CT3330

Hydraulic Structures

Caissons

Lecture notes

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M.Z. Voorendt, W.F. Molenaar, K.G. Bezuyen

TU Delft

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PREFACE

These lecture notes are the result of teamwork. Wilfred Molenaar initiated the work and gave valuable directions for improvement. Kees Bezuyen advised on the design approach and Henk Jan Verhagen helped with the reasoning behind the closure of the tidal basin in the design example. Cor Ramkema gave valuable comments based on his experience with caisson design and construction. Improvements to the use of the English language were made by Joris Schoolderman. All this help is highly appreciated. I am also greatly indebted to Professor Han Vrijling for giving me the opportunity to in this way pass on a part of the knowledge and experience of generations of hydraulic engineers.

Delft, January 2011
Mark Voorendt

READER TO THESE LECTURE NOTES

These lecture notes on caissons are part of the study material belonging to the course 'Hydraulic Structures 1' (code CT3330), part of the Bachelor of Science education and the Hydraulic Engineering track of the Master of Science education for civil engineering students at Delft University of Technology. Many of the principles and engineering techniques treated in the BSc curriculum have to be applied when designing a caisson. The challenge for students is to combine (and refresh) the already gathered knowledge and build up experience to develop a broader perspective on the design of hydraulic structures in general.

Because of their high impact on the design, construction aspects are also treated in these lecture notes. We, however, realise that these aspects differ considerably from site to site, hence this important matter surely cannot be discussed in all its details and varieties. Despite the endeavour to sketch the design and handling of caissons to its full extent, these lecture notes should therefore not be considered as a complete guide for caisson design. Instead, the general way to deal with a broad range of aspects that has to be taken into account will be illustrated. It should be emphasised that the designer should use all his imaginative powers and common sense to deal with possible future problems related to the project at hand. The ancient philosopher Lao Tse already noticed that it is easier to solve a problem before it occurs. This is why in engineering practice making an appeal on the experience of colleague-designers is very useful and if circumstances differ too much from preceding cases, scale model experiments should be carried out, to prevent problems during construction and operation. Especially problems occurring in the latter case can be rather time-consuming and very expensive to solve.

For the sake of the user's convenience, relevant parts of the other course material for Hydraulic Structures (the 'general' lecture notes and the 'manual') have been copied into these lecture notes (mostly in the appendices), so only the underlying volume has to be consulted to prepare a first conceptual caisson design. Subjects very specific for immersed tunnel elements, that in a way could be considered as caissons, have been omitted because they are dealt with in the course on bored and immersed tunnels (CT5305).

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

1. Introduction to caissons

1.1 Definition

The name 'caisson' is French and is to be translated as a 'large chest', which refers to the general shape of caissons. In civil engineering a caisson could be defined as a retaining watertight case (or box), in order to keep out water during construction, but also for more permanent purposes. Caissons are always part of a larger structure, such as a breakwater, substructure or foundation. Therefore, caissons serve a wide variety of purposes in bridge, quay, lock head, breakwater or many other projects. Caissons are also the result of a development of prefabrication to avoid the painstaking and costly construction of in-situ concrete 'in the wet'. Frequently, caissons are prefabricated and transported to their final position at a later moment in time. There they will be handled mainly in two ways, dependent on the caisson type.

1.2 Types

Generally spoken, two main types of caissons can be distinguished in civil engineering: standard caissons and pneumatic caissons (Figure 1-1). In some literature also an 'open caisson' type is mentioned: more or less a standard caisson without bottom plate, but this type is not considered in these lecture notes.

1.2.1 Standard caisson

The standard caisson generally will be prefabricated, transported over water, and immersed until it rests on the river or sea bed, where it has to fulfill its function. For positioning and to prevent undue settlements of the caisson, the bed has to be prepared with a stone layer, concrete pads or a sill. The standard type of caisson, sometimes referred to as 'box caisson', has a bottom plate, side and head walls to enable it to float. Sometimes there is a roof, which can be prefabricated or constructed after partial immersion. A variation to the standard type is the sluice or flow-through caisson, which has temporary gates in the walls in order to reduce the current in the remaining gap after one or more caissons are put in place besides each other.

1.2.2 Pneumatic caisson

The other main caisson type, the pneumatic caisson, is constructed on groundlevel and has to be subsided into the soil, which can be achieved by digging from within the caisson under compressed air. Because of the 'diving bell principle' used for this caisson, it is called a 'pneumatic' caisson. Below the bottom plate there is an enclosed work space where workmen can dig and from where excavated soil can be removed. A cutting edge (*snijrand*) around the bottom plate facilitates the subsidence into the soil.

The advantages of pneumatic caissons are the needlessness for dewatering (with pumps), the relatively small space requirements around the caisson and the possibility to subside it without major dredging works. A major disadvantage is the necessity to work in compressed air, which requires worksmen to make use of slow decompression afterwards, to avoid caisson illness.

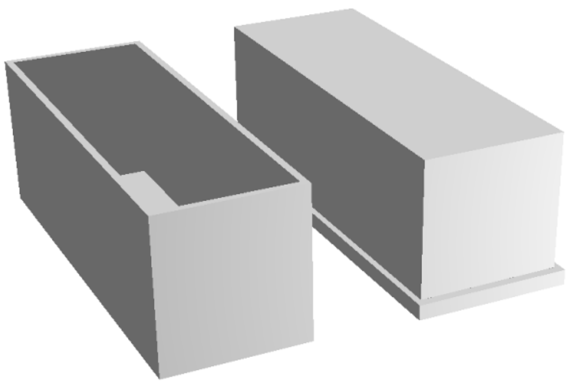

Figure 1-1 Schematics of the two main caisson types: standard caisson (left) and pneumatic caisson (right)

1.3 Final positions of caissons / where caissons end up

The question 'where caissons end up?' will be answered looking at the position where the caisson remains by far the largest part of its service life. During its service life a caisson usually is part of a larger structure and often preferred to in-situ construction if the spot is difficult to reach. In the Figures below the most common positions of caissons in some typical hydraulic structures are shown; no doubt there is a variety of other possibilities. In these structures, caissons can be used stand-alone, or lined up (Figure 1-2 and Figure 1-3). With respect to the connection with the soil, a distinction can be made between free standing and partially or completely embedded caissons (Figure 1-4 and Figure 1-5).

Single or Stand-alone

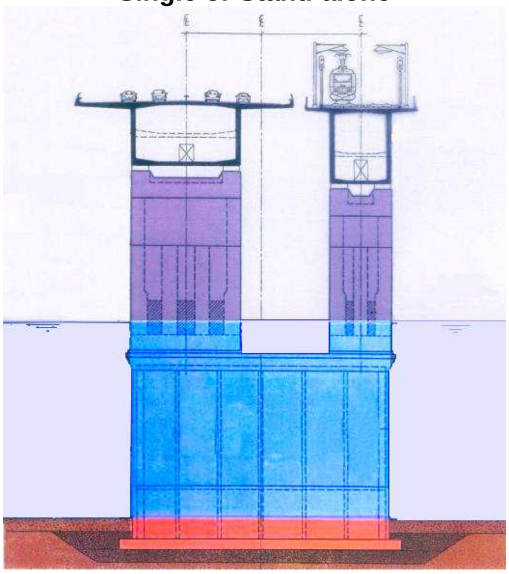

Figure 1-2 Stand-alone caisson used for the foundation of a bridge pier

Joint or In line

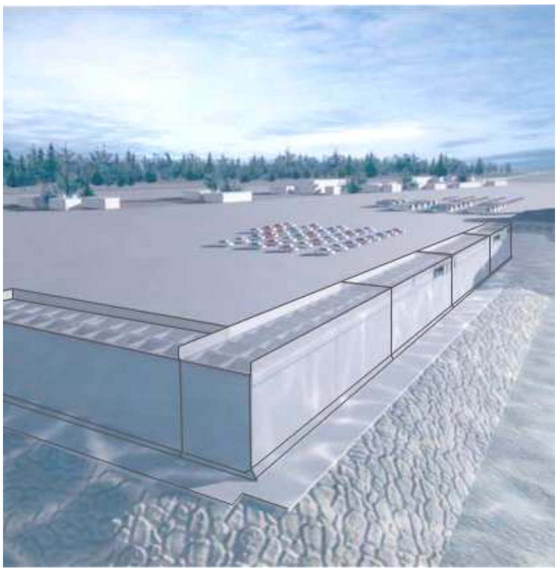

Figure 1-3 Caissons standing in line used for a quay

Especially for bridges, design teams are often split into a substructure and a superstructure design team. Caissons always belong to the substructures and are usually the most important structural element of the foundation.

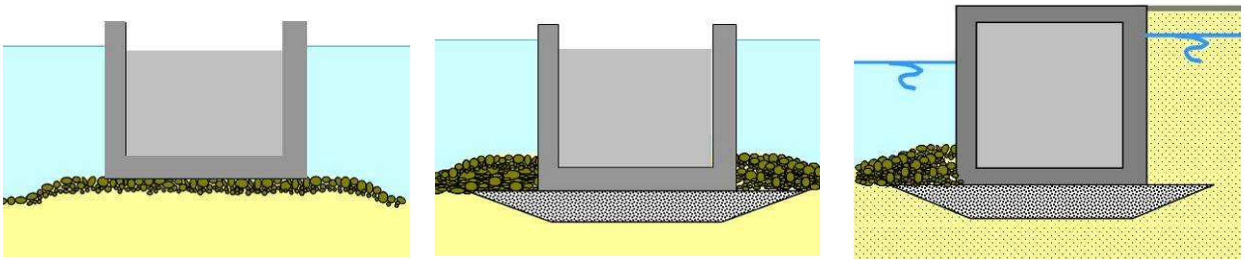

Figure 1-4 Caisson standing free (left) and partially embedded (middle & right)

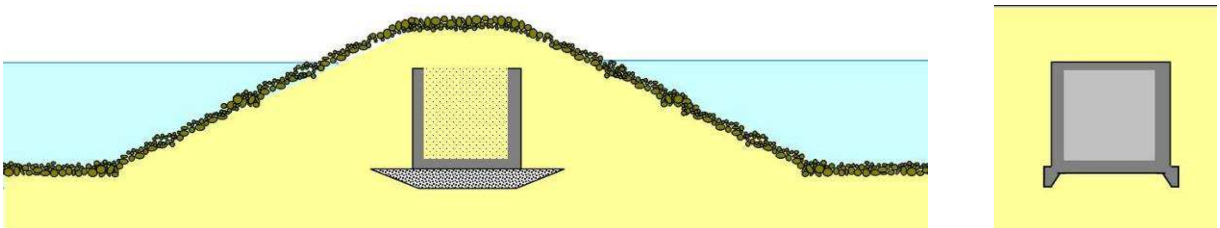

Figure 1-5 Completety embedded/covered caissons; standard caisson (left), pneumatic caisson (right)

1.4 Functions

The main functions of caissons generally are soil or water retention and transfer of vertical and horizontal loads into the subsoil. Less frequent functions are provision of space for equipment or machinery, and locking through of ships, if the caisson is part of a lock or barrier.
More specifically with respect to application, one could distinguish the following applications:

- closure of breaches in dikes and dams (closed and flow-through caissons)
- breakwater
- quay wall
- storage
- tunnel element
- foundation for bridge pier, lighthouse, wind mill, etc.
- specials:
  - casing for hydro-electric plant
  - gate for a dry-dock.

Of course a combination of functions can be made, like in Monaco, where a 352 m long caisson functions as breakwater, quay and car parking. In Monaco this structure is known as ‘digue flottante’, Anglo-Saxons describe the structure more correctly as a floating breakwater. Another example of the combination of functions has been suggested in the MSc-thesis of Krol [2007], where a quay wall existing of caissons is also used for the storage of crude oil.

1.5 Construction of standard caissons

Figure 1-6 shows a flow chart for the construction of (standard) caissons. Depending on the specifics of the project, often determined by the geography of the final caisson location, the activities shown may be in or excluded. Though the activities are presented more or less in a linear time sequence in reality they may take place simultaneously.

Chapter 3 deals in more detail with standard caisson construction.

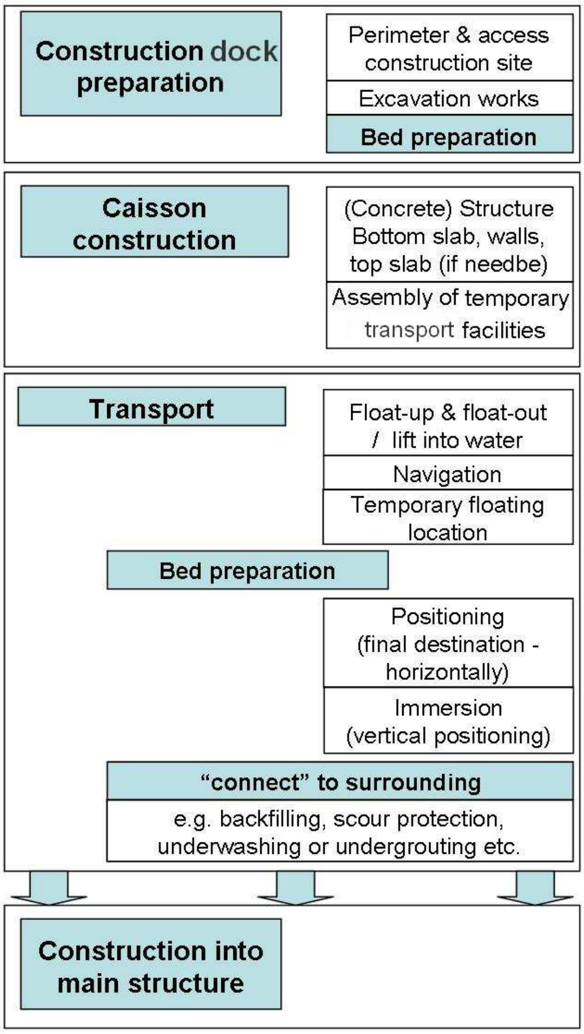

Figure 1-6 Flow chart for construction of standard caissons

2. Caissons through the ages

2.1 Ancient times

Caissons in civil and military engineering have been used since the era of the Roman Empire for various purposes. The first application of caissons found in the research for these lecture notes, is in about 250 years BC, in Alexandria, Egypt, where watertight caissons have been used to construct quay walls. A timber mould (*mal*) was constructed as part of a timber caisson and mortar blocks were cast in this mould. With help of the floating caisson, this mould was then positioned at the required location for the quay wall [*De Gijt, 2010*].

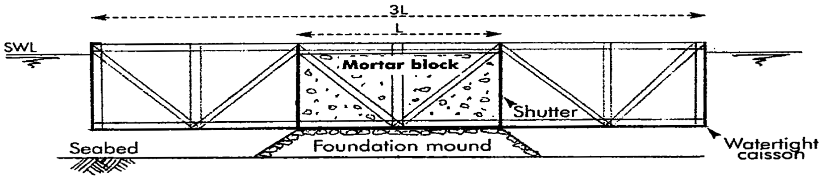

Figure 2-1 Floating caisson used to transport a mortar block, Alexandria 250 BC

Later on in history, 13 years BC, king 'Herod the Great' ordered the construction of the port of Caesarea, Judea, which became the largest on the eastern Mediterranean coast. The mole (*havenhoofd*) was built of floating units: timber casings that were prefabricated, transported over water (floating) and on the right location immersed by ballasting with stone (Figure 2-2). The dimensions of these caissons were 15 x 5.5 x 2.7 m; the water displacement was 220 tons. [*Bernshtein 1994*]

About 1500 years later, (1552) the first Russian Tsar, Ivan the Terrible, used caisson-type timber frameworks in the assault of Kazan, Russia. These caissons were prefabricated and then floated down the Volga River to be used in the assault. Lots of other construction works were carried out to defeat Kazan, e.g. the digging of tunnels to undermine the town (with help of gunpowder). [*Bernshtein 1994*]

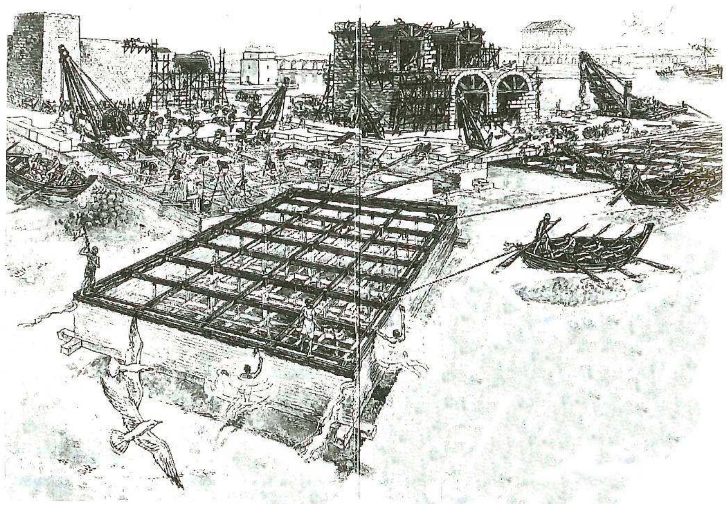

Figure 2-2 Timber caisson for the mole of the port of Caesarea, Judea, about 13 BC

Some two centuries later, Robert Weldon, a British engineer, invented a ship elevator, which he called a 'Hydrostatick Caisson Lock'. This caisson lock is a type of canal lock, and was intended to raise and lower ships in the Shropshire Canal (a tub boat canal built to supply coal, ore and limestone to the industrial region of east Shropshire). The vertical transport of ship took place in an immersed, sealed caisson box that moved up and down in a big water container, a cistern (Figure 2-3). Several tests were carried out from 1792 to 1799, first on a half-scale prototype model. During one test (the last one), the invited investors almost suffocated because the caisson got stuck, after which they withdrew from the project. In 1817 another test of a caisson lock was carried out in north London (Camden lock), but this was also not very successful [*Somersetshire coal canal society, 2008*]. No matter how interesting this invention might be, this type of lock is not what we normally denominate as a caisson.

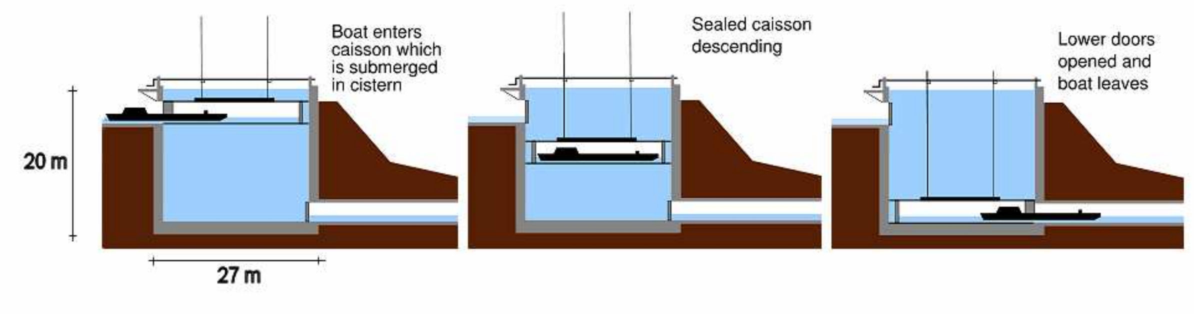

Figure 2-3 Robert Weldon's caisson lock at Oakengates

During the mid-19th century, cribworks with compressed air were used to construct shafts in coal mines. The overpressure drove the water out of the shaft, so workers could continue digging deeper in dry work space. This was first carried out in France (1841) and later (from 1850) also in English coal mines (Figure 2-4). [*Nebel, 2007*]

Engineers found out that in the same way they could construct foundations for bridge piers, which they first did in Vichy (France), later also in England (e.g. for the piers of the Royal Albert Bridge in Cornwall, 1859 and the Firth of Forth railroad bridge in Scotland, 1890) and the United States of America (e.g., the Brooklyn Bridge in New York and the Mississippi Bridge in St. Louis), followed by other countries. [*Nebel, 2007*]

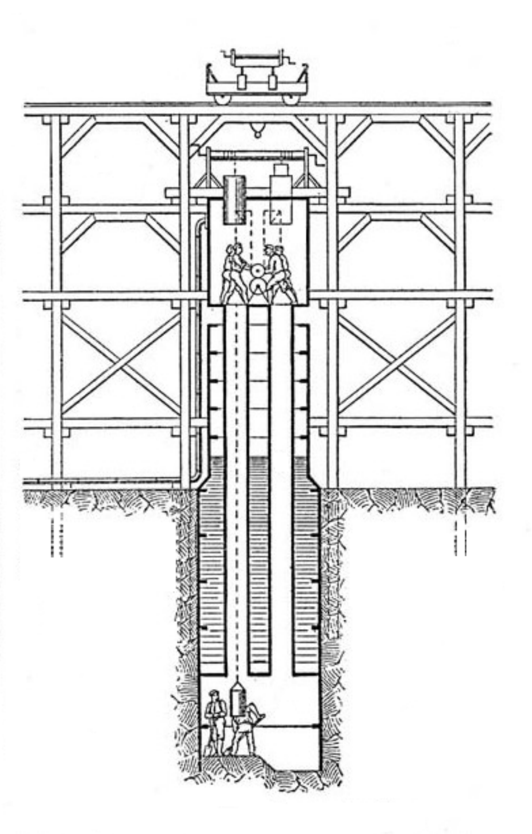

Figure 2-4 Impression of the application of pneumatic caissons in mine shafts

The functioning of a pneumatic caisson is as follows. Because of overpressure inside the airtight partition of the caisson, dry workspace is created for workers to allow them to dig (Figure 2-5). They have access to the work space through an air lock which ensures that the overpressure is maintained. Water locks are used to get excavated earth out of the work space: Wells (or shafts) are placed in the caisson in such a way that the lower endings of these tubes reach a pit in the soil below the inside water level. The water remains in the tube because of the compressed air inside the caisson. The water rises in the tube and in this way the air is locked out. Excavated earth and stones can be dumped into the pit, from where it can easily be removed by simply reaching under water with buckets. Because of the excavation and with help of the cutting edges and weight of the caisson it will dig itself into the soil. Sometimes extra mass on top of the caisson is needed for this.

This principle is illustrated with the Brooklyn Bridge foundation (New York, Figure 2-6). At the time of completion, 1883, this bridge, with a length of 1825 metre, was the largest and the first steel-wire suspension bridge in the world. The piers are founded on caissons, which were dug into the soil until a layer of bed rock was reached. The caissons were made of timber and were lined on the inside with boiler iron to make it air-tight. The drawing shows men working in compressed air. Also the muck tubes can be clearly distinguished. On top of the timber caisson, the masonry pier was built up, adding ever more weight. Brooklyn Bridge at this moment still rests on these original timber piers. [*Harper's Monthly, 1883*]

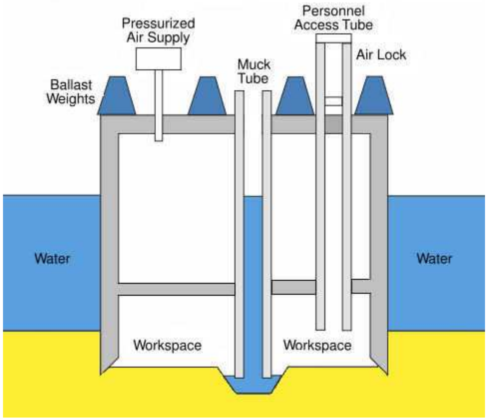

Figure 2-5 Schematic of a pneumatic caisson

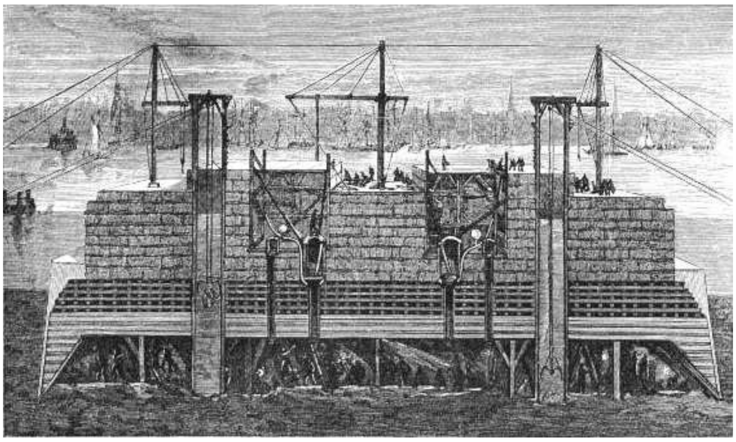

Figure 2-6 Foundation of a pier of Brooklyn Bridge, New York

This type of basement construction seemed to be very favourable, but for one reason or another, considerable numbers of workmen suffered from internal injuries or even died. These health problems soon appeared to be caused by the compressed air. Especially Paul Bert, who was a French physiologist and politician, studied the results of quick pressure changes on the human body. He discovered that the main problem was the decompression. If human beings get out of compressed air too abruptly, inert gases in the human body (generally nitrogen) which are normally dissolved in body fluids and tissues, will come out of physical solution and form gas bubbles. This, of course, is not favourable to the human body. Therefore, to prevent injuries and worse, workmen were advised, later as a standard prescription, to adapt slowly to atmospheric conditions in decompression tanks, after having worked in overpressure. [*The Columbia Electronic Encyclopaedia*, *2007*]

The experience with the founding of bridge piers on caissons nevertheless was positive, so the French structural engineer Gustave Eiffel selected this same method to found his prestigious tower for the World's Fair of 1889 in Paris. Variations on this theme of caisson foundation leads to applications like foundations for lighthouses (e.g., Baltimore, 1908) and basements for the supports of power transmission lines (like the 100-metre high supports over the Kakhovka water storage (Dnipro, Ukraine)).

It must have been around the time of the construction of the Eiffel Tower that engineers first thought of the idea to place digged-in or immersed pneumatic caissons in a row and then connect them by removing the head walls (after the joints had been made watertight). In this way they could construct tunnels, like was done for the metro in Paris (1906). These types of tunnel elements can be treated like caissons in many respects, but generally spoken, tunnel elements are not referred to as caissons.

2.2 Twentieth century

Another important field of application of caissons nowadays are ports and harbours, like in the beginning of caisson history. The harbour of Rotterdam, for instance, needed to be extended around a century ago. Pneumatic caissons were used to construct new quay walls below water level (Figure 2-7).

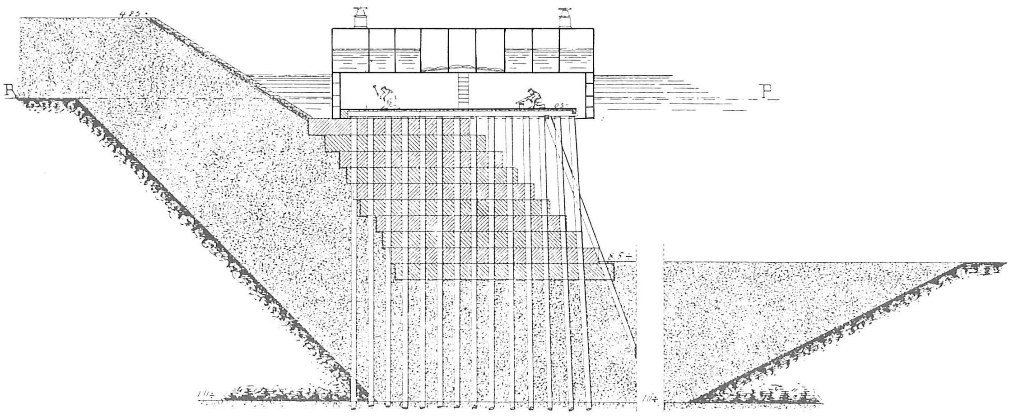

Figure 2-7 Pneumatic caisson for quay wall construction below water level, in Rotterdam

Standard caissons, the immersion type, were used for upgrading of old quays. They were put on top of the old piles (Figure 2-8). Later, reinforced concrete caissons were placed directly on the sand bed, forming a permanent structural element (Figure 2-9). These elements had a height of 11 metres and a length of 25 metres. The walls were tapered (*taps toelopend*), hence the shape of the structure's cross-section was trapezoidal, which was economic as regards the volume of material needed. They were ballasted with concrete and sand. These kinds of caissons were also used in other places of the world, like in the harbour of Talcahuano in Chilli and Surabaya (Indonesia) where they served as breakwaters.

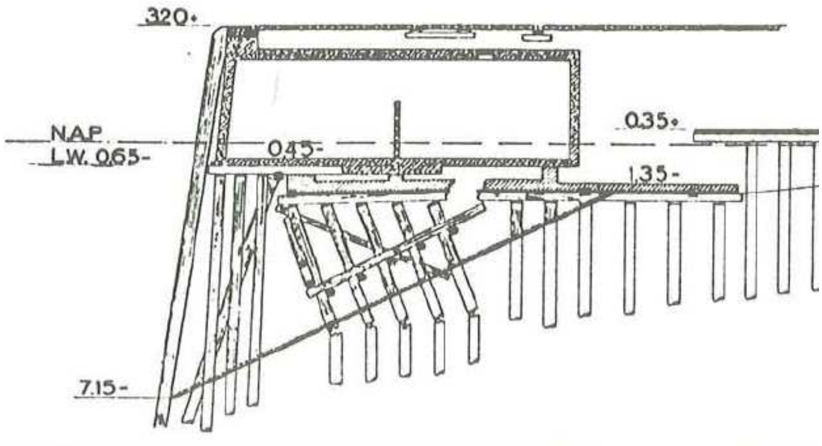

Figure 2-8 Upgrade of old quay walls in Rotterdam

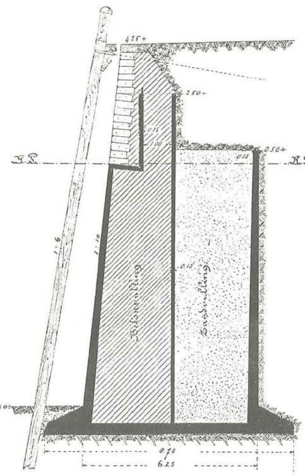

Figure 2-9 Quay walls without pile foundation in Rotterdam

Experience learned that considerable disadvantages of the applied trapezoid caisson shape are the laborious construction and the expensive formwork. Therefore engineers improved the shape by making it rectangular to avoid these drawbacks. [*HBG 1977*]

Caissons were also used in the Second World War during the Allied invasion of Normandy, France (June 1944). They were developed for the off-loading of cargo on the beaches. These caissons with code name 'Phoenix' were the appropriate solution for the rapid assemblage of breakwaters as part of temporary harbours. The dimensions of these caissons varied from 62 m (length) x 18 m (height) x 19 m (width) (type Ax) to 53 m x 7.5 m x 8.5 m (type D) (Figure 2-10) and the total number of Phoenix caissons built was 147. The Phoenix caissons were prefabricated in England. Their application as part of a breakwater on location in Normandy is shown in Figure 2-11. [*Heijkoop, 2002*]

The allied forces bombed sea dikes in Walcheren, Zeeland, late 1944, to inundate the island. In this way they wanted to prevent the Germans to attack Great Britain from there. Bombs ruined the dike of Westkapelle (along with part of the village), the Nolledijk near Vlissingen, the sea dike near Rammekens and a dike close to Veere. However, when the war was ended, it required a great deal of effort to reclaim the flooded part of Zeeland. After several attempts with other means, like sand bags and even a ship, everybody was convinced that caissons were needed for successful closures. The allied forces offered some left-over Phoenix caissons that were not needed any more for war purposes [*Heijkoop, 2002*]. In Figure 2-12 the closure of the Rammekens gap can be seen. An impression of the organisation of the closure works and the impact on the inhabitants of Walcheren is depicted by A. den Doolaard in his novel 'Het Verjaagde Water' ('Roll back the sea') [*Den Doolaard 2001*].

On 1 February 1953, a severe storm surge occurred resulting in dike failure at many places in Zeeland and Zuid Holland, and more than 1800 casualties. Once again Phoenix caissons were re-used to close the gaps. The last gap, near Kruiningen along the Westerschelde, was closed on 24 July 1953. A committee, appointed by the Dutch government, investigated how to prevent disasters like the February 1953 storm surge. After extensive study, the committee advised to protect the Netherlands against a storm surge of more than one meter higher than on February 1st. This protection was advised to be achieved by shortening the length of the coast by closing off arms of the estuaries or rivers (the so-called Deltaplan), which was preferred rather than strengthening and heightening of much more kilometres of dikes. Another advantage of closing off with respect to safety was: if the new dams would fail then the old dikes could still resist a storm surge, in fact a double safety was created.

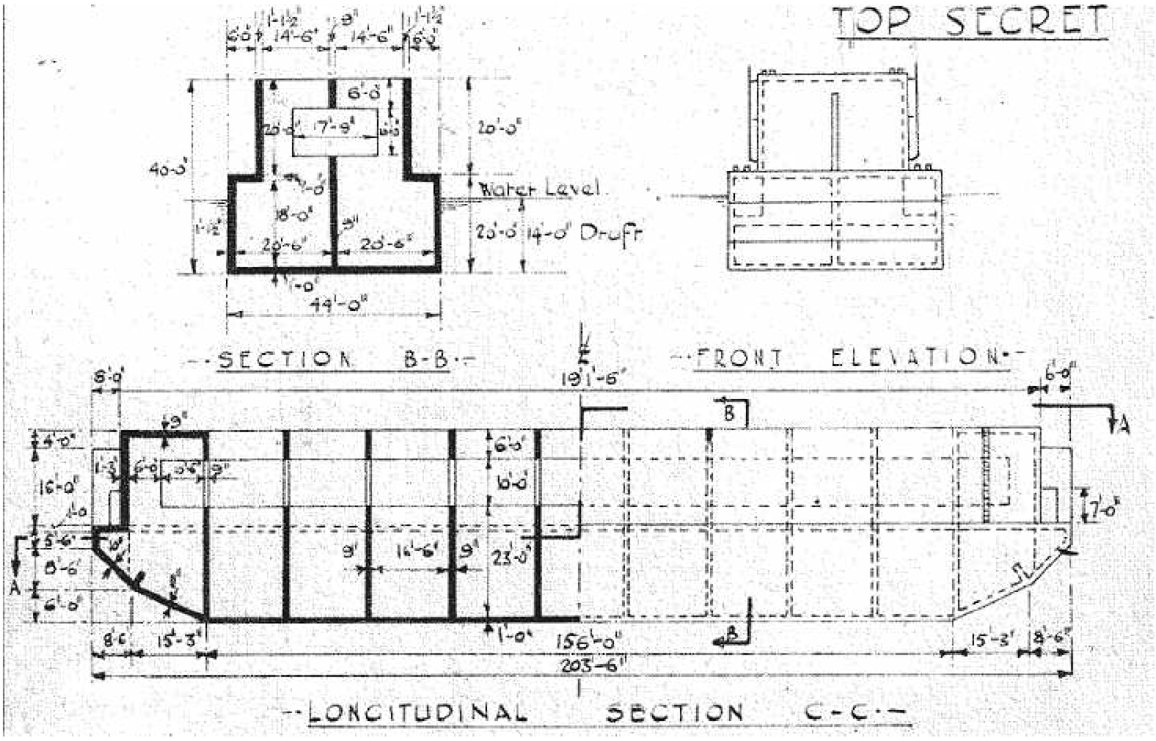

Figure 2-10 Technical drawing of a Phoenix caisson

The Deltaplan mainly comprehended closures of estuaries or sea arms. For every closure, the work or construction method had to be considered: a gradual or sudden closure.

The first closure in Zeeland, after the storm surge of 1953, was the closure of the Zandkreek (East of the Veersche Gat). Unity-caissons (*eenheidscaissons*) were used for this closure (Figure 2-13). They were prefabricated in the concrete factory of Kats (Noord-Beveland). The Zandkreek closure was relatively simple because the location is at the place where two opposing tides meet (*wantij*), so there is only a vertical water level variation and almost no horizontal flow. In this way, experience could be gained for more difficult closures like the next one near Veere.

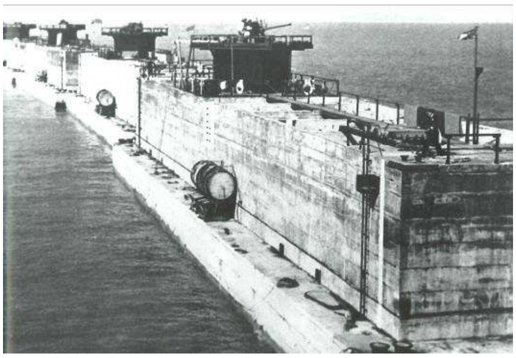

Figure 2-11 Phoenix caissons in Mulberry harbour, Normandy. Notice the anti-aircraft guns

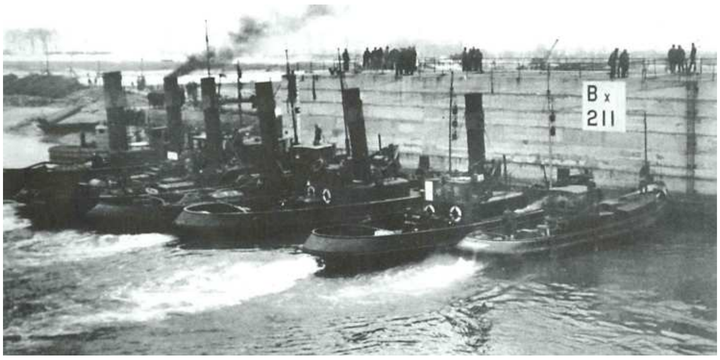

Figure 2-12 A Phoenix caisson closes the gap in the dike of Rammekens, Walcheren, 1946

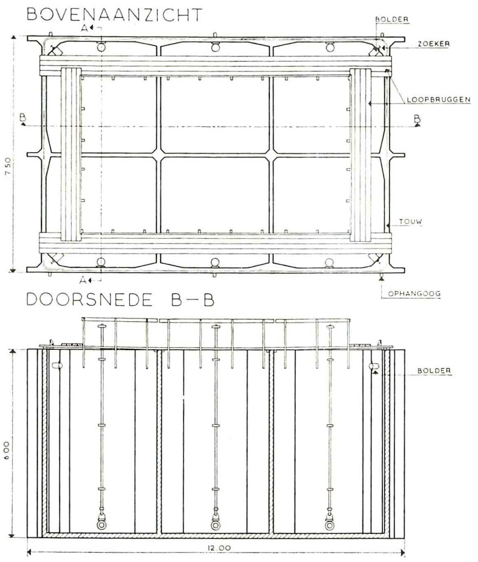

Figure 2-13 Unity caisson (eenheidscaisson)

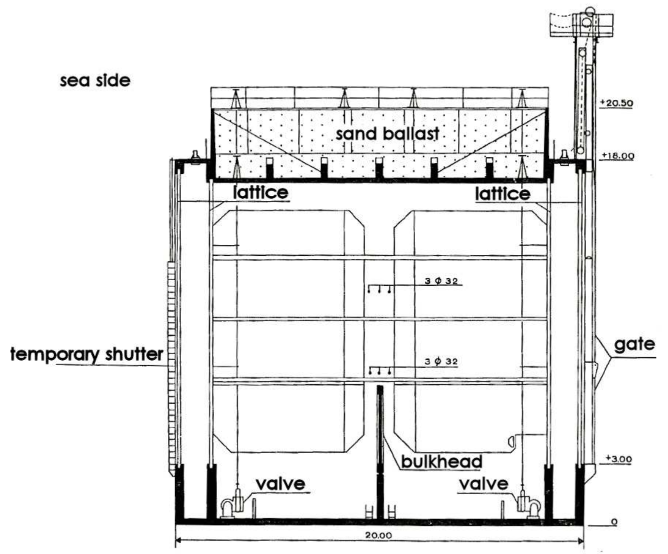

Figure 2-14 Sluice caisson with ballast box on top.
In this design the gates are positioned on one side.

The flow velocities in the Veersche Gat were expected to create problems after the partial closure with several caissons. That is why engineers invented sluice caissons (also known as flow-through caissons, or culvert caissons (*doorlaatcaissons*), see Figure 2-14), that allowed a discharge of about 50% of the original amount. During transport, temporary shutters on one side and closed gates on the other side kept out the water to make floating transport possible.

After placement of a caisson, the gates were opened and shutters removed, allowing water to flow through. When all caissons were immersed at their position, all gates were closed at once, which blocked all the flow immediately. Notice that the gates are at one side of the caisson. During transport, the other side of the caisson was closed with temporary shutters.

The next improvement to caisson design was made by positioning the steel gates in the middle of the element. This improved stability and flow pattern while gates are open. This type of sluice caisson is applied in the Volkerakdam and the Brouwersdam (Figure 2-15). The length-width ratio is 3.8:1, which proved to be very favourable for the manoeuvrability of the caisson during transport [*Deltadienst 1957-1987*].

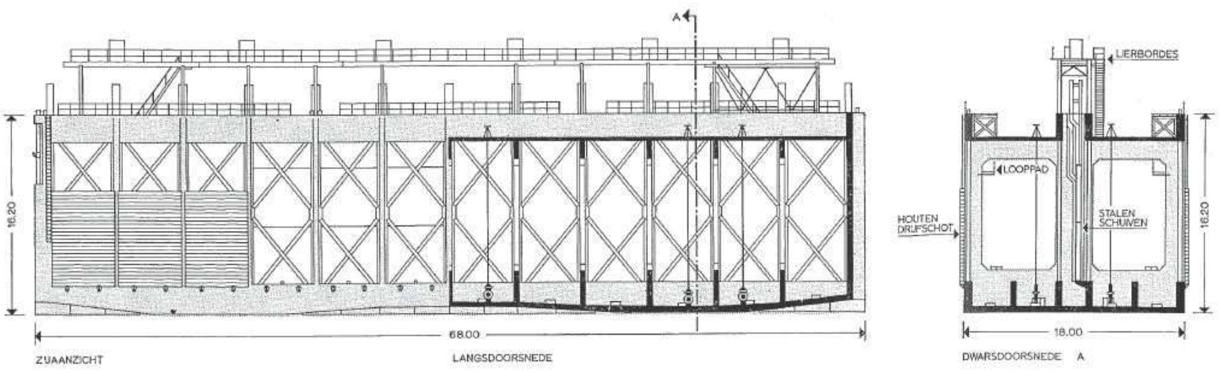

Figure 2-15 Cross-sections of sluice caissons Brouwershavensche Gat. The gates are positioned in the middle.

Caissons were also used in other parts of the Dutch Deltaworks. An overview is presented in Table 2-1.

Closure	**year of completion**	**type of caissons used**	**dimensions (LxHxW) [m]**	**numbers used**
Nolledijk	1945	Beetle	13.5 x 3 x 5	26
		Nl pontoon	? 1)	26
		intermediate pontoon	25 x ? x 18	1
		ship	? 1)	1
		Thames barge	? 1)	1
Westkapelle South	1945	intermediate pontoon	25 x ? x 18	3
		Beetle	12.5 x 3 x 5.5	3
		Whale boat	? 1)	1
Westkapelle North	1945	Phoenix Bx 2)	62 x 12 x 13.5	4
Veere	1945	Beetle	12.5 x 3 x 5.5	7
		invasion ship	40 x ? x ? 1)	2
Rammekens East	1945	Beetle	12.5 x 3 x 5.5	2
		invasion ship		1
Rammekens middle	1946	Phoenix Bx	62 x 12 x 13.5	2
		intermediate pontoon	25 x ? x 18 1)	1
		Beetle	12.5 x 3 x 5.5	4 (at least)
Brielsche Maas	1952	Phoenix caisson B2	62 x 10.5 x 13.5	1
Braakman	1952	Phoenix caisson Bx 3)	62 x 12 x 13.5	2
Kruiningen West	1953	unity caissons ?	11 x 6 x 7.5	6
Kruiningen Veerhaven	1953	unity caisson	11 x 6 x 7.5	ca. 15
		Phoenix Ax	62 x 18 x 19	1
		Phoenix Bx	62 x 12 x 13.5	1
Schelphoek	1953	Phoenix Ax	62 x 18 x 19	1
		unity caisson	11 x 6 x 7.5	several
Ouwerkerk	1953	Phoenix Ax	62 x 18 x 19	4
		unity caisson	11 x 6 x 7.5	17
Zandkreekdam	1960	unity caisson (closed)	11 x 6 x 7.5	14
Veersche-Gatdam	1961	sluice caisson	45.5 x 18 x 20	7
		Phoenix caisson Ax (for abutments)	62 x 18 x 19	2
Grevelingendam	1965	unity caissons	11 x 6 x 7.5	36
Volkerakdam	1969	sluice caisson	45 x 15 x 13	12
		abutment caisson	31 x 17.5 x 13→5	2
Lauwerszee	1969	sluice caisson	33 x 15 x 12	25
Brouwersdam	1972	sluice caisson	68 x 18 x 16	12
		abutment caisson	47 x 20 x 16.6→5	2
Eastern Scheldt	1986	sluice caisson	100 x 30 x 28	none 4)

note 1: an effort has been made to find the data, but has not been successful for the time being
note 2: used for a temporary breakwater
note 3: one Bx caisson had been altered into a sluice caisson
note 4: instead of a dam with caissons, a storm surge barrier with gates has been constructed

Table 2-1 Overview of caissons used in the Dutch Deltaworks and other closure works

The variation in dimensions through the years is shown in Figure 2-16.

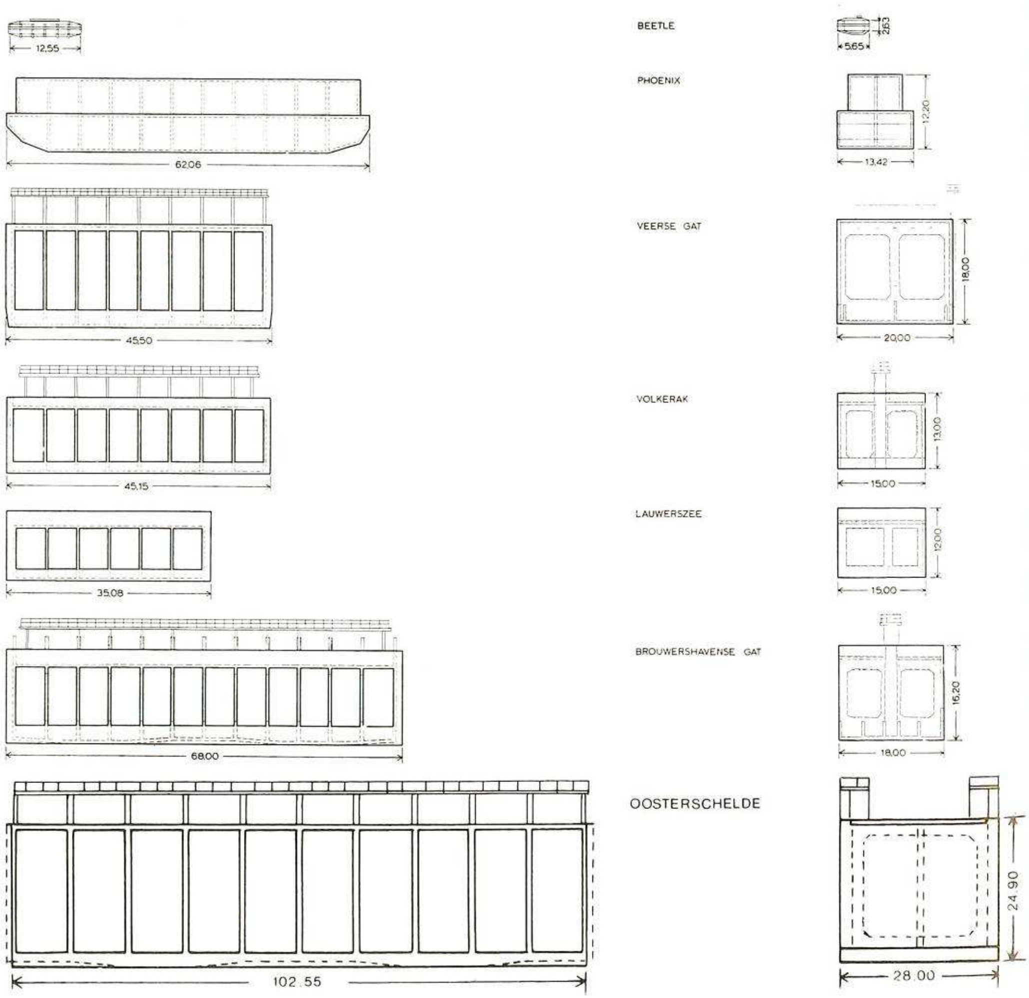

Figure 2-16 Various caisson dimensions in the Netherlands, 20th century

After a series of damage incidents (caissons toppled over) in the 1930s, for example in Catania (Italy) and Algiers (Mustapha breakwater), caisson breakwaters (and vertical breakwaters in general) were almost abandoned in favour of the rubble mound type except for some countries like Italy, Japan and Taiwan. Especially in Japan they have been applied in large numbers thereafter. The Japanese caissons were not floated to their destination but transported suspended on a large derrick barges (*drijvende bokken*) [*Oumeraci, 1994*].

In the 1980s, the worldwide need for breakwaters at greater depths required other solutions than the rubble mound type that had become favourable since the 1930s. Because of a number of improvements, the vertical breakwater type once again became an interesting possibility. The improvements, for instance, existed in the availability of more reliable wave data, more knowledge about wave breaking and impacts on structures, and the availability of large-scale testing facilities. [*Oumeraci, 1994*]

2.3 Nowadays use of caissons - special applications

Caissons nowadays are used for a wide variety of applications. Pneumatic caissons, for example, are still used for the construction of metro tunnels, like for the Amsterdam Noord-Zuidlijn underpass of the Damrak (between the underpass of Amsterdam Central Railway Station and the bored tunnel under the Damrak, see Figure 2-17). Three caissons for this part of the tunnel were constructed at street level and later subsided into the soil by washing away the mud below. The caissons went down 1.5 m per day until they reached a solid sand layer at 20 to 25 m depth. One of the caissons has been used as the starting shaft for the tunnel boring machine (TBM).

In the late 1980s, an artificial island has been constructed in the Beaufort Sea, North of Canada, to study the possibilities of oil extraction under severe ice conditions at sea. Four caissons (LxHxW = 70 x 11 x 15 m) were placed in a square and in this way formed the boundaries of the island. They were immersed on the sea bed and filled with sand, as well as the space between the caissons (Figure 2-18). More details about this project can be found in the lecture notes 'Hydraulic Engineering. Artificial Island in the Polar Sea. Dome Island in the Beaufort Sea' [*Vrijling, 2000*].

Figure 2-17 Caisson for the metro of Amsterdam

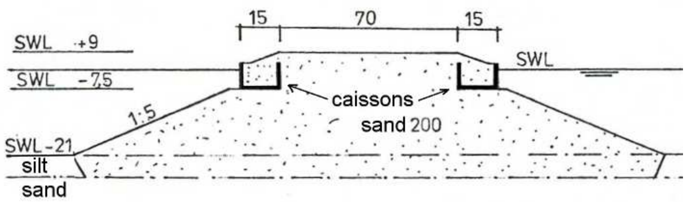

Figure 2-18 Cross-section Dome Island in Beaufort Sea

In Barrow, England, the concrete structure of a new navigation lock was designed as a pneumatic caisson. The shipyard of Barrow built ever bigger submarines, which required the construction of a bigger lock. This was carried out in 1989-1991 by Ballast Nedam. The new navigation lock exists of a U-shaped concrete caisson, 50 x 50 x 26 m, with a steel gate. The lock stability is guaranteed by a floor thickness of 9 metres. Monitoring during immersion, especially of groundwater, helped to prevent damage to existing structures. Shutters of 16 metres height retained soil and water during the immersion of the caisson [*Lievense, 2008*].

In Monaco, a floating breakwater has been constructed for the extension of the harbour. The main caisson has been built in Algeciras, Spain (near Gibraltar) from where it was towed to Monte Carlo, Monaco. This prestressed caisson has a length of 352 metres, is 28 metres wide, has a height of 19 metres and weighs 160 000 tonnes ($1.6{\cdot}10^6$ kN). It is anchored to a steady platform by an abutment caisson. An enormous steel ball-and-socket joint attaches the caisson to the land based abutment caisson (*landhoofdcaisson*). This steel articulation is specially designed to allow rotation and to resist loads up to 100 000 kN. The offshore end of the floating caisson is anchored by two sets of fixed anchors in water depths of over 55 metres. Besides its primary function, the breakwater provides berthing space for liners on the sea and harbour side. For about half its length, the caisson provides parking for 360 cars on precast floors on four levels (inside the box) and the other half contains two floors of boat stores. [*Hydro International, 2008*]

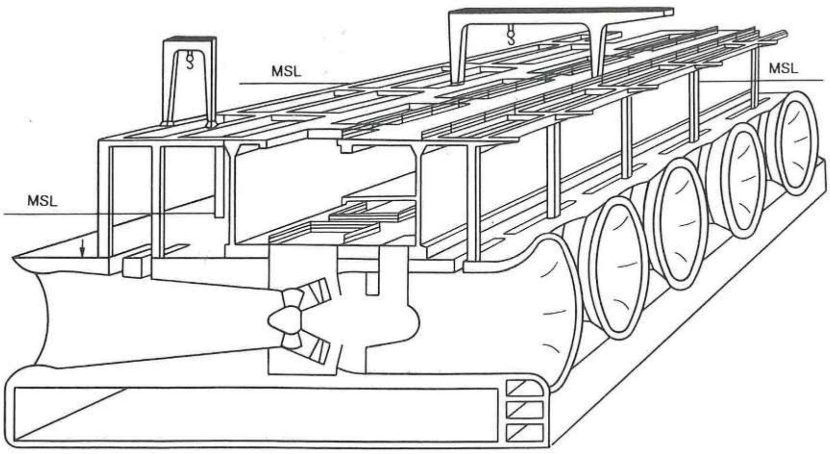

Figure 2-19 Tidal power plant in Australia

In Australia, caissons are used to accommodate turbines to obtain electric power from tidal currents in sea, see Figure 2-19. [*Bernshtein, 1996*]

For the port of Tangiers, Morocco, about forty-four cylindrical caissons were use to construct a breakwater (start of the construction in 2003, see Figure 2-20). Their final height is 35 metres, weighing 7 900 tonnes each. The shape of the caissons is rounded to reduce wave forces. To prevent ingress of chloride ions and to reduce cracking, a special concrete quality has been developed. The service lifetime of the breakwater is 100 years. [*Bouygues, 2005*]

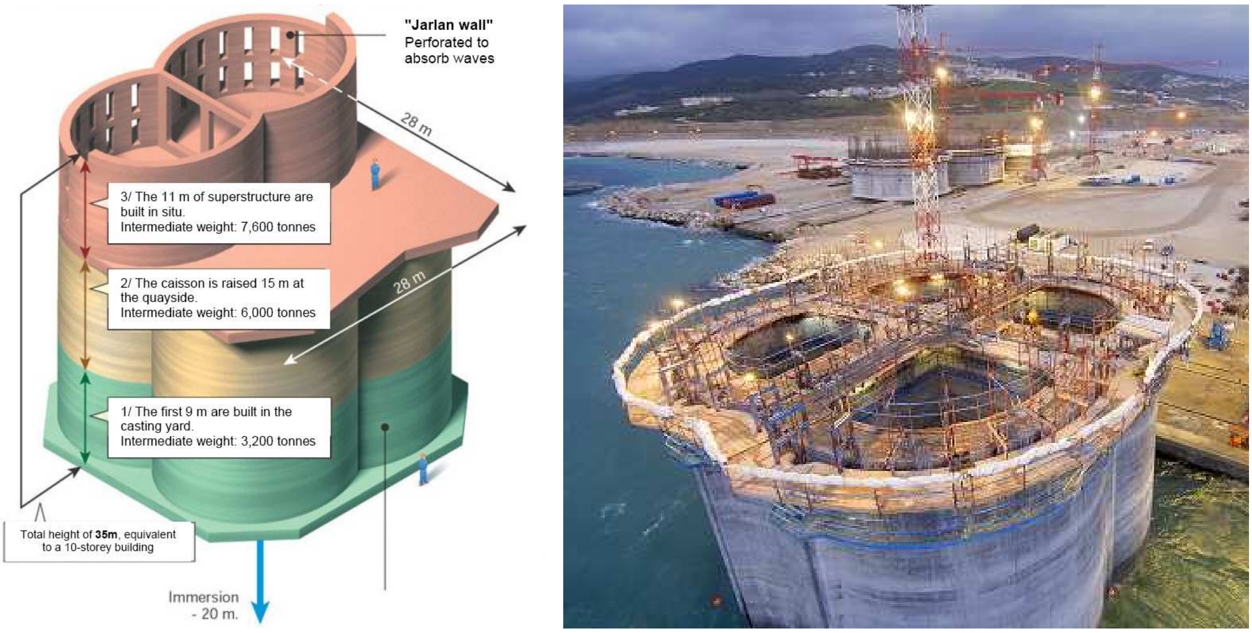

Figure 2-20 Caisson breakwater in Tangiers, Morocco

The high speed railroad bridge over the Hollandsch Diep waterway (Netherlands, constructed around 2005) is two kilometres long, of which 1200 metres is over water. The bridge is supported by eleven piers and two abutments. This bridge is a composite type bridge made of steel and concrete, necessary to obtain extreme rigidity, due to the vibrations caused by the passing high speed trains. The bridge pier foundation consists of driven steel tubes. These tubes are 19 to 33.5 metres long and have a diameter of 3 m. The wall thickness of the tube varies between 35 and 45 millimetres. After piling, concrete caissons of 10 x 25 x 2.65 metres were placed above the piles. The bottom of the caissons has holes in it, sealed with steel plates. The holes exactly match the position of the foundation tubes. The caissons were immersed in such a way that the holes ended up on top of the piles. After removing the steel plates, 12.5 m long steel meshes were placed in the remaining holes, followed by a replenishing with underwater concrete. Later, when the concrete had hardened, the caissons were immovably connected with the steel tubes. During construction, watertight partitions were put on the walls of the caissons. After immersion of the caissons, these partitions still protruded far above the water surface. In this way, construction pits were created. These were dewatered for further construction work. [*HSL Zuid project organisation, 2008*].

Figure 2-21 Caisson pier foundation with water tight partitions during construction of the HSL bridge over the Hollandsch Diep

In September 2008, an enormous concrete caisson for transhipment of liquid natural gas has been towed from its construction dock in Algeciras, Spain, to the Adriatic Sea near Venice. The structure has a length of 180 metres, width of 88 metres, height of 48 metres and a total weight of 450 000 tonnes (Figure 2-22). A yearly amount of about eight billion cubic metres of gas originating from Qatar has been shipped to this transhipment station and then via a 17 km long tube pumped to the Italian main land.

Figure 2-22 Caisson for LNG transhipment being towed to the Adriatic Sea near Venice

Undoubtedly caissons will continue to prove their value as (part of) solutions for many structural problems in the future.

3. Construction of standard caissons

Standard caissons are generally prefabricated 'in the dry' in a construction dock. When ready, the dock is inundated and the caisson can be transported over water to the actual site using its own buoyancy. There it is immersed to the river, sea or estuary bed and ballasted heavily enough to remain at its place and fulfil its function. The life cycle of caissons consists of the following stages:

1. Idea / initiative
2. Planning and design, laboratory tests
3. Prefabrication
4. Transport
5. In-situ construction
6. Operation, maintenance
7. Upgrading, removal & reuse or demolition

These stages are explained in the next sections, starting with prefabrication. General design aspects are treated after construction aspects, because it makes sense to know what aspects should be taken into account before bothering about how to estimate dimensions and how to check calculations.

The closure of a sea arm using caissons has been chosen as a project case to illustrate the caisson life cycle. As mentioned before, a number of sea arms were closed in the Netherlands to provide better protection during storm surge conditions. The 'Driemaandelijkse berichten van de Deltacommissie' (three-monthly reports of the Delta committee) provide a treasure of useful information for construction aspects of caissons used for tidal closures. Another objective of this chapter in the lecture notes is to pass on the experience of civil engineers who were actually involved in the Dutch Deltaworks (1953 – 1986).

Before deciding in favour of a caisson closure at all, it should be considered if a sudden closure should be preferred over a gradual closure. This topic actually goes beyond the scope of these lecture notes, but some background information will be provided in the following 'intermezzo'.

Intermezzo 1: over-all closure procedure

In case of a gradual closure of a sea arm or estuary in tidal areas, the difference in water level inside and outside will increase as the closure gap decreases. The bigger the water head, the higher the flow velocities through the narrowing gap. During construction of the sill (*drempel*), when it gets higher and higher, current velocities will increase until the condition of a free surface flow or overflow (*volkomen korte overlaat*) is reached. In this condition further reduction of the gap width or depth will no longer result in an increase of flow velocities. However, this maximum velocity should not exceed the critical velocity with respect to scour or damage to bed protection. If this is the case, caissons could be used in this last critical stage of the closure to stop the flow suddenly.

Closing the last flow channel, as well as the building up and finishing of the dam body, should take place within one work season. In case of big closure works this implicates that the closure itself should be done early in the work season. Preparatory works should be carried out in the preceding work season, which requires sufficient robustness to survive the storm season. Generally, the final closure of the last flow channel should be realised during one single period of slack water, usually when the tide turns. In case of wide gaps that cannot be closed in a single turn of the tide, or only a few quiet tidal turns, sluice caissons will have to be applied.

Usually closures start on both sides of a channel or sea arm. One caisson at a time is immersed during slack water (*kentering*) on alternating sides, for example in the case of the Veersche Gat closure (Figure 3-1 and Figure 3-2), or simultaneously on both sides if the channel is wide enough.

To reduce risks due to unforeseen circumstances it is highly advisable to draw up a work plan in advance. The aspects to be considered depend highly upon the specific circumstances, but the following list gives a first idea of what could be included in the plan:

- Closure moment if possible during low slack water
- Maximum flow velocity
- Maintaining position during the immersion process (e.g., use of an anchored pontoon)
- Procedure for the inlet of ballast water
- Planning of the ballasting with sand and the application of rubble (including delivery)
- The fill-up of the space between the joints of the caissons.

[*Deltadienst 1957-1987, deel 7*]

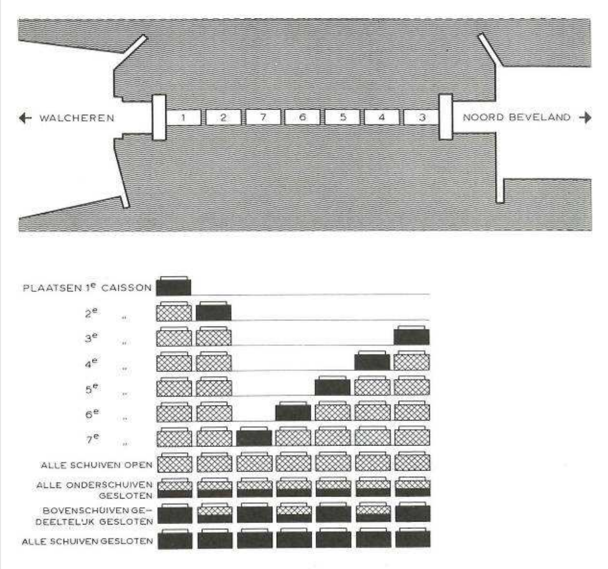

Figure 3-1 Closure scheme caissons Veersche Gat

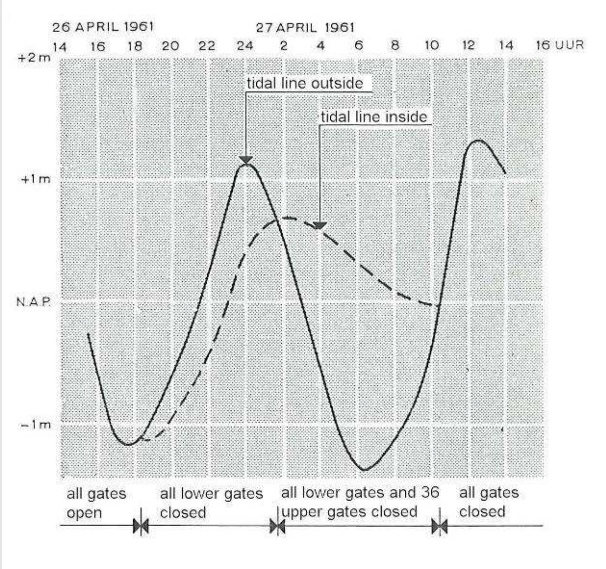

Figure 3-2 Closure scheme gates Veersche Gat

3.1 Prefabrication

It is advisable to prefabricate caissons at a location as close as possible to its final destination. If there is enough space and there are no restrictions for dewatering, a construction site in the form of a construction pit (*bouwput*) often is the cheapest solution. To make work with work, the excavated soil could be deposited in the surrounding slopes or dikes. It is sensible to carry out soil mechanical and hydrological tests in advance to find out whether or not differential settlement should be expected, determine counter measures, and to estimate the number of pumps needed for dewatering the construction pit.

Alternatively, e.g. when the available area is limited, a construction dock with sheet pile walls can be used to prefabricate the caisson(s). Depending on the use of the dock, it should have simple or more sophisticated facilities to float out the caissons and to close it off again[1]. It should be accessible over land without too much difficulty, e.g. by providing an access ramp. The earth that has been excavated to dig the dock should be deposited somewhere, preferably not too far away and should eventually be dumped back after completion of the job. There should also be space for a concrete batch plant, a reinforcement bar yard *(wapeningsvlechterij),* an area to assemble and store formwork elements (*bekistingsdelen*), and storage for other materials. Accommodation for workmen and commissioners should not be forgotten.

If several caissons have to be built one after another, storage space should be provided somewhere, preferably outside the dock to store the finished caissons. It could also be considered to construct the caissons in one line behind each other, which is advantageous for the use of travelling cranes and gantry cranes (*portaalkranen*). It could also make the use of lorries unnecessary.

Before inundating the construction dock, the caisson should be tested on water tightness, especially in the case of sluice caissons where timber shutters can cause leakage. For the Volkerak caissons, an inflow of 10 cubic metres per hour due to leakages was acceptable; in practice it appeared to be only half of that. Other measures that should be carried out before inundation are: clearing up the dock, digging free the element for a better water flow, if necessary applying slope protection at LWL / HWL and near the place of water inflow.

To prevent caissons starting to float in an uncontrolled way during dock inundation, the caissons should be ballasted with water in advance to keep them on the ground. Free ballast water requires bulkheads to prevent unacceptable water movement. Therefore, ballast tanks are sometimes preferred. The required water pipes and pumps should be tested before the actual filling takes place. River water usually is clean enough for this purpose, so no intolerable fungus or stench should occur because of this.

The construction dock can be inundated if the caisson is ready for transport. In practice there should be at least 0.50 to 0.70 m clearance below the caisson to float it out of the dock. If the dock bottom consists of too small sediment particles, there is a realistic probability that the caisson will not float up during dock inundation,

[1] The dock for construction of tunnel elements for the Spijkenisse extension of the Rotterdam metro tunnel was closed with a caisson!

as if it were glued to the dock bottom. This happened, for instance, during construction of sluice caissons for the Deltaworks. The solution for the problem was to replace the sand bed with a gravel or special drainage bed. Rijkswaterstaat recommends a gravel layer of 0.30 m thickness and a nominal grain diameter of 50 mm [*Rijkswaterstaat, 2005 (SATO)*]. Additionally, drainage tubes in the gravel layer can be used to let in water beneath the caisson. Alternatively, water could be injected under the caisson through small tubes that were cast in the caisson bottom plate, but normally the above described gravel layer should suffice.

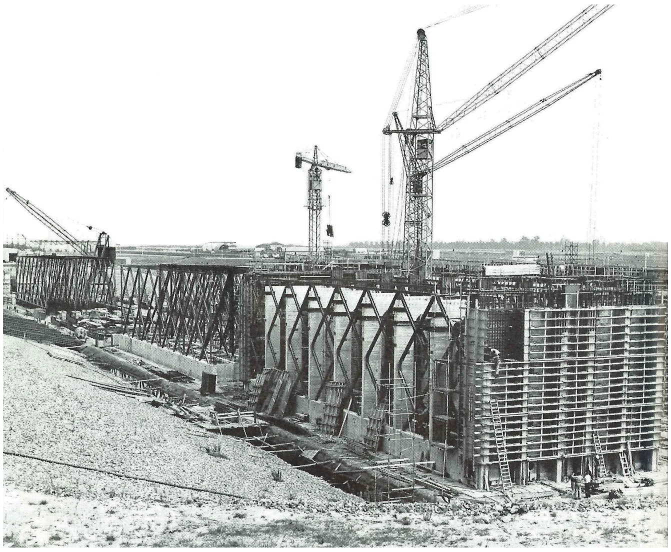

Figure 3-3 Sluice caissons for the Volkerak closure in construction dock

Figure 3-4 Filling of a ballast tank inside a caisson

The inundation of a construction dock can take up to 10 working days depending on its size. With regards to stability of the dock itself, the hydrological properties of the subsoil and surrounding dikes should be taken into account. The water level within the dikes should be raised along with the dock water level. The dock water level should be measured with help of piezo meters (*peilbuizen*). Dewatering should be phased out before inundation starts. Inundation can normally be stopped if the dock water level is about 0.5 m above the outer average water level.

Once the dock is inundated, the water ballast tanks can be emptied. Usually an elaborate plan (scheme) is drawn up dealing with all details concerned (such as the order of emptying tanks, the relation between draught and dock water level and directions concerning safety of workmen). Low tide is often preferred to start the floating up to avoid extra buoyancy. Anchored cables in combination with winches (*lieren*) avert uncontrolled horizontal drifting away.

3.2 Transport

The possibilities of transport from the construction dock to the actual construction site should be studied in advance and, if required, appropriate measures should be taken to make transport possible. In exceptional cases transport over land can be considered. However, these lecture notes concentrate on transport over water, which is usually accomplished with help of several tug boats and pusher tugs (*sleep- en duwboten*). The transport route should be checked for obstacles in both the horizontal and vertical direction. The presence of bridges or navigation locks, for example, could severely restrict possibilities of transport. Aspects that should also be considered are: width of the navigation channel, crossings with other waterways (side currents!), flow velocities, winding (*bochtigheid*), tidal movements and shipping. A transport permit should be obtained in advance from the waterway administration. Also weather conditions should be considered when planning the transport. High wind velocities could complicate navigability, but the resulting wave heights probably have more severe impact. Fog could hamper orientation and lead to unwanted run aground of the caisson. If the schedule is too tight, weather conditions can result in major problems. To prevent these, floating radar stations can be provided. Nowadays the global positioning system (GPS), which works with satellites, makes it possible to determine location, speed, direction, and time much more easily.

Caissons should be provided with bollards to connect towing ropes and hawser holes (*kluisgaten*) to let anchor cables through. To enable handling by workmen, temporary gangways with railings and ladders should be provided.
Bat phones (*portofoons*), CB-band (*mobilofoons*) and mobile phones can be used for communication between

the caisson, boats, measuring station and the shore. If the distance is short enough, shouting and signalling will prove highly effective.

The channel leading out of the dock should have sufficient depth to allow the floating caisson to pass, which also applies to the whole waterway to the destination. In general there should be at least 1.00 m keel clearance below the caisson. If this keel clearance is not available, some extra dredging work should be carried out, extra to the dredging work required for the exit out of the dock. Depending on the length of the river or estuary bed to be travelled through, and its shallowness or depth, it could be more economic to change the design of the caisson to reduce its draught than to do all the dredging work.

During transport (and later on during immersion), the floating caisson should be sufficiently stable; it should be guaranteed that it does not tilt to an unacceptable degree. Tilting of unstable caissons can be caused by mooring forces, wave motions, inlet of water during immersion, forces exercised by tugboats, etc. Free water inside caissons is normally spoken avoided during transport. The caisson should also be navigable enough during transport, to facilitate and control its movements. This requirement has consequences for both the proportions of the caisson, as well as the power and manoeuvrability of the tug boats (Figure 3-5). If stability problems cannot be acceptably solved by changing the dimensions, extra ballast could be connected to the element (below the point of gravity) during transport. Other possibilities for stabilisation during transport are the use of stabilising pontoons or vessels (stabilisers), or linking two elements to the caisson. See Appendix 6 for more details on stability.

Close to the final construction site, where the elements will be immersed, a place for temporary berthing should be provided to enable disassembly of transport facilities and prepare the caisson for immersion. Sometimes tug boats suffice to keep the caisson in position until the right moment has arrived for immersion.

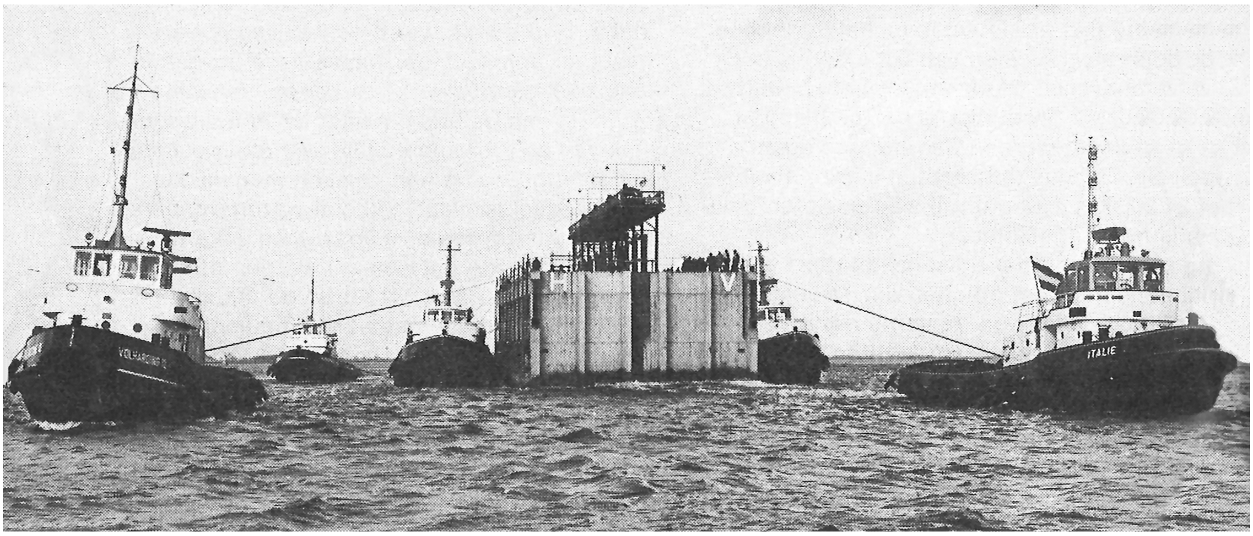

Figure 3-5 Transport of a sluice caisson with tug boats towards the Volkerak

3.3 Positioning and Immersion

The following aspects of the positioning and immersion of standard caissons will be treated in this section: preparation of the foundation, positioning, immersion, ballasting and building the dam body, finishing, back filling and reclamation.

3.3.1 Bed preparation

The preparation of the subsoil is not the main topic of these lecture notes, so some information about preparatory works has been put in a second intermezzo.

Intermezzo 2: preparation of the foundation

In this intermezzo a distinction is made between placement of caissons directly on the bed of a river or estuary and placement on a sill. In some special cases, pile foundations can be applied, but this is far more complicated and will be avoided if possible.

Placement of the caisson directly on the bed

Whenever a large prefabricated section is used as spread foundation, 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 break-waters 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.

Because of the required filter thickness, or other reasons, it could be necessary to do some dredging works. First a (cutter) suction dredge can start with rough dredging work, later this has to be levelled out more evenly. This is necessary to prevent that the caisson will later rest unequally on the sill, which would introduce high stresses in the caisson concrete, especially in the middle or extremities of the side walls. Even if the sill has been built up carefully and has been levelled afterwards, unevenness can appear nonetheless. In case of the Veersche Gat closure sill height differences occurred of up to 0.50 m (but not more than that) at short distances from each other. However, during positioning of the caisson these irregularities were partly levelled out by the caisson. Nevertheless it has occurred that only 25% of the bottom plate was in contact with the subsoil! The width of an immersion trench for tunnel elements is about the width of the element plus four metres on both sides. The depth of this trench should be about the draught of the element plus 0.60 m and the slopes should be relatively gentle, about 1:5, dependent on the soil type, tidal flow etc..

Dredged material, coming available during the preparation of the bed for the sill, particularly if it is sand, could be used to build up the dam core in a later stage.

Special treatment will be required if the dredged material is polluted. It is advised to make agreements in advance with licensing authorities (*vergunning-verlenende instanties*) about what to do with possibly polluted dredged material and consider what measures should be taken with regards to health and safety on the work.

Bed protection has to be applied almost immediately after the dredging has completed. This is to prevent erosion or sedimentation modifying the required depth. This implicates that construction of the scour protection (immersion of mattresses or geotextile variants and stone dumping) has to follow as soon as possible after the dredging work.

It is very difficult to obtain a smooth finish to the bottom, especially when large areas are involved. In some cases an attempt is made to do this by applying a layer of suitable material and smoothing it off. For 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 (fall pipe). It is then smoothed off by a levelling beam that is pulled over the bed, using 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 horizontal as possible and at the required level. The fall or discharge pipe is also assembled to the frame. Since the frame has relatively limited dimensions, compared to the whole bed to be prepared, it must be frequently moved by the floating cranes to provide following parts of the site with a level sand or gravel bed.

The surface will never become entirely smooth. For the transfer of forces to the subsoil it must be assumed that this is not optimal. In some places there will be no sand or gravel under 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 in some levelling). The bottom plate of the caisson must be dimensioned to resist these higher local 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 may have to be thicker over the entire surface (see §4.2.4).

Placement of the caisson on a sill
Large differences in water level on both sides of the dam could lead to piping under the caissons. To prevent scour, the sill should be built up like a filter. Damage can be avoided if the filter is designed in such a way that it prevents basic material to be moved away through it. This is called a geometrical tight filter. Another method to prevent this kind of damage is to build up the sill in such a way that the flow velocity in the superjacent filter layer is so low, that the basic material will not come into motion. Furthermore a filter can prevent damage if erosion of the basic layer will be small enough during the operational phase of the structure. The basic material in these last two types can pass through the filter, dependent on the hydraulic loads, but in such a controlled way that it does not lead to damage.

The dam sill of the Veersche Gat, for example, consisted of several layers on the existing fine sand bed. The sill has been constructed on a nylon mat that was unrolled on the sand bed. The first layer on the mat consisted of fine gravel (5-20 mm, layer thickness 0.40 m), followed by layers of coarse gravel (layer thickness 0.9 m), light ballast stone (0.6 m) and finally heavy rubble (0.6 m). The Brouwershavensche Gat sill top layer consisted of 10-300 kg rubble. On both sides of the Veersche Gat dam, protection of the channel bed was provided with fascine mattresses (*zinkstukken*). An example of a sill construction is presented in Figure 3-6. If the top of the sill is equally high over its entire length (i.e. horizontal), all caissons can have the same dimensions. This is advantageous for construction (only one size of formwork is needed so it can be re-used several times) and costs, although some extra dredging or heightening of the channel bed could be required to realise a horizontal sill.

A change of flow patterns could also lead to unwanted sedimentation, for instance on the sill, but also further away from the closure dam which should be prevented or remedied. It could be very wise to perform soundings and soil drillings around the location line. In this way knowledge can be gained about the resistance and composition of the soil and for instance it could be estimated if erosion could lead to liquefaction (*zettingsvloeiing*) that could cause instability of the sill or abutment. To avoid strong, erosive currents over weak soil, the closure scheme can be adjusted with help of the gained information about the soil.

Foundation on piles
If it turns out that a shallow foundation will bring severe problems, e.g. to the stability or displace-ment of the caisson, deep foundation can be considered. In that case, piles have to be driven in advance and later on the caisson will have to be lowered over these piles onto the channel bed. The pile heads should fit into the recesses of the caisson, where a structural joint can be constructed. A disadvantage of this method is the extension of the piles above the sill, which reduces the acceptable clearance of the caisson considerably. Another disadvantage is the connection between piles and caisson, which has to be made under water. A second method for deep foundations is to drive the piles through hollow intermediate walls in already immersed caissons. Visual inspection of the connection is possible now, but the extra walls are a disadvantage, because of the extra weight and the reduction of the wet cross section.

[*CT3330 - General lecture notes, Deltadienst 1957-1987*]

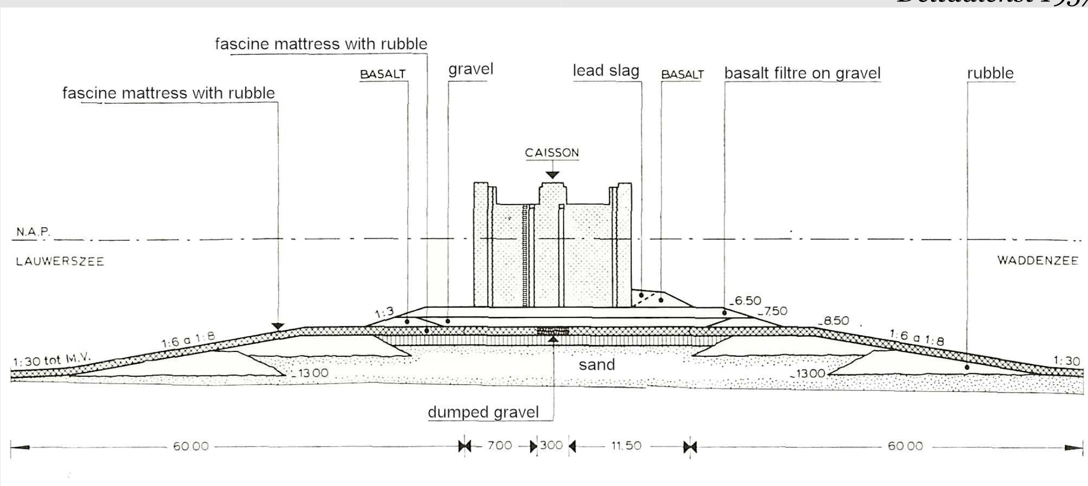

Figure 3-6 Cross-section of a sill with caisson, Lauwerszee

3.3.2 Positioning

Usually caissons are transported from the construction dock to their destination during high water level. They will be 'parked' (remaining connected to the tug boats) outside the closure channel until the flow velocity drops. If the current velocity is low, also the forces on the caisson will be low which is favourable for the positioning and immersion, because the horizontal flow or current forces on the caisson are proportional to the square of the velocity. Figure 3-7 shows how low current velocities occur during the turn of the tide, known as slack water (*laag- en hoogwater-kentering*). In general the low tidal turn is a better moment to immerse caissons because of the shorter immersion height (the shorter the immersion height, the less risk of failure), unless the course of the flow conditions appears to be much better during high tidal turn or if the clearance during low tidal turn might be too small.

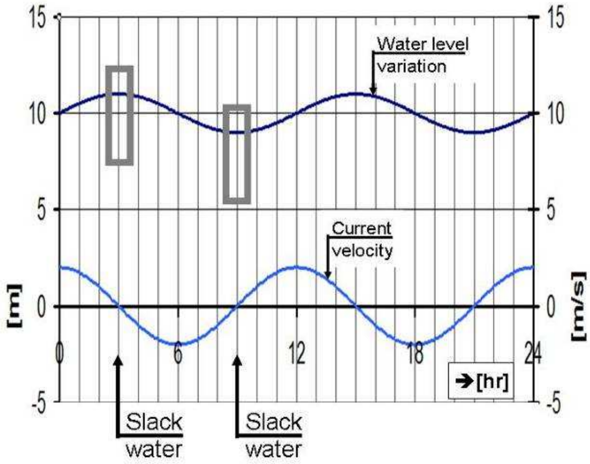

Figure 3-7 Immersion at slack water

The final positioning of a caisson could be achieved with help of only tug boats or tug boats in combination with cables from floating equipment, anchors or dead-man beds. Sometimes pontoons or temporary quays with a fixed position are used. The first caisson put in position and immersed is an excellent 'anchor' point for the following caissons to be immersed. Hinge connected (*scharnierend verbonden*) to the preceeding caisson the one to be immersed can easily be manoeuvered into the right position by tug boats. See Figure 3-8 for an example.

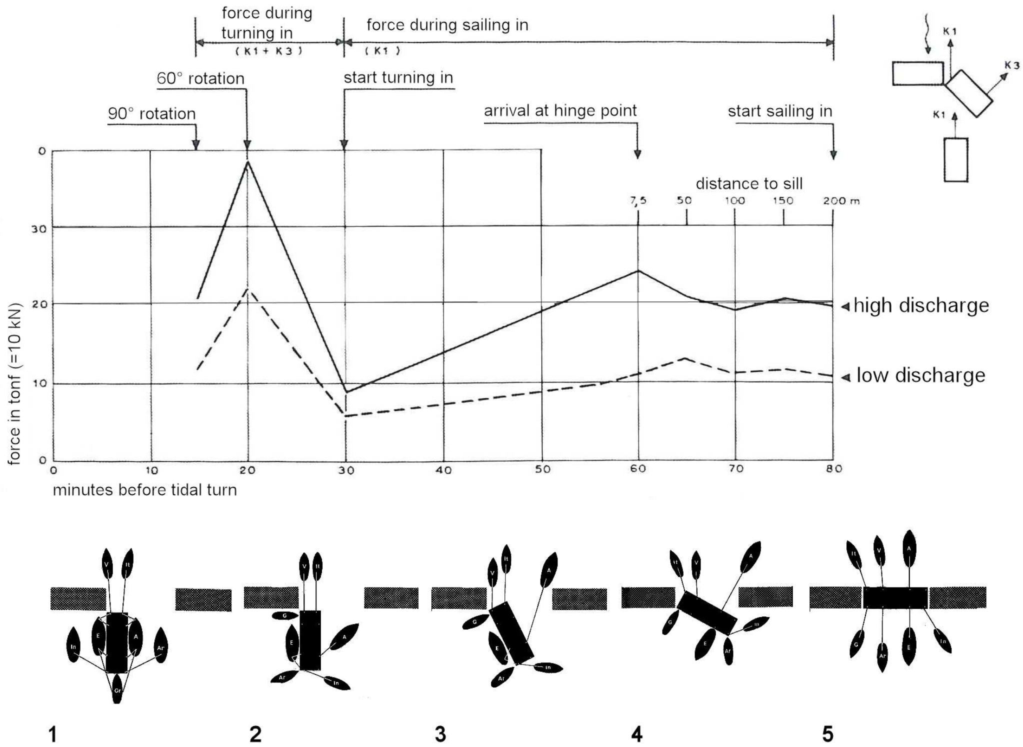

Figure 3-8 Forces on a caisson during positioning and manoeuvring stages (Volkerak closure)

Like during transport, global positioning system (GPS), which works with satellites, is of great help to determine location, speed, direction, and time.

Many of the requirements set for in situ foundations are also applicable for the foundation of prefab sections. The requirements are:

- a good transfer of forces to the subsoil, thus sufficient contact area between the bed and bottom of structure.
- the slope of the bed where the prefab section has to be positioned should suffice to what is required, see Figure 3-9.
- settlements may not lead to unacceptable deformation of the finished structure unless adjustment options are available.

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 3-9-a, with an exaggerated scale, shows the consequences of a misshaped underwater slope. In this case the slope is too steep.

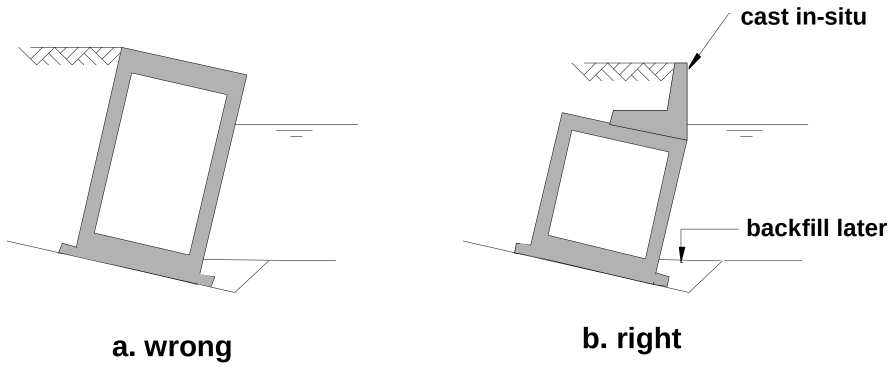

Figure 3-9 construction error in the positioning of a quay wall-caisson (exaggerated)

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 in order to even out the apron and provide a vertical berth face for safe mooring of ships. Figure 3-9-b shows an option that is often used to correct the wrong slope or angle of a caisson. After positioning less high caissons a reinforced concrete L-wall is cast above the waterline; naturally this must be well anchored to the caissons. The front face of the L-wall is vertical, so that ships can moor alongside. The inaccuracy in the slope of the bed can be counteracted in this way.

Even in case of a bottom bed of the right slope, large caissons cannot always be accurately positioned in the horizontal plane, e.g. due to unfavourable weather conditions during the sinking procedure. Their front faces could end up not being in the same plane unless preventative measures were taken such as the use of "shear keys". The resulting protruding angles could result in damage to berthing ships. Here as well, the quay face can be made smooth by constructing an L-wall on top of the caissons after positioning.

> **Note 1.** Caissons placed directly on the bottom have the disadvantage that erosion, for example caused by ship 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.

> **Note 2.** Figure 3-9-b 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 has another advantage; it mobilises the vertical weight of the fill at the back, which helps to counteract the active soil pressure, in order to satisfy the stability criteria (ΣH, ΣM, ΣV).

The dam heads adjacent to the caissons should preferably have vertical walls, which facilitates the connection of the caisson with the dam head. If the closure gap is situated very close to the dikes, special abutment caissons (*landhoofdcaissons*) can be convenient with respect to the connection of the dam with the dike. Abutments can be helpful to protect preparatory works during storm season preceding the next work season when the closure will be completed. Special attention should be paid to the shape of the abutment caisson with respect to the flow lines of the water. It could be favourable if part of the site is free of strong turbulence and flow.

3.3.3 Immersion

A caisson or tunnel element can be immersed if, after transport, it is moored on a mooring pontoon, derrick barge (*bok*), temporary quay, or if it is held in position by tug boats or anchors. Before immersion begins, transport facilities, like frames, bollards, navigation lights and generators, should be removed. Per project and per element it should be considered what kind of immersion facilities should be provided in and on the element, at the abutments or shores, and in the river or estuary. For instance, measurement towers, entry shafts and hoisting brackets (*hijspunten*) have to be installed. The dredged trench (in case of tunnel elements) should be checked not too long in advance on erosion, siltation with sand or mud and the density of the water.

The caisson should contain valves to let in water so that it can be immersed in a controlled way. The valves should be positioned in such a way that the caisson will remain balanced during the inlet of water. This can especially be a problem if the opening of the valves does not occur simultaneously. Hindrance of the water flow inside the caisson is another thing to be dealt with, as well as hindrance of water inlet from the outside to the inside. For example, bulkheads or girders could obstruct the flow inside the caisson, and openings on the outside could be blocked by other caissons (if the openings are positioned in the head walls) or the river bed (if positioned in the bottom plate). Therefore, in practice, valves are positioned on several sides of the caisson depending on the valve operation scheme. Valves in the bottom plate have the additional advantage that instability of the caisson during the last part of the immersion, when water will be forced away, will be reduced.

For proper placement of the caisson, the flow velocity of the water through the gap should not exceed 0.30 m/s. Immersion during low water slack is preferrable, because then the flow velocities and the time required for immersion will be minimum. The element should be gradually immersed with vertical (and sometimes also horizontal) steps and preferably be manoeuvred against a fixed positioning point (*aanslagpunt*). First the ballast tanks are filled with water until the element stops floating and is suspended on the immersion facilities. Then the winches on the immersion pontoons or barges are step-by-step eased off.

The required time for immersion of the Zandkreek unity caissons, for example, was about 6 minutes. Including mooring along the crane pontoon and positioning, it took well over 20 minutes. The dump of 120 tonnes of rubble per caisson lasted about 1½ hour [*Deltadienst 1957-1987*]. The filling of ballast tanks in tunnel elements can last up to about 2 or 3 hours [*Rijkswaterstaat 2005 (SATO)*].

If a considerable number of caissons have to be immersed in a relatively short period, the caissons could be linked to each other, up to five pieces. These linked caissons should be sufficiently rigid so that tug boats can handle them, but on the other hand there should be some flexibility to avoid unacceptable forces in the links between the caissons. The fact is, linked caissons tend to swing under influence of beating waves, current and wind. It normally is not necessary to already link the caissons near the construction dock, so the transport to the site will be single.

The mutual connection of the immersed caissons is another point of concern. The space between two caissons can be made watertight with help of staggered ribs on the head walls or extensions of the side walls (unity caissons) or sand bags. Even torpedo nets could be used for this purpose, like they did in Zeeland, and other inventive solutions will be found when one is in big distress.

Special attention should be paid to water overpressure in the soil underneath the just immersed caisson. The weight of the caisson has to be taken over by the soil within a few minutes. The soil has to be highly compressed in a short period to take up the sudden increase of stress. This compression can only take place if water can be squeezed out of the soil. If water cannot flow away fast enough, there will be water overpressure underneath the caisson which can endanger its stability. This can also occur due to wave impact, especially if the soil exists of loosely packed sand. Wide caissons are unfavourable for this phenomenon because of the relatively long time needed for flow-off.

Because the procedure of the immersion of sluice caissons is a special case, compared to standard caissons, this type of caissons is treated in another intermezzo.

Intermezzo 3: sluice caissons

For a description of the principle of sluice caissons, reference is made to section 2.2

The handling of sluice caissons during navigation and immersion does not deviate too much from standard caissons. The procedure after putting the sluice caissons on the sill or bed, however, is different.

Shutters have to be removed and gates must be raised first after the immersion of sluice caissons to let the water flow through until the moment of closure. When all sluice caissons are positioned on the sill, all caisson gates should be lowered at the same time during low slack water to effectuate the closure (low slack water lasts a bit longer than high slack water). If water level differences on both sides of the caisson are expected to cause severe initial piping below the caissons, the tidal movements in the semi-blocked estuary can be dampened by closing one or more sluice caissons before actually closing all gates at once (Figure 3-2). However, this could lead to unacceptable flow velocities in the direct neighbourhood of the caissons.

The gates of sluice caissons in principle remain open until all caissons have been immersed and the connections have been made watertight. Although sluice caissons let through water, the discharge will be substantially diminished, up to half of the original. Because of this, horizontal water pressure will be exerted onto the open sluice caissons, so there is a possibility that the caissons will slide aside or topple over. In this case adaptations to the design are required (e.g., ballast boxes on top of the caisson or increase of the caisson width) or rubble can be applied along the caisson which can also help prevent piping under the caisson.

Even though immersed sluice caissons have openings to let water flow through, they reduce the total flow capacity considerably. The discharge coefficient of caissons used in the Delta Works varied from 0.60 to 0.85, dependent on the water depth. The discharge decreases as a result of the narrowed flow opening, but the resulting flow velocities in the flow channel will increase. This is why the proposed procedure for the closure of the Veersche Gat had to be changed. First it was thought that the latest positioned caisson should remain closed, so that the new to be placed caisson could be immersed under the lee (*luwte*) of the preceding one. However, the closed gates of the sluice caisson caused turbulence, resulting in damage to the bed. Therefore it has been decided that the sluice caissons should be opened after immersion and only closed during slack water for the purpose of rubble dumping. Even in this case the flow velocity will increase as the open channel opening will decrease resulting in higher attack of the sill and water bed.

The final ballasting of sluice caissons normally occurs from both sides, when the dam body is being suppleted.

3.3.4 Ballasting

After immersion of a caisson, the ballast water has to be replaced by sand or rock to improve stability. After the final closure caissons will be ballasted, usually with sand, but some extra ballast might be needed in the flow-through stage (in the case of sluice caissons) before closure. In that case extra rubble ballast boxes have proven to be very useful. Ballast boxes have to be replenished with ballast sand as soon as possible after immersion. Quick dewatering of the sand-water mixture can be advanced with help of a drainage system of synthetic tubes in the side walls of the ballast boxes.

The main function of the dump of rubble aside of the caisson is the prevention of sliding away. An additional advantage of rubble is the reduction of seepage under the caisson. Dumping of rubble is time-consuming and rather complicated, so it appeared worthwhile to think of other means to avert sliding. One solution is to increase the ballast weight, but in case of sluice caissons this requires extra ballast boxes on top of the caissons. Sometimes ballast boxes are positioned on the bottom plate of the caissons, which could be favourable for stability, but here they should not hamper the flow too much. Another way to avoid expensive rubble dump is to increase the dead weight of the entire caisson, but this has direct implications to the draught in the floating stage, so with this another problem arises which also must be solved (probably leading to an increase in costs).

After the caisson has been ballasted, some settlement of the sill could occur. In case of the Veersche Gat this mounted up to 0.15 m.

3.4 Finishing the structure as a whole

Caissons generally are only a part of a larger structure. After immersion of the caisson, this larger structure still has to be completed. Here below this process will be described for a closure dam.

Before building the actual dam body, it could be considered to fill up gaps, if these are observed, in the sill below the (tunnel) element. These gaps leave the element partially unsupported and result in unwanted extra tensile stresses in the concrete. A sand-water mixture can be injected under the element with help of pipes. These pipes can be part of the caisson structure, or part of the sill. The pump capacity, mixture concentration, discharge, positioning of injection points, sand supply (e.g., by ship), etc. are to be studied and planned in advance.

Immediately after the closure large amounts of sand have to be supplied on both sides of the caissons. This can continue for several weeks and is intended to prevent piping and sliding aside. After the backfilling the caissons will permanently be part of the complete structure, like the dam body in Figure 3-10.

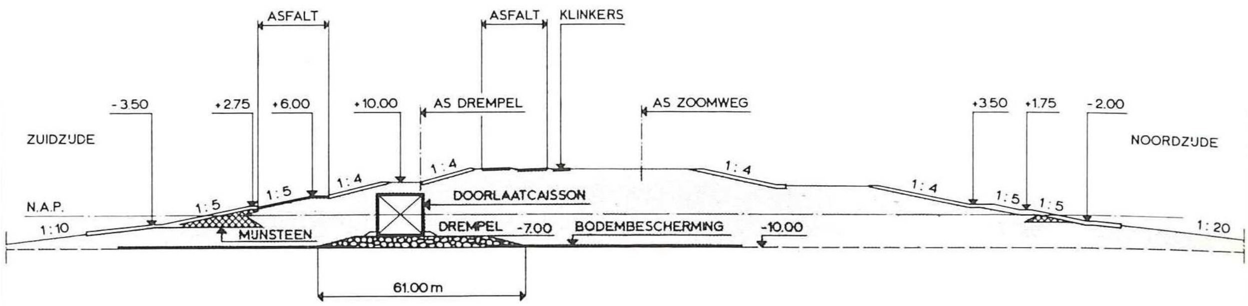

Figure 3-10 Cross-section of the Volkerak dam

3.5 Maintenance and control

Maintenance and control are carried out to make sure that the construction as a whole fulfills its function during its lifetime. Serious calamities and unnecessary reduction in the value or life time have to be prevented. Regular inspection and maintenance is therefore required. To realise this, a maintenance handbook has to be drawn up. The inspection and preventive maintenance activities should be based on a cyclic pattern. Unforeseen calamities (which should be avoided) require incidental repair.

The maintenance and control phase starts with a zero-measurement to register the initial qualities of the structural parts. Measures should be defined if deviations from these qualities exceed defined tolerances. The frequency of inspections and maintenance should be indicated, as well as the replacement frequency of specific parts. A registration and archive system should assist in this phase of maintenance and control.

Maintenance and control of a caisson that is part of a dam body is almost impossible. Therefore, collapse or other severe damage of an embedded caisson should be prevented if this would have unallowable consequences for the structure as a whole (i.e., if this will threaten the fulfilment of its primary function).

Breakwater caissons, however, can be partly inspected with more ease. The usefulness of inspection is proven by the fact that many failures of vertical breakwaters were preceded by prior warning due to previously experienced less violence storms, or failure case histories of similar structures [*Oumeraci 1994*].

3.6 Final stage

With respect to upgrading, re-use or demolition, it could be very convenient if the caisson can float up again. During the design it should be reckoned with that measures that allow the caisson to immerse can be reversed in a later stage.

4. Design of standard caissons

4.1 Design method

If caissons are used, a large scale prefabrication construction method is used for at least for a considerable part of the structure. Generally, prefabrication of caissons has the disadvantage of finding and building a caisson construction site and the subsequent transport problems. The advantage, however, is that the huge in-situ construction problems can be avoided.

Design has progressed through a number of stages, or has dealt with a lot of issues already, when the decision to use caissons is taken. Figure 4-1 shows the issues that have to be dealt with during design; there will be a stage when more general design and analysis, resulting in the Basis of Design (see issues on the left hand side of the Figure), shifts into the more hardcore engineering work (see issues on the right hand side).

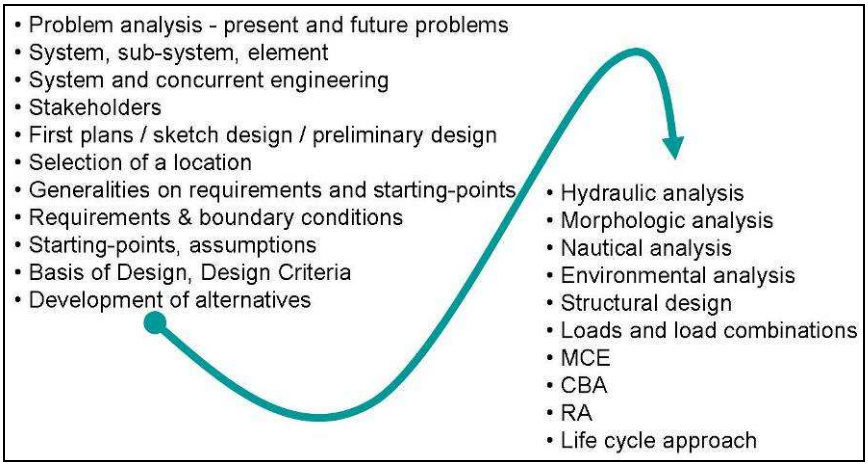

Figure 4-1 Issues to be dealt with during design

To arrive at a Basis of Design for the caisson another design loop, further analysis dedicated to caisson fabrication and transport, will be necessary. Look for instance at the previous chapter on construction aspects of caissons and see how functional requirements can be derived (like the presence and place of bollards, gates, valves etc.). In Appendix 1 some more background regarding the design process of hydraulic structures in general has been included. Structural design of the caisson will be the subject of this chapter.

To keep in control of structural design and more important, not to overlook a load that should have been included, failure mechanisms and load situations, see Table 4-1, will be considered in the first structural design loop or example calculation for a standard caisson in sections 4.3 through 4.4.

Failure Mechanisms	Load situations

• static stability
   – during (floating) transport
   – during immersion
• dynamic stability
• shear criterion caisson-subsoil
• turn-over criterion
• vertical bearing capacity
• scour
• strength of the concrete structure
	• building pit phase
	• floating phase:
	   – during transport
	   – during immersion
	• founded phase:
	   – immediately after immersion
	   – final phase (use phase)
	• removal phase

Table 4-1 Failure mechanisms and load situations to be considered during structural design of a standard caisson

However, to start structural design, the dimensions of the caissons should be known, at least a first estimate. Determination of the caissons dimensions wil be the subject of next section.

4.2 Determination of the main dimensions

Experience taught that a 'standard recipe' for the design of caissons cannot be given. Specific project requirements and local circumstances generally differ too much to make the design that simple. However, an overall approach that in most cases leads to good results is a two-step approach: first determine the main dimensions and then, step 2, check the dimensions based on a number of basic engineering calculations. It cannot be avoided that some steps have to be repeated if requirements are not met: In that case the initial dimensons have to be reconsidered and previously done checks have to be repeated. Figure 4-2 illustrates how the design process gets into iteration when determining the dimensions of the caisson.

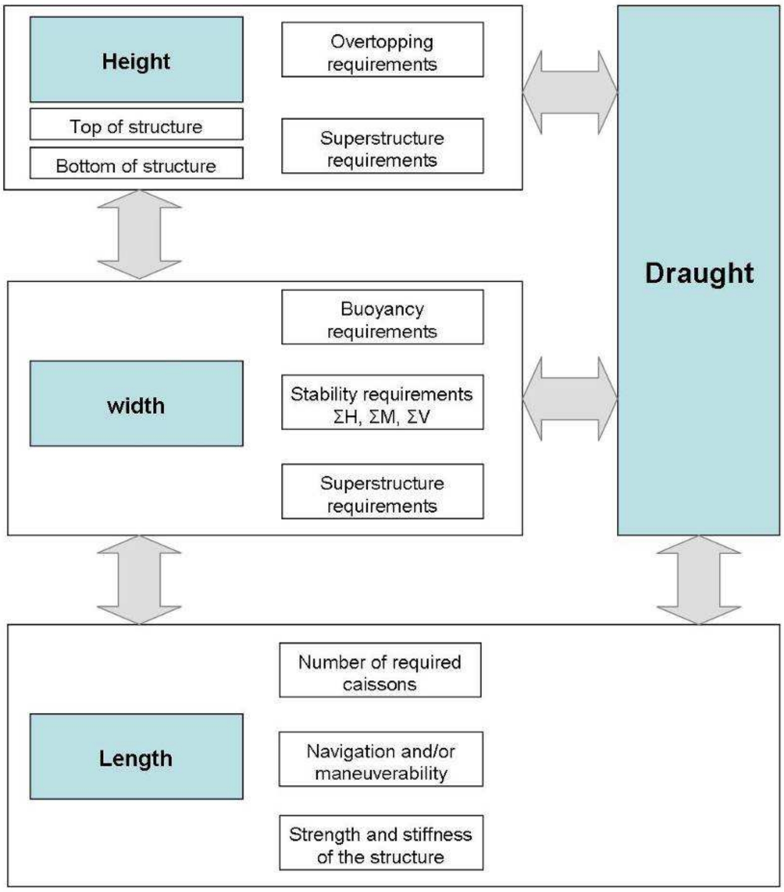

Figure 4-2 Iteration to find the caissons main dimensions

Looking at the height, width and length of caissons, generally the height is the dimension which is most easily derived from one of the main functional requirements; e.g. the retaining height of the quay, the complete structure demanded by the commissioner *(opdrachtgever)*. In few situations there will be a main functional requirement regarding caisson width, often the width is the result of considerations with regard to the foundation, and the length will always be the result of practical engineering considerations (maybe except bridge piers). Therefore, in the following Sections, the main dimensions will be dealt with in the sequence height, width, length to arrive at the main dimensions of the caisson. Some attention will be paid to the caisson’s internal walls and/or bulkheads and the caisson cover or caisson roof.

Note: Especially the design of sluice caissons can be rather complicated. On one hand, the structure should be lean and slender to reduce flow resistance through the open gates and to facilitate manoeuvrability during transport. The depth of the construction dock, the weight in the floating phase and stability are better if the draught is not too much. On the other hand, the structure should be sufficiently strong and stiff to resist the horizontal forces, without sliding away or toppling over, as well as without inadmissible deformations.

For sluice caissons the height also depends on the following three factors:

- the required discharge in relation to the sill depth
- the required dimensions of the bottom case or bottom style with respect to the overall stiffness of the caisson and the possibility to dump rubble
- the minimum required height of the underside of the ballast box on top of the caisson regards wave height.

4.2.1 Height

Often determining the required caisson height is a good start for design. The final situation, not a temporary construction stage, is in most cases governing for the height of the complete structure and for the caisson as one of its elements. For breakwaters and quay walls, top of caisson will be almost equal to the top level of the final or complete structure. However, in case of closure dams, soil layers frequently are put besides and on top of the caisson to finish the dam, hence the caisson height is determined by an intermediate stage. Usually the height of caissons for bridge piers is determined by construction stages, except for bridges with an archtectonic mark.

The height determination as described below is based on breakwaters and quay walls. For other hydraulic structures and/or dependent on specific project circumstances engineering judgement will be required to include or exclude items relevant for determining the caisson height.

The height by simple definition is the difference between Top of Structure (ToS) and Bottom of Structure (BoS); so determining the height comes down to finding these two levels. To start with the latter: BoS for caissons used in breakwaters and quay walls, usually depends on the original bed level or on the level of the sill or soil improving (gravel) bed constructed on the original bottom.

ToS for breakwaters and quay walls (i.e., uncovered caissons) depends on:

- astronomical tide
- wind set-up
- height of wind waves
- refraction, shoaling, breaking, reflection and diffraction of wind waves
- overtopping (and eventually wave run-up)

and occasionally:

- extra freeboard
- seiches
- shower oscillations and shower gusts
- relative sea level rise

The tidal fluctuations depend on moon and sun cycles in combination with the oceanic configuration. They can be predicted fairly reasonably and are published by various authorities. In the Netherlands they can be found on the internet (http://www.getij.nl) and also in yearly paper publications. For the calculation of the tidal variations and the wind set-up see Appendix 1.
If the tidal variation is required for a remote location, or because the calculation model for wind set-up contains a number of uncertainties, whilst great detail is required regarding maximum water levels, a different approach is possible in practice, i.e. site observations or water level measurements. Using as many observed storm surge levels as possible, a frequency distribution (an empirical relation) can be produced. The Gumbel distribution and the exponential distribution are commonly used frequency distributions. By extrapolation, the design storm surge level can be determined for a previously selected small exceedance frequency.

The design height of wind waves should preferably be determined from extrapolated measurements. In case of insufficient or unreliable wave measurements, the significant wave height and wave period can be estimated using the Bretschneider method or the Groen & Dorrestein nomograms, see Appendix 3. It should be taken into consideration that the height of waves approaching the coast can change. This is due to refraction, shoaling, breaking, reflection or diffraction. For the calculation of the influence of these phenomena, look at the mentioned appendix.

Overtopping of water can be prevented by designing the structure up to a sufficiently high level. Authorities often prescribe the allowable exceedance frequency of a maximum water level and obviously this has to be treated as a major design requirement. Generally the momentary water level fluctuation due to waves is not included in the specified exceedance frequency. Frequently it is not strictly prohibited that (some) waves overtop the structure, on the condition that the main function is not jeopardised. This is taken care of by means of analysing the wave attack on the structure regarding maximum overtopping discharges, dependent on the type and size of the slope.

Maximum overtopping discharges for breakwaters and quay walls according to the European Overtopping Manual (2007):

- with respect to stability and strength of the structure: 50 to 200 ℓ/s/m if the inner slope is well protected
- regards to equipment and machinery on the quay wall or structure: 10 ℓ/s/m.

Appendix 4 deals with overtopping design rules in more detail.

Note. According to PIANC recommendations [*PIANC 1976*] an eventual extra freeboard of 1.3 to 1.5 H_u should be applied for caisson breakwaters. H_u is the design wave height related to the limit-state of use = $H_{z1/10}$, which is the average height of the highest one tenth of all wave heights. Because of the risk of failure of breakwaters due to wave impact, a too high toe berm should be avoided. PIANC prescribes that the berm toe is at least at -1.72 H_d. H_d is the design wave height related to the limit-state of rupture. The water depth at the vertical structure should be at least twice the expected wave height.

Consequences regarding ToS, or the height of the caisson, as a result of extra freeboard, seiches, shower oscillations and shower gusts, or relative sea level rise should be dealt with in similar ways as shown for tidal levels or waves. Keep in mind to use an appropriate level of precision, i.e. in initial design stages it is best to round off to decimal meters, centimetre or even millimetre business should be postponed to detailed engineering stages.

Intermezzo 4: reduction of the wave load on caisson breakwaters

The disadvantage of traditional fully vertical caisson walls is the exposure to large shock wave forces (in case they are not embedded in a dam). If overtopping or wave shocks will become problematic, several structural measures can be taken. One solution could be to use cylindrical caissons instead of the square type, see Figure 4-3a.

A wave front hitting a row of cylindrical caissons will reduce the resulting horizontal force up to 40% (according to physical model studies) because the wave front does not hit the entire contact area at once. A disadvantage of these cyliders is, however, high splashing of waves in between the cylinders.

Another way to reduce wave forces is the use of a superstructure on top of the caisson, see Figure 4-3b. Superstructures, however, could have negative impact on overtopping so this has to be studied in advance. A third wave force reducing measure is the use of absorbing chambers in caissons, see Figure 4-4a. If properly designed, these absorbing chambers dissipate wave energy resulting in smaller horizontal wave forces, reduced wave reflection, less wave overtopping and less scouring in front of the caisson structure. The caisson type depicted in Figure 4-4a, by the way, is not suitable for high wave periods.

Rubble mounds in front of the caisson can have the same function. The rubble mound can either be immediately in front of the caisson, see Figure 4-4b, or at a distance. The rubble mound induces wave breaking in front of the caisson and thus reduces the height of the waves and their impact on the caisson.

[*Haile, 1996-1*]

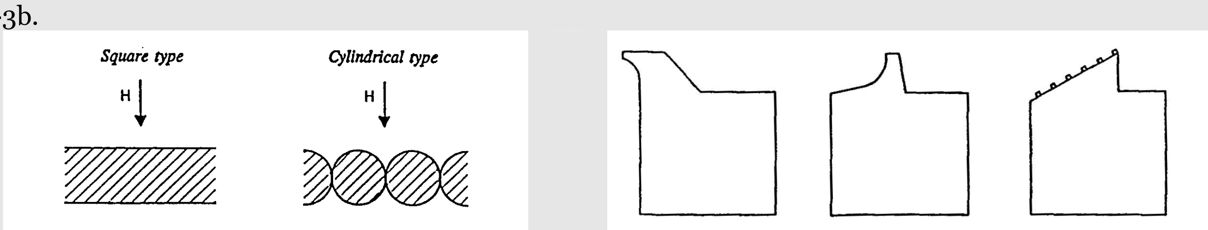

Figure 4-3 a. Cylindrical type caisson (top view) b. Superstructures (cross-section)

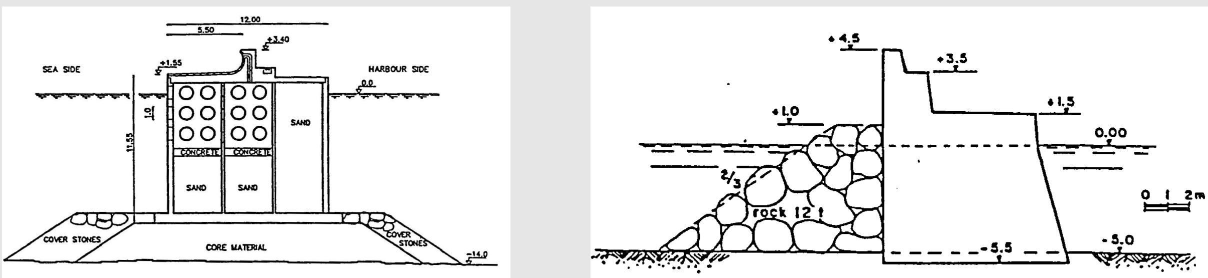

Figure 4-4 a. Absorbing chambers (cross section) b. Rubble mound in front of caisson (cross-section)

4.2.2 Width

Generally the caissons width is not subject to specifications resulting from one of the main functions of the structure; strength and stiffness throughout the life cycle stages and float conditions in the transport stage result in width requirements. Practical considerations and experience show it is best to determine the width required for transport first.

Once the caisson height has been selected, the width of the caisson has to be determined considering the required keel clearance during the floating transport stages of the caisson; this appears to be governing in most cases. In the equilibrium equation for floating objects (weight = buoyant force) the weight of the caisson has to be determined using a best guess for the width in order to be able to compute the draught. Wall thicknesses of 0.50 m and a bottom plate thickness of 1.00 m have proven to be reasonable start values (under normal circumstances). If the computed draught does not suffice to the keel clearance requirement the work has to be redone again adjusting the width. Alternatively the draught can be calculated using the minimum clearance requirements, followed by computation of the width of the caisson.

Horizontal and vertical forces on the structure and the resulting overturning moments in the final situation may result in adjustments of the width that would have been sufficient for the floating conditions. See Section 4.3.

4.2.3 Length

To decide on the length of the caisson simple rules of thumb based on construction practices could be used, or more realistic and advisable, immersion, navigation characteristics during transport and the resulting caisson strength and stiffness can be taken into consideration.

Longer caissons have the advantage that the total number of immersions, thus the total risk, will be reduced. However, (cross) currents during immersion, especially catching the caisson's longer side, are a problem in view of positioning, or even a risk considering collision to the previously immersed caisson. Hence, the length of caissons has to be limited to maintain manoeuvrability of the caisson. Another advantage of reducing the number of caissons is the reduction in the number of joints or shear-keys to be constructed, see Figure 4-5. The two walls with the connection joint require more concrete, labour and so on, than for instance the extra internal wall of a longer caisson, if needed at all. Hence, considering construction it is beneficial to construct caisson with bigger lengths, up to a certain limit.

Regarding navigation, again, manoeuvrability is the key thing. Some length:width ratios are more favourable for navigation than others. The length/width ratio of 2.2 / 1 of the Veersche Gat unity caissons was less favourable with respect to manoeuvrability. Relatively much power was needed to control the floating caissons under all circumstances. For the closure of the Brouwersdam, caissons were used with a length/width ratio of 3.8 / 1, which proved to be easily navigable. Tow tests at the Maritime Research Institute Netherlands (MARIN) showed that a length/width ratio of 3 / 1 is sufficient for navigation.

From a strength and stiffness point of view it would be beneficial if all the outer walls of the caisson were of the same thickness and constructed with similar and more or less homogeneous amounts of reinforcement. Note that reality is quite often different, especially the homogeneity of vertical and horizontal reinforcement because caisson walls generally are of the 'deep beam' type (*hoge wand ligger*). With prudence a simple rule of thumb to be used could be to select a length of the caisson equal to about two times the caisson height. Figure 4-5 shows the load transfer from the long side wall to supporting walls and caisson bottom to illustrate the 2/1-ratio; think about bending moments in supported and cantilevering beams that are proportional to $\frac{1}{8} q \ell^2$ and $\frac{1}{2} q \ell^2$ respectively. The previous observations are made for caissons without internal walls. Although not less true, in most cases strength and stiffness considerations are less decisive for selecting the caissons length.

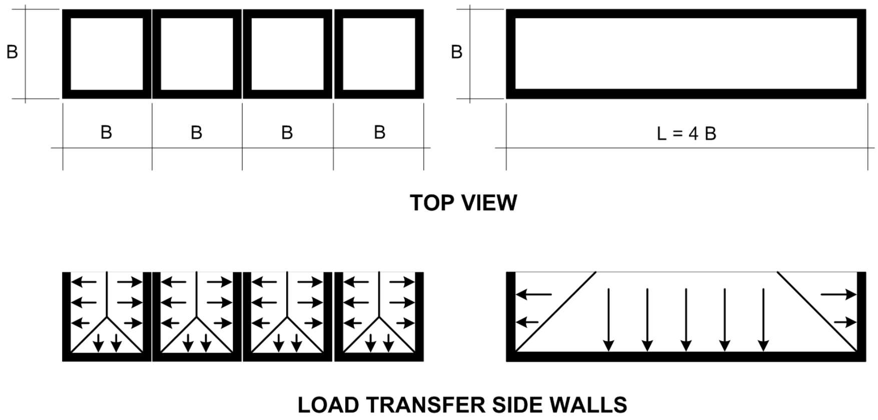

Figure 4-5 Top view and load transfer in side walls; 4 caissons versus 1

4.2.4 Thickness of the concrete elements

The governing load situation with respect to the dimensioning of external concrete elements (walls, roof and floor) could very well occur during the floating phase. The fact is that, during floatation, a considerable water load is acting on the outside of the caisson walls, but it is not balanced by counterload of ballast on the inside, thus no water or soil load resisting the outside load. The reinforced concrete structure then will have to bear the water load acting on the outside of the caisson all by itself. If, however, the bed under the caisson is not smooth, the governing load condition could occur when the fully loaded caisson rests on eventual bumps or big boulders on the bed. The caisson in this case is not evenly supported by the subsoil, which causes concentrated loads. This implies a considerable increase of bending moments and stresses in the concrete, compared to the floating phase, which is not unlikely to cause torsion of the entire caisson.

Failure caused by an unlevelled bed can be prevented by ensuring that the bed is reasonably smooth, or by reckoning with concentrated loads under the caisson during the design. A smooth bed can be accomplished by accurate dredging or precise rubble dumping, eventually followed by follow-up treatment and monitoring. A design measure to cope with this problem is to incorporate special beams or ridges in the bottom plate, so that the caisson will rest on these beams and not on the elevated floor in the first place, to prevent concentrated loads on the bottom plate. Another way to handle uneven loads is to dimension thicker walls and bottom plate to bear eventual concentrated loads due to bumps. Bent Hansen invented a theory to deal with this redistributed soil pressures. Mr. Hansen subdivided the floor in several parts and indicated parts with less and more acting stress, see Figure 4-6.

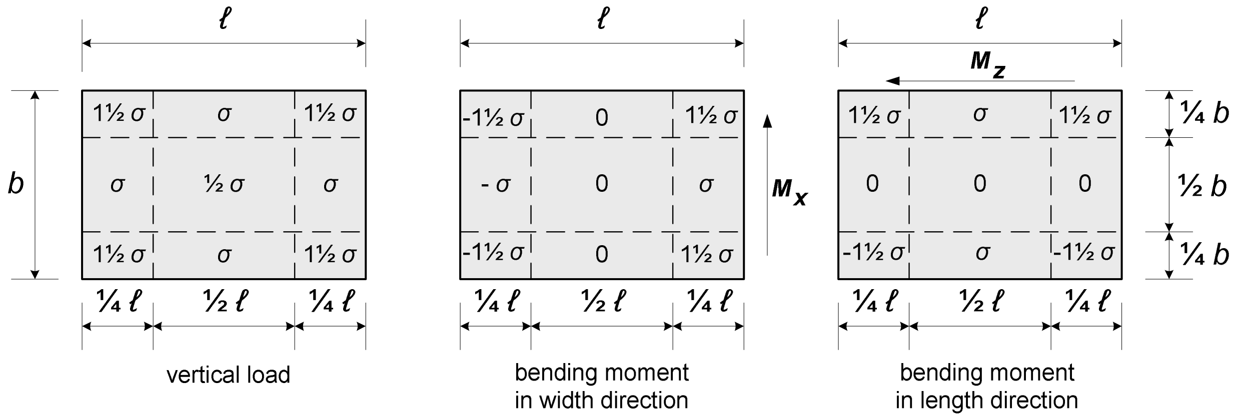

Figure 4-6 Soil pressure redistribution in the caisson floor, according to Bent Hansen

In this figure, the average stress caused by the vertical load F_y is:

$$\sigma = \frac{F_y}{\ell \cdot b}$$

The average stress caused by bending moment M_x is:

$$\sigma = \frac{64 \cdot M_x}{15 \cdot b \cdot \ell^2}$$

and the average stress caused by M_z is:

$$\sigma = \frac{64 \cdot M_z}{15 \cdot b^2 \cdot \ell}$$

The necessity for internal walls inside the caisson mainly depends on strength and stiffness, considering the caisson stand-alone and as part of the whole structure, albeit that increased floating stability in relation to ballast water may have a large influence. If internal walls are mainly intended to improve the floating stability, they are often called 'bulkheads' (*slingerschotten)*.

Internal walls or bulkheads may be constructed as high as the outer caisson walls or to intermediate heights; their wall thickness in most cases will be less than the outer wall thickness. Say the internal wall thickness is 60-80% of the outer wall thickness, thus 0.30-0.4 m for the first, if the last is 0.5 m.

It is advised to take the presence of internal walls into consideration in an early stage of the design, e.g., by adding 5 to 10% of the concrete self-weight. In this stage of the design the number of bulkheads has to be guessed based on experience or reference projects.

The presence of a roof depends on functional requirements. If there are no specific reasons to include a roof in the design, it could better be omitted because it requires extra material and quite some construction effort to build. A first estimate for the caisson roof thickness would be 1 meter; do note that functional requirements may easily increase this to 2 or even 3 meter.

4.2.5 Draught

If the found dimensions conflict with the required maximum draught, there are mainly three methods to reduce the draught of the caisson:

- reduction of the weight, generally by decreasing the thickness of walls or bottom slab, but it may result in finishing caisson construction ‘after’ immersion as well
- increase buoyancy of the structure, generally by increasing the width and/or the length of the caisson
- adding additional buoyancy during transport, for instance with help of drift bodies

Thus values for the caisson height, width and length are found, plus the estimated wall and bottom thicknesses. However, it should be checked if these dimensions suffice with respect to all load situations that can be expected. This is treated in the next section.

4.3 Design checks

4.3.1 Static stability of caissons (during transport and immersion)

The stability of a floating object is its ability to counteract forces eventually overturning the caisson, see Appendix 6. Tilting of caissons during transport or immersion could cause inconvenience or (worse) damage and should therefore be restricted. The design should provide a righting moment if tilt is initiated so that the caisson will turn back to its neutral position. Tilting around the length axis ('surging') is the most crucial, but in some cases tilting around the width axis ('pitching') should be considered as well. See Figure 4-7 for an overview of possible motions and their nomenclature.

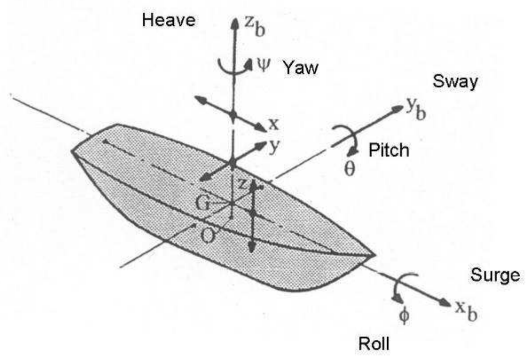

Figure 4-7 Ship or caisson motions

The stability of floating elements is usually checked by calculating the metacentric height, which is the distance between the centre of gravity and the meta centre. The meta centre is the intersection point of the *z*-axis (this is the vertical axis if the element is in its stable position) and the action line through the buoyant force. Caissons are considered to be stable if the metacentric height is at least 0.50 metre (see Appendix 6).

It is emphasised here that the metacentric height changes if a caisson is filled with (ballast) water, which is the case during submersion. It often does not suffice to just calculate the metacentric height regarding only one inner water level (see Figure 4-8 for an example).

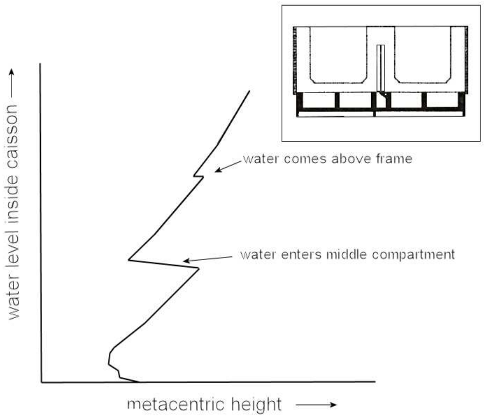

Figure 4-8 Example change of metacentric height depending on inner water level

4.3.2 Dynamic stability of floating caissons

If the dimensions (length or width) of a floating element are too small compared to the length of the waves or swell, the element will start swaying on the waves. In practice, a rule of thumb is being used:

$L_w < 0.7 \cdot \ell$ and $L_w < 0.7 \cdot b$ (dependent on the direction of the waves relative to the caisson)

where:
L_w = wave length [m]
ℓ = length of the caisson [m]
b = width of the caisson [m]

If this requirement is not fulfilled, problems due to swaying of the element can be expected.

If the natural oscillation period (*eigenperiode*) of the caisson more or less equals the wave or swell (*deining*) period, inadmissible and increasing motions will occur. The design should therefore include a dynamic stability check and eventually the dimensions have to be adapted (see Appendix 6 for the calculation method of dynamic stability).

If the natural oscillation period is a problem and adjustments of the design or additional measures do not offer a solution (or are too expensive), another solution would be to transport and position the caisson at the final location during more favourable conditions as far as waves and swell are concerned. However, this may result in serious work delays and thus larger costs. Therefore costs should be optimised considering the costs of additional measures and/or design and construction changes on one hand and the costs of possible delays on the other hand.

4.3.3 Shear criterion caisson-subsoil

The total of the horizontal forces acting on a caisson (on a shallow foundation) should be transferred to the subsoil (Figure 4-9). The friction force of the subsoil should resist the resulting total horizontal force. This friction force is determined by the total of the forces acting on the caisson in the vertical direction (or the vertical components of the forces), multiplied by a friction coefficient *f*. In equation form:
$\Sigma H < f \ \Sigma V$

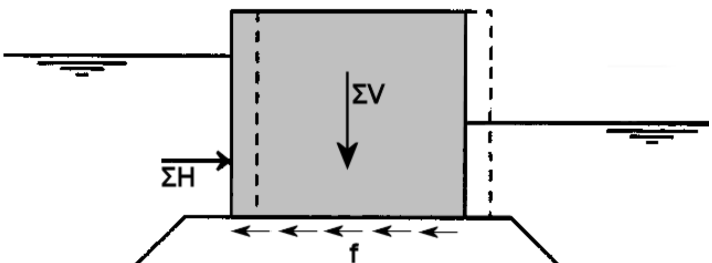

Figure 4-9 Slide-off principle sketch

The friction coefficient *f* takes several mechanisms into account. The most critical of these should be used:
1. friction between structure and subsoil: $f = \tan(\delta)$, with δ = friction angle between structure and subsoil. If δ is unknown, it can be approximated: $\delta \approx ⅔\ \varphi$ (φ is angle of internal friction of the subsoil). The friction coefficient for caisson-rubble is about 0.5.
2. Internal friction of the subsoil: $f = \tan(\varphi)$, where φ is the angle of internal friction of the subsoil.
3. A deeper soil layer with a low sliding resistance.

The impact of waves should also be considered in the force and moment equilibriums. See Appendix 7.

4.3.4 Rotational stability
Contrary to compression stresses perpendicular to bottom of the structure and friction acting in the plane of the structure (bottom) and the soil, tensile stresses perpendicular to the bottom of the structure can not develop. Considering the stability of shallow foundations a tensile force between structure and the soil will not enter the force equilibrium equation. Especially the adhesive and cohesive properties of sand are very poor. If the resulting action force intersects the core of the structure, the soil stresses will be positive (= pressure) over the entire width. The core is defined as the area extending to 1/6 *b* on both sides of the gravity centre line, see Figure 4-10.

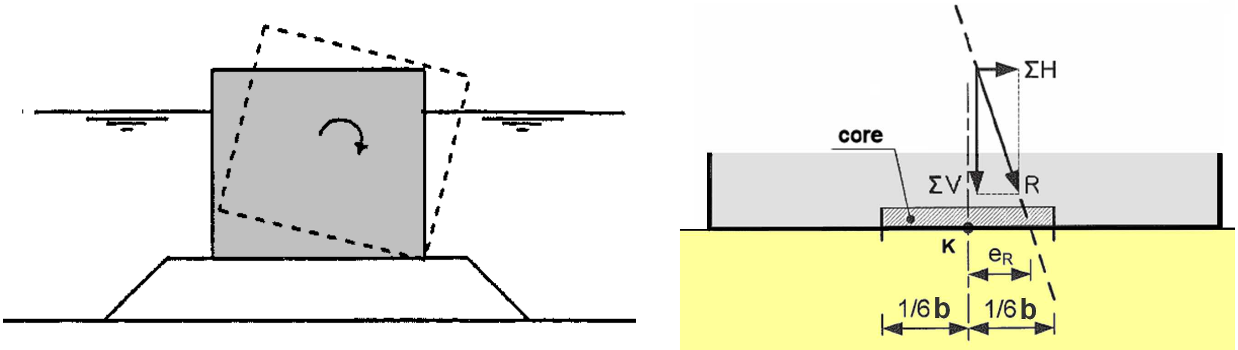

Figure 4-10 The action line of the resulting force should intersect the core of the structure

It should be checked that:

$$e_R = \frac{\Sigma M}{\Sigma V} \leq \frac{1}{6} b$$

where: e_R = distance from the moment centre (K) to the intersection point of the resulting force with the bottom line [m]
ΣV = total of the acting vertical forces (or vertical components) per structural element [kN]
ΣM = total of the moments, preferably around point K, per structural element [kNm]
b = width of the structural element [m]

4.3.5 Vertical bearing capacity

The required vertical effective soil stress should not exceed the maximum bearing capacity of the soil, otherwise the soil will collapse. The maximum acting stress on the soil can be calculated with:

$$\sigma_{k,\max} = \frac{F}{A} + \frac{M}{W} = \frac{\sum V}{b \cdot \ell} + \frac{\sum M}{\frac{1}{6} \ell\, b^2}$$

where:
- F = normal force [kN]
- A = area perpedicular to the normal force [m^2]
- M = acting moment [kNm]
- W = section modulus [m^3]
- ΣV = total of the acting vertical forces (or vertical components) [N]
- b = width of the structural element [m]
- ℓ = length of the structural element [m]
- ΣM = total of the moments, preferably around point K, halfway the width [kNm]

The bearing capacity can be calculated according to TGB 1990 (NEN 6744), which gives the Brinch Hansen method for determining the maximum bearing capacity of a foundation. This method takes into account the influence of cohesion, surcharge including soil coverage and capacity of the soil below the foundation (see Appendix 8).

As a rule of thumb (instead of the Brinch Hansen calculation), the bearing capacity of densely packed sand is often assumed to be 500 kN/m^2 (= 0.5 N/mm^2).

4.3.6 Piping and scour

Groundwater flow under or around a water or soil retaining structure is caused by a potential difference across the structure. Piping can occur at the plane separating the impermeable structure and the loose grains. Piping is the flow of water through a pipe-like channel that has been created by internal erosion. This phenomenon can occur along the foundation plane of a structure but also along a retention wall. Piping is also possible in dikes. Little “sand volcanoes” are created where the water flows out at ground level.

Empirical formulas based on research describe the critical situations in which piping can occur. The most famous are the Bligh and Lane formulas. According to these formulas there is a limit state with a critical ratio between the differential head and the seepage distance. More recent research has confirmed this. The calculation of the required seepage distance is explained in Appendix 9.

Bottom scour may affect the waves in front of the structure and can lead to gradual dislocation of the sill and can decrease the geotechnical stability of the breakwater (Figure 4-11). The scour depth in front of vertical breakwaters may, under the worst conditions, reach values up to 0.7 times the original water depth.

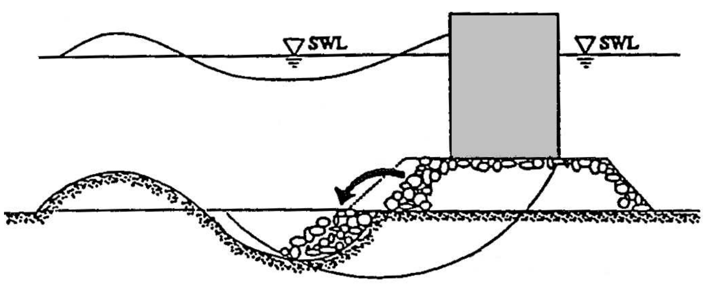

Figure 4-11 Effect of bottom scour on breakwater stability

Scour can be prevented by applying geometrically tight granular or geotextile filters. A granular filter should be designed in such a way that grains in the basic layer cannot pass through the holes of the filter. (i.e. the pores in the grain packet which are interconnected by small pore channels). If the diameter of the pore channels D_c is smaller than the diameter of the governing grains of the basic layer D_b, no transport can take place irrespective of the value of the water level slope or the direction and type (stationary or not) of the flow. For filter design rules one is referred to Appendix 10.

4.3.7 Strength of the reinforced concrete structure

The required strength of the structure depends on the shape and dimensions of the structure and the strength properties of the construction materials. The dimensions of the caisson may have been chosen with respect to requirements other than strength or stiffness alone. In any case, it should be checked whether or not all loads (in all load situations and/or combinations) actually can be resisted by the structure.

The first step in this check is to schematise the structure and the acting loads in the distinguished stages of its lifetime. Especially the maximum bending moment and maximum shear force should be checked for Ultimate Limit States (ULS). Crack width and fatique are typical phenomena checked for Serviceability Limit States (SLS). With help of construction mechanics principles, the normal force, shear force and moment diagrams (*normaalkrachten-, dwarskrachten- en momentenlijnen*) can be determined.

In the first step the structure will be checked by means of hand calculation; typical cross-sections will be evaluated. To do so the structure is schematised from a 3D to a 2D, even into a 1D structure. For the resulting wall, beam or column elements spreadsheets are generally readily available to check dimensions and/or the properties of the concrete and steel. See Figure 4-12 for a sketch of the decomposition of a caisson into several elements.

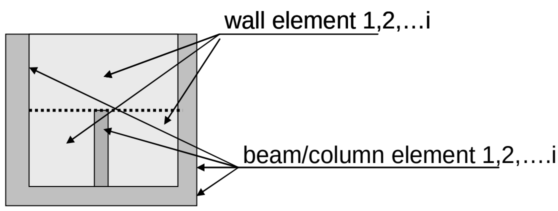

Figure 4-12 Schematisation

If the number of elements increases and/or the structure is statically undetermined, which generally is the case, a hand calculation may become rather complicated. In these situations the use of 2D plane frame calculation software helps to determine the ‘hot spots’, i.e. the location where sectional forces are either maximum or minimum, of the structure. Tie-strut models of the considered (cross)section may also assist in the analysis.

Bending moments

Considering concrete sections, it has to be determined how much tension and compression reinforcement is required. The dimensions of the section and/or the strength properties are generally acceptable when the main reinforcement amounts to 100 kg/m^3 of concrete; obviously this figure is only a rough first estimate. Reinforcement in the other direction, shear reinforcement or crack width limiting reinforcement may double or triple this figure.

The thickness of the concrete should be sufficient to resist the bending moment. For a first estimate Table 7-12 of appendix 11 can be used to find the thickness of the concrete wall able to resist the bending moment (under the assumption of an economic reinforcement percentage).

Shear stress

The shear stress (τ) criterion can be checked using the maximum shear force V. A little bit simplified:

$$\tau_d = \frac{3}{2}\frac{V_d}{b \cdot t} \leq \tau_1$$

where

τ_d = the design value of the shear stress [N/mm^2]
V_d = the design value of the shear force in the considered cross-section [N]
t = thickness of the concrete part that should bear V_d [mm]
τ_1 = the maximum allowable shear stress if no shear reinforcement is applied [N/mm^2]:

$$\tau_1 = 0.4 f_b + 0.15 \sigma'_{bmd}$$

f_b = design value of concrete compressive strength [N/mm^2]
σ'_{bmd} = average design value of concrete compressive strength [N/mm^2]

For more detailed equations see Appendix 11.

More detailed structural analysis

The first step hand calculation provides a fairly accurate indication of the dimensions of the concrete structure. However, since the structure was schematised into a 2D or even 1D element a more precise calculation should follow taking into account, e.g. the spread of forces in multiple directions. To do this work generally 3D FE (finite element) calculations are performed.

Generally the output of 3D FE calculations produce the cross-sectional forces in typical cross-sections. Obviously these have to be checked against dimensions and the amount of available reinforcement. More advanced FE programs have a modular structure including the modules to do these checks.

4.4 Example 'hand calculation' standard caisson

4.4.1 Situation and hydraulic boundary conditions

Given is a fictitious small estuary somewhere in the Netherlands. The estuary has a size of 20 km^2, is about 1 km wide and 20 km long. For the location and the hydraulic boundary conditions see Figure 4-13. It has been decided to close off this estuary in order to shorten the coastline, hence to reduce the number of dikes to be improved because of expected sea level rise.

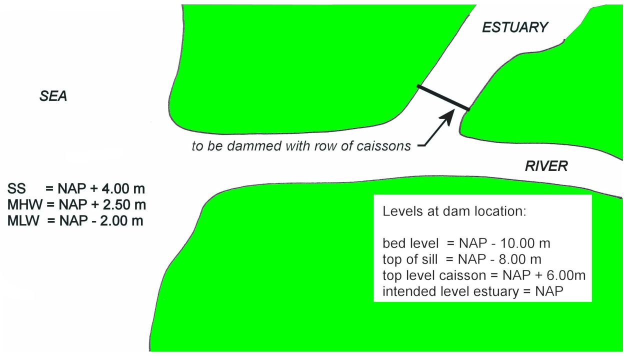

Figure 4-13 Sketch of the estuary and location of the dam

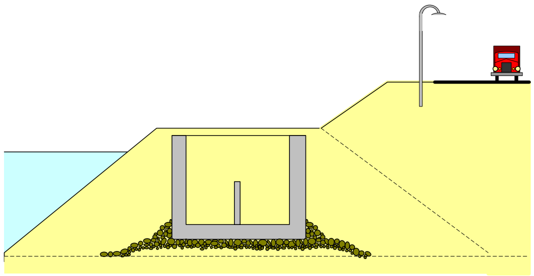

Figure 4-14 Typical cross section of the closure dam (final stage including the caisson core)

The use of caissons is an interesting option for the cosure of this estuary, see Figure 4-14 for a typical cross section. The caissons will have to be placed on a sill, which should be as high as possible to lower the cost of caisson construction. The sill, however, reduces the area for water flow, thus increases flow velocities, which may create a problem for caisson positioning and immersion. There are requirements with respect to the final dam height, but the commissioner has given the required height of the caissons for the temporary situation with only a row of caissons as well. Of course closed caissons are to be preferred given the high costs of sluice caissons. However, one has to check first if a closure with closed caissons is feasible taking into account the hydraulic boundary conditions.

Closing an estuary with (closed) caissons is only possible if the flow velocity in the last closure gap will not too high, and during slack water there has to be sufficient time to immerse the caisson in a safe way. In practice this means that the maximum flow velocity in the closure gap is U_0 = 2.50 m/s in both directions.

Figure 4-15 shows a graph that can be used to determine the maximum sill height in semi-diurnal tide conditions, given the maximum flow velocity of 2.5 m/s during immersion (reference is made to course CT5308, chapter 12, for further details). Originally this graph contains lines for three tidal amplitudes: 2.0, 1.0 and 0.5 m. In this example the tidal difference is 4.50 m, so the tidal amplitude is 2.25 m. To find the required waterdepth above the sill, an extra line has to be drawn. This line is seperately indicated in the graph.

Assume that the (last) caisson has a length of approximately 50 m. The estuary to be closed has a size of 20 km^2. Then the ratio $B/B_s = 20{\cdot}10^6/50 = 0.4{\cdot}10^6$ (B is area of the estuary, B_s is the width of the last gap in the closure). With the B/B_s ratio of $0.4{\cdot}10^6$ and using some extrapolation the graph shows that the minimum water depth should be 8 m. Assuming MWL at 0 m NAP and using the bed level of NAP – 10, the resulting sill height is 2 m, which is a feasible value.

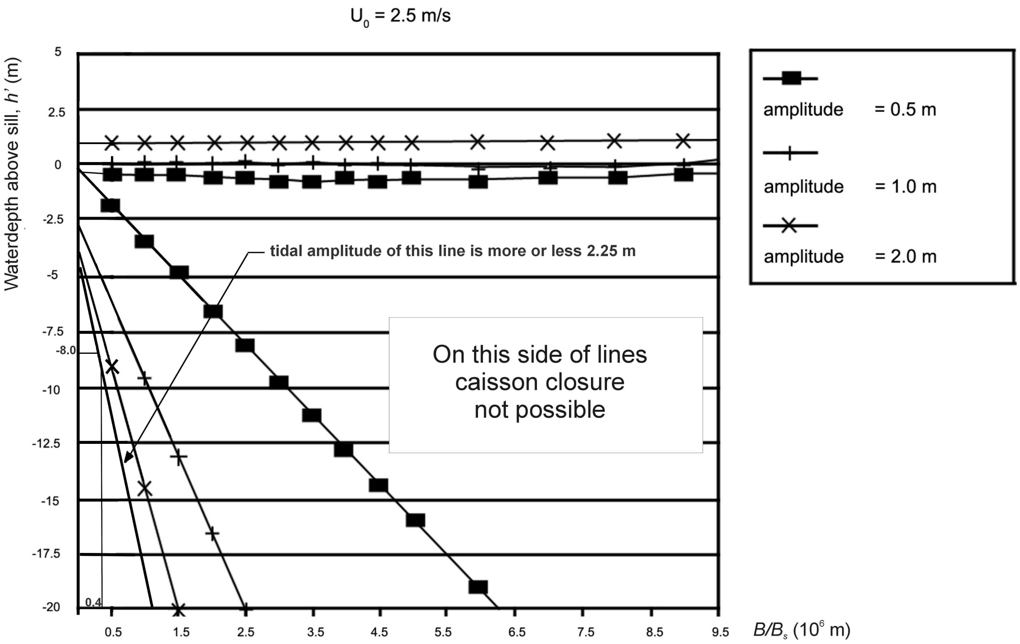

Figure 4-15 limiting conditions for a semi-diurnal tide

Based on experience with this type of caisson in this situation, the sequence to work through the design stages and calculations is more or less chronologically. The example is intended to show the development of a conceptual design. During later design stages (not worked out in these lecture notes) the conceptual design has to be verified with detailed calculations, computer programmes and possibly physical model testing. To get a grip on the design and a feeling for the structure in these initial design stages, extreme load conditions are used without load or material factors or other safety factors.

4.4.2 Phase I: Caisson in building dock

The first phase in the life cycle of a caisson is the building dock phase. Because of the lack of external loads this phase is not governing for structural design of the caisson. Attention should be paid to ensure that formwork removal (*ontkisting*) is not started before the required strength of the concrete has been reached. The caisson has to be floated out; in case of insufficient depth, the draught of the caisson should be reduced for instance by increasing the width.

4.4.3 Phase IIa: Floating caisson – estimating the main dimensions

For the considered type of caissons, the floating phase often appears to be governing for the caisson width. During floatation there should be equilibrium between the buoyant force and the weight of the caisson. Using a given height, the minimum required keel clearance, and a length-width ratio of 3 (a good ratio from navigation point of view) the main dimensions of the caisson can be estimated.

The level of the sill top is given: NAP – 8.00 m, and also the required top level of the caisson: NAP + 6.00 m. Therefore, the caisson height is easily be calculated: h = 6.00 - (-8.00) = 14.00 m.

The draught (d) of the caisson is limited by the required minimum keel clearance. This means the buoyant force should be large enough. To determine the buoyant force (F_b), the under-water volume of the caisson (V_{uw}) has to be computed:
$V_{uw} = b \cdot \ell \cdot d$ [m^3]

A length-width ratio of $\ell = 3b$ has proven reasonable with respect to navigability, so:
$V_{uw} = b \cdot 3b \cdot d = 3\,\ell^2 d$ [m^3]

The buoyant force then is:
$F_b = V_{uw} \cdot \gamma_w = 3\,b^2 \cdot d \cdot \gamma_w$ [kN], where b and d are unknown parameters.

The allowable draught has to be determined considering various bed and water levels. Two situations that have to be examined anyway are transport and positioning & immersion. Note the investigation shall not be limited to these two situations alone.

During transport there should be at least 1.00 m keel clearance. It should be taken into account that the water level in relatively small canals will lower a bit because of the moving caisson. Transport will take place during MHW (mean high water). (This is a simplification: transport will take some time during which the water level will change, so in reality this should be investigated in more detail).
The bed level is at NAP – 10.00 m and MHW is at NAP + 2.50 m, so with 1.00 m clearance the maximum allowed draught is: d = 2.50 - (-10.00) - 1.00 = 11.50 m.

During positioning above the sill, the manoeuvres will be more careful, so a keel clearance of 0.50 m suffices here. The positioning will take place immediately before immersion, so at MLW (mean low water) (Another assumption to be checked with the caisson transport project manager!).
The sill level is at NAP – 8.00 m, MLW is at NAP – 2.00 m, so with 0.50 m keel clearance the draught is:
-2.00 - (-8.00) - 0.50 = 5.50 m.

Both conditions should be valid, so $(d \leq 11.5 \text{ m}) \wedge (d \leq 5.5 \text{ m}) \rightarrow d \leq 5.5 \text{ m}$
Then:
$F_b = 3b^2 \cdot d \cdot \gamma_w = 3b^2 \cdot 5.5 \cdot 10 = 165b^2$, where the caisson width W is the only unknown parameter. This can be solved using the force equilibrium. To determine the weight of the caisson, the thickness of walls and bottom slab has to be guessed for the time being. With some experience this can be done rather accurately. If experience lacks, this will probably imply that the design cycle will be extended with some extra iterations. At the moment a wall thickness of t_w = 0.50 m and a bottom thickness of t_b = 1.00 m is assumed. A roof is not necessary in this case. Therefore, the weight of the caisson is:

$F_w = \{ \ell \cdot b \cdot h - (\ell - 2t_w) \cdot (b - 2t_w) \cdot (h - t_b)\} \cdot \gamma_c$
$\ell = 3 \cdot b$ and h = 14.00 m, so $F_w = 75\, b^2 + 1300\, b - 325$

In the floating condition, equilibrium exists between the buoyant force and weight:

$F_w = F_b \rightarrow 75\, b^2 + 1300\, b - 325 = 165\, b^2 \rightarrow b = 0.25$ or $b = 14.19$ [m]

The only realistic solution is b = 14.19 → 15 m. With b = 15 m, the draught will be 5.34 m.

So, the caisson dimensions for the time being are:
h = 14.00 m
b = 15.00 m
ℓ = 45.00 m
t_w = 0.50 m
t_b = 1.00 m

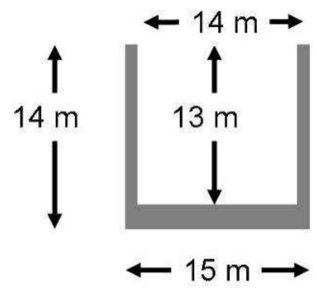

Figure 4-16 cross section of the caisson

4.4.4 Phase IIa: floating caisson - strength check

If it can be assured that the caisson will rest on a reasonably flat bed (after immersion), the floating phase is likely to be governing for the moments and stresses in the concrete side-walls. This is caused by the combination of high water pressure outside, while there is no pressure working inside-out in the empty caisson.

> **Note**. In this example calculation it turns out that the governing load for structural wall design appears to occur during floatation, which is only a temporary load condition.
> 1. It might be quite uneconomical to let a temporary load condition govern the design.
> 2. The use of proper load factors in SLS and ULS cases could dramatically change the conclusions on governing load combinations, hence on the dimensions of the caisson
> 3. Bumps in the bed or big boulders lying on the bed can cause high concentrated loads if the caisson will be placed on top of them. This could be more critical than the high unsupported water loads on the outside of the caisson during floating transport. In this example, however, a smooth flat bed is ensured.

For a first calculation, a cross-section in the middle (length direction) is considered. The influence of the head walls is neglected, which works out on the safe side because in reality the head walls take over part of the horizontal forces. Also, in this stage of the design, the effect of eventual internal walls is neglected for simplicity reasons.

The water pressure reaches its maximum value at the lowest point of the caisson:
$p_{max} = d \cdot \gamma_w = 5.34 \cdot 10 = 53.4$ kN/m^2.

The resulting horizontal force due to water pressure on a side wall reaching to the bottom of the structure is: $R_H = ½\, p_{max}\, d = ½ \cdot 53.4 \cdot 5.34 = 143$ kN/m. This is equal to the maximum value of the S-diagram acting on the walls (Figure 4-17e). The equation for the S-diagram in the walls is $p(x) = ½ \cdot p_{max} \cdot x^2 \; \ell\, d$.

The dead weight of the wall is: $t_w \cdot h \cdot \gamma_c = 0.50 \cdot 14 \cdot 25 = 175$ kN/m' in vertical direction (= 350 kN/m^2).

With these results the normal and shear force diagrams can be sketched (Figure 4-17d and e).

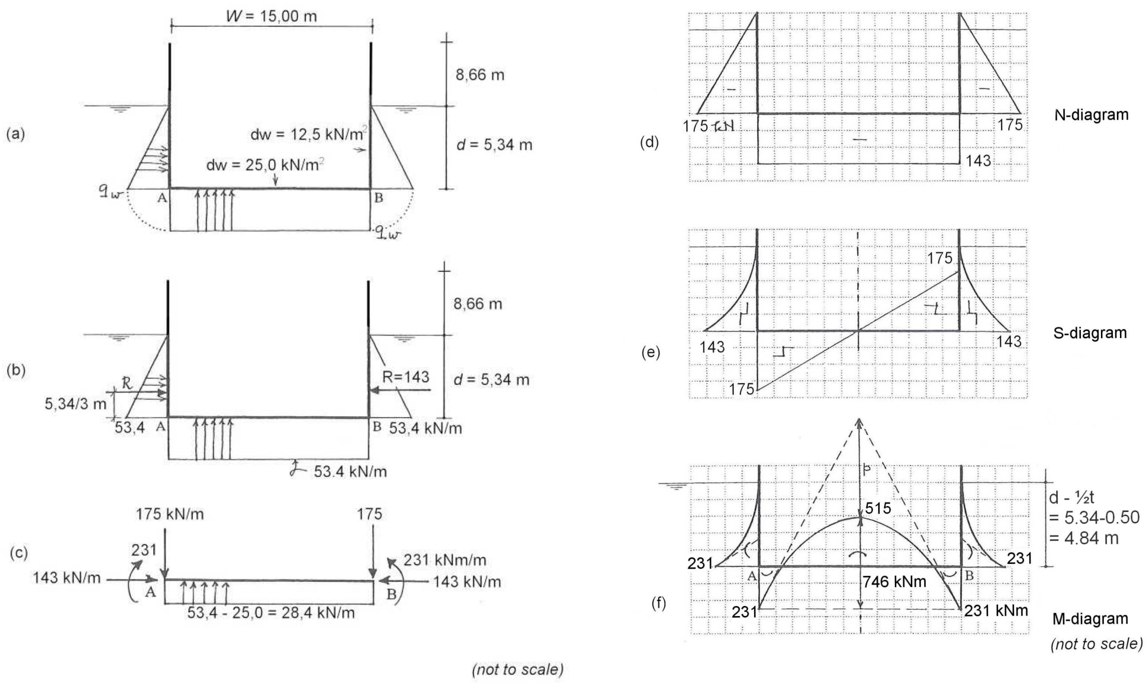

Figure 4-17 Diagrams of load, normal force, shear force and bending moment

The Dutch standard TGB 1990 prescribes a shear stress criterion:

$$\tau = \frac{3}{2}\frac{V}{b \cdot t} \leq \tau_1 \quad \text{where } \tau_1 = 0.4 f_b + 0.15\sigma_{bmd}'$$

Critical shear planes appear to be close to the lower corners of the caisson, see Figure 4-18.

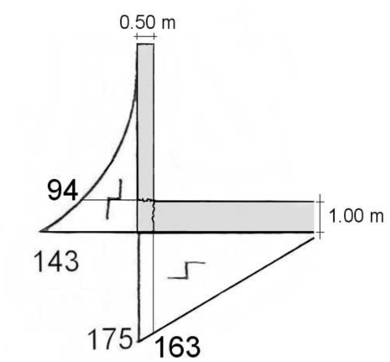

Figure 4-18 detail S-diagram

For the bottom plate:

$$\tau_b = \frac{3}{2} \cdot \frac{163}{0.9 \cdot 1.0} = 272 \text{ kN/m}^2 = 0.272 \text{ N/mm}^2$$

For the side walls:

$$\tau_w = \frac{3}{2} \cdot \frac{94}{0.9 \cdot 0.5} = 313 \text{ kN/m}^2 = 0.313 \text{ N/mm}^2$$

For concrete quality B45, the tensile strength f_b is 1.65 N/mm^2, so 0.4 f_b = 0.660 N/mm^2, which suffices for both τ_b and τ_w (even without the contribution of $0.15\sigma_{bmd}'$).

For the moment diagram, the loads are supposed to act on a schematized structure. Therefore, the dimensions of the system lines are used for the calculation of the moments. The bending moment due to water pressure in the side walls is maximum at the height of the horizontal system line (middle of the bottom plate):

$M = R_H \cdot e = 143 \cdot (5.34 - 0.50)/3 = 231$ kNm/m

The vertical load acting on/in the bottom plate is caused by vertical water pressure and the dead weight of the concrete plate. The field moment resulting from this is: $M' = {}^1/_8\, q\, \ell^2 = {}^1/_8\, (53.4 - 25.0)\cdot 14.5^2 = 746$ kNm/m. Taking the bending moments of the corners into account, the total resulting field moment in the middle of the bottom plate is $M - M' = 231 - 746 = -515$ kNm/m. This is sketched in the M-diagram in Figure 4-17f.

With help of the found maximum moment and Table 7-12 (Appendix 11) the required wall thickness can be estimated. To do so, an economic reinforcement percentage has to be chosen. A value of 1% has proven to be reasonable.

For this percentage, Table 7-12 gives a value for $\dfrac{M_d}{b \cdot t^2 \cdot f'_b}$ of 150, so:

for the bottom plate: $t_b = \sqrt{\dfrac{M_d}{b \cdot 150 \cdot f_b'}} = \sqrt{\dfrac{515}{1.0 \cdot 150 \cdot 27}} = 0.36 \text{ m} < 1.00 \text{ m}$

for the side walls: $t_w = \sqrt{\dfrac{M_d}{b \cdot 150 \cdot f_b'}} = \sqrt{\dfrac{231}{1.0 \cdot 150 \cdot 27}} = 0.24 \text{ m} < 0.50 \text{ m}$

So the thicknesses of the wall and bottom plate are sufficient to withstand the acting forces.

4.4.5 Phase IIa: floating caisson - static stability

To avoid instability of the caisson (too big rotation around the length axis), its metacentric height should be more than 0.5 meters.

The weight of the caisson is $F_w = \{ \ell \cdot b \cdot h - (\ell - 2t_w)\cdot(b - 2t_w)\cdot(h - t_b)\} \cdot \gamma_c =$
$\{45 \cdot 15 \cdot 14 - 44 \cdot 14 \cdot 13\} \cdot 25 = 36\,050$ kN

The position of the gravity centre G is:

$$\overline{\text{KG}} = \frac{\sum V_i \cdot e_i \cdot \gamma_i}{\sum V_i \cdot \gamma_i} = \frac{\sum V_i \cdot e_i}{\sum V_i} = \frac{45 \cdot 15 \cdot 14 \cdot 7.0 - 44 \cdot 14 \cdot 13 \cdot (1 + 6.5)}{45 \cdot 15 \cdot 14 - 44 \cdot 14 \cdot 13} = 4.22 \text{ m} \quad \text{above the underside.}$$

The draught of the element is:

$$d = \frac{F_w}{b \cdot \ell \cdot \gamma_w} = \frac{36\,050}{15 \cdot 45 \cdot 10} = 5.34 \text{ m},$$

so $\overline{\text{KB}} = \frac{1}{2} d = 2.67$ m.

The area moment of inertia of the area that intersects the water surface is: $I = \frac{1}{12} \cdot \ell \cdot b^3 = \frac{1}{12} \cdot 45 \cdot 15^3 = 12\,656 \text{ m}^4$.

The volume of the displaced fluid is: $V_{uw} = \ell \cdot b \cdot d = 45 \cdot 15 \cdot 5.34 = 3\,604.5 \text{ m}^3$.

Then $\overline{\text{BM}} = \dfrac{I}{V} = \dfrac{12\,656}{3\,604.5} = 3.51 \text{ m}$.

The metacentric height is: $h_m = \overline{\text{GM}} = \overline{\text{KB}} + \overline{\text{BM}} - \overline{\text{KG}} = 2.67 + 3.51 - 4.22 = 1.96 > 0.50$, so there is no problem with the static stability.

4.4.6 Phase IIa: floating caisson - dynamic stability

Considerable swinging of the caisson on the waves or swell should be avoided. Based on experience, the following rule of thumb is often used for a check:
If the dimensions (length or width) of a floating element are too small compared to the length of the waves or swell, the element will start swaying on the waves. In practice, the following rule of thumb is being used:

$L_w < 0.7 \cdot \ell$ and $L_w < 0.7 \cdot b$ (dependent on the direction of the waves relative to the caisson)

where L_w = wave length or swell length [m]

If this condition does not apply, problems due to swaying of the element can be expected.

In our case: $L_w < 0.7 \cdot 48.0 = 33.6$ m if the wave direction is parallell to the length axis of the caisson
In our case: $L_w < 0.7 \cdot 16.0 = 11.2$ m if the wave direction is perpendicular to the length axis of the caisson

Compared with normal wave conditions, this will be no problem in a river (assume that the building dock is upstream along the river). In reality, actual local wave data should be checked anyway in a later design stage!

It should also be avoided that the periods of wave movements come close to the natural oscillation period of the structure. Hence the natural oscillation period has to be calculated.

For the time being, the influence of the head walls is neglected. It can be checked easily that this is reasonable for the current dimensions.

First I_{xx} around the z-axis (vertical axis) is calculated:

$$I_{xx} = \frac{1}{12} \cdot t_b \cdot b^3 + 2 \cdot \left\{ \frac{1}{12}(h - t_b) \cdot t_w^3 + (h - t_b) \cdot t_w (\frac{1}{2}b - \frac{1}{2}t_w)^2 \right\} = 967 \text{ m}^4$$

I_{zz} is around the x-axis, the horizontal axis in width direction:

$$I_{zz} = \frac{1}{12} \cdot b \cdot t_b^3 + b \cdot t_b \cdot (\overline{\text{KG}} - \frac{1}{2}t_b)^2 + 2 \cdot \left\{ \frac{1}{12} \cdot t_w \cdot (h - t_b)^3 + (h - t_b) \cdot t_w (t_b + \frac{1}{2}(h - t_b) - \overline{\text{KG}})^2 \right\} = 370 \text{ m}^4$$

Notice that Steiners theorem has been used here for the calculation of I_{xx} and I_{zz} (see Figure 4-19).

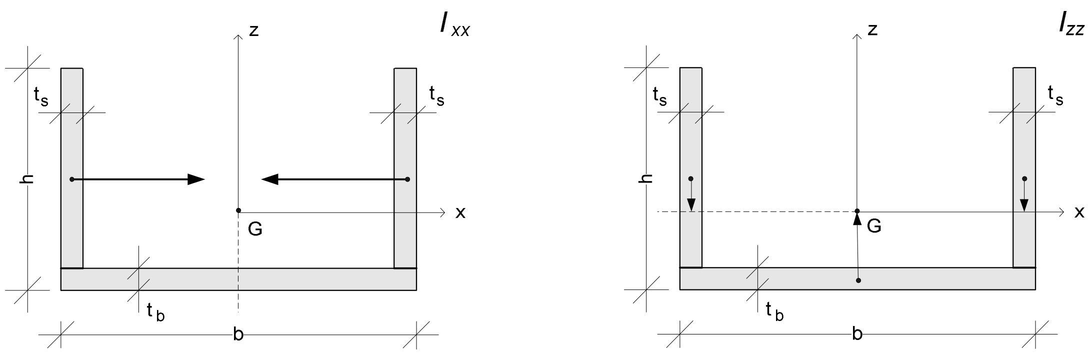

Figure 4-19 Indication of translation direction for application of Steiners theorem for I_{xx} (left) and for I_{zz} (right).

The polar moment of inertia: $I_p = I_{xx} + I_{zz} = 967 + 370 = 1337$ m^4.

The area of concrete in the cross-section is: $A_c = (14 \cdot 15) - (13 \cdot 14) = 28$ m^2.

The polar inertia radius then is: $j = \sqrt{\frac{I_p}{A_c}} = \sqrt{\frac{1337}{28}} = 6.91\,\text{m}$

The natural oscillation period is:

$$T_0 = \frac{2 \cdot \pi \cdot j}{\sqrt{h_m \cdot g}} = \frac{2 \cdot \pi \cdot 6.91}{\sqrt{1.96 \cdot 9.81}} = 9.90 \text{ s}$$

For normal river conditions, also natural ascillation will be no problem.

4.4.7 Phase IIb: immersing caisson - static stability

During phase IIb the situation is slightly different from phase IIa, the caisson is still floating, but some ballast water has been let in to increase the draught. This means that the weight and the value of $\overline{KB}$ have increased, but $\overline{KG}$ is now lower. The underwater volume will be increased now, but the moment of inertia will be decreased considerably which will also significantly reduce the value of $\overline{BM}$. This results in a smaller value of $h_m = \overline{GM} = \overline{KB} + \overline{BM} - \overline{KG}$. In other words: as soon as water has been let in, the caisson will be dramatically less stable. First only 10 cm of water inside the caisson will be considered:

The weight of the concrete caisson is still $F_{w,\ concrete}$ = 36 050 kN and $F_{w,\ water}$ = 616 kN

The calculation of position of the gravity centre G is a bit more complicated than without ballast water because of the different densities of concrete and water:

$$\overline{KG} = \frac{\sum V_i \cdot e_i \cdot \gamma_i}{\sum V_i \cdot \gamma_i} = \frac{(45 \cdot 15 \cdot 14 \cdot 7.0 - 44 \cdot 14 \cdot 13 \cdot 7.5) \cdot 25 + (44 \cdot 14 \cdot 0.10 \cdot 1.05) \cdot 10}{(45 \cdot 15 \cdot 14 - 44 \cdot 14 \cdot 13) \cdot 25 + (44 \cdot 14 \cdot 0{,}10) \cdot 10} = 4.17 \text{ m}.$$

The draught d of the element is:

$$d = \frac{F_{w,Total}}{b \cdot \ell \cdot \gamma_w} = \frac{36\,666}{15 \cdot 45 \cdot 10} = 5.43 \text{ m},$$

so $\overline{KB} = \frac{1}{2}d$ = 2.72 m.

The area moment of inertia of the area that intersects the water surface is:

$$I = \frac{1}{12} \cdot \ell \cdot b^3 - \frac{1}{12}(\ell - 2t_w)(b - 2t_w)^3 = 2595 \text{ m}^4.$$

The volume of the displaced fluid is: $V_{uw} = \ell \cdot b \cdot d$ = 3 667 m^3.

Then $\overline{BM} = \frac{I}{V} = 0{,}708$ m.

The metacentric height is: $h_m = \overline{GM} = \overline{KB} + \overline{BM} - \overline{KG}$ = 2.72 + 0.71 - 4.17 = -0.74 m << 0.50 m, so now the caisson is unstable! The instability will decrease when more water is let in, but at d = 6.00 m, the metacentric height still is insufficient: -0.40 m.

So, the design should be changed. The remedy is to add one or more bulkheads (*slingerschotten*) to improve stability. This; however, implies that also draught and strength should be checked again.

In this example, one bulkhead of 6.50 m height will be added in the length direction of the caisson. It will have no structural function and the wall thickness t_{bh} is assumed to be 0.40 m (Figure 4-20).

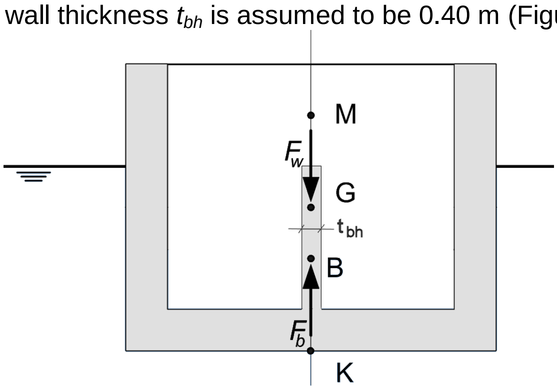

Figure 4-20 Cross-section with bulkhead

The draught will be more now because of the increased weight:
F_w = 38 910 kN
d = 5.76 m, which is more than the allowable 5.50 m.

To decrease the draught more buoyant force is required, which can be achieved by increasing the width. If b = 16 m is assumed, ℓ = 3b = 48 m and F_w = 42 730 kN, then d = 5.56 m, which is probably not a big problem.

The strength of the structure should also be checked again. This can be done with the help of normal, shear and moment diagrams like demonstrated in Section 4.4.4. The check shows that strength is not a problem with the increased dimensions.

Now the static stability during transport can be re-checked:

$\overline{KG} = 4.12\text{ m}$, which is less than in the previous situation, as could be expected.

d = 5.56 m, so $\overline{KB} = 2.78\text{ m}$.

I = 16 384 m^4 and V_{uw} = 4270 m^3, so $\overline{BM} = \dfrac{I}{V_{uw}} = 3.84\text{ m}$.

Then h_m = 2.5 m > 0.5 m.

The dynamic stability during transport should also be checked again:
I_{xx} = 1075 m^4 and I_{zz} = 552 m^4, so I_p = 1627 m^4.
A_c = 31.6 m^2, so j = 7.18 m and T_0 = 10.2 s. This is much longer than normal inland wave periods.

The static stability during immersion should now be improved:
Check with only 10 cm of water:
$\overline{KG} = 4.07\text{ m}$,

d = 5.65 m, so $\overline{KB} = 2.83\text{ m}$.

I = 13 337 m^4 and V_{uw} = 4339 m^3, so $\overline{BM} = \dfrac{I}{V_{uw}} = 3.07\text{ m}$.

Then h_m = 1.83 m > 0.5 m, which is sufficient.

For security, the immersing caisson will also be checked for the situation in which it is just above the sill, so d = 6.00 m.

From the force equilibrium it can be derived that F_w now is 3350 kN.

Then the height of the water inside the compartments is $h_W = \dfrac{F_W}{V} = 0.49\text{ m}$, and $\overline{KG} = 3.91\text{ m}$.

$\overline{KB} = 3{,}00\text{ m}$, I = 13 337 m^4 (as before) and V_{uw} = 4608 m^3, so $\overline{BM} = \dfrac{I}{V_{uw}} = 2{,}89\text{ m}$,

so h_m = 1.98 m > 0.5 m, which is still sufficient.

4.4.8 Phase III: just immersed caisson - only ballasted with water

A storm surge is not likely in this phase because transport and immersion should be avoided if the forecasts are that bad. Assume a water height of MHW plus a little extra for safety. Assume NAP + 3.00 m. This water height will occur on both sides of the caisson as long the estuary is not closed off entirely.

Now the buoyant force is F_b = 84 480 kN (d = 11 m). The weight of the ballast should be at least:
$F_{ballast} = F_b - F_{w,\,concrete}$ = 84 480 - 38 910 (see above) = 45 570 kN to accomplish an equilibrium of vertical forces. This corresponds with a water height inside the caisson of

$$\frac{F_{ballast}}{(\ell - 2t_W)(b - 2t_W - t_{bh})\gamma_W} = \frac{45570}{47 \cdot 14.6 \cdot 10} = 6.6\text{ m}.$$

This is a little bit higher than the bulk heads, but that should be no problem if the valves remain open.
This phase is not governing for strength because there is now counter pressure from inside the caisson (by the ballast water).

4.4.9 Phase III: caisson fully ballasted with sand - shear criterion

After the caissons are all placed and completely ballasted with sand, a condition of storm outside and wind setdown inside can occur. For this situation the shear criterion should be checked.

The storm surge level is given: NAP + 4,00 m. Assume a wind set-down inside of 0.50 m. A friction factor f = 0.5 is usual for concrete on stone or sand.

The shear criterion: $\Sigma V > \frac{\Sigma H}{f}$ is more critical if ΣV is smaller, so do not take eventual surcharge (*bovenbelasting*) into account.

The resulting vertical force per caisson is:

$\Sigma V = F_{w,\ concrete} + F_{w,\ ballast} - F_b = F_{w,\ concrete} + (\ell - 2t_w)(b\text{-}2_{tw}\text{-}t_{bh})(h\text{-}t_b)(\gamma_{ballast}) - (b \cdot \ell \cdot p)$

$= 38\,910 + (47 \cdot 14.6 \cdot 13 \cdot 20) - (16 \cdot 48 \cdot ½ \cdot (120 + 75)) = 142\,442$ kN

(where p = average water pressure below the caisson [kN/m^2])

The resulting horizontal force is:

$\Sigma H = F_{h,\ static\ sea\ side} - F_{h,\ static,\ estuary\ side} = 21\,060$ kN (per caisson)

It can now be verified that indeed $\left(\Sigma V = 142\,442\right) > \left(\frac{\Sigma H}{f} = \frac{21\,060}{0.5} = 42\,120\right)$.

4.4.10 Phase III: caisson fully ballasted with sand - turn-over criterion

The turn-over criterion $e_R = \frac{\Sigma M}{\Sigma V} \leq \frac{1}{6}b$ is more critical if ΣM is maximum (so with the highest difference in water levels) and if ΣV is minimum, so also here do not include the surcharge.

It is most convenient to calculate ΣM around the point in the middle, at the bottom of the structure.
ΣM = 3137 kNm/m (this is clockwise) and ΣV = 150 122 / 48 = 3128 kN/m, so indeed

$$\left(e_R = \frac{\Sigma M}{\Sigma V} = \frac{3137}{3128} = 1.00\right) \leq \left(\frac{1}{6}b = \frac{1}{6} \cdot 16 = 2.67\right).$$

4.4.11 Phase III: caisson fully ballasted with sand - bearing capacity subsoil

Obviously the bearing capacity of the subsoil should not be exceeded in this phase. The formula to be used to calculate the maximum acting stress on the soil under the caisson (i.e., the sill) is the following:

$\sigma_{k,\max} = \frac{\sum V}{b \cdot \ell} + \frac{\sum M}{\frac{1}{6}\ell b^2}$, which is more critical if there is a surcharge present. Assume 15 kN/m^2.

Including the surcharge on the caisson, the total vertical load is:

$\Sigma V = 142\,442 + 48 \cdot 16 \cdot 15 = 153\,962$ kN (per caisson).

ΣM does not change now, so ΣM = 150 576 kNm.

$$\sigma_{k,\max} = \frac{\sum V}{b \cdot \ell} + \frac{\sum M}{\frac{1}{6}\ell \cdot b^2} = \frac{153\,962}{16 \cdot 48} + \frac{150\,576}{\frac{1}{6} \cdot 48 \cdot 16^2} = 274 \text{ kN/m}^2$$

The situation, however, differs from the standard load situation described in the Dutch standard TGB1990 (NEN 6744), because of the presence of the sill. The vertical load is assumed to spread through the sill under an angle of 45 degrees. The sliding plane with width W' then develops below the sill in the original estuary bed, like indicated in Figure 4-21.

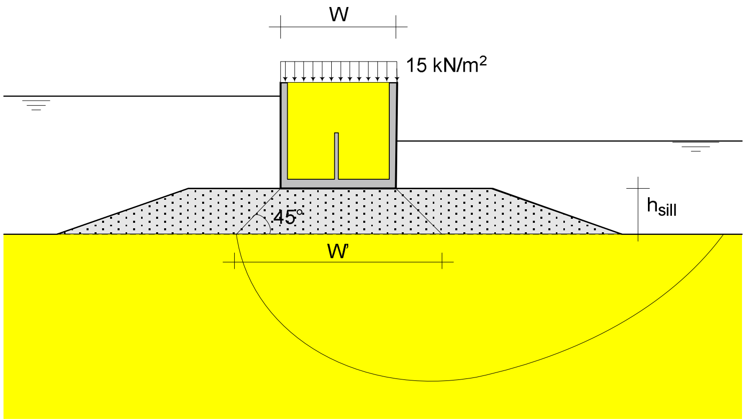

Figure 4-21 assumed sliding plane under the caisson

This implies that the maximum acting stress on the layer below the sill, and not directly below the caisson, should be compared with the bearing capacity. This means that $\sigma_{k,max}$ due to the caisson weight, reduces with a factor *W*/*W* ', but an extra pressure because of the weight of the sill above the area with width W' should also be taken into account: to $\sigma'_{k,\max} = \frac{b}{W'} \cdot \sigma_{k,\max} + \gamma'_{sand} \cdot h_{sill} \cdot 1 \cdot 1$ (for one of the caissons placed in the middle of the row).

The pressure resulting from the weight of the caisson spreads to width *W'* at the original bed level:
$W' = b + h_{sill} \cdot \tan(45°) = 20$ m.

The maximum acting stress at bedlevel then is:

$$\sigma'_{k,\max} = \frac{b}{W'} \cdot \sigma_{k,\max} + \gamma'_{sand} \cdot h_{sill} \cdot 1 \cdot 1 = \frac{16.0}{20} \cdot 274 + 10 \cdot 2.00 \cdot 1 \cdot 1 = 239 \text{ kN/m}^2$$

This resulting maximum effective soil stress (*effectieve korrelspanning*), the soliciting stress, should not exceed the vertical bearing capacity. The resisting bearing capacity shall be calculated according to the theory of Prandtl & Brinch Hansen:

$$p'_{\max} = c' N_c s_c i_c + q' N_q s_q i_q + 0{,}5 \gamma' W' \cdot N_\gamma s_\gamma i_\gamma$$

The cohesion of sand is negligible. The effective soil stress next to the caisson is also assumed to be negligible, because the sill is not present over the entire width of the sliding plane and it would be too favourable if its effect would be taken fully into account. Thus the bearing capacity equation can be simplified to:

$$p'_{\max} = 0{,}5 \gamma' W' \cdot N_\gamma s_\gamma i_\gamma ,$$

where:
relative specific weight $\gamma' = \gamma_s - \gamma_w = 20 - 10 = 10$ kN/m^2.
angle of internal friction $\varphi' = 30°$

factor for surcharge $N_q = \frac{1+\sin\varphi'}{1-\sin\varphi'} \cdot e^{\pi \tan\varphi'} = 18.40\ [-]$

factor for subsoil $N_\gamma = (N_q - 1) \cdot \cot\varphi' = 20.09\ [-]$

shape factor for the foundation $s_\gamma = 1 - 0.3 \cdot \frac{b}{\ell} = 1 - 0.3 \cdot \frac{20}{48} = 0.9$

factor for horizontal load $i_j = \left(1 - \frac{\Sigma H}{\Sigma V + A \cdot c' \cdot \cot\varphi'}\right)^3 = \left(1 - \frac{21\,060}{153\,962 + (16 \cdot 48) \cdot 0 \cdot \cot 30°}\right)^3 = 0.64$

so $p'_{\max} = 0.5 \cdot \gamma' \cdot W' \cdot N_\gamma s_\gamma i_\gamma = 0.5 \cdot 10 \cdot 20 \cdot 20 \cdot 0.9 \cdot 0.64 = 1152 \text{ kN/m}^2$

Thus it appears that ($\sigma_{k,max}$ = 247) < (p'_{max} = 1152 kN/m^2), even in this critical phase.

4.4.12 Phase IV: operational phase
In the operational phase, the caisson is fully filled with sand and completely embedded in a sand body. Although some extra, relatively small loads from trafic are to be expected, this phase appears not to be very critical for any of the design aspects of the caisson. In a later, more detailed design, the actual loads in this phase could be governing for the reinforcement calculation of the concrete.

In this case, the operational phase is not of very big interest, but in a design every phase should be considered to be sure that governing load conditions are not neglected.

4.4.13 Phase V: removal phase
The last phase is the removal phase. It could be very convenient if the caisson could be transported away in floating condition again. This implies that the caisson should remain a watertight box, or at least it should not be too difficult to make it watertight again in this phase. It should also be possible to remove the ballast from the caisson, so sand, instead of concrete for instance, is preferable as ballast material.

After removal, the caisson could be demolished so that the broken concrete can be reused, but more value is gained if it could as a whole be reused in another project.

4.4.14 Conclusion of this example design
According to this first rough hand calculation, a caisson with the following dimensions meets all the considered requirements.

- width b = 16 m
- height h = 14 m
- length ℓ = 48 m
- wall thickness t_w = 0.5 m
- thickness bottom plate t_b = 1.0 m
- wall thickness bulkhead t_{bh} = 0.4 m
- height bulkhead h_{bh} = 6.5 m
- reinforcement percentage: 1.0%

5. Construction of pneumatic caissons

5.1 Preparation

Pneumatic caissons are usually constructed above ground level. Prior to the actual construction, sand bodies are applied upon the ground level and should then be stabilised. The sand bodies form the counter mould (*contramal*) of the work chambers. Formwork is put into the sand body on the place of the future entrance to the work chamber to initiate excavation later on. A blinding (*werkvloer*) with starter bars (*stekeinden*) is cast on top of the sand body. The starter bars should later on connect the fill concrete with the structural concrete of the caisson. They should also prevent that parts of the blinding let off during subsidence.

5.2 Caisson construction

The cutting edges (eventually prefabricated) and the bottom plate are cast as a whole. Together they form the 'work chamber'. After hardening of the work chamber, the walls and eventually the roof can be cast. This can either be done in one cast. Otherwise the inner walls can be cast before the outer walls and the roof depending on the cooling mechanism used. If freshly cast concrete has been cooled, impermissible shrinkage cracks should not occur in structural parts that are cast onto already hardened parts.

To prevent air leakage, joints of timber caissons, which were used in the past, were packed with oakum (*uitgeplozen touw*). Also the joints of former steel caissons, subjected to air pressure, must be well caulked (*waterdicht gemaakt*). Nowadays reinforced concrete is commonly used for caissons because it is considered to be the best material for caissons and also to be the most economical. Reinforced concrete cassions, however, also need precautions because the material frequently is not proof against loss of air pressure through the concrete. This can be accomplished by painting the inner surfaces with bituminous or other suitable paint. Special attention should be paid to joints, the number of which should be limited as much as possible.

5.3 Subsidence

When the caisson itself is ready, soil can be dug from within this work chamber which causes the caisson to subside into the ground. Nowadays the soil is often removed after it has been mixed with water. In this way it can be removed by suction through a high-pressure tube. Excavation was traditionally done by hand, but nowadays preferably by water canons or sand pumps. Sometimes the air pressure can be used to blow out mud and wet soil through a pipe. The excavated soil is removed by a hydraulic transport system. There should be a basin close to the site where the removed sand-water mixture can be deposited.

The caisson is subsided under its weight. This is usually done by gradually building up, which is especially favourable if there will be ballast on top of the caisson anyway in its final stage (like a pier, for example) (see Figure 5-1). Ballast can aternatively be applied inside the caisson.

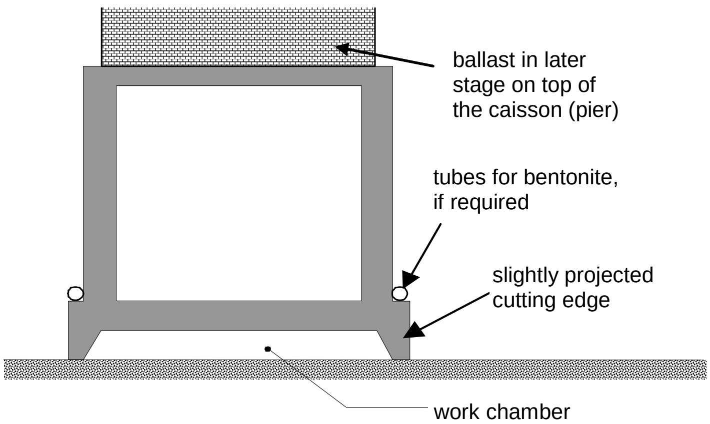

Figure 5-1 Measures to improve subsidence of a pneumatic caisson

It should be given thought that a heavy dead weight can cause a caisson to subside even before excavation starts, leading to fill of the working chamber (and possibly even the shafts) with soil. If this will happen, bothersome delay is inevitable. It should for this reason be avoided that the caisson is too heavy.

The air in the work chamber has to be kept under compression to prevent ground water from intruding. Compressed air, on the other hand, also functions as an air bag that lifts the caisson. By varying the air pressure, the pressure on the cutting edges can be regulated in order to influence the subsidence and position of the caisson. The pressure in the working chamber is adjusted to balance, or slightly exceed the water pressure at the depth of the cutting edge. To boost sinking, the air pressure is often reduced more considerably when worksmen are out of the working chamber. The cutting edge must be sufficiently below the inside soil surface to prevent serious loss of air through the soil. The height of the cutting edges can therefore measure up to two metres. If air pressure and excavation near the cutting edges do not sufficiently regulate the subsiding process, a sliding plane of bentonite can be applied between the caisson wall and soil. An other measure to better control the subsidence process is to project the edges of the work chamber slightly outside the profiles of the structure (this should be considered in advance).

Excavation generally occurs aloof of the cutting edges to leave them embedded in soil and thus avoid loss of air. If, however, the subsidence needs correction because the caisson is not kept level, excavation should occur close to the cutting edges where greater subsidence is desired. Eventual obstacles that come across during excavation, like remainders of old structures, can be removed through the air lock. Large obstacles like boulders (*zwerfkeien*) can be subsided along with the caisson by washing away the soil beneath them.

When the caisson has reached its final depth, it is generally sealed with concrete, which means that the work chamber is completely filled up or injected completely with concrete. To prevent the caisson from sinking lower than the desired level before the concrete seal is fulfilling function, the caisson can be propped (*gesteund*) well from the soil up to the roof. In that case, the caisson roof has to be sufficiently strong. Otherwise, the props will need to be placed under the ribs in the roof. In some cases the caisson is subsided onto previously applied foundation piles to assure fixation.

The subsequent stages of pneumatic caisson subsidence are depicted in Figure 5-2. This example comes from the steel caisson which has been used for the river pier of Lambeth Bridge in London.

The application of pneumatic is limited to depths up to about 30 m below the free-water surface, because of the high water pressure at these depths. This limitation may sometimes be overcome where it is possible to lower the groudwater level by pumping. This can only be done in case of suitable surface and subsoil conditions. Lowering of the groundwater level can even be accomplished in river or estuary beds if the caisson is surrounded by a cofferdam and an impermeable layer separates the river bed from the artesian water.

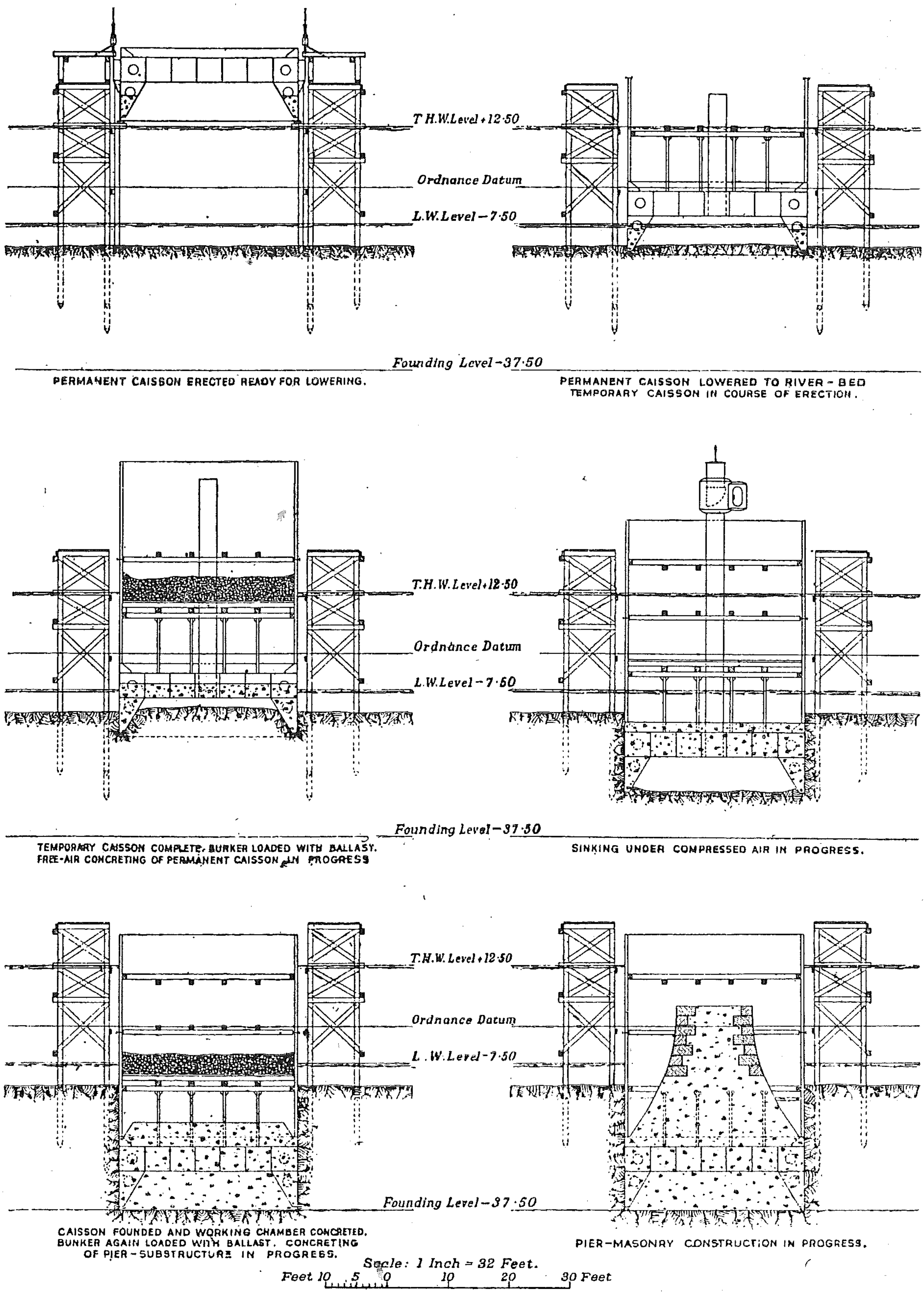

Figure 5-2 Stages in subsidence of a pneumatic caisson

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Appendix 1 Design Method

Design may be complicated due to the many issues, see Figure 7-1, to be dealt with and due the cyclic nature of the process, see Figure 7-3. The cyclicity alone is source of chaos during design; reason why efforts are made to structure the design process, see Figure 7-2.

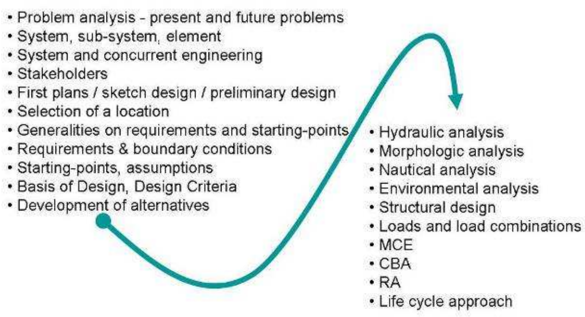


Figure 7-1 Issues to be dealt with during design

But how to obtain the best design? It has to be kept in mind that a structured design process resulting in a “relative best” solution is by no means a guarantee for obtaining the “absolute best” solution or design. The best solutions are often found after a flash of inspiration followed by hours of trans-piration (Free interpretation of Albert Einstein’s original quote).
How to provoke this flash of inspiration? Isn’t this creative phenomenon completely in contradiction with the well argued structured approach? Isn’t creativity synonymous to chaos? Yes and no, the designer, certainly the ambitious one, has to deal with it, has to be able to switch between the creative mind for production of alternatives and the methodical approach for selection etc. in order to keep the design process going in the right direction.

General scheme for structured design approach:

To achieve convergence of the design process a structured design approach should be used. An example of a structured design process is shown in Figure 7-2. Before the analysis phase generally an initial definition of the problem and an initial set of objectives have been produced. In the analysis phase, the project or structure is analyzed on:

- Functions (*Functieanalyse)*
- Operational aspects *(Procesanalyse)*
- Requirements or boundary conditions *(Randvoorwaarden)*
- Starting-points or Assumptions *(Uitgangs-punten)*

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

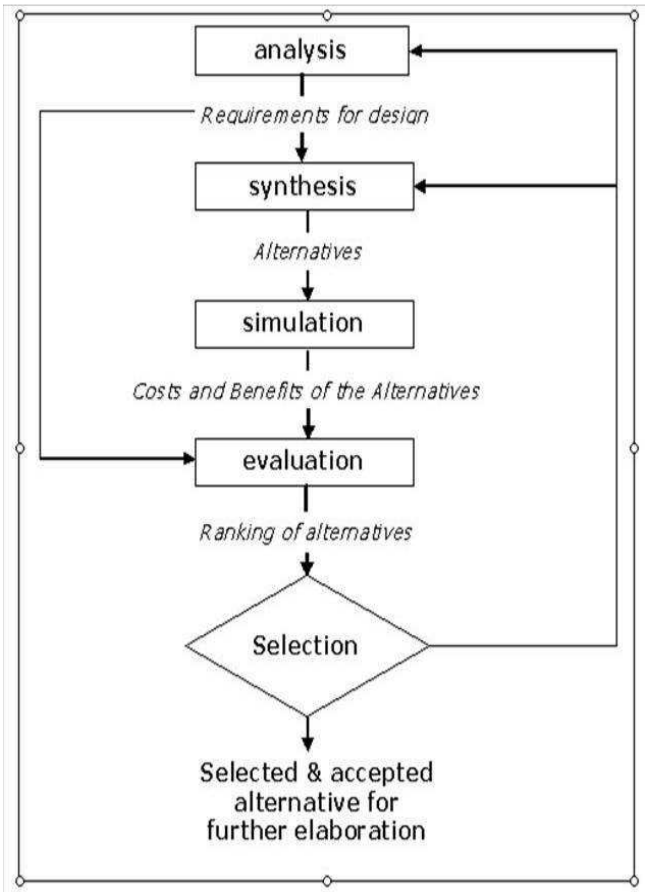


Figure 7-2 Flow chart structured design approach

Usually the analysis phase results in a List of Requirements or Specifications. In Dutch: “Programma van Eisen (PvE)”; in Anglo-Saxon literature “Basis of Design” is an expression frequently used. “Terms of Reference (ToR)” is often found in tender or contract documents. The left-hand-side of Figure 7-3 shows a number of contributive sources to the Basis of Design, depending on the project there may be less or many more contributors.

After development of alternatives (synthesis), based on the Program of Requirements and elaboration (simulation) up to level where meaningful comparison is possible (evaluation), one alternative or solution will be selected for detail design. All going well, the process converges into a solution or a design, see the right-hand-side of Figure 7-3. Note that further elaboration of alternatives, especially the feedback generated by evaluations or selection procedures, is an important contributor to further development of the Basis of Design.

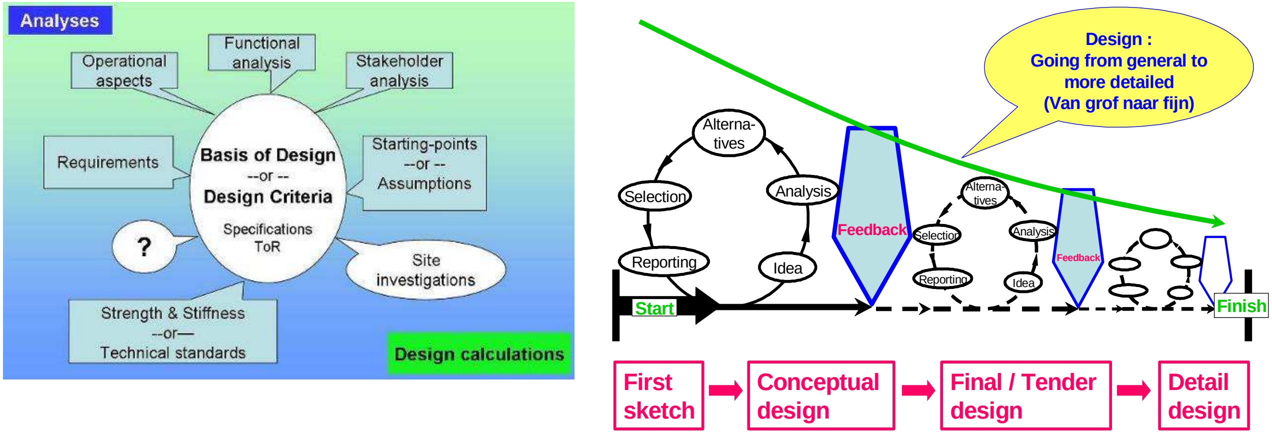


Figure 7-3 Contributions to the Basis of Design (left); Cyclic nature of design (right)

When design has progressed into more final conceptual design or design for tender, has moved out of idea or feasibility stages, making engineering or design calculations becomes the more dominant, more time consuming activity.

Appendix 2 Water level rise due to astronomical tide and wind set-up

FROM: Manual Hydraulic Structures CT3330

In shallow seas, deltas, closed off creeks, and lakes, wind fields can influence the water level quite considerably by damming up the water (wind set-up). Figure 7-4 shows a model to approximate the wind set-up.

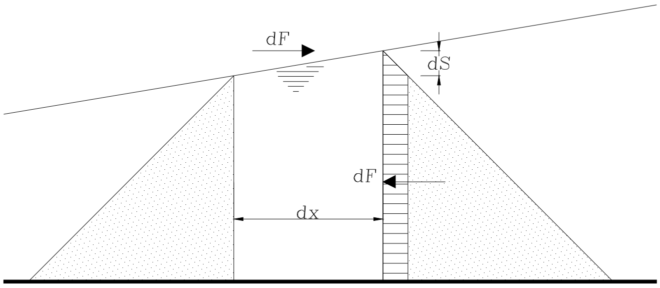


Figure 7-4 Balance of forces in case of wind set-up

The wind set-up in the equilibrium state is approximately:

$$\frac{dS}{dx} = C_2 \frac{u^2}{gd}$$

in which: S = total wind set-up [m]
C_2 = constant ≈ $3{,}5 \cdot 10^{-6}$ to $4{,}0 \cdot 10^{-6}$ [-]
d = water depth [m]
u = wind velocity [m/s]

If the wind set-up is small compared with the water depth, in an area with a horizontal bed, the slope $\frac{dS}{dx}$ is constant.
The formula shows that the wind set-up increases with increasing wind velocity and fetch and decreasing water depth. The wind set-up is therefore of importance in river deltas, lakes and shallow seas. In coastal areas where the sea is deep, wind set-up hardly ever occurs. In the North Sea, the Wadden Sea and the IJsselmeer, the set-up can be as much as a couple of meters. In 1953 the rise in Vlissingen was 3.05 m.

If the water level at the edge of a basin is known, the course of the water level in the basin can be calculated with a simple numerical solution (Heun method):

$$d_x = d_{x-\Delta x} + \frac{dS}{dx}\Delta x - \Delta z_{x-\Delta x,x}$$

in which: d_x = the water depth in point x [m]
$d_{x-\Delta x}$ = the water depth in point x-Δx [m]
$\Delta z_{x-\Delta x,x}$ = difference between the height of the bed in x and x-Δx [m]

In a closed basin or a lake, the total amount of water cannot change. This means that, provided the slope may be assumed constant, the surface of the water (by approximation) will tilt around the gravity line of the basin surface, perpendicular to the wind direction. The water in the area away from the wind is subjected to wind set-down (see Figure 7-5).

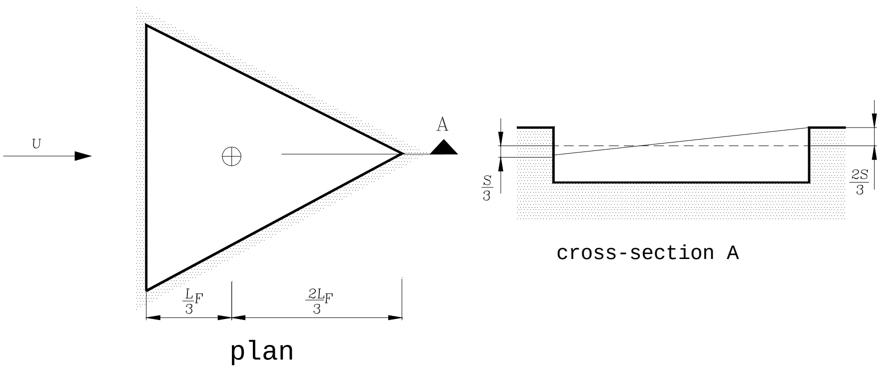


Figure 7-5 Wind set-up in a closed basin

The combination of tides and wind set-up is important, particularly for the design of most hydraulic engineering structures. If high tide coincides with strong winds (storm) a storm surge occurs.

It is more or less possible to predict the timing and the water levels of a spring tide. The wind set-up in front of the coast can be calculated with a given wind velocity. By analysing wind data, it is possible to calculate the probability of occurrence of a spring tide and a certain wind set-up. Such a combination is known as a storm surge.

Because the calculation model for wind set-up contains a number of uncertainties, a different approach is used in practice. Using as many observed storm surge levels as possible, a frequency distribution (an empirical relation) is made. The Gumbel distribution and the exponential distribution are commonly used frequency distributions. By extrapolation, the design storm surge level can be determined, for a previously selected small exceedance frequency. See the high water level measurements for Hoek van Holland in the following figure.

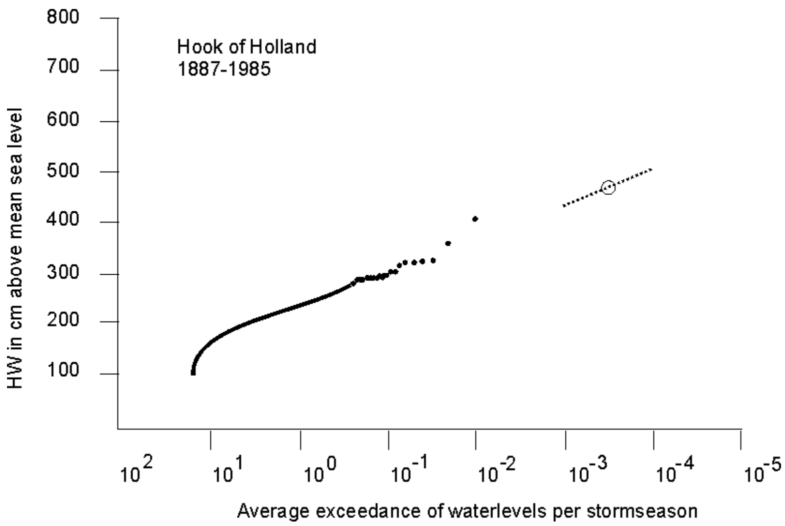


Figure 7-6 Probability distribution high water level Hoek van Holland

HW in this figure includes two phenomena: tide + wind set-up. For some hydraulic engineering structures, the water level during an extreme storm surge, based on a probability of exceedance of once per 10 000 years, is normative. The table below gives water levels corresponding to certain probabilities of exceedance for a number of places (see Tidal Tables for the Netherlands, "Getijtafels voor Nederland", RIKZ).

HW/year	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 7-1 (Extreme) storm surge levels

For foreign projects, however, there are often too few data to determine the design storm surge level on a statistical basis. In such cases the models for the tide and the wind set-up have to be used to estimate the storm surge level.

Appendix 3 Determination of the design wave height

FROM: Manual Hydraulic Structures CT3330

The most important waves are waves generated by the wind. If a structure is to be dimensioned for a certain wave height, this wave height has to be known (measurements). If no measurements are available, the significant wave height and wave period can be estimated using the methods of Bretschneider or Groen and Dorrestein.

When wind waves approach the coast, a number of changes occur, caused by the change of water depth. Due to the smaller depth, the wave velocity decreases and the wave front turns so it runs increasingly parallel to the depth contours (refraction). As a result, the wave crests become narrower, the wave becomes more concentrated and the wave height increases. At the same time, the wave velocity decreases, thereby reducing the wavelength, causing a further increase of the wave height (shoaling). So, the wave height increases and the wave length reduces. At a certain point, the waves are so steep that they break. This section considers these three points:

1. Refraction
2. Shoaling
3. Breaking of waves

Besides that, the effect of an obstacle is also discussed. The two most well known consequences are:

1. Diffraction
2. Reflection

Unless stated otherwise, regular waves are assumed in all these points.

A3.1 Design height of wind waves

The most important waves are waves generated by the wind. If a structure is to be dimensioned for a certain wave height, this wave height has to be known (measurements). If no measurements are available, the significant wave height and wave period can be estimated using the Bretschneider method:

$$\tilde{H} = 0.283\tanh(0.53\tilde{d}^{0.75})\tanh\left(\frac{0.0125\tilde{F}^{0.42}}{\tanh(0.53\tilde{d}^{0.75})}\right)$$

$$\tilde{T} = 7.54\tanh(0.833\tilde{d}^{0.375})\tanh\left(\frac{0.077\tilde{F}^{0.25}}{\tanh(0.833\tilde{d}^{0.375})}\right)$$

in which:

$\tilde{H} = \frac{gH_s}{U^2}$

$\tilde{T} = \frac{gT_p}{U}$

$\tilde{F} = \frac{gF}{U^2}$

F = fetch [m]

U = wind velocity at an altitude of 10 m [m/s]

$\tilde{d} = \frac{gd}{U^2}$

d = water depth [m]

T_p = peak wave period (= most common period) [s]

After some time, the significant wind wave thus depends on the wind velocity *u*, the water depth *d* and the fetch *F*. The formulae above also apply to deep (sea)water. The following then applies:

$$\lim_{d\to\infty}(\tanh(0.53\tilde{d}^{0.75})) = 1$$

Besides the Bretschneider formula given above, the Groen and Dorrestein nomograms are also very well known. These nomograms are shown in Figure 7-7 and Figure 7-8. One must use these nomograms cautiously; neither is dimensionless. Because Groen and Dorrestein used different data sets than

Bretschneider, their findings are not identical.

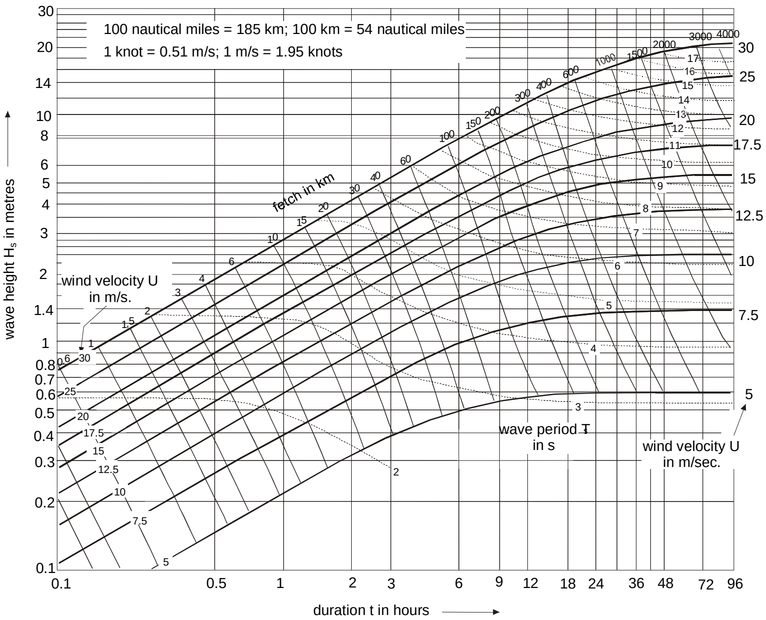


Figure 7-7 Nomogram for deep water

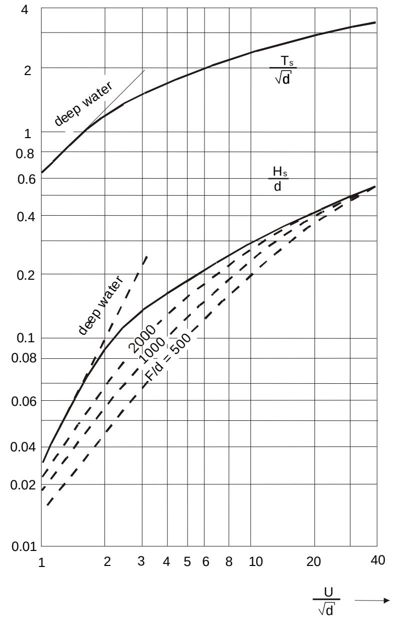


Figure 7-8 Nomogram for shallow and transitional water

The significant wave height H_s is the average of the highest 1/3 of the waves. This wave occurs regularly and is therefore a lot lower than the design wave height H_d. If the effects of shallow water can be disregarded with a small wave height, one can assume a Rayleigh distribution. The probability of exceedance of a given wave height within a given wave field is:

$$\Pr(H > x) = e^{-2\left(\frac{x}{H_s}\right)^2}$$

Therefore, the probability that the design wave height H_d is exceeded during a storm with N waves is:

$$\Pr(H > H_d) = 1 - e^{-N \cdot e^{-2(H_d/H_s)^2}}$$

For a storm along the coast one can assume $T_{storm} = 2$ h. For rivers and the IJsselmeer $T_{storm} = 4$ h can be supposed. Presuming $T_{wave} = 3$ sec, the number of waves N along the coast is:

$$N = {T_{storm}}/{T_{wave}} \approx 2400$$

If one allows an exceedance probability $\Pr(H > H_d) = 0.10$, the design wave height H_d is:

$$H_d = 2.25 H_s$$

To ascertain the design wavelength one may assume that the shape of the energy spectrum essentially does not change for light and heavy storms, so:

$$L_d \approx L_s$$

A3.2 Refraction

If a wave approaches a sloping coastline at an angle, the propagation velocity will vary along the wave crest due to the difference in water depth along the wave crest. After all:

$$c = \sqrt{\frac{g}{k}\tanh(kd)} = c_0 \cdot \tanh(kd)$$

In shallow water the propagation velocity is smaller. Therefore, with decreasing depth, the wavelength shortens. The wave front decelerates in the first part to reach shallow water. The wave front will thus turn. This causes a bend of the propagation velocity towards the coast. This phenomenon is known as refraction.

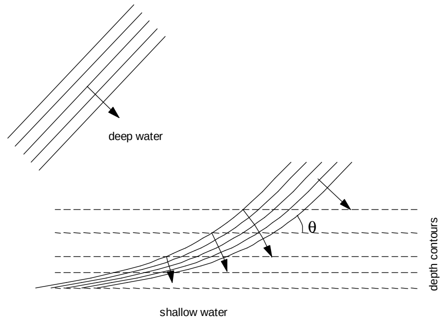


Figure 7-9 Refraction

In the case of a coast with straight parallel depth contours, the angle between the wave crest and a contour line can be derived directly from the local depth and the angle between the wave crests and a parallel contour line in deep water (see Figure 7-9). This relation is called Snell's law and reads as follows:

$$\frac{\sin(\theta_1)}{\sin(\theta_o)} = \tanh(k_1 d_1) \qquad \text{for:} \qquad \frac{\sin(\theta_1)}{\sin(\theta_o)} = \frac{c_1}{c_0} = \frac{c_0 \tanh(k_1 d_1)}{c_0} = \tanh(k_1 d_1) = \frac{L_1}{L_0}$$

in which: θ_o = the angle between the wave crest and the contour line in deep sea

θ_1 = the angle between the wave crest and the contour line in shallow water

If one knows the angle of the wave ray, the wave height and wavelength in shallow water, one can calculate the wave ray angle in deep water using the above equation or the dotted line in Figure 7-10.

If the angle of the wave ray, the wave height and the wavelength in deep water are known, the angle of the wave ray in shallow water cannot be calculated using the equation above. The equation requires the wavelength in shallow water, for which the following applies:

$$L_1 = L_0 \tanh(k_1 d_1) \quad \text{met:} \quad k_1 = \frac{2\pi}{L_1}$$

So $\frac{\sin(\theta_1)}{\sin(\theta_o)}$ as a function of d/L_0 is an implicit function, which has to be solved iteratively. The inverse solution is given as a solid line in the graph below, plotted against the relative water depth d/L_0 (water depth d in shallow water, divided by the wavelength L_0 in deep water).

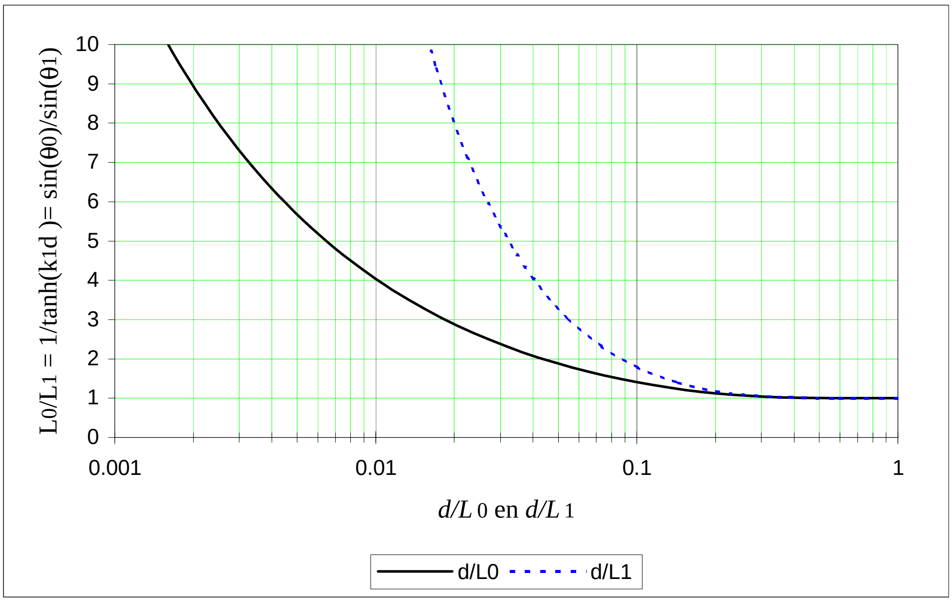


Figure 7-10 Relation between depth and wavelength
(d = depth, L_1 = wavelength at considered depth, L_o = wavelength in deep water)

Depending on the concentration or spread of wave rays, the wave height will increase or decrease. Generally, for the wave height in shallow water:

$$H = K_s K_r H_o$$

in which: K_r = the refraction coefficient [-]
K_s = the shoaling coefficient (see next section) [-]

In the case of straight parallel depth contours, the wave height decreases with a factor:

$$K_r = \sqrt{\frac{b_0}{b_1}} = \sqrt{\frac{\cos(\theta_o)}{\cos(\theta_1)}}$$

with: b = the wave crest width
This is because the wave crest width b continues to increase while the wave crest turns, which causes a reduction of the energy density and thus also of the wave height. The change of wavelength in shallow water also leads to a change of the energy density and the wave height, but that phenomenon is called shoaling and is covered in the next section.

Refraction also occurs when a wave enters an area with a current (along the coast). In this case, the wave will turn more or less in the direction of the current.

A3.3 Shoaling

When the water depth decreases, the propagation velocity and the wavelength are reduced with a constant period. This influences the wave height.

The wave energy per unit of surface area equals:

$$E = \frac{1}{8}\rho g H^2$$

The group velocity is:

$$c_g = nc \qquad \text{with} \quad n = \frac{1}{2} + \frac{kd}{\sinh(2kd)} \qquad \text{and} \quad c = \sqrt{\frac{g}{k}\tanh(kd)} = c_0 \cdot \tanh(kd)$$

The energy flux is the amount of energy that passes a certain point per unit of width. This energy flux equals:

$$F = E \cdot c_g = \text{constant}$$

and is constant for non-breaking waves (no loss of energy) and straight approaching waves (no change of width).

The wave height is therefore:

$$H^2 = \frac{\text{constant}}{\text{n} \cdot \text{c}}$$

The wave height in a shallow area, H_1, is therefore dependent on the wave height in deep water, H_0 , according to:

$$\frac{H_1}{H_0} = K_s = \sqrt{\frac{c_{g;0}}{c_{g;1}}} = \sqrt{\frac{c_0 n_0}{c_1 n_1}} = \sqrt{\frac{\frac{1}{2}}{\tanh(kd)n_1}} = \frac{1}{\sqrt{\tanh(kd)\left(1 + \frac{2kd}{\sinh(2kd)}\right)}}$$

The shoaling coefficient is therefore a function of the wave number k and the water depth d:

$$K_s = \frac{1}{\sqrt{\tanh(kd)\left(1 + \frac{2kd}{\sinh(2kd)}\right)}} \qquad \text{with:} \quad kd = 2\pi\frac{d}{L}$$

This solution is represented by the dotted line in Figure 7-11. If the wave height and wavelength are known in a certain shallow area, these can be used to calculate the wave height in deep water. The inverse, using this solution and a known wave height and a known wave length in deep water to calculate the wave height in shallow water is not possible. The problem is that L and thus also k are dependent on the depth d and on themselves, for:

$$L = L_o \tanh(kd) = L_o \tanh(\frac{2\pi d}{L})$$

This is an implicit function. The shoaling coefficient, a function of the water depth d and wavelength in deep water L_0, can therefore only be solved iteratively. This solution is represented by the solid line in Figure 7-11.

Besides shoaling, refraction also influences the wave height, for this see the previous section.

The wave height H_1 of regular (non–breaking) waves in shallow water depends on the wave height in deep water H_0, the refraction coefficient K_r and on the shoaling coefficient K_s, according to:

$$H_1 = K_r K_s H_o$$

In Figure 7-11 the shoaling coefficient is given as a function of the relative water depth (water depth d in shallow water, divided by the wavelength L_0 in deep water).

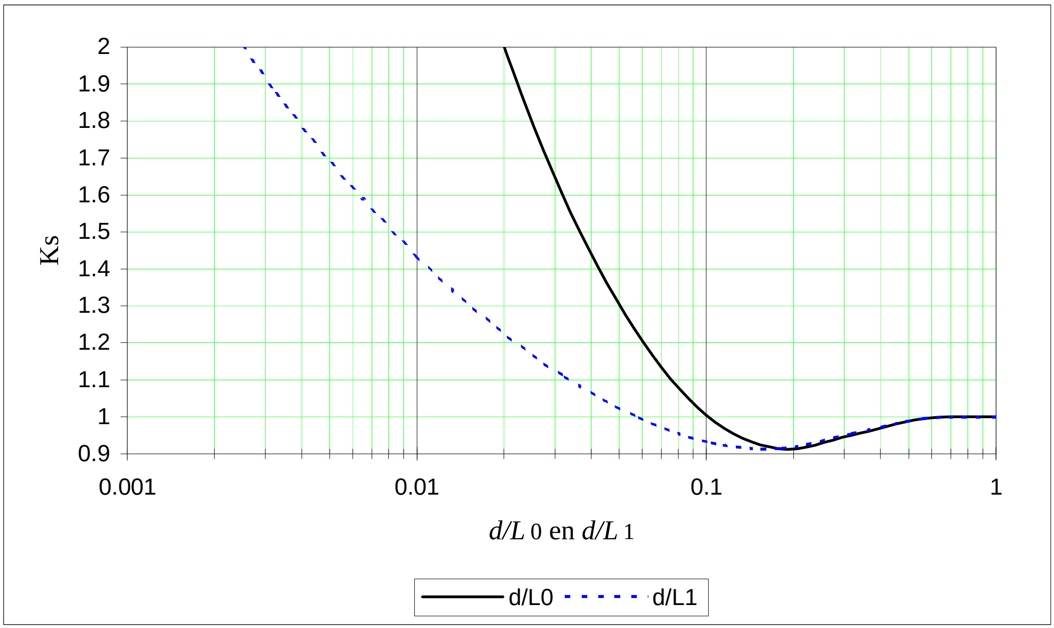


Figure 7-11 Relation between K_s and d/L (L = wavelength at considered depth, L_o = wavelength at deep sea)

For the refraction parameter:

$$K_r = \sqrt{\frac{\cos(\theta_o)}{\cos(\theta_1)}}$$

has been found. To calculate the refraction ($\theta_0 \Rightarrow \theta_1$), see previous section.

A3.4 Breaking

Due to the decreasing wavelength and the increasing wave height in shallow areas, the steepness of the wave increases. Theoretically the wave breaks at a steepness of H/L = 1/7. The depth also limits the wave height. It has been theoretically deduced that an individual wave will break when $\frac{H}{d} \geq 0.78$ (there are also more complex formulas e.g. by Miche). However, individual waves with a ratio of $\frac{H}{d} = 1.2$ have been observed.

When calculating breaking for a wave spectrum:

$$\frac{H_s}{d} = 0.4 \sim 0.5$$

is often used. The way in which a wave breaks on a smooth slope with a constant slope angle depends on the steepness of the wave and the slope of the bed. This is characterised in the breaker parameter (Iribarren):

$$\xi = \frac{\tan\alpha}{\sqrt{H_s/L_o}}$$

in which: α = angle of the slope [º]
H_s = significant wave height [m]
L_o = wavelength in deep water [m]

Depending on the value of the breaker parameter, different types of breaking occur.

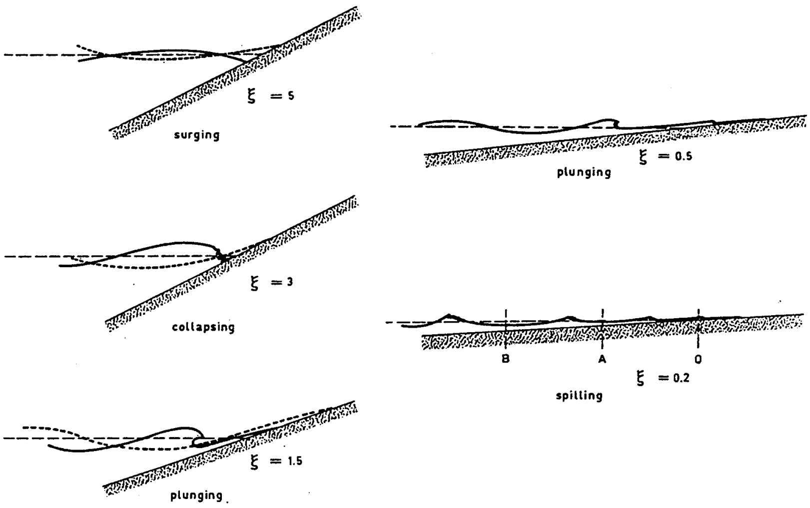


Figure 7-12 Types of breaking

Notes:

- The depth profile need not be constant in time. Shallows can appear or disappear. It is therefore important to find out if the bed consists of rock or sand.
- The depth can depend on the tide and on the wind set-up (storm surge).
- Changes of depth also mean changes of refraction, shoaling and breaking.

A3.5 Reflection

If waves run into a structure they can break or reflect. The reflection can be partial or complete. Characteristics of completely reflected waves are:

- the energy of the reflected wave equals the energy of the incoming wave
- the period of the reflected wave equals the period of the incoming wave
- the reflected wave is in phase with the incoming wave.

The consequence of the above is that a standing wave with (in case of complete reflection) a wave height twice the size of an incoming wave is created in front of the structure. If the reflection is partial, the wave height of the standing wave will be less. In general the following applies:

$H = (1+\chi)H_i$

in which: H = the wave height of the standing wave
H_i = the wave height of the incoming wave
χ = the reflection coefficient ≤ 1

The value of χ depends on the permeability, roughness and slope of the structure and on the steepness of the incoming waves and the water depth in front of the structure.

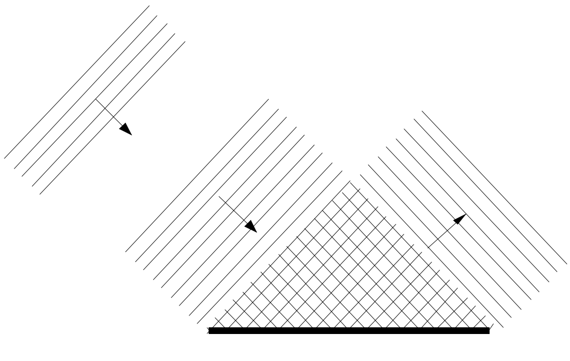


Figure 7-13 Reflection

A3.6 Diffraction

If there's an obstacle in the course of a wave (e.g. a breakwater or an island), wave motion still occurs in the shadow zone behind the obstacle. The transfer of energy apparently not only takes place in the wave direction. The wave crests bend round the object shaped like circular arcs. This phenomenon is called diffraction.

The wave height changes due to diffraction, whereby the wave height on the lee side of the object is smaller than that of the incoming wave, whilst the wave height next to the object is often larger than that of the incoming wave.
Generally:

$H = K_d H_o$

where: K_d = the diffraction coefficient

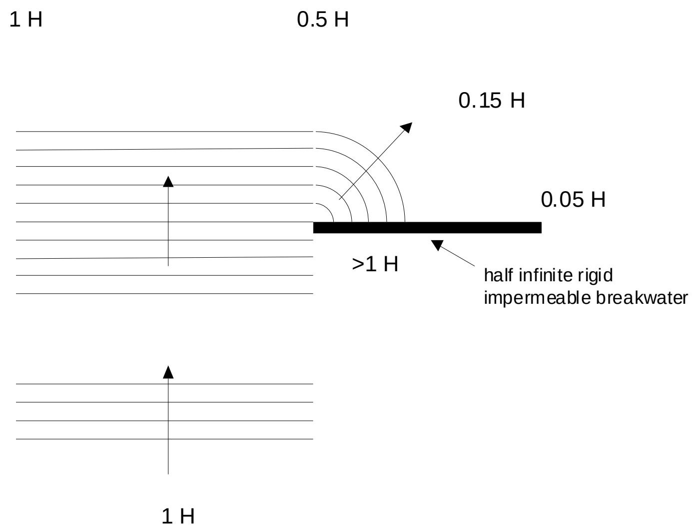


Figure 7-14 Diffraction

The "Shore Protection Manual" (CERC 1984) gives a large number of diagrams for the estimation of K_d with different wave directions. The Shore Protection Manual can be found on the internet with search request 'coastal engineering manual'. Figure 7-15 shows the diagram for waves moving straight towards a breakwater. The distance between the wave lines equals the wavelength L.

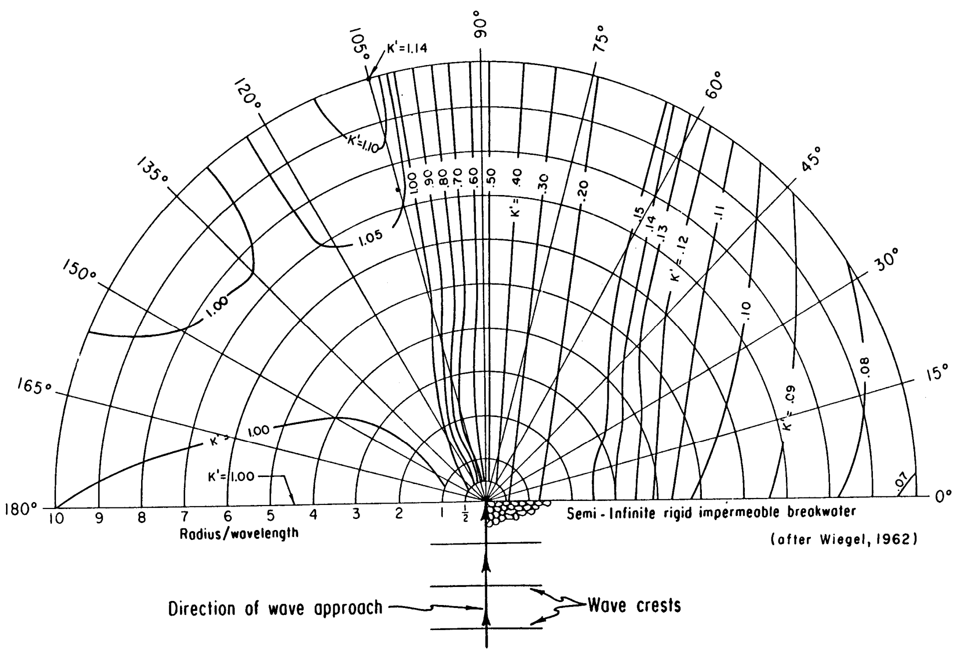


Figure 7-15 Diffraction coefficients for straight incoming waves

Appendix 4 Overtopping

FROM: Manual Hydraulic Structures CT3330

Overtopping waves can jeopardise a civil engineering work if they cause erosion or softening of the foundations. Wave overtopping can also cause a nuisance to the surroundings. To prevent severe wave overtopping, the design of the structure should therefore include a sufficient freeboard above the design water level.

For wave overtopping considerations, the wave run-up R_n simply exceeds the crest height z.
The wave overtopping is usually characterised by an overtopping discharge q per metre of the water defence, averaged over time. This discharge depends on the wave height, the wave steepness, the slope and the existing freeboard. This concerns overtopping discharges averaged over time. In reality, a far larger discharge can occur for a short time, depending on the percentage of overtopping waves. With the results of several investigations, one can derive a global relationship between all of these factors. One distinguishes between overtopping discharges for vertical walls and for sloping walls.

European Overtopping Manual

In August 2007, the EurOtop team released the "Wave Overtopping of Sea Defences and Related Structures: Assessment Manual", in short: "European Overtopping Manual" (EurOtop 2007). That manual gives "guidance on analysis and/or prediction of wave overtopping for flood defences attacked by wave action." It replaces the older Dutch "Technical Report Wave Run-up and Wave Overtopping at Dikes" of
TAW, Technical Advisory Committee on Flood Defences, author: J.W. van der Meer, and two other (foreign) reports.

The Overtopping Manual gives design rules for overtopping:

Hazard type and reason	**Mean discharge** q (l/s/m)
Embankment seawalls / sea dikes	
No damage if crest and rear slope are well protected	50-200
No damage to crest and rear face of grass covered embankment of clay	1-10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

Table 7-2 Limits for overtopping for damage to the defence crest or rear slope [European Overtopping Manual 2007]

The principal equation used for wave overtopping is:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = a \cdot e^{\left(\frac{-b\, R_c}{H_{m0}}\right)}$$

where:

$\frac{q}{\sqrt{g \cdot H_{m0}^3}}$ = dimensionless overtopping discharge [-]
q = overtopping discharge [m^3/s/m]
$\frac{R_c}{H_{m0}}$ = the relative crest freeboard [-]
R_c = crest height [m]
H_{m0} = estimate of significant wave height from spectral analysis = $4\sqrt{m_0} \approx H_s$ [m]

See Figure 7-16 for a definition sketch.

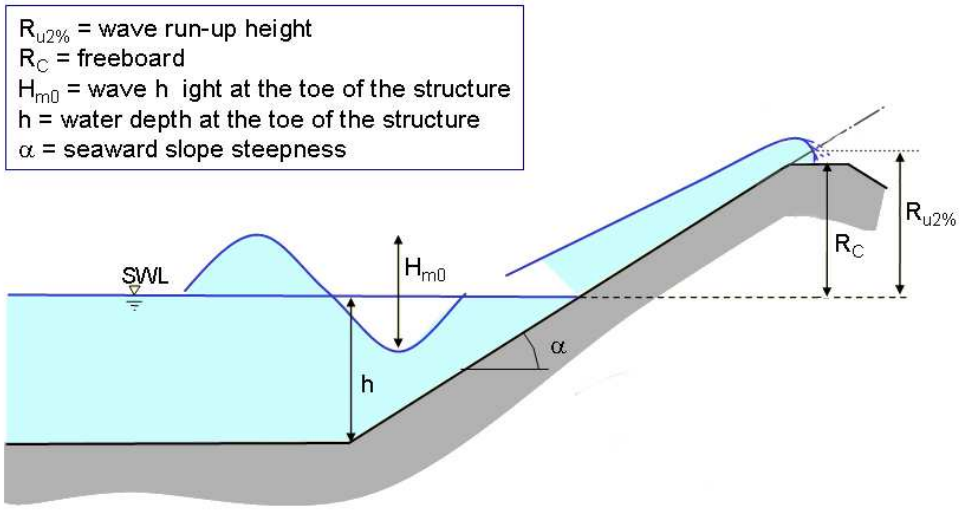


*Figure 7-16 Definition of some parameters for the calculation of overtopping [*European Overtopping Manual 2007*]*

Additional required parameters:

$$a = \frac{0{,}067}{\sqrt{\tan\alpha}}\gamma_b \cdot \xi_{m\text{-}1{,}0} \qquad [-]$$

$$b = \frac{4{,}3}{\xi_{m\text{-}1{,}0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \qquad [-]$$

$\xi_{m\text{-}1,0}$ = breaker parameter (see below) [-]
γ_b = correction factor for the presence of a berm (see below) [-]
γ_f = correction factor for the permeability and roughness of the slope, sometimes written as γ_R (see below) [-]
γ_β = correction factor for oblique wave attack [-]

$$\gamma_\beta = 1 - 0{,}0033|\beta| \quad for: 0° \le \beta \le 80°$$

$$\gamma_\beta = 0{,}736 \quad for: \beta > 80°$$

Note: γ_β has different values for run-up and overtopping calculations!
γ_v = correction factor for a vertical wall on top of the crest (see below) [-]

The breaker parameter, also referred to as surf similarity or Iribarren number, is defined as:

$$\xi_{m\text{-}1{,}0} = \frac{\tan\alpha}{\sqrt{H_{m0}/L_{m\text{-}1{,}0}}}$$

where tan α is the slope of the front face of the structure and $L_{m\text{-}1,0}$ being the deep water wave length:

$$L_{m\text{-}1{,}0} = \frac{g \cdot T^2_{m\text{-}1{,}0}}{2 \cdot \pi} \quad [m]$$

In another shape, the overtopping equation is: $q = a \cdot e^{\left(\frac{-b\, R_c}{H_{m0}}\right)} \cdot \sqrt{g \cdot H^3_{m0}}$.

The European Overtopping Manual gives a maximum of $q_{\max} = 0{,}2 \cdot e^{\left(\frac{-2{,}3 \cdot R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)} \cdot \sqrt{g \cdot H^3_{m0}}$.

The wave steepness s_0 (= H_{m0}/ $L_{m\text{-}1,0}$) varies from 0.04 (steep storm waves) to 0.01 (long waves due to swell or wave breaking).

Berm influence γ_b
A berm reduces the wave run-up. The reduction factor γ_b (a.k.a. 'shoulder reduction') is dependent on the length of the berm B_b and the water level.

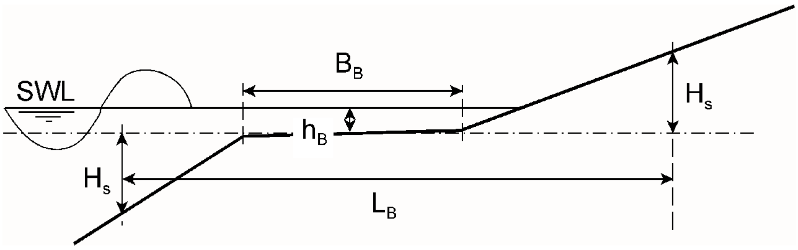


Figure 7-17 Definitions berm reduction

At both sides of the berm, the slope is intersected at a vertical distance H_S from the horizontal centre plane of the berm, giving a length L_B. h_B is the distance between SWL and the berm level (can be negative or positive). γ_b finally becomes:

$$\gamma_b = 1 - \frac{B_B}{L_B}\left[0.5 + 0.5\cos\left(\pi\frac{h_B}{x}\right)\right]$$
$$x = z_{2\%} \quad \textit{for } z_{2\%} > -h_B > 0 \quad (\textit{berm above SWL})$$
$$x = 2H_S \quad \textit{for } 2H_S > h_B \geq 0 \quad (\textit{berm below SWL})$$

With limits: $0,6 \cdot \gamma_b < 1$. The equation above shows that a berm on SWL is most efficient. For more information, see TAW (2002)

Roughness γ_f

The reduction factor γ_f that takes the roughness and the permeability of the surface into account is:

- 1.00 for asphalt, concrete with a smooth surface
- 0.95 for concrete blocks, block mats
- 0.70 for gravel, gabions
- 0.60 for quarry stone (rip-rap)
- 0.50 for cubes (random positioning)
- < 0.50 for X-blocs, tetrapods, dolosses (see European Overtopping Manual for more data)

Influence of vertical walls (γ_v)

The reduction due to (relatively small) vertical walls on top of the slope can mount up to 35%. This topic has to be studied further, but based on experience up to now the next equation could be applied:

$$\gamma_v = 1.35 - 0.0078 \cdot \alpha_{wall}$$

where α_{wall} is the slope of the wall (for 100% vertical walls, $\alpha_{wall} = 90^o$).

The European Overtopping Manual restricts the equation above with the following conditions:

- the average slope of 1.5 H_{m0} below the still water line to the foot of the wall (excluding a berm) must lie between 1:2.5 to 1:3.5.
- the width of all berms together must be no more than 3 H_{m0}.
- the foot of the wall must lie between about 1.2 H_{m0} under and above the still water line.
- the minimum height of the wall (for a high foot) is about 0.5 H_{m0}. The maximum height (for a low foot) is about 3 H_{m0}.

Appendix 5 Alternative caisson width calculation

FROM: General Lecture Notes Hydraulic Structures CT3330

If there is enough clearance (buoyancy) along the entire transport route, still a certain minimum freeboard should be maintained. In that case, the starting point for determining the caisson or tunnel width is the necessary profile of free space that has a surface area of H m^2 (Figure 7-18). Around this is the reinforced concrete structure (walls, floor a roof; with bigger tunnels and also often, the partitioning walls, not shown here) with a surface area of C m^2. The section is floated to the site and then immersed with the aid of water that is admitted into the ballast tanks inside the tunnel section. After the tunnel has been founded on the layer of fill sand the layer the temporary ballast is replaced by the permanent ballast: a layer of non-reinforced concrete with an area of B m^2.

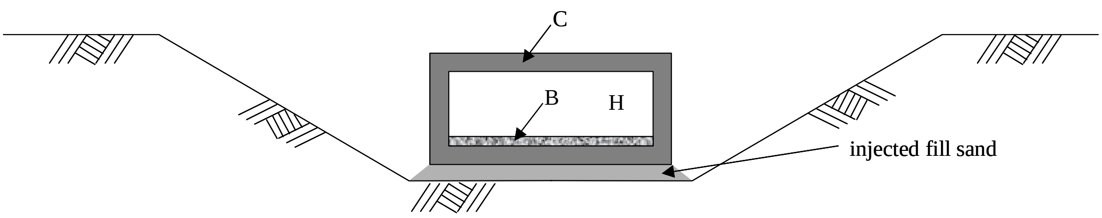


Figure 7-18 Definition sketch for the vertical equilibrium calculation (tunnel element in this case)

If the weight of the temporary watertight partitions on the ends of the tunnel section as well as that of the further immersion equipment (including that of the ballast tanks) is discounted, the following equation per running meter of tunnel can be derived:

<u>During the floating transport:</u>
When an immersed tunnel section is transported via an inland waterway the designer will usually strive for a small freeboard, say 1% of the external height of the element, so that later as little ballast as possible will be required. This means that the weight corresponds to 99% of the maximum buoyancy. If the volumetric weight of reinforced concrete is 25 kN/m^3 and that of water 10, then:

$$25\,C = 0.99 \cdot 10 \cdot (B + C + H)$$

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

$$25\,C + 23B = 1.08 \cdot 10 \cdot (B + C + H)$$

Solving these two equations gives:
$C = 0.701 \cdot H$ m^2
$B = 0.0693 \cdot H$ m^2

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

In a following round of the design cycle an investigation will be 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 7-18, by for example adding bevelling (*afgeschuinde randen*) 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 floating transport impossible.

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

- partly or entirely prestress the transverse section.
- use light concrete rather than concrete with a gravel aggregate.
- make the space (*H*) bigger than that deriving from the functional requirements (the required profile of free space).
- do not make the element free-floating (for example use barges with extra buoyancy capacity).
- combinations of the above.

Appendix 6 Stability of floating elements

FROM: Manual Hydraulic Structures CT3330

To ensure that floating elements do not undesirably move or rotate, they should be statically and dynamically stable. The stability of a floating element depends on forces and moments, and the shape of the element.

A6.1 Static stability

A6.1.1 Equilibrium of vertical forces

Vertical forces establish an equilibrium if the buoyant force (*opwaartse kracht*) equals the weight of the floating body (including all ballast). This buoyant force has the same magnitude as the weight of the displaced volume of fluid (Archimedes' principle: a floating body displaces its own weight of fluid).

A vertical equilibrium is usually reached if the element is floating, or if it is resting on the bottom of the water body. If there is no equilibrium, a completely immersed element will move upward or downward until an equilibrium state is reached. An element will move upward if the buoyancy is more than the total weight of element. Then, at a certain moment, the part of the element rising out of the water will result in a decrease of the buoyant force in such an amount that this buoyant force equals the weight of the element. It then stops moving upward. An element, conversely, will sink if the weight of the element exceeds the buoyant force, until it reaches the bottom. The bottom will resist the downward directed force and will stop the element moving down.

A6.1.2 Equilibrium of moments

For the design of large-scale prefabricated elements, vessels and dredging equipment, not just the weight is of importance. There must also be an assurance that the elements do not tilt in an unacceptable degree during the floating transport or the immersing procedure. Tilting can be initiated by the mooring forces, wave motions, and inlet of water during immersion, etc. Elements must therefore be designed or equipped in such a way that a rotation, caused by external factors, is corrected by a righting moment that will return the element to its original position.

If the sum of moments around the point of rotation (= point of gravity) equals zero, the element will not incline to tilt. This principle is illustrated in Figure 7-19, where equilibrium is not reached yet. It will tilt in such a way that an equilibrium will be reached.

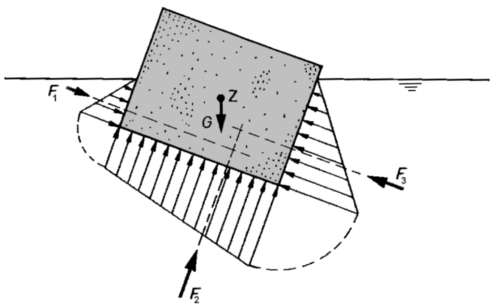


Figure 7-19 Forces acting on a floating element [Nortier, 1991]

A6.1.3 Static stability - metacentric height

A check of the equilibrium of moments (previous paragraph) is sufficient if an element is floating in still water. In reality, however, this is rarely the case. This is why also the 'sensitivity to tilting' has to be taken into account. A measure for the resistance to tilting is given by the 'metacentric height'. The principle is illustrated in Figure 7-20. The left side depicts the cross-section of a floating element (like a caisson). On the right side the same element is showed in tilted position. The rotation angle is φ.

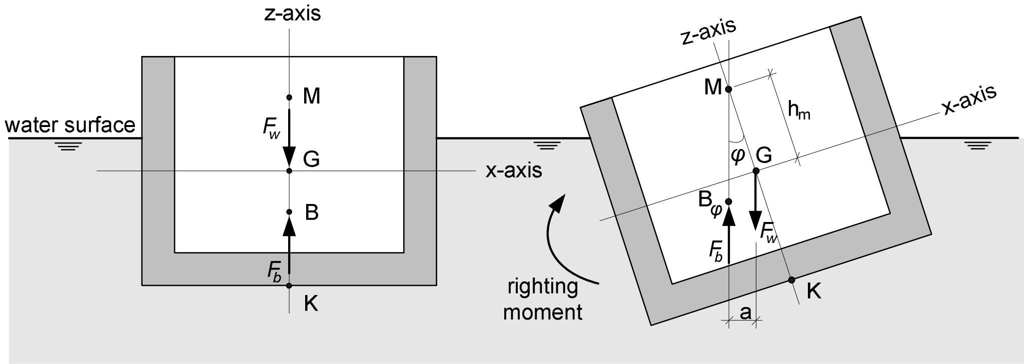


Figure 7-20 Floating element

Indicated are three points, which are of importance in the evaluation of the stability:

- B is the centre of buoyancy (*drukkingspunt*), the point of application of the buoyant force F_b in state of equilibrium (the state in which the axis of symmetry of the element is vertical). B is therefore the centre of gravity of the displaced water. In a rectangular container (caisson), B is found halfway between the water surface and the bottom of the element. In tilted position the centre of buoyancy shifts to a new position because the geometry of the displaced volume has changed. The shifted centre of buoyancy is indicated with Bφ and the horizontal shift is *a* [m].

- G is the centre of gravity (*zwaartepunt*) of the element. If the element is filled with a layer of gravel or water for the benefit of the immersing procedure (not shown in *Figure 7-20*), this weight should also be taken into account when calculating G. Not only will this ballast lower the centre of gravity, it will also increase the draught and will therefore raise B relative to the bottom. If the element heels over, the centre of gravity generally remains fixed with respect to the element because it just depends upon the position of the element's weight and ballast. The centre of gravity at the same time is the rotation point.

- M is the metacentre; the point of intersection of the axis of symmetry, the z-axis, and the action line of the buoyant force in tilted position. For small rotations (φ < 10°) the metacentre is a fixed point (see lectur e notes OE4652, 'Floating Structures' for a proof). The determination of point M is explained below.

For static stability, rotation of the element should be compensated by a righting moment caused by the buoyant force and the weight of the element. This is the case if M is located over G: the line segment $\overline{GM}$, also known as the metacentric height h_m, must be positive.

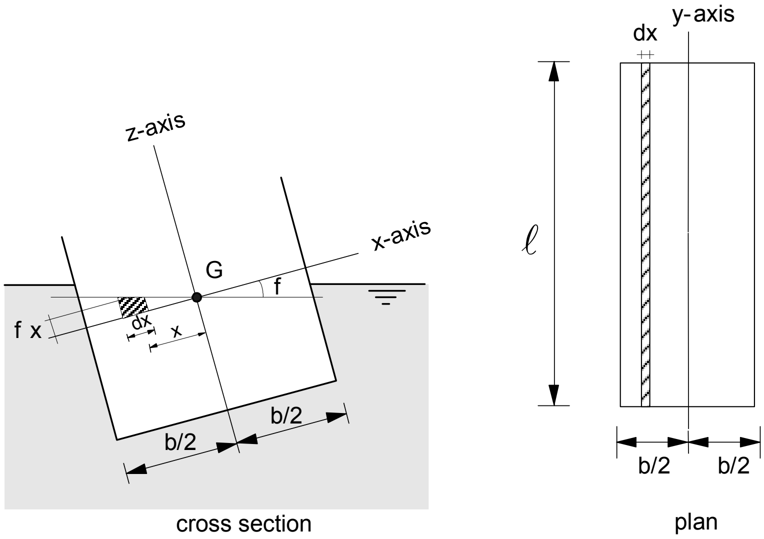


Figure 7-21 Tilted element

Figure 7-21 shows an element with a rotation φ. The part dx, which has been forced under water by the rotation, experiences an upward force:

$$dF_b = \varphi\, \ell\, dx\, \rho_w\, g\ ,$$

in which ρ_w is the volumetric mass of water. This equation is only valid for rotations smaller than 10°in which case $\tan\varphi \approx \varphi$ [rad]!

Relative to G this gives a moment $\mathrm{d}M = x\, dF_b = \varphi x^2 \ell\, dx\, \rho g$.

Over the entire width this means a righting (corrective) moment of:

$M = \int_{x=-½b}^{+½b} \varphi x^2\, \ell\, \mathrm{d}x\, \rho\, \mathrm{g}$, which could be rewritten as $M = \varphi \rho g \int_{x=-½b}^{+½b} x^2 \ell\, \mathrm{d}x$, which appears to be the same as $M = \varphi\, \rho\, g\, I$, in which I (actually I_{yy}) is the area moment of inertia (a.k.a. 'second moment of area'), relative to the y-axis, of the plane intersected by the waterline.

The point of application of the buoyant force (F_b) in a state of rest, is the centre of pressure B. A rotation φ leads to a translation of the line of action of F_b over a distance a (see Figure 7-20):

$$a = \frac{M}{F_b} = \frac{\varphi \rho g I}{\rho g V} = \frac{\varphi I}{V}$$

In this V is the volume of the immersed part of the element (= the volume of the displaced water).

The distance between the centre of pressure and the metacentre therefore is:

$$\overline{\mathrm{BM}} = \frac{a}{\varphi} = \frac{I}{V}$$

For small rotations (φ < 10°) the metacentre is a fixed point, but $\overline{\mathrm{BM}}$ increases because the position of B will go down. In case of considerable rotations, the metacentre displaces upward and sideways in the opposite direction in which the ship has rolled and is no longer situated directly above the centre of gravity. If M is positioned above G, a righting moment $F_b\, h_m\, \varphi = \rho\, g\, V\, h_m\, \varphi$ is created, which tries to return the element to its stable position.

For small seagoing vessels a metacentric height h_m of at least 0.46 m is required. A ship with a small metacentric height will be "tender" - have a long roll period. A low metacentric height increases the risk of a ship capsizing in rough weather and more likely to develop "synchronized rolling". It also puts the vessel at risk of potential for large angles of heel if the cargo or ballast shifts. If a ship with low h_m is damaged and partially flooded, the metacentric height will be reduced further and will make it even less stable.

For large ships h_m should be at least 1.1 m, but not too large because in that case the vessel will be too 'stiff': it will snap back upright too quickly after a wave or wind gust has passed, which will cause heavy stresses in the structural parts of the vessel, maybe shifting of the cargo and not unlikely sea sickness of the persons on board.

The requirements for caissons and tunnel elements are less tight: 0.5 m suffices for h_m. If M is positioned below G, the element is unstable and will tilt.

A6.1.4 Check design static stability

The check of the static stability (in this case also known as the outset stability, because only small rotations of the element are investigated) is made up of the following steps:

- Calculate the weight F_w and the position of the gravity centre G of the floating element with reference to K ($\overline{\mathrm{KG}}$). K is the intersection of the z-axis with the bottom line of the element. In general,

$$\overline{\mathrm{KG}} = \frac{\sum V_i \cdot e_i \cdot \gamma_i}{\sum V_i \cdot \gamma_i},$$

where V_i = volume of element i [m^3]
γ_i = specific weight of element i [kN/m^3]
e_i = distance between gravity centre of element i and reference level (i.e., a horizontal plane through point K) [m]

- Calculate the draught d of the element.

- Locate the centre of buoyancy B and calculate its position above the bottom of the element. This distance is $\overline{\mathrm{KB}}$. In case of rectangular elements, $\overline{\mathrm{KB}} = \frac{1}{2}d$
- Determine the shape of the area at the fluid surface and compute the smallest area moment of inertia *I* for that shape (this is the most unstable). For rectangular elements: $I = \frac{1}{12} \cdot \ell \cdot b^3$.
- Compute the volume of the displaced fluid *V*.
- Compute $\overline{\mathrm{BM}} = \frac{I}{V}$
- Calculate $h_m = \overline{\mathrm{GM}} = \overline{\mathrm{KB}} + \overline{\mathrm{BM}} - \overline{\mathrm{KG}}$
- Theoretically, if $h_m > 0$, the body is stable. In practice, $h_m > 0.50$ m is recommended. If $h_m < 0.50$, additional measures are required.

Besides the static stability, the dynamic stability (the oscillation) must also be checked. This will be explained later in this chapter.

A6.1.5 Measures for unstable elements

If the element is unstable, the design should be altered or additional measures have to be taken.
Examples of design alterations are:

- widening of the element, thereby increasing the area moment of inertia *I* (usually the floor thickness will increase too, because of strength requirements).
- making the floor of the element heavier. This lowers G and increases the draught (if the transport route allows for this), which raises B relative to the bottom of the element. Unfortunately *V* also increases, which decreases $\overline{\mathrm{BM}}$, but the other effects dominate, so the stability is increased.

Examples of additional measures are:

- adding ballast to the element (below the point of gravity) during transport.
- the use of stabilising pontoons or vessels (see Figure 7-22a), which increases *I*.
- linking two elements during the floating transport (see Figure 7-22b), which increases *I*. Before the elements are disconnected at their destination, extra ballast must be applied to ensure the stability of the individual elements.

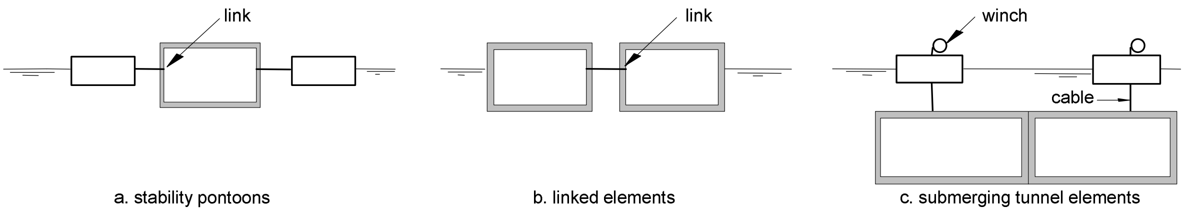


Figure 7-22 Stabilizing measures

If the required stability is achieved, one can opt to alter the design or use additional measures. Of course combinations of both are also possible.

<u>Stability during immersion</u>
Floating tunnel elements generally owe their stability to their large area moment of inertia. Once the elements have been given extra ballast, they immerse under water and no longer have a plane intersected by the waterline. The area moment of inertia is then zero. Stability is then only achieved if B is positioned above G. However, the elements are lowered on four cables using winches placed on four pontoons (see Figure 7-22c). This way, the elements are lowered accurately and in a controlled fashion. The element and the four pontoons together act as one system, which, around the pontoons, does have a plane that is intersected by the waterline and thus has an area moment of inertia. By positioning the pontoons as closely to the corners of the element as possible, large moments of inertia arise, both in the transverse direction and alongside the element.

<u>Water ballast in the immersing process</u>
The use of water as ballast to immerse elements is attractive because it is a fast method and filling up with water can be simply accomplished by opening the valves (mostly placed in or just above the floor).

If a closed element (i.e., with a roof) is completely filled with water, it acts in effect as a solid mass. This means that its weight can be regarded as being concentrated at its centre of gravity. If the element is only partly filled, or completely but not having a roof, the water surface is free to move and therefore possesses inertia. This causes a destabilising effect for the element, which can also be observed from Figure 7-23a. Due to the rotation, the depth of the ballast water increases on the left and reduces on the right. This results in a moment which amplifies the rotation. If the inner space of the element is partitioned, as is shown in Figure 7-23b, the moment driven by the ballast water decreases.

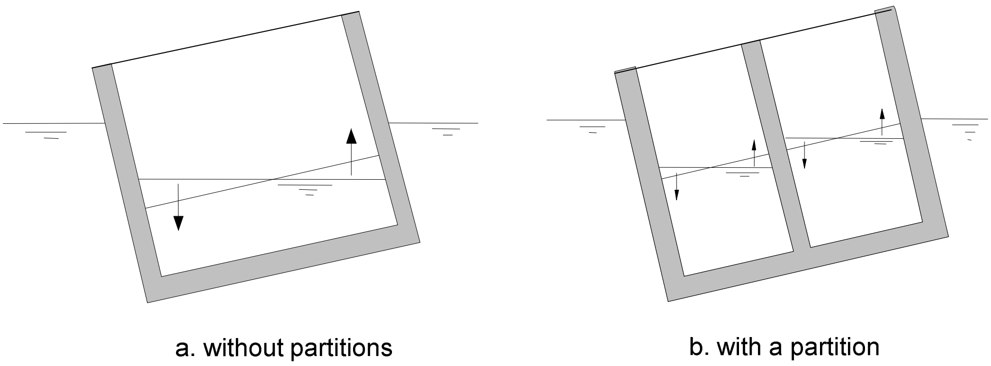


Figure 7-23 Water ballast

The unfavourable influence of the ballast water on the stability can be discounted by defining I as:

$$I = I_u - \sum I_i$$

in which:

I_u = area moment of inertia of the plane intersected by the waterline [m^4]
I_i = area moment of inertia of the ballast water area relative to the gravity centre line of the compartment concerned [m^4]

Not only in transverse direction, as treated above, but particularly lengthways is the creation of compartments with partitions beneficial for the stability!

The partitions, which incidentally won't always be used and not for every type of prefab element, also have the following advantages:

- Smaller spans (and so smaller moments and shear forces) of outer walls, floor and if present, roof.
- Realization of a better flow of forces. Concentrated loads (*puntlasten*) like shipping and crane wheel loads can be resisted more easily.
- Correcting the tilt (trimming) during the immersing process, by letting more (or less) water into one compartment than into the other.

Notes

- *Water is let into ballast tanks during the immersion of tunnel elements. These tanks are of limited sizes, in order to keep I_i as small as possible.*
- *Working without tanks: letting water straight into the elements, immediately leads to tilting, especially lengthways.*
- *Rainwater tanks used by farmers are often used as ballast tanks. These tanks are easy to fix and remove and are also cheap.*
- *The mentioned stability problems do not occur if sand or gravel is used as ballast material, provided that the material is spread evenly (so there is no unnecessary tilt) and that a coincidental unwanted tilt does not lead to the sliding of the ballast material.*

A6.2 Dynamic stability

Not only the static stability but also the dynamic stability should be checked. If an element is transported over water, it will be affected by waves or swell (*deining*). This can cause the element to sway (*slingeren*), which can cause problems with respect to navigability and clearance (*kielspeling*).

A6.2.1 Sway

If the dimensions (length or width) of a floating element are too small compared to the length of the waves or swell, the element will start swaying on the waves. In practice, the following rule of thumb is being used:

$L_w < 0.7 \cdot \ell_e$ and $L_w < 0.7 \cdot b_e$ (dependent on the direction of the waves relative to the caisson)

where:

L_w	=	wave length	[m]
ℓ_e	=	length of the floating element	[m]
b_e	=	width of the element	[m]

If this condition does not apply, problems due to swaying of the element can be expected.

A6.2.2 Natural oscillation

Worse than just swaying on the waves or swell is the movement of an element if the period of the water movements comes close to the natural oscillation period (*eigenperiode*) of the element. In order to prevent this, one must ensure that the natural oscillation period of the element is significantly larger than that of the waves or swell. For example, long swell was a problem with caissons in the bay of South Africa. If the natural oscillation period is a problem and adjustments of the design or additional measures do not offer a solution, or are too expensive, the transportation and positioning above the definite location should take place in favourable conditions as far as waves and swell are concerned. However, this can lead to serious delays and thus larger costs. Therefore one should conduct a cost optimisation: on the one hand the costs of additional measures and/or design alterations, on the other hand the costs of possible delays.

Ignoring the hydrodynamic mass (the additional water mass) and damping, the natural oscillation period of the floating element is:

$$T_0 = \frac{2\,\pi\, j}{\sqrt{h_m\, g}}$$

in which:	T_0	=	natural oscillation period	[s]
	h_m	=	metacentric height	[m]
	g	=	gravitational constant	[m/s^2]
	j	=	polar inertia radius of the element	[m]

The polar inertia radius (*polaire traagheidsstraal*) can be found according to:

$$j = \sqrt{\frac{I_{polar}}{A}}\ ,$$

where A is the area of concrete in a vertical cross-section.

The polar moment of inertia I_{polar} is a measure for accelerated rotation and should in this case be considered around the y-axis:

$$I_{polar} = \int_A r^2 dA = I_{xx} + I_{zz}\ ,$$

where I_{xx} = polar moment of inertia around the z-axis, and I_{zz} = polar moment of inertia around the x-axis, both in relation to the centre of gravity G. A large natural oscillation period is gained by a large polar inertia radius. A large metacentric height, however advantageous for the static stability, decreases the natural oscillation period.

If the natural oscillation frequency is much larger than the wave or swell frequency, the element is dynamically stable for oscillations (rotation). Wave period statistics for the Dutch North Sea can be found on internet: http://www.golfklimaat.nl.

The same type of calculation has to be carried out for dynamic stability for pitching (vertical translation instead of rotation), but this calculation method is not discussed here. A good example of pitching is the rolled up mat that was used for the Oosterschelde storm surge barrier. This floating roll weighed 9000 tonnes and moved 0.60 m up and down in calm seas (H = 0.20 m en T = 9 s). Pitching can also be a problem for dredging vessels, because the cutter can hit the seabed.

Appendix 7 Wave loads

FROM: Manual Hydraulic Structures CT3330

A7.1 Non-breaking waves

Civil engineering works located on the sea side of a breaker zone, can be subjected to loads by non-breaking waves. Non-breaking waves also occur in waterways and lakes in which the wave height is limited. Unlike around slender structures, the wave pattern is influenced by a wall. The wave height in front of the wall is determined by refraction and diffraction (see §A3.5 and §A3.6).

There are five methods to calculate the load on a wall due to non-breaking waves. They are given in the table below with a description of when they are applied.

No	Method	Design phase	Notes
1	Rule of thumb	preliminary estimate	conservative
2	Linear theory	preliminary (and final) design	-
3	Sainflou	preliminary design	simple!
4	Rundgren	final design	not in this handbook
5	Goda	final design	also for sills!

Table 7-3 Summary of methods

A7.1.1 Rule of thumb

According to linear wave theory for non-breaking waves against a vertical wall, the wave height H in front of the wall is double the incoming wave height H_i, in the case of total reflection. In short:

$H = 2H_i$ and with $H = 2a$: $a = H_i$ is valid.

This causes a temporary water level rise. If this is considered as a stationary load, the following rule of thumb can be applied to calculate the maximum wave pressure against a wall:

$$F_{max} = ½\rho g H_i^2 + d\rho g H_i$$

in which:
- H_i = the wave height of an incoming wave (= 2 a_i) [m]
- a = amplitude of the wave (half the wave height) [m]
- d = depth of the breakwater [m]

This can be used for a quick estimate of the upper boundary value of the wave load.

A7.1.2 Linear theory

For non-breaking waves against a vertical wall, the force on a wall can be determined using the pressure distribution in a vertical, taken from wave theory. As mentioned before, according to linear wave theory, the wave height H in front of the wall:

$H = 2H_i$ and with $H = 2a$: $a = H_i$ is valid.

The maximum pressure against a wall in case of reflection is then:

$$p = \rho g H_i \frac{\cosh(k(d+z))}{\cosh(kd)} \qquad \text{for } -d < z < 0$$

$$p = \left(1 - \frac{z}{H_i}\right)\rho g H_i \qquad \text{for } 0 < z < H_i$$

In which:
- H_i = wave height of an incoming wave [m]
- k = the wave number of the incoming wave (= 2π/L) [m^{-1}]

The force per linear metre follows from integration over the water depth:

$$F = \int_{-d}^{0} \rho g H_i \frac{\cosh(k(d+z))}{\cosh(kd)} dz + \int_{0}^{H_i} \left(1 - \frac{z}{H_i}\right)\rho g H_i \, dz$$

$$= \rho g H_i \left(\frac{(\exp(kd) - \exp(-kd))}{2k\cosh(kd)} + \frac{H_i}{2} \right)$$

In the case of a large wavelength, the wave pressure approaches the hydrostatic pressure (= rule of thumb). Figure 7-24 gives an example of this. The figure illustrates the wave pressures for different wavelengths, which are to be added to the hydrostatic pressure corresponding to the still water level.

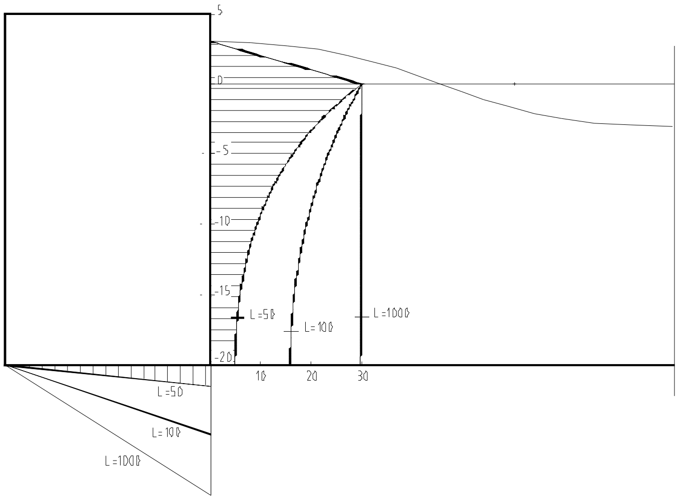


Figure 7-24 Linear wave theory: wave pressure

A7.1.3 Sainflou

In practice another simple approximation is often used for the total force on a wall, this method is known as Sainflou's method. This approach is shown schematically in Figure 7-25.
The approach is based on Stokes' second order wave theory, the waves have the shape of a trochoid and complete reflection (χ = 1). In this case the mean water level increases to MLS + h_0.

$$h_0 = \frac{1}{2} k H_i^2 \coth(k\,d)$$

where: h_0 = the height increase of the middle level [m]
H_i = the wave height of an incoming wave [m]
k = the wave number of the incoming wave [m^{-1}]

Sainflou and Stokes's second order wave theory lead to the same maximum pressures at middle water level and near the bed as the linear theory; viz.:

$$p_1 = \rho g H_i$$

$$p_0 = \frac{\rho g H_i}{\cosh(k\,d)}$$

The pressure between p_0 and p_1 is assumed to be linear. Therefore Sainflou leads to an overestimation of the load for steep waves.

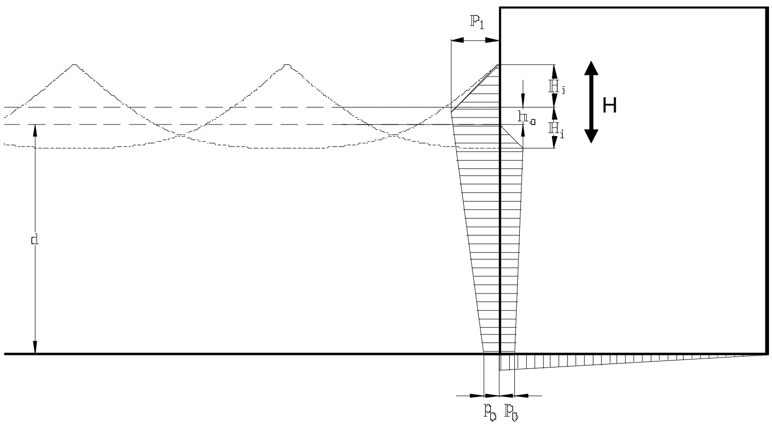


Figure 7-25 Sainflou: wave pressure

A7.1.4 Rundgren

Based on adapted higher order wave theory, Rundgren adapted Sainflou's formulas. The adapted formulas were used to make the graphs in CERC (1984).
In these graphs, overtopping and oblique approach are taken into account, which reduces the load.
Rundgren's wave theory is not covered in this manual.

A7.1.5 Goda

Goda (1985, 1992) made a general expression for the wave pressure on a caisson on a rockfill sill. This expression can also be used for broken and breaking waves. Worldwide Goda's equations are used often for the design of vertical breakwaters, see Figure 7-26. Goda's equations don't have an analytical base but rather an empirically foundation.

For the determination of the design wave height H_D and the design wavelength L_D, see the method in the manual CT3330 in §20.3. Goda proposed his own formula for H_D and L_D, however, these are not dealt with in this manual.

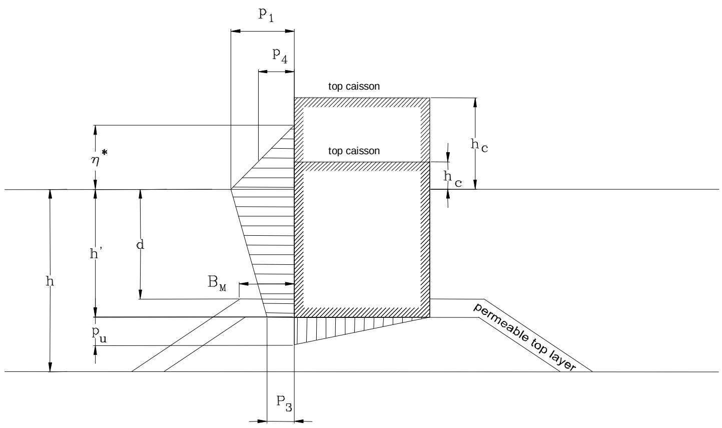


Figure 7-26 Goda (modified by Tanimoto): wave pressure

The sill height is h - d and the sill width is B_m.

The maximum wave pressures are:

$$p_1 = 0.5(1+\cos(\beta))(\lambda_1\alpha_1 + \lambda_2\alpha_2\cos^2(\beta))\rho g H_D$$

$$p_3 = \alpha_3 p_1$$

$$p_4 = \alpha_4 p_1$$

$$p_u = 0.5(1+\cos(\beta))\lambda_3\alpha_1\alpha_3\rho g H_D$$

in which:

α = the angle of the incoming wave

$\eta^* = 0.75(1+\cos(\beta))\lambda_1 H_D$

$\alpha_1 = 0.6 + 0.5\left(\dfrac{4\pi h/L_D}{\sinh(4\pi h/L_D)}\right)^2$

$\alpha_2 = \min\left(\dfrac{(1-d/h_b)(H_D/d)^2}{3}, \dfrac{2d}{H_D}\right)$

$\alpha_3 = 1-(h'/h)\left(1-\dfrac{1}{\cosh(2\pi h/L_D)}\right)$

$\approx \dfrac{1}{\cosh(kd)}$ (<u>without</u> sill)

$\alpha_4 = 1-\dfrac{h_c^*}{\eta^*}$

$h_c^* = \min(\eta^*, h_c)$

$\lambda_1, \lambda_2, \lambda_3$ = factors dependent on the shape of the structure and on wave conditions; (straight wall and non-breaking waves: $\lambda_1 = \lambda_2 = \lambda_3 = 1$)

h_b = water depth at a distance $5H_D$ from the wall

H_D = design wave height

L_D = design wavelength.

d = water depth above the top of the sill

h' = water depth above the wall foundations plane

h = water depth in front of the sill.

A7.2 Breaking waves

A7.2.1 Introduction
For the description of conditions in which a wave breaks.
For unbroken waves, the pressure distribution in the wave is a measure for the force on the wall. In the case of breaking waves this is not so. For those waves it is mainly the velocity with which the water particles hit the wall that is of importance. The shape of the breaking wave and possible air that is caught between the structure and the breaking wave largely influence the maximum wave shock and the course of the pressure distribution in time. The load due to breaking waves is still a point of research. The dynamic character of the load is an essential facet of breaking waves. Due to the collision between the wave and the structure a transfer of impulse takes place. At the moment of impact a relatively high pressure occurs, which only lasts a very short time (in the order of 1/100 s). Because of the short time span, this pressure is not representative for the stability of a structure (due to the inertia of mass). This pressure can be of importance for the strength of the structure (partial collapse).
It is better to prevent the wave shocks of breaking waves on the structure. In most cases it is therefore more economical not to place too high a sill in front of a straight wall. Thus, in most cases, the waves won't break and the load of the non-breaking waves is governing (maatgevend).

The sections below describe three models for breaking and broken waves. These models are:
- Minikin
- CERC 1984, broken waves
- Goda-Takahashi

These models are no more than rough estimates.

A7.2.2 Minikin
Minikin's model is based on both laboratory tests and on prototype measurements. Figure 7-27 gives a diagram of the model. It is based on a maximum dynamic pressure at the still water level and on a parabolic decline to zero over the distance $H_b/2$ above and below the still water level plus an increase of the hydrostatic pressure as a result of the displacement of the water surface.

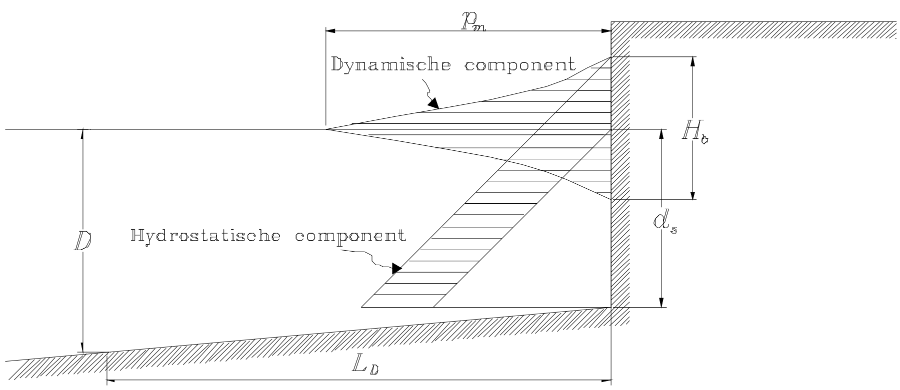


Figure 7-27 Minikin: broken wave pressure

The maximum pressure is:

$$p_m = \frac{1}{2} C_{mk} \pi \rho g \frac{H_b}{L_D} \frac{d_s}{D} (D + d_s)$$

where:	C_{mk}	=	coefficient of the impact ≈ 2	[-]
	H_b	=	breaker height	[m]
	d_s	=	depth in front of the wall	[m]
	D	=	depth at one wavelength in front of the wall	[m]
	L_D	=	wavelength at depth D	[m]

Minikin found $C_{mk} \approx 2$.

The resultant force according to Minikin is:

$$F = \frac{P_m H_b}{3} + \frac{\rho g H_b}{2}\left(\frac{H_b}{4} + d_s\right)$$

Note:
Minikin's method is unfortunately described incorrectly in CERC (1984). In the original publication by Minikin (1963), the pressure on the wall was expressed in tonnes per square foot. This is not correct. It should be ton force per square foot. This mistake was overseen in conversion to SI units for the CERC 1984 and has lead to a formula for p_m which gives values that are far too large. This is why many publications warn against Minikin's method, mentioning that the equation gives values that are 10 to 15 times too large, whilst the original method actually gave far lower values. One is advised not to use equations derived from Minikin (except for the corrected equations given above).

A7.2.3 CERC 1984

According to CERC 1984, the model for broken waves merely gives an indication of the load. If accurate estimates are needed of the maximum load on a structure due to breaking waves, more thorough research must be carried out for the specific situation.
Like Minikin's model, the model assumes a dynamic and a hydrostatic component of the water pressure on the structure.

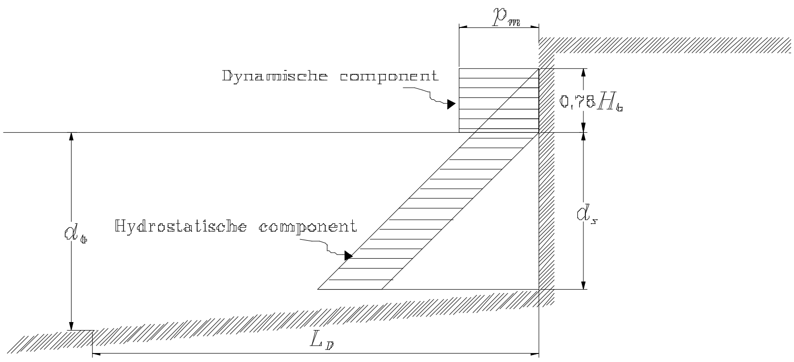


Figure 7-28 CERC 1984: broken wave pressure

The dynamic component is derived from the wave propagation velocity c at the moment the waves started to break. The broken wave is considered a translation wave with the propagation velocity:
$c = \sqrt{g d_b}$.
The dynamic pressure is:

$$p_m = \rho g \frac{c^2}{2g} = \frac{\rho g d_b}{2}$$

where: d_b = the water depth where the wave broke [m]

As in Minikin's model, the hydrostatic component of the load is caused by the displacement of the water surface. The total load as a result of the broken wave is therefore:

$$F = \rho g h_c \left(\frac{d_b}{2} + \frac{h_c}{2} + d_s\right)$$

in which: h_c = the height of the broken wave (translation wave) = 0.78 H_b [m]

A7.2.4 Goda-Takahashi

Goda's model was already given in the previous chapter. According to Takahashi and others (1994), a couple of factors need to be adjusted for waves that break on the berm of the sill on top of which a caisson has been placed:

$$\lambda_1 = \lambda_3 = 1$$

$$\lambda_2 = \max\left(1, \frac{\alpha_I}{\alpha_2}\right)$$

where: α_I = impulse coefficient

The impulse coefficient is determined with the following equations:

$$\alpha_I = \alpha_n \alpha_m$$

$$\alpha_m = \min\left(\frac{H_D}{d}, 2\right)$$

$$\alpha_n = \frac{\cos(\delta_2)}{\cosh(\delta_1)} \qquad \text{if } \delta_2 \le 0$$

$$\alpha_n = \frac{1}{\cos(\delta_1)\sqrt{\cosh(\delta_2)}} \qquad \text{if } \delta_2 > 0$$

$$\delta_1 = 20\,\delta_{11} \qquad \text{if } \delta_{11} \le 0$$

$$\delta_1 = 15\,\delta_{11} \qquad \text{if } \delta_{11} > 0$$

$$\delta_2 = 4.9\,\delta_{22} \qquad \text{if } \delta_{22} \le 0$$

$$\delta_2 = 3.0\,\delta_{22} \qquad \text{if } \delta_{22} > 0$$

$$\delta_{11} = 0.93\left(\frac{B_M}{L_D} - 0.12\right) + 0.36\left(\frac{h-d}{h} - 0.6\right)$$

$$\delta_{22} = -0.36\left(\frac{B_M}{L_D} - 0.12\right) + 0.93\left(\frac{h-d}{h} - 0.6\right)$$

where: B_M = width of the berm in front of the wall (see Figure 7-28) [m]

The dimensions of the berm have an important influence on the extent of the load. Figure 7-29 shows this influence for an example.

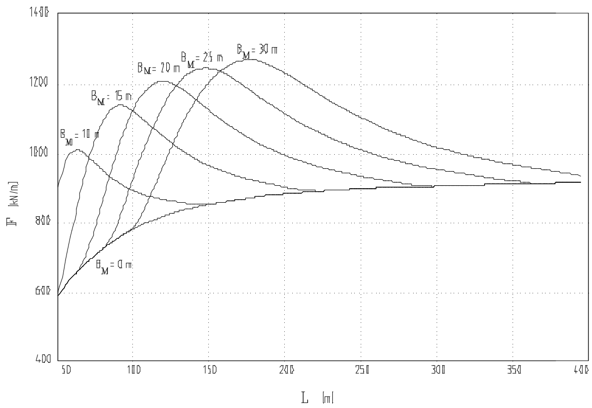


Figure 7-29 Influence of a berm on the wave load (H_D = 6 m, h = 9 m, h' = 7 m, d = 5 m, $h_c = \infty$)

Appendix 8 Bearing capacity of the soil

FROM: Manual Hydraulic Structures CT3330

The TGB 1990 (NEN 6744) gives the Brinch Hansen method for determining the maximum bearing force *F* of a foundation. This method is based on Prandtl's theoretical sliding surfaces. One distinguishes between drained and undrained situations. Undrained situations occur in cohesive impermeable soils, in which pore water pressures increase directly after the load is applied. In undrained soil an undrained shear strength c_u is used instead of *c'* in calculations. One also uses φ = 0.

Long-term loads on clay and peat are, of course, calculated as on drained soils!

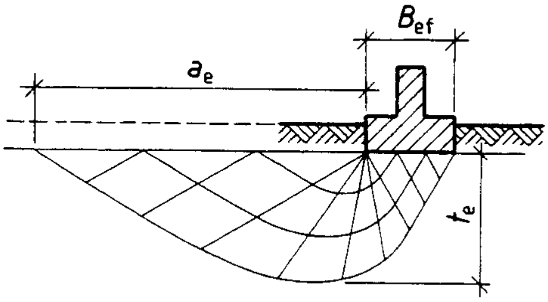


Figure 7-30 Prandtl's sliding surfaces

Brinch Hansen extended Prandtl's formulas to include reduction factors for the influence of a possible shear force H and the relation between the foundation's width *B* and length *L*.
The following discusses a method which is somewhat simplified, relative to the TGB 1990, and applies for well-permeable soil.

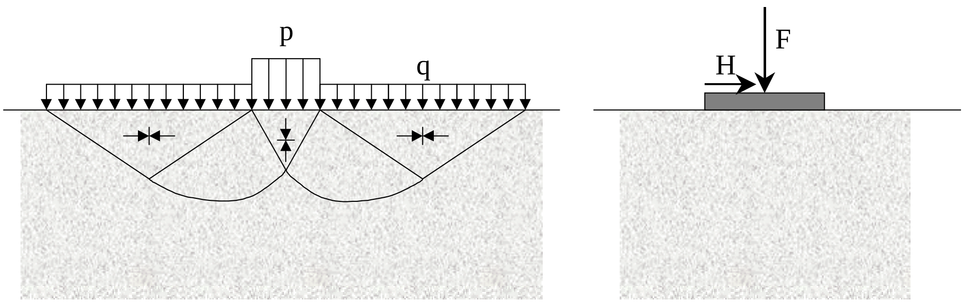


Figure 7-31 Prandtl and Brinch Hansen

The maximum bearing capacity can be approximated by:
$F_{\max} = p'_{\max} \cdot A$
where:
$p'_{\max} = c'N_c s_c i_c + q'N_q s_q i_q + 0{,}5\gamma' B \cdot N_\gamma s_\gamma i_\gamma$,
consisting of contributions from cohesion (index $_c$), surcharge including soil coverage ($_q$) and capacity of the soil below the foundation ($_\gamma$) (Figure 7-31).

The <u>bearing capacity factors</u> are:

$$N_c = (N_q - 1)\cot\phi' \qquad N_q = \frac{1+\sin\phi'}{1-\sin\phi'} e^{\pi \tan\phi'} \qquad N_\gamma = 2(N_q - 1)\tan\phi'$$

The <u>shape factors</u> ($B \le L \le \infty$) are:

$$s_c = 1 + 0.2\frac{B}{L} \qquad s_q = 1 + \frac{B}{L}\sin\phi' \qquad s_\gamma = 1 - 0.3\frac{B}{L}$$

The <u>inclination factors</u> to deal with an eventual inclined direction of the resulting force ($B \le L \le \infty$) are:

For drained soil:

For H parallel to L and $L/B \ge 2$:

$$i_c = \frac{i_q N_q - 1}{N_q - 1} \qquad i_q = i_\gamma = 1 - \frac{H}{F + Ac'\cot\phi'}$$

For H parallel to B :

$$i_c = \frac{i_q N_q - 1}{N_q - 1} \qquad i_q = \left(1 - \frac{0.70H}{F + Ac'\cot\phi'}\right)^3 \quad i_\gamma = \left(1 - \frac{H}{F + Ac'\cot\phi'}\right)^3$$

For undrained soil:

$$i_c = 0.5\left(1 + \sqrt{1 - \frac{H}{Af_{undr}}}\right)$$ for the rest, see drained soil, above.

Only the part of the foundation slab which has effective stresses underneath is included in the effective width B. The factors for the bearing force are also given in the figure below.

$\phi_{e;d}$	N_c	N_q	N_γ
0°	5	1	0
5°	6.5	1.5	0
10°	8.5	2.5	1
15°	11	4	2
20°	15	6.5	4
22.5°	17.5	8	6
25°	20.5	10.5	9
27.5°	25	14	14
30°	30	18	20
32.5°	37	25	30
35°	46	33	46
37.5°	58	46	68
40°	75	64	106
42.5°	99	92	166

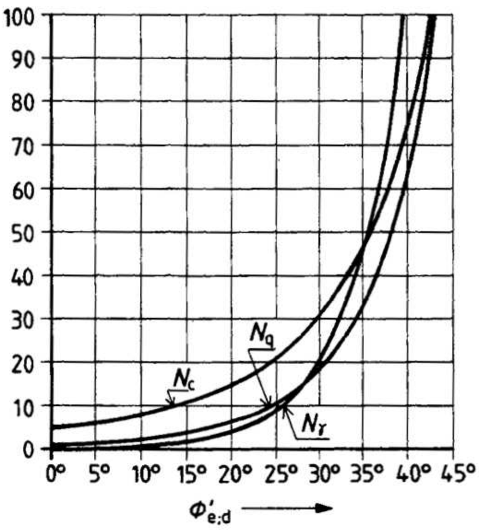
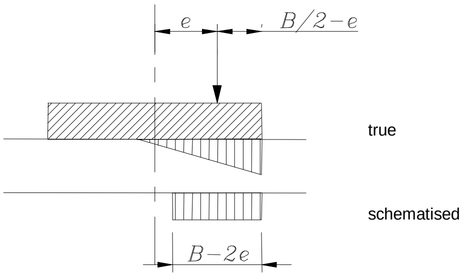


Figure 7-32 Bearing capacity factors as functions of the angle of internal friction

Clarification of the symbols used above:

p'_{max}	=	maximal average effective stress on the effective foundation area	[kPa]
A	=	the effective foundation area	[m^2]
c'	=	(weighted) cohesion (design value)	[kPa]
q'	=	effective stress at the depth of but next to foundation surface (design value)	[kPa]
	=	$\sigma'_{v;z;o} = \gamma_{f;g} \cdot \sum_{i=1}^{i=n} \left(d_i \cdot \gamma_{car}\right) - u$ (design value)	
u	=	the water pressure	[kN/m^2]
n	=	the number of horizontal soil layers between the construction depth and the soil cover level	
γ'	=	(weighted) effective volumetric weight of the soil below construction depth (design value)	[kN/m^3]
d_i	=	the thickness of layer i	[m]
γ_{car}	=	the characteristic volumetric weight of the soil for which: for a soil layer above groundwater level: $\gamma_{car} = \gamma_{rep}$; for a soil layer under groundwater level: $\gamma_{car} = \gamma_{sat;rep}$;	[kN/m^3]
γ_{rep}	=	the representative value of the volumetric weight with a natural humidity	[kN/m^3]
$\gamma_{sat;rep}$	=	the saturated volumetric weight	[kN/m^3]
$\gamma_{f;g}$	=	the load factor for a favourable load	
χ_c	=	factor for the influence of cohesion	

χ_q	=	factor for the influence of the soil cover	
χ_γ	=	factor for the influence of the effective volumetric weight of the soil	
ϕ'	=	(weighted) effective angle of internal friction (design value)	[°]
H	=	the shear force, i.e.: component of the force in the plane of the foundation surface (design value)	[kN]
F	=	component of the exerted force perpendicular to the foundation surface (design value)	[kN]
f_{undr}	=	the design value of the undrained shear strength = c_u	[kPa]
L	=	the length of the effective foundation area, for circular slabs: $L = B$	[m]
B	=	the width of the effective foundation area, for circular slabs: $L = B$ For an eccentrically loaded foundation B is approximated by B-2e	[m]



Note

- The factors for the bearing force are too conservative. This is because it is assumed that the factors do not influence each other, which is incorrect.
- The factors for the shape of the foundation and particularly for the horizontal load are not substantiated scientifically but they are based on empirical relations, experiments, calculations, etc.
- For a horizontal load H one not only has to apply a reduction in the calculation of the vertical bearing force F, but one also has to check if sliding can occur (e.g. using Coulomb).
- The reduction of the vertical bearing force F as a result of the horizontal load H is considerable. Assuming H/F = 0.30 results in i_q = 0.50!

Appendix 9 Piping

FROM: Manual Hydraulic Structures CT3330

Groundwater flow under or besides a water or soil retaining structure is caused by a potential difference across the structure. Piping can occur at the plane separating the impermeable structure and a loose grain layer. Piping is the flow of water through a pipe-like channel that has been created by internal erosion. This phenomenon can occur along the foundation plane of a structure but also along a retention wall. Piping is also possible in dikes. Little “sand volcanoes” appear where the water flows out at ground level.

Empirical formulas based on research describe the critical situations in which piping can occur. The most famous are the Bligh and Lane formulas. According to these formulas there is a limit state with a critical ratio between the differential head and the seepage distance. More recent research has confirmed this. For the results of this research and the design rules that have been derived from it, one is referred to Sellmeijer (1988).

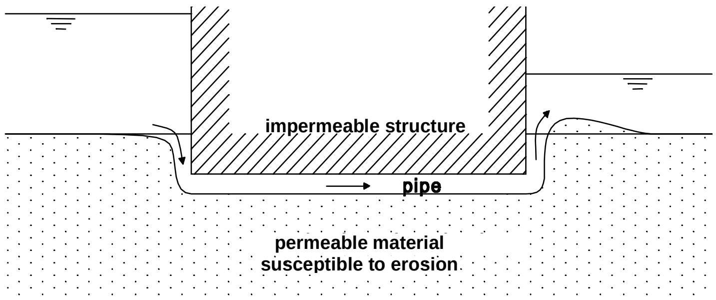


Figure 7-33 Piping

W.G. Bligh was the first to compose a formula for a safe seepage distance. E.W. Lane extended Bligh’s theory on grounds of an investigation of over 200 masonry dams. His formula poses that vertically placed structural parts are less likely to lead to a “pipe” (or 'line' or 'path') than horizontally placed parts. This is because the walls of a vertical line are more likely to collapse due to gravity, thereby blocking the line, than horizontal lines. In Lane’s formula slopes with an angle of 45° or larger are treated as vertical, slopes at less than 45° are considered horizontal.

Piping method:	**Bligh**		**Lane**	
Safe seepage distance:	$L \geq \gamma \cdot C_B \cdot \Delta H$		$L \geq \gamma \cdot C_L \cdot \Delta H$	
True seepage distance:	$L = \sum L_{vert} + \sum L_{hor}$		$L = \sum L_{vert} + \sum \frac{1}{3} L_{hor}$	
	C_B	i_{max}	C_L	i_{max}
Soil type:				
Very fine sand / silt /sludge	18	5.6 %	8.5	11.8 %
Fine sand	15	6.7 %	7.0	14.3 %
Middle fine sand	-	-	6.0	16.7 %
Coarse sand	12	8.3 %	5.0	20.0 %
(fine) gravel (+sand)	5-9	11.1 – 20.0 %	4.0	25.0 %

Table 7-4 Safe seepage distance for piping

where:
- L = total seepage distance [m]
- C_B = Bligh’s constant, depends on soil type [-]
- C_L = Lane’s constant, depends on soil type [-]
- ΔH = differential head across structure [m]
- γ = safety factor (1.5) [-]
- i_{max} = maximum (allowed) hydraulic gradient = $\Delta H / L$ [-]

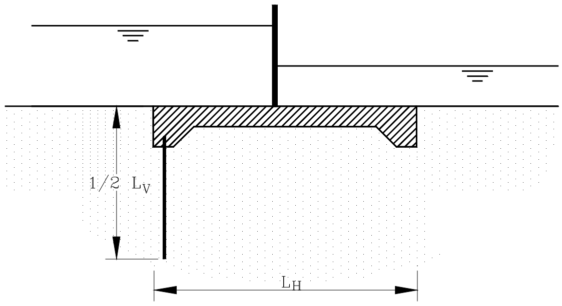


Figure 7-34 Horizontal and vertical seepage paths

Bligh assumes $L = \sum L_{vert} + \sum L_{hor}$ and Lane assumes $L = \sum L_{vert} + \sum \frac{1}{3} L_{hor}$. On grounds of Lane's assumption his soil constants also undergo changes (see Table 7-4).

In the Dutch design practice, both methods are being applied. Bligh's method is favorite for the design of dikes, whereas Lanes' method is used to estimate if piping will occur under water retaining structures because of the possibility of vertical piping lines.

J.B. Sellmeijer more recently (1989) developed a mathematical model to describe piping. The design rules resulting form this model lead to more favourable dimensions for the required horizontal piping line, compared to Bligh's method. Sellmeijers original model, however, is not applicable for vertical piping lines and is only suitable for the dimensioning of dikes. That is why he formulated additional design rules for the dimensioning of heave (*hydraulische grondbreuk*) behind seepage screens. These new design rules can be used if the piping criterion of Lane doesn't suffice. The Sellmeijer method, however, lies beyond the scope of this manual.

If a structure does not meet requirements for piping, the following solutions are possible:
1. using (longer) sheet piling upstream as a screen against seepage
2. grout columns (making the soil impermeable and cohesive) (upstream)
3. inserting a diagonal protective textile in the ground (in front of the structure)
4. inserting a filter structure (downstream)

Notes
1. *The full and one third part of respectively the vertical and horizontal planes may only be taken into account if the following conditions are satisfied:*
   - *The building material must be in direct contact with undisturbed soil*
   - *The walls must be closed and must have a water tight connection with the rest of the structure*
2. *The first condition means that if a structure has a pile foundation, which may allow settlements that create a split between the structure and the soil, the seepage path along the bottom slab may not be counted, unless extra measures are taken to seal the split.*
3. *One must not consider underseepage only, but must also take backward seepage into account. Cut off walls to prevent this are obviously placed in the same plane as the normal cut off walls to prevent under seepage. Furthermore, these walls are extended sideways beyond the loose ground of the building site.*

It is very important to compact the soil around the structure so it erodes less easily.

Appendix 10 Scour Protection

FROM: Lecture notes Introduction Hydraulic Engineering CT2320

Bottom scour may affect the waves in front of the structure, which can lead to gradual dislocation of the sill and can decrease the geotechnical stability of the breakwater (Figure 7-35). The scour depth in front of vertical breakwaters may under the worst conditions reach values up to 0.7 times the original water depth.

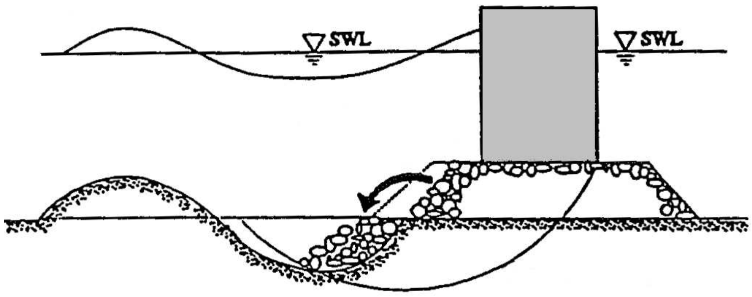


Figure 7-35 Effect of bottom scour on breakwater stability

Scour can be prevented by applying geometrically tight granular filters and more or less impermeable layers like concrete, asphalt and geotextile (see lecture notes of course CT4310 'Bed, bank and shore protection').

A10.1 Horizontal dimensions

K.W. Pilarczyk studied the behaviour of scour holes near hydraulic structures in order to find rules for an optimal length of the bottom protection [The Closure of Tidal Dams, §2.4.9. 'Local Scour', Delft University Press, 1987]. Pilarczyk found that the stones in the top layer of the bottom protection cause turbulence of the water flow. This leads to considerable sediment transport through the protection layer, thus causing erosion below this layer and the development of a scour hole in front of the bottom protection. The scour hole is mainly characterized by the upper scour slope (β) and the maximum scouring depth (h_{max}) (Figure 7-36). The presence of a scour hole with a too steep scour slope or a too big scouring depth will lead to sliding away of a soil section, or liquefaction (*zettingsvloeiing*) under the bottom protection. The influence of these mechanisms should not reach to the structure, to avoid its failure.

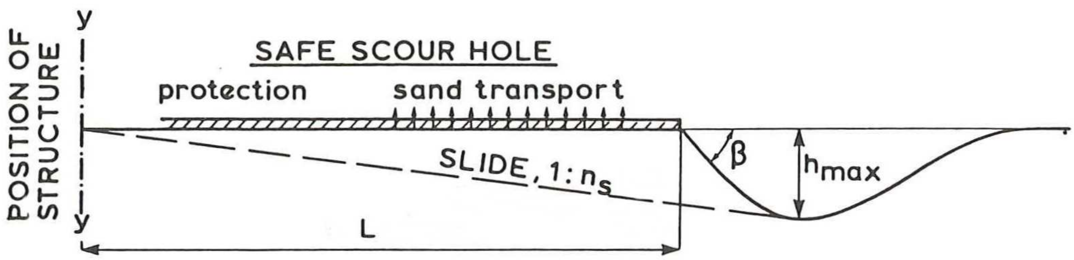


Figure 7-36 length of bottom protection

For a first estimate, the required length of the bottom protection can be calculated with:

$$L \geq \gamma \cdot n_s \cdot h_{max} \,,$$

where:

γ	= safety factor (≥ 1.0)	[-]
$1{:}n_s$	= average slope of the slide	[-]
h_{max}	= maximum scouring depth	[m]

$n_s \approx 6$ for densely packed, or cohesive material, $n_s \approx 15$ for loosely packet material

The upper scour slope, β, is usually much less steep than the natural slope of sediment under water. Usual values for β vary between 18° and 26°.

If no information is available, the maximum scouring depth may be assumed to be of the same magnitude as the (initial) water depth. This assumption, however, is rather rough and only to be used as a first indication. The calculation of the scour hole is extensicely treated in course 'Bed, Bank and Shore protection' (ct4310).

A10.2 Top and filter layers

(Partly translated from lecture notes Introduction Hydraulic Engineering CT2320)

A granular filter should be designed in such a way, that grains in the basic layer cannot pass the holes of the filter. The holes in the granular filter exist of the pores in the grain packet that are interconnected by small pore channels. If the diameter of the pore channels D_c is smaller than the diameter of the governing grains of the basic layer D_b, no transport can take place, irrespective of the value of the water level slope, the direction and type (stationary or not) of the flow.

In general, there are three types of scour protection, viz.:

- granular filter
- asphalt concrete
- geotextile

The granular filter type is the most commonly used and will be dealt with in the following.

A granular filter should be designed in such a way, that grains in the basic layer cannot pass the holes the filter. If the diameter of the pore channels D_c is smaller than the diameter of the governing grains of the basic layer D_b , no transport can take place, irrespective of the value of the water level slope, the direction and type (stationary or not) of the flow.

It has been empirically found that the diameter of a characteristic pore channel D_c in a granular layer meets with: $D_c \approx 0.2\, D_{f15}$ (D_{f15} is the size of the holes in a seave, through which passes 15 percent by weight of the material, see Dutch standard NEN 2560).

The governing grain size of the basic layer is about D_{85}. If grains with the governing diameter cannot be transported, also smaller grains cannot be transported.
A layer is geometrically tight, if:

$$\frac{D_{f15}}{D_{b85}} < 5 .$$

This only applies if the sieve curve of the basic material does not differ too much from the curve of the filter material. If a layer is widely graded, internal instability can occur. In that case, the small grains can be transported through the channels of the big ones. Internal stability can be expected if:

$$\frac{D_{60}}{D_{10}} < 10 ,$$

which should also be the case for a filter layer.

To avoid overpressure perpendicular to the separating layer, the filter layer should more permeable for water than the basic layer. In general, smaller grain diameters imply smaller permeability. In graded layers principally small grains of about D_{15} determine the permeability. During the lifetime of the structure, the diameters of the pore channels can decrease because of siltation with material from the basic layer, or because of deterioration (*verwering*). It is difficult to check up on this effect and eventual repairs are practically nearly impossible. Therefore, an extra requirement for permeability should be met:

$$\frac{D_{f15}}{D_{b15}} > 5$$

For geotextile filters, similar rules apply. A geotextile is considered to be tight if the larger fraction of the grains of the basic layer cannot pass the characteristic pores of the geotextile:

$O_{90} < D_{b90}$, which is very strict, and usually: $O_{90} < 2D_{b90}$

where O_{90} = the diameter of the fraction of which 90 weight-percentage remains on the geotextile after 10 minutes of sieving (so it is not an indication for the pore size distribution!)

If the basic material has a wide gradation (D_{b60}/D_{b10} > 10), D_{b90} in the equation can be replaced with D_{b50}.

To avoid overpressure in vertical direction, the filter should be more permeable than the basic material. In this case the permeability coefficient of the geotextile should be more than the permeability coefficient of the basic layer:

$$k_{ng} > \gamma\, k_b$$

where:

k_{ng}	=	permeability coefficient of the geotextile	[m/s]
γ	=	safety coefficient, dependent on the composition of the basic material γ = 2.5 for a uniform grain distribution, γ = 10 for a highly graded material (D_{60}/D_{10} > 10)	[-]
k_b	=	permeability coefficient of the basic layer	[m/s]

The permeability can be determined with help of a straightforward test, using a glass tube filled with soil, connected with two reservoirs of water - see lecture notes CT2090 'Soil Mechanics', section 8.1.

Appendix 11 Concrete strength

FROM: Manual Hydraulic Structures CT3330

Concrete is a commonly used and very suitable construction material, particularly for non-moving parts of hydraulic structures. The design of concrete structures is a profession in itself. It should be realised that the theory of concrete for hydraulic engineering purposes has other emphases than the theory for the more common utility construction branch, because of the following reasons:

1. In utility construction one can often schematize structural element as bending beams. In structural hydraulic engineering the concrete structures are often not slender and often have complex 3-D shapes.
2. In hydraulic structures the concrete parts below the water surface are under pressure. A pretensile stress is present on all sides. This does not exist in utility construction.
3. The reinforcement steel in hydraulic structures in sea water must be well protected from corrosion. This is why often prestressed reinforcement is used to reduce the crack width to zero.

A11.1 Properties of concrete

For design calculations European standards should be used: for concrete NEN-EN 1992-1-1:2005 (Eurocode 2: Design of concrete structures). For the theory about this subject, the course of CT2051 and CT3051 is recommended. The prescribed characteristics for concrete classes currently available in the Netherlands are presented in Table 7-5 (EN 206-1 table 7 and EN 1992-1-1 table 3.1).

Concrete class (old)	Concrete class	$f_{ck,cil}$ (MPa)	f_{ck} (MPa)	f_{cm} (MPa)	f_{ctm} (MPa)	$f_{ctk,\ 0.05}$ (MPa)	$f_{ctk,\ 0.95}$ (MPa)	E_{cm} (GPa)
B15	C12/15	12	15	20	1.6	1.1	2.0	27
B25	C20/25	20	25	28	2.2	1.5	2.9	30
B35	C30/37	30	35	38	2.9	2.0	3.8	33
B45	C35/45	35	45	43	3.2	2.2	4.2	34
B55	C45/55	45	55	53	3.8	2.7	4.9	36
B65	C55/67	55	67	63	4.2	3.0	5.5	38

Table 7-5 Characteristics of concrete classes

$f_{ck,cil}$ = characteristic compressive cylinder strength of concrete at 28 days [MPa]
f_{ck} = characteristic compressive cube strength [MPa]
f_{cm} = mean value of concrete cylinder compressive strength after 28 days ($f_{cm}= f_{ck}+8$) [MPa]
f_{ctm} = mean value of axial tensile strength of concrete [MPa]
- $f_{ctm}=0.30 \cdot f_{ck}^{(2/3)}$ ≤ C50/60;
- $f_{ctm}=2.12 \cdot \ln(1+(f_{ck}/10))$ > C50/60

$f_{ctk,\ 0.05}$ = characteristic axial tensile strength of concrete ($f_{ctk,\ 0,05} = 0.7 f_{ctm}$ 5% fractile) [MPa]
$f_{ctk,\ 0.95}$ = characteristic axial tensile strength of concrete ($f_{ctk,\ 0,95} = 1.3 f_{ctm}$ 95% fractile) [MPa]
E_{cm} = secant modulus of elasticity of concrete ($E_{cm} = 22\ [(f_{cm})/10]^{0,3}$) ($f_{cm}$ in MPa) [GPa]

The design value for concrete compressive strength can be computed as follows:

$$f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_C}$$

The design value for concrete tensile strength can be computed as follows:

$$f_{ctd} = \frac{\alpha_{ct} \cdot f_{ctk,0.05}}{\gamma_C}$$

Where:

f_{cd} : design value of concrete compressive strength [MPa]
f_{ctd} : design value of concrete tensile strength [MPa]
α_{cc} : coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied (α_{cc} = 1.0) [-]
α_{tc} : coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied (α_{tc} = 1.0) [-]
f_{ck} : characteristic compressive cylinder strength of concrete at 28 days [MPa]

$f_{ctk,0.05}$	:	characteristic axial tensile strength of concrete (5% fractile)	[MPa]
γ_c	:	partial safety factor for concrete	[-]

An overview of the partial safety factors for materials for ultimate limit states for concrete (γ_C) and steel (γ_S) is given in Table 7-6.

Design situations	γ_C for concrete	γ_S for reinforcement steel	γ_S for prestressing steel
Persistent & Transient loads	1.5	1.15	1.1
Accidental loads	1.2	1.0	1.0

Table 7-6 Partial safety factors for material

A11.2 Properties of reinforcement steel

For reinforcement steel TGB 1990 gives material properties for some steel classes:

Steel type		f_{yk} [N/mm^2]	f_{yd} [N/mm^2]	ε_{uk} [N/mm^2]
Bars	FeB 220 HWL	220	190	5.00
	FeB 400 HWL, HK	400	350	4.00
	FeB 500 HWL, HK	500	435	3.25
	FeB 500 HKN	500	435	2.75
Wire fabrics (*wapeningsnetten*)	FeB 500 HKN, HWN	500	435	2.75

Table 7-7 Characteristics of reinforcement steel classes according to the old TGB 1990 standard

Where:

f_{yk}	:	characteristic yield strength of reinforcement	[N/mm^2]
f_{yd}	:	design yield strength of reinforcement	[N/mm^2]
ε_{uk}	:	characteristic strain (*rek*) of reinforcement or prestressing steel at maximum load	[-]

The TGB 1990 standard has been replaced by a NEN standard that shows some differences regarding the characteristic values of reinforcement steel classes. The NEN standard (NEN 6008) applies only in a limited part of Europe. The most frequently used reinforcement steel class is B500B.

Reinforcement steel classes	$\varnothing$ [mm]	R_e [MPa]	R_m/R_e [-]	A_{gt} [%]
B500A	4-16	500	1.05 (1.03 for $\varnothing$ ≤ 5.5mm)	3.0 (2.0 for $\varnothing$ ≤ 5.5mm)
B500B	6-50	500	1.08	5.0
B500C	6-50	500	1.15 (1.13 for $\varnothing$ ≤ 12mm)	7.5 (7.0 for $\varnothing$ ≤ 12mm)

Table 7-8 Characteristics of reinforcement steel classes according to the NEN 6008 standard

Where:

$\varnothing$	:	nominal diameter	[mm]
R_e	:	characteristic yield strength of reinforcement (f_{yk})	[MPa]
R_m	:	characteristic tensile strength of reinforcement (f_{tk})	[MPa]
R_m/R_e	:	minimum ratio tensile strength/yield strength (f_{tk}/f_{yk})	[-]
A_{gt}	:	minimum percentage total elongation at maximum force	[-]
A	:	indicates a smooth, dented or ribbed profile	[-]
B	:	indicates a dented or ribbed profile	[-]
C	:	indicates a ribbed profile	[-]

Commonly used reinforcement bar diameters in Hydraulic Engineering are $\varnothing$ 12,16,20,25 and 32.

The Young's modulus of reinforcement steel (E_s) is $2.0 \cdot 10^5$ N/mm^2

A11.3 Properties of prestressed steel

In principle the function of prestressing is to prevent the occurrence of cracks in the concrete structure by creating compressive stresses in a structural member where one normally would expect tensile stresses. The elimination of tensile stresses does not only result in the prevention of cracks in the concrete, but also in a more economical use of materials (slender structures). In Figure 7-37 the principle of prestressing is visualized for a simply supported beam (*vrijopgelegde balk*) on two supports. The load on the beam results in compressive stresses above the centroid (*neutrale lijn*) and tensile stresses below the centroid as indicated in the left stress diagram. As the result of prestressing a normal force is exerted, resulting in an evenly distributed compressive stress over the whole cross-section of the beam (middle stress diagram). This compressive stress eliminates the tensile stress at the underside of the beam and reinforces the compressive stress at the topside, resulting in compressive stresses over the whole cross-section (right stress diagram).

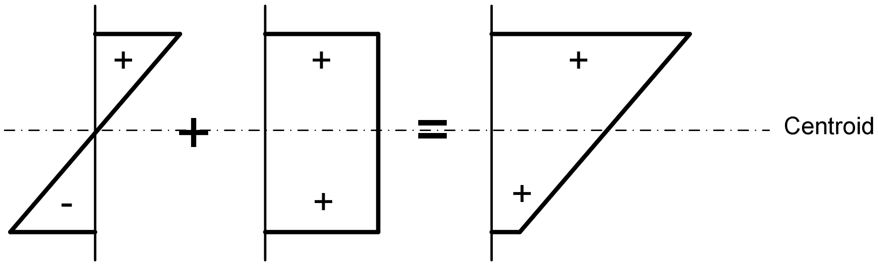


Figure 7-37 the principle of prestressing.

The following methods for prestressing concrete are used:

1) Pre-tensioning (*voorspanning met aanhechting, VMA*):
   This principle is mainly used in the prefabrication of concrete members. In the factory the tendons (*voorspanwapening*) are pre-stretched before the concrete is poured. After the concrete has hardened enough the tendons are released. The force present in the tendons is absorbed via adhesion (friction) between the steel and the concrete.
2) Post-tensioning (*voorspanning zonder aanhechting, VZA*):
   Here the tendons are situated in a protective tube. After pouring the concrete they are slightly tensioned in order to prevent attachment with the cement water. After a certain period, when the concrete has reached a strength determined by the structural engineer, the tendons are pre-stressed to approximately 20% of their capacity. This is called pres-stressing the dead weight (*eigen gewicht aanspannen*). When the concrete has reached its ultimate strength the tendons are pre-stressed to 100% of their capacity.

There are three types of prestressing steels, namely: wire (*voorspandraad*), strands (*voorspanstreng*) and bars (*voorspanstaven*). The properties of these three types are described in the following standards, NEN-EN 10138-2 (draft) “wire”, NEN-EN 10138-3 (draft) “strand” and NEN-EN 10138-4 (draft) “bars”. Furthermore the NEN- EN 10138-1 (draft) “general requirements” and NEN-EN 1992-1 are applicable. In Table 7-9 the characteristic values for certain diameters of all three types of prestressing steel are presented, for information regarding other available diameters the reader is referred to the standards mentioned above.
The following symbols are use in Table 7-9:

- d : nominal diameter
- S_n : nominal cross-sectional area
- f_{pk} : characteristic value for the tensile strength of prestressing steel
- $f_{p0.1pk}$: characteristic 0.1% yield boundary for prestressing steel
- ε_{uk} : characteristic strain (*rek*) of reinforcement or prestressing steel at maximum load

The design value for the tensile strength is equal to: $f_{pd} = \dfrac{f_{p0.1k}}{\gamma_s}$

The design value for the characteristic yield boundary can be computed as follows: $\varepsilon_{ud} = 0.9 \cdot \varepsilon_{uk}$.

The Young’s modulus for prestressing steel (E_p) is $2.05 \cdot 10^5$ N/mm^2 for wire and bars and $1.95 \cdot 10^5$ N/mm^2 for strands.

Steel type	d [mm]	S_n [mm^2]	f_{pk} [N/mm^2]	$f_{p0.1k}$ [N/mm^2]	ε_{uk} [-]
Wire					
Y1860C	4.0	12.57	1860	1599	0.035
Y1770C	6.0	28.27	1770	1521	0.035
Y1670C	8.0	38.48	1670	1437	0.035
Y1570C	10.0	78.54	1570	1299	0.035
Strand					
Y1860S3 class A	6.5	23.40	1860	1598	0.035
Y1770S7 class A	16.0	150.00	1770	1587	0.035
Y1960S3 class B	6.5	21.10	1960	1687	0.035
Y1960S7 class B	9.0	50.00	1960	1680	0.035
Bar					
Y1030H	26.0	531	1030	834	0.035
Y1030H	40.0	1257	1030	835	0.035
Y1230H	26.0	531	1230	1079	0.035
Y1230H	40.0	1257	1230	1080	0.035

Table 7-9 Characteristic values for prestressing steels.

A11.4 Concrete cover

The concrete cover on the outer reinforcement bar of a structure serves to protect the reinforcement against external influences such as rain water, soil, corrosive liquids or fumes or the like, which can lead to corrosion of the reinforcement. When the concrete cover is too thin or insufficiently dense there is a risk that the reinforcement starts to oxidize (*roesten*). This will lead to reduction of the bar diameter and hence the force that the reinforcement can absorb decreases. Since rust has a larger volume than the original steel there is a probability that the concrete cover is pushed of the reinforcement. This will lead to further corrosion and a further decrease of the absorbable force. It is be obvious that the reinforcement in an aggressive environment requires a thicker concrete cover than in a dry environment. Hence the thickness of the concrete cover depends on the environment in which the concrete structure is located. The environment characteristics are expressed via an exposure classification, see Table 7-9. The minimum required concrete cover on the outer reinforcement bar for each exposure class is presented in Table 7-10.

Exposure classification			
Class	Corrosion induced by	Class	measure of humidity
X0	no risk		very dry
XC	carbonation	XC1 XC2 XC3 XC4	dry or persistently wet wet, seldom dry moderate humidity alternating wet and dry
XD	chlorides (excl. seawater)	XD1 XD2 XD3	moderate humidity wet, seldom dry alternating wet and dry
XS	seawater	XS1 XS2 XS3	exposed to salt in the air, no direct contact with seawater persistently submerged tidal-, splash- and spray-zone
XF	freeze/thaw attack	XF1 XF2 XF3 XF4	not fully saturated with water, without de-icing salt not fully saturated with water, with de-icing salt fully saturated with water, without de-icing salt fully saturated with water, with de-icing salt
XA	chemical attack	XA1 XA2 XA3	weakly aggressive chemical environment moderately aggressive chemical environment highly aggressive chemical environment
Notation: XC1 X stands for exposure; the letter indicates process that causes the corrosion and the number indicates the humidity			

Table 7-10 Exposure classification of the environment in which the structure is situated.

Exposure classification	Concrete cover (c) [mm]		
	slab, wall	beam, pad footing (*poer*), truss (*spant*)	column
X0	-	-	-
XC1	15	25	30
XC2 to XC4 XF1 and XF3	25	30	35
XD1 to XD3 XS1 to XS4 XF2 and XF4 XA1 to XA3	30	35	40

A surcharge of 5 mm to the minimum concrete cover should be applied in case of:

- a finished (*nabewerkt*) surface;
- an uncontrollable surface;
- concrete with a characteristic cube compressive strength (*kubusdruksterkte*) $f'_{ck} < 25\ \text{N/mm}^2$

Note that when the situations above occur simultaneously, the surcharges should be superimposed.

Table 7-11 Minimum concrete cover on the outer reinforcement bar

A11.5 Reinforced and prestressed concrete

To design reinforced or prestressed concrete structures the following limit states have to be considered:

1) Ultimate limit states, leading to failure of the structure;
2) Serviceability limit states; leading to restriction of use of the structure.

A11.5.1 ultimate limit state:

- fracture due to bending and / or normal force
- fracture due to shear force
- fracture due to punching
- fracture due to torsion

A11.5.2 serviceability limit state:

- unacceptable deformation
- unacceptable cracking (*scheuren*)

For a more elaborate consideration of the limit states, reference is made to the TGB 1990 (NEN 6720, chapter 8). Bending and shear force are discussed briefly here because they are of importance in a preliminary design.

A11.5.3 Bending and/or normal force

The limit state involving bending and normal force is:

$$M_{\text{Ed}} = M_{\text{Rd}} \quad \text{en} \quad N_{Ed} = N_{Rd}$$

in which:

M_{Ed} = design value of the maximum occurring bending moment
M_{Rd} = maximum allowable bending moment
N_{Ed} = design value of the normal force
N_{Rd} = maximum allowable normal force

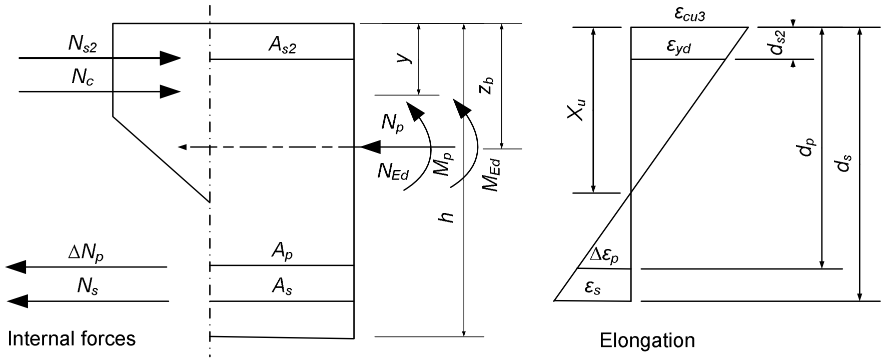


Figure 7-38 Forces and strains in concrete

The maximum allowable moment and normal force are:

$$M_{\mathrm{Rd}} = (N_p + N_{\mathrm{Ed}})(z_{\mathrm{b}} - y) + \sum N_{\mathrm{s}}(d_{\mathrm{s}} - y) + \sum \Delta N_{\mathrm{p}}(d_{\mathrm{p}} - y)$$

$$N_{Rd} = N_c + N_{s2} - N_p - \Delta N_p - N_s$$

Where:

- N_p : design value for the effective normal compression force as a result of the pre-stressing force
- M_p : design value for the effective moment force as a result of the pre-stressing force
- N_{Ed} : design value of the normal force (excluding pre-stressing force); if the normal force is a tensile force replace $+N_{\mathrm{Ed}}$ with $-N_{\mathrm{Ed}}$
- N_c : design value of the compression resultant = $0.75 \cdot x_u \cdot f_{cd}$
- f_{cd} : design value of concrete compressive strength
- N_s : tensile force in the reinforcement steel
- $N_{s,2}$: compressive force in the reinforcement steel
- ΔN_p : increase of the force in the pre-stressing reinforcement relative to the initial pre-stressing force ($\Delta N_p = A_p \cdot \Delta\sigma_{pu}$)
- A_p : cross-sectional area of the pre-stressed element
- $\Delta\sigma_{pu}$: increase of the stress in the pre-stressing reinforcement relative to the initial pre-stressing stress.
- y : distance between the compression stress resultant and the edge with most compression = 7/18 x_u (for $\leq C50/60$)
- x_u : height of the concrete compression zone
- d_s : the distance between the tensile reinforcement and the edge with most compression
- d_{s2} : the distance between the reinforcement in the compression zone and the edge with most compression
- d_p : the distance between the pre-stressing steel and the edge with most compression
- z_b : the distance between the elastic line of gravity and the edge with most compression
- h : total height of the structure
- ε_{cu3} : ultimate compressive strain in the concrete

When determining x_u one must take into account that: $\varepsilon_{cu3} = 0.0035$.

Furthermore, there are requirements for the maximum value of x_u if the normal force is small ($N_{Ed} < 0.1 \cdot f_{cd} \cdot A_c$) due to the rotation capacity, for this the reader is referred to TGB 1990 (NEN 6720 art 8.1.3).

To calculate the required reinforcement, the requirement should be satisfied that the reinforcement steel must yield before the concrete will fail and the minimum of the reinforcement percentage must be large enough to be sure there will be no brittle failure when cracking of the concrete occurs (*brosse breuk*). If the structure is mainly loaded by a moment force, the required reinforcement steel can easily be calculated with help of Table 7-12. Note that Table 7-12, Table 7-13, Table 7-14 and the flowchart below only apply to reinforced concrete and not for pre-stressed concrete. The flowchart is used to compute the reinforcement percentage needed in a structural member when the bending moment for the ultimate limit state is known.

Given M_u (kN/m), b (m), d (m) and f_{cd}

↓

Compute $M_u/b \cdot d^2 \cdot f_{cd}$ (kN/m^2)

↓

Look up ρ for the chosen concrete class

↓

Check ρ_{min}

↓

Check ρ_{max}

↓

Compute A_s with $A_s = \rho \cdot b \cdot d \cdot 10^4$ mm^2

Figure 7-39 Flowchart for the preliminary design of reinforcement using the GTB-tables.

$\frac{M_d}{bd^2 f_{cd}}$	ψ	k_x	k_z	ρ [%]				
				C20/25	C28/35	C35/45	C45/55	C53/65
10	0,010	0,013	0,99	*0,03*	*0,05*	*0,06*	*0,08*	*0,09*
20	0,020	0,027	0,99	*0,07*	*0,10*	*0,13*	*0,15*	*0,18*
30	0,030	0,240	0,98	*0,10*	*0,15*	*0,19*	*0,23*	*0,27*
40	0,041	0,055	0,98	*0,14*	0,20	0,25	0,31	0,37
50	0,051	0,068	0,97	0,18	0,25	0,32	0,39	0,46
60	0,062	0,083	0,97	0,21	0,30	0,39	0,47	0,56
70	0,073	0,097	0,96	0,25	0,35	0,45	0,55	0,66
80	0,084	0,112	0,96	0,29	0,41	0,52	0,64	0,75
90	0,095	0,127	0,95	0,33	0,46	0,59	0,72	0,85
100	0,106	0,141	0,94	0,37	0,51	0,66	0,81	0,95
110	0,117	0,156	0,94	0,40	0,56	0,73	0,89	1,05
120	0,129	0,172	0,93	0,44	0,62	0,80	0,98	1,16
130	0,140	0,187	0,93	0,48	0,68	0,87	1,06	1,26
140	0,152	0,203	0,92	0,52	0,73	0,94	1,15	1,36
150	0,164	0,219	0,91	0,57	0,79	1,02	1,24	1,47
160	0,176	0,235	0,91	0,61	0,85	1,09	1,34	1,58
170	0,188	0,251	0,90	0,65	0,91	1,17	1,43	1,69
180	0,201	0,268	0,90	0,69	0,97	1,25	1,53	1,80
190	0,214	0,285	0,89	0,74	1,03	1,33	1,62	1,92
200	0,227	0,303	0,88	0,78	1,10	1,41	1,72	2,04
210	0,240	0,320	0,88	0,83	1,16	1,49	1,82	2,16
220	0,253	0,337	0,87	0,87	1,22	1,57	1,92	2,27
230	0,267	0,356	0,86	0,92	1,29	1,66	2,03	2,39
240	0,281	0,375	0,85	0,97	1,35	1,75	2,13	2,52
250	0,295	0,393	0,85	1,02	1,43	1,83	2,24	2,64
260	0,310	0,413	0,84	1,07	1,50	1,93	2,35	2,78
270	0,325	0,433	0,83	1,12	1,57	2,02	2,47	2,91
280	0,340	0,453	0,82	1,17	1,64	2,11	2,58	3,05
290	0,356	0,475	0,81	1,23	1,72	2,21	2,70	3,19
300	0,372	0,496	0,81	1,28	1,80	2,31	2,82	3,34
310	0,388	0,517	0,80	1,34	1,87	2,41	2,94	3,48
320	0,405	0,540	0,79	*1,40*	*1,96*	*2,51*	*3,07*	*3,63*

Table 7-12 Reinforcement percentages for rectangular cross-sections, reinforced with B500B, loaded by bending without normal force, With M_u in kNm; b and d in m^1 and f_{cb} in N/mm^2

Where: M_u = ultimate absorbable bending moment (*breukmoment*)

k = ratio between the strength of concrete and steel $\left(k = \frac{f_{yd}}{f_{cd}}\right)$

f_{yd} = design yield strength of reinforcement $\left(f_{yd} = \frac{f_{yk}}{\gamma_s}\right)$

f_{cd} = design value of concrete compressive strength $\left(f_{cd} = \frac{f_{ck}}{\gamma_c}\right)$

$\psi = k{\cdot}\rho$ = mechanical reinforcement percentage

$k_x = \frac{x_u}{d}$ $\quad$ $k_z = \frac{z_u}{d}$

x_u = height of the of the compressive zone (*hoogte drukzone*) $\left(x_u = d \cdot \frac{\rho \cdot k}{0.75}\right)$

z_u = arm of internal leverage (*inwendige hefboomarm*) $\left(z_u = d \cdot (1 - 0.52 \cdot \rho \cdot k)\right)$

d = effective height of the cross-section (*nuttige hoogte*) ($d = h$ - (c + ½ $\varnothing$)

ρ = reinforcement percentage

b = cross-sectional width

h = height of the cross-section

c = concrete cover

$\varnothing$ = bar diameter (*kenmiddellijn*)

A_s = total cross-sectional area of the reinforcement

	C20/25	C28/35	C35/45	C45/55
ρ_{min}	0.15	0.18	0.21	0.24

Table 7-13 Minimum reinforcement percentage (ρ_{min}) for B500B.

	C20/25	C28/35	C35/45	C45/55
ρ_{max}	1.38	1.94	2.49	3.05

Table 7-14 Maximum reinforcement percentage (ρ_{max}) for B500B.

To check an already existing concrete structural member, the maximum allowable bending moment can be computed using the following equation (see course CT2051):

$$M_u = A_s \cdot f_{yd} \cdot d \cdot (1 - 0.52 \cdot \rho \cdot k) \quad \text{and} \quad M_{ed} \leq M_u$$

Where:

M_{ed} = design value for the bending moment in the ultimate limit state

M_u = ultimate absorbable bending moment

A_s = total cross-sectional area of reinforcement

k = ratio between the strength of concrete and steel $\left(k = \frac{f_{yd}}{f_{cd}}\right)$

f_{yd} = design yield strength of reinforcement

f_{cd} = design value of concrete compressive strength

ρ = reinforcement percentage ($= \frac{A_s}{b \cdot d}$)

b = width of the concrete structure

Automatically the equation can also be used to calculate the necessary reinforcement, when the load is known.

A11.5.4 Shear force

The design value for the shear resistance $V_{Rd,c}$ **without** shear reinforcement is given by

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}\right] \cdot b_w \cdot d \quad [\mathrm{N}]$$

With a minimum of

$$V_{Rd,c} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d \quad [\mathrm{N}]$$

Where
- f_{ck} = characteristic compressive cylinder strength of concrete at 28 days in MPa
- k = $1+\sqrt{\dfrac{200}{d}} \le 2.0$ with d in mm
- ρ_1 = reinforcement ratio for longitudinal reinforcement $= \dfrac{A_{sl}}{b_w \cdot d} \le 0.02$
- A_{sl} = the area of the tensile reinforcement, which extends ≥ (l_{bd} + d) beyond the section considered
- b_w = the smallest width of the cross-section in the tensile area [mm]
- σ_{cp} = compressive stress in the concrete from axial load or prestressing = $N_{Ed} / A_C < 0.2 \cdot f_{cd}$
- N_{Ed} = the axial force in the cross-section due to loading or prestressing [in N] (N_{Ed}>0 for compression)
- A_C = the area of the concrete cross-section [mm^2]
- k_1 = a coefficient, in the Netherlands: 0.15
- $C_{RD,c}$ = a coefficient, in the Netherlands: $0.18/\gamma_c = 0.18/1.5 = 0.12$
- v_{min} $= 0.035 \cdot k^{3/2} \cdot f_{ck}^{\ 1/2}$

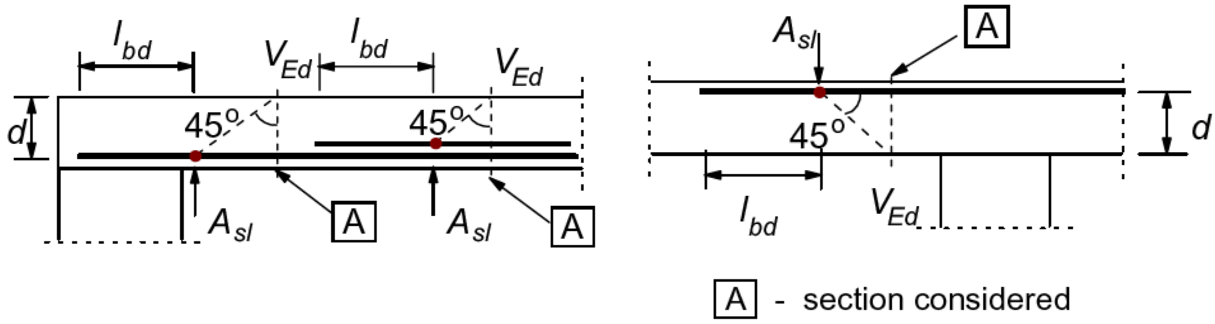


Figure 7-40 Reinforced concrete structural member without shear reinforcement.

The design of members **with** shear reinforcement is based on a truss model (*vakwerkmodel*).
In Figure 7-41 the following notations are shown:
- α = angle between shear reinforcement and the beam axis perpendicular to the shear force (measured positive as shown in the figure)
- θ = angle between the concrete compression strut and the beam axis perpendicular to the shear force
- F_{td} = design value of the tensile force in the longitudinal reinforcement
- F_{cd} = design value of the concrete compression force in the direction of the longitudinal member axis.
- b_w = the smallest width of the cross-section in the tensile area
- z = arm of internal leverage, for a member with constant depth, corresponding to the bending moment in the element under consideration. In the shear analysis of reinforced concrete without axial force, the approximate value z = 0.9·d may normally be used.

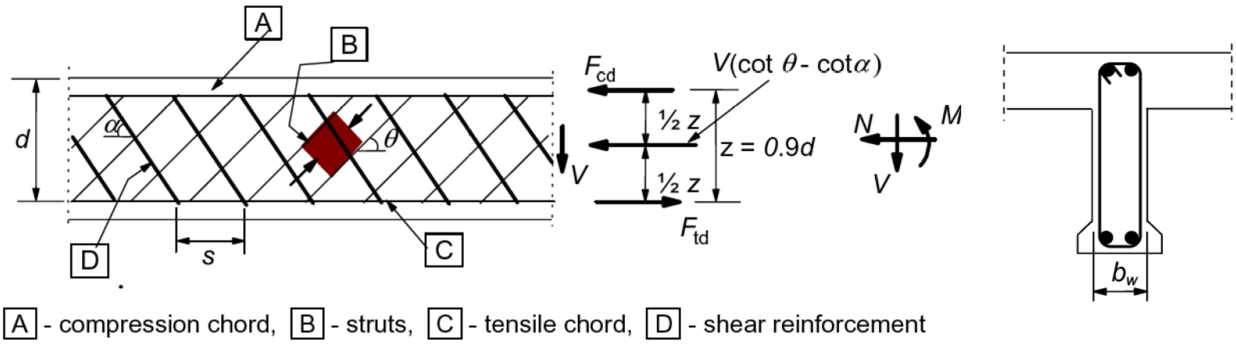


Figure 7-41 Inclined shear reinforcement

The angle θ should be limited. The recommended upper and lower limits are: $21.8° \le \theta \le 45°$.

A11.5.5 Vertical shear reinforcement

For members with vertical shear reinforcement, the shear resistance, V_{Rd} is the smaller value of :

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot \cot\theta \qquad \text{(stirrups governing)}$$

Where:

A_{sw} = the cross-sectional area of the shear reinforcement (two times because the reinforcement crossed two times the cross-sectional area of the concrete).
s = the spacing of the stirrups
f_{ywd} = the design yield strength of the shear reinforcement

And

$$V_{Rd,\max} = \frac{\alpha_{cw} b_w z \cdot v_1 \cdot f_{cd}}{\cot\theta + \tan\theta} \qquad \text{(concrete compressive struts governing)}$$

Where:

v_1 = strength reduction factor for concrete cracked in shear. Recommended is that v_1 = v and $v = 0{,}6\left(1 - \frac{f_{ck}}{250}\right)$

α_{cw} = coefficient taking account of the state of the stress in the compression chord. The recommended value of α_{cw} is as follows:

- 1 for non pre-stressed structures
- $\left(1 + \sigma_{cp} / f_{cd}\right)$ for $0 < \sigma_{cp} \le 0.25 \cdot f_{cd}$
- 1.25 for $0.25 \cdot f_{cd} < \sigma_{cp} \le 0.5 \cdot f_{cd}$
- $2.5 \cdot \left(1 - \sigma_{cp} / f_{cd}\right)$ for $0.5 \cdot f_{cd} < \sigma_{cp} \le 1.0 \cdot f_{cd}$

σ_{cp} = the mean compressive stress, measured positive, in the concrete due to the design axial force. This should be obtained by averaging it over the concrete section taking account of the reinforcement. The value of σ_{cp} need not be calculated at a distance less than $0.5 \cdot d \cdot \cot\theta$ from the edge of the support.

The maximum effective cross-sectional area of the shear reinforcement, $A_{sw,max}$ for $\cot\theta = 1$ is given by:

$$\frac{A_{sw,\max} \cdot f_{ywd}}{b_w \cdot s} \le \frac{1}{2} \cdot \alpha_{cw} \cdot v_1 \cdot f_{cd}$$

A11.5.6 Inclined shear reinforcement

For members with inclined shear reinforcement (*schuine dwarskrachtwapening*), the shear resistance is the smaller value of:

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha \qquad \text{(stirrups governing)}$$

and

$$V_{Rd,\max} = \frac{\alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} \cdot (\cot\theta + \cot\alpha)}{1+\cot^2\theta} \qquad \text{(concrete compressive struts governing)}$$

The maximum effective shear reinforcement, $A_{sw,max}$ for $\cot\theta = 1$ follows from:

$$\frac{A_{sw,\max} \cdot f_{ywd}}{b_w \cdot s} \leq \frac{\frac{1}{2} \cdot \alpha_{cw} \cdot v_1 \cdot f_{cd}}{\sin\alpha}$$

Note
In the walls of many hydraulic structures, there are large areas in which the shear force has reached its maximum while the bending moment is zero. In this case pure tension is found in the concrete wall, which needs special attention.

A11.6 Stiffness of the concrete structure

For statically indeterminate structures, the stiffness (EI) of the elements used in the calculations has considerable influence, not only on the resulting deformation and displacements, but on the flow of forces through the structure (global force effect) and the resulting internal forces in each individual member (local force effect) as well. Figure 7-42 illustrates this effect for a U-shaped cross section on a pile foundation. The correct stiffness has to be used in hand or computer calculations to find the governing (internal) load distributions M, N and V.

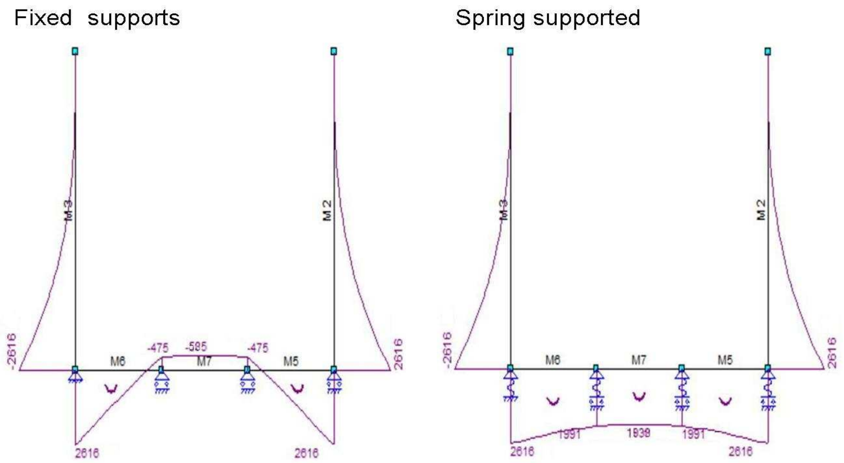


Figure 7-42 Influence of foundation stiffness on force distribution in the structure

Unfortunately the stiffness of a reinforced concrete section or element changes depending on crack development. There is a significant difference in bending stiffness between the non-cracked and the cracked concrete cross-section. After occurrence of the first cracks, further loading will go hand in hand with a decreasing stiffness of the concrete. This is easily demonstrated by a *M-κ* diagram, here *κ* is curvature (*kromming*), see Figure 7-43.

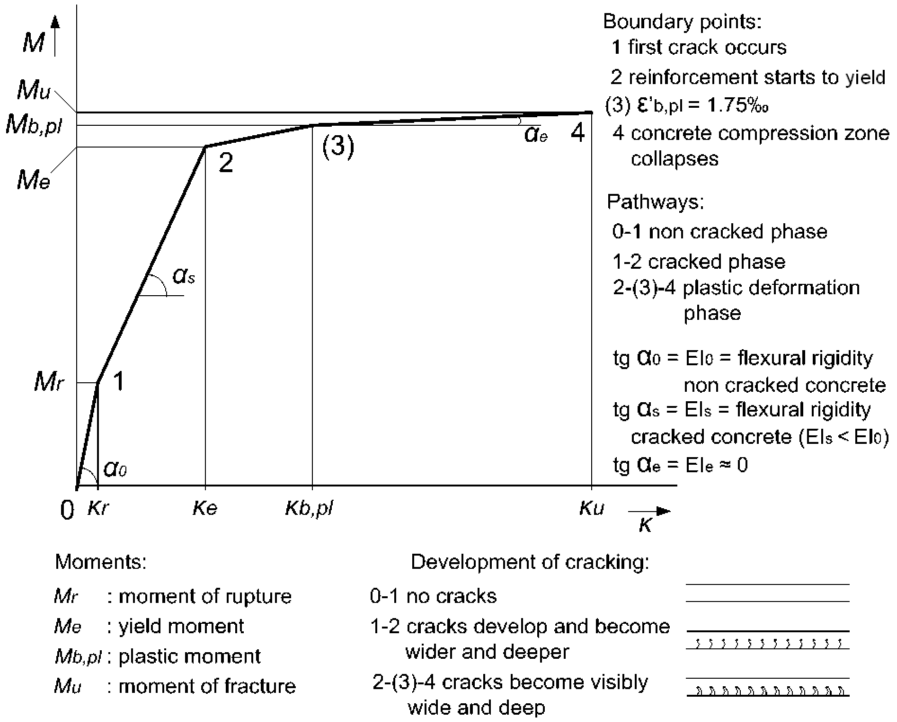


Figure 7-43 *M-κ*-diagram.

The bending stiffness of a concrete section, having to resist a certain M, is equal to the tangent of the line in the M-κ-diagram:

$$EI_x = \tan\alpha_x = \frac{M_x}{\kappa_x} \qquad \textit{and} \qquad \kappa_x = \frac{M_x}{EI_x}.$$

So, for a certain bending moment the intersection with the line in the *M-κ*-diagram has to be determined first to find the value of the curvature on the *x*-axis. Finally the tangent of the line connecting the origin with the intersection point can be found, using the equation above and hence the bending stiffness corresponding to that moment.

To construct a *M-κ*-diagram, all the *M-κ* combinations have to be computed. The curvature *κ* can be determined as follows:

$$\kappa = \frac{\varepsilon_b^{'} + \varepsilon_s}{d}$$

To find the correct stiffness of the whole structure the *M-κ*-diagram has to be constructed for every different concrete section, for each type of concrete and reinforcement percentage (*ρ*). This is a lot of work, often too much work for the level of precision required. In the following subsections first an approximation of concrete stiffness will be presented, then development of the M-κ-diagram will be further explained for detailed calculations.

A11.6.1 First design calculations with concrete EI guestimate

For uncracked cross-sections the bending stiffness of concrete EI_0 can be guestimated/computed as follows:

$$EI_0 = E_b^{'} \cdot I$$

Where:

$E_b^{'} = 22250 + 250 \cdot f_{ck}$ for $15 \le f_{ck} \le 65$ (NEN6720)

$E_b^{'} = 35900 + 40 \cdot f_{ck}$ for $65 \le f_{ck} \le 105$ (CUR 97)

$I = \frac{1}{12} \cdot b \cdot h^3$ for rectangular cross-sections

For cracked cross-sections the bending stiffness EI_g can be computed as follows:

$$EI_g = 0.5 \cdot E_s \cdot A_s \cdot h^2$$

Where:

E_s = bending stiffness of steel

A_s = area of the reinforcement steel present in the cross-sectional area of the beam
f_{ck} = characteristic compressive strength
b = width of the cross-section
h = height of the cross-section
[*source: 'construeren in gewapend beton' - part 2, Kamerling 1978*]

A11.6.2 More detailed calculation of concrete stiffness with M-K diagram

In this subsection the critical points of pure bending, i.e. bending moment M without normal force N, will be explained using stress-strain diagrams (*spanning-rek diagrammen*) in the end leading to the M-κ-diagram, see Figure 7-43.

Non-cracked beam (ongescheurde balk)
At the instant that the concrete tensile strength f_{ctd} is reached, the deformation diagram and stress diagram look like depicted in Figure 7-44. The bending moment equals the moment of rupture M_r (*scheurmoment*) and the concrete is just not cracked. In this stage the concrete's compressive strength is still very small because the mean value of the axial tensile strength of concrete f_{ctm} is much smaller than the design value of the concrete compressive strength f_{cb}, so that $\varepsilon_b^{'} << 1.75\text{‰}$.

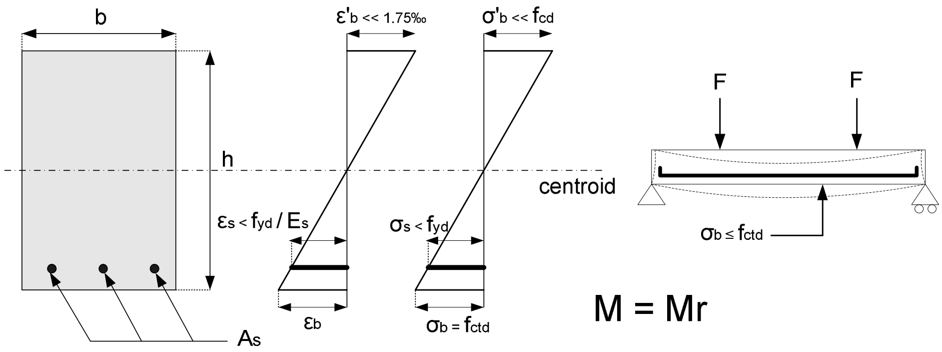


Figure 7-44 Deformation and stress diagram for a non-cracked beam.

Cracked beam
When the load only increases a little the tensile zone in the concrete will crack and the tensile forces will be concentrated in the existing reinforcement. The centroid (*neutrale lijn*) displaces in upward direction. The load can be increased further until the reinforcement reaches its yield stress f_{yd}. The corresponding deformation and stress diagrams are depicted in Figure 7-45. The concrete is cracked so it does not have a tensile strength any longer. The deformation of the concrete at the compression side of the beam $\left(\varepsilon_b^{'}\right)$ is still smaller than 1.75‰. The corresponding bending moment is the yield moment (*vloeimoment*). At this point the deformation of the steel changes from elastic to plastic.

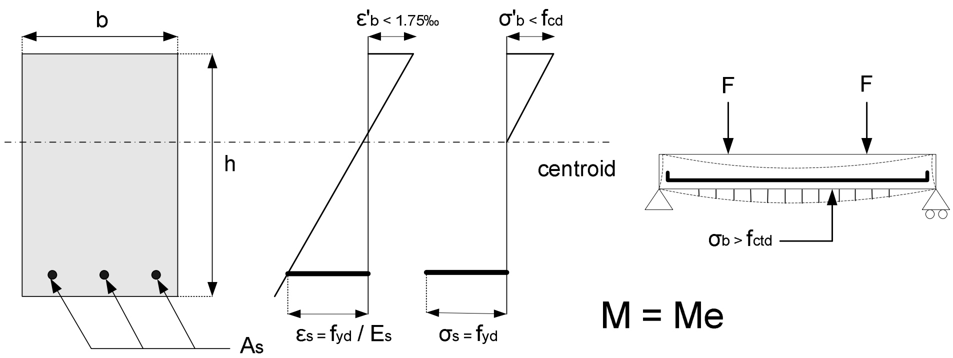


Figure 7-45 Deformation and stress diagram for a cracked beam.

A11.6.3 Compression strain in concrete (*betonstuik*)

When the load on the beam is increased further, at a certain moment the deformation of the concrete at the compression side of the beam will reach the value of 1.75‰ in the extreme pressure fibre (*uiterste drukvezel*). At the moment the compression strain of 1.75‰ is reached and the corresponding bending moment is equal to the plastic moment $M_{b,pl}$. The corresponding deformation and stress diagrams are depicted in Figure 7-46.

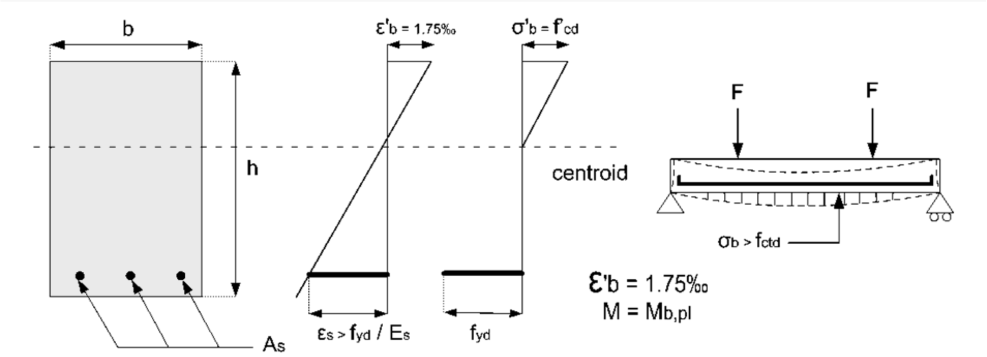


Figure 7-46 Deformation and stress diagram when the compression strain in the concrete has reached a value of 1.75‰.

Let the load on the beam increase even further and the compression strain in the concrete will reach eventually a value of 3.50‰ in the extreme pressure fibre. Now the beam has reached its point of collapse, the corresponding bending moment is the moment of fracture M_u (*breukmoment*). The corresponding deformation and stress diagrams are shown in Figure 7-47.

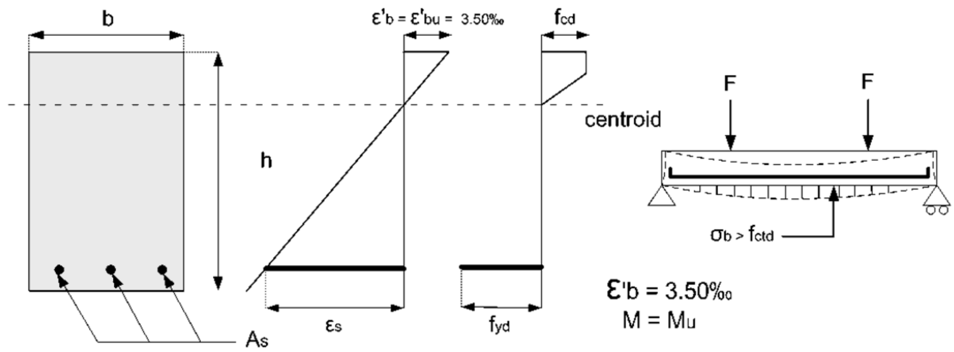


Figure 7-47 Deformation and stress diagram when the compression strain in the concrete has reached a value of 3.50‰.

Appendix 12 Construction costs

Below a cost calculation method for hydraulic engineering structures will be presented. The method is used mainly in the tender stage by contractors; however, it is suitable for use in other design stages and other parties.

A	Direct costs		
		Material costs	
		Equipment	
		Labour	
		Subcontractors	
		Temporary structures	
B	Indirect costs		
		Design	2 – 5 %
		Temporary site facilities	
		Staff on construction site	
		Supervision by client on site	
C	Uplift		
	(Staartkosten)	CAR - Insurance	1 – 1.5 %
		Overhead (Head office contractor)	5 %
		Risk	5 %
		Profit	10 – 15 %

Total construction costs T: T = A + B + C

Or

Total construction costs T: T = (1.3 – 1.4) * A

All costs VAT exclusive *(geen BTW meenemen)*.

The above paints a deceitful simple procedure to determine the total construction costs.