


Figure 2.7: Analyses that can be used to set up the Basis of Design

2.3.2 Requirements

The main functional requirements can be derived from the desired system functions as specified in the design objective. They specify the expected design result and have to be given or agreed upon by the client. Requirements are demands of the client that have to be met. If the functions of all main aspect systems are included in the design objective, the Programme of Requirements will cover the entire scope of the design. Design alternatives (to be developed in the next design phase) finally have to meet all the requirements.

To obtain a clear overview of requirements, it can be helpful to group them by type:

- **functional requirements**, which qualitatively or quantitatively describe the desired behaviour, or performance, of the system or subsystem under defined conditions;
- **aspect requirements**, which describe specific characteristics of the system that supports the primary functioning of the system;
- **external interface requirements**, which originate from the fact that the system often crosses or borders adjacent elements in its surroundings when one element influences the other;
- **internal interface requirements**, which originate at the boundaries between subsystems or elements within the system that is under design. The subsystems created by different disciplines, like mechanical, electrical and civil engineering should be well-connected as well.

For educational purposes, this course uses a main distinction of two types of requirements:

- **functional requirements**, to be derived from the design objective;
- **structural requirements**, inherent to a well-functioning structure and ensuring structural integrity:
  - constructibility
  - stability (overall)
  - dimensional stability (*vormvastheid*)
  - strength

The functional requirements have to be used in a **functional-spatial design** phase, and the structural requirements are dealt with in a succeeding **structural design**. Section 2.4.3 explains the difference between the two types of design.

Safety requirements are 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. 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.

To give only one example: the level of accepted risk of the storm surge defence in New Orleans was in the order of a 1:100 years storm, while in Holland this level is based on loads with an average 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.

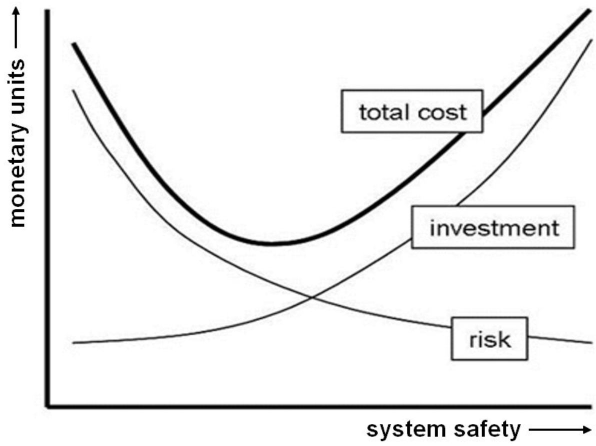


Figure 2.8: Reducing the total cost by balancing investment and risk

There may be strong sentiments in society opposed to expressing safety in mere cost figures. Decision techniques based on Multi Criteria Analysis (MCA) can therefore be more helpful to reach a decision.

Going into more and more detail, so from macro-system, via system, sub-system to element (the so-called technical decomposition), the requirements at macro-system level have to be decomposed in a consistent structured way (the so-called functional decomposition). Design tools like fault-trees (*foutenbomen*) and event-trees (*gebeurtenissenbomen*) can be of great help to clarify how subsystems or elements do or don't contribute to the performance of systems at a higher level. The main allowable failure probability (the failure probability requirement) can thus be subdivided towards allowable failure probabilities per component and per failure mode.

An example of a fault tree is shown in Figure 2.9. In this example, 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.

Fault trees are treated in more detail in Section 6.4.4.

2.3.3 Evaluation criteria

Evaluation criteria are values, or 'qualities' that can be used to compare verified alternatives. The evaluation criteria are derived from the clients' wishes and stakeholders' interests, which are typically non-quantifiable. They usually consist of values such as human life, nature, culture, spatial quality, but can involve more technical values as well, such as maintainability and disturbance to the surroundings by construction activities.

Many stakeholder interests will be translated into evaluation criteria. The criteria will be different or show overlap to greater or lesser extents. Sometimes conflicts with other criteria or requirements will result. Unfortunately, it seems to be a (bad) habit to introduce important criteria or even requirements rather late

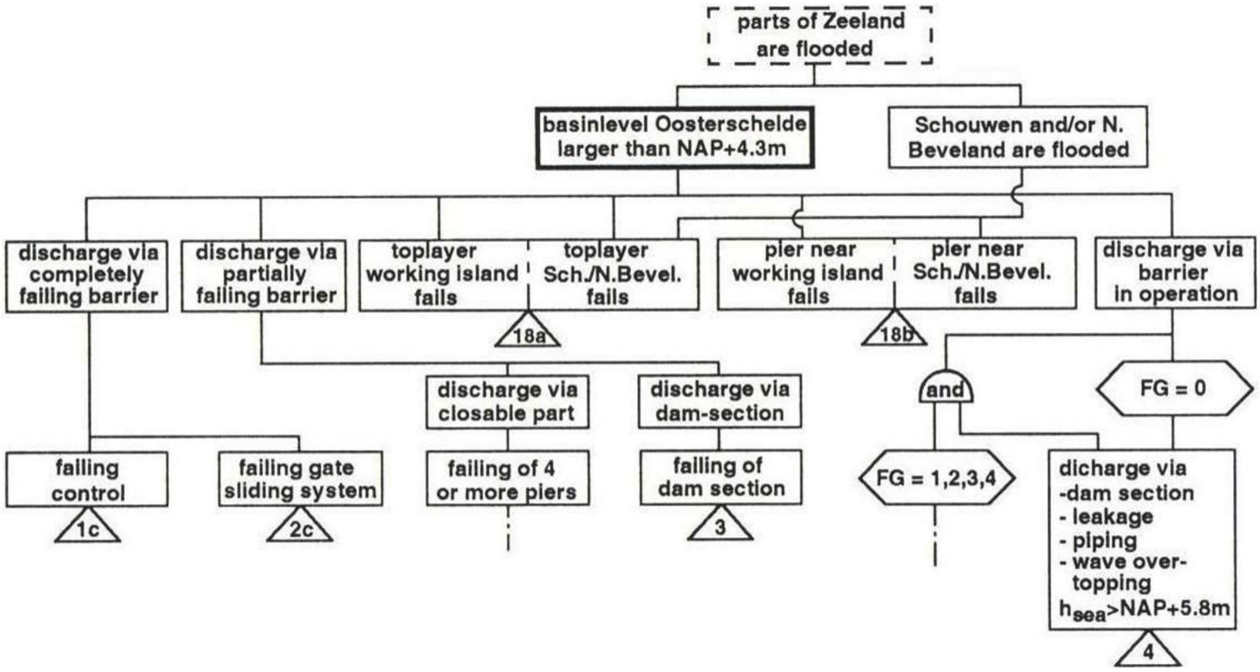


Figure 2.9: Fault tree for flooding of Zeeland - the Netherlands

in the design process. The objectives of the (local) society generally result in criteria of a different or extra category, the category of environmental friendliness and aesthetics.

Examples:

- Local society does not want high cranes at their skyline or noisy trans-shipment 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 borders 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).

Examples of evaluation criteria are:

- limitation of impact on the surroundings (noise, smell)
- accessibility of the surroundings during construction
- accessibility of the construction site (over land, over water?)
- ease of construction
- shortness of the construction period
- extendibility
- adaptability
- sustainability
- durability
- possibility of inspection
- possibilities for repair
- ease of maintenance
- aesthetics
- visual integration in the surroundings
- spatial quality
- social safety
- energy savings (use of renewable energy)

Section 2.6 explains more about the use of evaluation criteria in the Evaluation design phase.

Criteria versus requirements
Optional wishes of the client are usually formulated as evaluation criteria to be able to compare verified alternatives in the Evaluation phase. Interests of secondary stakeholders (stakeholders other than the client) are usually formulated as criteria as well, but the client could decide to formulate them as requirements if he considers them essential. A clear distinction should therefore be made between requirements and criteria. All generated concepts have to comply with the requirements formulated for the design cycle under consideration before they can systematically be evaluated. Therefore, requirements should not be used as criteria for the evaluation of alternatives.

It could be argued that several requirements could be complied to even better than strictly needed, for instance with a lower failure probability than required, which would be a reason to include a requirement as an evaluation criterion. The drawback is that the alternative, functioning better than strictly required, will possibly be more expensive. That would make it less feasible in an economic sense.

Sometimes, stakeholder interests are formulated as requirements, instead of evaluation criteria: stakeholder interests become major design considerations and social acceptance of the project by the concerned stakeholders is very likely. However, it is difficult to satisfy all stakeholders, because of conflicting interests. Another disadvantage is that, if all major stakeholders' interests would be formulated as requirements, the selection of the best alternative can only be based on a comparison of costs because, in that case, there are no criteria left for the evaluation. This can feel unnatural or uncomfortable for those who are familiar with the basic design method, but it is the consequence of the approach.

A major drawback of formulating stakeholders' interest as requirements, is that stakeholders often do not consider the over-all consequences of formulating their interests as requirements. Firstly, they often do not oversee the consequences for society as a whole of putting forward their interests as requirements. Secondly, they often have single-issue interests, and an overarching design and decision process is needed to balance all the stakeholders' interests (next to the client's requirements). This can only be done if these interests are formulated as criteria.

2.3.4 Boundary conditions

Boundary conditions (*randvoorwaarden*) are site-specific parameters that can comprise many aspects given by the surroundings where the solution has to be realised. They often limit the possibilities and could pose a major challenge in the realisation of generated concepts.

Boundary conditions can, amongst others, be subdivided into natural boundary conditions, artificial boundary conditions and legal boundary conditions:

- **Natural boundary conditions**
  - hydraulic conditions (water levels, wave heights, structural erosion or accretion);
  - meteorological conditions (wind velocity and direction, temperature, humidity);
  - geo-technical conditions (soil properties, layers, groundwater, ground levels, land subsidence);
  - geological conditions (presence of hills, mountains, rivers, lakes, sea, earthquake conditions).
- **Artificial boundary conditions**
  - nautical conditions (intensity, fleet composition, max currents);
  - road traffic (intensity, traffic composition).
- **Legal boundary conditions (laws and regulations)**
  - laws;
  - regulations;
  - municipal plans;
  - etc.

The presence of the new structure will influence its direct surroundings. Without further thinking, one could think that the hydraulic boundary conditions influence the solution, but in the opposite direction the boundary conditions will be influenced by the hydraulic structure. According to strict Systems Engineering and mathematical definition, the conditions at the boundary are not or cannot be influenced by the presence of the structure. Literally, the boundary has to be chosen at such a distance, that it cannot be influenced by the structure.

Sometimes, it is difficult to find useful data for these boundary conditions. While in the Netherlands long series of wave and water level measurements are available and can be used to calculate water levels corresponding to required safety levels, abroad this is not always the case. For example, soil properties are required for the design of foundations, but, even in the Netherlands, sounding graphs are not always readily available for the exact project location. The magnitude of wind and other loads is uncertain, so it is a typical task for an engineer to deal with these uncertainties. Data acquisition campaigns can be carried out to do additional measurements. Typical questions to be answered for such campaigns are then: how many data are needed, at what distance from each other, in what sample frequency, with what accuracy and with what measurement devices? If there are no sufficiently reliable data available, especially for extreme conditions, governing boundary conditions can be obtained through mathematical calculations, computer simulations, scale models or prototypes (if well validated).

Examples of boundary conditions:
At the waterside of a hydraulic structure there are the typical natural boundary conditions like:

- bathymetry of the dry and wet part of potential building location (with help of Geographic Information System (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

At 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!)

Boundary conditions versus requirements
Requirements are forced upon the project by the client and boundary conditions are forced upon by the environment. They should be considered as matters that cannot be influenced by secondary stakeholders and certainly not by the design team or engineer.

2.3.5 Starting points

Starting points (*uitgangspunten*) are design decisions imposed by the client. Starting points thus restrict the solution room (*ontwerpruimte*). An example is the instruction to specifically create a tunnel under a river, rather than to create a more generically formulated traffic connection between two riversides. Another example is the planned economic lifetime of a system.

Starting points versus requirements
There is a typical difference between requirements (*eisen*) and starting points (*uitgangspunten*). Requirements specify how the design objective should be achieved. They are defined by the client and describe in what way the final structure or system should perform. Starting-points comprise design choices already made by the client, restricting the number of possible design solutions, the 'design space' (*ontwerpruimte*). An example of

a starting-point for instance is that a river-crossing is constructed as a steel bridge, leaving out all kinds of other bridges, tunnels and ferries as possible solutions.

Compared to requirements, starting points are more open to discussion, i.e. the quantification is a matter of agreement between the client (and possibly other 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.

2.3.6 Requirements and boundary conditions changing in time

If 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 then 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:

- sea level rise because of melting icecaps (0,5 m, 1 or 2 m?) or increasing river run-off (e.g. maximum run-off of River Rhine from 15 000 to 18 000 m^3/s), 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 storm surge barrier tempers the in- and outgoing tidal stream, so creates sandbanks in front of the barrier and so reduces the governing wave height.

2.4 Design phase 3: Development of concepts

2.4.1 Creating concepts

Having established a Basis of Design, the time has come to come up with shapes and techniques that potentially can solve the problem. The design objective should be kept in mind, to prevent the creation of irrelevant design concepts with lacking subsystems. However, to not restrict creativity too much, the designer should not be afraid to come up with concepts that are not yet entirely feasible. After all, the succeeding design stages will ensure that the final solution is feasible. In the case that new and innovative concepts are desired, for instance if spatial design is an important aspect of the objective, it is advised to formulate the requirements, who in fact are specifications of the design objective, only after the development of concepts. The reason is that creativity will be restricted, if the designer already has the requirements in mind .

There are several techniques that can be used to generate potential concepts. The most commonly used technique is brainstorming. This is especially useful, if traditional solutions or extrapolation do not seem to result in an acceptable solution. A brainstorm is usually carried out by a group of four to fifteen people. The group members just quickly come up with ideas that could potentially lead to reaching the design objective. The ideas should be listed or sketched (simple hand drawings). Premature criticism should be avoided in this design phase and ’freewheeling’ (expressing any idea one can think of) is welcome. It is encouraged to build upon the ideas of others, or combine elements of various concepts. Brainstorming is suitable for relatively simple problems with an open formulation. If highly specialised knowledge is needed, other methods might be more suitable

Design-trees and morphological charts can be used to generate alternative (partial) solutions with a more structured approach. Both methods decompose complex functions into elementary functions and, the other way around, combine elementary solutions into a new composition. The generated idea are thus not completely new, but most of the time they are extrapolations 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 can result in successful stepwise or incremental innovation.

The starting point of a morphological chart is the function analysis. It breaks down the overall system function into sub-functions at several levels. Often, there are already solutions for several of the sub-functions, while others still have to be generated. The morphological method results in a matrix of sub-functions (parameters) and solutions (components), which enables the description of potential principal solutions by combining solutions for each sub-function. Figure 2.10 shows an example of a morphological chart. The morphological chart is most suited to engineering problems, less for spatial designs.

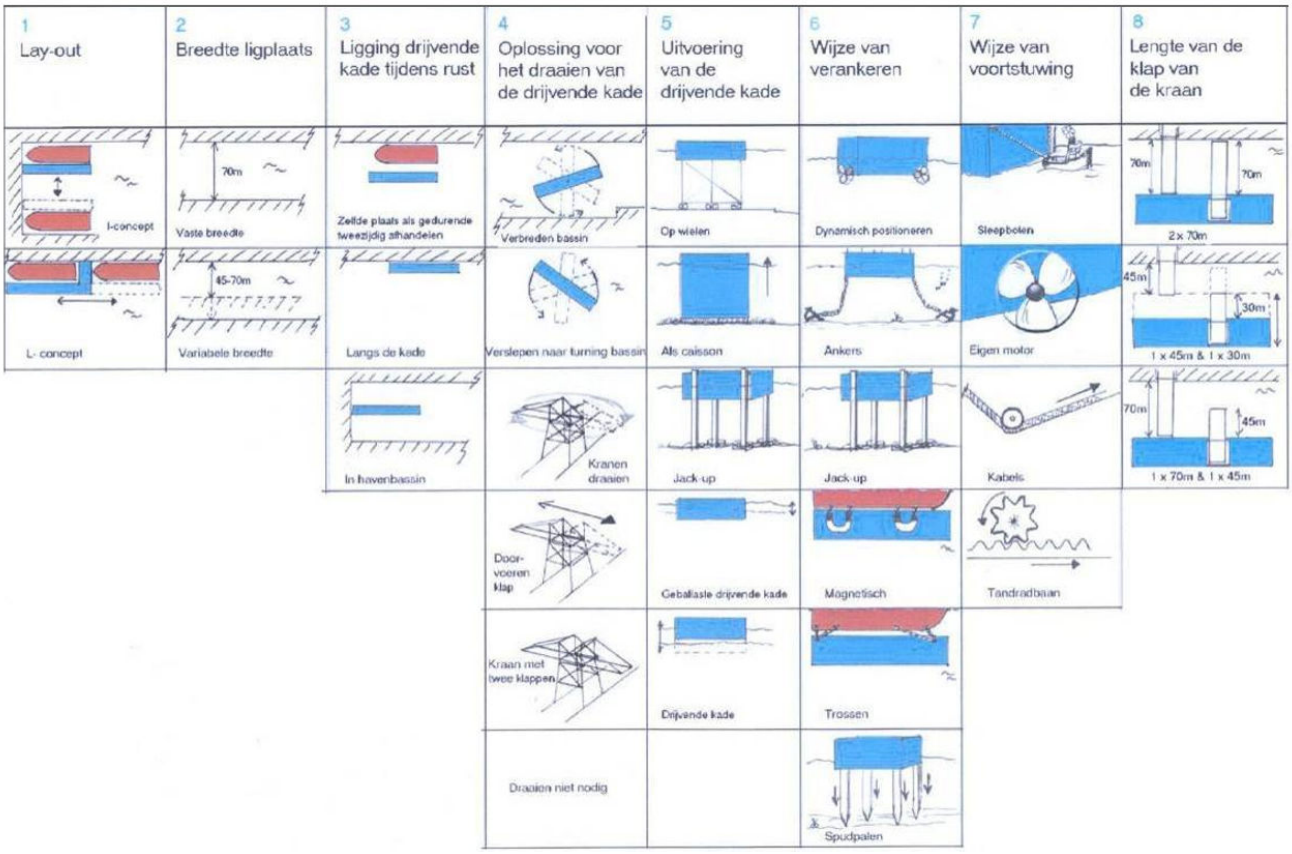


Figure 2.10: A morphological chart for design of a floating quay – graduate student Christian Paus, 2004

Examples:

- If a quay wall has to be constructed in much deeper water, there is search for floating or semi-floated and pre-stressed anchored solutions (see offshore oil platforms).
- If a storm surge 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 wave impact on the gate in a storm surge 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 quay walls 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 quay wall or storm surge barrier in 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.

More techniques to generate ideas, like fish trap modelling, using analogies or metaphors and synectics, are described in the Delft Design Guide (Van Boeijen et al, BIS Publishers).

Professional engineers will base their first design concepts on theory, 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 quay wall may on the one hand have up to seven different functions like a place for berthing ships, supporting cranes for the trans-shipment 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 caisson type, gravity type block walls, the combi-wall with concrete relieving platform, etc.. Most of the times one of these basic solutions will suffice for 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 will have to be found. See the combined breakwater-quay wall-storage facility-garage in Monaco.
- A storm surge barrier mostly has 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.

2.4.2 First plans / sketch design / preliminary design

Although the design process still is in a very early (feasibility) stage, as there are only objectives in terms of a desired situation and a first set of functional requirements, there is already a strong need to 'see' the solution. The first plans, 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, the design can move from feasibility design stage into technical design. Figure 2.11 shows an example of a first sketch of a small navigation lock.

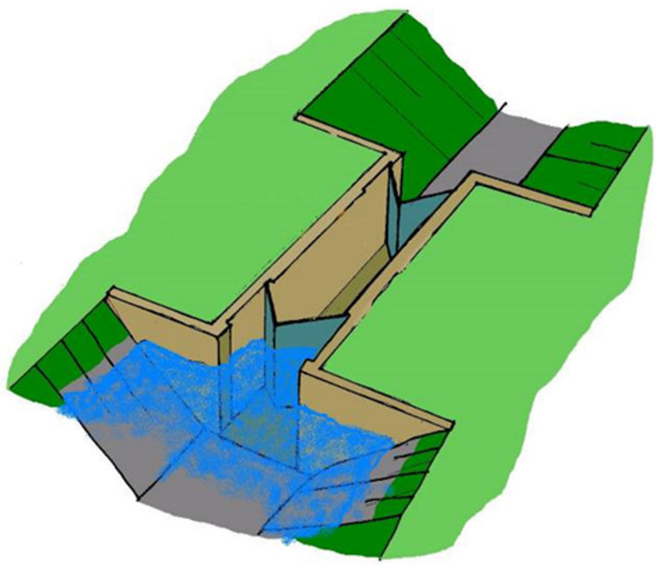


Figure 2.11: First sketch of a small lock for recreational purposes

The design used in the 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 not yet a technical solution worked out for the specific situation at hand.

2.4.3 Separating functional-spatial design from structural design

In an ideal case, spatial-functional and structural design are developed in a combined process, but due to complexity, it can be decided to first create and verify spatial-functional concepts, and only thereafter create and verify structural concepts.

A functional-spatial design consists of:

- Finding a technical solution, or key element, to fulfil the (main) functions of the system or structure (this can, amongst others, be gates, water intake systems, outflow systems or pumps);
- Determining the components that are needed for the structure or system to fulfil its function (for a tunnel, for example, the following (sub)systems have to be considered: earth- and water free underground space, ventilation, traffic control, pumping system, lighting, firefighting installations, emergency and escape, etc.);
- Determining the main dimensions of the structure or system, needed to fulfil the main function and estimate the dimensions for the main structural elements by rules of thumb or by scaling reference structures.

Notice that a **hydraulic design** can be a substantial part of the functional-spatial design. The height of a dike, for example, is a key feature of a flood defence to make it functional (and differs from a structural design that mainly deals with stability and strength). Determining the dimensions of the flow-through openings of the closable part of a storm surge barrier to allow sufficient tidal propagation in the hinter laying sea arm is another example of a hydraulic design that enables the engineer to find the main dimensions.

A structural design consists of coming up with ideas to ensure constructibility and structural safety (= structural integrity) of the system or structure:

- constructibility
- stability (overall)
- dimensional stability (*vormvastheid*)
- strength

Most of the work in a structural design is in the Verification phase, but in the phase of Conceptualisation, an inventory can be made of construction and design techniques that could be interesting to consider (it has not yet to be decided what is really feasible yet – that will be done in the Verification phase). For instance, for the design of a metro tunnel, the following techniques can be inventoried:

- earthen cofferdam (natural slopes) (*bouwput*)
- braced cofferdam (*bouwkuip*)
- cut & cover (*wanden-dak*)
- pneumatic caisson (*pneumatische caissons*)
- standard caisson / immersed tunnel (*afzinkbare tunnel*)
- sequential excavation (*sequentiële afgraving*)
- tunnel boring (*geboorde tunnel*)

These different techniques can be considered as alternative concepts that have to be verified in the following design phase (see Section 2.5). Making this inventory is advised for beginning engineers - professionals will have this knowledge and experience already in their minds.

2.5 Design phase 4: Verification of the concepts

2.5.1 The purpose of the verification design phase

In the Verification phase, the created concepts are verified to check whether they will function properly as defined in the project objective and specified in the programme of requirements, defined at the detailing level of the design loop under consideration. The designer has to verify functionality, as well as structural safety, serviceability, durability, constructibility, maintainability, reuse of all elements, subsystems and the system as a whole. A wide range of theories, formulas, tables, scale models, prototypes, drawings and research methods from technical and behavioural science are at the disposal of the designer. They all model reality, more or less simplifying it.

Simulations can be used as a verification tool, for example to check a transport infrastructure system to detect bottlenecks. Calculations are suitable to determine the main dimensions of the system or the structure. Typically, during this design phase, the technical system and the main dimensions are derived from functional requirements and the dimensions of structural members (like the thickness of the floors, walls, columns, etc.) follow from requirements regarding structural integrity in more detailed design loops.

If functional requirements cannot, or only problematically, be quantified, the effects of concepts can be simulated and the extent of meeting these requirements can be checked with help of interviews, questionnaires, focus groups or user observations. If requirements cannot be assessed, so, if it cannot be decided whether the fulfilment of a requirement is acceptable or not and only a degree of fulfilment can be indicated, a Multi Criteria Analysis can be made. This method is usually applied as a tool for the evaluation phase of a design process, in which case the criteria consist of not-mandatory interests of stakeholders, but it can be used in the Verification phase as well with not-assessable requirements. See the Section 2.6 for a description of the Multi Criteria Analysis.

2.5.2 Selection of a location

The selection of a location that is suitable for the system so it can fulfil its desired functions and meet the external aspect requirements, presents a "chicken and egg" dilemma, because a rough design is needed to select a location, but opposite, a global location is needed to make a first design. This is a typical loop in an iterative design process.

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

- 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;
- necessary adjacent extra works (connecting waterways, roads, dams, etc.);
- rough estimate of total life-cycle 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. Requirements 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. A Multi Criteria Analysis can be used to do the actual selection of the location (this tool is explained in Section 2.6.2). Another suitable method is the sieve analysis, where all unsuitable locations are indicated on a map or the area. The remaining area is where there is no objection to realise the new projects, see Figure 2.12 for an example.

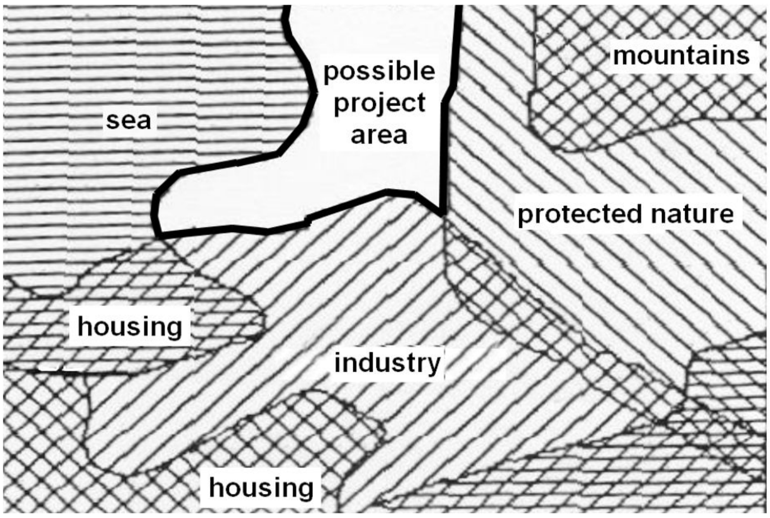


Figure 2.12: Sieve analysis (modified from lecture notes 'Integraal ontwerp en beheer, 2018)

A Potential Surface Analysis is another method with the same goal. Instead of indicating all unsuitable areas, it is marked per criterion what locations are suitable. The areal map is therefore provided with a raster. The degree of suitability is indicated per location with scores. The location with the highest score is the optimal project location.

2.5.3 Functional-spatial design

The functional-spatial verification ensures the functioning of the system or structure. It implies checking (= verifying) the concepts against the programme of requirements, taking the boundary conditions into account. This means that there should be a working technical system, and sufficient space to let it functioning.

Functional design comprises the technical functioning of a system, so the required technical components will have to be identified and given shape. For a navigation lock, for example, the following main components are commonly necessary to let it function:

- waiting or lay-by berths
- guard, guide or approach wall, lead-in jetty
- lock heads
- closing elements - 'gates'
- filling and emptying system
- lock chamber
- bottom protection
- cut-off screens (*kwelschermen*) to prevent seepage (*kwel*) and piping (*zanduitspoeling*)

In addition, the main dimensions will have to be found, starting with the lock chamber, to ensure that the governing vessel will fit in it and the waiting times are acceptable. The top of the structure will mostly depend on wave-overtopping requirements over the gates, for which the water levels and wave conditions have to be known. The filling and emptying system should be designed in such a way that the levelling time (*schuttijd*) as well as the generated currents remain acceptable.

A hydraulic design is a typical form of a functional-spatial design. It can consist of the determination of a weir height with help of a backwater curve, or the determination of the culvert width of a dewatering sluice with the stage-discharge relation, including energy losses because of contraction and friction.

2.5.4 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 the 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 storm surge 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 has led to a minimum gate opening, while the storm conditions work in the opposite direction.

After analysis and initial design, a first structure can be 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. 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 or by physical model testing. Although the first aim is to check the hydraulics, like in case of the discharge sluice in Figure 2.13 it should work the way it is supposed to work, the second aim is to determine the hydraulic loads on the structure.

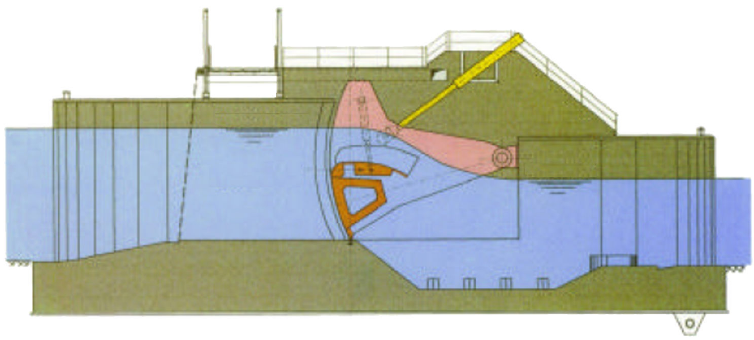


Figure 2.13: Complicated flow through a discharge sluice (source unknown)

Morphological analysis

Sedimentation and counterpart erosion are two aspects that should be given special attention. Although primarily caused by hydraulics and shipping, they may the other way around 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 quay wall at the 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 high sedimentation rate requires sounding and dredging for every new ship arrival.

- Caisson-type quay walls at the port of Rotterdam faced throughout the years growing depth of berthing places. 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 many hydraulic models like Delft3D to forecast overall tides and velocities, and 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 v3). This is even worse if erosion is caused by local flow from the 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 for the reduction and management of 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[1]-guidelines for approach channels, or ship simulation and physical model testing. If a structural design of a weir, for the governing closed condition, results in dimensions that are insufficient for ship manoeuvres in open condition, or the resulting current velocities in the remaining water section are too high for navigation, 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.

textbfEnvironmental analysis
The delay or cancellation of many construction projects due to underestimating the environmental consequences could have been prevented by carrying out thorough environmental analyses. 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:

- The construction of the Western Scheldt 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 Wadden Sea and the consequences for the flora and fauna were not investigated in satisfying detail.

[1]PIANC is the Permanent International Association of Navigation Congresses, the world association for waterborne transport infrastructure.

2.5.5 Structural design

In general, the following steps are taken within the structural design of a hydraulic structure[2]:

1. Choose a suitable main construction method:
   - in situ / prefab / combination;
   - 'in the dry': construction pit, notice:
     – sealing of floor: clay / underwater concrete;
     – sealing of walls: vertical walls / natural slopes + dewatering;
   - 'in the wet': caissons, piles, under water construction;
2. Figure out what steps are needed for construction: draw a construction sequence (bouwstrip), that includes the operational phase (see below):
   - check constructibility;
   - make an inventory of all load situations.
3. Provide stability for temporary and permanent structures (during critical load situations). Perform the following checks:
   - piping
   - bearing capacity and settlement of the subsoil: foundation: shallow or deep?
   - uplift: tension piles/anchors or additional ballast needed?
   - lateral shear;
   - rotational stability;
   - embedded depth of retaining walls;
   - potential support of walls (struts / anchors / permanent structure);
   - ensure dimensional stability (*vormvastheid*) of the load-bearing structure;
   - scour;
   - earthquake impact;
   - human and animal actions.
4. Check the strength of the structural elements + connections (+leakage);
5. Check deformations and displacements.

Notice that both the Serviceability Limit State (SLS) (*uiterste gebruikstoestand*) and the Ultimate Limit State (*uiterste grenstoestand*) should be considered.

If necessary, e.g. if 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 through.

The construction & use sequence

A construction sequence (*bouwstrip*) is a series of simple sketches, mainly cross-sections, where one main construction activity is carried out per sketch (see Figure 2.14 for an example). Such a sequence already gives an idea whether the proposed structure is constructable. In addition, it gives direction for the structural design process and provides an overview of the load situations to be used in the structural verification.

Loads and load combinations for hydraulic structures

The strength and stability checks are in fact checks of failure mechanisms, or limit state functions, where it should be demonstrated that the design loads won't exceed the resistance by the structure. While doing this, a certain safety philosophy should be taken into account, which is often prescribed by national standards (enforced by law) or recommended in structural guidelines. The governing conditions and accordingly critical load combinations should be determined per failure mechanism, because they can be different for the various modes of failure. For uplift, for example, high (ground)water levels in combination with a minimum of self-weight and minimum ballast will be governing, while for the soil bearing capacity it is the situation with low

[2]This sequence is looks straight-forward, but it is iterative. The sequence is intended to support the structural design process, but should not be used as a generic recipe that always ensures a well-functioning structure. Furthermore, it does not cover all types of structures. For proposed sequences on soil embankments and structures in soil embankments, see Voorendt, 2017 (Design principles of multifunctional flood defences, Section 5.3.2). For floating caissons for closure dams, reference is made to the lecture notes CTB3355 on caissons.

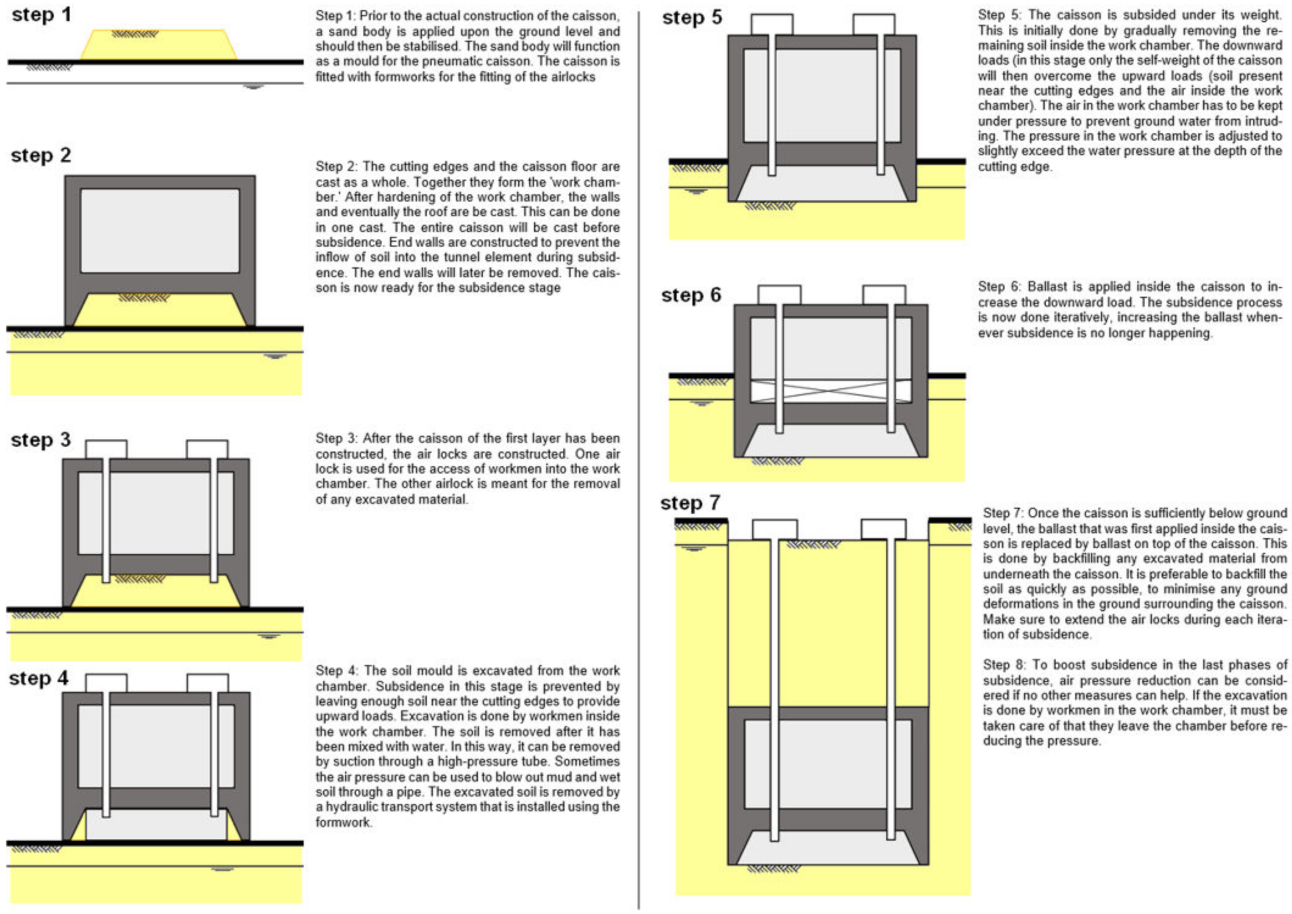


Figure 2.14: Example of a construction sequence for a pneumatic caisson (M. Smit, 2020).

water levels and maximum weight. The construction sequence is very helpful in determining the governing conditions.

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

- self-weight of the hydraulic structure
- hydraulic loads (hydrostatic and dynamic) due to tide, discharge, setup, sea level rise, etc.
- loads caused by primary users of the structure, like vessels (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)

For instance, for a wave load at least 4 parameters 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.

Notice that, next to the self-weight and several of the life loads, most of the loads act in the horizontal direction.

In general, horizontal loads are governing (*maatgevend*) for 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 notice 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 the 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 tree (*foutenboom*) in Figure 2.9, a few pages back. 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 NEN 6702). Moreover, the probability of occurrence is generally different for every project because it varies with the number of bottom events.

In the text above, several 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 self-weight are working together and should be combined with wisdom. 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 (Figure 2.3).

The sequence and associated characteristics of design calculations 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. $1/10$ - $1/8ql^2$ for bending moments or $5/384ql^4/\mathrm{EI}$ 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.
2. **2D or plane frame calculations, simple software**. Based on the hand calculations, the obvious errors in the shape of the structure will have been removed and basic shapes and rough dimensions of structural elements will have been established. Generally, the structure as a whole will thereafter be modelled 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, quay walls, 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, cost much money.

Figure 2.15 shows two levels of schematisations for lock gates.

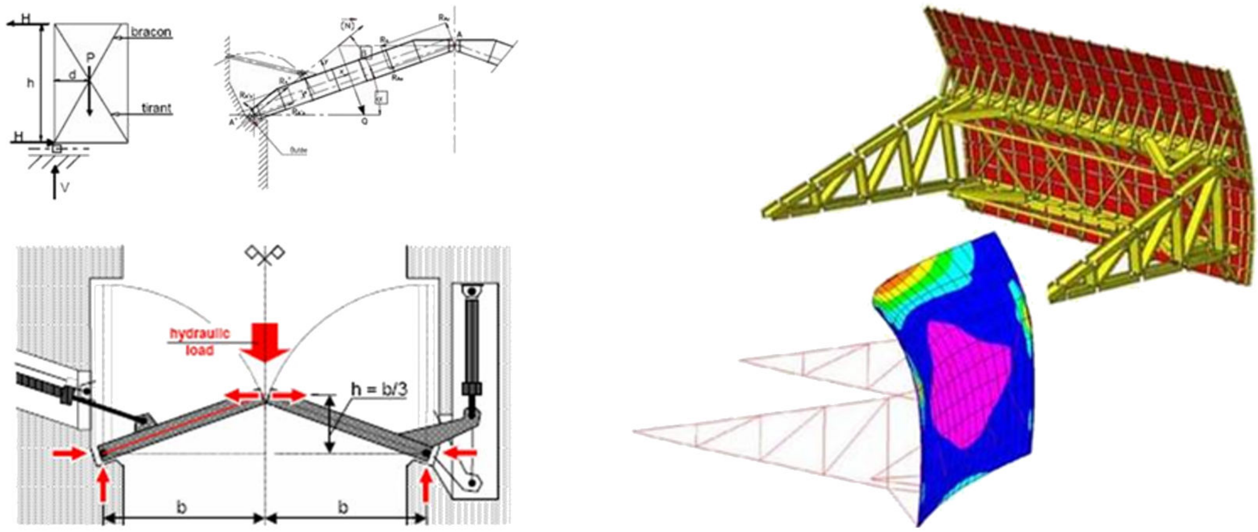


Figure 2.15: Left: First schematisation of mitre gates; hand or 2D software calculations; Right: Finite Element Method (FEM) calculations - 3D

Structuring and motivating calculations
It is strongly recommended to explicitly follow the following steps for every structural calculation:

1. Make a situation sketch by hand (cross-section) including water levels and ground levels;
2. Choose the most critical load situation (from the construction sequence) and sketch a load diagram by hand; motivate selected methods for estimating the magnitude of the loads;
3. Think of a way to resist the loads and figure out how they can be transferred to the foundation and further towards the subsoil, thus the resisting forces;
4. Make a hand sketch of the structure or element, indicating components, materials and connections (this can be in various shapes);
5. Make a mechanical scheme of the supporting structure or element; Motivate your assumptions and simplifications, for instance when considering only two dimensions instead of three;
6. Check the overall stability of the structure, including soil bearing capacity and settlements. Furthermore, check dimensional stability (*vormvastheid*). Consider both SLS and ULS conditions. Mention the source of the model or equation and motivate why it is applicable to the actual situation. If it is not (entirely) applicable: reason how the model can be changed, or find another approach to do the check;
7. Check the strength of the structural elements, considering potential failure mechanisms. Determine critical cross-sections and draw shear force and moment diagrams. Mention the source of the model or equation and motivate why it is applicable to the actual situation. If it is not (entirely) applicable: reason how the model can be changed, or find another approach to do the check;
8. Check whether deflections/displacements are acceptable (SLS);
9. Check whether the designed component fits in the entirety (connections, space);
10. Reflect on the outcome of the calculation: is the magnitude reasonable? Comparison with similar structures under similar conditions will provide information about the order of magnitude.

An example of a calculation according to these steps is provided in the Appendix to these lecture notes.

Often, the adopted work method 'from general into more detailed' calculations prevents situations where fundamental changes to the design are required due to 'details' that cannot be solved, but it does not work like that 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 storm surge 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 completely 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.

Memorise the above with the following slogan that is often heard in design land:

"*The DEVIL is in the DETAILS*"

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

2.6 Design phase 5: Evaluation and Selection of the best alternative

2.6.1 The purpose of the Evaluation phase

The Evaluation design phase aims at finding a right balance between the created values and the sacrifices needed to achieve these values. It thus evaluates the feasibility and acceptability of design concepts, to enable the selection of one preferred alternative.

The Evaluation phase succeeds the Verification phase, where potential design concepts are assessed against the requirements. It must be ascertained that the design alternatives meet all requirements at the moment of Evaluation, otherwise the comparison between alternatives would be 'unfair', because 'good' alternatives would have to compete with 'not fully matured' concepts. Even worse: it could lead to selection of non-functional solutions. The outcome of the Verification phase is a limited number of realistic design alternatives.

Finding the right balance between created values and sacrifices can be done in an intuitive way, but a more systematic and transparent way of evaluating by making a list of the advantages and the disadvantages per concept is a better basis for making a decision. If not all advantages and disadvantages have the same importance, they can get different weighting factors and can be compared with help of a multi-criteria analysis (explained below).

A difficulty in estimating the balance of created values and sacrifices is that not all values and sacrifices are (objectively) quantifiable. Furthermore, (part of) the values and sacrifices often reside outside the project boundaries. The feasibility of project alternatives can therefore be considered at different levels of economic focus:

- political science, where a multi-criteria evaluation is a suitable tool, but also broad societal debates can be used;
- macro economy, where cost-benefit analyses are used;
- micro economy, where cash flows are analysed.

The suitable type or level of evaluation thus depends on the scale of the project and its owner. Broad debate is usually appropriate to large-scale infrastructure, but for 'smaller' projects, a multi-criteria analysis seems to be more suitable because the marginal cost of the project would not balance with the marginal value obtained with this approach. Politicians include aspects like environment, health, culture, aesthetics, etc. in their considerations. Cost-benefit analyses include elements like growth, employment, currency and inflation.

Cash-flow only comprises financial profits and losses (Bakker, 2000).

2.6.2 The Multi Criteria Analysis

The Multi Criteria Analysis, is a method by which the relative merit of alternatives can be compared using a range of quantitative and qualitative criteria. It is a decision-making tool, used to evaluate problems when one is faced with a number of different design alternatives and wants to find the best solution regarding different and possibly conflicting objectives. The Multi Criteria Analysis is also referred to as multi-objective decision making, multi-objective decision support system, and multi-criteria decision aid. The Multi Criteria Analysis can be used in the Evaluation phase of an engineering or integrated design method.

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 euros, 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 Multi-Criteria Analysis 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.

The identification of evaluation criteria

In a multi-criteria approach, the values, or 'qualities' of the design alternatives are determined with the help of relevant qualitative criteria. The criteria are derived from the clients' wishes and stakeholders' interests, which are typically non-quantifiable. They usually consist of values such as human life, nature, culture, spatial quality, but involve more technical values as well, such as maintainability and disturbance to the surroundings by construction activities.

Section 2.3.3 already gave an overview of potential evaluation criteria. These criteria should all be clarified, because the meaning of the terms is often ambiguous and they can easily overlap. For instance, what is meant with 'sustainability' or 'spatial quality', and what is the difference between 'aesthetics' and 'visual integration in the surroundings'?

For a clear evaluation, the number of criteria should not be too large. It can therefore be useful to cluster several criteria by considering the characteristics of the assignment and the ideas of the designer and client that have led to the concepts, because these aspects indicate what is considered important for the solution.

It should be noticed that these evaluation criteria can be formulated as requirements, if the client considers them of high importance. For instance, vibrations during the driving of foundation piles can be disastrous for surrounding buildings. In that case, it should be formulated as a requirement that vibrations caused by construction activities will be limited to a certain extent. The limitation of the noise of pile driving can be defined as a requirement as well, rather than an evaluation criterion, if for example the duration of the total pile driving works is rather long and there is a hospital in the vicinity.

Costs should not be considered as an evaluation criterion, because the criteria represent values that are created, whereas costs represent the sacrifices that are needed to create these values. Values and costs are contrary aspects and should therefore be dealt with differently in the evaluation of design alternatives (see later in this section).

The difference in the importance of the criteria

Weighting factors are assigned to these criteria, because not all criteria are equally important. The client should be involved in estimating the weighting factors of the criteria and in scoring the alternatives. The designer could, for instance, present the table with unweighted criteria and discuss with the client what criterion should have more or less influence. The designer and client then immediately get a feeling of the sensitivity of the weights and scores.

The unconscious inclusion of criteria that have a more or less similar meaning leads to over-weighting of one aspect. For example, including 'sustainability', 'spatial quality' and 'integration in the surroundings' as criteria could give a similar aspect a relatively too high value (unless it is specified that these terms have different meanings). Under-weighting of aspects is possible as well, if too many criteria are grouped in one term. If, for instance ecological, social and cultural values are considered to be important aspects for the comparison of verified design alternatives, it is better to not group them as one criterion 'spatial quality'.

The relative importance of the criteria can be included with help of weight factors per criterion. This can easily be done by comparing the criteria in pairs in a matrix, as shown in the example below (Table 2.2). For each cell, it is indicated with a '0' or a '1' what criterion is more important. If the criterion listed in the header row is more important than the criterion in the header column, it is marked with a '1' in the intersecting cell, otherwise with a '0'.

row / column		a	b	c	d	e	Total
aesthetics	a		1	1	1	1	**4**
disturbance during construction	b	0		1	1	1	**3**
adaptability	c	0	0		1	0	1
user experience	d	0	0	0		0	**0**
environmental impact	e	0	0	1	1		**2**

Table 2.2: Determination of the relative weights of the criteria

After having filled out the entire matrix, the scores per criteria are summarised per row. A total criterion weight of '0' should be changed into '1', because despite it is not more important than any of the other criteria, it should still be part of the evaluation. Otherwise it should not have been chosen as one of the criteria. The other scores should then be multiplied with 2, to better distinguish the lowest weighting criterion.

The order of importance of the criteria can now be determined, followed by a calculation of the weighting factors per criterion, see Table 2.3.

Scoring the alternatives per criterion

Subsequently, the design alternatives get scores per criterion, for instance on a scale of 1 to 5, or 1 to 10. The scores can best be determined by comparing the alternatives. If, for example, alternative 1 would be quite good, but alternative 2 even better and alternative 3 only moderate, their scores could be 8, 9, 5 (on a scale from 1 to 10). The scores should all be motivated.

The total values of the alternatives can now be calculated by multiplying the scores with the weighting factors and summarising the products per alternative, see Table 2.4.

After having determined the total score, or value, per alternative, it is easy to compare them to each other. In the example, alternative 3 creates the most value. However, the costs have to be included as well to make a definite choice. This can be done by dividing the values per alternative by their costs. The result is a value-cost ratio per alternative, which is more or less a 'value for money' ratio. Net present value calculations are used to represent future costs and revenues.

			weighting factor
aesthetics	a	8	8/21 = 0,38
disturbance during construction	b	6	6/21 = 0,28
environmental impact	e	4	4/21 = 0,19
adaptability	c	2	2/21 = 0,10
user experience	d	1	1/21 = 0,05
		Σ = 21	Σ = 1

Table 2.3: Determination of the weighting factors per criterion

			alternative 1		alternative 2		alternative 3	
		WF	Score	SC * WF	Score	SC * WF	Score	SC * WF
aesthetics	a	38	3	114	7	266	7	266
disturbance during construction	b	28	6	168	3	84	8	224
environmental impact	e	19	5	95	8	152	6	114
adaptability	c	10	7	70	7	70	7	70
user experience	d	5	5	25	8	40	6	30
Total score				472		612		704

Table 2.4: Determination of the scores per alternative

Preferably, the net present values of the costs are used, calculated to present time (or the start of the project), as future investments or expenses have less influence. If it is difficult or time-consuming to make cost calculations, rough estimates can be made (be aware of the risks of making rough estimates). In our example, alternative 1 could be estimated at 100 fictitious monetary units. If alternative 2 would be about 30% more expensive, it could be estimated at 130 and if alternative 3 would be about twice as expensive as alternative 1, it gets a fictitious monetary value of 200. The value-cost ratios then are:

- alternative 1: 472/100 = 4,72
- alternative 2: 612/130 = 4,71
- alternative 3: 704/200 = 3,52

The selection of the best solution

After having performed the multi-criteria analysis, the best alternatives can be proposed to the client and the consequences of choosing an alternative can be discussed. In this selection process, the values obtained by the realisation of the project should be carefully balanced against the required sacrifices. The chosen solution has to be affordable, which means that the client has to have sufficient solvency (the degree to which the assets exceed the liabilities) and liquidity (the ability to pay short-term obligations). In addition, in a capitalist economy, the dividend of the project has to be competitive with other dividends in the economic market. It could be useful to consider the value and costs of alternatives during longer periods, because that could alter the attractiveness of each alternative. For instance, if not only construction costs are considered, but the costs of maintenance and demolishing or re-use are included as well, sustainable alternatives could become more interesting, if long-term trend changes within the lifetime can be foreseen. It should be noticed, that cash inflow and outflow have less effect, if they take place further in the future.

If the client likes the 'winning' alternative and is able to afford that solution, a new design cycle can be started, where more detailed calculations and drawings are generated for the subsystems within the system that was designed in the previous cycle. The objective and programme of requirements and all further design steps are then aimed at the subsystem and as a consequence, they will be more specific than in the previous design cycle. A new design cycle could also be used to re-consider the programme of requirements, for instance if the costs of the best alternatives appear to be unaffordable. To complete the design project, the final results have to be communicated to the client and other involved parties in a final report. Professional engineers document their solutions in such a way that they can be constructed or manufactured (Hertogh and Bosch-Rekveld, 2014, Delft Design Guide, 2013, Roozenburg, 1995 and IID, 2013).

The higher the value-cost ratio, the better the alternative. However, value-cost ratios give no insight in the absolute magnitude of values and costs. Therefore, basing the choice of a preferred alternative on only value-cost ratios is too simple. After all, two alternatives can have the same value-cost ratios, but different values and different costs.

The ratio of values and costs of the alternatives can therefore better be drawn in a graph, like in Figure 2.16, which shows the value-cost ratios of five alternatives. Alternatives 1, 2 and 4 in this graph have about the same value-cost ratio, but there are big differences in the absolute magnitudes of values and costs. The quadrants should be drawn with their origin in a chosen reference alternative for comparison with other alternatives. Other alternatives situated in quadrant I are always more interesting than the reference alternative, because they create more value for less money. Alternatives in quadrant III are always less preferable, because they create less value for more money. Alternatives in quadrants II and IV need more consideration and here the available budget could be a deciding factor. It could be helpful to consider the value and costs of alternatives during longer periods, because that could shift their position in the graph. For instance, if not only construction costs are considered, but also the costs of maintenance and demolishing or re-use, sustainable alternatives become more interesting.

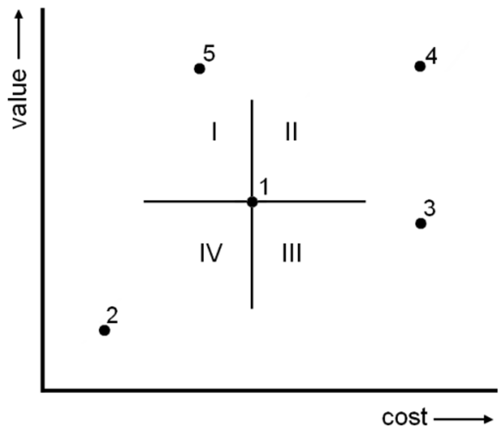


Figure 2.16: Example of a value-cost graph

If the client is not satisfied with the proposed alternative, or is not able to pay for it, four design optimisation moves are possible. Firstly, it should be checked whether the right evaluation criteria have been used with correct weighting factors. Secondly, it can be endeavoured to enhance the preferred, but not yet fully satisfactory, alternative by combining it with strong elements of other alternatives. Thirdly, it can be attempted to find possibilities for extra functionality (surplus value) that will generate more value and/or income. Fourthly, the design objective and the programme of requirements can be adjusted to make it less ambitious.

Points of attention
When doing a multi-criteria analysis, take good notice of the following:

- Requirements should not be formulated as evaluation criteria, because all alternatives should already comply to all requirements before they can be evaluated;
- 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.
- Sometimes, one criterion dominates or overrules all the others. In MCA-terms, such a criterion has a very high weighting factor compared to the other criteria. For example, military constraints (second bridge in the storm surge barrier of Saint-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.
- Evaluation criteria represent values that will be created or added by the structure or system, additional to what has strictly been required. Costs do not represent these values, but indicate the expenses that are needed to create these values. To not mingle values and expenses, costs should not be formulated as criteria. It is better to divide the total values per alternative by the total costs per alternative. This results in value-cost ratios that can be compared to each other. The alternative with the highest value-cost ratio is the best solution (if the right criteria, the right weighting factors and the right scores have been assigned).
- 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.

2.6.3 Cost-Benefit Analysis

The Cost-Benefit Analysis (CBA) in financial terms is a technique that enables the discounting of expenditures and revenues over time and normalises them 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 a cost-benefit analysis 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 cost-benefit analysis is more appropriate.

A cost-benefit analysis 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 cost-benefit analysis, 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. Table 2.5 illustrates the use of a cost-benefit analysis in a much later design stage for the selection of the type of gate.

Option / Costs (€)	Vertical lift gate	Sector gate	Suspended flap gate	Bottom flap gate
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

Table 2.5: Results of NPV computations used for gate selection for a new weir in the Meuse near Sambeek (4th PIANC WG26 meeting, Rotterdam, March 2004)

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 net present value.

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 quay walls and jetties, there are direct benefits like quay dues and (part of the) harbour dues. The trans-shipment activities will also generate indirect income to society in terms of employment, the possibility of added value to the goods that are trans-shipped, etc. For most public works, like weirs, storm surge barriers and public quay walls, however, there are no direct benefits like payments for use, for protection, trans-shipment 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. If people feel safe behind the dikes and people or goods are easy to trans-ship and to transport, the value of buildings increases and there is a positive attitude to (more) investments.

Evaluation model / evaluation instruction

Generally hydraulic structures require considerable public or societal investment on the 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.

2.6.4 Risk analysis

Taking care of risks during design: some notes

Risk is the product of the probability of an undesired event and its consequences (material and/or immaterial). 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;
- delays;
- malfunctioning 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.

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:

- quality assurance and quality control (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 storm surge 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 load factors for different piers. At that moment supporting steel structures were added to mobilise a sufficient amount of steel cross-section!
- The Eastern Scheldt storm surge 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 gates fails!
- If a quay wall is designed as a combi-wall, like a chain of interconnected sections of about 40 meter and each section can withstand the active ground pressure with help of more than two anchors, there is

much robustness or redundancy in that structure against ship collision, deterioration of anchor rods, local overloading by life loads, etc.

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 too high, 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 2.17 distinguishes three main components in a risk analysis:

1. 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.
2. The quantitative analysis, calculates the probability of failure, quantifies the consequences and calculates the risks.
3. In 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 analysed. 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 report 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 2.18. 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.
- 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 analysed.
- 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.

2.6.5 Life cycle approach

The main functional requirements come from the client as the main stakeholder during the expected long period of normal use. The client 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 multiple stakeholders are active, who may have extra requirements to the hydraulic structure (see Figure 2.19).

Example(s):

- The main user of a quay wall is the trans-shipment 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 up-gradability or re-usability or extra quality requirements in terms of sustainability or break-ability.

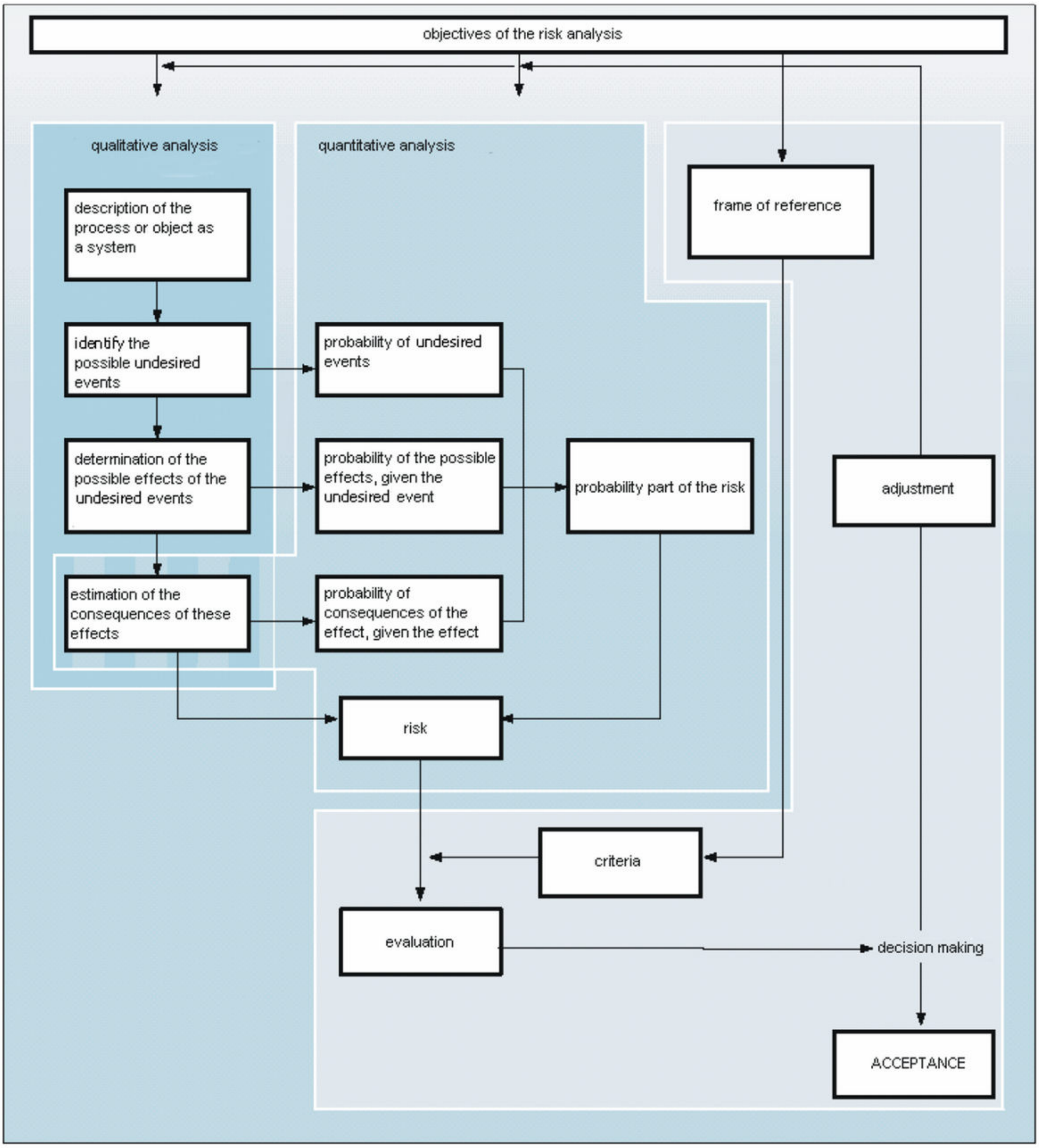


Figure 2.17: Components of a risk analysis, according to CUR report 190

- The local environment may have extra requirements for the aesthetics of the structure (old fashioned outlook, kneeling cranes?).

So, on the one hand anticipating at future life-cycle 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 disposable quay wall" and, opposite in the case of less uncertainty about future (re)use, this will lead to a very flexible and durable solution.

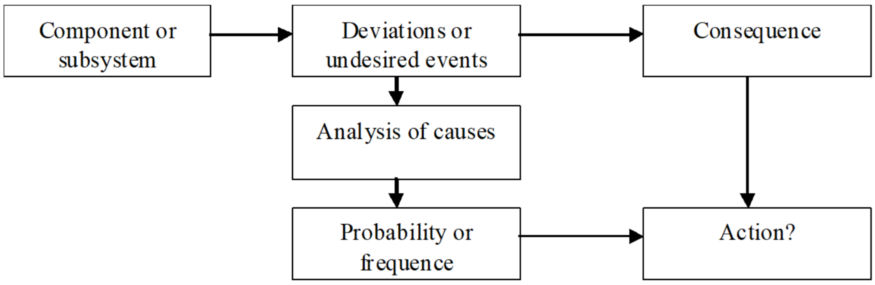


Figure 2.18: FMEA schematic approach (from CUR 190)

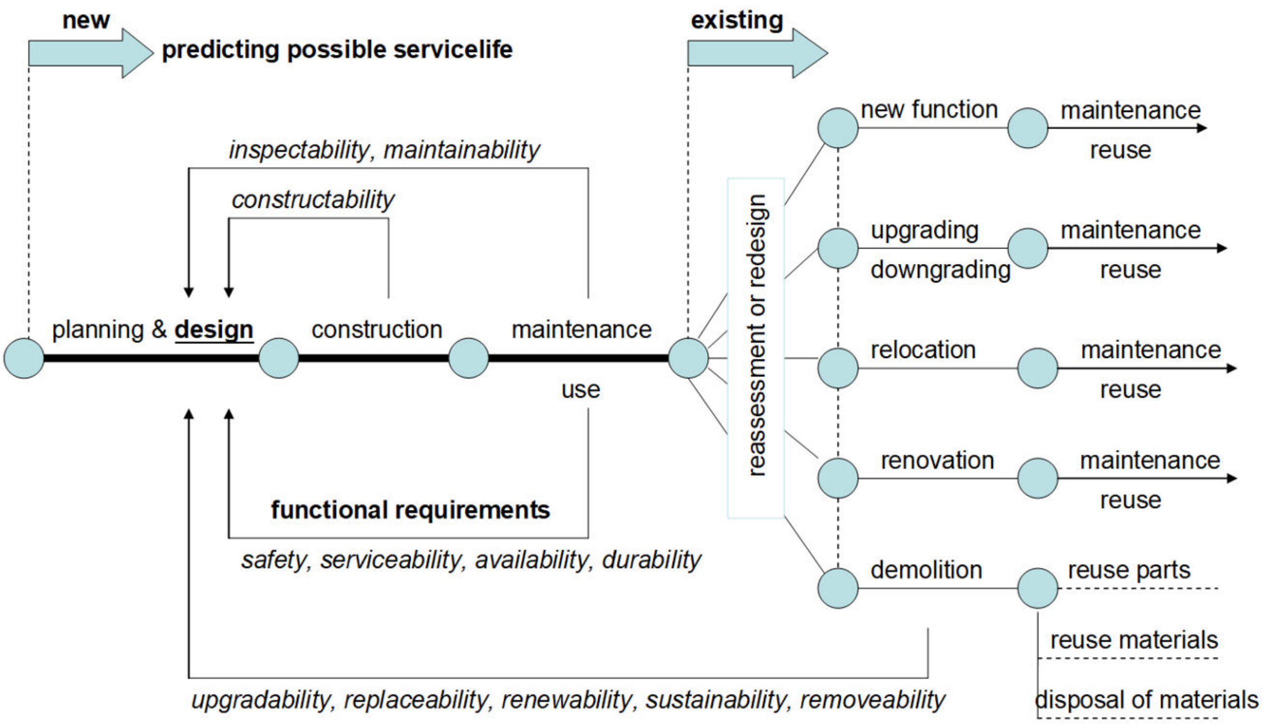


Figure 2.19: Life cycle stages and performance criteria (Van der Toorn, 2006)

2.6.6 Formalising the decision

In case of infrastructural projects, decision-making is regulated by law or other procedures. Everyone should be offered the possibility to consult and participate in the design project. This can be organised in different stages of the design. The decision is usually formalised by signing a legal document, according to which a next detailing loop is started, or the design project will be finalised. In the last case, the final results will be communicated to the client and other involved parties in a final report. Engineers usually document their solutions in such a specific way that they can be constructed or manufactured.

If the client agrees upon the proposed alternative and is able to afford that solution, a new design cycle can be started, where more detailed calculations and drawings are generated for the subsystems within the system that was designed in the previous cycle. The objective and programme of requirements and all other phases then aim at the subsystem and they are more specific than in the previous design cycle. A new cycle could also be used to re-consider the programme of requirements, for instance when the costs of the best alternatives appear to be unaffordable.

2.7 Design phase 6: Integration of subsystems

Because this engineering design method, as described in this chapter, can be used at detailing levels where several subsystems can be developed simultaneously, the solutions for the discriminate subsystems and components will have to be integrated into a complete, functioning system. If the external and internal interface requirements were formulated and verified correctly, the integration of subsystems into a working system should be no problem. However, if interface requirements are not met, it should be noticed and adjusted when putting the pieces of the puzzle together.

Total costs, spatial aspects and planning have to be considered as well. Technical drawings indicate how the structure has to be constructed, material quantities and equipment should be estimated, and a construction planning should be prepared.

2.8 Design phase 7: Validation of the result

It is recommended to subsequently carry out a final check on the validity of the entire design, whether the design objective has been adequately formulated and correctly translated into requirements. Validation is the confirmation, obtained through objective evidence, that the system will perform its intended functions.

Validation is a check whether the right system has been designed, whilst verification is a check whether the system design in itself is correct. If the requirements do correctly define the desired system, and the system is verified against the requirements, the resulting system will fulfil its purpose. Part of the validation is a check whether the used requirements and criteria are applicable to the design and a judgement whether the used design method is suitable for its purpose.

In practice, the Validation becomes more important because contract managers presently tend to have a poor sense of the reality. It is therefore proposed to add this phase to the usual design sequence in real practice.

3

DESIGN AND CONSTRUCTION

3.1 Design for construction

Hydraulic engineering structures are situated in, on, under, adjacent to and above water bodies. 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 larger 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 mottos in hydraulic engineering therefore is:

"*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 a large influence on design. Design and construction cannot not be considered separately.

Notice 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, if 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).

For a jetty the (prefab) piles can be driven into position using a crane and flying hammer working from a barge or pontoon and several 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

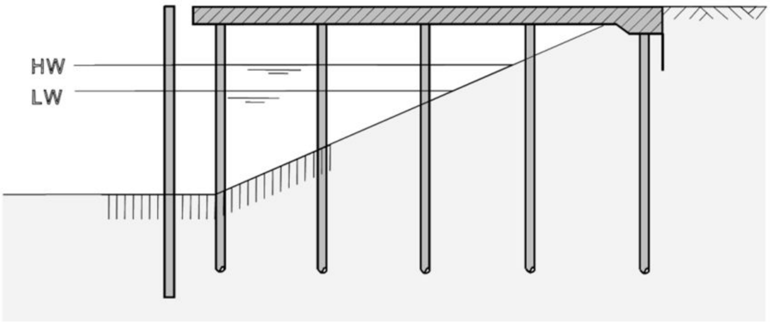


Figure 3.1: Construction above water - principle of a jetty or open berth quay

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 exceedance 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.);
- 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 in the 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 much space. Alternatives can be used for the construction pit, if there are objections to this (space required, dewatering damage). It is possible to divert the entire watercourse or part of it before starting the work as well, in which case 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 Euphrates in Babylon (presently in Iraq). Figure 3.2 and Figure 3.3 give random examples of diversion/widening after completion of construction works besides the water course.

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.

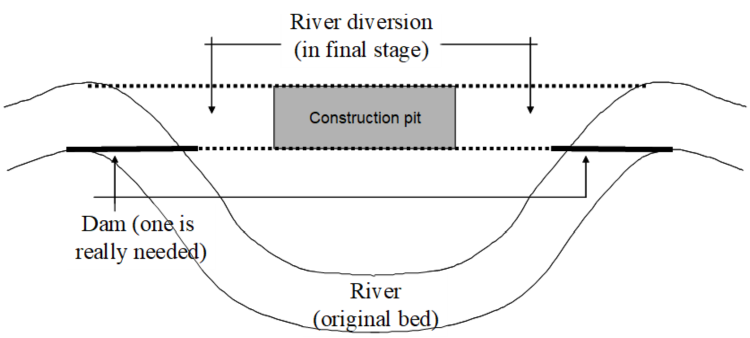


Figure 3.2: Pit outside the watercourse and (later) diversion to connect the lock

For the construction of the quay wall of Figure 3.3, only a limited construction depth and a limited reduction of the groundwater level were required. 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. It must be emphasised that the L-wall and the gravity wall are only two of the many types of quay wall structures.

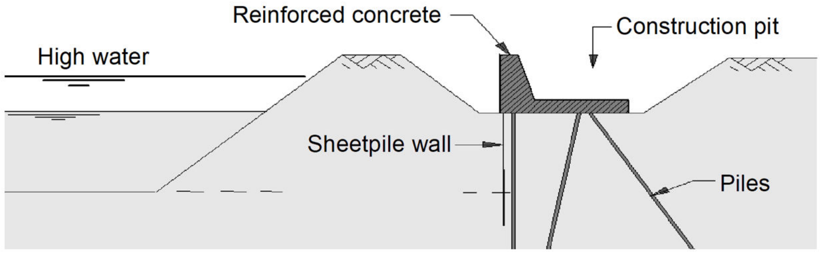


Figure 3.3: Construction pit for a quay wall

These 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 regarding 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 earlier, but not later than that.

3.2.3 Construction in the watercourse within a temporary retaining structure

A 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.

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, can be 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 surrounded by a dike
- cofferdam sland using sheet pile walls
- peninsula cofferdam using sheet pile walls

Example: Construction pit in the water, surrounded by a dike

It was decided to construct a dam for flood defence 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 discharge sluice was constructed as part of the dam. The construction pit in fact is a polder in the middle of the Haringvliet, see Figure 3.4.

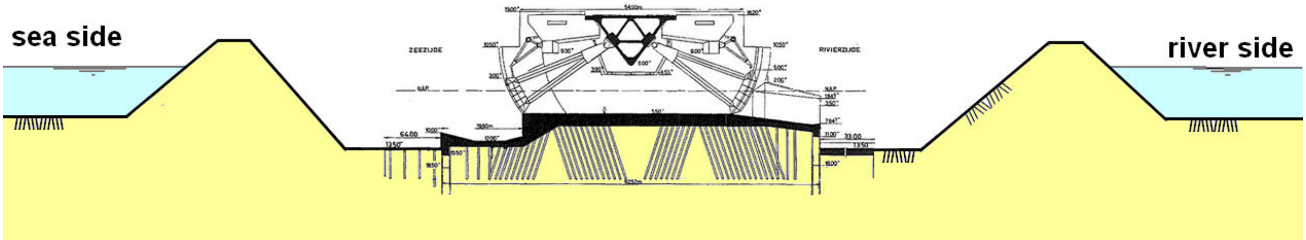


Figure 3.4: Construction pit in a waterway: discharge sluice in the Haringvliet (schematic, not to scale!)

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. 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 dewatering 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.

Figure 3.5: left: Construction pit (period 1958-1968) (Beeldbank RWS);
right: overview Haringvlietdam (Google Earth, 2022)

The channels were deepened, especially north of the island, while the construction pit was still there. After completion of the structure, channels were dredged to provide access to the discharge sluice in the shallow parts and the ring dike was removed. The current velocities declined as a result of the increased wet cross-section of the Haringvliet. 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.

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).

The sequence of construction is:

1. driving the sheet pile walls from a barge with a crane on top of it;
2. placing the struts;
3. excavation under water;

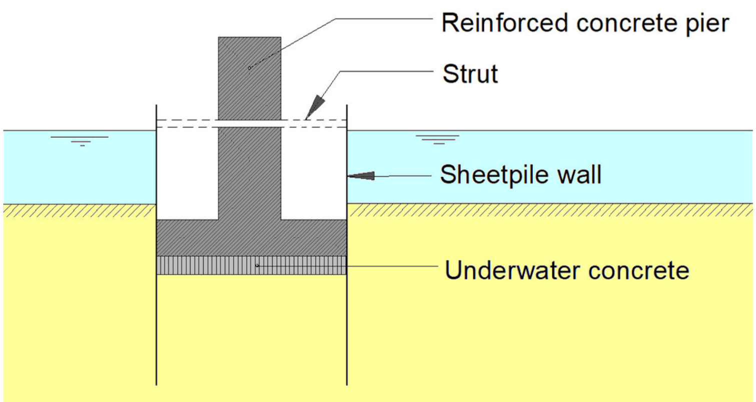


Figure 3.6: Cross-section of a cofferdam for a pier in a river

4. casting the underwater concrete floor (If necessary, this can be anchored to the previously driven sheetpiles to prevent heave as a result of the water pressure);
5. pumping dry the cofferdam;
6. construction of the piers;
7. filling the cofferdam with water;
8. removal of the struts and sheet pile walls;
9. The sheet pile walls can then be jacked just above the pier foundations by divers if an anchor between (underwater) concrete and the sheets was used, otherwise the whole sheet pile wall can be pulled out.

A sheet pile wall is vulnerable to collisions, so, the safety of the people who are working within the cofferdam is at stake. 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, 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 around 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.

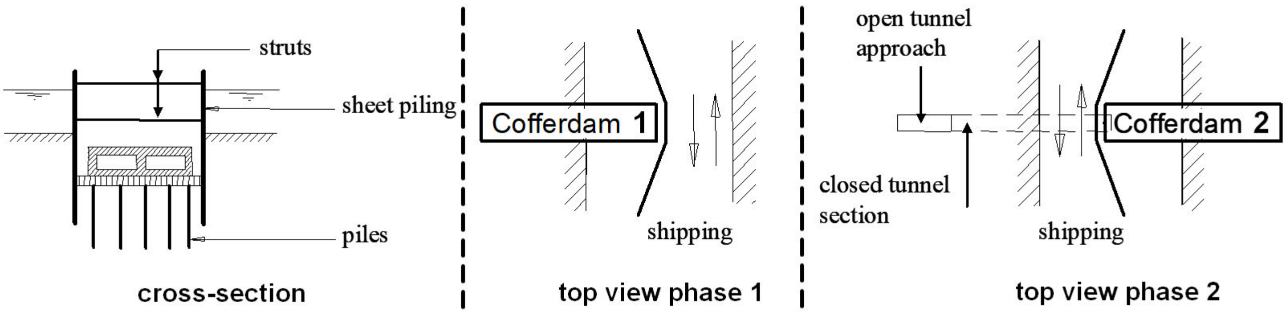


Figure 3.7: Cofferdam constructed in-situ for a tunnel

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 around the part of the tunnel that has already been completed. This is not a simple matter, but it will not be considered here.

The narrow passage increases the likelihood of ship collision with the cofferdam. Guiding or 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 several other places, but it does lead to a closed tunnel element that is longer than would have been necessary for crossing the waterway with its original width.

3.2.4 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 depends on the period during which the retaining structure is used and 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 dike around the construction pit in the Haringvliet was designed for sea levels with an annual probability of exceedance of 0,02 (construction phase), whilst for the discharge sluice in the Haringvlietdam an annual probability of $2{,}5\text{x}10^{-4}$ was used (operational phase). It is not possible to give a general rule for this and each case must be individually investigated.

Other issues 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

There is no prefabrication without in-situ construction, but there are many projects were prefabrication 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 tunnelling. Since the tunnel elements are manufactured outside the watercourse, immersed tunnelling 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 elements that were to be immersed. After the last tunnel element 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 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, with similar natural conditions, still completely different construction methods were used for large parts of the works:

The island like the construction pit for the Haringvliet discharge sluices could only be constructed because the estuary was wide enough. Sufficient wet cross-section remained available 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 (about NAP - 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 discharge 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 sandbanks.

The prime reason for closure of the Oosterschelde was protection against 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 Eastern Scheldt 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 designed later, large-scale prefabrication would have been considered for the construction of the outlet sluices, because the technology of prefabrication has been 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

IN-SITU CONSTRUCTION

For construction the most important task is to make ‘dry’ what otherwise would be ‘wet”. In this chapter the following in-situ construction methods aiming at that objective are considered:

- construction pits;
- cofferdams;
- geotextile / membrane;
- cut and cover method;
- pneumatic caissons;
- bored tunnels.

All these cases involve methods in which at least the following two requirements 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. **Structural safety** This concerns vertical, horizontal and rotational stability as well as strength: 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.); and strength of the structural components (walls, roof, piles, etc.)

4.1 Construction pits

4.1.1 Introduction

The difference between a construction pit (*bouwput*) and a cofferdam (*bouwkuip*) 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 combi-walls). 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 cofferdam, as an alternative option for the construction pit, is discussed as well in this chapter.

4.1.2 Construction sequence

The main sequence of construction stages is as follows (Figure 4.1):

- installation of the dewatering system;
- excavation;
- installation of piles from the bottom of the construction pit, if it is not possible to use a shallow foundation;
- installation of seepage screens to prevent piping, if necessary;
- installation of a concrete blinding (*werkvloer*) 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.

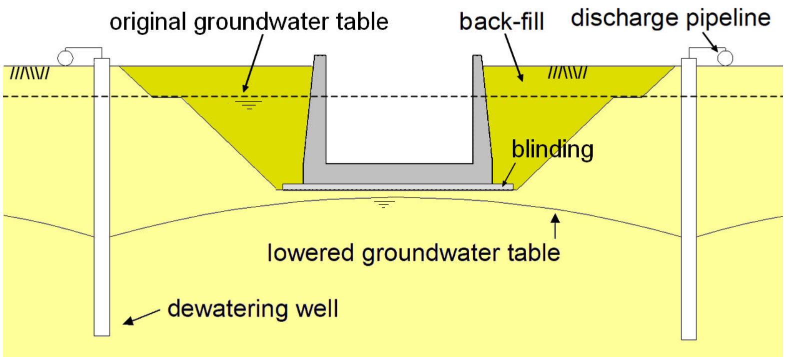


Figure 4.1: Cross-section of a construction pit with natural slopes

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.

4.1.3 Dimensions

With regard to the depth and dimensions of the bottom of the construction pit, the following should be noticed. 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 dewatering 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. The smoothness and the height of the bottom of a dredged pit 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 larger 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.

4.1.4 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 one 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.

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 can 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.

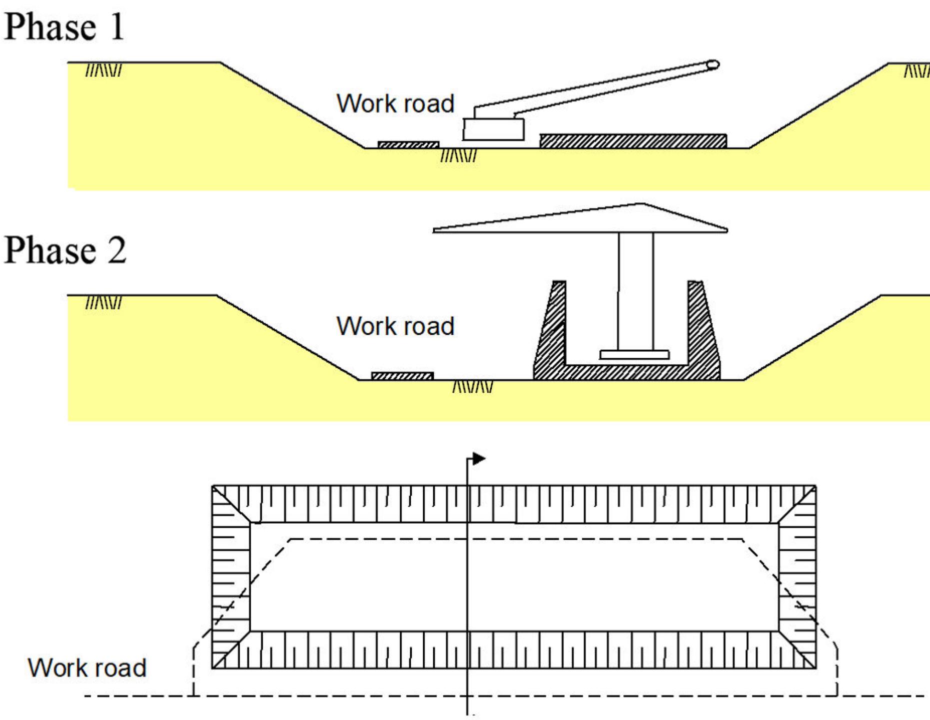


Figure 4.2: Construction in a construction pit

4.1.5 slope stability

The slopes of a building pit are designed to be as steep as possible to limit the amount of dredging and acquisition of land. The more quickly, steeper and higher a slope is constructed, the larger 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.

4.1.6 Dewatering

Nowadays, it becomes impossible more often to use dewatering installations, because of the disadvantages of this process:

- 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 Figure 4 3. During the construction of the aqueduct for the crossing of the A4 by Haarlemmermeer Ringvaart (ring canal) the canal was temporarily 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 4.3) the dike of the ring canal subsided as a result of which settlement occurred. The rotation of 1:10 is very much larger than what is considered acceptable (1:300).

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

- return drainage (*retourbemaling*)
- impermeable screens

With return injection the water that is pumped out is returned into the ground further away to maintain the groundwater level there. 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 noticed that return water always requires stronger pumping. The shorter the distance between the dewatering

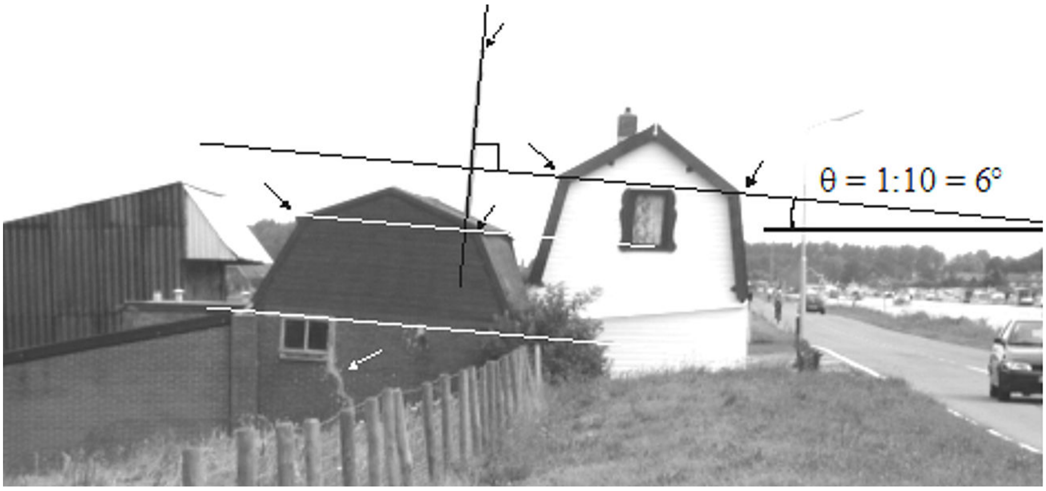


Figure 4.3: Settlement of a house (rotated) due to dewatering a construction pit

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, if 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).

4.1.7 Alternatives for the construction pit

The principle reasons for using a cofferdam 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).

In addition to these two points, more requirements may a part during the construction, such as:

- 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 Cofferdams

One of the most frequently used alternatives for the construction pit is the cofferdam. It consists of vertical walls and a horizontal sealing. 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.

4.2.1 Sealing layer

Fortunately, in the Netherlands and sometimes elsewhere, there is a good sealing clay or peat layer beneath bottom of the pit. In that case the options marked A and B in Figure 4 4.4 can be used when making a construction pit. With option 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.

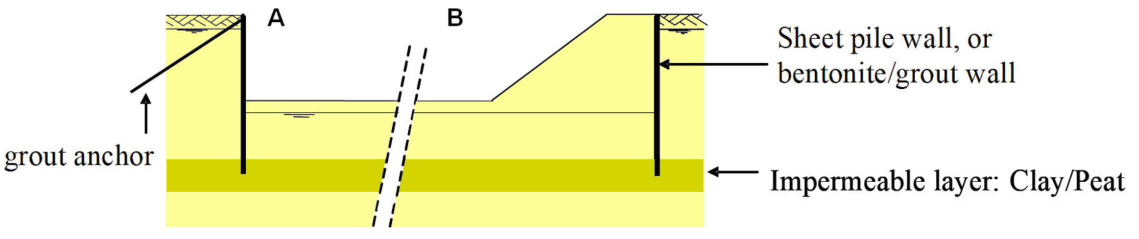


Figure 4.4: A natural sealing layer (Cofferdam - left; Construction pit - right)

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 ($k = 10^{-4}$ to 10^{-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 4.4A, 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.

4.2.2 Underwater concrete slab

If there is no natural sealing layer, usually an (underwater) concrete blinding (*werkvloer*) is anchored to tension piles as shown in Figure 4.5.

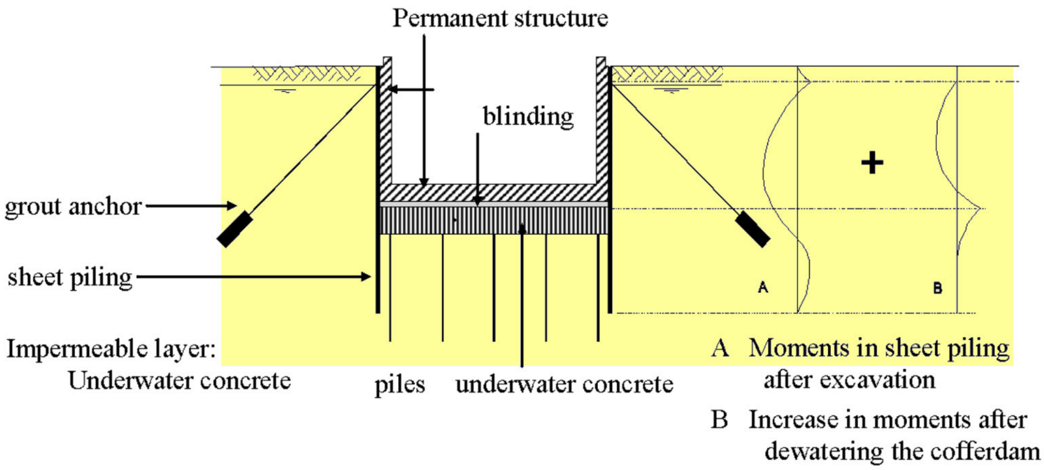


Figure 4.5: Cofferdam with underwater concrete slab

4.2.3 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 components that are briefly described below.

4.2.4 Vertical wall (sheet pile wall)

The vertical wall usually consists of sheet pile walls but sometimes of combi-walls or diaphragm walls. During 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 4.5).

To resist the horizontal water pressure, the sheet pile is supported by struts, or anchors, and the underwater concrete floor (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).

4.2.5 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.

4.2.6 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 Willemsspoortunnel).

4.2.7 Piles

Usually, pre-stressed 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.

4.2.8 Horizontal sealing

The easiest way to obtain a horizontal seal is to use an impermeable soil later (clay or peat). The leakage water has to be removed by a temporary or permanent pumping 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.

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.

4.2.9 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 of cofferdams (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 several 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. Disconnecting the welded-on steel transverse reinforcement is very expensive.

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, because of potential silt inclusions, uniformly good concrete covering of the reinforcement, etc.

Instead of using 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).

4.2.10 Cofferdam alternatives - no tension piles

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 uplift (Figure 4.6).

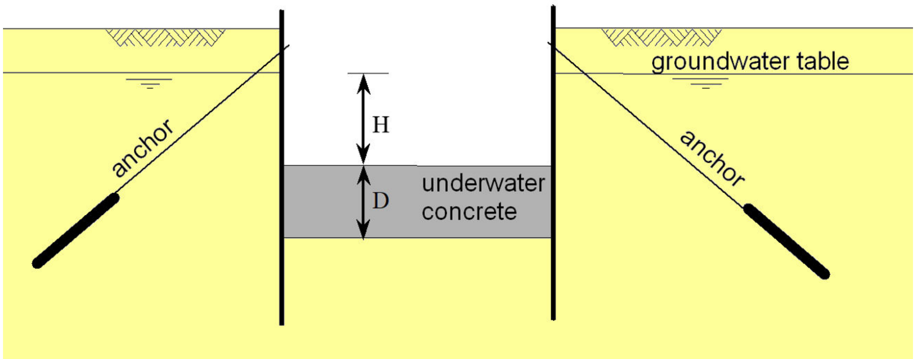


Figure 4.6: Cofferdam with unanchored concrete slab (gravity slab)

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.

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 (Tram tunnel Den Haag) but this concept does deserve further consideration

4.3 Membrane structure

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 4.7). A PVC or polyester membrane (1 mm for example) is laid and then covered by a layer of sand on which the road is constructed. A drainage system ensures that the water level above the membrane (precipitation and possibly leakage 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 to maintain the water level in the cutting at the same as the ground level. After the membrane has been immersed 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 factor 1,1 to 1,2).

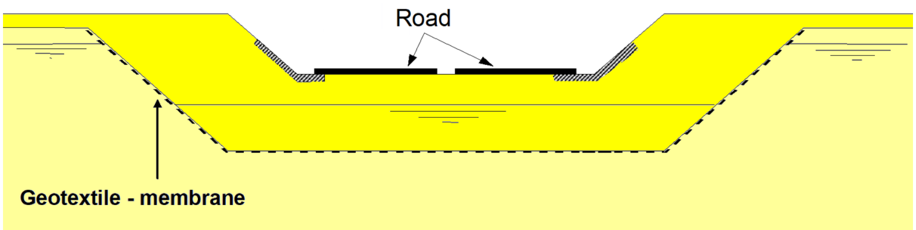


Figure 4.7: The use of a membrane for construction)

4.4 Cut-and-cover method

This method was developed for construction of metro lines in towns, like in Milano. Diaphragm walls are constructed, working from a shallow trench. The diaphragm walls are covered with 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 4.8).

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 (as in Figure 4.8)

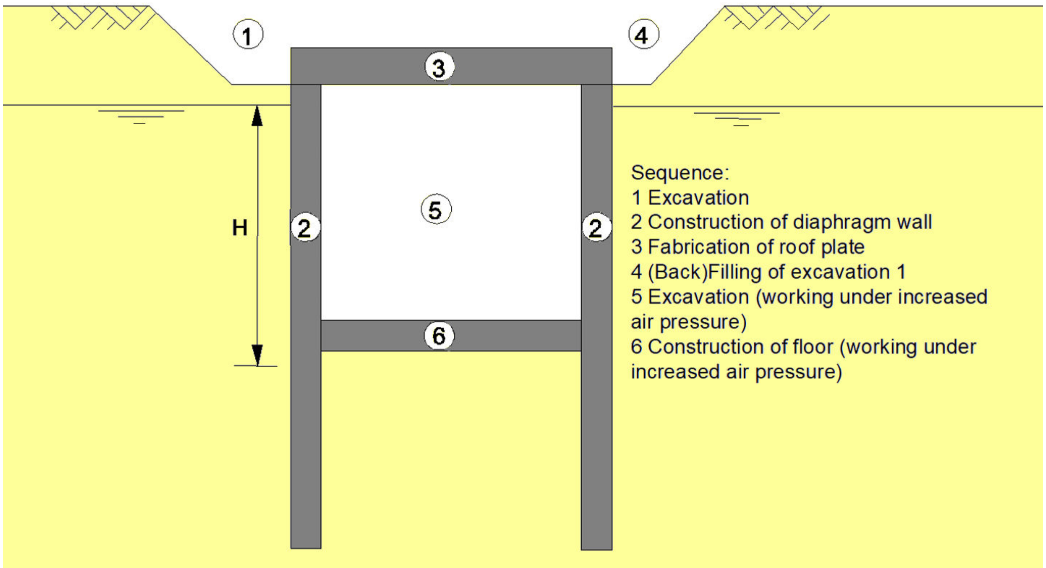


Figure 4.8: Cut-and-cover method)

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 Drechttunnel 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. 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 method

Pneumatic caissons are also used to build structures without the need for dewatering and which require little space. This method has been used for the metro line in Amsterdam and its use has also been 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 pneumatic 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 realistic alternative. For the entrance shaft of the Westerschelde tunnel, a 20 m high caisson of 20 000 ton was chosen (Figure 4.9). The heads of the new IJmuiden sea locks consist of pneumatic caissons with 4 m thick floors and 7 m thick walls. The dimensions of the largest caisson are L x B x H = 81 m x 55 m x 31,8 m.

A pneumatic 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. The side edges of the work chamber are projected slightly outside the profiles of the structure to limit soil friction during the immersion (Figure 4.10).

To further reduce the friction, 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.

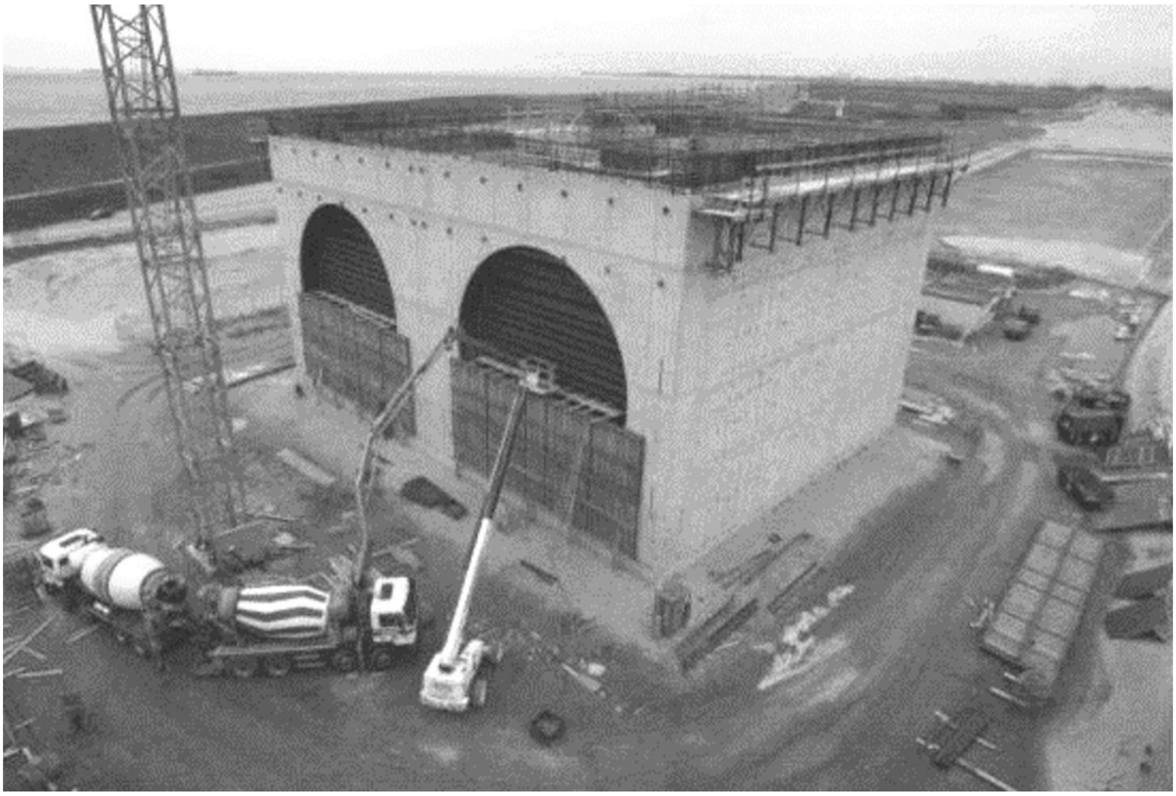


Figure 4.9: Entrance shaft of the Westerschelde tunnel constructed as a caisson at ground level)

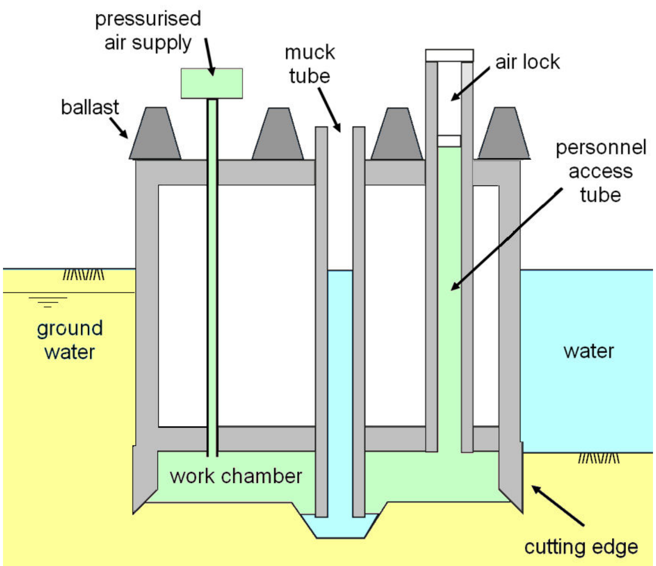


Figure 4.10: Pneumatic caisson cross-section/ Left: subsided into soil, Right: subsided under water (modified from Wikimedia Commons)

4.6 Bored tunnels

4.6.1 General

The great advantage of a bored 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.

Bored 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 bored 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 element 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, 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 bored 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 boring. 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 boring methods:

- fluid shield (hydro shield or slurry shield) (BS);
- earth Pressure Balance (EPB);
- pipe jacking;
- other.

4.6.2 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 (Figure 4.11).

The fluid pressure q must be considerably higher than the groundwater pressure p: ($q > p + 30$ kPa) to ensure that the drilling face does not collapse (blow-in), because in that case the excavation wheel becomes stuck and the ground above the tunnel caves in. However, 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 elements that are transported to the required position by rail.

4.6.3 Earth pressure balance shield

The earth pressure balance (EPB) 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 (Figure 4.12). The soil that is scraped off moves via holes in the excavation wheel into the excavation chamber.

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 become seriously contaminated. The advantage of this

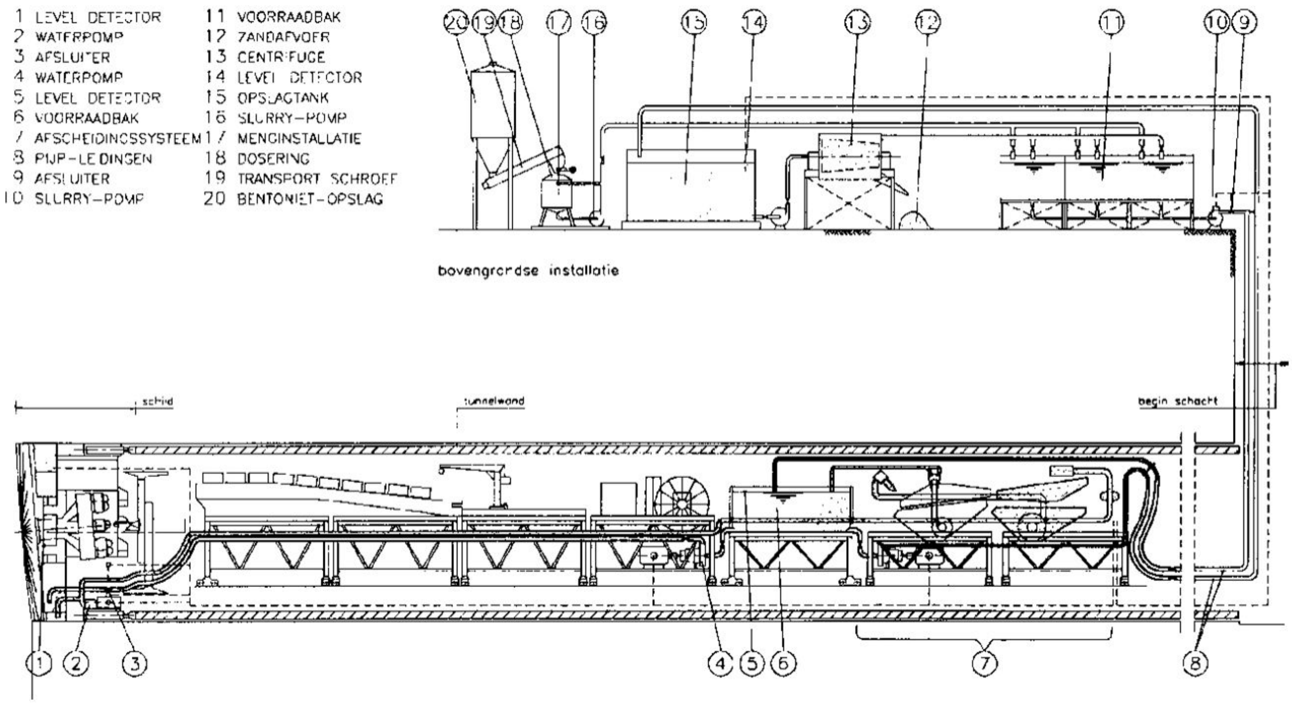


Figure 4.11: Fluid shield tunnel boring machine

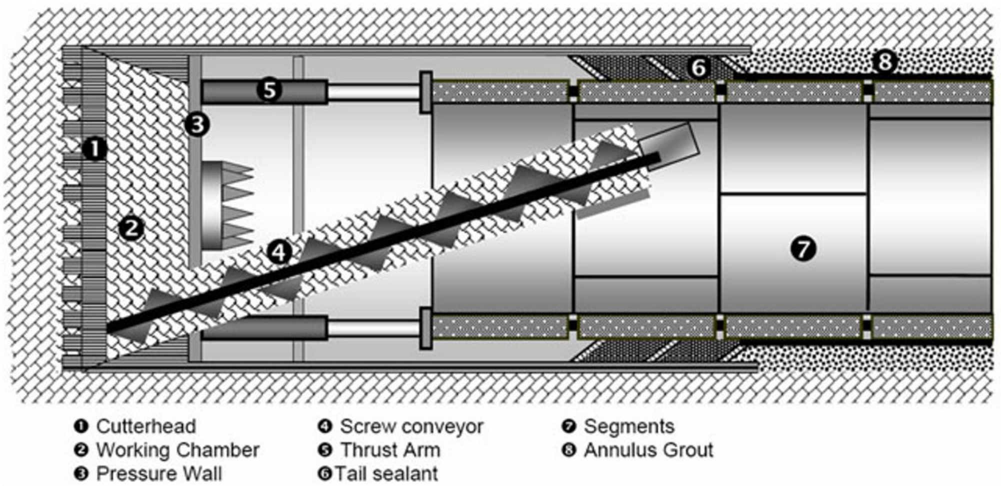


Figure 4.12: Earth shield tunnel boring machine with diagonal auger (www.p3planningengineer.com)

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 larger 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.

4.6.4 Pipe jacking

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

4.6.5 Other tunnel boring 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 boring machine, in which case an airlock is needed. 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

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 towed over water to the construction site. A construction dock is financially attractive, if little area is available at the construction site or if lowering the water table would cause damage to the surroundings.

For floating transport, use is made of the buoyancy of the sections. If 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 elements on submersible or non-submersible steel barges or transport vessels.

Installation, for example the positioning of self-floating elements on the bottom, can be carried out by the addition of ballast. Water that can be quickly admitted, is often used for this purpose. It shortens the time needed and reduces hindrance, for example to shipping traffic. The temporary water 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 element (that is not entirely self-buoyant) into position from the pontoons.

Connecting the elements 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, which is, in its use phase, entirely located 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, among other things, possible for:

- tunnels and underwater bridges;
- turbine-houses and outlet sluices for tidal power stations;
- the extension of existing locks;
- dams.

Immersed tunnel
A typical Dutch method of tunnel construction is based on immersion of tunnel elements. The tunnel elements are prefabricated in a construction dock, often a construction pit. Each element 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 dam closing off the dock must be removed by dredging, or if the dock is equipped with a gate, it has to be opened (Figure 5.1).

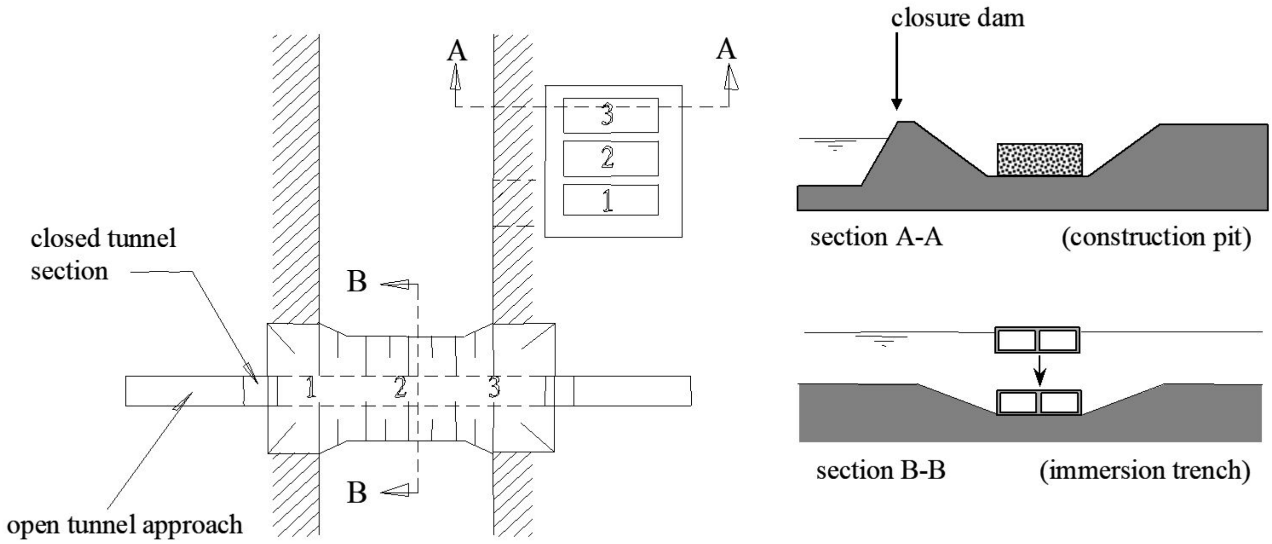


Figure 5.1: Construction site for an immersed tunnel

The land elements (open entrance plus connected closed tunnel) are built on each bank on dry sites at the same time as the floating elements. 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. Tug boats transport elements successively from the construction dock to the required position. Here, the elements are ballasted (water in tanks which are inside the element), as a result of which they immerse 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 element. The immersion must be controlled and accurate. The elements 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 becomes larger than its buoyancy capacity) the element immerses vertically, suspended on four vertical cables which are controlled by winches on the pontoons that remain floating (Figure 5.2).

The temporary ballast (the water in the tanks) is replaced by the permanent ballast after the tunnel elements have been positioned: for example, by a layer of un-reinforced concrete 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 hard-core 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

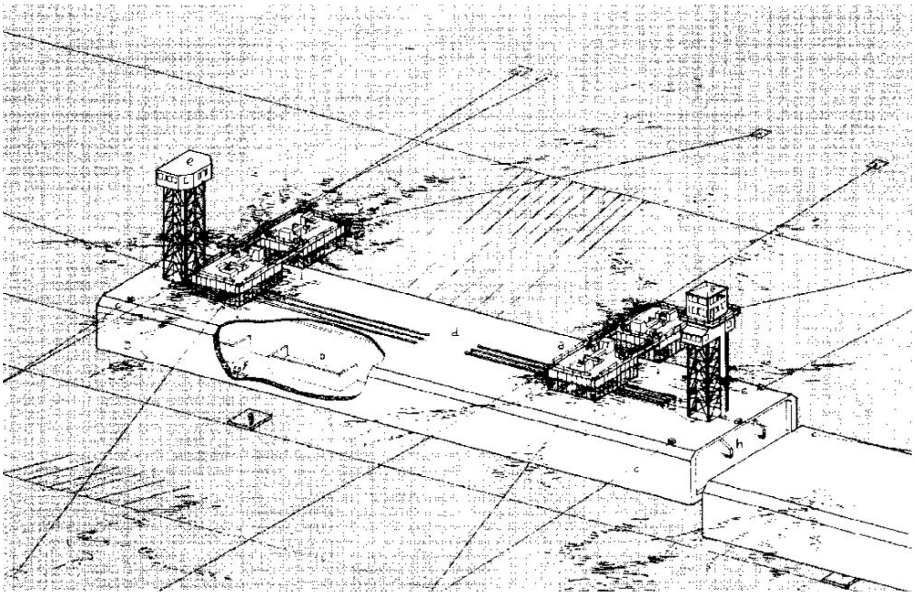


Figure 5.2: Immersion with the aid of pontoons and cables

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 larger distance from the site of the tunnel. However, it is necessary for the access route to be deep enough for the tunnel elements, 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 elements. 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 influences the design to a large extent. Large-scale prefabrication imposes many extra requirements:

- transport
  - sufficient buoyancy
  - stability during the transport
  - means of connecting to other elements
- 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 elements) 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. Elements 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

5.2.1 Self-propelled floating transport

Weight and buoyancy capacity

Already a distinction has been drawn between floating and non-floating elements as well as between elements 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 5.3. The tunnel is at the bottom of the trench and is founded on injected sand fill.

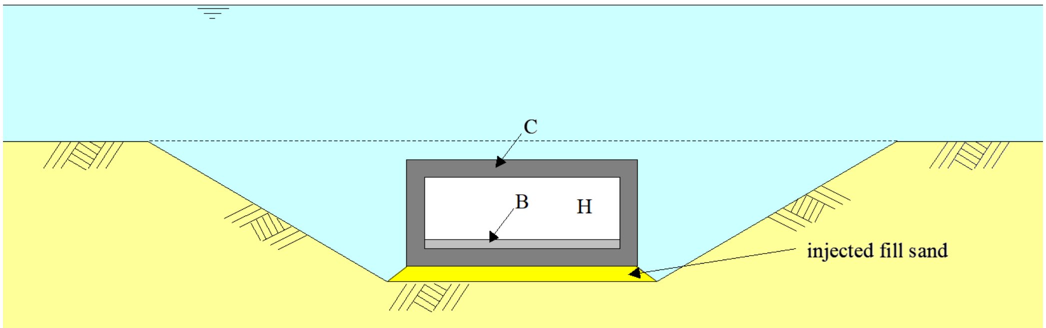


Figure 5.3: Cross-section of a just immersed tunnel on a sand fill. C = concrete volume; B = volume of ballast material; H = volume of he empty space in the tunnel.

For the determination of the proportion between the total cross-sectional area, the empty space and the area filled with concrete, two situations should be considered:

1. During floating transport: When an immersed tunnel element 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.
2. After immersion on the bottom (before the filling of the trench): The weight must now be larger than the buoyancy, say 8% more.

From the necessary hollow or free area, the chosen construction method and the limits set by these (freeboard, over weight), the amounts of construction and ballast concrete can be determined.

In the following loop 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, 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 pre-stress 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 element 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.

Large horizontal loads can occur in many structures. For example, after the placing of the piers of the storm flood surge barrier in the Oosterschelde, it was necessary to take wave loads into account. 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 (horizontal stability);
- Resultant of forces within the core of the basal area of the caisson (rotational stability);
- Foundation (grain) pressure under the basal area of the caisson not higher that the permissible value (vertical stability / bearing capacity).

It should be noticed that for the last two criteria this concerns a quick preliminary approach: larger 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 if 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 with 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 would be 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.

5.2.2 Transport by floating crane

The possibilities of using floating crane 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 large distances. The parts of the bridge over the Great Belt were made in Portugal, where it was 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 not allowed by the client. Figure 5 4 shows the floating crane and its arrival in Sweden with a complete bridge span.

A floating crane, named ‘Ostrea’, was especially constructed for the installation of the piers of the Ossterscheldt storm surge barrier (Figure 1 45). It was a U-shaped vessel with two portals, of 36 and 24 m height, for hoisting (*hijsen*) the piers. The lift capacity was 10 000 tonnes, while the piers weighed 18 000 tonnes.

Figure 5.4: Floating crane Svanen (Ballast Nedam) with bridge girder for the Westbridge over the Great Belt, Denmark

5.3 Foundations

The difference between in-situ and prefab construction with regard to the foundation
When construction in situ, two types of foundation are possible:

- on a shallow foundation, directly on the subsoil;
- on piles.

With a shallow foundation, first a concrete blinding is laid on the bottom, and on this, 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 elements, 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 elements. The requirements are as follows:

- a good transfer of forces to the subsoil, thus sufficient contact area between the bed and bottom of structur;
- the slope of the bed where the prefab element has to be positioned should suffice to what is required, see Figure 5.5;
- settlements may not lead to unacceptable deformation of the finished structure unless adjustment options are available.

Example: Prefab Caisson
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 5.5A, 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 to even out the apron and to provide a vertical berth face for safe mooring of ships.

Figure 5.5B shows an option that is often used to correct the wrong slope or angle of a caisson. After positioning lower 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. In this way, the deviation from the right slope angle of the bed can be counteracted. Even in case of a bottom bed of the right slope angle, large caissons cannot always be accurately positioned in the horizontal plane, e.g. due to unfavourable weather conditions during the immersion 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.

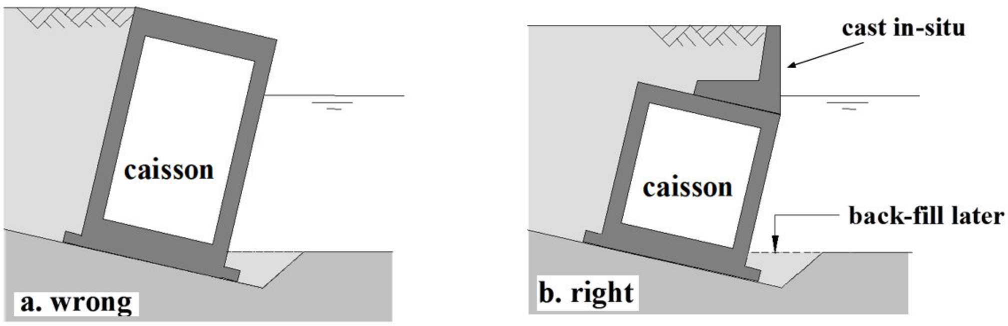


Figure 5.5: Construction error in the positioning of a quay wall-caisson (exaggerated)

Notes

- 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 5.5B 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 issues 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 the bottom as smooth as possible
It is very difficult to obtain a smooth finish to the bottom, especially if 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 the caisson example 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 immersed tunnels in the USA.

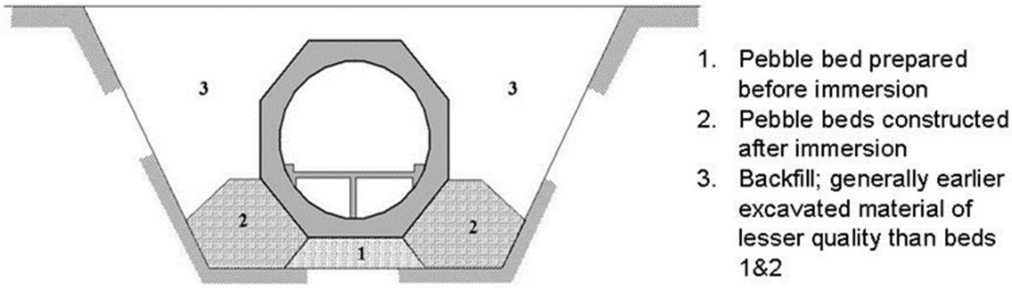


Figure 5.6: USA-type immersed tunnel

In Figure 5.6, the slopes of the trench are too steep (at least for Dutch soil conditions). The tunnel elements 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 elements, 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 element. 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 450, 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 element - 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), this is 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 5.7is 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 element touches one point on the bottom the element. Here it is assumed that the bottom slopes evenly in one direction, but actually the bottom may have a much more uneven profile. The tunnel element will tilt, if ballasting with water continues to be equally distributed over all the compartments. 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 element into the bottom in that position.

When more ballast has been 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

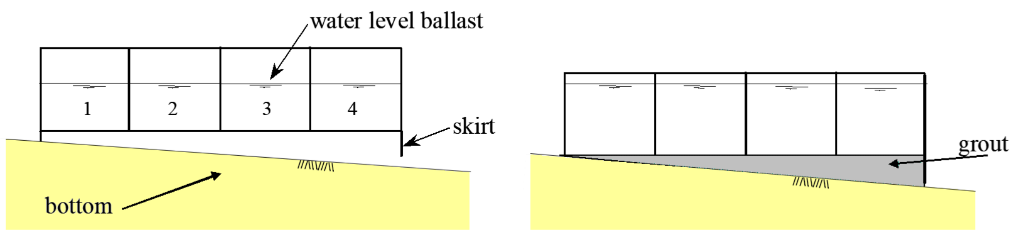


Figure 5.7: The principle of levelling

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 element 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 element 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 element to the underside of the skirts. In this way irregularities in the surface, possibly with lower values of the internal friction angle ϕ, 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 5.8. 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), to avoid high upward wave impacts. Pipelines leading to and from the seabed run through the columns. The box structure (105 x 105 x 32 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 levelling are steel sheet pile screens.

The construction sequence was as follows:

1. The skirts and part A, apart for some of the partitions and the roof of A, were pre-fabricated in Europoort in a construction dock with a dewatering facility. Part A was not entirely finished, to reduce the costs of construction in the dock.
2. After the filling of the dock and dredging of the access channel through the dam between the dock and open water, the element 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 element was then towed to a deep fjord in Norway, during which the upper section of part A was

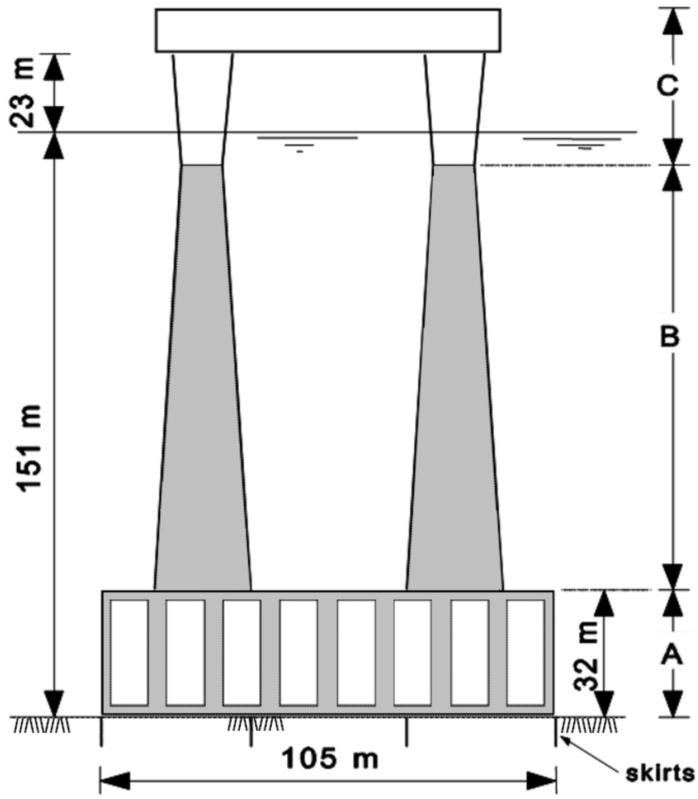


Figure 5.8: The Andoc platform

well above the water level. The large area of the waterline area induced a high inertial moment and thus a 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 immersed 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 little higher (waterline at 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 than 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 levelling 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 least of the Andoc-platform), much water must be able to 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 5.9 gives an example of this.

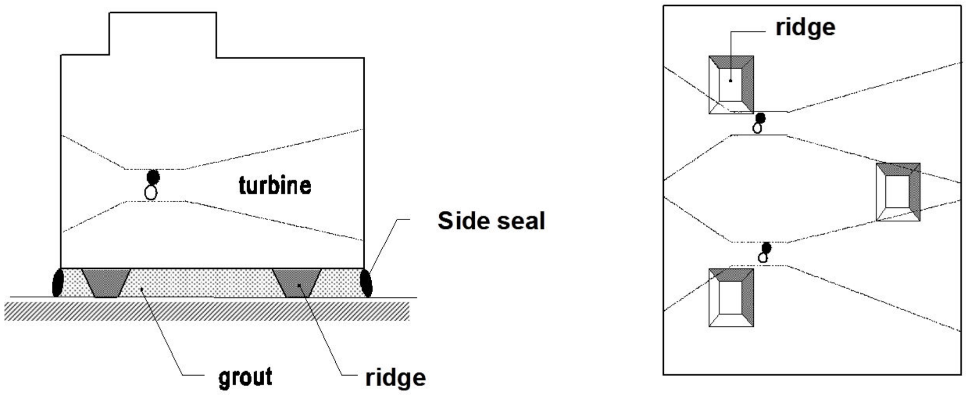


Figure 5.9: Turbine caisson (Left: cross-section, right: top view)

This example is a turbine-caisson (which houses 2 turbines) for a tidal power generator. The underside of the caisson is fitted with 3 ridges or supports, 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 5.10). A detailed description of this is described below.

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 can be closed by lift gates that are powered by hydraulic jacks and moved between guide rails in the sides of the piers. The traffic bridge, the lower sill beam (*dorpelbalk*) and the upper sill beam 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.

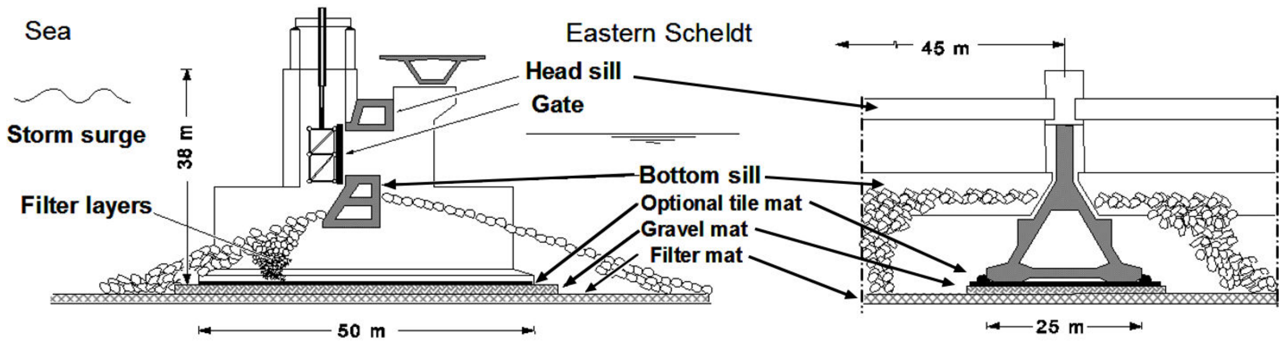


Figure 5.10: Storm surge barrier in the Oosterschelde: cross-section and longitudinal section

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 m^2, 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. The height of the gravel mat was measured before each pier was positioned. If this exceeded 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 installing the gravel bed, 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 5.10).

Temporary support on jacks

Themporary supports on jacks are used for the immersed tunnel shown in Figure 5.11. The tunnel is constructed entirely from reinforced or pre-stressed 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.

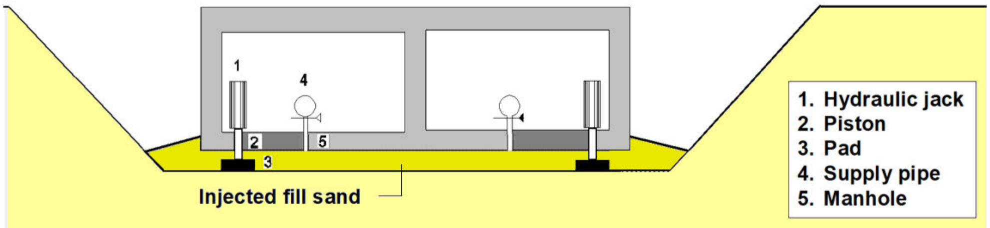


Figure 5.11: Positioning and foundation for European tunnels

The element is placed on three 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 that very accurate positioning of the section is necessary regarding 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 6x6 m2) 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 choosing 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 large 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.

Notes

- 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 a 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.

6

STRUCTURAL SAFETY

6.1 Basic principles

6.1.1 Introductory definitions

Safety is defined in the ISO-code 8402 as "*a state in which the risk of harm to people or material damage is limited to an acceptable level.*" This implies that safety is complementary to risk.

Risk in engineering is often quantified as the product of the probability of failure and the consequences of failure. Probability is the likelihood of an event and its consequences are quantified as the direct or total economic damage, or the loss of life, given that the hazardous event occurs.

What risk is considered acceptable can be determined using three criteria:

- Individual casualty criterion: the probability that an individual, continuously residing at a certain place during a year, will perish due to an undesired event;
- Multiple casualties criterion: the probability that a (large) number of individuals perishes due to an undesired event. This is considered a measure for societal disruption;
- Economic criterion: the investments in reducing the failure probability should balance the prevented loss of economic value.

The risk level that is considered acceptable is subjective and depends, amongst others, on the extent of voluntariness of exposure to the threat, the recognisability of the threat and social advantages of opposing the risk.

Reliability is the probability of a structure or system performing its required function adequately for a specified period of time under stated conditions (Reeve, 2010). In other words: reliability is the probability of non-failure.

6.1.2 Failure of structures

To determine the dimensions of structural elements of a civil engineering work, one needs to know the expected loads and material characteristics. One also needs structural design rules. Nearly all design rules are derived from failure modes and describe a certain limit state.

The type of solicitation (= load) and resistance depends on the regarding failure mechanism. It could be a force if a horizontal or vertical equilibrium is checked or a turning moment if a rotational equilibrium is considered. If the water retaining height of a flood defence has to be determined, loading and resistance are expressed as an elevation above reference level (m above NAP in the Netherlands).

For example, in case of the serviceability limit state of a breakwater, the resistance is defined as the maximum allowed wave height in the harbour and the solicitation is the occurring wave height in the harbour basin, which is influenced by the geometry of the structure.

In modern standards like the Eurocodes, this is often expressed as a dimensionless unity-check:

$$S/R < 1 \tag{6.1}$$

The relation between solicitation and resistance can also be expressed as a **limit state function** (*grenstoestandsfunctie*). A limit states is a condition just before failure. The general form of a limit state function is:

$$Z = R - S \tag{6.2}$$

If $Z < 0$, the structure will fail according to the given mode.

A **failure mechanism** (*faalmechanisme*) is a description of the way in which a structure is no longer able to fulfil its function. Not being able to fulfil a function can relate to persistent, transient, accidental or seismic situations. Failure is permanent if a structure collapses.

Several limit state types can be distinguished. Eurocode gives the following overview of limit states:

- Serviceability limit state (SLS, bruikbaarheidsgrenstoestand), indicating disruption of normal use.
- Ultimate limit state (ULS, uiterste grenstoestand), indicating collapse of all or part of the structure:
  - Loss of static equilibrium of the structure or any part of it, considered as a rigid body (EQU).
  - Internal failure of the structure or structural elements, including footings, piles, basement walls, etc., in which the strength of construction materials or excessive deformation of the structure governs (STR)
  - Failure or excessive deformation of the ground in which the strengths of soil or rock are significant in providing resistance (GEO)
  - Fatigue failure of the structure or structural elements (FAT)

Instead of FAT, Eurocode 7 (Geotechnical Design) mentions:

- Loss of equilibrium due to uplift by water pressure (buoyancy) or other vertical actions (UPL)
- Hydraulic heave, internal erosion and piping caused by hydraulic gradients (HYD).

An example of an ultimate limit state of a breakwater (textitgolfbreker) is the overturning of the breakwater as a result of the collapse of its foundation. Due to its collapse, the breakwater can no longer fulfil its function. An example of the serviceability limit state is the overtopping of a large number of waves over the breakwater, in which case there is no guarantee of calm water behind the breakwater.

Sometimes, a damage limit state is distinguished, indicating unacceptable damage but no immediate failure. However, the damage limit state is often included in the ultimate limit state. During the design process, one must take both the ultimate limit state and the serviceability limit state into account. In our breakwater example, the ultimate limit state refers to the stability and strength of the structure and subsoil whilst the serviceability limit state is related to the safety of use. (In the ultimate limit state, stiffness is of importance when deformation induces 'collapse' of the structure or structural element.)

In general, a structure does not collapse if its loading (solicitation) can be resisted:

$$S < R \tag{6.3}$$

where:

S	=	the load ("Sollicitation")
R	=	the resistance to failure due to the load, or the strength ("Resistance")

6.1.3 The need for safety factors

In practice, several kinds of uncertainties have to be taken into account while making an engineering design. There are four main categories of uncertainties:

1. physical or inherent uncertainties;
2. statistical uncertainties;
3. modelling uncertainties;
4. human error.

Physical uncertainties consist of randomness or variations in nature. Variables can differ in time (water level, for example), or in space (dike height). These uncertainties are mainly caused by a lack of data of loading or strength. Statistical uncertainties occur if the distribution function of the possible values for loading or strength is not exactly known, or if the parameters of the distribution function are determined with a limited amount of data. Modelling uncertainties consist of imperfectness of the models, or failure modes, describing natural phenomena. This can be caused by a lack of knowledge of these processes, or of over-simplification. Financial uncertainties (like construction costs and damage costs) are comprised in this category of modelling uncertainties. Finally, human error often forms a big threat to the reliability of a structure.

All these uncertainties can be taken into account by introducing a safety margin between loading and strength. There are various calculation techniques available to incorporate this margin in a structural design. These techniques are classified according to the following levels:

- Level 0: deterministic design;
- Level I: semi-probabilistic design;
- Level II: simplified probabilistic design;
- Level III: full probabilistic design.

These methods are briefly explained in the following sections.

6.2 Deterministic design (level 0)

Based on experience, or engineering judgement, overall safety factors (γ) were applied to create a margin between loading and strength. In general, in a deterministic approach, a structure is considered safe, if:

$$S \cdot \gamma < R, \tag{6.4}$$

where $\gamma > 1,0$ [-]

(Old Dutch TGB standards used $\gamma = 1,5$ for steel structures and $\gamma = 1,7$ for concrete structures until about 1990.)

In the Netherlands, the crest height of flood defences used to be based on the highest observed water level (often the water level that caused most recent flood), plus a freeboard fb (*waakhoogte*) of 0,5 to 1,0 metres to account for wave overtopping and uncertainties:

$$S + fb < R \tag{6.5}$$

where 0,5 m $\leq fb \leq 1,0$m

The estimation of these overall safety factors was not based on a quantification of the uncertainties, so it was very difficult to determine the extent of over-design (or under-design) relative to some desired level of safety. This can be overcome by using (semi-) probabilistic techniques, which is explained in the following sections.

6.3 Semi-probabilistic design (level I)

6.3.1 Theory

In semi-probabilistic design methods, load and strength variables are treated as stochasts, which means that their possible values are distributed around a mean value μ (Figure 6.1). The characteristic value of the strength R_k is the value that is exceeded by 95% of the samples. The characteristic value of the load S_k is the value that is exceeded by only 5% (in other words: the single tails represent 5% of the possible values).

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. The failure probability then is very low, especially when these characteristic values are multiplied by partial safety factors.

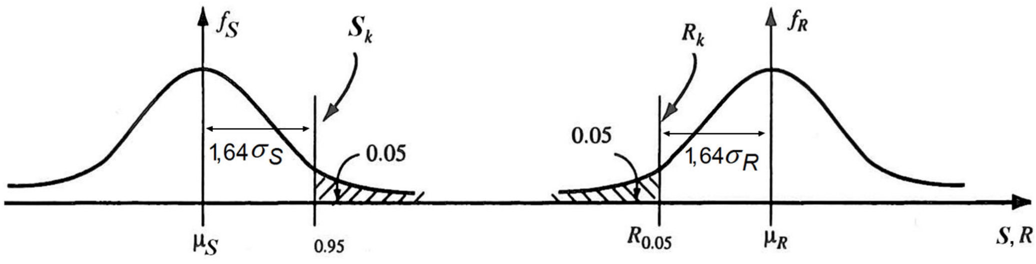


Figure 6.1: Characteristic values for load and strength

The characteristic values deviate from the mean values depending on the 'width' of the distribution, which can be expressed as a function of the standard deviation:

$$R_k = \mu_R - k \cdot \sigma_R \tag{6.6}$$

and

$$S_k = \mu_S + k \cdot \sigma_S \tag{6.7}$$

where:

R_k	=	characteristic value of the strength
S_k	=	characteristic value of the load
μ	=	mean value of strength μ_R or load μ_S
k	=	multiplication constant for the standard deviation to obtain the 5% / 95%value; k = 1,64 for a normal distribution
σ	=	standard deviation of strength σ_R or load σ_S:

$$\sigma = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(x_i - \mu)^2} \tag{6.8}$$

for a large number of samples, and:

$$\sigma = \sqrt{\frac{1}{N-1}\sum_{i=1}^{N}(x_i - \mu)^2} \tag{6.9}$$

for a limited number of samples

These characteristic values of strength and loading are used to obtain the representative values that are needed to evaluate the limit states (SLS or ULS).

Every load has four representative values:

- **the characteristic value** (the main representative value), which is found using statistical methods on a preferably large number of samples, as described above.
- **the combination value**. If the loads are time-dependent, it is too conservative to add up the representative values of all loads and to multiply them all by the same partial safety coefficient. After all, the maximum values of the loads do not necessarily all act on the structure at the same time. This can be overcome by using the Turkstra rule for the variable loads. According to Turkstra, one load is considered dominant in every combination of loads. In that case, only average values of the other loads should be taken into account. The Eurocodes don't work with averages of variable loads, but contain reduction factors for load combinations (see the following section on load combinations).
- **the frequent value**. The frequent value is chosen in such a way that it can only be exceeded during a short period. It is mainly used in the serviceability limit state and in extreme ultimate limit state.

- **the quasi-permanent value**. It is permissible that quasi-permanent values are exceeded during long periods of time. These values could be considered as time-averaged values. They are used for long-term effects in SLS, accidental combinations and seismic design in ULS.

These representative values can be obtained by multiplying the characteristic values by the combination factor ψ_0, frequent factor ψ_1, or quasi-permanent factor ψ_2 (see the following section for their values according to the Eurocodes).

The representative values for material properties are mostly the same as the characteristic values, anyway in the Eurocodes. In some foreign codes the representative material factors could differ from the characteristic values.

When determining the dimensions of the design in a limit state check, the required strength has to have a design value larger than the design value of the load. These design values are related to the representative values through partial safety factors:

$$R_d \geq S_d \Rightarrow \frac{R_{rep}}{\gamma_R} \geq \gamma_S \cdot S_{rep} \tag{6.10}$$

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
R_d	=	design value of the strength
S_d	=	design value of the load

In hydraulic engineering, for the estimation of water levels needed to estimate hydrostatic loads and water retaining heights, characteristic values are used with a low probability of exceedance, like 1/1250 or 1/10 000. This is based on statistic calculations of water level measurements.

6.3.2 Combinations of loads

The steps that have to be followed to obtain a design value of a load, needed for a design calculation, are:

1. Estimate the types of the load (permanent, variable or accidental);
2. Discern all realistic loads;
3. Estimate the partial load factors (see below) for all relevant combinations of loads;
4. Combine the loads in such a way that the most critical circumstances are obtained.

In case of a load combination with only one variable load, the magnitude of this load is obtained by multiplying with the concerning partial load factor. If more than one variable load is combined, the main variable load should be distinguished from other, possibly simultaneously occurring, loads. A simultaneously occurring load is always considered as a combination value.

Fundamental load combinations

For fundamental load combinations, the Eurocode distinguishes permanent and variable loads. Loads from pre-stressing are treated as a separate permanent load and the main variable load is treated apart from other variable loads.

The design value of the load effect Ed (combined loads) for persistent and transient load combinations (fundamental combinations) should, according to Eurocode 0, be calculated as:

$$E_d = E\left\{\sum_{j\geq 1}^{n} \gamma_{G,j} \cdot G_{k,j} + \gamma_p \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1}^{n} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right\} \tag{6.11}$$

For the limit states STR (internal failure or excessive deformation of the structure or structural member) and GEO (failure or excessive deformation of the soil), the most unfavourable of the following equations should be used:

$$E_d = E\left\{\sum_{j\geq 1}^{n} \gamma_{G,j} \cdot G_{k,j} + \gamma_p \cdot P + \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} + \sum_{i>1}^{n} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right\} \tag{6.12}$$

$$E_d = E\left\{\sum_{j\geq 1}^{n} \xi_j \cdot \gamma_{G,j} \cdot G_{k,j} + \gamma_p \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1}^{n} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right\} \tag{6.13}$$

in which:

$E\{...\}$	=	the combination of the permanent, pre-stressing and variable loads
$G_{k,j}$	=	characteristic value of permanent load j
$\gamma_{G,j}$	=	partial factor for permanent load j
ξ_j	=	reduction factor for unfavourable permanent load j
P	=	representative value for the pre-stressing load
γ_P	=	partial factor for the pre-stressing load
$Q_{k,1}$	=	characteristic value of the main variable load
$\gamma_{Q,1}$	=	partial factor for the main variable load
$\psi_{0,1}$	=	combination reduction factor for the main variable load
$Q_{k,i}$	=	characteristic value of variable load i
$\gamma_{Q,i}$	=	partial factor for variable load i
$\xi_{0,i}$	=	combination reduction factor for variable load i

It should be judged by the structural engineer what possible loads are useful to combine. The national annexes to Eurocode 0 give tables with values for the reduction factors γ, ψ and ξ, depending on failure state, load type and type of building.

Load combinations for accidental design situations

In case of combined loads for accidental design situations (fire or impact), the design value of the load effect E_d should be calculated as:

$$E_d = E\left\{\sum_{j\geq 1}^{n} G_{k,j} + P + A_d + (\psi_{1,1} or \psi_{2,1}) \cdot Q_{k,1} + \sum_{i>1}^{n} \psi_{2,i} Q_{k,i}\right\} \tag{6.14}$$

The choice between $\psi_{1,1}Q_{k,1}$ or $\psi_{2,1}Q_{k,1}$ should be related to the relevant accidental design situation (impact, fire or survival after an accidental event or situation). A_d is the design value of the accidental action.

Load combinations for seismic design situations

The design value of the load effect E_d during earthquake situations should be calculated as:

$$E_d = E\left\{\sum_{j\geq 1}^{n} G_{k,j} + P + A_{Ed} + \sum_{i\geq 1}^{n} \psi_{2,i} Q_{k,i}\right\} \tag{6.15}$$

Reduction factors for the combination of loads

Eurocode 0 recommends values for the load combination factor ψ. This factor is either 1, or ψ_1, ψ_2 or ψ_3 as indicated in Table 6.1.

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight $\leq$ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight $\leq$ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H $\leq$ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Table 6.1: Factor for the combination of loads for buildings (Eurocode 0)
Note: values mentioned in national annexes to Eurocode 0 may differ from this table

6.3.3 Partial load factors

Eurocode 0 gives design values of actions in persistent and transient design situations in ultimate limit state. Static equilibrium (EQU) for building structures should be verified using Table 6.2. The design of structural members (STR), not involving geotechnical actions should be verified with help of Table 6.3.

For the design of structural members like footings, piles and basement walls (STR), involving geotechnical actions and the resistance of the soil, verification should be done using one of the following three approaches:

- Design values from Table 6.4 are applied in separate calculations and Table 6.3 is used for geotechnical loads as well as for other actions on/from the structure. The dimensioning of foundations is carried out with the values mentioned in Table 6.4 and the structural resistance with Table 6.3.
- Table 6.3 is used for both the geotechnical loads as well as for other loads on or from the structure.
- Table 6.4 is used for geotechnical loads and Table 6.3 is simultaneously used for partial factors to other actions on/from the structure.

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup} G_{kj,sup}$	$\gamma_{Gj,inf} G_{kj,inf}$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The γ values may be set by the National annex. The recommended set of values for γ are :

$\gamma_{Gj,sup} = 1{,}10$

$\gamma_{Gj,inf} = 0{,}90$

$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables A1.2(A) and A1.2(B), a combined verification, based on Table A1.2(A), may be adopted, if allowed by the National annex, with the following set of recommended values. The recommended values may be altered by the National annex.

$\gamma_{Gj,sup} = 1{,}35$

$\gamma_{Gj,inf} = 1{,}15$

$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

provided that applying $\gamma_{Gj,inf} = 1{,}00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

Table 6.2: Partial factors for loads in EQU ultimate limit states (Eurocode 0)
Note: values mentioned in national annexes to Eurocode 0 may differ from this table

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10a)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The choice between 6.10, or 6.10a and 6.10b will be in the National annex. In case of 6.10a and 6.10b, the National annex may in addition modify 6.10a to include permanent actions only.

NOTE 2 The γ and ξ values may be set by the National annex. The following values for γ and ξ are recommended when using expressions 6.10, or 6.10a and 6.10b.

$\gamma_{Gj,sup} = 1,35$

$\gamma_{Gj,inf} = 1,00$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

$\xi = 0,85$ (so that $\xi\gamma_{Gj,sup} = 0,85 \times 1,35 \cong 1,15$).

See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.

NOTE 3 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self weight of the structure may be considered as coming from one source ; this also applies if different materials are involved.

NOTE 4 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1,05 to 1,15 can be used in most common cases and can be modified in the National annex.

Table 6.3: Partial factors for loads in STR/GEO ultimate limit states (Eurocode 0)
Note: values mentioned in national annexes to Eurocode 0 may differ from this table.

Persistent and transient design situation	Permanent actions		Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup} G_{kj,sup}$	$\gamma_{Gj,inf} G_{kj,inf}$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\ \psi_{0,i} Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE The γ values may be set by the National annex. The recommended set of values for γ are :
$\gamma_{Gj,sup} = 1{,}00$
$\gamma_{Gj,inf} = 1{,}00$
$\gamma_{Q,1} = 1{,}30$ where unfavourable (0 where favourable)
$\gamma_{Q,i} = 1{,}30$ where unfavourable (0 where favourable)

Table 6.4: Partial factors for non-geotechnical loads on structural members in STR/GEO ultimate limit states (Eurocode 0)
Note: values mentioned in national annexes to Eurocode 0 may differ from this table

Design values of load combinations in accidental and seismic design situations

For an overview of design values of combinations of accidental and seismic loads, see Table 6.5. The partial factors for loads for the ultimate limit states in the accidental and seismic design situations should be 1,0. Values for ψ are given in Table 6.1.

Design situation	Permanent actions		Leading accidental or seismic action	Accompanying variable actions (**)	
	Unfavourable	Favourable		Main (if any)	Others
Accidental (*) (Eq. 6.11a/b)	$G_{kj,sup}$	$G_{kj,inf}$	A_d	ψ_{11} or $\psi_{21}Q_{k1}$	$\psi_{2,i}\, Q_{k,i}$
Seismic (Eq. 6.12a/b)	$G_{kj,sup}$	$G_{kj,inf}$	$\gamma_I A_{Ek}$ or A_{Ed}		$\psi_{2,i}\, Q_{k,i}$

(*) In the case of accidental design situations, the main variable action may be taken with its frequent or, as in seismic combinations of actions, its quasi-permanent values. The choice will be in the National annex, depending on the accidental action under consideration. See also EN 1991-1-2.

(**) Variable actions are those considered in Table A1.1.

Table 6.5: Design values of loads for use in accidental and seismic combinations of loads (Eurocode 0)
Note: values mentioned in national annexes to Eurocode 0 may differ from this table

Partial load factors for serviceability limit states

For serviceability limit states the partial factors for loads γ_S should be equal to 1,0, except if differently specified in EN 1991 to EN 1999. See Table 6.6.

Combination	Permanent actions G_d		Variable actions Q_d	
	Unfavourable	Favourable	Leading	Others
Characteristic	$G_{kj,sup}$	$G_{kj,inf}$	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{kj,sup}$	$G_{kj,inf}$	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{kj,sup}$	$G_{kj,inf}$	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

Table 6.6: Design values of loads for use in load combinations for SLS (Eurocode 0)
Note: values mentioned in national annexes to Eurocode 0 may differ from this table

6.3.4 Partial material factors

Concrete

Partial factors for plain, reinforced, or prestressed concrete in ultimate limit states, γ_c and γ_s should be used as indicated in Table 6.7.

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

Table 6.7: Material factors for structures in plain, reinforced or prestressed concrete in ULS (Eurocode 2)
Note: values mentioned in national annexes to Eurocode 2 may differ from this table.

The partial factor γ_c should be multiplied by a factor k_f (recommended value is 1,1) for the calculation of the design resistance of cast-in-place piles without permanent casing.

Steel

For steel structures, partial factors as indicated in Table 6.8 should be used.

type of material resistance	partial material factor γ_m
resistance of cross-sections of all steel classes	1,0
resistance of members to instability assessed by member checks	1,0
resistance of cross-sections in tension to fracture	1,25
resistance of various joints	see Eurocode 1993-1-8

Table 6.8: Partial material factors for steel structures in ULS (Eurocode 3)
Note: values mentioned in national annexes to Eurocode 3 may differ from this table

Soil

The General Eurocode 7 'Geotechnical design' (EN 1997) specifies the use of partial factors for soil properties, which have to be prescribed per country in national appendices to the standard. The text below describes the general use of partial factors, so for practical use, the appropriate national appendix shall be used. In general, the following partial factors (γ_M) shall be applied for the verification of the equilibrium limit state (EQU), when including minor shearing resistances, see Table 6.9.

soil parameter	symbol	value
angle of internal friction [1])	$\gamma_{\varphi'}$	1,25
effective cohesion	$\gamma_{c'}$	1,25
undrained shear strength	γ_{cu}	1,4
prism compressive strength	γ_{qu}	1,4
specific weight	γ_γ	1,0
[1]) This factor relates to tanφ'		

Table 6.9: Partial factors for soil properties for equilibrium state verification (EQU) (Eurocode 7)
Note: values mentioned in national annexes to Eurocode 7 may differ from this table

For the verification of structural (STR) and geotechnical (GEO) limit states set $M1$ or set $M2$ of the partial factors on soil parameters (γ_M) shall be applied as given in Table 6.10. The most unfavourable set $M1$ or $M2$ shall be used.

For spread foundations and verifications of structural (STR) and geotechnical (GEO) limit states, set $R1$, $R2$ or $R3$ of the following partial factors on resistance (γ_R) shall be applied: $\gamma_{R;v}$ on bearing resistance and $\gamma_{R;h}$ on sliding resistance, see Table 6.11.

Partial resistance factors for various types of piles are given in Table 6.12, Table 6.13 and Table 6.14

Soil parameter	Symbol	Set	
		M1	*M2*
Angle of shearing resistance[a]	$\gamma_{\varphi'}$	1,0	1,25
Effective cohesion	$\gamma_{c'}$	1,0	1,25
Undrained shear strength	γ_{cu}	1,0	1,4
Unconfined strength	γ_{qu}	1,0	1,4
Weight density	γ_{γ}	1,0	1,0
[a] This factor is applied to tan φ'			

Table 6.10: Partial factors for soil properties. For the verification of structural (STR) and geotechnical (GEO) limit states verification (Eurocode 7)
Note: values mentioned in national annexes to Eurocode 7 may differ from this table

Resistance	Symbol	Set		
		R1	*R2*	*R3*
Bearing	$\gamma_{R;v}$	1,0	1,4	1,0
Sliding	$\gamma_{R;h}$	1,0	1,1	1,0

Table 6.11: Partial resistance factors (γ_R) for spread foundations (Eurocode 7)
Note: values mentioned in national annexes to Eurocode 7 may differ from this table

Resistance	Symbol	Set			
		R1	*R2*	*R3*	*R4*
Base	γ_b	1,0	1,1	1,0	1,3
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,0	1,1	1,0	1,3
Shaft in tension	$\gamma_{s;t}$	1,25	1,15	1,1	1,6

Table 6.12: Partial resistance factors (γ_R) for driven piles (Eurocode 7)
Note: values mentioned in national annexes to Eurocode 7 may differ from this table

Resistance	Symbol	Set			
		R1	*R2*	*R3*	*R4*
Base	γ_b	1,25	1,1	1,0	1,6
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,15	1,1	1,0	1,5
Shaft in tension	$\gamma_{s;t}$	1,25	1,15	1,1	1,6

Table 6.13: Partial resistance factors (γ_R) for bored piles (Eurocode 7)
Note: values mentioned in national annexes to Eurocode 7 may differ from this table

For retaining structures and verifications of structural (STR) and geotechnical (GEO) limit states, set $R1$, $R2$ or $R3$ of the partial factors on resistance (γ_R) shall be applied as mentioned in Table 6.15: $\gamma_{R;v}$ on bearing capacity, $\gamma_{R;h}$ on sliding resistance and $\gamma_{R;e}$ on earth resistance.

For slopes and overall stability and verifications of structural (STR) and geotechnical (GEO) limit states a

Resistance	Symbol	Set			
		R1	*R2*	*R3*	*R4*
Base	γ_b	1,1	1,1	1,0	1,45
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,1	1,1	1,0	1,4
Shaft in tension	$\gamma_{s;t}$	1,25	1,15	1,1	1,6

Table 6.14: Partial resistance factors (γ_R) for continuous flight auger (CFA) piles (Eurocode 7)
Note: values mentioned in national annexes to Eurocode 7 may differ from this table

Resistance	Symbol	Set			
		R1	*R2*	*R3*	*R4*
Temporary	$\gamma_{a;t}$	1,1	1,1	1,0	1,1
Permanent	$\gamma_{a;p}$	1,1	1,1	1,0	1,1

Table 6.15: Partial resistance factors (γ_R) for retaining structures (Eurocode 7)
Note: values mentioned in national annexes to Eurocode 7 may differ from this table

partial factor on ground resistance ($\gamma_{R;e}$) shall be applied. The recommended value for the three sets $R1$, $R2$ and $R3$ is given in Table 6.16.

Resistance	Symbol	Set		
		R1	*R2*	*R3*
Earth resistance	$\gamma_{R;e}$	1,0	1,1	1,0

Table 6.16: Partial resistance factors (γ_R) for slopes and overall stability (Eurocode 7)
Note: values mentioned in national annexes to Eurocode 7 may differ from this table

Eurocode 7 also gives partial material factors for the verification of the uplift limit state and hydraulic heave limit state. Furthermore, partial specific load factors are given for the geotechnical limit states mentioned above.

Important note. This chapter shows a selection of load and material factors from the General Eurocodes. Therefore, for design calculations in engineering practice, one is advised to consult the complete text of the Eurocode standards, including the relevant national annex.

6.4 Probabilistic design (levels II and III)

Both level II and level III calculations are probabilistic design methods. Level II methods are simplifications of full probabilistic design methods, level III. The full probabilistic design, level III, is explained first in the Section 6.4.1 and then the simplified methods, level II (Section 6.4.2).

6.4.1 Full probabilistic design (level III)

Level III-methods are full probabilistic approaches in which the probability density functions of all stochastic variables are described and included in the analysis. A probability density function is a function that describes the relative likelihood for a random variable to take on a given value. Figure reffig:ProbabilityDensityFunctions shows the probability density functions of the loading $f_S(s)$ and strength $f_R(r)$ as well as the resulting probability density function of the limit state $f_Z(z)$. The failure probability p_f is represented by the area where $Z < 0$ (the small grey area).

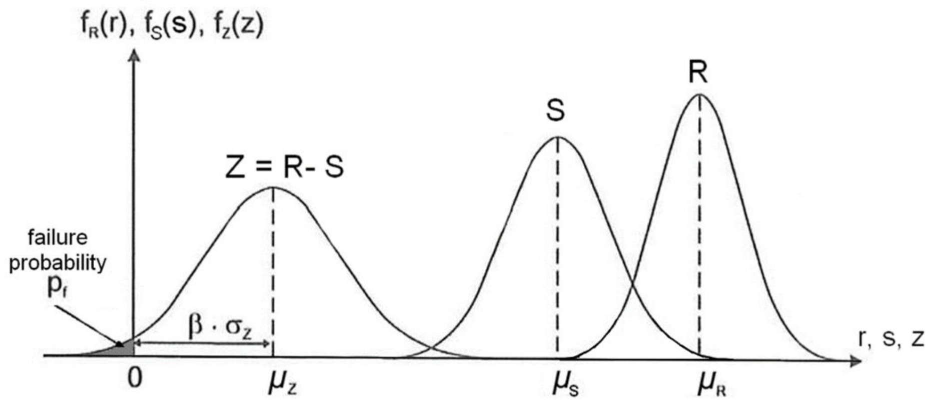


Figure 6.2: Probability density distributions of loading and strength, plus corresponding limit state function

A 'wide' distribution around the average limit state value μ_Z implies a large uncertainty, while a 'tight' distribution indicates a high certainty. The 'wideness' of the distribution should be judged relative to its mean value to obtain a good impression of the reliability.

The relationship between the stochastic variables of loading and strength can be mathematically described and the probability of exceedance can be calculated for the considered limit state. The probability of failure pf is the probability that the loading exceeds the resistance:

$$p_f = p(R < S) = p(Z < 0) \tag{6.16}$$

If loading and strength are independent, the failure probability can be calculated using:

$$p_f = \iint_{r<s} f_R(r) f_S(s) dr ds \tag{6.17}$$

$f_R(r)$ = probability density function of resistance
$f_S(s)$ = probability density function of loading

The product of $f_R(r)$ and $f_S(s)$ is the joint probability function $f_{RS}(r,s)$:

$$f_R(r) f_S(s) = f_{RS}(r,s) \tag{6.18}$$

Figure 6.3 shows such a joint density function $f_{R,S}$, including the $Z = 0$ line.

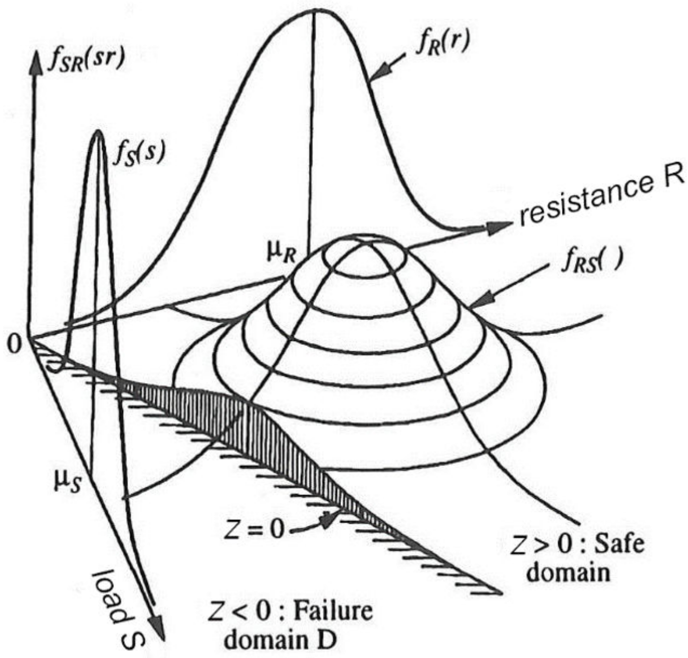


Figure 6.3: Joint density function (Melchers, 1999)

The failure probability can therefore be expressed as a function of the joint density function:

$$p_f = \int_{-\infty}^{r} \int_{-\infty}^{s} f_{R,S}(r,s)\,dr\,ds \tag{6.19}$$

If more than one failure mode or more than one structural element is considered, the failure probability can be formulated as an n-fold integral. For independent variables xi this looks like:

$$p_f = \iint_{Z(x)<0} \dots \int \prod_{i=1}^{n} f_{x,i}(x_i)\,dx_i \tag{6.20}$$

Solving this kind of integrals is a tough task, especially if n exceeds 5. However, in some cases with not too low failure probabilities, the integral can be solved with help of a Monte Carlo simulation. However, the difficulty of applying probabilistic techniques is that it requires a detailed knowledge of each variable and the relationship between these variables.

6.4.2 Simplified probabilistic design (level II)

Because of the drawbacks of a full deterministic design, methods have been developed to approximate the distribution functions of loading and strength. As a simplification, the limit state function is linearized and for most methods all parameters are considered to be independent and the probability density functions of loading and strength are replaced by normal distributions (also called Gaussian distributions):

$$f(x) = \frac{1}{\sigma_x\sqrt{2\pi}} \cdot e^{-\frac{(x-\mu_x)^2}{2\sigma_x^2}} \tag{6.21}$$

where x can represent load (s) and strength (r).

Depending on the order of approximation, first-order reliability methods (FORMs) or second-order reliability methods (SORMs) can be used.

These methods and more backgrounds on probabilistic design are treated in the lecture notes CIE4130 'Probability in Civil Engineering'. These probabilistic methods can be used to derive the partial safety factors needed for a semi-probabilistic design, see next section.

6.4.3 The derivation of partial safety factors and reliability indices for level I-calculations

A useful expression for judging the reliability of a structure is the reliability index β, which is related to the mean value and the standard deviation of the limit state distribution Z:

$$\beta = \frac{\mu_Z}{\sigma_Z} \tag{6.22}$$

β	=	reliability index
μ_Z	=	mean value of the limit state density function ($\mu_Z = \mu_R - \mu_S$)
σ_Z	=	standard deviation of the limit state density function ($\sigma_z = \sqrt{\sigma_R^2 + \sigma_S^2}$)

It can be seen in Figure 6.2 that $\mu_Z = \beta \cdot \sigma_Z$.

The influence of the distribution of the load or resistance on the distribution of the limit state function is usually expressed by the influence coefficient (*invloedscoëfficiënt*):

$$\alpha_R = \frac{\sigma_R}{\sigma_Z} \text{ and } \alpha_S = \frac{\sigma_S}{\sigma_Z} \tag{6.23}$$

α	=	influence coefficient for the strength (α_R) or load (α_S)
σ	=	standard deviation of the strength (σ_R), load (σ_S), or limit state function (σ_Z)

Level III and level II calculations can be used to calculate the partial factors used in level I calculations, if the reliability index β and influence coefficient α are known:

$$\gamma_R = \gamma_M = \frac{1 - k_R \cdot V_R}{1 - \alpha_R \cdot \beta \cdot V_R} \text{ and } \gamma_S = \frac{1 - \alpha_S \cdot \beta \cdot V_S}{1 - k_S \cdot V_S} \tag{6.24}$$

where:

k	=	factor indicating the limit of the representative value of strength (k_R) or load (k_S)
V	=	coefficient of variation for strength ($V_R = \sigma_R / \mu_R$) or load ($V_S = \sigma_S / \mu_S$)

The reliability index can be derived from the failure probability requirement for the failure mechanism under consideration:

$$\beta = \phi^{-1}(p_f) \tag{6.25}$$

where ϕ^{-1}= the inverse of the normal distribution function.

For the calculation of partial factors for building codes for steel and concrete structures, it is usual to adopt $k_R = 1{,}64$ and $k_S = 0$, but other values may also be chosen. The value of the reliability index β used for the determination of the partial factors depends on the severity of consequences (in Eurocode 0 indicated by consequence classes) and the reference period (life time) of the structure. Values of influence coefficients used for the determination of partial factors are based on calculations of failure probabilities for a number of reference cases.

6.4.4 Reliability of multiple component systems

A failure probability approach allows for calculating the reliability of systems composed of multiple structural parts (called components, elements or structural members). Each of these can fail by way of multiple failure mechanisms. Regarding system reliability, two basic types of combined system can be distinguished: a series system and a parallel system.

In a *series system*, the failure of each individual structural part $p_{f,i}$ will automatically lead to failure of the entire system $p_{f,sys}$. The system's strength R_{sys} is therefore determined by its weakest element R_i:

$$R_{sys} = min(R_i) \tag{6.26}$$

An example of a series system is a dike segment, which includes several dike sections and its crossing structures, e.g., navigation locks and sluices. If a navigation lock, as part of such a system would fail, it would lead to flooding of the area protected by the dike. If at least one of the dike sections would fail, for example due to macro-instability or piping, the same applies: the area will be flooded regardless other parts of the dike segment that are still well-functioning. Thus, the dike itself can be modelled as a series system with all possible failure mechanisms of its components.

A *parallel system* only fails if all structural elements have failed. The system strength is determined by its strongest element:

$$R_{sys} = max(R_i) \tag{6.27}$$

An example of a parallel system is a navigation lock with a double set of flood (mitre) gates. The lock is still functional if only one set of gates is damaged, because the other gate set will take over the function of the first gate. Another example of a parallel system is the multi-layered flood safety approach, where the allowable failure probability of a flood defence system can be increased by introducing evacuation plans and other damage-reducing measures. This actually is not very efficient in combination with the series protection system of dike ring areas.

The failure probability of a system can be calculated if the failure probabilities of the structural components are known, and if it is known whether these events are dependent or independent. The difference in failure probability of a dependent and an independent system is considerable. The failure probability of a dependent series system, for example, is equal to the largest failure probability of the individual subsystems. For an independent series system, the failure probability is equal to the summation of the failure probabilities of all subsystems. The correlation between failure mechanisms of flood defences predominantly comes from the main loading, for example an extreme water level. The influence of this loading on the various failure mechanisms, however, can be quite different. Another thing that plays a role in the determination of the dependency of flood defence systems is the spatial variation. In particular, this applies to the loads and resistances of dikes (the so-called 'length effect').

For *series systems* of two independent elements A and B, an upper bound of the system probability of failure is:

$$p_f = p(A \cup B) = p_f(A) + p_f(B) \tag{6.28}$$

The failure probability of *parallel systems* with two independent elements A and B is:

$$p_f = p(A \cap B) = p_f(A) \cdot p_f(B) \tag{6.29}$$

Table 6.17 gives the equations to calculate the system failure probability as described above for dependent and independent events. The equations give the lower and upper bound, for perfectly correlated, non-identical components and mutually exclusive events (the failure of a component or element is called an 'event').

There are various graphical methods to help obtain an overview of the elements that together define a system and allow calculation of the system failure probability. These methods are especially useful as a system becomes more complex and is composed of several series or parallel subsystems. The most common are the

System type	failure probability of dependent events	failure probability of independent events	failure probability of mutually exclusive events
series system	$max^n_{i=1,n} P_i$ lower bound	$1 - \prod^n_{i=1}(1 - P_i)$	$\sum^n_{i=1} P_i$ upper bound
parallel system	$min^n_{i=1,n} P_i$ upper bound	$\prod^n_{i=1} P_i$	0 lower bound

Table 6.17: Failure probabilities per system type (Voortman, 2003)

event tree, the fault tree and the cause-consequence diagram. These methods can be used as a tool in level II or level III calculations, because they visualise how the failure modes combine and interact.

Fault trees, or 'fault tree diagrams' are logic block diagrams that display the state of a system (top event) in terms of the states of its elements (basic events, which is failure of a structural component or failure mechanism). Logical operators, called 'gates', are used in a fault tree to combine events to obtain an event at a higher level. Logical operators (Boolean operators) can be AND, OR or NOT. Different sets of component failures are logically connected in a fault tree and the probability of failure of the top element, representing the failure of the system as a whole, can be calculated using the equations presented above, if values are assigned to component reliabilities. Figure 6.4 shows basic fault trees for a series and a parallel system. The difference is defined by the AND and OR gate.

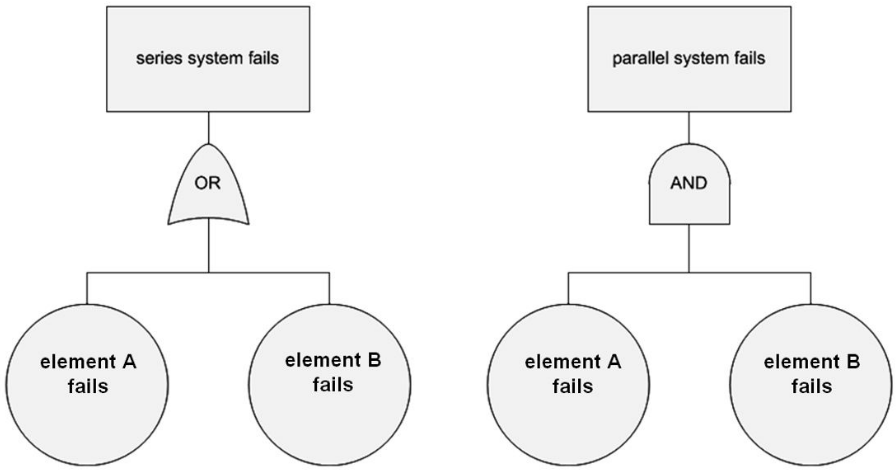


Figure 6.4: Basic fault tree for a series system (left) and a parallel system (right)

Figure 6.5 shows an example of a fault tree for the flooding of an area (the top event). The area is surrounded by a dike that is subdivided into several dike sections (*dijkvakken*). The fault tree only indicates how failure of separate dike sections contribute to the system failure, not how these dike sections can fail, i.e., dike section failure mechanisms are left out. The level of failure mechanisms per dike section should be added to this fault tree to obtain insight in their contribution to the failure probability. The failure probability of the top event is the maximum probability of system failure that is considered to be acceptable. This failure probability requirement should be specified by the government or client.

Fault trees have limited capacity to deal with dependence between various events and states. For simple fault trees, dependence can be included with careful considerations of the specific events and magnitude of the failure probabilities. For example, for two dependent series subsystems A and B, the $p_f(A \cup B) = p_f(A) + p_f(B) - p_f(AB)$, which is nearly equivalent to the independent case $p_f(A \cup B) = p_f(A) + p_f(B)$, as $p_f(AB) \ll p_f(A) \cdot p_f(B)$. One particular example where dependence must be addressed is the so-called

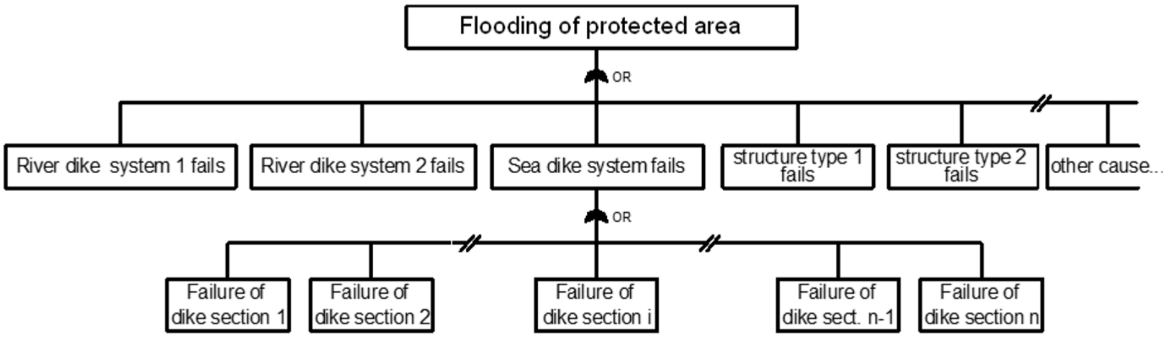


Figure 6.5: Example of a fault tree for flooding of an area being part of a dike segment (Mai Van, 2010, slightly modified)

common cause failure, where the occurrence of a specific loading condition affects only a part of the fault tree. A variety of special gates and procedures can be incorporated to deal with situations where dependence plays a role, and the reader is advised to consult the lecture notes of the course on probabilistic design (course code CIE4130) for more information.

Rules for creating a fault tree

When creating a fault tree, the following principles should be taken into account.

1. The top event of a fault tree represents the failure of a system, preferably formulated as a failing functionality (for example: 'flood of a polder' for a flood protection system).
2. The fault tree includes only those faults that contribute to this top event. For example, for weir complex (*stuw*) in a river, a complex consisting of a weir, fish passage and navigation lock, it does not make sense to define a top event that branches into, for example: 1) enabling shipping during low river discharges, 2) allowing passage of fish, 3) providing sufficient flood protection of the adjacent land areas. It is better to create separate fault trees per main function. If failure of a specific structural component appears in multiple fault trees, it has to comply to the strictest requirement, so the lowest failure probability. For example, structural failure of a lock gate can be an event in a fault tree for the shipping function as well as in a separate fault tree for the flood protection function.
3. The faults/events in a fault tree are not exhaustive: they cover only the most credible faults.
4. The top event (failure mode) is related to other events by gates. In this course, we only consider OR and AND gates.
5. An event can only be connected to one higher and one lower gate.
6. A gate connects lower events to a higher event in a logical (causal) way, at the right level.
7. Be aware of the difference between structural failure (like insufficient strength of a gate) and non-structural failure (like non-closure of a gate). These types of failure can be combined in one fault tree, because both types can lead to the loss of function of the entire system, but don't make non-logical relations.
8. Distinguish between failure during the construction and the use phase: do not combine them in one fault tree.
9. For independent events, the failure probabilities of OR-gates are combined by adding up the lower events, AND gates are combined by multiplying the lower events.
10. Keep in mind that in case of fault trees created for the (re)design of structures, i.e., where a maximum allowable failure probability is considered, the structural engineer determines how the top event failure probability is distributed to component failure probabilities throughout the tree. For a dike, it can for example be decided to increase the failure probability due to wave overtopping (by making the dike lower) and instead decrease the failure probability due to slope instability (by adding an inner berm). The cost reduction due to lowering the dike should then be weighed against the higher costs of adding a berm. The failure probabilities in a fault tree intended for a design are failure probability *requirements* that should be met by proper design of the component under consideration!

6.5 The safety of flood defences

This section mainly deals with the way in which the target safety of flood defences can be found. It basically answers the question 'What is safe enough?'

6.5.1 Introduction

Flood defences are hydraulic structures that protect the hinterland from being covered by water coming from oceans, seas, rivers, lakes and other waterways. There are three main types of flood defences:

- soil structures (dunes, dikes, dams);
- specific water-retaining structures (cofferdams, gravity walls, sheetpile walls, etc.);
- engineering structures (*kunstwerken*) (sluices, navigation locks, cut-offs, storm surge barriers, pumping stations).

Water-retaining engineering structures serve functions that are combined with flood defences, mainly as part of crossing infrastructures. They are usually equipped with movable closure means (gates) (ENW, Grondslagen voor Hoogwaterbescherming, 2016, TAW, Grondslagen voor Waterkeren, 1998).

There are three main design approaches for flood defences:

- deterministic design (in the Netherlands used until 1960)
- exceedance probability approach (in the Netherlands used from 1960 to 2016)
- failure probability approach (in the Netherlands used since 2017)

These approaches are explained in the following sections.

6.5.2 Deterministic design

Deterministic design is the classic approach, where the crest level (*kruinhoogte*) of a dike is based upon the highest observed water level, which is usually the water level that caused the last flood. An extra height, the freeboard (*waakhoogte*)[1], is added to account for wave overtopping and uncertainties. The freeboard is usually chosen between 0,5 and 1,0 m (excluding the surplus height to compensate for settlement):

$$h_{structure} > h_{water} + fb \tag{6.30}$$

where:

$h_{structure}$	[m]	=	crest level of the flood defence
h_{water}	[m]	=	highest observed water level
f_b	[m]	=	freeboard

The other dike properties, like the materials of the core and cover, the angles of the inner and outer slope and the presence of berms, were traditionally chosen based on experience (mainly 'trial and error'). The deterministic method was applied for a long time, when it was not yet possible to make stability calculations due to lacking numerical insight into failure mechanisms and difficulties in estimating the governing loads. The freeboard, a safety margin, was not based on the quantification of the uncertainties. Therefore, with this method, it is not possible to determine the extent of over- or under-design.

6.5.3 Exceedance probability approach

The performance of a flood defence is usually expressed by its reliability, which is the probability that the flood defence fulfils its function. Reliability is complementary to failure probability. The target failure probability of a flood defence should be such that it complies with an acceptable risk of flooding of the hinterland[2] . The settling of the acceptable flood risk level (safety level) is based on answering the question 'how safe is safe enough', because 100% safety cannot be achieved.

[1] Freeboard = *waakhoogte* in Dutch, which is sometimes erroneously translated into English as *wake height* and into Dutch as *vrijboord*, which has a completely different meaning! In Dutch, *waakhoogte* is also known as *kruinhoogtemarge*, as proposed in Voorschrift Toetsen op Veiligheid voor de tweede toetsronde 2001 - 2006.

[2] Flood risk is defined as the probability of flooding multiplied with the consequences of a flood.

Three criteria can be distinguished in settling the safety level:

- Economic risk: the investments in flood protection should balance the therewith obtained risk reduction;
- Single casualty criterion (*individueel risico*[3]): the probability that a human being, residing at one location during one year, will die because of a flood;
- Multiple casualties criterion (*groepsrisico*[4]): the probability that a number of people, residing at one location during one year, will die because of a flood.

The most severe of these criteria determines the acceptable flood probability of an area. The allowable flood probability can be translated to a maximum failure probability of the flood defence. This failure probability is related to an extreme water level with a certain exceedance probability. In the exceedance probability approach, flood defences are primarily designed to resist this design water level. If this water level becomes higher, the flood defence is considered to fail. Therefore, the height of a dike is the basic design parameter. Other properties, like the geometry (shape) and materials, are designed such, that they provide sufficient stability of the flood defence.

Section 42.3 of the Manual Hydraulic Structures (2023) explains how the extreme design water level can be determined. The way of determining a dike height that can resist water levels that occur with the target exceedance probability is described in the same chapter of the Manual.

The Dutch Delta Committee followed three approaches to find water levels with acceptable exceedance probabilities. See Appendix A for a description of these approaches.

6.5.4 Failure probability approach

In the failure probability approach, the height of a flood defence is calculated as part of a probabilistic computation. The allowable failure probability per dike section is distributed over various failure mechanisms. Usually, only the main failure probabilities are considered, like erosion of the inner dike slope due to overtopping waves, macro instability (sliding of soil bodies), piping or uplift, failure of revetment or erosion of the outer slope, and non-closure of closing means. A dike section will fail, if one or more of these failure mechanisms occur. The allowable failure probability per failure mechanism is derived from the required failure probability requirement per dike section by means of a standard distribution of failure probabilities over the main failure mechanisms (Figure 6.6).

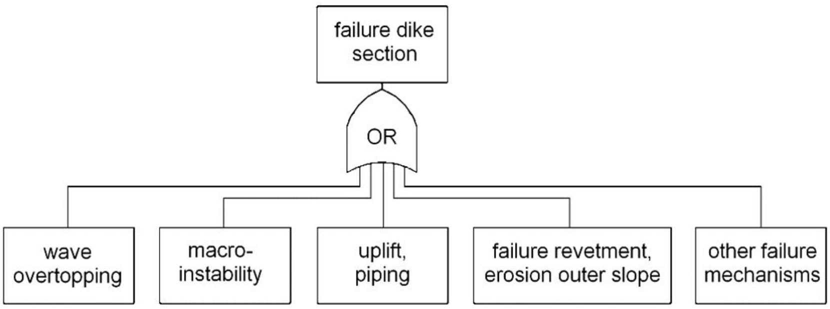


Figure 6.6: Basic fault tree of a dike section

In this approach, the height of a flood defence depends on the allowed probability of exceedance of a critical overtopping discharge. However, it is allowed to deviate from the standard failure probability. This implies that the height of a dike can also be lower than standard required, if the failure probability due to overtopping is compensated by, for example, a lower failure probability due to macro-instability (slip of a soil body), which can, for example, be obtained by a stability wall.

The determination of the height of a flood defence according to the failure probability approach is explained as part of the course of Flood Defences (CIE5314) and is not comprised in the present lecture notes.

[3]The Dutch term 'risico' here means 'probability,' which is not coherent with the definition or risk as we use it in engineering.
[4]See previous footnote

6.6 Standards and guidelines

6.6.1 The legal status of standards and guidelines

Standards, or 'codes', are statutory documents that have to be dealt with by force of law. In the structural engineering field, they are often material specific, or structure type specific. For instance, demands regarding the reference period and probability of failure of large hydraulic engineering projects can deviate from general standards. This means that, for instance, the partial safety factors (load and material factors) given in these standards cannot be applied. In addition, generally, characteristic values or load factors for typical hydraulic engineering loads, such as waves and current, are not specified in standards. The reason for this is obvious, as the statistics of waves and currents differ from one location to another and therefore cannot be generalised in a norm. Moreover, calculation rules regarding deviating reference periods are lacking in most standards and it is not specified how to deal with different acceptable probabilities of failure.

This is the reason why more specific guidelines, design handbooks and recommendations have been published and can be prescribed by the client. For probabilities of failure that deviate from the standards, one has to resort to probabilistic calculation techniques to determine the design values of the load and strength. For this, reference is made to course CIE4130 'Probabilistic design'.

6.6.2 Standards

The old Dutch TGB-standards ("Technische Grondslagen voor Bouwconstructies") were officially withdrawn per 31 March 2010 and were replaced by the Eurocodes, which have a similar structure as the TGB's. This manual sometimes still refers to the TGB-standards, which could be considered as outdated, however, for this course this is not a major problem because it deals with main principles in the first place.

Eurocodes

The basic Eurocode, EN 1990, describes the basic principles and load combinations. The loads for the design of buildings and other structures are elaborated in the ten parts of EN 1991. Material properties follow in EN 1992 (concrete), EN 1993 (steel), EN 1994 (steel-concrete), EN 1995 (timber), EN 1996 (masonry), EN 1997 (soil) and EN 1999 (aluminium). EN 1998 should be used for the design of structures for earthquake resistance. In addition to the general European standards, obligatory national supplements have been issued.

Below follows a list of Eurocodes. These codes are available (free for our students!) from the website of the library of Delft University of Technology (accessible from within Delft campus): http://www.library.tudelft.nl/, or more directly from: http://connect.nen.nl/.

EN 1990 – Eurocode: Basis of structural design
EN 1991 – Eurocode 1: Actions on structures
EN 1991-1-1 Densities, self-weight and imposed loads
EN 1991-1-2 Actions on structures exposed to fire
EN 1991-1-3 Snow loads
EN 1991-1-4 Wind loads
EN 1991-1-5 Thermal actions EN 1991-1-6 Actions during execution
EN 1991-1-7 Accidental loads due to impact and explosions
EN 1991-2 Traffic loads on bridges
EN 1991-3 Actions induced by cranes and machinery
EN 1991-4 Actions in silos and tanks

EN 1992 – Eurocode 2: Design of concrete structures
EN 1992-1-1 Common rules for buildings and civil engineering structures
EN 1992-1-2 Structural fire design
EN 1992-2 Bridges
EN 1992-3 Liquid retaining and containment structures

EN 1993 – Eurocode 3: Design of steel structures
EN 1993-1-1 General rules and rules for buildings
EN 1993-1-2 Structural fire design
EN 1993-1-3 Cold formed thin gauge members and sheeting
EN 1993-1-4 Structures in stainless steel
EN 1993-1-5 Strength and stability of planar plated structures without transverse loading
EN 1993-1-6 Strength and stability of shell structures
EN 1993-1-7 Strength and stability of plate structures loaded transversally
EN 1993-1-8 Design of joints
EN 1993-1-9 Fatigue strength
EN 1993-1-10 Fracture toughness assessment
EN 1993-1-11 Design of structures with tension components made of steel
EN 1993-1-12 Use of high strength steels
EN 1993-2 Bridges
EN 1993-3-1 Towers, masts and chimneys – towers and masts
EN 1993-3-2 Towers, masts and chimneys – chimneys
EN 1993-4-1 Silos, tanks and pipelines – silos
EN 1993-4-2 Silos, tanks and pipelines – tanks
EN 1993-4-3 Silos, tanks and pipelines – pipelines
EN 1993-5 Piling
EN 1993-6 Crane supporting structures

EN 1994 – Eurocode 4: Design of composite steel and concrete structures
EN 1994-1-1 General – common rules
EN 1994-1-2 Structural fire design
EN 1994-2 Bridges

EN 1995 – Eurocode 5: Design of timber structures
EN 1995-1-1 General rules and rules for buildings
EN 1995-1-2 Structural fire design
EN 1995-2 Bridges

EN 1996 – Eurocode 6: Design of masonry structures
EN 1996-1-1 Rules for reinforced and un-reinforced masonry
EN 1996-1-2 Structural fire design
EN 1996-2 Selection and execution of masonry
EN 1996-3 Simplified calculation methods and simple rules for masonry structures

EN 1997 – Eurocode 7: Geotechnical design
EN 1997-1 General rules
EN 1997-2 Ground investigation and testing

EN 1998 – Eurocode 8: Design of structures for earthquake resistance
EN 1998-1 General rules, seismic actions and rules for buildings
EN 1998-2 Bridges
EN 1998-3 Strengthening and repair of buildings
EN 1998-4 Silos, tanks and pipelines
EN 1998-5 Foundations, retaining structures and geotechnical aspects
EN 1998-6 Towers, masts and chimneys

EN 1999 – Eurocode 9: Design of aluminium structures
EN 1999-1-1 Common rules
EN 1999-1-2 Structural fire design

EN 1999-1-3 Structures subjected to fatigue
EN 1999-1-4 Trapezoidal sheeting
EN 1999-1-5 Shell structures

Other standards
Nederlands Normalisatie instituut: NEN 6702 Belastingen en vervormingen (TGB 1990)
Nederlands Normalisatie instituut: NEN 6740 Geotechniek, Basiseisen en belastingen
Nederlands Normalisatie instituut: NEN 6743 Geotechniek, Drukpalen
Nederlands Normalisatie instituut: NEN 6720 Voorschriften beton (VBC 1995)
Nederlands Normalisatie instituut, NEN 6008: “Steel for the reinforcement of concrete”, july 2008.
Nederlands Normalisatie instituut, NEN-EN 10080: “Steel for the reinforcement of concrete – Weldable Reinforcing steel - General”, juni 2005.
Nederlands Normalisatie instituut, NEN-EN 10138-1 Draft: “Prestressing steels – Part 1 to 4, september 2000.

6.6.3 Guidelines
Empfehlungen des Arbeitsausschusses "Ufereinfassungen" Häfen und Wasserstraßen EAU 2012. Arbeitsausschusses "Ufereinfassungen" Hamburg; Deutsche Gesellschaft für Geotechnik, Essen, Germany.
Handbook Quay Walls, second edition. SBRCURnet, Municipality Rotterdam, Port of Rotterdam. Published by CRC Press/Balkema, the Netherlands, ISBN: 978-1-138-00023-0 (eBook: ISBN 978-1-315-77831-0), 2014.
Handboek damwandconstructies, SBRCURnet-publicatie 166, 6e druk, Stichting CURNET, Gouda, 2012.
Hydraulische randvoorwaarden 2006 voor het toetsen van primaire waterkeringen. Technische Adviescommissie voor de Waterkeringen.
Ontwerp van Schutsluizen (2 delen). Ministerie van Verkeer en Waterstaat, Rijkswaterstaat, Bouwdienst.
Prediction of wind and current loads on VLCC’s, second edition. Oil Companies International Marine Forum: Witherby & Co Ltd., London, 1994.
Richtlijn Vaarwegen RVW 2020 Ministerie van Verkeer en Waterstaat, Rijkswaterstaat Adviesdienst Verkeer en Vervoer. Rotterdam, December 2020.
Wave Overtopping of Sea Defences and Related Structures: Assessment Manual ("European Overtopping Manual"). EurOtop 2018.
Coastal Engineering Manual. US Army Corps of Engineers; Coastal and Hydraulics Laboratory, 2006, United States.

Other useful guidelines and technical reports are produced by the ’Technische Adviescommissie voor de Waterkeringen’ which has been succeeded by the ’Expertise Netwerk Waterveiligheid’ (www.enwinfo.nl).

APPENDICES

A

ESTABLISHMENT OF BASIC LEVELS BY THE DELTA COMMITTEE

After the floods of 1953, the Delta Committee, according to its assignment, studied what level of flood protection should be considered high enough and how this safety level should be attained. The committee followed three ways of reasoning to find an acceptable safety level:

1. A historical study of high water levels, using studies of the Royal Dutch Meteorological Institute (KNMI);
2. An extrapolation of water level measurements to find what storm surge levels can be expected in future, using a statistical analysis of Rijkswaterstaat (carried out by Wemelsfelder);
3. An econometric optimisation, which comprised the execution of a cost-benefit analysis to find an optimum between investments in flood protection and the thereby obtained risk reduction, using studies of the Mathematical Centre (by Van Dantzig).

This appendix explains the details of these approaches and shows how the chosen exceedance probability principle was applied for the design of flood defences in the Netherlands.

A.1 First approach: historical study of high-water levels (approach main report)

The historical study is the approach of the main Delta report, in which the Delta Committee attempted to determine what maximum water level could physically be attained, starting with an attempt to find the highest storm surge level reached in the past (Chapter 3.0 of the final report). The storm surge of 1953 reached a level of NAP + 3,85 m at Hoek van Holland, which was the level of the normal astronomical tide (NAP + 0,81 m) plus a 'storm effect' of 3,04 m. That storm surge level was considered to have an average exceedance frequency of about 1/300 per year (Deltacommissie, 1960a, p. 30)[1].

It appeared that the water level of the 1953 storm surge exceeded all previously recorded water levels. The top level of 1953, NAP + 3,85 m, exceeded the highest recorded level of 23 December 1894 (NAP + 3,28 m) by more than half a metre. The most severe storm surge since 1800 occurred on 4 February 1825, when an area of 370 000 m^2 was flooded, almost three times as much as in 1953. Unfortunately, the maximum water level at Hoek van Holland in 1825 is not known, because no measurements were done at that location, but the committee concluded that it can be assumed that a storm surge like in 1825 would not have reached the level of 1953, even if the sea level rise since 1825 could be taken into account[2].

It was difficult to find out whether storm surges before 1825 were more severe than in 1953. Extensive description of the floods of 1421 (Saint Elisabeth's Flood), 1570 (All Saints Flood), 1686 and 1775 were available, but water levels were not measured at that time. The committee did not have the impression that these water levels exceeded the level of the storm surge of 1953. The circumstances during the storm surge of 1953, however, could have been worse. The Delta Report mentions that more unfavourable circumstances could

[1] Other mentioned frequencies are: 1/222 per year by the Mathematical Centre, (Deltacommissie, 1960c, p/ 72) and 1/250 per year according to the Storm Surge Report (RWS and KNMI, 1961, p. 108) and by Wemelsfelder, (Deltacommissie, 1960d, p. 77)

[2] For Texel, almost 200 km North of Hoek van Holland, the levels of 1825 and 1953 were comparable, but there they were considerably lower than in Hoek van Holland.

have caused an additional water level elevation of 1,15 m. An internal note of the Dutch contractor HBM explains that the 1,15 m elevation consisted of four components (Van der Pot, 1977):

1. The main contribution to the additional elevation of 1,15 m comes from the astronomical tide: 0,44 m should be added to the water level reached in 1953, because it was not as high as it could have been during the storm surge. Two days before the storm surge (i.e., on 30 January 1953, 0:44 o'clock) it was a full moon, which caused spring tide in Zeeland with a delay of about $2\frac{1}{4}$days. This means that on 1 February 1953, a spring tide occurred in the province of Zeeland, but it was not an extremely high one. This was caused by the distance between the moon and the earth, which was at a maximum on 1 February 1953 (the moon was in its apo-apsis) and thus the combined force of attraction of the moon and the earth was minimal.
2. The water level could have been an additional 0,30 m higher, if the course of the low pressure area of 1 February 1953 would have been the most unfavourable for the water levels along the Dutch South-Western coast.
3. If the maximum wind set-up would have coincided with the astronomical tide, the water level would have been another 0,21 m higher.
4. Resonance of the maritime basin, finally could have worsened the case with 0,20 m.

These effects, which could have aggravated the disaster, are presented in Table A.1. Adding these 1,15 m to the

effect	resulting elevation
extra tidal elevation possible to reach HAT	0,44 m
extra wind set-up possible, coinciding with HAT	0,21 m
extra elevation if more unfavourable course of low pressure areas	0,30 m
extra elevation due to resonance of the maritime basin	0,20 m
total	1,15 m

Table A.1: Additional effects that could have raised the extreme water level at Hoek van Holland in 1953

reached storm surge level of NAP + 3,85 m at Hoek van Holland, a 'basic level' of NAP + 5,00 m was obtained, which was finally chosen as a starting point by the Delta Committee. It was calculated excluding effects of future closure dams and other interventions, nor did it include effects of chart datum (NAP) subsidence or water level fluctuations of short periods.

The influence of the discharge of the main rivers on the storm surge level was taken into consideration. Rijkswaterstaat and KNMI (1961) describe that in 1953, the discharge of the rivers Rijn and Maas was lower than the usual winter average: there was only 67% of the average Rijn discharge (measured at Lobith) and 80% of the average Maas discharge (near Lith). This implies that the water level of the lower rivers could have been higher than in 1953. If the storm surge on 1 February would have coincided with the high discharge of 1941, the river levels would have been 0,13 to 0,50 m higher, depending on the location. For the calculation of the basic water level of Hoek van Holland, however, this river level elevation was not of any influence, because the water levels were measured at sea.

The physical approach as described in this section was criticised because any of the parameters that together constituted the 'storm effect' might still have been more unfavourable. The Royal Dutch Meteorological Institute (Koninklijk Nederlands Meteorologisch Instituut, KNMI) carried out studies that showed that considerably higher storm surge levels are physically possible. According to KNMI, a storm set-up of more than 5 m could be possible, about 2 m higher than observed during the storm surge of 1953 (Deltacommissie, 1960b). This would result in a basic level of NAP + 7,00 m (or even higher) near Hoek van Holland. The Delta Committee, however, considered this level impossible, because of meteorological reasons.

A.2 Second approach: Extrapolation of water level measurements (approach Wemelsfelder)

As already explained in the previous section, the Delta Committee assumed a water level of NAP + 5,00 m at Hoek van Holland as a basic level for further considerations. To find the corresponding exceedance frequency of this basic level, extrapolation of the found trend through storm water levels was necessary. Because of the uncertainties of the course of this line above NAP + 3,00 m, the Delta Committee had asked the Mathematical Centre in Amsterdam, with help of the Dutch Meteorological Institute, to assist. The Delta Committee also asked Rijkswaterstaat to study the problem, which was mainly carried out by Wemelsfelder, who was head of the Hydrometrical Department of Rijkswaterstaat. Their contributions can be found in the appendices of the Delta Report (Deltacommissie, 1960c,d).

In the statistical approach of Wemelsfelder, it is acknowledged that no maximum storm surge level can be found, but it was obvious that the likelihood of exceedance decreases substantially with the height of the water level. The exceedance frequency of extreme water levels was found by extrapolation of a series of water level measurements, far beyond the observation range. It should be noted that Wemelsfelder had access to a relatively large number of data, because water levels in the Netherlands were systematically measured since halfway the nineteenth century. Nevertheless, the measurement period was not long enough to obtain a good accuracy for modelling the tails of the water level distribution over time.

The Mathematical Centre, under guidance of David van Dantzig and prof. Jan Hemelrijk, studied the water level records of Rijkswaterstaat between 1888 and 1956 with aid of the Royal Meteorological Institute for making the selection of relevant data. Van Dantzig found an exponential function better suitable to describe the distribution of high-water levels than the Gumbel function that was proposed by Wemelsfelder. The results of both functions were comparable within the range of high water level up to NAP + 6,00 m, but the exponential function was easier to use in a mathematical sense and the extrapolation of the line beyond the measurement points was less arbitrary (Deltacommissie, 1960c, p.19).

Cor van der Ham, employee of the KNMI and the first weatherman on Dutch television, took several measures to 'homogenise' the data set (Deltacommissie, 1960c):

1. Measurement points were restricted to the months November, December and January, because of reasons of representativeness and only one measurement point was included per storm surge. The set of selected data was extensively analysed by the Mathematical Centre. It advised to use an exponential distribution with an exceedance frequency line that intersected a water level of NAP + 5,13 m at a frequency of 10-4 per year.
2. The selection was based on low pressure areas (causes of storms) instead of separate high-waters. Successive high-waters can namely be correlated (being caused by the same low pressure area), which is incorrect for a statistical analysis.
3. Van der Ham studied the tracks of the centres of the low pressure areas in the period between 1989 and 1953 that had caused a high or low water setup of more than 1,60 m near Hellevoetsluis. It appeared that not all low pressure areas were equally threatening. Only those, following a 'track' in a certain part of the North Sea, were considered dangerous for the Nether¬lands on meteorological grounds. Hoek van Holland was consequently chosen as a representative station for the Netherlands, insofar as it concerns the behaviour of severe storm surges. A selection was made of high water levels at Hoek van Holland with a set-up of minimal 0,50 m and a low pressure track through the corresponding window.

After the Mathematical Centre had presented its results, the Delta Committee consulted representatives of this institute and of the Department of Water Management (Directie Waterhuishouding en Waterbeweging) of Rijkswaterstaat. It was agreed to assume a work line as indicated in Figure 0 1: the thick line, with a bend at around NAP + 3,00 m. This graph shows the highest 30 storm surges plus the 40th surge. The relation between exceedance frequency and water levels is given by an exponential function, in accordance with the study of Wemelsfelder (1939), which results in a straight line through the part with not-extreme water levels, plotted on a half-logarithmic scale, see Figure A.1.

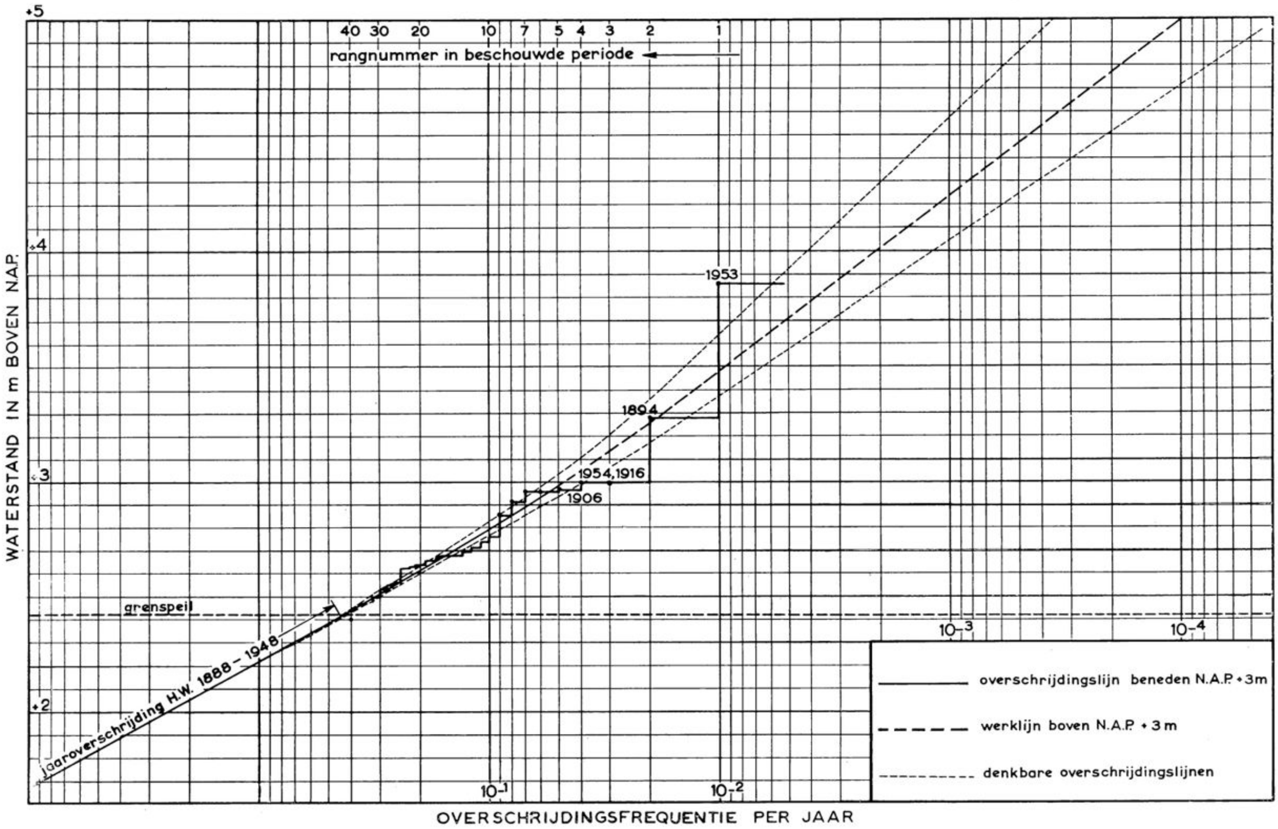


Figure A.1: Water level exceedance line at Hoek van Holland from measurements between 1859 and 1958 (Deltacommissie, 1960a)

The Delta Committee justified the bend just above NAP + 3,00 m by stating that, notwithstanding the fact that there are arguments to assume that the exceedance line above NAP + 3,00 m could deviate towards lower water levels than indicated by a straight line (downward deviation), the assumption was not supported by measurements. On the contrary, a deviation towards higher water levels was considered more likely, because of several highest measurement points. The presence of these highest measurement points was statistically not demonstrable, but if a larger class of distributions would be used as a base for the adoption of an exceedance line to the measurements, a considerable upward deviation would be obtained (Deltacommissie, 1960a). In a publication other than the contribution to the Delta Report, Van Dantzig gave an explanation for the bend: he suggested that the highest storm surges are caused by storms of a different type than the lower surges, which could cause a kink in the trend line. Van Dantzig remarked that the group of storms that followed the tracks, selected by Van der Ham and analysed by Hemelrijk of the Mathematical Centre, clearly resulted in a different straight line. The estimated halving height found by the Mathematical Institute was 0,21 to 0,25 m higher and the 95% confidence limit was 0,24 to 0,26 m higher. The estimated Wemelsfelder-line then became: $h = 2,03 - 0,75 \cdot log(p)$ (Van Dantzig, 1956).

After having initially agreed upon the work-line as indicated in Figure A.1, the Mathematical Centre made more calculations and found higher levels than NAP + 5,00 m for the average exceedance frequency of 10-4 per year. After more discussions with Rijkswaterstaat, the Mathematical Centre finally stated that it considered the level of NAP + 5,00 m 'not entirely unacceptable', though on the low side, as an estimate for the entirely statistically determined level with an exceedance frequency of 10^{-4} per year.

The relation between water level and exceedance frequency was found with the help of measurements over a long period of time. However, the level reached in 1953 was not included in this calculation. This omission is in line with the remark of Wemelsfelder, that 'the generic shape of a frequency curve should not include the highest, the one but highest and the two but highest levels' (Wemelsfelder, 1939). The highest measurements, namely, cannot be expected to be situated on the frequency curve, because the distribution of measurements becomes wider if the frequency decreases. Due to all the uncertainties, the Delta Committee advised to use the exceedance graph only with 'great caution' (Deltacommissie, 1960c).

The question then was what exceedance probability would be suitable as a criterion. Any chosen criterion is bound to be subjective, but the Delta Committee preferred to include flood consequences in the estimation of an acceptable safety level anyway. The committee considered a probability that an individual would die because of a flood reasonable, if this was 1% in a lifetime, or approximately 1% per 100 year (1 / 10 000 year). This is the exceedance probability that corresponds to the level of NAP + 5,00 m at Hoek van Holland according to the exceedance line preferred by the Delta Committee (Deltacommissie, 1960c; Valken and Bischoff van Heemskerk, 1963).

A.3 Third approach: Econometric optimisation (approach Van Dantzig)

Because the selection of a design level on the basis of physical or statistical considerations appeared to be unavoidably subjective, a joint economic and statistical basis was attempted as a third way of reasoning to solve the problem. The Delta Report therefore contains an econometric calculation, in which investments in protective measures are balanced with the therewith obtained flood risk reduction[3]. The insight into this approach was delivered by a contribution of Van Dantzig and Kriens (Deltacommissie, 1960c).

The main idea of the econometric optimisation was to calculate the total capital to be invested in flood prevention and the capitalized anticipated value of the margin of damage due to flooding, and then find the smallest value (Figure A.2). This principle was worked out analytically by the Mathematical Centre and graphically by Rijkswaterstaat. To estimate the risk reduction, Van Dantzig, of the Mathematical Centre, used the estimate of the Central Bureau for Statistics (Centraal Bureau voor de Statistiek, CBS) of $24 \cdot 10^9$ guilders for capital goods and sustainable consumptive commodities in central Holland (dike ring 14)[4]. This value was the magnitude of the consequences of a flood in case of complete loss of capital goods. Not included in this value were production deprivations and, to a much lesser extent, neither were losses of infrastructure, administered by the national authorities, social disruption and loss of lives. On the other hand, over-estimations were made in the econometric calculation: they consisted of partially preserved real estate in higher situated areas and partial preservation of productivity of the population. The net effect of this over- and under-estimation was that no adjustment in the estimation of the economic value of the area was made.

The calculations to find this optimum protection level, however, contained many uncertainties. To start with, only a tentative estimation could be made of the costs of the reinforcements of existing dikes, of the construction of new dikes, of carrying out other flood protection projects and of the capitalised expenditure of maintenance of such a large scale project. In addition, the magnitude of consequences of a flood was extremely difficult to estimate, because it was hard to forecast the economic developments, while the impacts of floods vary considerably. Furthermore, the selected rate of interest is an uncertain factor for the capitalisation of the damage. Yet, the uncertainties of extrapolation of the frequency curves are much bigger. Besides, the selection of the critical failure mechanism (wave run-up / overflow) introduces uncertainties, as many factors are not taken into account, such as factors related to dike construction. However, the population growth, as well as the economic development and numerous other imponderables, such as human suffering, loss of life, and disruption of daily life, were taken into consideration.

Van Dantzig was reluctant to express the value of human life in monetary units, out of ethical reasons. He considered to make a comparison with investments that were made in society to reduce other types of risk, or to look at the insurance benefits in case of loss of life, but these ideas appeared to lead to unacceptable or insignificant results. Therefore, Van Dantzig refrained from directly quantifying the value of human life. The same applies to cultural values. To nevertheless include non-economic values, Van Dantzig proposed to multiply the total economic value of a protected area with a factor to include non-economic values. He considered a multiplication factor of 2 'certainly not too high'. (Deltacommissie, 1960c). Instead of quantifying

[3] For the obtained risk reduction, the Committee used the present value of the imaginary insurance premium that would be required to cover the flood risk of the areas behind the dikes.

[4] In 2005 it was estimated at $290 \cdot 10^9$ euro (Rijkswaterstaat, 2005).

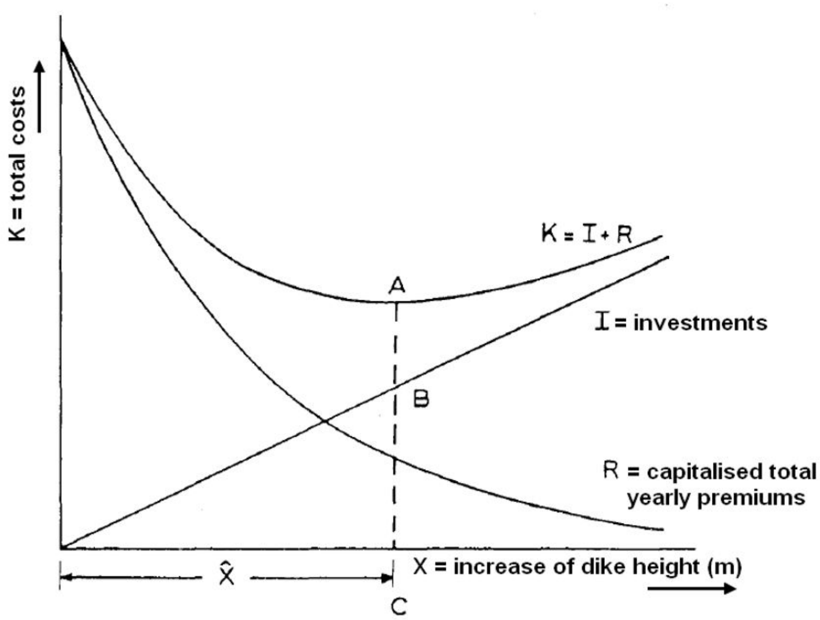


Figure A.2: Econometric estimate of investments and capitalised disaster damage, represented by yearly total premiums (Deltacommissie, 1960c)

human life, he reasoned what preventive investments per individual would be acceptable, compared to other types of disasters, and looked at life insurance premiums. He reasoned that even an amount of 100 000 guilders per individual, which would certainly go 'far beyond any sum that would be acceptable (...) as a norm for all cases, and would not lead to significant improvement of the flood protection level, corresponding to 0,03 m higher dikes. Any acceptable monetary equivalent for the loss of life, on the other hand, would not be feasible (Van Dantzig, 1956). He finally advised to base the factor on political considerations, not on mathematical, statistical, economic or technical analyses (Deltacommissie, 1960c).

It was then calculated by Van Dantzig that if a complete economic loss would occur with a probability of failure in a random year of 1/125 000, it would balance with the investments in risk reduction by flood protection, which were estimated 150 million guilders per year (net present value). After application of these dike reinforcement measures the flood risk, defined as the probability of occurrence of a flood in a year multiplied with the insured value, was estimated at 13,5 million guilders. This corresponds with a design water level of NAP + 6,00 m at Hoek van Holland, called the disaster level (*ramppeil*). The economic considerations mentioned in the foregoing analysis would be valid if a large number of risks could be insured on this basis. This, in fact, is not applicable to flood risk, because a succeeding flood would have a considerable influence on the outcome of the calculations. This is one of the weakest points of these econometric calculations. Notwithstanding the arbitrary outcome of the econometric approach, it gives a more insight in the involved factors than the previous described ways of reasoning (approaches 1 and 2) (Valken and Bischoff van Heemskerk, 1963).

Ultimately, the Delta Committee did not fully support the outcome of the advice of Van Dantzig: due to the lack of numerical insight in failure mechanisms it appeared impossible to determine the probability of failure of a dike. It also appeared that the assumption of a disaster with complete loss of goods in case of exceedance of the design level was overstated. The Delta Committee also deviated from the proposal of Van Dantzig to include non-economic losses in the analysis. Consequently, it did not adopt the advice to multiply the economic losses with a factor two to account for the loss of lives, because the assumption that a dike failure would result in maximal damage already implied an extra safety margin (RIVM, 2004).

The Delta Committee did not adopt the disaster level of NAP + 6,00 m as a design level, as proposed by Van Dantzig, because exceedance of the design level would, after all, not immediately result in maximum damage. Instead, the committee translated the failure probability criterion of 1/125 000 (with a corresponding disaster level of NAP + 6,00 m) into an exceedance probability of 1/10 000 (and corresponding 'design level' of NAP +

5,00 m) at the reference location of Hoek van Holland (Deltacommissie, 1960a).

A difference of opinion arose between the Delta Committee and Van Dantzig, on exactly this issue. Van Dantzig stated that the committee would in the future regret its too low standard (RIVM, 2004). The committee admitted that a maximum storm surge level could not be estimated, thus the probability of a disaster remained, whichever storm surge level would be selected as a base for reinforcement of primary flood defences. The committee recognised that other considerations could lead to higher safety standards, but was of the opinion that flood risk should not be regarded in an isolated way, instead should be considered in relation to other types of risks. With respect to this fact, the committee considered the proposed basic levels related to a 1/10 000 exceedance probability as an acceptable limit for the risk of storm surges. Moreover, levels based on a 1/10 000 norm would obtain a safety standard as much as 30 times higher than the storm surge level of 1953 (Deltacommissie, 1960a).

The Delta Committee thus found a probability of 1/10 000 during a random year acceptable for the exceedance of the design water level. Finally, it was calculated whether the investments needed to accomplish this safety level could be afforded by the Dutch state. The investments in flood protection for the first 20 to 25 years were estimated at 2,0 to 2,2 billion guilders in total, or 100 to 125 million guilders per year, assuming that construction works would take about 20 to 25 years. One year of investments equalled about 10% of the economic damage caused by the storm surge of 1953, which could be afforded in a short term without severe disruptions. Compared to the total of 27,6 billion guilders of total national expenditures in 1955, the protection of the Netherlands at the indicated level would cost 0,5% of these expenditures, which was considered affordable and acceptable (Deltacommissie, 1960a).

A.4 Basic design levels for other locations than Hoek van Holland

The Delta Committee strived for an equal flood risk all over the Netherlands. This implies that the parts of the Netherlands with a lower value (industry, houses, inhabitants etc.), it is accepted that they flood more frequently . The flood-prone part of the Netherlands is therefore divided into dike ring areas (*dijkringgebieden*), areas that are surrounded by dikes or other flood defences that have their own acceptable exceedance probability, which is stipulated by law (Water Act, Waterwet). These numbers are partly determined by using economic calculations, but have been adjusted by political considerations. See Figure A.3.

With help of the thus determined acceptable exceedance probabilities per dike ring area, the Delta Committee established 'normative high-water levels' (NHW) for a number of locations along the coast, where measurements had been carried out for many years. These normative high-water levels can be used as design water levels, if local and temporal effects are taken into account as well. The normative high-water levels were found by relating the water levels to acceptable exceedance probabilities and are listed in Table A.2 for a few locations.

exceedance frequency	Delfzijl	Den Helder	Scheveningen	Vlissingen	Bath
10^{-1}	4,10 m	2,75 m	3,05 m	3,85 m	4,75 m
10^{-2}	4,95 m	3,40 m	3,70 m	4,40 m	5,45 m
10^{-3}	5,60 m	3,95 m	4,40 m	4,95 m	6,10 m
10^{-4}	6,20 m	4,45 m	5,15 m	5,50 m	6,75 m
1 Feb. 1953	-	3,25 m	3,97 m	4,55 m	5,60 m

Table A.2: Normative high-water levels along the Dutch coast according to the exceedance probability approach

Along rivers, the normative high-water level can be determined with help of computer models that calculate the distribution of river discharges over the river branches.

The thus determined critical water level along coasts and rivers is the assessment level (*toetspeil*), as listed in the report Hydraulische Randvoorwaarden (Hydraulic Boundary Conditions) of Rijkswaterstaat. This book is used to assess the reliability of existing flood defences.

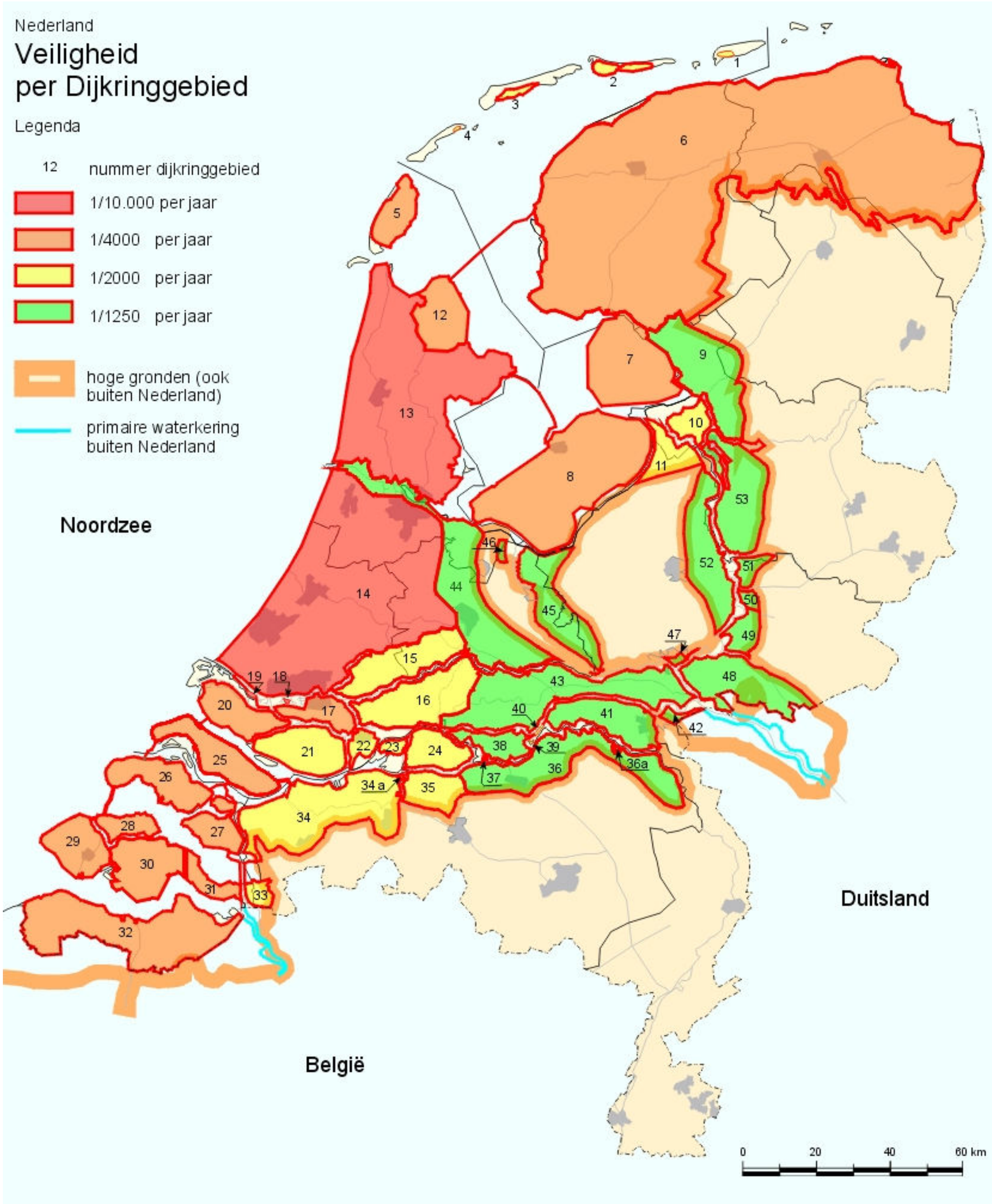


Figure A.3: Dike ring areas with an approximately equal flood risk, but different exceedance probabilities

A.5 Stipulation of the approach in guidelines and laws

In the Netherlands, the target safety level, based on an exceedance probability approach per dike ring, was used in internal guidelines of Rijkswaterstaat since 1960, according to the studies of the Delta Committee. After the completion of the Deltaworks, in 1996, the safety levels were stipulated in the Act on the Flood Defence (*Wet op de Waterkering*), to ascertain continuity in maintaining the safety levels. In 2009, this act was combined with several other acts in the Water Act (*Waterwet*). Since 1 January 2017, the policy changed to a failure probability approach per dike segment (change of the Water Act).

B

EXAMPLES OF SYSTEMATIC DESIGN CALCULATIONS

This appendix shows a clear and systematic elaboration of design calculations. The style of reporting such a calculation differs from the normal reporting style, where the information is merely presented in the shape of much text. For calculations, it is much more appropriate to present as much information as possible in the form of (simple) sketches (for example for boundary conditions) and lists (for the parameters required for a model or equation). Methods and parameters should be motivated as much as possible (but also as concise as possible) to avoid wrong assumptions or using methods beyond their limitations.

As mentioned in Section 2.5.5, the general form of structural calculation is as follows:

1. Make a situation sketch by hand (cross-section) including water levels and ground levels;
2. Choose the most critical load situation (from the construction sequence) and sketch a load diagram by hand; motivate selected methods for estimating the magnitude of the loads;
3. Think of a way to resist the loads and figure out how they can be transferred to the foundation and further towards the subsoil, thus the resisting forces;
4. Make a hand sketch of the structure or element, indicating components, materials and connections (this can be in various shapes);
5. Make a mechanical scheme of the supporting structure or element; Motivate your assumptions and simplifications, for instance when considering only two dimensions instead of three;
6. Check the overall stability of the structure, including soil bearing capacity and settlements. Furthermore, check dimensional stability (*vormvastheid*). Consider both SLS and ULS conditions. Mention the source of the model or equation and motivate why it is applicable to the actual situation. If it is not (entirely) applicable: reason how the model can be changed, or find another approach to do the check;
7. Check the strength of the structural elements, considering potential failure mechanisms. Determine critical cross-sections and draw shear force and moment diagrams. Mention the source of the model or equation and motivate why it is applicable to the actual situation. If it is not (entirely) applicable: reason how the model can be changed, or find another approach to do the check;
8. Check whether deflections/displacements are acceptable (SLS);
9. Check whether the designed component fits in the entirety (connections, space);
10. Reflect on the outcome of the calculation: is the magnitude reasonable? Comparison with similar structures under similar conditions will provide information about the order of magnitude.

B.1 Example1: Concrete cover

Given

The abutments of a storm surge barrier are made of reinforced concrete C35/45 with gravel diameters ≤ 28 mm. The design life time is 100 years. The abutments are cast in-situ in a cofferdam with no ensured quality control.

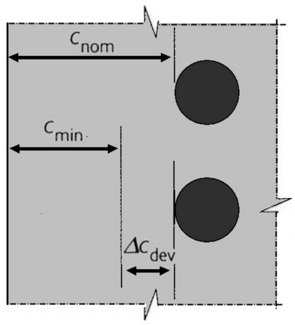
Asked

Calculate the minimum required concrete cover of the abutments according to Eurocode 2.

Elaboration

The required concrete cover c_{nom} according to Eurocode 2 (Manual HS §29.6) is:

$c_{nom} = c_{min} + \Delta c_{dev}$



1. Determination of c_{min}
c_{min} is the minimum cover and is the maximum of three values:
$c_{min} = \mathrm{MAX}\{c_{min,dur}, c_{min,bond}, 10\}$
where:

- $c_{min,dur}$ which is the minimum cover required for durability [mm].
- $c_{min,bond}$ which is the minimum cover required for bonding [mm].
- 10 mm

1a. Determination of $c_{min,dur}$
$c_{min,dur}$ depends on the exposure class and the structural class of the hydraulic structure. The procedure is as follows:

- Determination of the exposure class XS
The corrosion at the front wall of the abutment is induced by seawater. Part of the wall is persistently submerged and another part is in the tidal-, splash- and spray zone. The most severely exposed part is the tidal- and splash zone. According to Table 29-9 of the Manual HS this implies an exposure class of XS3, which was chosen for the entire front wall of the abutment.

- Determination of the structural class S
The standard structural class for hydraulic structures is S4, but may have to be adjusted, depending on the following criteria as mentioned in Eurocode 2 (Table 29-10 of the Manual HS):

- The design life of the hydraulic structure is 100 years (given), therefore the class has to be increased by 2.
- If the concrete used is ≥ C45/55 then the class must be reduced by 1, which is not the case since C35/45 is used (given).
- The slab geometry, the class must be reduced by 1 if the position of reinforcement is not affected by the construction process. This is not the case, because the abutment is constructed in situ inside a cofferdam.
- The ensured quality control of concrete manufacturing then the class must be reduced by 1.

This is also not the case: Quality control is not ensured, as the abutment is cast in-situ and not in a factory under controlled circumstances.

This results in a structural class S(4+2+0+0) = S6 for the front wall of the abutment.

- Determination of $c_{min,dur}$
According to Eurocode 2, Table 29-11 of the Manual HS, for exposure class XS3 and structural class S6, it can be found that cmin,dur equals 55 mm.

1b. Determination of $c_{min,bond}$
For $c_{min,bond}$, the minimum cover because of bond, it applies that:
$c_{min,bond} > \phi_{bar}$, 28 mm (assumed for the time being) or:
$c_{min,bond} > \phi_{bar}$ + 5 mm if the largest grain size is 32 mm: no gravel of > 32 mm will be used in the concrete mixture to avoid bad concrete casting.
$\rightarrow c_{min,bond}$ = 28 mm

1c. Resulting determination of c_{min}
$c_{min} = \text{MAX}\left\{c_{min,dur}, c_{min,bond}, 10\right\} = \text{MAX}\{55, 32, 10\} = 55\text{mm}$

This value should be increased in case of finished concrete surfaces, or floors cast on rough surfaces, but that is not the case for the abutment wall.

2. Determination of Δc_{dev}
Δc_{dev} = margin for construction intolerances = 5 mm in the Netherlands (Eurocode 2; Manual HS §29.6)

3. The resulting minimum concrete cover
$\rightarrow c_{nom} = c_{min} + \Delta c_{dev} = 55 + 5 = 60$ mm

Reflection on the result It is common for hydraulic structures to have concrete covers of 50 mm or more, therefore a cover of 60 mm for an abutment of a storm surge barrier seems a realistic value.

B.2 Example 2: Dike height

The example below is to demonstrate the right built-up of a technical calculation. In the present engineering practice, the crest level of a dike is usually calculated with a wave overtopping limitation, not with wave run-up. Furthermore, the determination of the governing wind direction often requires a more elaborate analysis of wind directions, velocities, water depths etc., which is not demonstrated here to keep the example clear for educational purposes.

Given

A sea dike with an orientation from north-west to south-east and an outer berm of 2 m wide at storm surge level. The outer slopes (above and under the berm) have a slope of 1:3. The revetment of the outer slope consists of concrete blocks (basalton). The toe of the dike is at CD - 1,00 m. The inner slope has an inclination of 1:4 and the crest width is 4,0 m. The coastline is straight and there are no harbour moles or groynes at the considered location.

More given parameters:

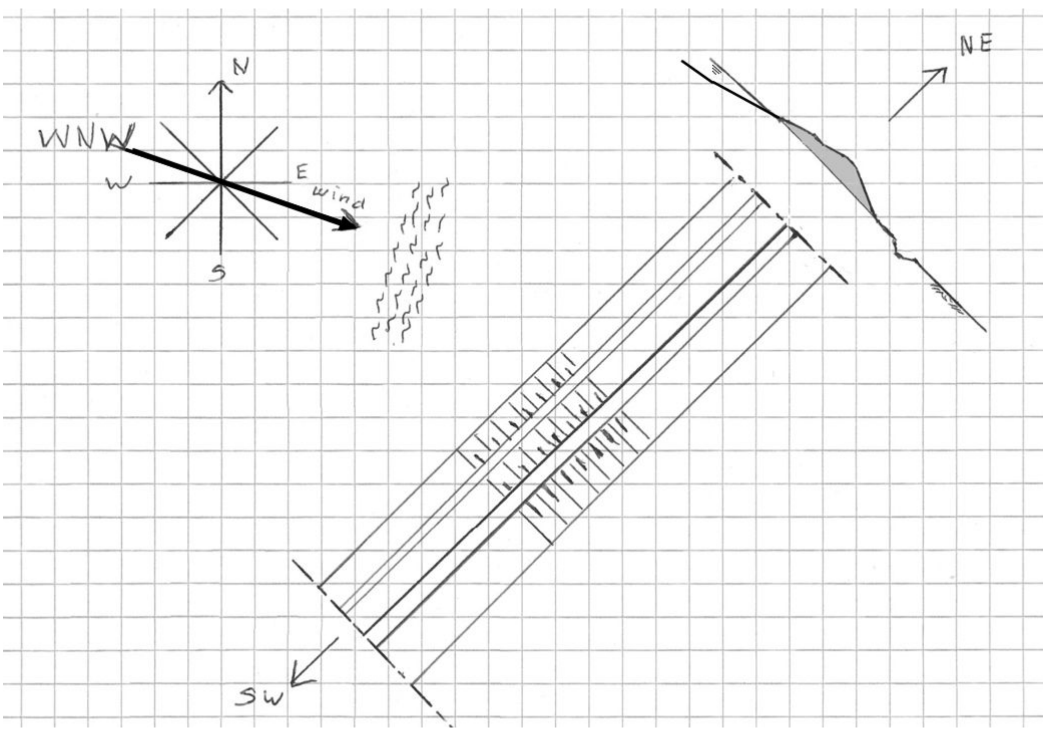
- The design water level is DWL = CD + 2,00 m (including wind setup and sea level rise during the design life-time and including the effect of short-term atmospheric depressions)
- The governing wind direction is WNW (West-North-West)
- The design significant wave height at sea is H_s = 2,0 m
- The design wave length at sea L_0 = 45 m

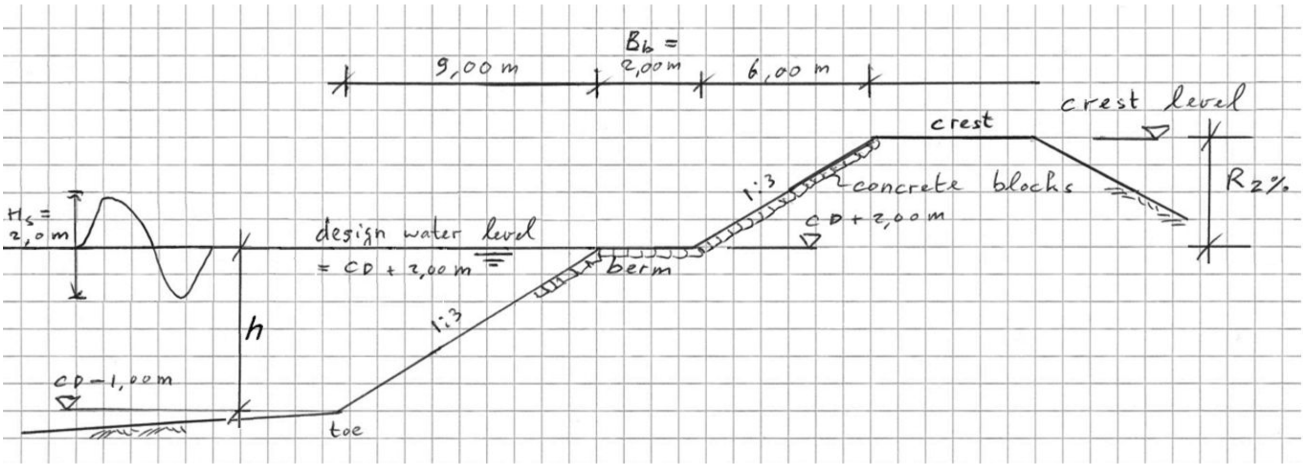
Asked

Determine the design crest level of a sea dike, based on a maximum wave run-up of 2%.
Mathematical surplus heights (for example due to the length effect) can be ignored in this example.
No safety factors have to be taken into account for this conceptual design.

Elaboration

Situation sketch (top view and cross-section)





Description of the main calculation method of the crest height

The crest level of the dike should be such, that it only allows 2% of the waves during a design storm to overtop the crest. The crest level, CL, consists of the governing water level (= design water level, DWL) + a run-up height $R_{2\%}$ that will only be exceeded by 2% of the waves:

CL = DWL + $R_{2\%}$

DWL is already given, so we only need to find $R_{2\%}$ It can be determined with the Van der Meer equation, which was included in the TAW-CUR guidelines and the Overtopping Manual of 2018:

$R_{2\%} = 1,75 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0} \cdot H_{m0}$ with a maximum of $R_{2\%,max} = 1,07 \cdot \gamma_f \cdot \gamma_\beta \left(4,0 - \frac{1,5}{\sqrt{\gamma_b \cdot \xi_{m-1,0}}}\right) \cdot H_{m0}$

where:

$R_{2\%}$	[m]	=	the 2% wave run-up height
γ_b	[-]	=	berm influence factor
γ_f	[-]	=	factor taking the slope roughness into account
γ_β	[-]	=	influence factor for oblique wave attack
$\xi_{m-1,0}$	[-]	=	breaker parameter
H_{m0}	[m]	=	estimate of the significant wave height from spectral analysis = $4\sqrt{m_0}$.

Motivation of the method

The Van der Meer equation as found in the Overtopping Manual of 2018 is the most advanced of the available run-up equations, including the effects of oblique wave attack, the presence of a berm, the roughness and slope of the outer slope and the wave steepness, which is the case for this calculation.

Critical circumstances

The situation that causes the higest wave run-up, is governing for the design. This is not necessarily the situation with a wave direction that is perpendicular to the dike, because another wind direction could be related to a a higher wind velocity, longer fetch or deeper water depth, which could result in a higher run-up. In this example, it was already given that the governing wind direction during a storm is from WNW. The water level during these circumstances is a high water level with the same return period as the storm (they are both related to the wind direction and wind velocity).

Motivation of the parameters of the Van der Meer equation

1. The design significant wave height H_{m0}

$H_{m0} \approx H_s$ (Holthuijsen, 2007) = 2,0 m (given)

However, $H_s < 0,4$ to $0,5h_{water}$ (the water depth at the toe of the structure), because otherwise the waves would break (Manual Hydraulic Structures 2019 §11.3). In this case:

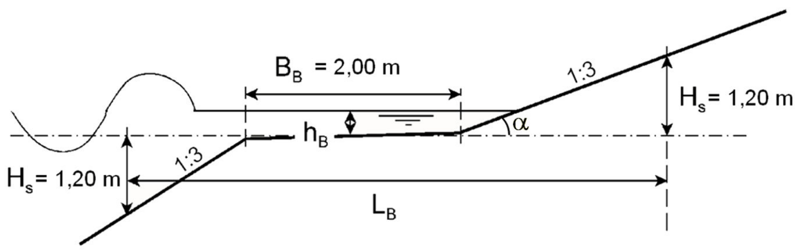
$H_{s,max} = 0,5 \cdot h = 0,5 \cdot 3,00 = 1,50$ m

Therefore, $H_s = 1,50$ m will be used in the run-up calculation.

2. The berm influence factor γ_b

γ_b is determined with the equation given in TAW technical report on wave run-up and overtopping (2002) as mentioned in the Manual Hydraulic Structures 2019:

$\gamma_b = 1 - \frac{B_B}{L_B}\left[0,5 + 0,5\cos\left(\pi \frac{h_B}{x}\right)\right]$ and $0,6 \leq \gamma_b \leq 1,0$



(not to scale)

where:

B_B	[m] =	berm width = 2,00 m (given)
L_B	[m] =	influence length of the berm: LB = BB + $2/tan(\alpha) \cdot H_s = 2,00 + 2 \cdot 3 \cdot 1,50 = 11,00$ m
h_B	[m] =	water depth above the berm. During critical conditions (storm surge), the water level is at its design height and the berm is at the same level, so h_B = 0,00 m.
x	[m] =	influence height of the berm: $x = R_{2\%}$ if the berm is above SWL $x = 2H_s$ if the berm is at or below SWL, which is the case: $\rightarrow x = 2H_s = 2 \cdot 1,20 = 2,40$ m (berm at SWL)

$\rightarrow \gamma_b = 1 - \frac{B_B}{L_B}\left[0,5 + 0,5\cos\left(\pi \frac{h_B}{x}\right)\right] = 1 - \frac{2,00}{11,00}\left[0,5 + 0,5\cos(0)\right] = 1 - \frac{2,00}{11,00} \cdot 1 = 0,82$

This is within the boundaries of $0,6 \geq \gamma_\beta \geq 1,0$, so OK.

3. The slope roughness influence factor γ_f

The outer slope consists of basalton block revetment (given), so γ_f = 0,90 (Overtopping Manual 2018, Table 5.2)

4. The influence factor for the angle of wave incidence γ_β

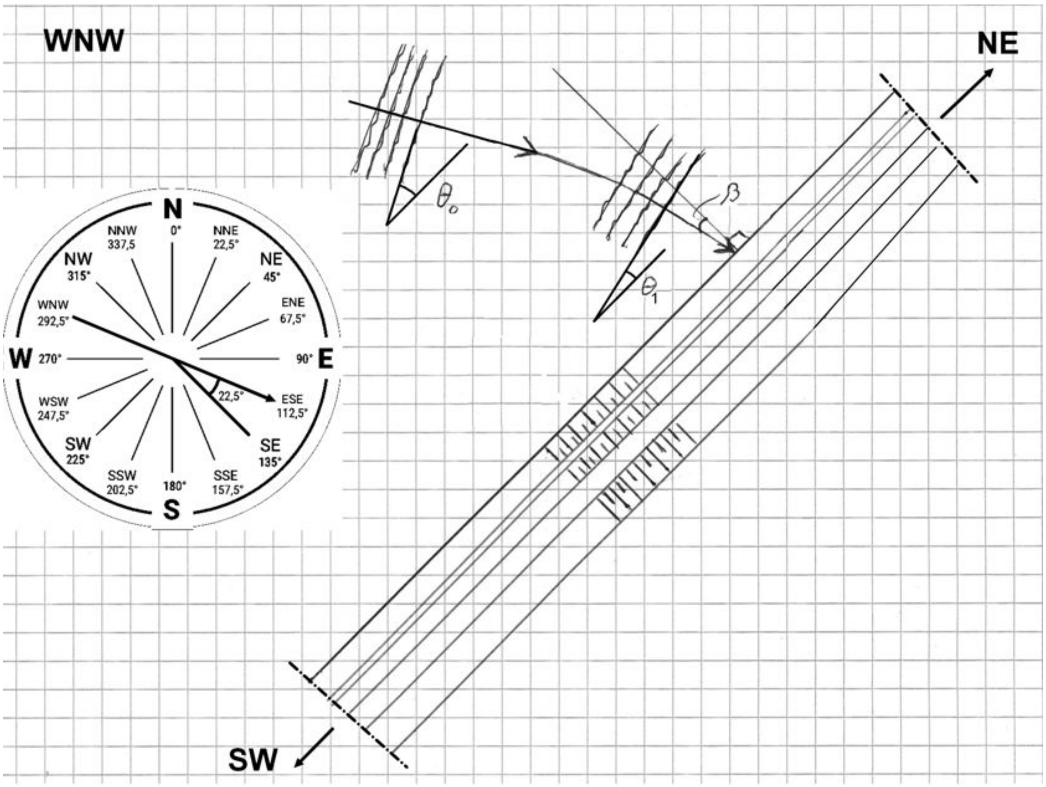
The wave incidence influence factor is given by:

$\gamma_\beta = 1 - 0,0022\beta$ for $0° \leq \beta \leq 80°$ (Overtopping Manual 2018, Equation 5.28)

where β [°] = the angle between the wave direction near the dike and a line perpendicular to the dike axis. β Depends on the governing wind direction and the dike orientation. The governing wind direction at sea is given: During the design storm conditions, the wind comes from the West-North-West. The waves at deep sea will have the same direction as the wind.

The wave direction can in principle change when the waves approach the dike because of diffraction or refraction. As the coast line is straight and there are no breakwaters or groynes in the vicinity, there is no diffraction near the dike. However, refraction can be expected, resulting in a deviation of the wave direction from the wind direction.

The wind and wave angles of approach at 'deep' sea from WNW direction and the dike orientation is from SW to NE, see figure below.



To calculate the change in wave approach angle due to refraction, Snell's law can be used, because we assume depth contour lines parallel to the dike:

$$\frac{sin(\theta_1)}{sin(\theta_0)} = \frac{L_1}{L_0}$$

in which:

θ_0	=	angle between the wave crest and the depth contour line at deep sea
L_0	=	wave length at deep sea
θ_1	=	angle between the wave crest and the depth contour line in shallow water
L_1	=	wave length in shallow water

(Holthuijsen 2007, p. 204)

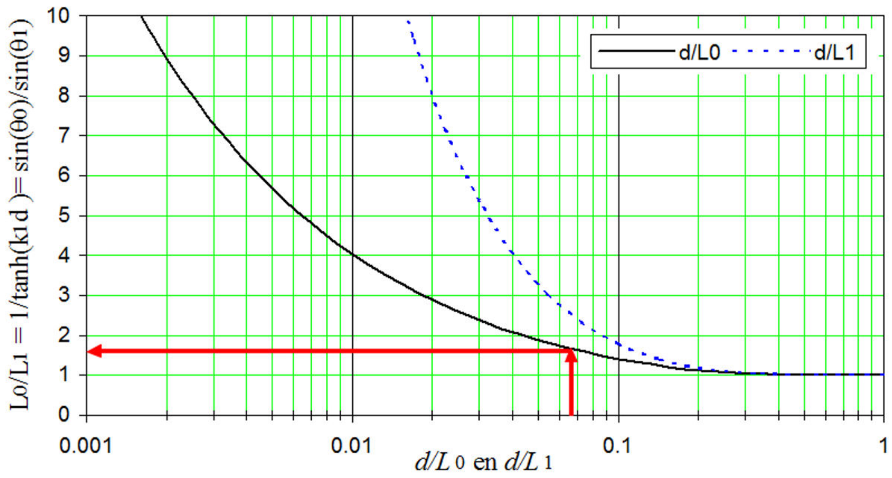
Snell's Law can be written as: $\theta_1 = \arcsin\left(\frac{L_1}{L_0}\right) \cdot \sin(\theta_0)$

in which all parameters are known, except for θ_1 and L_1. However, with d/L_0 known, L_1 can be found with help of the following graph (from Manual Hydraulic Structures, §10.1), which is a graphical presentation of Snell's Law (see graph).

$d = d_1 = h = 3{,}00$ m (water depth at the toe of the dike)
L_0 = deep water wave length during governing conditions = 45 m (given)
$\rightarrow d_1/L_0 = 3{,}00$ / 45 = 0,067

From the graph follows that, for $d_1/L_0 = 0{,}067$, $L_0/L_1 \approx 1{,}65$,

$\rightarrow L_1 = L_0/1{,}65 = 45/1{,}65 = 27{,}3$ m



θ_0 = the difference wave front with depth contour lines at sea'
The wave front is from NNE to SSW and depth contour lines are from NE to SW (like the dike) $\rightarrow \theta_0 = 90/4 = 22,5°$ (see the compass rose in the sketch).

$\rightarrow \theta_1 = \arcsin\left(\frac{L_1}{L_0}\right) \cdot \sin(\theta_0) = \arcsin\left(\frac{27,3}{45,0}\right) \cdot \sin(22,5) = 37,35 \cdot 0,38 = 14,29°$.

This θ_1 is the same as β in the equation for the wave incidence influence factor:

$\beta = \theta_1 = 14,29$

$\rightarrow \gamma_\beta = 1 - 0,0022\beta = 1 - (0,0022 \cdot 14,29) = 0,97$

(For simplicity, we could have chosen $\beta = 0°$, so $\gamma_\beta = 1,00$, which is slightly more conservative)

5. The breaker parameter $\xi_{m-1,0}$
The breaker parameter, defined by Iribarren (Battjes, 1974, see Manual HS §10.3), is:
$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{m0}/L_0}}$

where:

$\tan\alpha$	=	the angle of the outer slope of the dike = 1:3 (given)
H_{m0}	≈	H_s at sea = 2,00 m (given)
L_0	=	wave length at sea = 45,0 m (given)

$\rightarrow \xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{m0}/L_0}} = \frac{1:3}{\sqrt{2,00/45,0}} = \frac{0,3333}{0,2108} = 1,58$ [-]

Calculation of the run-up height
All parameters known, the run-up height can be calculated:
$\rightarrow R_{2\%} = 1,75 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0} \cdot H_{m0} = 1,75 \cdot (0,82 \cdot 0,95 \cdot 0,97) \cdot 1,58 \cdot 1,50 = 1,75 \cdot 0,76 \cdot 1,58 \cdot 1,50 = 3,15$ m

Check of the maximum value:
$R_{2\%,max} = 1,07 \cdot \gamma_f \cdot \gamma_\beta \left(4,0 - \frac{1,5}{\sqrt{\gamma_b \cdot \xi_{m-1,0}}}\right) \cdot H_{m0} = 1,07 \cdot 0,95 \cdot 0,97 \cdot \left(4,0 - \frac{1,5}{\sqrt{0,78 \cdot 1,58}}\right) \cdot 2,00 = 5,22$ m

Check of the maximum value of the run-up (validity of the Van der Meer equation):
($R_{2\%,max}$ = 5,22 m) > ($R_{2\%}$ = 3,15 m) → check OK

Furthermore, $R_{2\%}$ > 0,50 m minimum freeboard, which is the case → check OK

So, $R_{2\%}$ = 3,15 m

Calculation of the dike crest level
The crest level of the dike, CL, thus becomes:
CL = DWL + $R_{2\%}$ = CD + 2,00 m + 3,15 m = CD + 5,15 m

Reflection on the result
Compared to Dutch sea dikes that have a crest level of about 10 or 11 m above average sea level, a crest level of CD + 5,15 m seems to be a bit low. However, in our example, the design water level is only 2,00 m above average sea level (CD), compared to about NAP + 5,00 m for Dutch sea dikes. Furthermore, the significant wave height in our example is 2,00 m, which is quite low compared to Dutch sea dikes, where the significant wave height at sea is usually more than 8,00 m, which will reduce to half the water depth at the toe of the dike, resulting in a significant wave height of about least 2,5 m H_s. These Dutch circumstances are a bit higher than in our example, therefore it seems plausible that the crest level of our example is a few metres lower than the Dutch sea dikes.