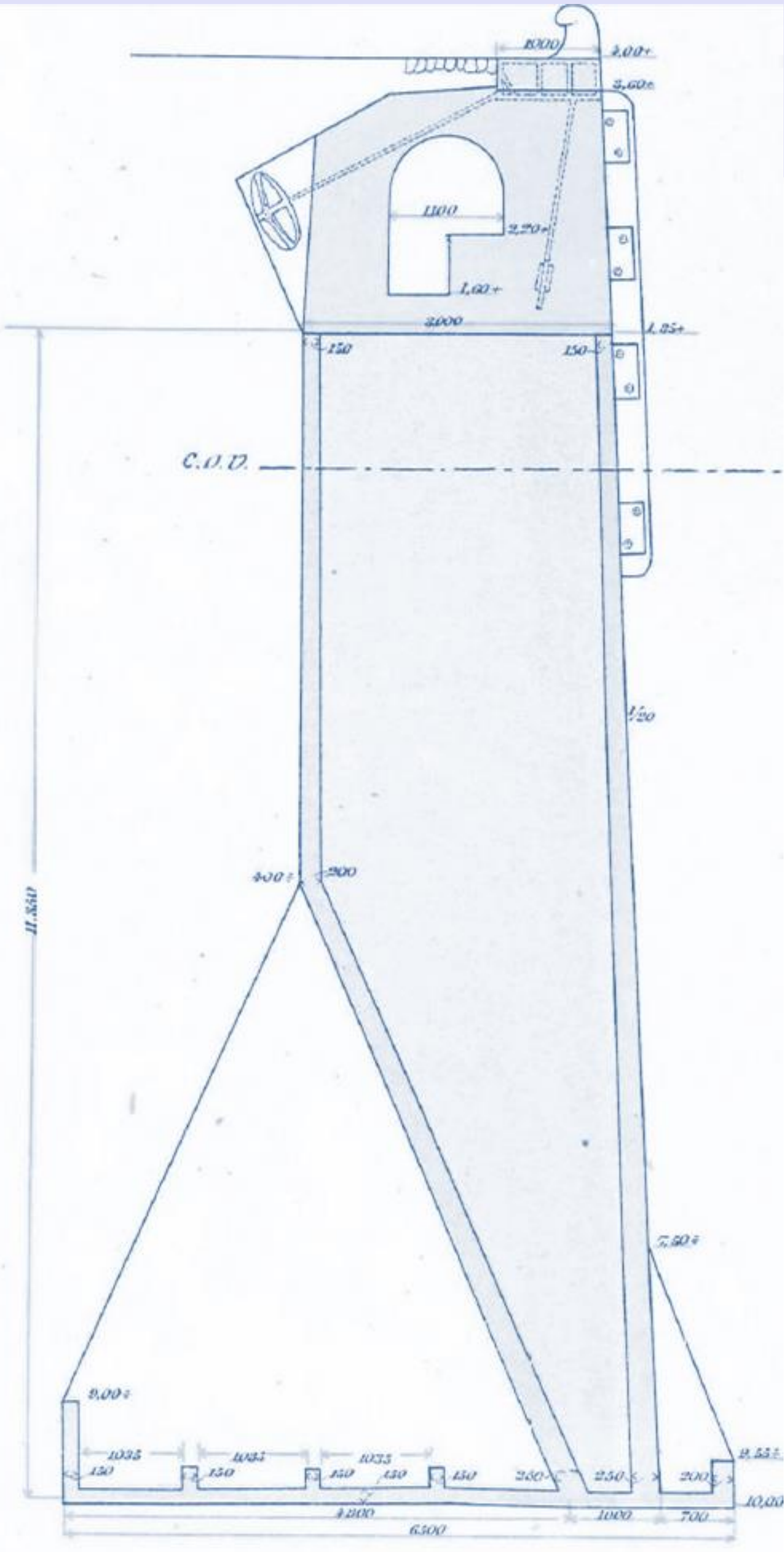
The overturning caisson *(kantelcaisson)*

Construction method reconsidered

Master Thesis

B.P. Korff



TU Delft bam

Overturning caisson

Construction method reconsidered

by

B.P. Korff

in partial fulfilment of the requirements for the degree of

Master of Science
in Civil Engineering

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Preface

This master thesis is conducted in partial fulfilment of the requirements for the Master degree in Civil Engineering, at the Delft University of Technology. The project was executed in cooperation with BAM Infraconsult B.V.

The first overturning caissons are, in my opinion, one of the most beautiful civil engineering structures ever made. The structures were designed in great detail and involved state of the art technologies for that time. A combination of the latest science and technologies allowed engineers to design and construct the very first reinforced concrete caisson quay walls. Luckily, the design was very well documented and preserved for over a century by the TU Delft Library.

The original report and drawings gave a lot of insight and revealed the major engineering and construction challenges. It was delightful to have such a historical project as a starting point. This background, combined with an already interesting topic, made it pleasurable to work on the thesis.

I would like to thank my graduation committee for their support during the graduation process. In particular, I would like to thank my daily supervisor, ir. L.A.M. Groenewegen, for his guidance and support. Not the less, feedback from all committee members was very helpful and enlightening. Furthermore, I would like to use this opportunity to thank all the professors, teachers and supervisors who contributed to the fundamental prerequisites for this thesis and future profession.

B.P. Korff,

Rotterdam, June 2017

Executive summary

The first overturning caissons were designed in 1903 by professor Kraus. The caissons had a geometry which differs from current caisson designs. The structures were designed as self-floating reinforced concrete boxes with a declined back-wall and counterforts. Due to these shape characteristics, the caissons could be referred to as hybrid counterfort caissons. However, another unique property of this concept is the horizontal construction method and horizontal floating position during transport. Due to this method, the caissons had to be turned in vertical position before placement. Because of this characteristic, the concepts are referred to as *overturning caisson* (Dutch: kantelcaisson).

The particular self-floating caissons were only applied for three quay wall projects. The concept was abandoned after the port expansion project for Tandjong Priok (Indonesia) in 1914. The Dutch contractor, Hollandsche Beton Maatschappij, learned from experience that vertically constructed rectangular caissons resulted in higher qualities, simplified formwork and easier transport at the same costs. However, without research, these arguments cannot be approved for current projects due to technical and economic advances for over one hundred years. This study is therefore performed to evaluate whether such a concept is feasible for current quay wall projects. Apart from the feasibility of the concept itself, smart sub-elements might be reusable for regular caisson designs.

The caisson geometry resulted in a material efficient retaining structure. The application of reinforced concrete was consciously considered. In the period 1903 to 1914, material savings resulted in a snowball-effect for other components in the construction process. Less concrete and less formwork area directly resulted into cost savings. Economic advantages were found in different elements within the construction process. Compared to a rectangular caisson, benefits of the original overturning caissons were:

- Less reinforced concrete;
- Less temporary equipment;
- Less formwork and falsework;
- Less weight (therefore reduced transport and launching costs).

Due to developments over time, caisson quay structures became larger and the robustness increased. The increased caisson dimensions are mainly caused by increased stability demands and larger design vessels. The need for more stringent stability calculations was outlined after several failures and excessive caisson deformations. A preliminary stability analysis based on original input parameters of the first overturning caissons revealed that the caissons lacked overturning stability (GEO) and resistance to forward sliding (GEO).

Besides the demand for larger caissons, technological developments resulted in improved production rates and higher labour efficiency. Developments in equipment, formwork and advanced concrete mix designs resulted in a cost reduction for the application of reinforced concrete. Because of this, material savings and a horizontal construction method do not directly lower the overall construction costs. High repetition factors and depreciation rates contribute highly to the economic feasibility of a caisson.

Since the urge for material savings reduced and more robustness is desired, only the most striking material saving aspects of the original concept are examined for modern overturning caisson designs. Elements such as tapered walls and stiffeners designed for the base plate are replaced by thicker uniform concrete elements. The declined back-wall and counterforts are considered as the most effective material saving feature and therefore kept for the modern designs. Additionally, keeping these characteristics allows a similar launching and transportation method.

When a caisson is designed according to current regulations and standards, the overturning concepts allow 8% to 15% material savings. If loads on the quay wall increase, the required caisson width becomes larger. This affects the feasibility of the concept. The material savings for the wide overturning caisson (15.65m) reduce to roughly 8%. Draught for the wider concept can be reduced from 12.40 metre for a rectangular caisson to 11.40 metre for the overturning caisson, which is a rather small reduction.

The largest material savings (15%) can be obtained when a rubble backfill is applied. These also result in a significant reduction of draught during transport. For this case, draught can be reduced from 12.60 metre to 9.60 metre for the overturning caisson. The draught of the rectangular caisson could be reduced, if sufficient floating stability can be guaranteed without ballast water (e.g. by sponsons). The overturning caisson has sufficient metacentric height, which allows transport without ballast water. The draught of the overturning caisson is therefore lower than a rectangular caisson with the same width. Due to these significant benefits, the construction method and economic feasibility for the concept with rubble backfill is further analysed.

Material savings for a gravity based structure are not free of charge. It is found that rectangular caissons have higher factors of safety compared to overturning caissons having the same width. Therefore, cost savings can also be made by further optimizing material consumption of the considered rectangular caissons, or savings on backfill materials. Furthermore, the caisson shape is not load reducing. The declined back-wall does not result in soil pressure reduction. On the contrary, the caisson heel results in a “trapped” soil wedge which prevents ground to reach an active pressure state. Destabilizing effects caused by soil are therefore slightly higher for a caisson with heel.

In terms of construction technology, equipment is nowadays highly exploited. Current slipforming techniques allow the reuse of formwork for over 100 caissons. Since the slipform is raised with steps of approximately 50mm, the forms are reused for hundreds of times for each caisson. The continuous work-flow results in efficient and a rather constant resource consumption. Due to these developments of equipment and techniques, caissons can be constructed relatively fast. Caisson production rates can be up to 1 each week, for a single slipform-system. A century ago, normal caisson construction rates were longer than one month.

Due to efficient vertical construction methods and high repetition, a horizontal construction method is not beneficial. It is found that this construction method requires approximately 25% more labour for concrete activities. The slipform construction technique allows higher productivity rates and provides convenient concrete quality, inspection and safety. However, a slipform method cannot be applied for constructing an overturning caisson. The irregular geometry results in a rather traditional construction method. Therefore, no intrinsic benefits are found for a horizontal construction method.

The procurement of a job-built formwork system for an overturning caisson is estimated to be more expensive than a slipform system. This is caused by the diversity of the formwork elements, the desired production rate and its low salvage value. For constructing an overturning caisson within a week, almost the entire concrete surface of a caisson must be available in forms. By the proposed construction method and technologies, no quality and safety setbacks are expected. However, as a result, the concrete mixture must be carefully designed and labour consumption on direct construction works increase.

The production costs of an overturning caisson, compared to a rectangular caisson become almost equal after 100 caissons. The costs per caisson remain higher than for a rectangular caisson. In terms of the reference project, for which 60 caissons are required, the overturning caisson is therefore estimated to be economically unfeasible. The overturning concept cannot be demonstrated as the most economical solution. This conclusion is however based on many assumptions regarding labour, assembly time, equipment and building material costs. For instance, if the custom designed formwork system can be reused for multiple projects, the economic potential increases.

Contents

1. Introduction

The reconsideration of an early 20th century construction method arose from engineering curiosity. A Dutch report *Caissonbouw* from the construction company Hollandsche Beton Groep (HBG) gives an overview of caisson applications between 1902 and 1977. It describes a remarkable construction method of the first reinforced caisson structures for quay walls in the beginning of the twentieth century. Economic advantages might be achievable when this concept is applied. Besides economic reasons, it is from engineering point of view interesting to investigate the concept to gain insight in former design approaches.

Many different quay wall structures have been built in the past. Some concepts are abandoned and others are refined and optimized over the last century. The particular concept of our interest is the so called “overturning caisson” (*Dutch: kantelcaisson*), designed by professor Kraus in 1903. It comprised the first reinforced concrete caissons for quay wall application. For several reasons, it was a state of the art concept in that time.

The concept is exceptional due to the horizontal construction method and turning during the immersion phase. For some reason, the concept has been abandoned after a few projects. The caissons might became too large for equipment at that time, or the method was considered as being too complex. Perhaps, other methods were just slightly more favourable. The degree of feasibility of this concept has shifted after more than a century of improvements in equipment and construction techniques. Therefore, this study is done to clarify the technical and economic feasibility of the “overturning-caisson” principle for future projects.

1.1. Document Structure

The document is divided into elements, which are chronologically in time. The document starts analysing the first caissons (1903) and ends with new caisson designs in the last chapters. Chapter 3, 4 and 5 contain three fundamental aspects of the feasibility study; technical requirements, construction technology and economic feasibility. These aspects return in chapter 6, 7 and 8 for the new caisson design. The document structure is thereby as follows:

1. Introduction 2. The first caisson quay walls	*analysis of original caisson and recognition of strengths*
3. Technical requirements 4. Construction technology 5. Economic shifts	*analysis of changes over the last century*
6. Technical feasibility 7. Construction technology 8. Economic feasibility	*analysis of current caisson designs*
9. Conclusions and recommendations	*conclusion regarding all considerations*

1.2. Challenge

This research topic arose from the desire to design caisson quay wall structures more efficiently. This study is performed in order to determine if aspects from the first (overturning) caissons can nowadays be used for caisson designs. The caissons have been designed and built over a century ago, which indicates that the concept is technically realizable. However, developments over the last hundred years have influenced the degree of attractiveness significantly. Because of this, it is unknown whether the concept is currently technically or economically feasible.

New techniques, equipment, materials or methods might be applied to further increase the competitiveness of this concept. On the contrary, some changes might have a negative contribution to the feasibility, such as safety factors and different maintenance demands. Or more stringent design criteria could have resulted in the need to increase the size of the caisson and thus making this option less attractive. The question; "*Is an overturning caisson economically feasible?"* will be answered with this study. And furthermore; "*Does a certain feasible region exist for the application of an overturning caisson?"*

The feasibility can be determined by making a comparison between an overturning concept and rectangular caisson quay structures. In order to determine the feasibility, the concept is reconsidered and adjusted to meet current design criteria. In order to recognise the differences, the following research question must be answered;

Is the overturning caisson concept feasible?	A. Is this caisson shape structurally more efficient?
	B. Is a horizontal construction method beneficial for caissons?
	C. Does the concept allow a simplified launching method?
	D. Do transport conditions become less governing for caisson designs (e.g. due to a reduced draught)?
	E. Does the feasibility of the concept depend on the required number caissons?
	F. Does the feasibility depend on the required retaining height?
	G. Does the concept improve safety and environmental aspects?

In addition, it must be clarified whether technological and economical shifts over the last century advance or obstruct the expected benefits. If the significance a certain aspect is increased, it may be smart to further exploit the particular benefit.

1.3. Significance

This concept might be a more economical solution than current concepts. In which degree the concept distinguishes itself from other structures has to be determined. Analysis of the first caisson concepts possibly results in a cost reduction and a simplified execution of the quay structure. This research shall thereby identify and quantify the advantages and disadvantages of different elements within the construction process of caisson quay structures.

1.4. Design approach and philosophy

The objective of this study is to design a caisson quay wall structure which is more economical than traditional box-shaped caissons. From the perspective of a client, who requests a quay wall, its value depends on its final performance (life cycle costs). The focus of this research is therefore to design a caisson quay wall which performance is equal or better than regular caisson quay walls, at lower costs.

1.4.1. Design objective

The design philosophy focusses on the essential aspects on the structure. It is intended to eliminate superfluous elements (waste) within the construction process. For instance, a quay wall is essentially a soil retaining structure, but certainly not only designed for this function. Different aspects, such as constructability and accessibility result in drastic design changes. By reconsidering these aspects, more straightforward design might be obtained which results in savings in the construction process.

Generally, construction and transportation phases require a considerable part of the total construction costs (De Gijt, 2010). These adjustments are always required up to a certain extent since the structure cannot simply be “wished in place”. However, a more economical L-shaped caisson might be able to reduce the amount of required design adjustments for transient and permanent situations. A comparison of a traditional box caisson and an L-shaped caisson is depicted in figure 1.1 below.

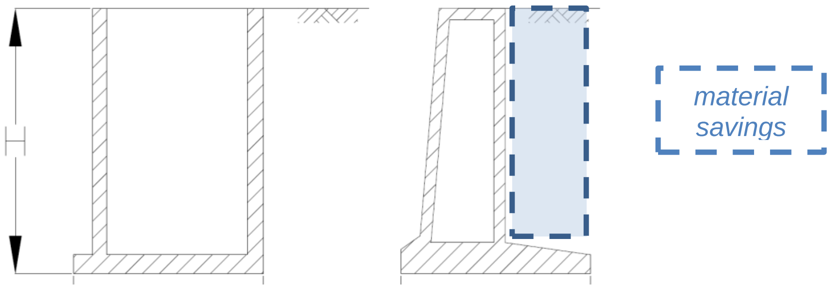


Figure 1.1. Fundamental shape differences

1.4.2. Design approach

A traditional design approach is visualized in fig. 1.2, which is normally an iterative process. On the next page (fig. 1.3), a design model for an overturning caisson is proposed. With respect to the construction of a caisson, adjustments on the geometry to provide sufficient buoyancy and floating stability during transport is a reduction of efficiency. An overturning caisson is expected to have less draught and can therefore be designed for operational conditions in a more straightforward manner.

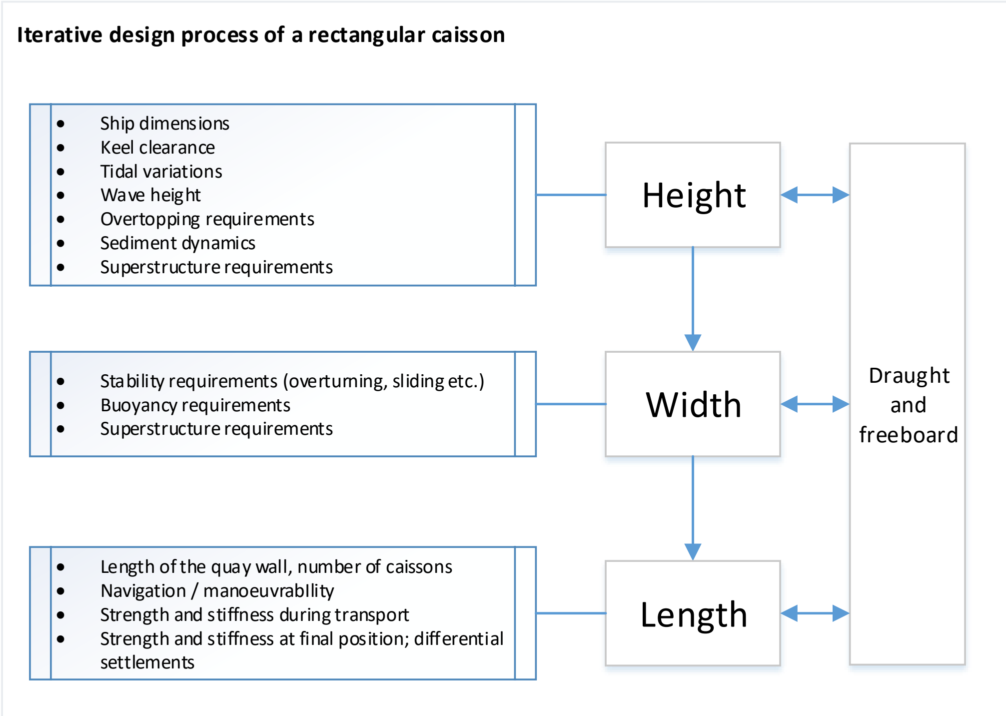


Figure 1.2 – Traditional design method for caissons (iterative)

The main goal is to increase construction and equipment efficiency through material savings and / or shape changes. It is thereby required to prevent sub-optimisations and apply an integrated design approach by considering the whole construction process from design to execution. The proposed design method should result in a less iterative process.

Figure 1.3 shows a linear design path, which would be an ideal situation. Ideally, the least number of iterations and design changes are desired. Adjustments for floatability should influence the design marginally. This scenario results in the possibility to design a quay structure directly for its purpose. However, this is in engineering practice an unrealistic scheme since there are always iterations required to derive an optimal design.

1.4.3. Design requirements and considerations

A quay wall should be designed with an appropriate degree of reliability. The structure must therefore meet strength, stability and serviceability requirements throughout its design working life, without significant loss of utility or excessive unforeseen maintenance.

The generally difficult constructability and maintainability below sea level in combination with severe environmental circumstances results in the need for a proper consideration of the design life and durability aspects. The combined influences of these conditions indicate that it is desired to design a quay structure which demands relatively little maintenance over its service life. Based on these considerations, ordinary berth structures in commercial ports are generally designed for a design life of 50 years or higher (Thoresen, 2014).

The Eurocode (NEN-1990) prescribes a design working life of 50 years for building- and other common structures and 100 years for bridges and other civil engineering structures. The British Standard (6349-1-1) prescribes a design working life of 50 years for quay walls and 100 years for flood defence structures. Considering these recommendations and codes, it is intended to design a durable reinforced concrete caisson, which could safely and efficiently be used over a life time of at least 50 years.

A different design approach could be an optimization in terms of technical and economic service life time, where it is aimed to equalize both life times to reduce costs. However, such a design approach would only be appropriate for specific projects and it would drastically change the feasible region of alternative concepts. The feasibility of the overturning caisson for a design life less than 50 years is therefore not considered.

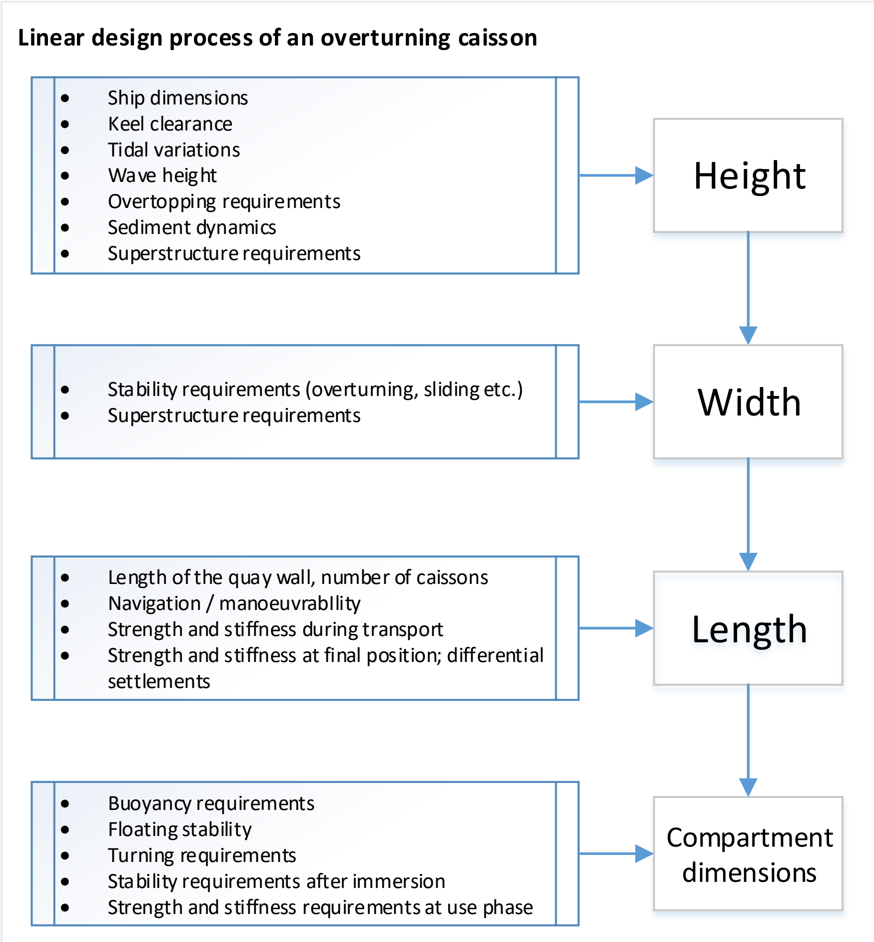


Figure 1.3- Proposed design process for overturning caissons

1.5. Economies of scale (process)

The feasibility of caisson quay wall structures largely depends on the possibility of exploiting scaling advantages. Within the construction industry, economies of scale is generally referred to as the repetition effect. This is an umbrella term which covers the possibility to improve productivity by repeating a process.

The repetition advantages can be categorized by *volume economies of scale* and *learning economies of scale*. Investments in equipment and labour can result in an improvement of the product and / or lower costs per unit. Learning economies lead to improvements in the existing production process or resources. According to a literature review on the repetition effect (A26), this can be illustrated in the following form:

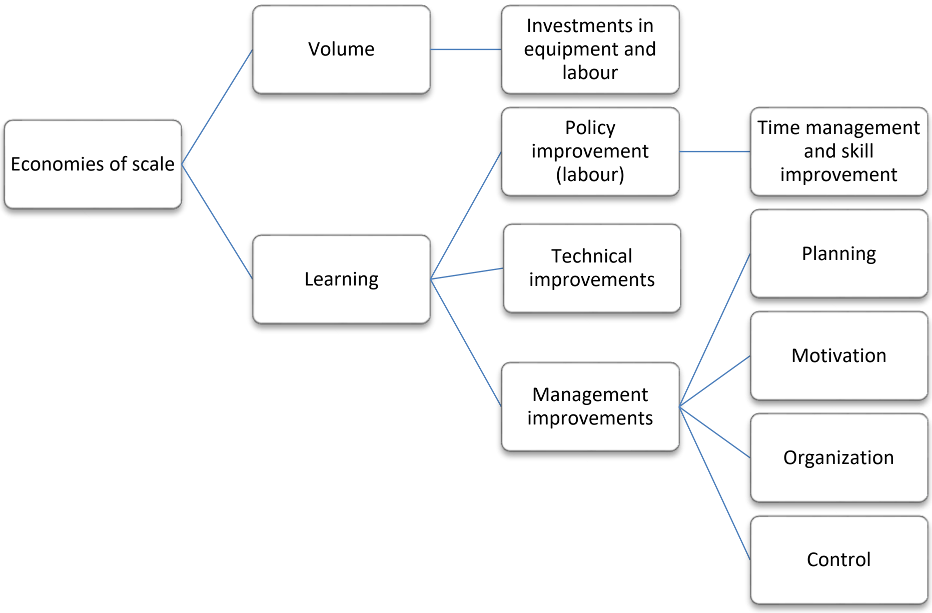


Figure 1.4. Economies of scale, after Pearson and Wisner (1993); ref. [A26]

Economies of scale, and thereby the depreciation and learning effects, are majorly important for the design of a caisson quay wall. The relative influence of investments in equipment reduces if it can be well exploited (depreciation). Constructing a single caisson would therefore be relatively expensive since many resources need to be addressed for a unique product. Learning factors such as man-hour efficiency (learning effect and assembly optimization), planning (critical paths) and shape similarities can reduce the price per unit as well, if the quantity becomes larger.

1.5.1. Volume economies

The initial investment costs of temporary equipment (such as formwork and falsework) spread thinner as the production increases. Therefore, the marginal cost of producing a caisson will generally become lower for each additional caisson. The production costs of an additional caisson can be formulated as:

$$\text{marginal costs} = \frac{\text{change in production costs}}{\text{change in total quantity produced}}$$

The change in production cost consists of fixed costs and variable costs. Fixed cost consist for instance of equipment which is required for construction. The variable costs increase for every produced object and consist of building materials, labour and utilities. The most economical way of production depends on the required number of caissons. When the required number of caissons is relatively high, investments in equipment for reducing production costs shall eventually pay off. These investments must thereby lead to a lowering of the labour, material, equipment or construction site costs.

1.5.2. Learning economies

Improvement of operational times can be achieved by repeating a particular process. The time reduction can be expressed by so called improvement, learning or experience curves. These curves usually show a decrease in man-hours or costs over a number of operations. A typical hypothetical learning curve is shown in figure 1.5. Normally, it has a downwards concave shape. It can be expressed as a logarithmic regression function of the form: $y = a \cdot x^b$. In which a and b depend on aspects such as the complexity of the operations, quality of work preparation and skill level of personnel. A more quantitative analysis of the learning effect for caisson construction is treated in chapter 8; Economic Feasibility.

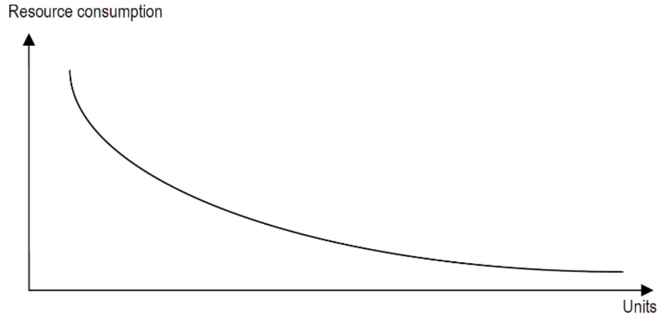


Figure 1.5. Hypothetical learning curve showing a classical concave shape; ref. [A26]

1.5.3. Example of scaling effects

A typical example of economies of scale is the shape difference of capitals on top of a concrete column. In figure 1.6, a typical formwork for a column head is shown for renovation works. For this piece of work, a carpenter had made a relatively simple shape of formwork. Figure 1.7 shows cone shaped capitals which are more efficient in terms of use of building materials. However, the formwork required for these capitals is far more expensive. It is only due to the possibility of repetition (reuse) that this solution can become more economical. Analogously, the most economical caisson shape and construction method depends on the required quantity.

Figure 1.6. Column head for renovation works

Figure 1.7. Multiple cone shaped column heads

1.5.4. Economic benefits

Compared to a traditional box caisson, the overturning caisson has a different balance between fixed and variable costs. Also balances within a cost category will be different. For instance, savings in building materials can result in increased labour costs. This could eventually result in no difference in the fixed costs per caisson. The produced quantity (number of caissons) is not from importance if the fixed costs per overturning caissons are identical to a box caisson. From a production process point of view, the fixed and variable costs can be defined as depicted in figure 1.8.

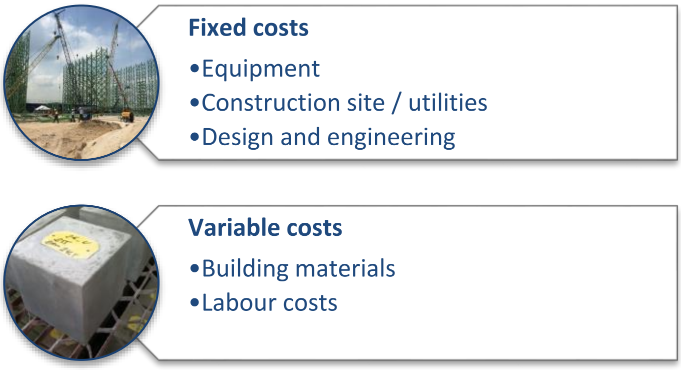


Figure 1.8. Fixed and variable costs from production point of view

Based on the fixed and variable costs distinction, the following three scenarios are possible for which the concept is beneficial:

a. Higher fixed costs and lower variable costs result in an economic design after a certain number of caissons;
b. Lower fixed costs and higher variable costs result in an economic design up to a certain number of caissons;
c. Lower fixed costs and lower variable costs; production costs are always lower.

The balance between these costs determines the feasibility range for the overturning principle. The particular cost balances are thereby also depending on the geographical location of the construction project.

1.6. Economies of scale (structural)

Besides possible scaling advantages due to repetition, a particular structure (caisson) can become more efficient for certain dimensions. This efficiency difference can be explained by the mathematical principle known as the square-cube law (Galileo Galilei, 1638). This law basically describes that, if an object is scaled dimensionally, the relationship between volume and area is non-linear.

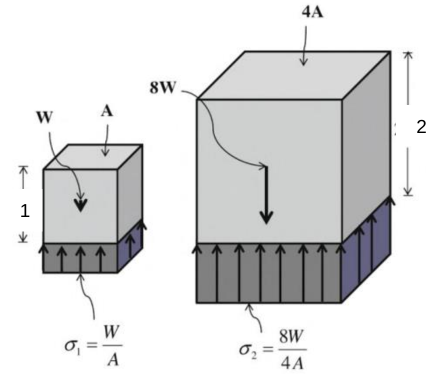


Figure 1.9. Galileo's example of the square-cube law [9]

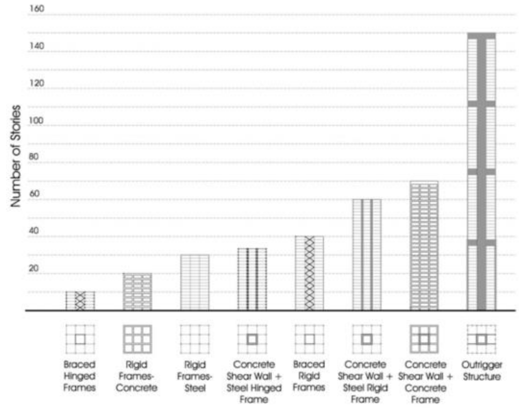


Figure 1.10. Increasing structural complexity for tall building [A27]

This is for example the reason why it becomes more challenging to build a high-rise building than low-rise. The weight of the used materials increase cubically, while the strength of materials increase by the area of the cross section, and thus squared. Therefore, buildings have different efficient structural frameworks for particular heights.

On the contrary, this scaling law is part of the reason why seagoing vessels have become larger over the past century. They become more efficient for increasing dimensions since the hull surface increases less in comparison with their potential deadweight tonnage. Since vessels have become larger, the maximum quay walls size has also increased over the last century.

Although quay heights have increased up to a height of roughly 20 to 30 metres, caisson structures remained quite similar in shape. The shape complexity actually decreased if one compares the efficiently designed caissons from before WWII with those constructed afterwards. This is unexpected since the required volume of construction materials increases disproportionally and loads could be transferred more efficiently for larger retaining heights.

2. The First Caisson Quay Walls (1903)

2.1. The port of Valparaíso

The first caisson concepts originate from a harbour improvement project in Valparaíso (Chile) in the beginning of the twentieth century. Valparaíso had become an important port due to its key position on trade routes through the Eastern Pacific. The combination of the geographical position and the second industrial revolution resulted in a strong growth of harbour activities. Due to relatively shallow water conditions, only smaller vessels were able to berth near the city in the late nineteenth century (fig. 2.1.).This resulted in the desire of an extensive harbour improvement which consisted of thousands metres of new quay walls. The primary goal of the project was to allow more and larger vessels to berth safely in the bay.

Figure 2.1. Panoramic view over the bay of Valparaíso (late 19th century)

Unfortunately, the construction works delayed due to a major earthquake which struck Valparaíso in 1906. There was extensive economic damage and thousands of people died. A few years later, the opening of the Panama Canal in 1914 reduced the port activities drastically. This combination of events resulted in such a large setback that the port lost its vital role as major transhipment point. Because of this, the originally planned harbour improvement works were never fully executed. However, the quay wall designs were very well documented by *Comision Kraus*, which resulted in the application of the particular caisson designs in other ports over world (appendix A).

2.1.1. Quay wall design for Valparaíso

From geotechnical point of view, the bay was characterized by relatively hard soil and rock bottom, with similar cross-sections near the shoreline. The combination of a large total quay length of over 3 kilometres and a desired quay height of 14 metre resulted in favourable conditions for the application of caissons. *Comision Kraus* recommended this structure extensively for the improvement project of the port of Valparaíso. The fundamental difference between earlier caisson quay concepts was the application of reinforced concrete as building material. Preceding caissons quay walls had only been built by Romans with materials such as wood.

There were mainly two quay structures designed which are worth mentioning. One type is referred to as "*cases of armed concrete"* and the other as *"floating blocks of masonry"*. The cases and floating blocks, nowadays referred to as *caissons*, were state of the art civil engineering structures at that time. Professor Kraus recommended reinforced concrete caissons for sheltered sites of the harbour and masonry caissons for unsheltered quay parts. It was noted in the report that masonry structures were commonly used and that it had already proven its capability to resist great pressures from beating of the sea.

2.1.2. Reinforced concrete caissons (*cases of armed concrete*)

From the report of the commission Kraus (1903), the main design considerations seemed to be to resist the beating of the sea and provide enough rigidity for transport and placement. Furthermore, enough strength was required to resist the water pressure during transport.

In order to satisfy these conditions, specific concepts have been made for different locations. The designs differed from robust masonry cases up to economical L-shaped caissons. Wave actions and rigidity during transport were the most important design conditions for all concepts.

With respect to the L-shaped caissons, the compartments were filled with so called “weak concrete” at the final position which guaranteed its stability and strength. The concrete fill was also beneficial for enlarging the resistance against ship collision and berthing forces. The reinforced concrete wall thickness of 150 millimetres (near water level) in combination with a maximum concrete compressive strength of 15 N/mm^2 was therefore not sufficient to ensure a durable and save structure by itself.

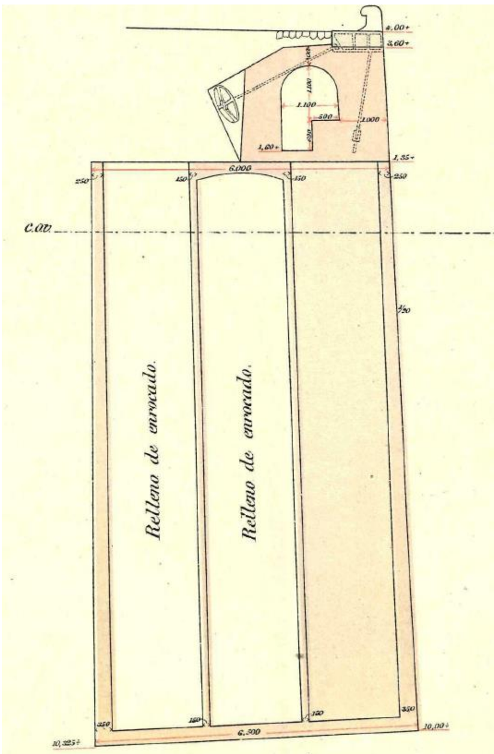


Figure 2.2. Box caisson (Valparaíso, 1903)

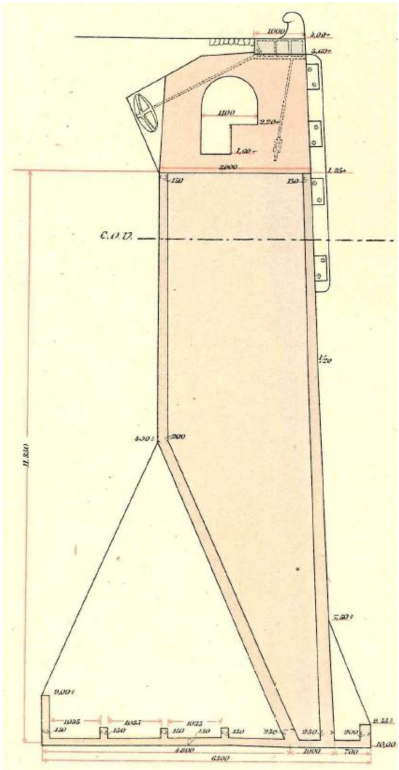


Figure 2.3. L-shaped caisson (Valparaíso, 1903)

2.1.3. Masonry caissons (*floating blocks of masonry*)

For unsheltered site of the harbour, which was prone to high wave forces, it was decided to use a more robust design. This resulted in a caisson made out of masonry rubble stone. It was planned to be floated to its final position and then filled with a cheap material, as for instance "sand concrete". This was similar to the planned breakwaters.

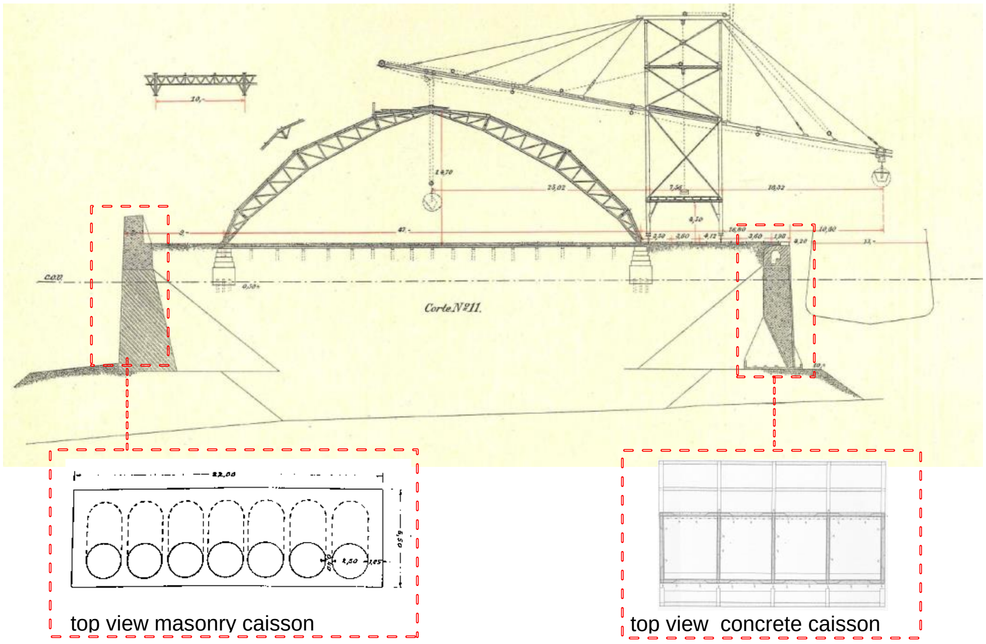


Figure 2.4. Sheltered quay wall (right) and unsheltered quay wall (left) designed by prof. Kraus for the Dock el Baron in Valparaíso, Chile (1903)

The outer quay walls have been designed to perform a double duty. It must take the place of a real breakwater in cases of storms, and has to function as a mooring place in normal conditions.

A remarkable aspect is that masonry cases, reinforced blocks and reinforced L-shaped caissons all have a width of 6.50 metre. Apparently, this was considered to be a safe value, although the weights of concepts differ considerably. The gravity based structures have therefore different factors of stability. Furthermore, it can be concluded that the bearing capacity was not influencing the decision. Nevertheless, it was noted that soil improvement was required for some parts of the harbour basin which consisted of mud.

The dimensions of the walls of the breakwaters were determined with relation to the strength of the waves. Global measurements (by means of a dynamometer) resulted in an advisable wave pressure of 30 tonne per square metre (300 kN/m^2) at the height of the surface of the sea. At that time, it was observed that a relatively large *total of great resistance* could be obtained when masonry breakwaters were applied. The breakwaters had cylindrical shaped compartments, which were mainly designed for an even spread hydraulic water pressure during transport.

The masonry cases were not able to float by themselves. Therefore sponsons or air cases were designed to increase the buoyancy and floating stability during transport. Furthermore, a steel frame was required to improve its strength during floatation. Although the sponsons could be reused, the masonry caissons were more costly than the reinforced or "*armed*" concrete alternatives. For this reason, the masonry blocks have only been designed for locations prone to high wave loads.

2.2. Caisson characteristics

This section regarding caisson characteristics is based on the harbour improvement report of Valparaíso by *Comision Kraus* (ref. [1]), which was published in 1903. The analysis reveals opportunities and threats of the concept. Lessons learnt from analysing the first caissons are used for further caisson concepts.

2.2.1. Caisson shape and dimensions

For the largest part of the quay, the retaining height (incl. superstructure) was 14.00 metre. The standard type of sheltered quay wall was designed to function from -10.00m to +4.00m relative to chart datum. Other quay walls were designed without superstructure and had a sloping revetment up to +5.00m CD. The width of the caissons varied from 3.00 metre at the top and 6.50 metre at the bottom (baseplate). To sum up, the main dimensions of the caissons for the largest part of the sheltered quays were:

Caisson dimensions	
height	11.35m
width	6.50m
length	10.00m
Retaining measures	
total height	14.00m
water depth:	CD -10.00m
coping level:	CD +4.00m

The shape and caisson characteristics are discussed using the drawing (fig. 2.5) and corresponding numbers of each element. Noteworthy is that for each reinforced element, the concrete cover amounted probably 10mm. The impact of this aspect on durability is discussed in chapter 3 and appendix C.

Superstructure (1)
The superstructure was made out of ordinary masonry. A canal had been economized for placing water and gas-pipes and the conducting electric cables. The pipes were designed to be placed above an altar, while a deeper part of the canal could serve for workman.

The total height of the superstructure amounted 2.65 metre. The superstructure starts from +1.35 metre above mean sea level, guarantying relatively easy placement and maintenance works. This is 0.35 m above the anticipated value for extreme sea level. The top level of the structure was CD +4.00m.

Backwall (2)
Caisson walls were tapered; the outer walls started for at a thickness of 250 millimetres at the bottom and attenuates to 150 mm at the top. This reduced the amount of reinforced concrete and thereby the total construction costs. The declined lower part had various benefits.

Compartments (3)
The caisson was designed with four compartments (3 inner walls). The thickness of inner walls amounted 150mm. Reasons for partitioning the caissons were; (1) the lateral walls had to be able to resist the pressure and (2) they had to diminish the movement of ballast water during transport. The first 4 metres of the compartments were originally designed to be filled with concrete which was casted under water. In this way, the walls were not

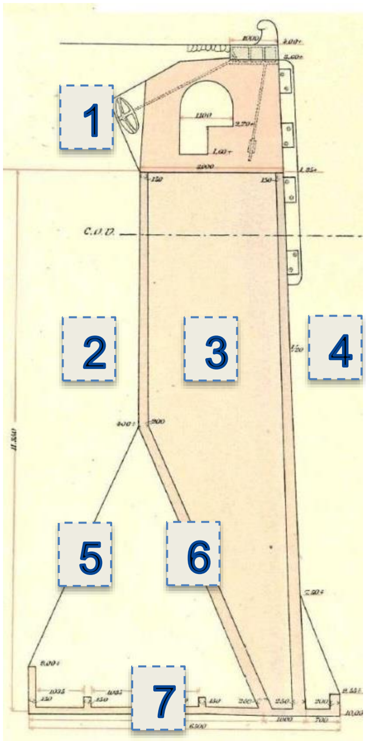


Figure 2.5. Main elements of the caisson, indicated by numbers

exposed to high pressures. The remaining of the compartments were designed to be filled with a concrete made out of 200 kg/m^3 of Theil lime, which probably particularly desired due to its ability to resist actions of sea water due to a low iron content[1].

Frontwall (4)
The frontwall was designed with an inclination of 1/20, which ensured its stability during placement and its service life. Similar to the backwall, the wall thickness varied from 150 millimetre at the top, to 250 millimetre at the bottom of the caisson.

Counterforts (5)
The baseplate and longitudinal walls were connected by means of five triangular consoles, or counterforts. These provided strength and rigidity of the connection of the baseplate between the compartments itself.

Declined back wall (6)
The declined back-wall allowed a nearly vertical floating position after turning. Due to this declination, added ballast water acts beneficial for a vertical floating position. The slender design in combination with a distributed weight over the height of the caisson resulted in a low draught (relative to caisson height). The horizontal (fig.2.9; $\alpha \approx 10°$) floating position resulted in a draught less than 4 metre.

The declination also resulted in less use of reinforced concrete. For instance, a larger wall-span for the side walls could be realized. Namely, the lower part of the caisson, which encounters the highest pressure during immersion, has the lowest wall-span. The upper compartment walls have the largest wall-span of 3.00 metre. Furthermore, no further increase of wall-pressure occurs after turning. This is due to its triangular shape, which causes water pressure inside the compartments to rise faster than outside (in contrast to the described La Goulette caisson in appendix A).

According to Tsinker (1997, 2014), the rear wall and base slab were designed to reduce the horizontal soil thrust on the wall. In practice however, the declination does not result in a reduction of horizontal soil pressure since the soil is trapped within the heel and counterforts. The applicable soil pressure theory for overturning caissons is addressed in more detail in appendix L.

The declination probably affected the casting method of the backwall considerably. Casting under an angle of 20 degrees could only be realized with a very low slump concrete mixture or with an additional formwork *(tegenkist)* which closes the wall. In this case, the buoyancy of the formwork must be considered. This was probably challenging, since internal ties (which could connect both formwork sides) were probably not well developed to resist substantial hydraulic pressures during immersion.

Baseplate (7)
The baseplate was made out of reinforced concrete. It had a slight concave shape in order to facilitate its stable position in the rubble bed. This guaranteed two supports in case of bed variations. The rubble prism behind the wall provided anchorage to the quay.

Forces acting on the plate are mainly transferred to the perpendicular (compartment) walls and counterforts. The orthotropic plate (depicted in figure 2.6) is, due to its shape, highly efficient in terms of material use. These reinforced concrete stiffeners required formwork with a lot of corners (stiffener size was just 150mm). Attention was required in terms of reinforcement (cutting, bending and placement), formwork construction and concrete pouring. Especially if one realizes that the baseplates were casted in vertical position. This implies that concrete kickers *(opstort)* could not be used for constructing the stiffeners. If the baseplate was therefore casted at once, the small stiffeners on the plate where probably prone to honeycomb formations (*grindnesten*).

[1] Practical treatise on Limes Hydraulic Cements and Mortars, Professional Papers of the Corps of Engineers (U.S.A.), No 9, by Q.A. Gillmore, A.M., fifth edition, 1879

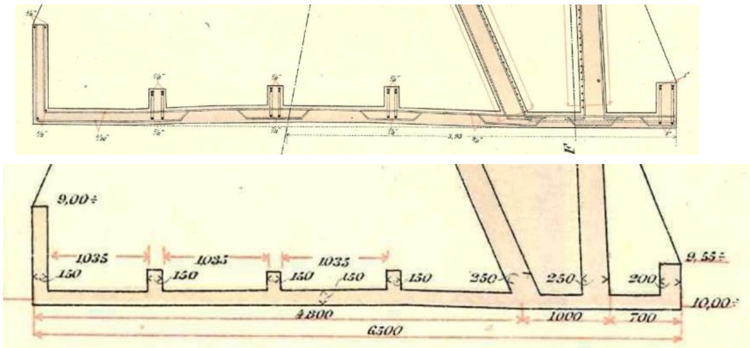


Figure 2.6. Detailed view of the reinforced concrete baseplate (original drawing) above: reinforcement layout, below: cross-sectional dimensions

The total width of the baseplate amounted 6.50 metre, which is approximately equal to 0.6H. The width / height ratio is thereby small compared to current caisson designs, which are generally in the order of 0.8H. Especially if one includes the superstructure (+2.65m), the total width/height ratio becomes more or less 0.5.

2.2.2. Material properties

The concrete strength was determined based on experiments conducted before the caisson was designed. The experiments had been performed with beams and plates of an ordinary form and composition. The rupture tension had been determined as well as the deformation undergone by the material according to the forces to which it had been subjected. Rupture of “*tension iron*” appeared to be not governing at these tests, on which it was concluded that the “*limit of the elasticity*” of the plate was decisive.

The concrete compressive strength (f_{ck}) had been determined to be 15 N/mm^2 and its characteristic tensile strength (f_{tk}) amounts 2.0 N/mm^2. The capacity of iron totalled 250 N/mm^2 (f_{yk}).

2.2.3. Reliability

A safety factor of 2 was applied on material properties with temporary loading (e.g. during transport). Where a factor of 2.5 was applied on material properties with long term loading (e.g. final at stage). Combined with the mentioned safety factor of 2.5, a design compressive strength of 6 N/mm^2 remained.

2.2.4. Construction method

The construction method was planned in great detail by *Comision Kraus*. The most striking aspects of this approach were the horizontal casting and launching of the caissons. The elements were designed to be prefabricated on shore in horizontal position and transported to water by custom made lorries. Major benefits of this method are the costs savings of a (floating) dry-dock and horizontal casting. A significantly larger area could be casted as floors instead of walls. Floors could be casted more economically since there was less falsework and formwork required and it required less labour to place reinforcement and cast the concrete.

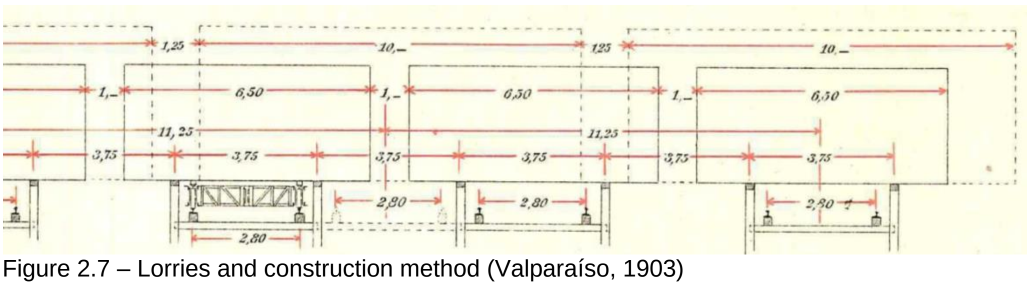


Figure 2.7 – Lorries and construction method (Valparaíso, 1903)

Comparing a theoretical vertical construction method with the applied method provides insight in the advantages that could be obtained. Significantly more walls would be required if the caisson would be constructed in vertical position. Differences between casting walls and floors are:

- Horizontal forms hold *weight* of fresh concrete;
- Vertical forms hold *pressure* of fresh concrete (which can be equal to hydrostatic);

Therefore, the maximum casting height of walls can be limited by formwork pressure. Horizontal casting is on the other hand not limited by a certain formwork pressure since it only has to carry the weight of the fresh concrete (fig 2.8 and 2.9). A concrete slab having a thickness of 250mm would therefore exert a vertical pressure of 6.25 kN/m^2, while a concrete wall, having a height of for instance 3.00m, exerts a horizontal pressure of 3.00 x 25 = 75 kN/m^2. Hence, the length of a concrete element could be increased without pressure restrictions, while a wall casting height of just 3 metres already results in considerable formwork pressures. Especially in the years of the design of the first caissons, when little experience and limited techniques were available, this aspect could have influenced the design considerably.

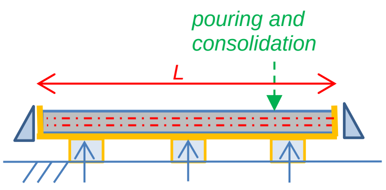


Figure 2.8. Formwork pressure: weight of slab

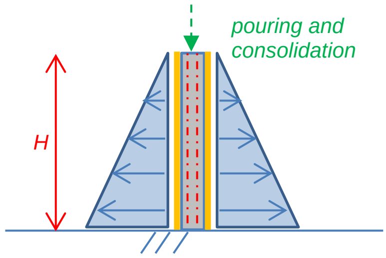


Figure 2.9. Formwork pressure: hydrostatic pressure

Additional differences between figures 2.8 and 2.9 are the accessibility and working conditions for activities such as; reinforcement fixing, concrete pouring and consolidation. The horizontal casting method improves these conditions considerably when little techniques are used. Also the concrete free-fall distance could easily be reduced in case of horizontal casting, which prevents segregation of aggregates.

The caissons were planned to be constructed onshore. This resulted in the possibility to construct caissons simultaneously. The report revealed that it was intended to construct 171 regular blocks at once and 320 in total. The sheltered quay wall caissons were planned to be built in portions of 114 caissons at a time and 260 in total. The construction of over 100 caissons simultaneously was the most economical solution in 1903, this is however currently unusual.

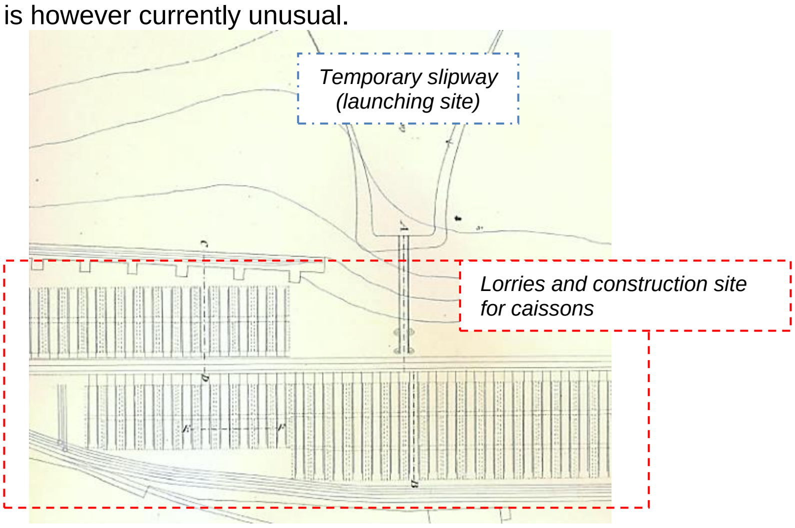


Figure 2.10. Execution plan of reinforced concrete caissons (half of the construction site)

2.2.5. Launching method

Launching was meant to take place with help of temporary timber bridges. The declination of this slipway was designed to be 1:10, which was similar to the floating position of the caissons, which was approximately 10 degrees. This resulted in low concrete tensile stresses during the launching process; it was calculated that stresses of 0.45 N/mm^2 would not be exceeded. The slipway launching method was considered to be the most economical solution due to shallow water conditions near the construction site.

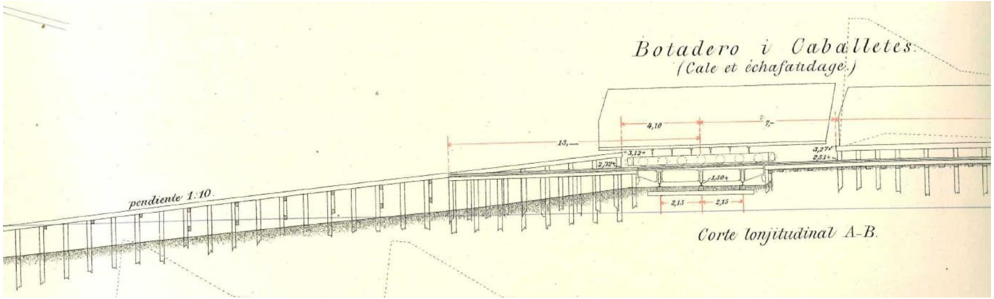


Figure 2.11. Transport and launching plans of the caissons (Valparaíso, 1903)

The maximum draught of the overturning caisson was roughly 3.80 metres (ref 1), which is less than 0.35H (where H = caisson height). This is a significant reduction compared to rectangular caissons where a rule of thumb of 0.5H can be used[2]. Savings can be made during construction of the caissons, whether it is decided to build the overturning caissons on ground level, where slipway launching or a ship-lift is required or when it is decided to use a temporary dry-dock. The particular launching method shall always be more economic due to the reduced depth.

Loads induced by slipway launching can be estimated to be equal to the hydrostatic pressure in combination with a certain additional dynamic load (Tsinker 1997). This additional hydrodynamic pressure (P_{qs}) on the caisson is usually treated as quasi-static. Its value can be estimated as a function of an empirical coefficient (c) and the speed of caisson launching. The coefficient is conservatively estimated to be in the order of 0.85 – 1.00, where a speed (V) of 5 m/s is usually assumed for preliminary design. This results in the following expressions for the estimation of added dynamic pressure:

$$P_{qs} \approx cV^2$$

$$P_{qs} \approx 1.0 \cdot 5.00^2 = 25 \text{ kPa} = 25 \text{ kN/m}^2$$

This pressure is a considerable amount of additional load, but is not necessarily a dominant design load. In other words, slipway launching is not necessarily influencing the structural design and geometry.

2.2.6. Transport

One of the most striking aspects of the original caisson is its equilibrium floating positions. The first floating position was under an angle of approximately 10 degrees. This could be achieved through an economic design in terms of material use and a slender caisson (height / width ratio ≈ 2).

The floating position after launching is shown in figure 2.11. A benefit of this position was that it corresponded to the desired horizontal construction and launching method. Also, the caissons could be floated in shallower water conditions, which reduced the required length of the slipway and/or the dredging works. The draught of the original caisson during transport amounted approximately 3.90 metres.

The low relative draught was caused be two major design characteristics. Firstly, its limited weight resulted in a small amount of displaced water (∇). Secondly, the centre of

[2] This is valid if one assumes 20% concrete and 80% compartment volume. A concrete/water density ratio of 2.5 gives a draught of 0.20 x 2.5 x H = 0.50H

gravity (G) was located near the middle of the caisson (fig. 2.12.). The angle of floatation would further decrease if G would be shifted towards the middle. The small angle of floatation was therefore obtained by the enlarged upper part of the caisson. The slope of the back-wall itself was not important in this floating phase, but the weight distribution

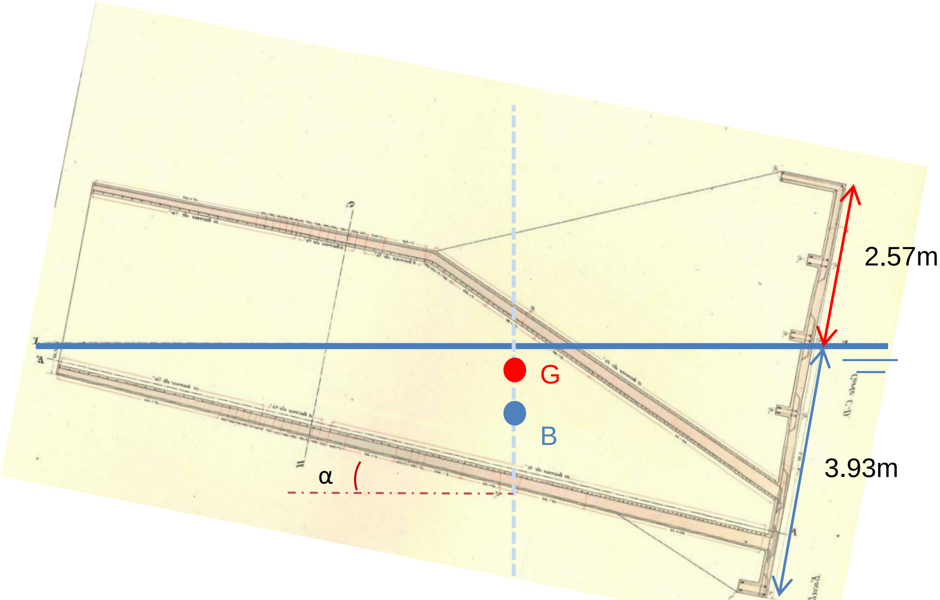


Figure 2.12. First floating position of the caisson

Due to the relatively large moment of inertia of displaced water, the horizontal floating position is rather stable. Form appendix B.9, the metacentric height of the simplified model appeared to be in the order of 3 metres above the centre of gravity, which is more than sufficient in terms of transport requirements. The caisson will therefore have the ability to remain upright after small disturbances. Considering this, the metacentric height is expected to be a less significant design aspect, compared to rectangular caissons.

2.2.7. Turning & Immersion

When ballast water is applied, point G shifts and enforces a different floating angle. The volume of displaced water will increase and its centre of buoyancy (B) shifts horizontally to the position G'. When ballast water is continuously added, rotation will slowly progress until the heel of the caisson scoops water. The heel will quickly be filled with water and the floating position will become more or less vertical.

In example, figure 2.13 shows two floating objects. The left object is in equilibrium and has a horizontal floating position. The right object has a shifted centre of gravity which implies that a rotation is initiated if no other external force is applied. The shape of the displaced water must change in such a way, that its centre of buoyancy (B) returns above or below the shifted centre of gravity (G').

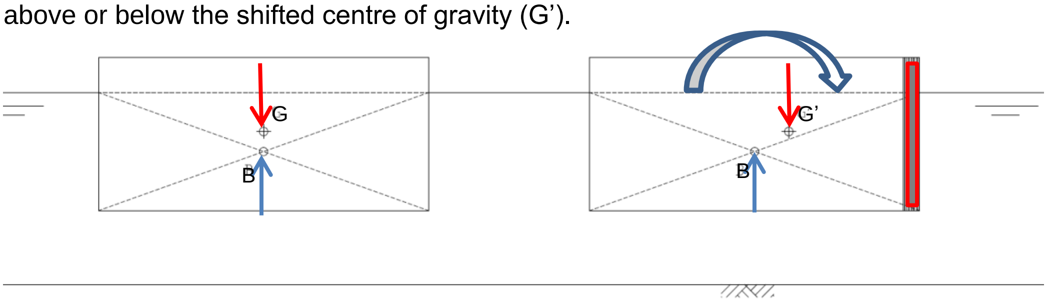


Figure 2.13. Change of floating position

At a particular horizontal shift of G, equilibrium cannot be found by minor changes of the floating position. Turning of the object is then required to find equilibrium. In case of the overturning caisson, a new equilibrium point shall be reached with point G positioned vertically below point B. The enlarged compartments at the upper part of the caisson contribute to floating stability after turning and during the immersion process. In appendix B.10, an analysis of allowable positions for the centre of gravity is made. It is found that the relative displacement influences this region. The more relative displacement, the smaller the allowable region for G. Such a region can be used as tool for determining the geometry of an overturning caisson.

The first overturning caissons were designed for a nearly vertical floating position after turning. During ballasting, its floating position became more and more vertical. A vertical position was obtained in combination with approximately 10% of the compartment volume with water. This was in combination with a remaining freeboard of approximately 1.65 metre, which implies that during low water level, a remaining keel clearance of 0.30 metre was obtained. Irrespective of its vertical floating position and keel clearance, it was decided to assist the caissons during turning and immersion with a sheerleg (fig. A.12). The used sheerleg is estimated to have a capacity of 50 tonnes, which is roughly a quarter of the caisson weight (220 tonne).

The vertical floating position could be obtained by a well-thought-out geometry. A simplified model with straight walls (fig. 2.14) has no intrinsic vertical floating position after turning. The absence of a toe and the rectangular shape of the compartments result in a floating equilibrium which is not vertical, and shall not become vertical due to the ballast increments.

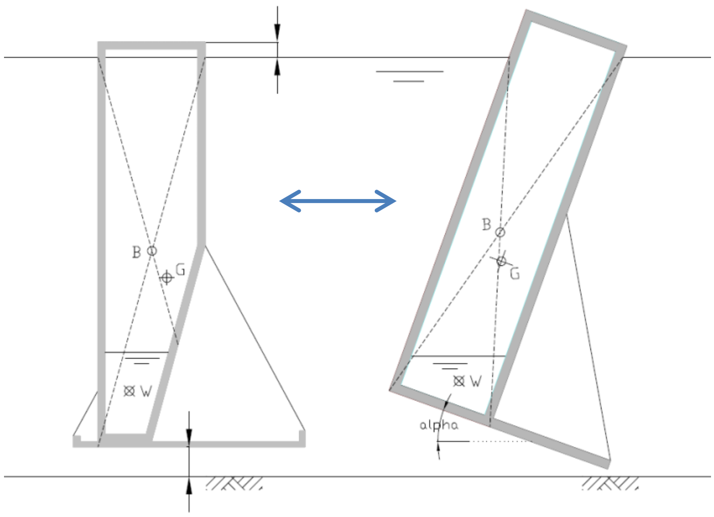


Figure 2.14. Schematic position after ballasting of the original caisson (left) and the simplified model (right)

The caisson with a rectangular shaped compartment has its buoyancy point relatively close to the front-wall. The original caisson, with a triangular back-wall shifts the buoyancy point towards the back-wall, which is desired for a vertical floating equilibrium. Therefore, the declination functions in two manners:

1. The buoyancy centre (B) shifts backwards;
2. Ballast water shifts the centre of gravity (G) forwards;

The buoyancy centre is well-positioned due to declined back-wall. This can be seen by comparing a rectangular geometry with a triangular which have equal surface areas. If these surfaces represent a certain displacement, both have different geometric centres (centroids), which indicate a different location of the buoyancy point.

A comparison between those geometries is depicted in figure 2.15. Both having the same area (H x W). Nevertheless, the buoyancy point for the rectangular geometry, with a straight back-wall is positioned at a horizontal distance of W/2, where the buoyancy point for the triangular geometry is positioned on a horizontal distance of 2W/3 from the front.

The centre of mass also changes due to the adjusted geometry. However, the distance from the front-wall remains equal for both geometries. The remaining differentiating aspect is a change in length of the walls; where the hypotenuse is by definition longer than the adjacent. The increased mass of the declined back-wall results in a slight shift towards the (undesired) back of the caisson. The mass increase is however less significant than the shape change.

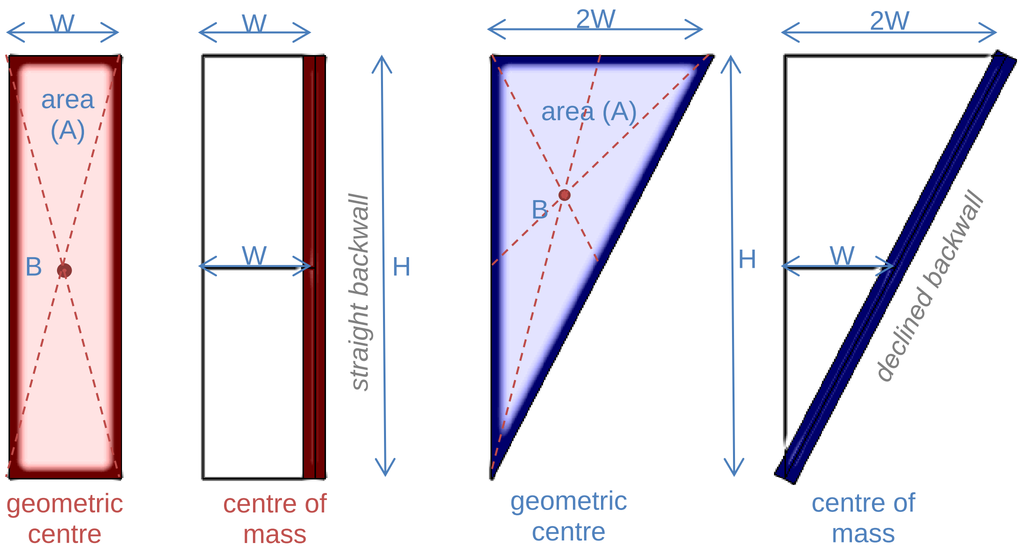


Figure 2.15. Declined versus straight back-wall: geometry differences affecting the floating position

Nevertheless, the declined back-wall is not compulsory to obtain a vertical floating position. It can also be obtained by designing an additional compartment or ballast tank. This makes that the geometric centre (or buoyancy centre B) can be adjusted independent of the centre of gravity (G), which simplifies the vertical positioning.

2.3. Structural capacity

Loads during immersion were governing for the structural design of the concrete caisson. This is expressed in the report as follows:

> *"The sides and the bottom of this case are so calculated that they can resist the corresponding pressure of the water at this depth, a pressure will naturally not increase when the case has a greater depth, owing to the water introduced."*

The case was designed to resist the hydrostatic pressure directly after the turning operation. This state is obviously governing due to the caisson geometry and the increasing hydrostatic pressure from the presence of ballast water. In other words, the water level inside the compartments rises faster than the immersion rate of the caisson.

The walls could be designed with a thickness of 150mm at the top and 250mm at the bottom of the caisson. This limited wall thickness was possible due to the limited hydrostatic pressure on the top of the caisson. The schematic load case on the front-wall of the caisson is as follows:

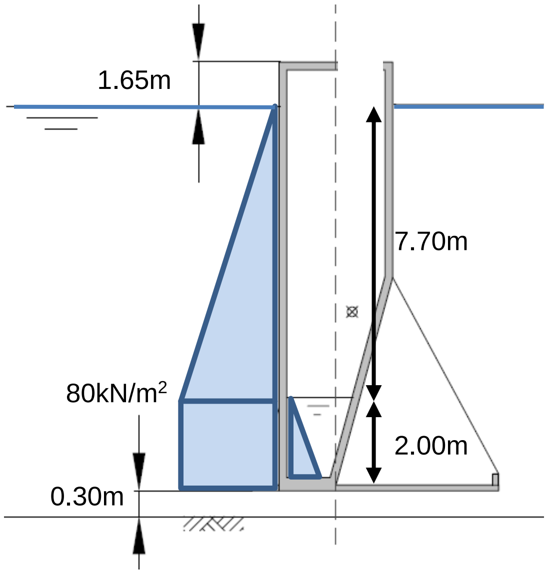


Figure 2.16. Hydrostatic pressure during immersion

The original strength parameters for the reinforced caisson are listed in the table below.

Material characteristics (1903)	**Value**
Concrete	
Concrete compressive strength	15.00 N/mm^2
Concrete tensile strength	2.00 N/mm^2
Overall safety factor (during transport)	2.00
Design value of concrete compressive strength	7.50 N/mm^2
Design value of concrete tensile strength	1.00 N/mm^2
Reinforcement	
Steel ("iron") yield strength	250 N/mm^2
Bar diameter (lower part of caisson)	1/2" ≈ 12.7mm

The structural capacity is analysed in appendix B. The most striking aspect from the analysis is that the shear capacity of the walls is just sufficient at the critical depth. The walls are thereby rather thin, but nevertheless able to resist the hydrostatic pressure. After immersion, the compartments were filled with a low quality concrete. This had to result in a durable structure with limited use of reinforced concrete.

2.4. Economic characteristics

The harbour improvement project in Valparaíso consisted of several quay structures. Especially the largest quay site was designed with great precision. The most economical solution, which also had to fulfil all requirements, seemed to be a reinforced concrete L-shaped caisson. This carefully designed caisson type was estimated to cost about 1445 Chilean pesos ($), where the rectangular caisson type was estimated to cost 2500 pesos per running metre. The costs of excavation and the rubble backfill were initially not included for both cases.

The report revealed a list of prices and elements from prior the tender phase of the quay walls. The cost of rubble stones are found in a cost estimate supplement and added to the existing table. Based on this, the total quay cost per running metre (excl. excavation and dredging) is calculated. This cost overview is presented in table 2.1.

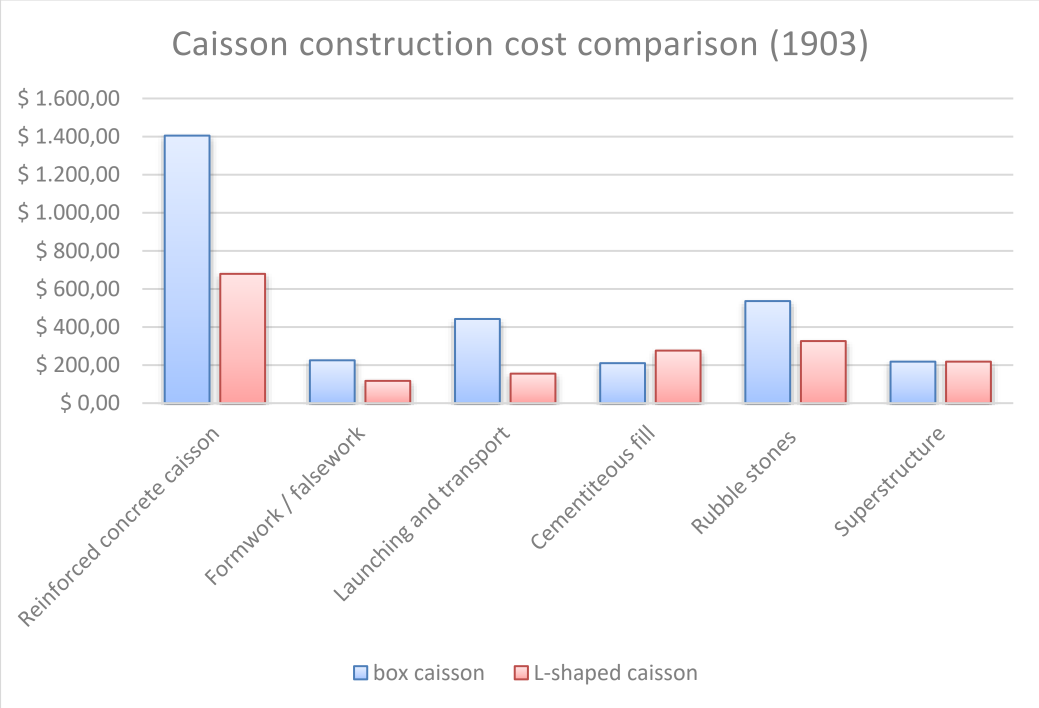
Price per line metre quay (1903)	*Box caisson*	*L-shaped caisson*
Floating case of armed concrete	$ 1 405,-	$ 679,-
Filling up of sand concrete	$ 210,-	$ 276,-
Superstructure composed of masonry with a mortar of cement, including the coping-stone	$ 218,-	$ 218,-
Construction and keeping in repairs of the moulds and launchers for the floating cases	$ 225,-	$ 117,-
Operation of launching, transporting and placing the cases, with mortmain of the material needed	$ 442,-	$ 155,-
Rubble stones for compartment fill, backfill and bed foundation	$ 536,-	$ 326,-
Total Chilean pesos ($):	$ 3 036,-	$ 1 771,-

Table 2.1. Caisson quay wall costs per running metre

It can be seen that the rectangular caissons were about 3 times more expensive to launch, transport and place compared to the economic L-shaped solution. However, both caisson types have been designed for this project. This reveals that the L-shaped caisson design was not forced, for instance, by shallow water conditions during transport. The L-shaped caisson was purely designed to satisfy all the conditions at the lowest possible price.

The rectangular caissons (shown in figure 1.1) were only designed for locations with a generally rough sea. The L-shaped caisson was particularly designed for the sheltered parts of the harbour. The planned sheltered quay wall length totalled 3481 metres, which is, also for today's standards a large quay length.

Based on the original cost estimate, the differences per running metre quay for the sheltered harbour are expressed in the bar chart below. It can be seen that the largest savings were made by reducing the amount of concrete. The material savings thereby resulted in a chain reaction and reduced the costs of other components as well.



From the volumes and given prices per metre quay, the following price per units has been calculated:

Price per unit (1903)	*Price per element*	*Quantity*	*Price per unit*
Floating case of armed concrete	$ 679,-	9.60 m^3	71,75 \$/m^3
Sand-concrete compartment fil	$ 276,-	23.00 m^3	12,00 \$/m^3
Superstructure composed of masonry with a mortar of cement, including the coping-stone	$ 218,-	6.75 m^3	32,30 \$/m^3
Construction and keeping in repairs of the moulds and launchers for the floating cases	$ 117,-	75.40 m^2	1,55 \$/m^2
Operation of launching, transporting and placing the cases, with mortmain of the material needed	$ 155,-	9.60 m^3	16,15 \$/m^3
Rubble stones for backfill and foundation bed	$ 326,-	65.00 m^3	5,00 \$/m^3
**Prices in Chilean Pesos ($)*			

The price per unit of reinforced concrete was exceptionally high. It was for example more than 5 times higher than the cost of “sand-concrete” which is a mixture of sand, cement and water. It was also over 10 times more expensive than the procurement of rubble stones. Due to these large price differences, it was highly beneficial to save on the amount of reinforced concrete.

2.5. Synthesis

The counterfort caissons resulted in a material efficient retaining structure. The application of reinforced concrete was consciously considered. The optimization resulted in materials savings, but as a consequence, a relatively complex shape had to be constructed. The complex shape had no significant influence on the construction costs. A rectangular caisson for the same port expansion project had a relatively simple shape, but was estimated to cost almost twice as much. The reason for this cost difference can be addressed to material and weight savings.

The minimum wall thickness amounted just 150mm, where the lower walls reached thicknesses up to 250mm. Although relatively thin walls were constructed with inferior material properties, it was sufficient for the temporary loads during transport and immersion. After placement, the compartments were filled with a sand-cement mixture which increased structural longevity.

For a regular caisson, the compartment and caisson dimensions are intrinsically linked to the draught magnitude. For the overturning caisson, the potential water displacement by the compartments is only from importance during immersion since the caisson has two floating equilibrium positions. The draught restriction in vertical floating position is limited by the water levels and depth during placement.

The savings of building materials resulted in a snowball-effect for other components in the construction process. Also a horizontally constructed rectangular caissons was significantly more expensive. Compared to a rectangular caisson, the benefits of the counterfort concept were:

Benefits of the caisson (1903)	**Cause**	**Result**
Reinforced concrete	Economically designed caisson due to: - Counterforts (L-shape); - Declined backwall; - Thin and tapered walls; - Orthotropic base plate;	Over 50% savings on reinforced concrete costs
Reduced amount of rubble stones required	Less foundation bed and backfill required due to reduced weight	Approximately 40% savings of rubble stone
Temporary equipment (e.g. formwork)	Less walls, resulting in less formwork area; Less construction height, requiring less temporary supports;	Over 50% savings on formwork and falsework costs
Launching	Less weight and less draught due to the economical design resulting in a relatively simple slipway launching method	Almost 65% savings on launching costs
Transport	The design resulted in a small width-to-height ratio	Low draught

Results compared to a rectangular caisson designed for the port of Valparaíso in 1903, having the same retaining height.

The reduced launching costs were obtained by the reduced weight and shape of the caissons. Due to the efficient use of materials, the caissons could be constructed with an element length of 10 metre. This was approximately 3 times longer than the rectangular caissons, which reduced the number of launches drastically. All in all, significant advantages could be obtained. The low draught (approximately 1/3 the caisson height) of the overturning caisson is also listed as an advantage, while the draught of the first block caisson was actually lower. However, the relative draught is still low compared to current rectangular caisson designs.

Developments over more than a century of caisson construction resulted in a shift of benefits and priorities. Somehow, the concept became less attractive to build. A literature study (appendix A) revealed that the concept is only built for three projects in the years 1908, 1911 and 1914. The Hollandsche Beton Groep (HBG) decided to build only rectangular caissons. An overview of the first caisson projects is presented in the table below. It can be seen that the method of construction was initially horizontal, disregarding the geometry. After the Chilean port expansion projects, the method for rectangular caissons changed to vertical.

Project	Caisson construction methods	
	Counterfort caisson	Rectangular caisson
Valparaíso (1903)	Horizontal	Horizontal
Talcahuano (1908)	Horizontal	Horizontal
Surabaya (1911)	Horizontal	Vertical
Tandjong Priok (1914)	Horizontal	Vertical
Later HBG projects	*Withdrawn*	*Vertical*

After this period, horizontal construction methods were rarely applied. Two other caissons are found which make use of a turning principle; one project in Gdynia (1927) and one in Tunis (≤1967). Remarkable is that a feasibility study is performed for a counterfort caisson quay wall in 1986. This so called Camilla caisson has great similarities to the first counterfort caissons, but no record is found of any application.

The geometry of the first caissons of the first caissons did not affect the construction method. Later, the construction method started to depend on the shape of the caisson. The geometry is thus affecting the method of construction, launching and transport. It is therefore presumed that optimal construction methods are found for rectangular caissons, while a counterfort caisson might still be feasible with a horizontal construction method. Reasons for the withdrawal and new opportunities for counterfort caissons are sought in the following aspects:

Technical feasibility
- Operational conditions;
- Stability requirements;
- Durability requirements;

geometry, weight, displacement

Construction technology
- Construction technology;
- Executional aspects;
- Launching techniques;
- Transport and immersion requirements;

equipment, falsework, formwork, crane capacity, floating equipment

Economies
- Shifts in labour, materials and equipment;
- Design, risks and safety;

resource consumption

3. Technical Requirements (1903 – 2017)

Designing a reinforced concrete quay wall structure for sea harbours requires a well-considered design approach. The quay wall must meet all functional requirements during its service live in an economical way. The structure must thereby fulfil its tasks under actions and influences which are likely to occur during the execution and operational phase.

A certain balance must be found between aspects as; construction costs, durability and robustness. Also awareness of a high number of various loads throughout the design life of the quay structure is a prerequisite. This chapter provides an overview of changed technical aspects and requirements from 1903 to 2017. The addressed aspects are considered from the perspective of technical feasibility, which involves geometry, weight, displacement, stability and durability of the concept.

3.1. Structural developments

The development of large concrete gravity based quay wall structures started with unreinforced structures such as the Langton Dock Wall in Liverpool (1881) and block-wall structures such as constructed in Bougie, Algeria (1904), both depicted in figure 3.1. The retaining height of the shown block-wall was approximately 10 metres, the unreinforced concrete quay retained almost 12 metres and the caisson quay wall retained 14 metres soil.

Predecessors of the reinforced caissons were unreinforced concrete structures and concrete block-walls. The unreinforced concrete structures have relatively high self-weight compared to modern reinforced concrete caissons. These unreinforced variants are nowadays still widely applied, which indicates that the more recent developed reinforced concrete caisson is not particularly an improvement for all conditions.

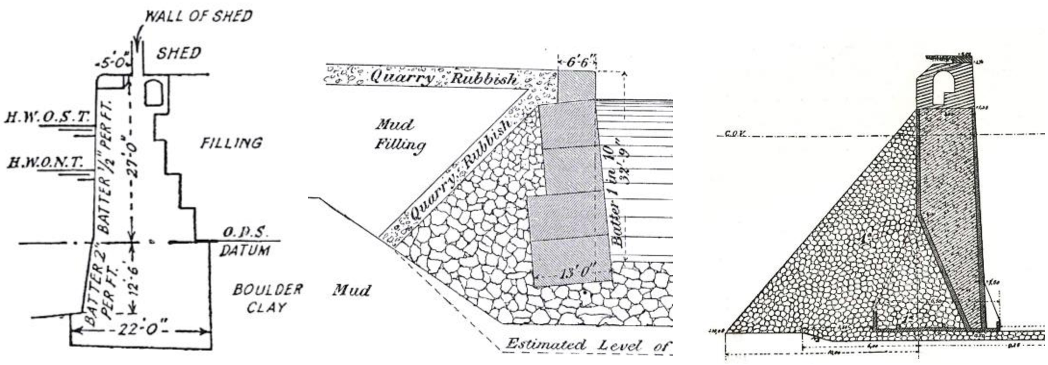


Figure 3.1. Concrete quay wall structures, gravity based; Liverpool (left), Bougie (mid) and Talcahuano (right).

3.1.1. Reinforced concrete

Development of this composite material, reinforced concrete, enabled engineers to reduce the weight of retaining walls considerably. However, since the weight of the quay wall ensures its stability, it must be compensated by a particular compartment fill, soil anchorage (e.g. an anchor plate) or sufficient heel embedment. Therefore, a reduction of self-weight must result in an increased width in case of gravity quay walls.

The first economical counterfort caissons were (from load transfer point of view) still very similar to the heavy weight unreinforced structures, since the caisson compartments were filled at its final position with a sand-cement mixture. The relatively low quay loads and high weight resulted in a slender retaining structure (B ≈ 0.5H), which is from load transfer point of view very similar to unreinforced concrete structures.

3.1.2. Compartment fill and backfill material

The reason for filling compartments with concrete was to increase the stability *and* strength of the structure. The overturning stability (equilibrium) increases when the compartments are filled with a heavy material. At the same time, a greater structural resistance can be obtained due to the connection of structural elements.

The cementitious fill was required because of the low quality concrete (5 to 15 N/mm^2) and slender walls in combination with relatively high loads and the probability of (accidental) ship collisions. For this reason, caissons in the port of Rotterdam (around the year 1920), have been designed with dedicated compartments at the front of the wall for the application of an unreinforced concrete fill. Therefore, risks of ship collision influenced the design significantly. Cementitious fills slowly vanished from the designs, starting with projects where compartments were only filled with concrete if these locations were subjected to a high risk of ship collision.

In the years following after the Second World War, the unreinforced compartment fill was vanished from all caisson quay designs at the port of Rotterdam. Reasons for this can be for example: an increased concrete quality, increased wall thickness (e.g. due to increasing quay height), increased insight in material behaviour and/or improved manoeuvrability of ships.

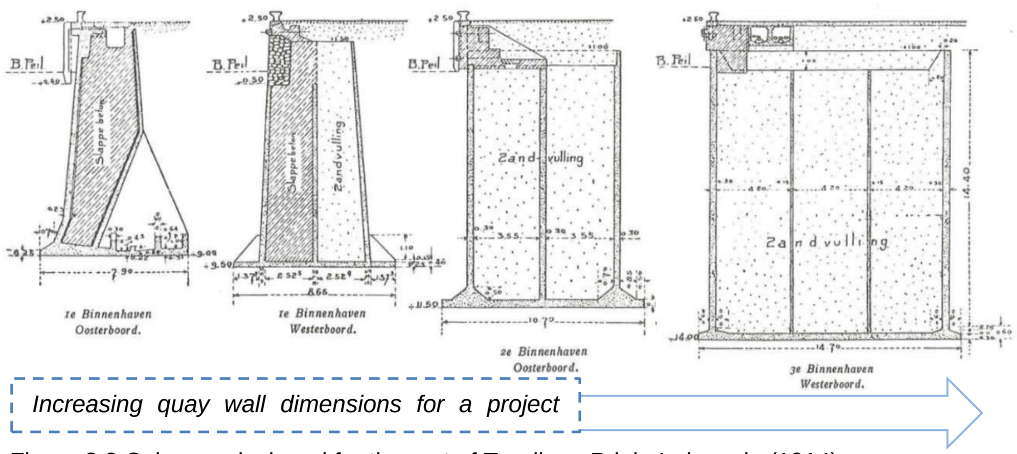


Figure 3.2 Caissons designed for the port of Tandjong Priok, Indonesia (1914)

Analysing different types of caissons built for a project in Tandjong Priok (1914), it can be seen that also the rectangular caisson *(1^e Binnenhaven Westerboord)* has been filled with unreinforced concrete. Filling the right (instead of left) compartment with such a heavy material would increase the stability substantially more. This would be more reasonable if one considers the distribution of foundation pressure. The reduction of eccentric loads could also result in less differential settlements in transversal direction.

On the condition that a rectangular shaped caisson is an economic design, the width can be further increased without major cost increase. At this point, it is often more economical to increase the width until locally sourced sand can be used as backfill material, as Tsinker (2014) describes:

> *"In general, reduction in caisson width does not save much concrete required for its fabrication. However, significant saving in the total cost of a quay wall may result from a reduction in volume (and cost) of stone bedding."*

The statement can be verified by comparing the two situations; a caisson with rubble backfill and a caisson with a sand backfill which therefor needs to be widened (fig. 3.3.).

The two situations are schematically depicted in the figures below. The main point at issue is when a certain solution becomes more economical. Initially, the difference is verified by considering a simplified example. This is however insufficient for disproving the general feasibility of the overturning concept.

Figure 3.3. Design choice: rubble backfill or increased caisson width

Simplified feasibility consideration

A cross section of a traditional caisson consists of approximately 20% concrete. A caisson width-to-height ratio is normally in the order of 0.80. Based on this, a simple expression can be made which provides the caisson concrete volume per running metre quay for a particular height (H):

$V_c = 0.20 \times 0.80\ H \times H = 0.16\ H^2\ [m^3/m^1]$

And, based on a triangular backfill with a slope of 45 degrees, the rubble stone volume per running metre amounts approximately:

$V_b = 0.50 \times H \times H = 0.50\ H^2\ [m^3/m^1]$

The costs of reinforced concrete are estimated to be €300/m^3 and the rubble stone backfill is expected to be €50/m^3 more expensive than locally sourced sand. The cost of a backfill thereby becomes:

$C_b = 0.5\ H^2 \times €50/m^3 = €\ 25\ H^2\ [m^{-1}]$

And for this value, a caisson volume can be increased up to:

$V_{d,add} = €25\ H^2\ /\ €300/m^3 = 0.08\ H^2\ [m^3/m^1]$

For this example, the caisson can be widened to 150% of its original size. After this point, a rubble backfill becomes more economical. Note however that the feasibility is highly affected by cost the difference between rubble and reinforced concrete. Also the particular shape of the caisson and backfill changes the outcome.

The question remains whether the enlarged width increases the stability in such a way that the lack of a rubble backfill is justified. Due to a higher soil shearing angle, the horizontal thrust can be reduced by 50%. This can be seen by the difference in K values (appendix L.5.) for soils with a shearing angle of 30 degrees compared to 45 degrees. A rubble backfill therefore reduces loads significantly, which can justify the higher procurement costs.

If soil pressure is the main source of destabilizing actions, the change to a rubble fill can highly influence the required width of the gravity retaining wall. If however other quay loads are present in a considerable amount, the impact of a rubble fill will become less significant to the overall design.

3.2. Operational and stability conditions

From the stability analysis (appendix B) from the original caisson, it can be seen that the original caisson offers an inadequate reliability in terms of (GEO) stability. If the caisson would be designed according to the British Standard and Eurocode, the caisson width should be increased to approximately 8.50 metres. This equals approximately 75% of the caisson height, which is still rather slender when the superstructure is included. The differences of safety levels are presented in table 3.1. Based on the preliminary calculations, values above 1.00 indicate sufficient stability.

SLS verification	Forward sliding (GEO)	Overturning (EQU)	Foundation pressure (GEO)
Original caisson design (width = 6.50m)	1.20	1.55	0.75
Adjusted caisson design (width = 8.50m)	1.60	2.65	1.10

Table 3.1. Stability analysis of original overturning caisson

For the enlarged caisson, the total width-to-height ratio, including the superstructure amounts (8.50/14.0=) 0.61. The PIANC Seismic Design Guidelines for Port Structures (2002) notes that a relatively small ratio of 0.75 will exhibit a predominant tilting failure mode rather than horizontal displacements. This results in extreme cases in collapse of the wall, where wider caissons are often only associated with excessive deformations. If a slender design is preferred, the increased risks in relation to earthquake loadings must be well-considered.

The required width increase of approximately 30% results in a different caisson geometry. Possible stability improving measures are presented in the table below.

Measure	Disadvantage
Drainage to equalize water levels	Increased maintenance costs and uncertainty in drainage behaviour;
Apply a backfill with higher shearing angle	Procurement of rubble stones; more expensive than locally sourced sand;
Increase the weight of the structure, e.g. by filling the compartment with concrete	Increased eccentricity results in higher soil pressure near the toe and lower soil pressure near the heel of the caisson;
Filling compartments only with water	Less sliding capacity and overturning stability (EQU) due to decreased weight
Provide alternative soil anchorage (e.g. by anchor plate)	An additional construction phase shall be required, which increases equipment and labour costs;
Increasing the heel width	Less buoyancy, increased draught, different floating position;
Increased toe-width	Less buoyancy, increased draught, different floating position.

Table 3.2. Possible measures to increase global caisson stability

3.3. Quay height

The quay height for international ports has increased drastically over the last century. Figure 3.4 shows the quay depths for the Port of Rotterdam from the year 1855 to 2000. The first port activities were near the city centre and moved away from the city (Botlek, Europort and Maasvlakte) to facilitate larger vessels and increased transhipment quantities.

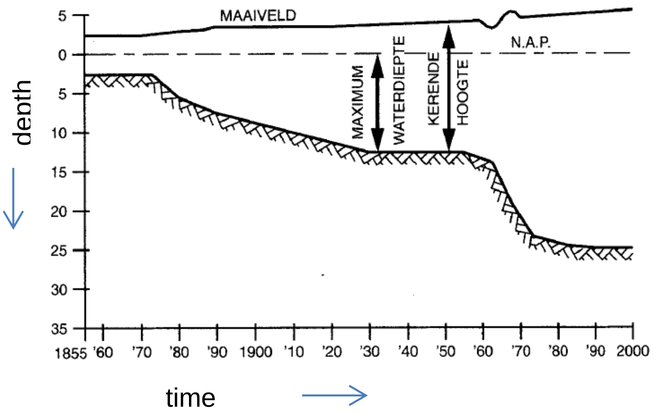


Figure 3.4. Increasing quay height for the port of Rotterdam (Kademuren, verleden, heden en toekomst)

It is expected that the draught of large vessels will not increase drastically in the coming years (De Gijt, 2010). Due to nautical constraints, such as depth and dimensions of sea straits, the draught is expected to be restricted to a value in the order of 25 metres. Therefore, the demand for quay wall structures with a height of 20 to 30 metre is expected to continue.

For this thesis a quay wall height of 21 metre is considered to be a good representation of the current demand for port expansions. After subtracting a few metres for water level differences and flood safety, such a height is still large enough for New Panamax ships (draught = 15.2m). Also various Post Panamax (ULCV) categories are able to berth at this quay height.

3.3.1 Scaling considerations

It would be reasonable to consider a linearly scaled concept of the original caisson design for larger quay heights. A caisson compartment could be enlarged as presented in figure 3.5. However, if the caissons and compartments are scaled by a certain factor larger than one, the wall-span and immersion pressure both increase. For instance; if the caisson height is doubled and the compartments are equally scaled, the wall thickness must increase drastically (see appendix J). The maximum shear force induced by hydrostatic pressure becomes 4 times higher, which indicates that the wall thickness must be increased by a factor 4 to obtain an equal shear stress. Besides this scaling aspect, the shear capacity itself decreases. According to the Eurocode 2, a size factor must be taken into account for larger concrete sections (>200mm) loaded in shear.

When a single compartment is analysed and the wall-spans are all doubled, its volume (and displacement) becomes four times larger, while its circumference is doubled. Since the walls are loaded by a pressure twice as large, and the spans are also scaled by this factor, the wall-thickness must be increased by a factor 4. The amount of concrete (and weight) is then increased by a factor 8.

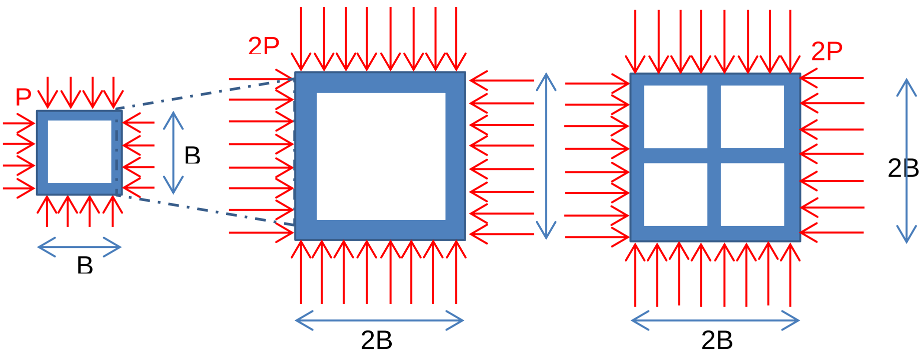


Figure 3.5. Compartment scaling effects

This effect results in limiting dimensions for caisson compartments. In general, larger caissons do not allow larger wall spans. Compartments can efficiently be designed with wall-spans up to approximately 3.50 to 4.00 metre. Actual limits depend on the particular design conditions. In terms of material use, adding transverse walls becomes more efficient than increasing spans.

3.4. Transport

The caisson transport can be divided into three different phases. During the first phase, the caissons must be transported from the construction site to the launching area. The weight and size of the elements are important aspects during this operation. After this, the caissons are launched and then transported over water to its final location. A schematic overview of traditional caissons is presented in figure 3.6 below.

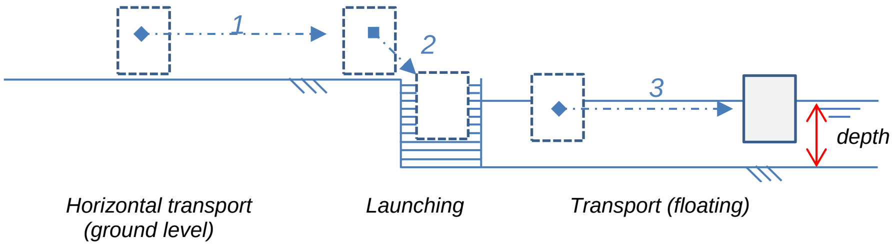


Figure 3.6. Traditional caisson transport phases

The technical characteristics of the caisson elements will determine the eventual transport costs. A lower caisson weight shall, for instance, result in a reduced demand for hydraulic jacks for transport over land. Weight reductions also affect the required lifting and launching capacity. Furthermore, the weight and shape could reduce the required towing capacity. These examples had large impact over a century ago, but are currently considered to be insignificant. The factors which are estimated to have high impact are related to draught and floating stability. An overview of the estimations is presented in table 3.3 below.

	Transport element	Influencing factors	Estimated significance
1.	Transport over land	weight, size	little
2.	Launching	weight, size	little
		draught	high
3.	Transport over water	weight, size, hydrodynamic drag	little
		draught, floating stability	high

Table 3.3. Estimated significance of differentiation within transport phases

The importance and consequences of draught changes are depending on local conditions. The draught cannot be problematic if the water-depth on the transport route is for instance larger than the caisson height itself. When the water depth is sufficient, the construction and launching procedure may still be affected by a change in draught. For instance, a dry-dock may become shallower or a slipway may become shorter.

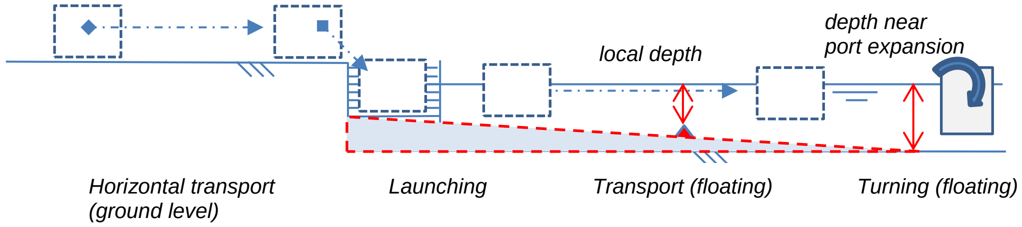


Figure 3.7. Overturning caisson transport phases

For a particular shape, the lowest theoretical draught can be obtained when the centre of gravity is located at half the caisson height. The caisson would then simply float in a straight equilibrium position. For a counterfort-caisson, the centre of gravity is off centre. The weight on one side increases when the heel-width must be increased to meet operational requirements. This changes the floating equilibrium position and increases the draught. The relation between floating position and centre of gravity is explained in more detail in appendix B.10.

The reduced draught is not free of charge; the caisson needs to be turned near its final location. The ideal second floating position (during immersion) would be completely vertical. As the asymmetry of the caisson becomes larger for a width of 8.50m, it becomes more challenging to find a vertical floating equilibrium.

3.5. Durability aspects

Nowadays, it is known that durability of reinforced concrete is not some given characteristic of the material itself. Many different aspects influence the durability and life time. Only if the structure is designed and built properly, the desired performance can be achieved. Durability aspects are perhaps even more important than the compressive strength, since the majority of the problems are associated with degradation due to poor durability, rather than lack of strength.

Durability of concrete can be defined as the ability to resist attack from environment in which it is placed. The attack can be either physical or chemical. Examples of different attacks are presented in table 3.4 below.

Physical attack	**Chemical attack**
Abrasion	Sulphates
Impact	Chlorides
Ice growth (freeze thaw)	Carbon dioxide
Permeation / diffusion	Alkalis
	Acids

Table 3.4. different forms of attack on a concrete structure

Form the examples of physical attacks, abrasion and (ship) impact are from importance for quay wall design. On the other hand, chlorides and carbon-dioxide are from major importance when considering chemical attack. These can influence the concrete quality and induce corrosion of carbon steel reinforcement.

3.5.1. Recommendations

From extensive research and experience over the last century, knowledge is obtained which currently enables one to design a durable reinforced concrete structure in marine environments. A relatively large concrete cover of at least 60 mm, in combination with a proper concrete mixture is required to obtain a quay design which can fulfil a service life of at least 50 years. Legally, the cover depth could be reduced to the requirements of the Eurocode. However, these cover depths are rather low compared to quay design recommendations.

The minimum concrete cover cannot be reduced significantly by applying high cementitious concrete mixtures or high strength concrete for instance. Most standards use the concrete cover depth as main parameter for different exposure classes. The BS 6349 specifies a concrete cover to mixture relation, but only starts from 55mm (XS3 and 50 years design life) and prescribes a high concrete quality.

Besides legal obligations by several European codes, the cover depth requirements are well-founded by theoretical background and field measurements. This makes a significant cover depth reduction for carbon steel reinforced concrete inacceptable, irrespective to the particular project location.

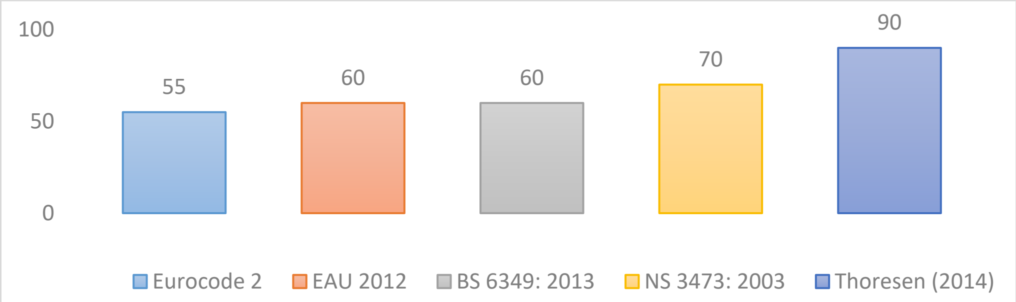


Figure 3.8. Concrete cover depth regulations and guidelines for tidal splash zones in marine environments and a 50 year design life.

For quay wall structures, other measures such as coatings, stainless steel or non-metallic reinforcement does not seem to result in a significant reduction of the concrete cover. Besides the fact that a significant cover reduction due to those measures would legally questionable, the earlier mentioned desired robustness will be compromised. In addition, those solutions would threaten the economic feasibility drastically.

The following schematization (fig. 3.9.) has been made to provide an overview in terms of concrete cover, robustness and degradation. In terms of robustness, a minimum front- and back-wall thickness of 300mm is recommended. The design of separation walls is limited by executional and economical aspects and can therefore not be reduced any further than 250mm.

Particular degradation sources for marine structures:

- mechanical loads (berthing / mooring)
- abrasion (e.g. by wave impact)
- chloride ingress
- carbonation
- freeze-thaw actions
- temperature gradients
- humidity gradients

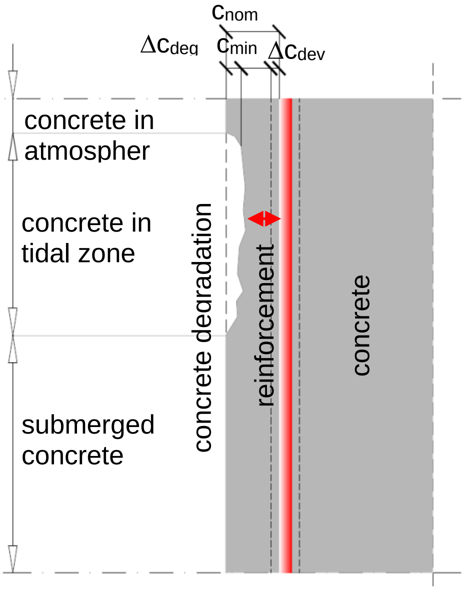


Figure 3.9. Degradation mechanisms with respect to the concrete cover (exaggerated schematization)

Measures to prevent degradation:

- Provide sufficient concrete cover (≥60mm)
- Apply high concrete grade (≥C35/C45)
- Apply a low water / cement ratio (≤0.45)
- Adjust cement type (e.g. apply CEM II or III[3])
- Mitigate concrete crack width

Besides the listed measures, good casting and curing conditions are desired to obtain a durable reinforced concrete structure.

3.6. Synthesis

Various aspects for caisson quay wall design changed over the last century. The first caissons were designed as relatively small, slender and fragile concrete self-floating structures which compartments were filled on their final location with a sand-cement mixture. Its design was thereby highly influenced by temporary construction phases.

The used concrete strength is nowadays much higher. The applied concrete class in 1903 was comparable to C12/15, which is currently the lowest available grade according to the European EN-206-1 regulations. This low grade is currently rarely used for structural applications. The significantly higher concrete grades (normal strength concrete up to C45/55) are, besides its strength increase, compulsory for a durable structure. However, the improvement of mechanical characteristics has not resulted in more slender structural elements.

Due to changed stability and robustness demands, structural elements became thicker and larger, while the required caisson width also increased. These changes where in favour of the longevity of the structure. Considering the original design, using a modern stability calculation approach, its width must be increased by approximately 30%. In contrast to rectangular caissons, a change of geometry results in a different floating equilibrium position. The design became increasingly influenced by serviceability requirements.

[3] According to CUR-Leidraad 1, CEM I would not be desirable, while actual field measurements (e.g. Gaal 2004) show no significant durability increase when blast furnace slag cement is applied

4. Construction Technology (1903-2017)

4.1. Workability and execution

4.1.1. Historical overview of mixing and casting

The first patents for making concrete in mobile steam driven mixers appeared around the year 1900 in the United States. The first mixer trucks arose after the invention of Stephen Stepanian in 1916. These trucks were developed to replace the horse-drawn concrete mixers as shown in figure 4.1.

After this invention, various mobile mixer trucks were designed by others. Due to these developments, ready mixed concrete plants (batching plants) became more popular. It was only after the Second World War that the mixer trucks became faster and less cumbersome. Also, the trucks became after this period capable of transporting several cubic metres of concrete.

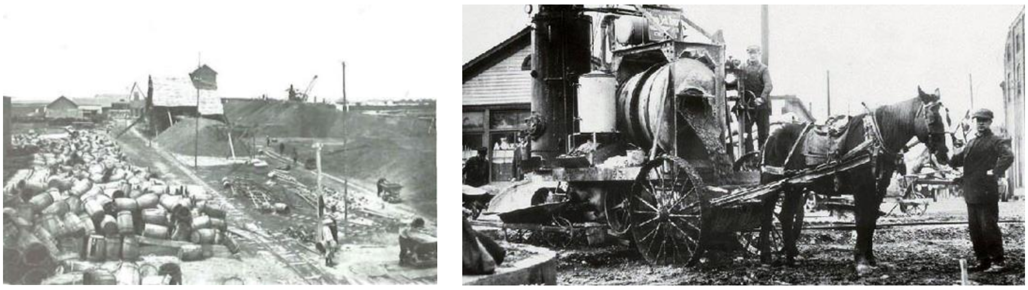


Figure 4.1. Left: stone crusher, aggregates and empty cement barrels (Surabaya, Indonesia, 1911). Right: batching by a steam driven mixer and horse drawn transport (USA, 1916)

It is not surprisingly that concrete mixing and casting has changed drastically over the last century. Technology has improved the efficiency and speed of concrete mixing and casting. Over hundred years ago, processing concrete was labour intensive and state of the art technologies had to be used for simply making a workable concrete mixture. Also transport from the mixer to the actual pouring place was a labour intensive task.

4.1.2. Concrete mixture quality

Professor D.A. Abrams (1918) was the first who described the water-cement ratio as being a key aspect for concrete characteristics. He found a relation between the water-cement mass ratio and strength. This relation prescribes that the strength increases when the w/c ratio decreases. It can be expressed in the form of:

$f_c = \frac{A}{B^{w/c}}$ Abrams' formula (1918)

The formula was not yet presented when the first caissons were constructed. However, disregarding the knowledge, the formula could not instantaneously lead to higher concrete strength. Workability for lower water cement ratios remained a dominant aspect. It was only after the development of plasticizers and superplasticizers that lower cement ratios became practically feasible. This resulted that for the first caissons, a 28 days concrete strength of just 15 N/mm^2 was reached.

Modern concrete mixtures and execution methods (e.g. casting, curing) resulted in large improvements in terms of strength and quality. Nowadays, concrete grades up to C45/55 are characterised by the Eurocode as normal concrete. The modern concrete mixtures, high green-strength and curing compounds allows fast demoulding and reuse of formwork.

4.1.3. Traditional formwork (1903-1920)

The first caissons were designed and built with traditional timber formwork. Due to the scarce application of reinforced concrete, advanced formwork systems were hardly developed. However, due to its simplicity, timber formwork has several advantages, such as; easy handling (light weight), flexible usage and easy replacement of components. Also the procurement costs of timber formwork is lower than for steel.

Traditional timber formwork is generally labour intensive and the formwork parts can be used for a limited number of times (≈ 5 times). Also in terms of quality management, timber formwork can require more attention since it is more sensitive to moisture, humidity changes and individual craftsmanship.

As discussed in section 2.2.4, the first caissons (Valparaíso) were planned to be built in portions of 114 caissons time. This resulted of approximately 4 times full occupation of the construction site (L-shaped and rectangular caissons). The scale of the simultaneous construction of caissons is shown in figure 2.10.

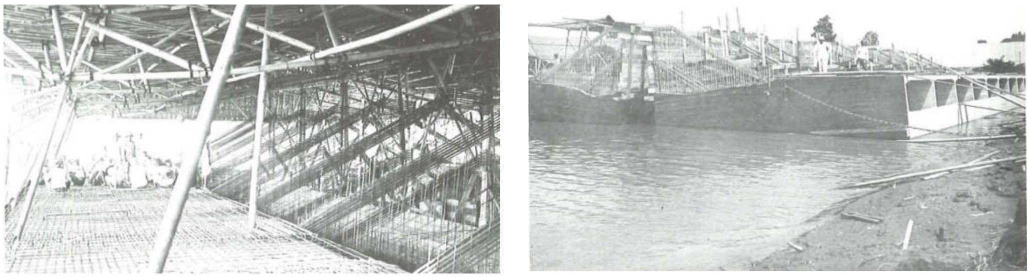


Figure 4.2. Construction of caissons (horizontally) with the traditional timber formwork method (Tandjong Priok, 1914)

The caissons for Tandjong Priok (fig. 4.2) were first constructed under temporary shelter and the counterforts were finished while the caissons were floating. The construction process must have been similar to the following sequence:

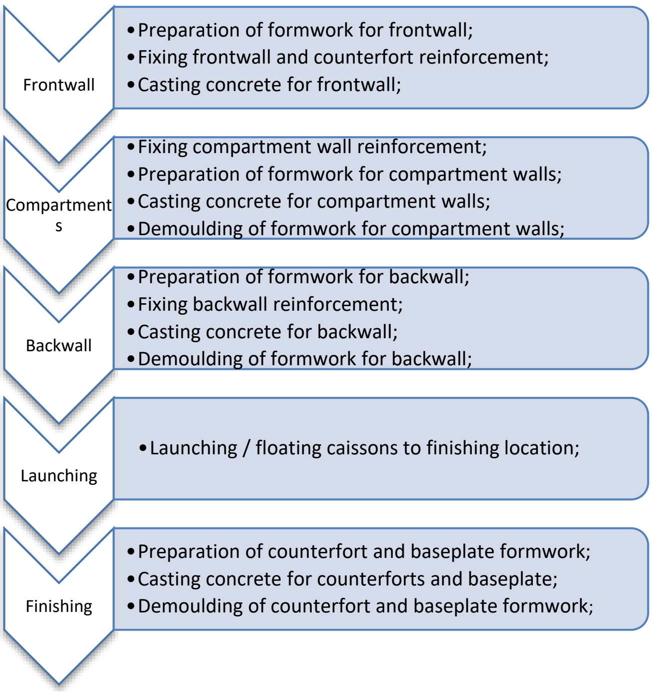


Figure 4.3. Construction sequence overturning caisson (1914)

One caisson required more than a month construction time due to its labour intensive formwork techniques and irregular construction process. On the contrary, the caissons were built with limited use of equipment. Due to lean equipment use and relatively slow construction speed, it was efficient to construct multiple caissons simultaneously. Repetition benefits were thereby mainly obtained by constructing identical caissons. The construction of one single caisson consisted of various different components. Repetition advantages within a single caisson could only be obtained by (vertical) construction of the separation walls and counterforts.

4.1.4. Maas formwork method (1920-1960)

From the year 1920, the casting method became similar to the currently used construction method of apartment buildings (*gietbouw*). The process of the box method (fig. 4.4.) was patented by Ir. Maas. It allowed much higher repetition of formwork and relatively fast construction of caissons. The construction time of one caisson was approximately 40 to 45 days. Concrete for caisson walls was mixed by the proportion (cement, sand and gravel) of 2:3:5. After mixing, the concrete was poured through timber and steel gutters. The fresh concrete was very thin, which allowed easy flow, but also resulted in segregation of aggregates. Therefore, concrete had to be remixed before it was poured into formwork. This indicates that a lot of water was used and probably a high water-cement ratio.

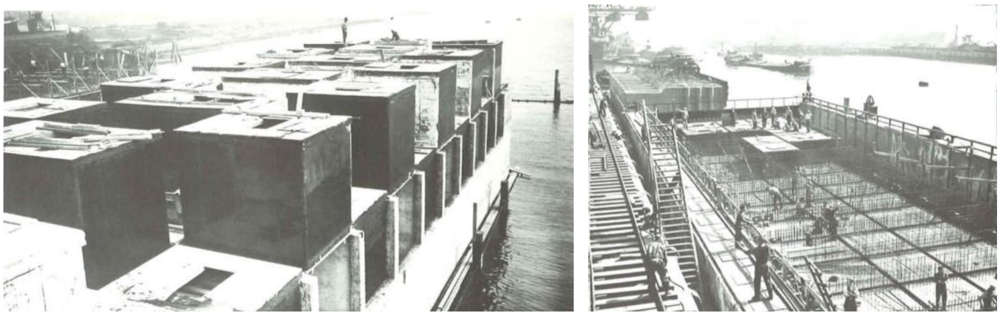


Figure 4.4. Construction of caissons by the Maas formwork system (The Netherlands, Waalhaven, left photo: 1920-1921, right photo: after 1945)

In order to obtain an efficient production rate, multiple (±6) caissons were constructed simultaneously. The activities and corresponding duration were as follows:

Activity	Duration
Formwork preparation	1 day
Formwork demoulding	1 day
Reinforcement fixing	1 day
Pouring concrete floor	1 day
Pouring concrete walls (± 3 metre high)	1 day
Concrete hardening	1 - 2 days

Table 4.1 – Caisson construction Maas method, 1920)

Theoretically, if a modern 18 metre high caisson would be constructed with the Maas formwork method, 6 casting cycles would be required. Considering that formwork preparation, demoulding, rebar fixing and pouring and hardening concrete took approximately 6 days, only the caisson walls could be constructed in at least 36 working days.

4.1.5. Climbing and slipforming systems (1970-present)

Since the 1970's, the climbing and slipforming techniques are applied for constructing caisson quay walls. This technique is characterized by its continuous work-flow and high repetition factor. The forms can be supported by the concrete structure itself or by large temporary gantry structures, as shown in figure 4.5. The systems are designed to be reused without high maintenance costs. Slipforms can for instance be reused for over 100 caissons without large maintenance costs. Therefore, such a system can efficiently be applied since caissons are generally large concrete structures with a generic shape over the height.

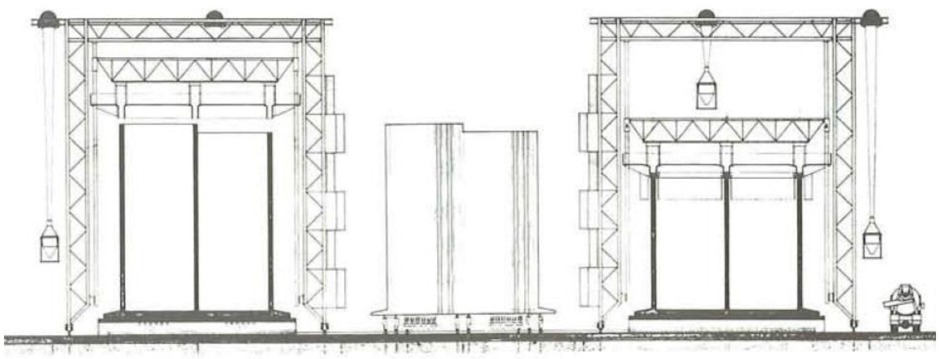


Figure 4.5. Gantry method applied in Saudi Arabia, Dammam (1977)

The vertical slipform method is suited for marine due to the high degree of durability that can be achieved with the process. Due to the continuous process of slipforming, all concrete is poured “fresh in fresh”, which means that there are no construction joints. There is also no need for form ties and rebar bolsters when using this construction method, resulting in a durable structure.

The choice for applying a traditional slipforming technique or a gantry system largely depends on the required number of caissons and use of equipment. The construction of a caisson requires approximately 14 days if traditional slipforming is applied. The construction time can be reduced to one week or 6 working days if the gantry slipform system is used.

A gantry system requires a relatively large initial investment in temporary structures, but it allows the construction process to advance more efficiently. The main advantages of this particular slipforming technique are:

- The slipform decks can be used for material storage and provide good working conditions (e.g. low risk of falling and a roof provides protection for the sun/rain);
- The gantry is equipped with cranes, additional required crane use is marginal;
- The gantry provides support for the slipform; there are no jacks in the concrete itself;

Due to the applied formwork techniques, reinforcement is generally lifted without pre-assembling it into a mesh or net. The rebars are generally placed manually while the slipforming process continues. The used lifting equipment varies from small mobile cranes up to fixed construction cranes or gantry cranes. The particular choice for equipment largely depends on the size of the caisson and used slipform technique.

For a concrete caisson, the described slipforming process (fig. 4.6.) can normally be finished in one week. For an 18 metre high caisson and s slipform jack-up height of 50mm, the process would be repeated 360 times. This results in a high repetition factor in terms of equipment exploitation and learning factors. The continuous process allows 4 to 5 metre climbing each 24 hours. This is in contrast to the formwork repetition for the previous Maas method would only allow 6 times repetition.

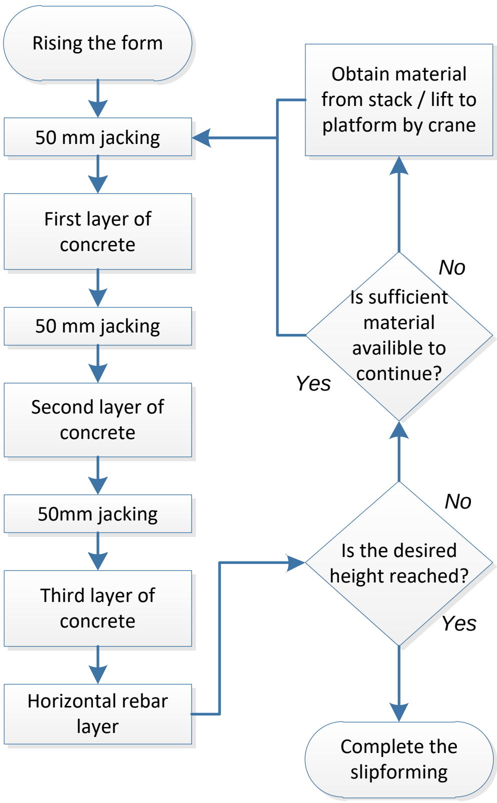


Figure 4.6. Example of the slipforming process (after: Slip-Form Application to Concrete Structures (2008))

The slipforming technique affects the rest of the building process as well. Regular reinforcement placement, concrete pouring and finishing is required in order to maintain the continuous work-flow. Jacking rates and the availability of building materials must be tuned to obtain the most efficient solution. A rather small aspect, such as the choice for a concrete pump or buckets can already make a significant difference[4].

4.1.6. Slipforming learning effects

The degree in which the learning effect reduces the working hours depend on various aspects. The type and conditions under which construction takes place affect the learning rate drastically. In order to quantify the learning effect, Hijazi et al. (1992), proposed learning rates for different construction processes.

Description	Learning rate
Structure of ordinary complexity; high rise, office buildings	95%
Construction elements requiring many operations to complete; erection and fastening structural units, concreting	90%
Construction elements requiring few operations; masonry, painting	85%
Construction elements requiring few operations and on-assembly line basis; formwork panels, rebar bending	80%

Table 4.2. Learning rates proposed by Hijazi et al. (1992), [ref. A26]

[4] Slip-Form Productivity Analysis for Concrete Silos, Sharifi, S, et al. (2006)

The applicability of learning curves for floating caisson construction is studied by Panas and Pantouvakis (2013). Here, the slipform construction method on a floating semi-submersible barge was analysed and simulated. No reference to a particular port expansion project has been made, but the project shows great similarities to caissons shown in figure A.1 and A.2 (appendix A.1.), which are constructed for the port of Piraeus in Greece.

It was found that for slipforming activities, a labour learning rate was achieved of approximately 80%. This rate is in line with the values presented in table 4.2. It appeared that performance prediction without knowledge of past performance provides rather high simulation errors (fig. 4.7). If the past construction performance of 4 caissons is used as input, the final working hour consumption could be estimated with an error varying from 19% to 26%. The accuracy varies for respectively a simulation or statistical approach. However, the more project performance data was used as input, the more accurate the predictions became.

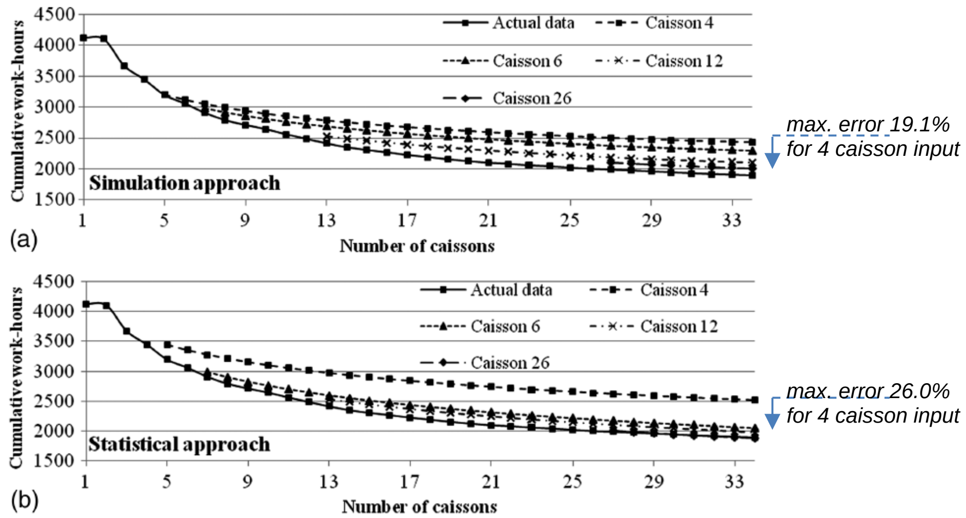


Figure 4.7. Performance prediction of floating caisson construction (Panas and Pantouvakis (2013))

This study on slipforming construction techniques for floating caisson construction shows the sensitivity of varying input parameters. Also the availability of historical data (experience) is needed to obtain a reasonable simulation output. From the graphs, it can be seen that the initial labour consumption starts at approximately 4,000 working hours per caisson and reduces by over 50% after constructing 34 caissons. At this point, the most significant decrease of labour consumption has been reached. The working-hours required for construction can be expressed as:

$$y = a \cdot x^b$$

with:
a= constant depending on the initial working hours
b = the learning index (slope of the asymptote)
x = cumulative unit number

However, the learning effect is over estimated since the expression results in:

$$\lim_{x \to \infty} a \cdot x^b = 0$$

This endless reduction of working-hours cannot be true. Based on figure 4.7, it is therefore assumed that the learning effect reaches the lowest working-hour consumption after 40 caissons. This lowest level is assumed to be half (50%) the original value.

4.1.7. Conclusions

The fist projects were planned to be constructed with many caissons simultaneous. Benefits of repetition were found by constructing identical caissons. This was the most economical solution due to the relatively slow working speed, low reuse of formwork and low labour costs.

Nowadays, equipment is more exploited. New slipforming techniques allow the reuse of formwork for over 100 caissons. Since the slipform is raised with steps of approximately 50mm, the forms are reused for hundreds of times for each caisson. The continuous work-flow results in efficient and a rather constant resource consumption. Due to these developments of equipment and techniques, caissons can be constructed relatively fast (approx. 1 per week). Because of these reasons, typically just 1, 2 or 3 caissons are nowadays constructed simultaneously.

Research on repetition effects show that labour efficiency increases drastically when a process is repeated. The exact learning rate and reduction on labour consumption after a certain number of caissons is however hard to predict

The high investment costs are therefore compensated by the increased efficiency and high depreciation over the project. Furthermore, the quality control and management has improved for modern formwork techniques. Additionally, the improved construction technologies are able to cope with the more stringent formwork and falsework requirements.

4.2. Heavy lifting and launching equipment

4.2.1. Launching techniques

The slipway launching method used for the first caissons is nowadays still applied for vessels and caissons. However, there are currently many alternatives such as the syncrolift (ship lift), dry-dock or a floating dry dock. The most economical launching method is thereby not limited (anymore) to certain shapes or sizes. Among others, the decision for a particular launching method depends on the locational aspects as the:

- Available space for construction and launching;
- Local ground conditions;
- Bathymetry: depth and slope of the seabed near construction site;
- Distance from construction site to final location;
- Future plan of use of the construction site;
- Possibility of combining harbour improvement works (e.g. dredging);

A mild slope near the construction site can be beneficial for slipway launching since a floating dock cannot reach the construction site without dredging works. The magnitude of required adjustments to the construction site, such as dredging works and temporary quay walls shall determine the most economical solution. The feasibility of a launching method also depends on caisson properties such as the:

- Geometry and dimensions;
- Weight;
- Draught;
- Number of caissons;

discussed in section Technical Requirements (3.4)

A caisson with low weight and low draught, as for the first overturning caissons, can relatively easily be launched by a slipway. However, caissons currently have a draught in the order of 10 metres (see appendix E-H). This launching method becomes less feasible if local conditions allow this depth after a large distance from the shoreline. Then, the extensive slipway and/or dredging works do not outweigh the benefits of a floating dry dock (FDD). The floating dock could launch the caissons and reduce the draught to a few metres. Therefore, on the condition that the location of the construction site is determined and fixed, the feasibility of a particular caisson launching method can change if the characteristics of the caisson change drastically.

A floating dry dock is currently often used for construction and / or launching caissons. An overview of used launching methods is presented in the pie chart below. The data is based on 58 reference projects from the period 1963-2015 by a specialized contractor[5]. The presented pie chart is therefore based on biased data, and only gives an impression of used launching methods over the world.

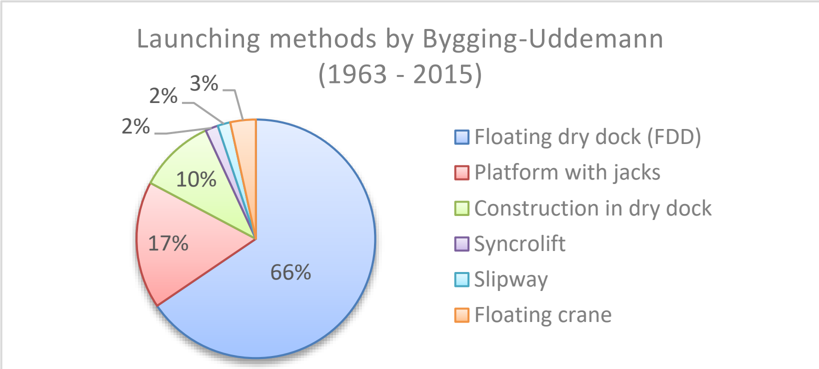


Figure 4.8. Caisson launching methods (Bygging-Uddemann, 2015)

A benefit of this launching method is that the procurement of the FDD can be smeared over multiple projects. This depreciation is a considerable advantage compared to the other launching methods since these do not allow complete reuse. A syncrolift, dry-dock and slipway have a more permanent character. These methods become increasingly beneficial when there is a local demand for launching other ships or structures after the initial quay project has finished.

The principle of caisson launching with a FDD is shown in figure 4.9. The caissons are (for instance) transported by multiple 250 tonne hydraulic jacks and arrive after several construction stages at the FDD. Note that the capacity of these jacks is even larger than the total weight of the first overturning caissons (220 tonne). Transport of heavy structures has definitely simplified over one century due to the invention and advancement of hydraulic lifting jacks.

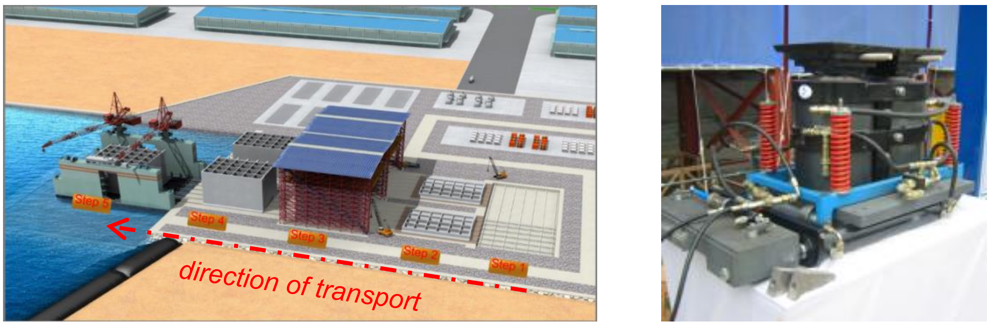


Figure 4.9. Caisson transport and launching; 5 phase transport to floating dock (left) and 250 tonne jack (right) (Bygging-Uddemann 2017)

[5]Based on reference projects from Bygging-Uddemann: http://www.bygging-uddemann.se/wp-content/uploads/2015/06/Caissons.pdf

4.2.2. Floating cranes (sheerlegs)

Technical improvements over the last century allow caissons to be lifted completely by the sheerleg itself. Floating sheerlegs had a maximum capacity in the order of 70 tonne in the beginning of the 20th century, which has increased to an enormous lifting capacity of 10,000 tonne (2015).

Although the use of reinforced concrete was rather limited, the first caissons (1903) were too heavy (220 tonne) to be lifted entirely into position. The use of buoyancy was therefore almost compulsory in that time. Nowadays, the lifting capacity of floating cranes is often larger than the entire weight of regular sized caissons, which weigh in the order of 2,000 to 4,000 tonne[6]. However, using this kind of heavy lifting equipment is still expensive, which makes it often uneconomically to apply for caisson quay walls (see appendix N). The design considerations for a floating-in caisson has therefore shifted from technological restricted feasibility to economic feasibility.

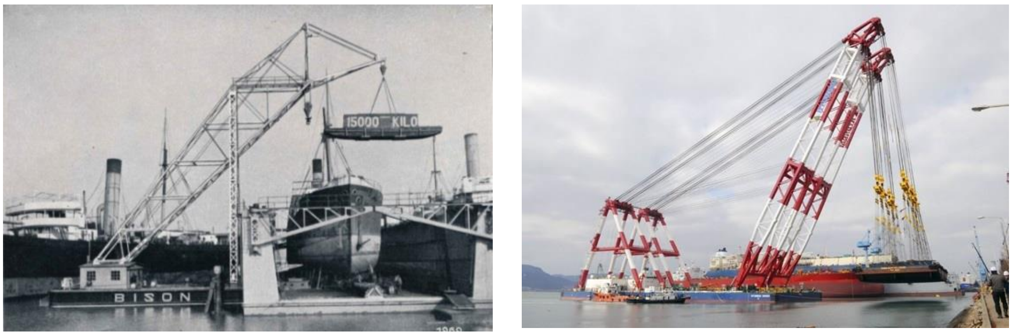


Figure 4.10. Left: Bison 66 tonne sheerleg (1910), image: rdm-archief.nl
Right: Hyundai 10,000 tonne sheerleg (2015), image: hhi.co.kr

4.2.3. Conclusions

The choice for applying a launching technique is currently not limited by technology or availability of equipment. Modern floating equipment, such as floating dry docks and sheerlegs are able to lift entire caissons of extraordinary dimensions. It is therefore mostly an economic consideration which is highly influenced by local conditions and possible depreciation rates.

However, as discussed in section 3.4, the caissons characteristics can influence the feasibility of a particular launching method. If for instance the draught of a caisson is significantly reduced, a floating dry dock might become superfluous and a relatively simple slipway can be used. Therefore, benefits within the launching phase are only achievable when significant draught decrease is obtained.

[6] See for instance the weight calculations of reference caissons (appendix E-H)

4.3. Synthesis of design differences

Construction technology and equipment is currently more drastically improved. A century ago, it was challenging to build a 220 tonne caisson in less than a month, where the current construction speed has increased to one 10,000 tonne caisson a week. Due to the relatively fast construction techniques, it is not necessary to build many (usually less than 5) caissons simultaneously. The high investment cost of the high performance formwork can therefore still be profitable.

These aspects have influenced the design of caisson quay walls considerably over the last century. Combined with the analysis from the previous chapters, an overview of differences is presented in the table below.

Design differences	**Caisson design** 1903 - 1930	**Caisson design** 1970 - 2017
Operational stability	Sufficient width ≈ 0.50H	Sufficient width ≈ 0.75H
Wall thickness (inner)	≥150mm	≥250mm
Wall thickness (outer)	≥150mm	≥300mm
Concrete strength	C12/15	C45/55 (NSC)
Concrete durability	High w/c ratio required for workability 10mm cover	Low w/c ratio CEM II or CEM III 60mm cover
Concrete execution	Labour intensive, cumbersome, low concrete quality	Less labour intensive and improved equipment
Construction time of one caisson	> 1 month	< 1 week
Formwork and falsework	Timber formwork	Slipform / steel formwork systems
Lifting equipment (floating)	Max. 66 tonne	Max. 10,000 tonne
Safety & Design	Depending on height of construction	Depending on method of construction

Table 4.3. Design differences (1903-2017)

The savings of building materials is a distinguishing property of the overturning caisson principle. However, these savings are not applicable when it is at the expense of reliability and durability aspects. A new balance must be found between material savings, durability and maintenance aspects.

Overall, the challenge of constructing a reinforced concrete caisson quay wall shifted over the last century from a technical to an economic challenge. The most economical solution can be found when the complete life cycle is considered.

5. Economic Shifts (1903 – 2017)

5.1. Construction costs: gravity based quay walls

In order to quantify the present value of the first quay walls, the historical data and original cost estimate is transferred to a 2008 values to verify relative differences between more recent gravity quay wall construction projects. The values can therefore be compared to figure 5.1, which shows cost variations of gravity quay walls relative to the retaining height (De Gijt, 2010).

The 2008 values of the first caissons are calculated based on the cost estimate from 1903 (ref. A1). The reference provides all cost estimates in Chilean pesos, which cannot easily be transferred into present values. Hyperinflation, fixed exchange rates (crawling peg) and even changes to other currencies (e.g. peso – escudo – new peso) over the last hundred years make it almost impossible to provide a reasonable estimate. However, from the year 1885, the Chilean peso was changed to a gold coinage, which pegged the peso to the British pound with a rate of 13.33 peso to 1 pound. From this knowledge, the quay construction costs could be calculated following inflation rates of the British pound[7]. Based on this path, the following quay wall construction costs per running metre are found:

- ■ Rectangular caisson quay wall: € 23.600 / m^1
- □ L-shaped caisson quay wall: € 13.750 / m^1

After correction for inflation, the cost of the first rectangular caisson quay wall is about average for the particular height of 14 metre. The economic L-shaped caisson is significantly less costly. It is positioned in the lower section of the data points, which indicates the economic advantages.

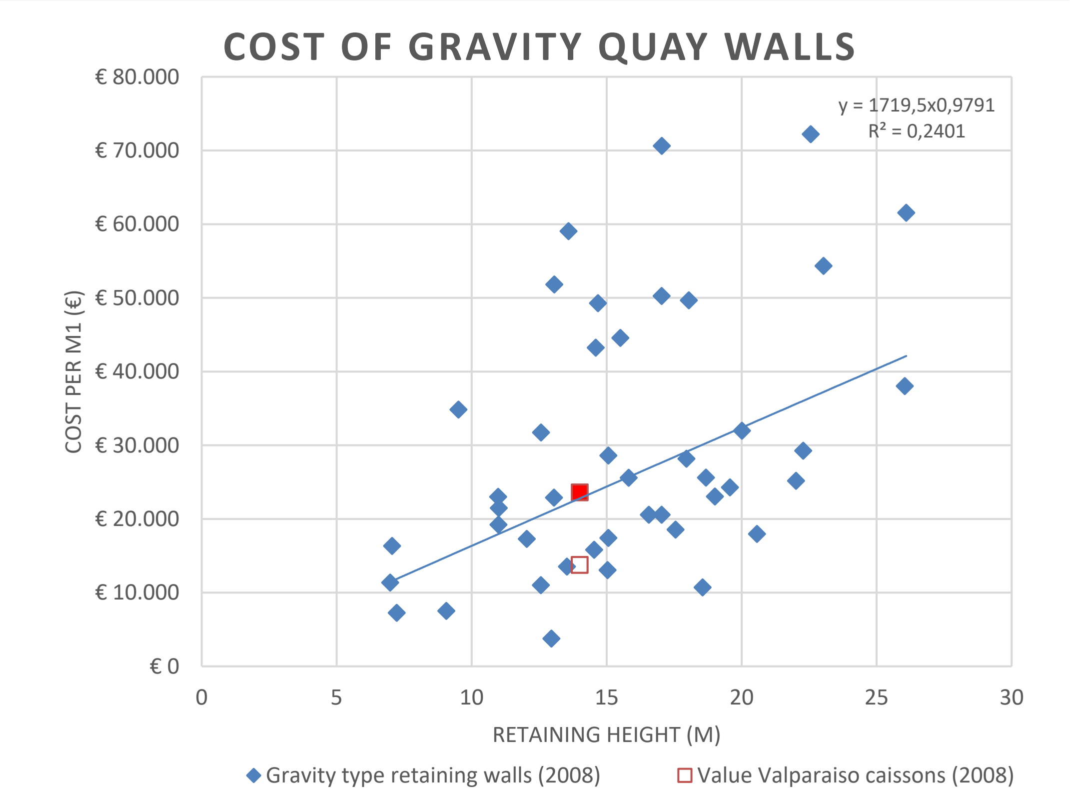


Figure 5.1. Cost of gravity quay walls (after De Gijt, 2010)

[7] Inflation: the Value of the Pound 1750-1998, House of Commons (Twigger, R., 1999)

5.2. Historical deviations per element

The overturning concept resulted in a material efficient quay wall structure. Especially the application of reinforced concrete was consciously considered. The accurate and lean engineering had large economic advantages due to the relatively high price of reinforced concrete as building material. From the volumes and given prices per metre quay, the following price per units has been calculated for the first caissons (1903):

Price per unit	*Original Value 1903*	*Present Value 2017*	*Procurement 2017*	
	Chilean Pesos ($)	*Euro (€)*	*Euro (€)*	*Rate*
Reinforced concrete*	71.75 $ / m^3	650 € / m^3	200 € / m^3	0.31
Sand-concrete (weak) compartment fil	12.00 $ / m^3	110 € / m^3	120 € / m^3	1.10
Rubble stone backfill and foundation bed	5.00 $ / m^3	45 € / m^3	60 € / m^3	1.33
Masonry superstructure	32.30 $ / m^3	300 € / m^3	300 € / m^3	1.00
Construction and repair of formwork, falsework	1.55 $ / m^2	14 € / m^2	70 € / m^2	5.00
Launching, transport and placement	16.15 $ / m^3	150 € / m^3	120 € / m^3	0.80

Table 5.1. Historical deviations - *Concrete including an estimate of 50kg/m^3 reinforcement steel

The provided prices rates per unit (table 5.1) include labour hours. The first column shows the original value, retrieved from historical data and the cost estimate. The second column shows the present value after correcting for inflation. Furthermore, an estimate is made for the procurement of the listed elements. It can be seen that the costs of two elements; reinforced concrete and launching are estimated to be drastically reduced, while on the other hand, the costs of rubble stones, and traditional formwork has increased. This economical shift reduces the urgency to reduce the application of reinforced concrete. However, if a reduction of building materials can be obtained without raising costs of other elements, still significant cost savings are expected.

It must be noted that the launching costs from table 5.1 are uncertain estimations. The values are based on based on the cost-data presented in appendix M from which a simplified estimate is made, as presented below:

Launching cost estimation
Based on second hand procurement and 100% depreciation:

5,000 tonnes floating dock:	*€ 3,000,000,-*
120 tonnes sheerleg:	*€ 1,000,000,-*

Direct launching costs: € 4,000,000,- / (3481 x 9.60) = € 120,- /m3

Reasons for cost deviations

The cost of reinforced concrete was three times more expensive. This, while a small decrease would be expected if one observes that reinforced concrete consists of steel, gravel, sand and cement. A possible explanation for the drastic change in price would be that the complete process of casting concrete has been greatly improved over the years, as also discussed in the previous chapter.

The procurement raw materials such as gravel, sand and cement lowered significantly over the last century. The trends can be seen in the plotted graphs in appendix N, from de Gijt (2010). On the other hand, the cost of steel and riprap has generally increased over the last century. These trends correspond to the rate changes given in the table above. The drastic cost reduction of reinforced concrete cannot only be explained by a shift in value of building materials. Probably, the improved construction methods, construction technologies and material science are responsible for the lower costs. On the other hand, traditional formwork costs have increased over the last century. The exceptionally low price of formwork explains why shape complexity was of minor importance.

5.3. Cost components: formwork and reinforced concrete

A price-shift can also be seen by comparing the 1903 cost components of reinforced concrete and formwork with a recent slipformed caisson project (>50 no. caissons). The outcome of the comparison is presented in figure 5.2., where both costs components are considered including labour costs.

The concrete price of the first caissons was determined by 85% from the use of concrete and reinforcement itself. Just 15% of the costs originated from formwork and falsework. Due to the minor contribution of formwork costs, the caisson geometry could become rather complex without significant cost changes. In combination with high costs of reinforced concrete it was highly beneficial to save on building materials.

These costs components are nowadays distributed differently. From a slipformed caisson reference project, recent costs are analysed. For a complete caisson, the cost division of formwork to reinforced concrete is approximately 38% to 62%. By analysing the caisson walls only, it is found that the costs of slipforming is approximately equal to reinforced concrete (both including labour costs).

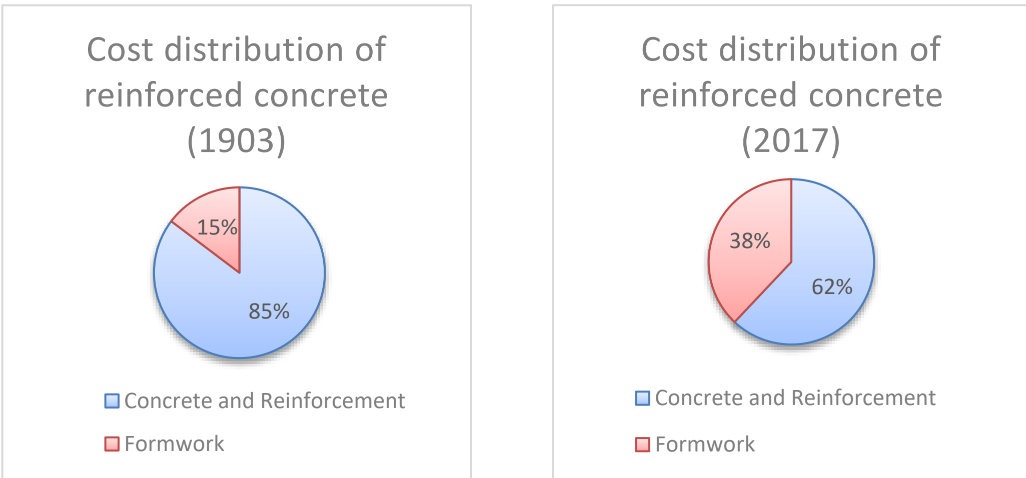


Figure 5.2. Comparison between the cost distribution in 1903 and 2017

5.4. Synthesis

After corrections for inflation, the total costs for the first caisson quay walls turn out to be similar to current projects. The economical L-shaped caissons are however on the lower side of the scatter. Historical deviations per element show large cost variations

The cost variations cannot be explained by shifts in procurement costs of raw building materials. Developments of construction techniques and technologies must therefore be responsible for the lowered costs of reinforced concrete. From a comparison between the first caisson projects and a resent project, it is found that the costs of formwork increased from 15% to 38% of the concrete cost distribution. Lowered concrete costs indicate that savings are nowadays less significant to the overall construction costs.

The application of reinforced concrete is estimated to cost approximately 1/3 of the original value. On the other hand, the price of formwork and falsework increased by a factor 5. These aspects substantiate the choice for the complex overturning geometry and recent shifts to increasing concrete use and rectangular shapes.

6. Technical Feasibility (2017)

6.1. Considered designs

The technical feasibility study comprises caisson designs based on conditions presented in appendix D. This chapter is thereby focussed on determining main dimensions of the caissons. The first overturning caissons were constructed over a century ago, and are therefore theoretically constructable. However, as discussed in chapter 3 and 4, technical requirements have changed over the last century. An overview of the most noteworthy developments are presented in table 6.1 below.

Technical feasibility developments	Development	Consequence
Operational conditions	Increased quay loads	Width increase and changed floating equilibrium position
Stability requirements	More stringent operational stability demands	Width increase and changed floating equilibrium position
Quay height	Larger quay height	Weight increase
Durability aspects	Increase of concrete cover and design life	Weight increase
Material properties	Increased concrete and reinforcement strength	Weight reduction

Table 6.1. Overview of technical feasibility developments.

It is not intended to find the significance of all individual elements presented in the table. By designing a new overturning caisson concept according to the Eurocode and British Standard, the developments are inherently taken into account as a whole.

For the feasibility study, four different caisson designs are made and analysed. The primary goal is to compare the performance of an overturning caisson with a traditional box caisson. Secondly, the influence of width increase is analysed. In order to compare structures properly, their width and height of concepts is kept equal. One concept is thereby bound to the governing design width of the other concept. The following performances are compared:

1. Global stability – performance in terms of stability factors;
2. Launching and transport – performance in terms of draught and floating aspects;

The first step is to design an overturning caisson which satisfies the floating and operational stability requirements. After this, a rectangular caisson is designed having the same width. Because of this sequence, the rectangular concept will contain superfluous operational stability. Alternatively, transport conditions such as draught and floating stability can differ for each design. The used approach allows a comparison in terms of global stability and the floating equilibrium positions (draught). On the other hand, material savings are not entirely fair to compare since not all designs are optimal.

The rectangular caissons are not primarily designed for their operational conditions, while the overturning caissons are optimized on these conditions. This is done in order to obtain acceptable transport characteristics in terms of draught and floating stability. For a slender box caisson which is designed for operational conditions only, a vertical floating position becomes less feasible. This statement might not be trivial and is therefore further substantiated in section 6.5.

The analysis of the overturning caisson is thereby more elaborated since these concepts had more uncertainties in terms of stability, strength and floating conditions. The analysis can be found in the appendix as follows;

- Appendix F: Overturning caisson (12.60m)
- Appendix G: Rectangular caisson (12.60m)

rubble backfill

- Appendix H: Overturning caisson (15.65m)
- Appendix I: Overturning caisson (15.65m)

sand backfill

The first two listed caissons are designed with a smaller width, which is achieved by applying a high quality backfill. For the other designs, locally-sourced sand is used from dredging activities. Therefore, the geometry and backfill are varied, while the other conditions are kept equal. The conditions can be found in appendix D and E of the report.

6.2. Caissons with sand backfill

For the wide overturning caissons (15.65m) with a sand backfill, material savings of approximately 8% can be obtained compared to a rectangular box caisson. Additionally, the draught is reduced.

6.2.1. Design differences

The most important differences between the L-shaped overturning caisson and the rectangular box caisson are listed in the table below. The most striking aspects are draught and material savings. However, material savings are rather limited (≈8%). A reason for this is the internal transverse wall, which is required to resist the hydrostatic pressure during immersion. This transverse wall also increases the complexity during construction and increases labour and resource consumption.

Caisson properties	L-shaped caisson – sand backfill	Rectangular caisson – sand backfill
Width	15.65m	15.65m
Base plate	24.45 x 15.65 m^2	24.50 x 15.65 m^2
Base plate	230 m^3	230 m^3
Walls	964 m^3	1,052 m^3
Concrete volume	1,194 m^3	1,275 m^3
Concrete volume per running metre	47 m^3/m^1	52 m3/m^1
Weight per running metre	1,199 kN/m^1	1,309 kN/m^1
Draught	11.40 m	10.60 m / 12.40 m

Table 6.2. Caisson properties (wider caissons)

6.2.2. Transport

The draught benefits of the overturning concept are negligible for the 15.65 metre wide concept (fig. 6.1.). In fact, an unballasted caisson has a larger draught than a regular caisson having the same width. However, the examined rectangular caisson does not have a stable floating position by itself. The floating stability can be increased by adding ballast water, which results in an increase of the draught compared to the overturning design. All in all, it is expected that this advantage does not contribute significantly to the overall economic feasibility.

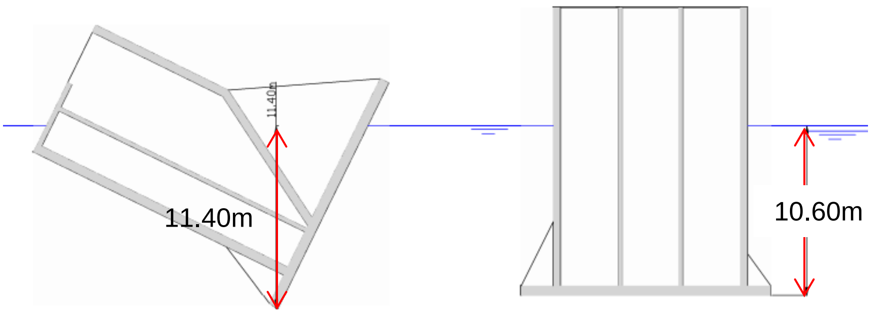


Figure 6.1. Draught comparison: overturning caisson (left), rectangular caisson (right)

6.2.3. Immersion

The second floating equilibrium position would be similar to the drawing (fig. 6.2) below. It can be noticed that the floating position is considerably deviating to vertical. This is caused by the asymmetrical shape and relatively heavy counterfort walls.

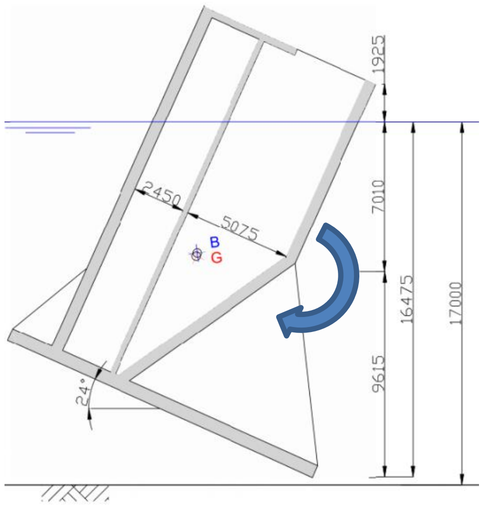


Figure 6.2. Second floating position (unballasted caisson)

The shape of displaced water (fig 6.3 left) is rotated in such a way that the buoyancy point (B) acts on the same vertical axis as the centre of gravity (G). Due to the rotation of the caisson, the shape of displaced water is varied by the upper triangle (hatched red), which changes the floating position. The declined back-wall causes a reduction of buoyancy (hatched green) and a shift of the buoyancy point which is actually almost equal to the deviation of the waterline.

A straight back-wall would be a solution to obtain a more symmetrical shape and more vertical floating position. The separation walls could then be designed at an equal distance from the front and back-wall. However, the total width (at the top) of almost 9 metres must then be divided into straight compartments with 4.50 metre spans. This is a rather large wall-span for an immersion depth of at least 17.00 metre and the technical feasibility of such a concept is not evaluated.

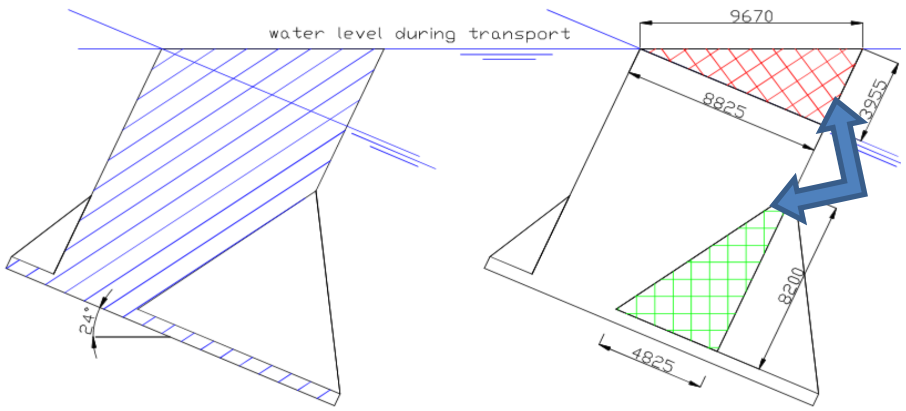


Figure 6.3. Shape of water displacement: floating position after turning

The enlargement of the compartments resulted in the requirement of an additional transverse wall. An advantage of this separation wall is the possibility to deviate water levels within the compartments. When the compartments are accurately filled, it is possible to obtain a vertical position after turning. Additionally, a vertical position could be maintained during further immersion since the water levels in the front- and back-side can be controlled separately.

The vertical position could be obtained with approximately 9.00m^3/m^1 water. This makes the first stage of immersion possible in two manners: 1. filling the front compartments before the heel flows over. 2. Filling the heel with water and obtain a vertical position during immersion by applying ballast. The first option would introduce an increased change of displacement. If the turning-process is controlled by a floating crane, a larger crane capacity would be required. On the other hand, if the turning phase is completed, a vertical position is obtained which simplifies the remaining immersion phase.

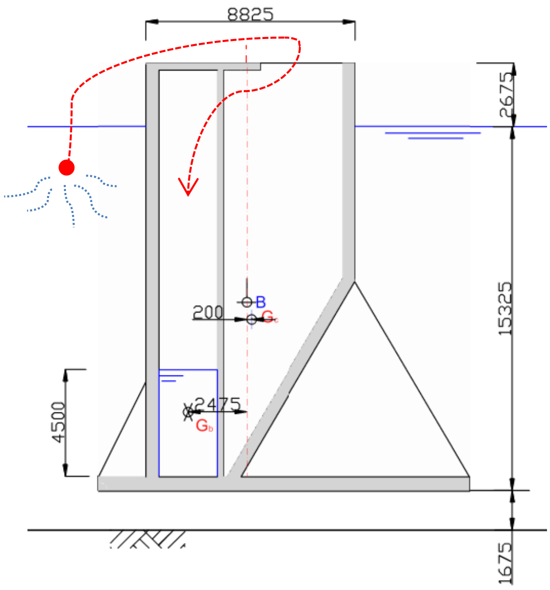


Figure 6.4. Vertical floating position with partially ballasted front-compartments

Adding ballast water to the front compartments of the caisson cannot be done from the top of the caisson since it is closed to prevent water flowing into the compartments during transport. Therefore, openings must be made at the inner side of the separation walls. This allows water to be pumped into the compartments by some detour (fig. 6.4 red dotted path). When the compartments must be filled with sand for the operational phase, the front compartments becomes more difficult to reach. This makes the filling more time consuming than the relatively simple method for rectangular caissons.

6.2.4. Operational stability

All concepts are designed to resist prescribed actions during their service life, the rectangular caissons have significantly more stability. This is caused by the increased use of materials (weight) and due to the different position of its centre of gravity. Additionally, the sloped back-wall of the L-shaped caisson causes increased soil pressure before failure. To sum up, the overturning caisson has:

- Less weight due to concrete material savings;
- An eccentrically positioned centre of gravity;
- An increased horizontal soil pressure during failure (trapped soil wedge);

These factors combined result in the safety parameters presented in the table below. The table lists the governing mechanisms and the ratio of safety above the requirements provided in the British Standard (BS-6349) and Eurocode. A factor of 1.0 satisfies the design criteria sufficiently.

Caisson failure mechanisms	L-shaped caisson – sand backfill	Rectangular caisson – sand backfill
GEO – SLS Overturning	1.1	1.2
GEO – ULS Overturning	1.6	2.3
GEO – ULS Sliding	1.0	1.3
EQU – ULS Overturning	1.8	2.3

Table 6.3. Caisson failure mechanisms (wider caissons)

6.3. Caissons with rubble backfill

For the most slender caisson (12.60m) with a rubble backfill, material savings of 15% can be obtained compared to a rectangular box caisson. Also the draught is reduced by more than 10%.

6.3.1. Design differences

The differences between the L-shaped overturning caisson and the rectangular box caisson are listed in the table below. The most striking aspect is the amount of material savings.

Caisson properties	L-shaped caisson – rubble backfill	Rectangular caisson – rubble backfill
Width	12.60m	12.60m
Base plate	24.25 x 12.60 m^2	24.25 x 12.60 m^2
Base plate	183 m^3	183 m^3
Walls	802 m^3	983 m^3
Concrete volume	985 m^3	1,166 m^3
Concrete volume per running metre	41 m^3/m^1	48 m^3/m^1
Weight per running metre	1,016 kN/m^1	1,202 kN/m^1
Draught	9.60 m	10.70 m / 12.60 m

Table 6.4. Caisson properties (slender caissons)

6.3.2. Transport and immersion

The overturning caissons have sufficient floating stability without ballast water. This is beneficial compared to other slender rectangular caissons, which lack intrinsic floating stability. It could therefore be a design consideration to widen the rectangular caissons in order to increase the metacentric height. When the caisson is widened, a rubble backfill might become superfluous. Perhaps, this is part of the reason why rubble backfills are nowadays less extensively applied for caisson quay walls. Therefore, an overturning caisson in combination with a rubble backfill results in synergy advantages, where a regular caisson tends to obtain limited benefits of a rubble backfill.

However, these savings require an entirely different construction method and assistance of a (120 tonne) floating crane during immersion. It is therefore not evident whether this results in a reduction of marginal costs.

The caisson heel can be filled with approximately 13 m^3/m^1 water before it reaches the critical state for which water can freely enter the heel of the caisson. The heel has a total volume of approximately 28 m^3/m^1. Therefore, a volume of 15 m^3/m^1 shall quickly fill up the remaining part (hatched in blue). Due to this change of the centre of gravity and buoyancy, a new equilibrium floating position shall be found.

The velocity of the turning process and sway magnitude due to the dynamic actions are unknown (not calculated). If these conditions result in unacceptable behaviour, a floating crane can be used as possible solution to control the turning process. The added weight of water that enters the heel can then be carried by the floating crane. The total added weight due to a volume of 15 m^3/m^1 water amounts:

$$W_{water} = \gamma_w \cdot V \cdot L_{caisson}$$
$$W_{water} = 10.30 \cdot 15.00 \cdot 23.25 = 3{,}592 \text{ kN/caisson}$$

Assuming that the centre of gravity of the entering water is on the same location as the compensated lift of the floating crane, a controlled turning process would require a 400 tonne capacity floating crane. However, if a sheerleg must reach the caisson from the front of the wall, jib hoist could be required which can result in a larger size (>400T) sheerleg.

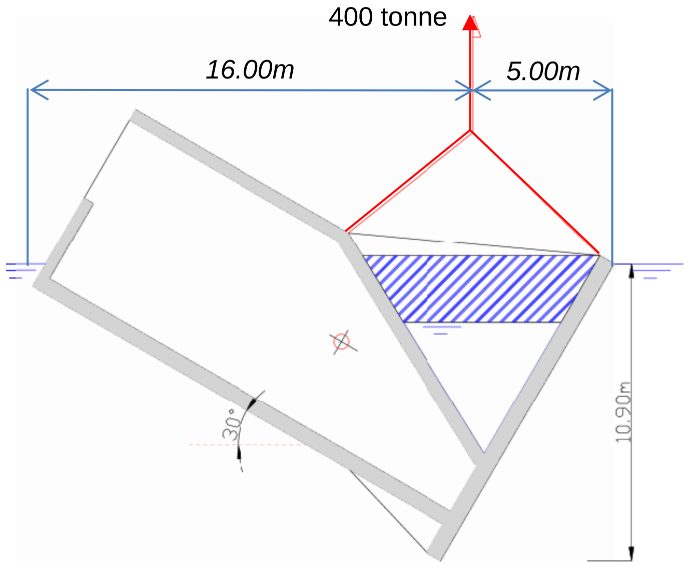


Figure 6.5. Turning process assisted by a 400 tonne sheerleg

When the turning progresses, the required uplift becomes lower. However the caisson is designed to be self-floating, a certain crane assistance remains necessary since a vertical position is not intrinsically obtained. An upward force of approximately 120 tonne would provide equilibrium in a vertical floating position. The equilibrium position is shown in figure 6.6. This is in contrast to the first caissons, which were designed to float almost vertically after turning.

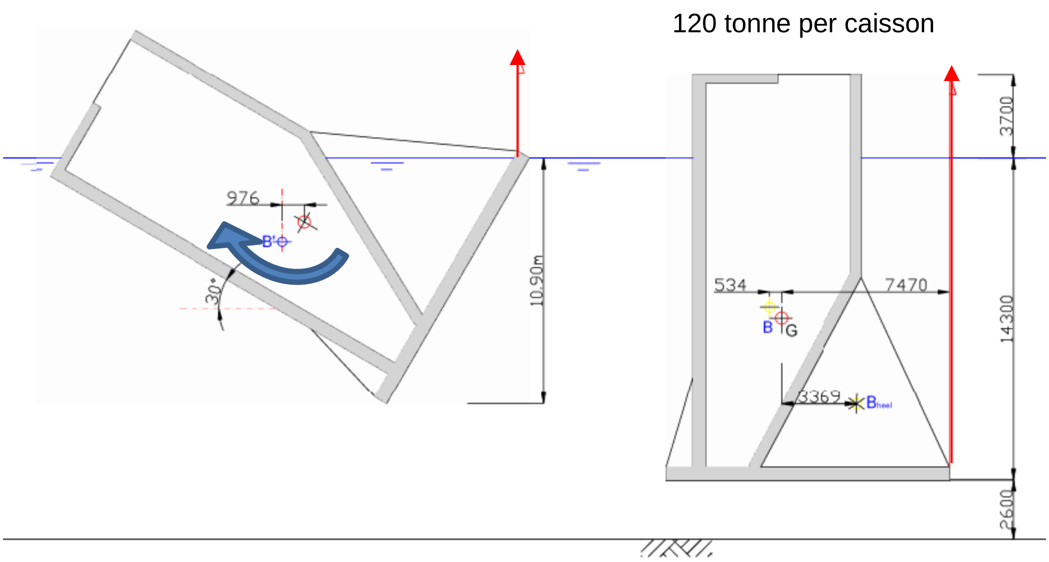


Figure 6.6. Change of buoyancy point and turning process

6.3.3. Operational stability

The difference in operational stability for the slender caissons is similar to the previously presented operational stability factors of the wide caissons (section 6.2.4.). The safety parameters corresponding to the slender caissons are presented in the table below. The table lists the governing mechanisms and the ratio of safety above the requirements provided in the British Standard (BS-6349) and Eurocode. A factor of 1.0 satisfies the design criteria sufficiently.

Caisson failure mechanisms	L-shaped caisson – rubble backfill	Rectangular caisson – rubble backfill
GEO – SLS Overturning	1.0	1.5
GEO – ULS Overturning	1.2	2.0
GEO – ULS Sliding	1.4	1.8
EQU – ULS Overturning	1.6	2.4

Table 6.5. Caisson failure mechanisms (wider caissons)

6.4. Evaluation of design changes

6.4.1. Comparison of caisson geometry (1903 vs 2017)

The cross-section of a modern overturning design is compared to a linearly scaled model of the original design (see figure 6.7 below). The width of the new caisson design has increased over 25%. Additionally, the wall and floor thickness has increased, which results in increased use of reinforced concrete per running metre quay.

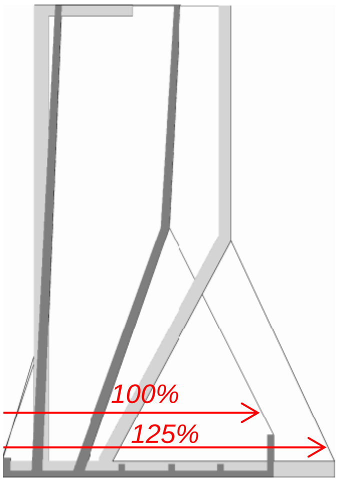


Figure 6.7. Comparison between a linearly scaled caisson (dark grey) and the new design (light grey)

6.4.2. Change of floating equilibrium position

The first caissons were designed as relatively slender caissons with a height-to-width ratio of 1.75. This resulted in a large benefit to float the caissons in a horizontal position. When the caisson width increases, the advantage of horizontal transport becomes less significant. The difference between caisson draughts is shown in figure 6.8. It can be seen that the original caisson has a limited draught of approximately 6.00 metre, where the new design ends with 9.60 metre. This increase is caused by the changed geometry and increased material use.

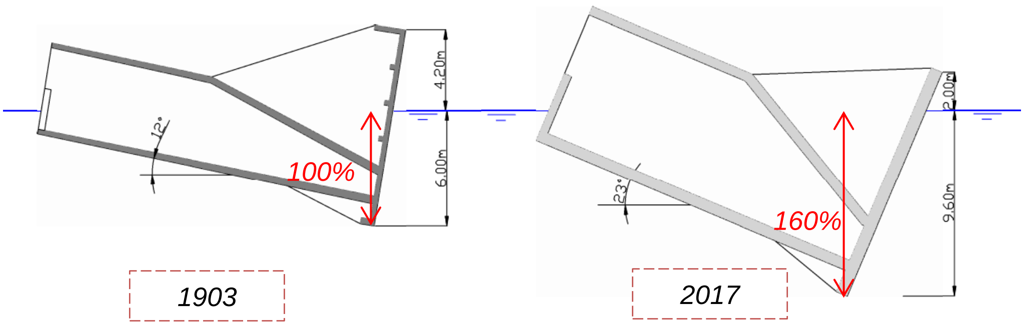


Figure 6.8. Comparison of floating positions; linearly scaled caisson (dark grey) and new design (light grey)

For the previously discussed overturning caisson concepts, the enlarged heel resulted in a more diagonal floating position. Based on this, it can be seen that the significance of an intended horizontal floating position decreases when the width becomes larger.

The desire of a horizontal floating position decreases when the floating position is analysed from the perspective of a rectangular caisson. Namely, a vertical floating position becomes more favourable when the caisson width increases, while the horizontal floating position becomes less favourable.

This is caused by the following aspects that are affected by widening a caisson:

(1) The floating equilibrium position of an overturning caisson becomes increasingly diagonal;
(2) The draught of an vertical floating object decreases and the intrinsic stability of a vertical floating position increases;

These aspects are substantiated in the following sub-sections.

Floating equilibrium position (1)
The changed floating position becomes clear when the original caisson is drawn next to the current design for a rubble backfill. Due to width and weight increase, the draught has increased drastically. The advantage of the particular caisson shape is thereby mitigated. The calculation for the floating equilibrium position can be found in the corresponding appendix F.

For circumstances where load reducing measures are taken (rubble backfill), the degree of benefits reduced due to changed design requirements over the last century. It is therefore questionable whether this solution is currently economical when one compares it to a rectangular caisson.

Draught and floating stability (2)
The draught of a horizontally floating object increases by width (b) increments, while the draught (d) remains equal or reduces when the object is considered in vertical position. The comparison between floating positions is schematized in figure 6.9. Besides draught considerations, floatation of light-weight slender objects can be limited by stability requirements.

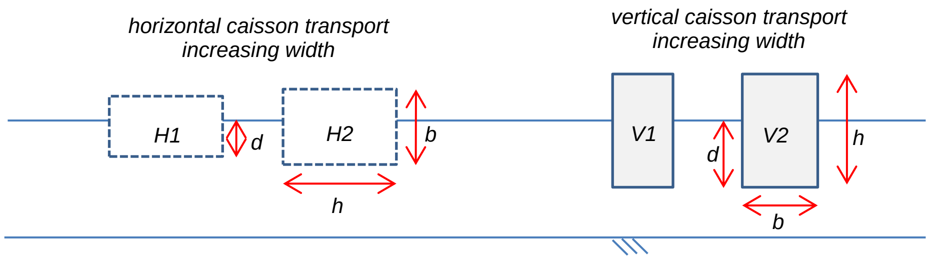
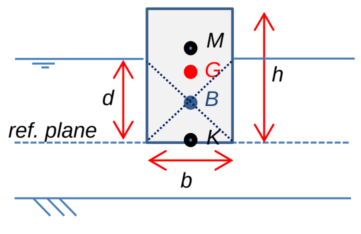


Figure 6.9. Caisson transport-shape relation

When rectangular floating objects are considered, such as presented in figure 6.9, the required width for intrinsic floating stability can be calculated. The point for which a vertical floating element becomes stable, is considered in the next sections. The relevant properties of the analysed rectangular caissons are presented in appendix G and I. The general calculation of floating stability is substantiated in appendix J.4.

A generalized rectangular floating object (caisson) is considered using the notations shown in figure 6.10. Dimensional parameters are denoted by lower case letters (h,d, and b), while the stability parameters are denoted by upper case letters (K, B, G, M). Distances from the bottom of the caisson (K) are denoted as for instance KG, which implies the distance from keel to gravity centre.



Notation	Description
h	Height of caisson
d	Draught of caisson
b	Width of caisson
K	Keel (bottom of caisson)
B	Buoyancy point
G	Centre of gravity
M	Metacentric height

Figure 6.10. Notations for floating stability analysis

The primary requirement for floating stability is a positive metacentric height (M above G). This height is influenced by the width (b), draught (d) and height of point G. The height of the centre of gravity varies when ballast water is added. The essential variables are therefore:

(1) draught;
(2) width;
(3) height of centre of gravity.

Relations can be defined based on these variables. A stability region can then be plotted for rectangular floating objects. The draught and width are considered to be most important design aspects and therefore taken as variables. The height of the centre of gravity is kept constant and considered separately for the unballasted and ballasted situations.

Intrinsic stability of a rectangular floating object
The stability of a floating object is analysed by defining dimensionless parameters. The parameters are related to the total height of the object in order to obtain an outcome which is valid for different caisson dimensions. From calculations presented in appendix J.4, the stability region is plotted as the elliptical curve presented in figure 6.11.

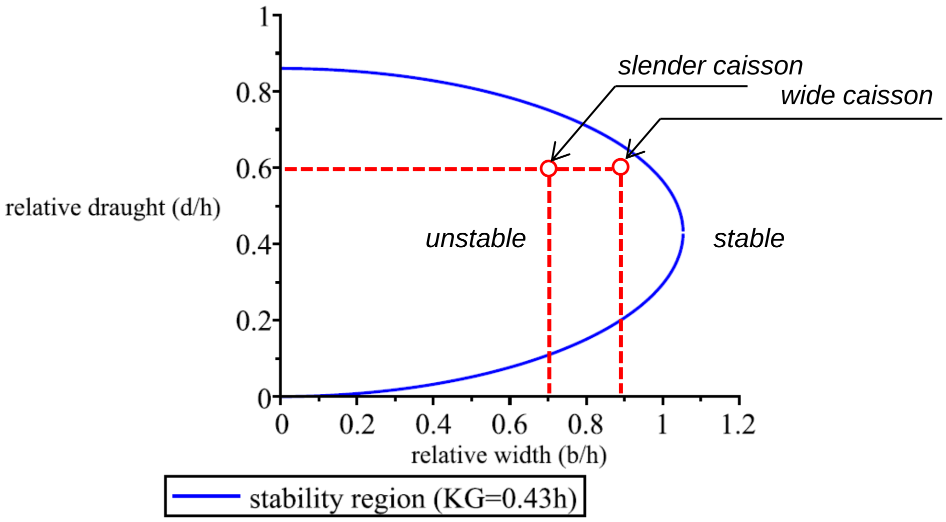


Figure 6.11. Stability of the considered rectangular caissons (unballasted)

The relative draught of both caissons is similar (0.59), while the relative width varies (0.70 and 0.87). The intersection points in the graph show that both objects are unstable without ballast. However, the wide caisson is almost stable by itself. This point is already located near the blue boundary.

Ballasted stability of a rectangular floating object
A similar approach can be used for the analysis of the floating stability of ballasted caissons. Due to the weight increments, the centre of gravity reduces and the draught increases. This results in the change of parameters.

The stability region for the ballasted situation is presented in figure 6.12. It can be seen that the slender caisson is just outside the boundary and has a positive metacentric height. However, the wide caisson has considerably more stability and does therefore require less adjustments for transport.

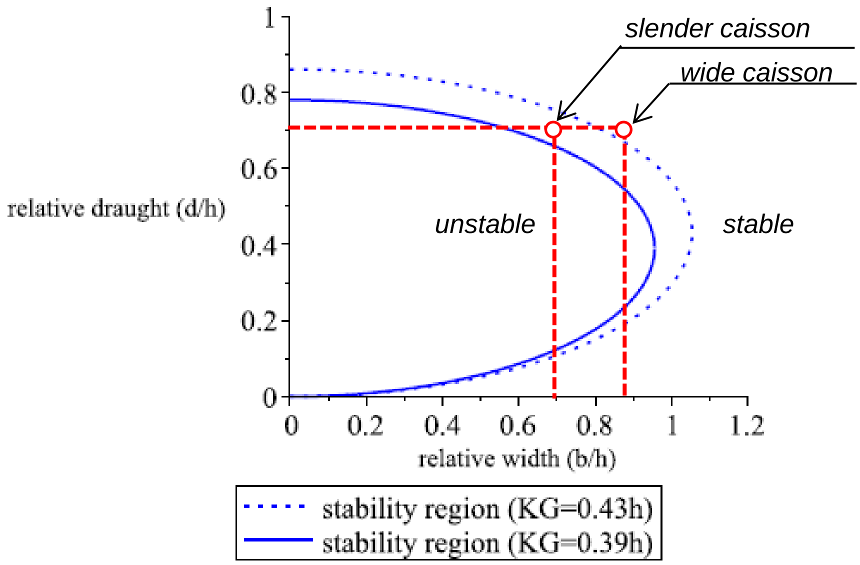


Figure 6.12. Stability of the considered rectangular caissons (ballasted)

6.4.3. Synthesis

Considering the first caissons designed in 1903, one can imagine that the stability criterion is hard to meet for a vertical floating caisson. At this point in time, a floating box caisson with a high slenderness and thin concrete walls (starting from just 150mm), have resulted in low relative width and weight. Without adjustments to the design, these objects would be unstable in vertical position. This can be seen from the figures 6.11 and 6.12. When a high slenderness object is considered, objects can only be stable for very low and high draughts. Different design requirements and economic shifts over the last century resulted in wider and heavier caissons, which additionally improved floating stability conditions. Subsequently, the urgency of horizontal transport diminished over time.

When one starts designing a caisson from the objective of horizontal transport, the following aspects reduce the advantages or even result in disadvantages of the floating position:

(1) Increasing caisson width;
(2) Increasing caisson weight;
(3) Lowered centre of gravity (e.g. increased bottom-slab thickness).

Transport requirements can be met without adjustments when a caisson reaches the relative width of roughly 0.80 and a relative draught of roughly 0.60. In terms of transport, regular caissons are not expected to profit from a horizontal floating position.

6.5. Conclusion

From a technical point of view, the most striking advantages of the caisson geometry are draught and material savings. Compared to a rectangular caisson, the overturning caisson combined with a rubble backfill results in the largest reductions. This is caused by the reduced horizontal quay load and the subsequently increased caisson slenderness. Nevertheless, draught and material use are nowadays significantly higher than for the original concept from 1903. Reasons for the mitigation of benefits are;

- Stability demands → increased width; less caisson slenderness;
- Durability demands → larger concrete cover and wall thickness;

The strengths of the overturning concept are therefore reduced, but still present. In terms of material savings and transport, the overturning caisson combined with a rubble backfill is the most beneficial. However, also construction technologies and economic aspects must be considered to give a decisive answer whether the concept is feasible.

Material savings

When designing a caisson according to current standards, the overturning concepts allow 8% to 15% material savings. The largest material savings can be obtained when a rubble backfill is applied. If quay loads increase, the required caisson width must become larger which results in a reduction of material savings relative to a rectangular caisson.

Lowered draught

For a rectangular caisson, ballast water must be applied to obtain sufficient floating stability. The L-shaped caisson has a lower draught and larger metacentric height during its first floating position. Due to the increased stability, no ballast water is required during transport. Because of this, the overturning concepts have a lower draught than their rectangular counterparts. The development of caisson designs in relation to draught can be made clear by the following tables. From table 6.6, it can be seen that the technical design changes mitigate draught benefits drastically.

Original caisson concepts	Linearly scaled overturning caisson (1903)*	Modern overturning caisson (2017)
Floating stability	sufficient	sufficient
Draught	6.00 metre	9.60 metre
*The linearly scaled model lacks operational stability and durability requirements and is only used as benchmark.		

Table 6.6. The linearly scaled overturning caisson compared to a modern concept

Nevertheless, from table 6.7, it can be seen that draught benefits are still present. This is predominantly caused by insufficient floating stability of the rectangular counterpart.

Rubble caisson concepts	Modern overturning caisson (2017)	Modern rectangular caisson (2017)	Modern rectangular caisson (2017)
Floating stability	sufficient	insufficient	sufficient
Draught	9.60 metre	10.70 metre	12.60 metre

Table 6.7. Caisson concepts designed in combination with a rubble backfill

For caissons with a sand backfill (table 6.8), draught benefits become insignificant. Only stability requirements result in a lower draught of 1.00 metre. It is however reasoned that this problem can also be solved by external sponsons or floating bodies. In combination with such temporary stabilizing elements, draught can then be lower than the comparable overturning caissons.

Sand caisson concepts	Modern overturning caisson (2017)	Modern rectangular caisson (2017)	Modern rectangular caisson (2017)
Floating stability	sufficient	insufficient	sufficient
Draught	11.40 metre	10.60 metre	12.40 metre

Table 6.8. Caisson concepts designed in combination with a sand backfill

7. Construction Technology (2017)

7.1. Construction and formwork techniques

As discussed in the previous chapter, significant material savings can be made when the overturning concept is applied. The required formwork area can be reduced by almost 50%. This reduction is obtained by the increase of horizontal elements (which require less formwork), in combination with element- and material- reduction. Comparing the two caisson designs from respectively appendix F and G, the following overview can be made:

Caisson properties	L-shaped caisson – rubble backfill	Rectangular caisson – rubble backfill
Concrete volume	985 m^3	1,166 m^3
Reinforcement*	101 kg/m^3	92 kg/m^3
Horizontal form area	650 m^2	350 m^2
Declined form area	450 m^2	0 m^2
Vertical form area	2400 m^2	5500 m^2

*estimated average amount of reinforcement (appendix F.7)

As described in chapter 4.1, formwork for caisson construction is nowadays well-exploited. The shutters can be reused for over hundred times. This makes the required formwork area not an intrinsic advantage. Namely, as a consequence of the caisson geometry, the construction method changes drastically in terms of applicable construction technologies and working schedules. The associated construction method must therefore be considered to define actual benefits.

7.1.1. Formwork system

For constructing a large number of caissons, a job-built formwork system for the construction of an overturning caisson is the most economical solution. These forms are designed and constructed to meet the requirements of the particular project. The rather complex caisson shape and high repetition justify high formwork investments. The considered reference project (appendix D), can be made with 60 caissons with a length of 24 metre, which number is large enough for job-built formwork.

Current techniques allow the formwork to be able to resist high fluid pressures with limited deformations. Formwork can be designed to be reused for over 100 times, which is more than sufficient for the considered reference project. The overturning caisson can therefore be constructed without large concern of formwork limitations.

The shape complexity basically results in the traditional formwork method described in section 4.1. Other construction methods, such as the later developed Maas formwork system, a climbing or slipform technique cannot be applied. Also repetitive tunnel formwork systems cannot efficiently be applied due to the irregular shape of an overturning caisson. Repetition can therefore be found in the number of caissons, and not within the formwork elements of a caisson.

7.1.2. Declined back-wall

Casting a declined wall must be well-considered before the caisson construction starts. Normally, the formwork is closed on both sides. The upper side must then be prevented to be lifted by the concrete liquid pressure. The upper formwork elements can be fixed by:

- Connecting formwork by ties;
- Adding ballast to the upper formwork elements.

A disadvantage of applying ties is the additional labour required for installation, removal and concrete patching. On the other hand, adding ballast to the upper formwork section results in an increased load on scaffolds.

It can also be decided to cast concrete without an upper formwork element (*tegenkist*). The technical feasibility of this casting method is presented in the Dutch journal Betoniek 2012-12. The angle under which the caisson back-wall is designed is similar to this project in Nijmegen. This method appeared to require special attention to the concrete mixture to obtain an optimal workability. Also concrete finishing and curing was rather challenging. Therefore, custom equipment was made to be able to finish the top layer in a safe and consistent manner.

Figure 7.1. Concrete formwork sloped at 27 degrees for the "Promenadebrug Nijmegen, the Netherlands (2015). Photo: Betoniek 2015-12

Working on a sloped element introduces accessibility and safety concerns. Examples of activities which are affected by the slope are:

- Formwork cleaning and maintenance;
- Reinforcement placement and fixing;
- Casting, finishing and curing concrete.

In order to minimize the required working time on the sloped forms, it is advised to prefabricate reinforcement meshes as much as possible. The concrete finishing and curing activities can be minimized by applying a closing formwork element (fig. 7.3). On the other hand, using a closed form limits the location for pouring and vibrating concrete. The concrete must then flow over the declined form downward through the rebars. At this moment, concrete aggregates should not be separated from the cement matrix. After pouring, compaction becomes challenging since mechanical vibrators must be lowered from the top. To overcome these obstacles, self-compacting concrete may be required.

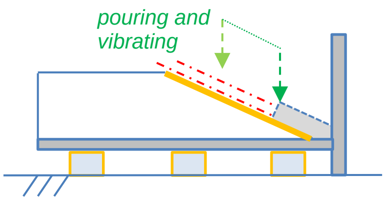


Figure 7.2. Without upper form; concrete pouring and compaction possible over entire wall

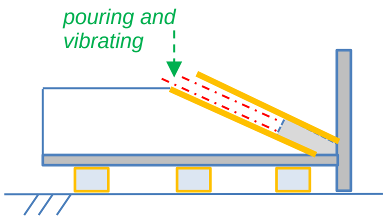


Figure 7.3. Closed form; concrete pouring and compaction only possible from top

Both casting solutions (fig. 7.2 and 7.3) therefore have their pros and cons. This brief consideration does not prove that a particular casting method prevails, but disregarding the chosen solution, it can be concluded that significant investments are required to obtain the desired concrete quality and performance.

7.1.3. Concrete casting plan

In contrast to caissons constructed with a slipforming technique, the concrete casting for an overturning caisson cannot be continuous. Time differences between elements result in two aspects which require more attention;

- Restrained deformations during hardening;
- Construction joints.

Restrained deformations result in an increased risk of cracking and can enlarge crack-width. Depending on the chosen construction method and process, a certain hardening time difference occurs which result in strain differences. Time between two casts is therefore desired to be as short as possible. If this is an insufficient measure, restrained deformations can be prevented by application of low shrinkage concrete mixtures. A final and more detailed study on maturity and thermal behaviour must be performed to conclude whether (cost-significant) adjustments are required to the mixture.

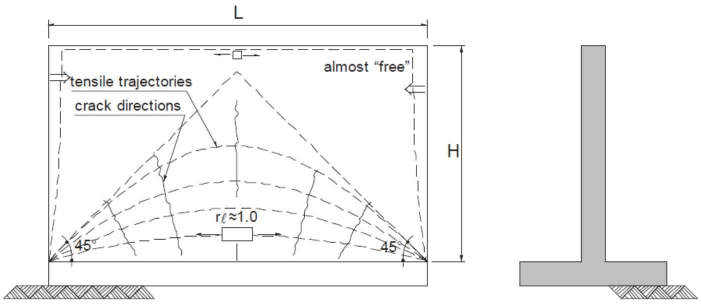


Figure 7.4. Deformations of a wall cast on an already hardened floor slab; degree of restraint, direction of tensile trajectories and cracks indicated[8]

Construction joints do not necessarily reduce concrete quality, but it requires extra attention. The surface of the joint must be rough, clean and moistened. Furthermore, sufficient starter bars cross the construction joint.

The number of joints is basically determined by the casting sequence. Some construction joints, such as wall-slab connections, cannot be avoided. The least number of joints is thereby obtained when dividing the casting into three phases:

1. Horizontal element: front-wall
2. Vertical elements: compartment walls, counterforts and baseplate
3. Horizontal element: back-wall

This division results in mainly horizontal construction joints. A large benefit of this type of joint is that rebars can vertically extent the concrete element without adjustments to formwork panels. However, the connection between the back-wall and counterforts would still require vertical construction joints.

A connection between wall elements requires more effort. Formwork must be adjusted for vertical joints, since reinforcement is able to pass for the future connection, without leaking fresh concrete. This can be achieved by designing custom reinforcement connection systems, or by systems which are already on the market. However, these systems must be able to retain considerable concrete liquid pressures to allow a fast construction process.

Division of construction into 3 phases results in an unequal casting schedule. The front-wall ($223m^3$), vertical elements ($521m^3$) and back-wall ($205m^3$) differ considerably in their quantity. It is therefore proposed to divide the vertical elements into 3 subsections. This results in a more evenly spread concrete supply and processing time.

[8] CIE5130: Concrete Structures under Imposed Thermal and Shrinkage Deformations (TU Delft)

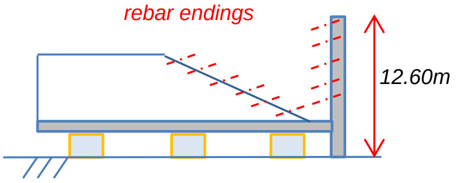


Figure 7.5. Rebar starters / endings in vertical elements

7.2. Construction process

By considering aspects from the previous section, the caisson construction is divided into three main sites; a formwork and falsework yard, an assembly line and a reinforcement prefabrication yard. The formwork yard is required to store and maintain the different shapes of shutters and scaffolds. Similarly, the reinforcement yard is required to prefabricate and store all the steel reinforcement. The assembly line, which consists of three construction phases (A, B, C), is located in between the formwork and reinforcement sites. A schematic overview of the proposed construction site is depicted in figure 7.6.

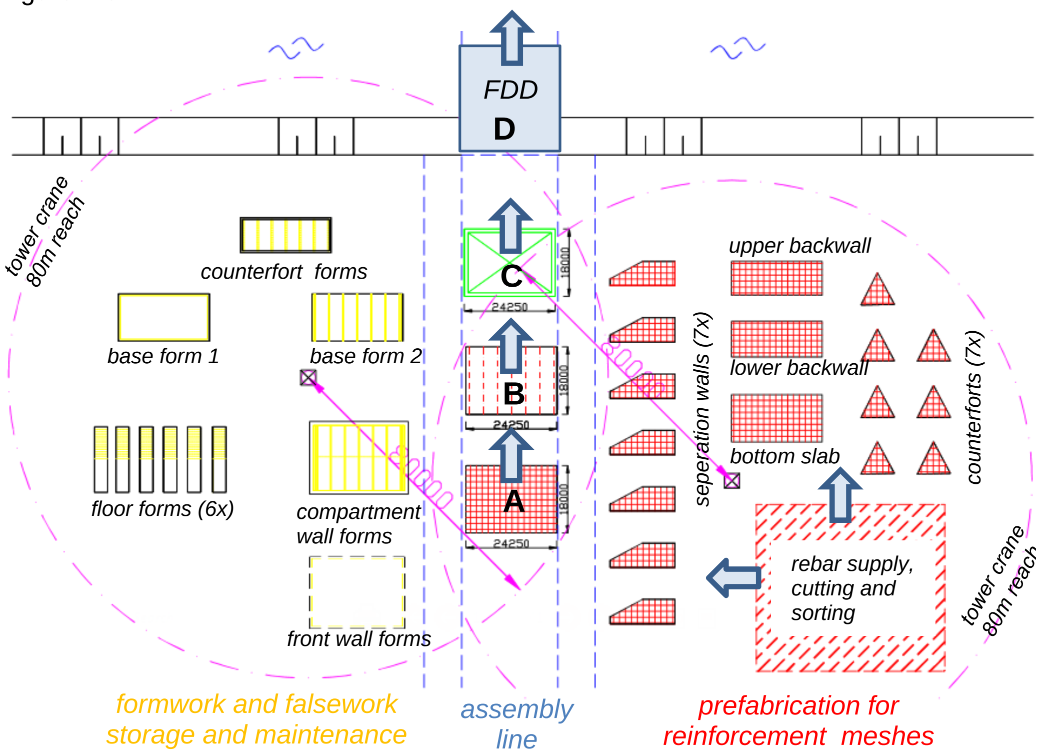


Figure 7.6. Proposed construction site layout for the overturning caisson (12.60m)

Caisson transport on the assembly line is done by hydraulic jacks, similar to the method shown on figure 4.9. Next to the assembly line, 2 x 10 metre construction roads are planned for concrete mixer trucks and concrete pumps.

7.2.1. Equipment

On the construction site drawing (fig. 7.6), two 80 metre reach tower cranes are indicated. This tower reach is nowadays feasible and allows relatively easy transport of (pre-assembled) building materials. However, regular tower cranes have a capacity up to 300 tonne-metre. This implies that just a few tonne (3.75t) can be lifted at its maximum reach.

The weight of formwork systems can be considerable, if the elements are not divided into sub-sections. In example, the weight of formwork elements for the port of Botany (appendix A) amounted 30 to 100 tonne. This, while the length of the constructed counterfort elements is not even half the length of the new overturning design. It is therefore expected that the formwork elements can weigh up to 100 tonne, if not subdivided. Also the prefabricated reinforcement meshes can weigh 10 to 20 tonne, if the reinforcement sections are kept as depicted in figure 7.6. Therefore, these formwork systems and prefabricated reinforcement meshes cannot be lifted (at once) by regular tower cranes. Three measures can be taken to overcome this problem:

1. Increase tower crane capacity to extraordinary values;
2. Use heavy lifting crawler cranes and SPMT's;
3. Divide formwork, falsework and reinforcement meshes.

Tower crane capacity (1)
One of world's largest tower cranes, a Kroll K-10,000, would be able to lift complete formwork systems over such a reach. However, sufficient lifting capacity not the only condition which has to be satisfied. The large and heavy weighing elements require decent guidance during each crane operation to prevent spinning and swinging. For instance, local weather conditions (wind) can result in undesired sway of the elements, increasing executional risks.

Working under these suspended loads must be avoided as much as possible to facilitate safe working conditions. The proposed construction yard is designed to be able to cope with this aspect. Construction workers on the assembly line may have to temporarily avoid areas where elements are lifted. Proper supervision and guidance on planning must be provided.

However, in terms of equipment procurement, such cranes are rather scarce and may cost over 3 million euros (appendix N). On top of this, the scarcity if this equipment shall increase mobilization cost and may result in high depreciation rates over the project. Combined with the previously described operational and safety aspects, this solution is considered to be uneconomical if only exploited for one quay 1200 metre long quay project.

Crawler cranes (2)
The formwork elements can be transferred at once, for instance, by SPMT's (Self-Propelled-Modular-Transporter) and heavy lifting crawler cranes. This method however, does not tackle the increased labour consumption. The lifting and transport of these elements is time consuming and requires high additional investments in heavy lifting equipment.

Divide elements (3)
For an economic construction process, the formwork systems and reinforcement meshes can be divided into several sub-elements, which allows lifting by regular tower cranes. This shall result in a drastic man-hour increase since many formwork elements must then be erected, coupled and secured on site. Also the reinforcement meshes must be lifted in different sections. This increases the work load on the assembly line, since all meshes erected and coupled locally.

In conclusion, it can be seen that none of these proposed solutions is highly economical. The construction method can thus be simplified, but labour consumption shall increase. The economies of this trade-off depend on the required number of caissons, where high investment costs can be justified if marginal costs reduce (as explained in section 1.5). It is assumed that division of elements to sections of 5 to 10 tonne results in an optimum in terms of equipment-to-labour cost division. This scenario allows lifting by regular tower cranes and crawler cranes and therefore reduces the equipment costs drastically.

7.2.2. Formwork and falsework yard

A formwork and falsework site (figure 7.6, yellow) is useful for storage and maintenance of the formwork panels. The high quality forms can be re-used for the construction of all caissons. The following formwork components are distinguished on the drawing:

- Front-wall form (4x)
- Compartment wall forms (7x)
- Back-wall forms (6x)
- Base form: front and back (4x)
- Counterfort forms (7x)

In total: 28 formwork elements

The vertical formwork elements consist of three form-types; compartment wall-forms, counterfort-forms and baseplate-forms. The compartment forms and counterfort forms are both sub-divided into 7 elements to meet lifting equipment capacities. The horizontal elements consist of two form types; front-wall forms and back-wall forms. The back-wall form has a declined section (such as fig. 7.1), and consists of 7 elements which can be placed in between the compartment walls. This results in a total number of formwork elements of 28.

As described in section 7.2.1, the time required for placement and securing formwork within tolerances can be reduced when the elements are connected and have a fixed centre to centre distance.

7.2.3. Reinforcement prefabrication yard

The reinforcement for the front-wall is fixed and casted at location A (fig. 7.6.). This element requires the largest amount of reinforcement and is therefore preferred to be fixed locally. The reinforcement for the remaining elements is fixed at the dedicated prefabrication site (figure 7.1, red). The following prefabricated meshes are distinguished on the drawing:

- Compartment and side-walls (7x)
- Straight backwall (1x)
- Declined backwall (1x)
- Bottom slab (1x)
- Counterfort walls (7x)

In total: 17 prefabricated reinforcement meshes

All reinforcement elements must be fixed and ready for assembly within one working week. This also holds for the 7 compartment walls and 7 counterforts, which means that one rebar fixing team must be able to fix one element a day. This process allows daily repetition, which reduces the working hours and lowers the probability of failures. Furthermore, the rebars are fixed on ground level which is beneficial in terms of safety. In terms of lifting capacities, the backwall and bottom slab reinforcement meshes must be sub-divided in at least 2 elements.

7.2.4. Assembly line

The assembly line (figure 7.6, blue) is designed to maximize the utilization of equipment and labour. The total construction time directly increases if the structure would be fixed to one location. For instance, other construction activities cannot start before the front-wall is casted.

Since the front-wall is the largest structural element and therefore requires the most labour and construction time, it is preferred to be casted separately on location A. On the same location, the side walls and compartment walls are constructed. When the concrete is sufficiently hardened, the element is transported to location B. At this point, the remaining caisson elements are constructed using prefabricated reinforcement (fixed on site) and the stored formwork systems. When all elements are casted, the caisson is transported to location C. Here, curing, inspection and repairs can be done. The construction phases are as follows:

A. First construction site

The reinforcement for the front-wall and toe is fixed on the assembly line. When the reinforcement mesh is finished, the front-wall and toe are casted. On a following working day, construction of side-walls and compartment walls is started. The reinforcement for these elements is already prefabricated. The total duration of phase A is allowed to be 7 days.

B. Second construction site

From prefabricated reinforcement, the back-walls, counterforts and base-slab are constructed. The total duration of phase B is allowed to be 7 days.

C. Curing and inspection site

At this location, concrete is cured and inspected. If the concrete appears to have anomalies, it can be repaired before launching. Space between the caisson and ground surface is preferable for inspection of the front-wall. This allows time for concrete to develop its strength. The total duration of phase C is allowed to be 7 days. If curing and hardening time needs to be extended, additional storage space can be made in front of the launching facility.

D. Launching facility

Due to the weight and draught similarities, the feasibility of a caisson launching method is unaffected. Similar to regular caissons, a floating dry dock is used.

7.2.5. Construction sequence on location A and B

The proposed construction method results in a construction sequence as presented in table 7.1. Six working days are planned for both construction sites. On top of this, one day is needed for demoulding, finishing, curing and transport to the next location. Therefore, it is estimated that at least one week is required for both locations. As can be seen from the table, a tight schedule must be followed. Apart from reinforcement prefabrication, the caisson construction involves daily changing activities.

Location	Activity	Duration
A	Front-wall	4 days
	Formwork preparation	0.5 day
	Reinforcement fixing	2.5 days
	Casting concrete 223m^3	1 day
	Compartment walls (incl. sides and joints)	2 days
	Formwork preparation	0.5 day
	Reinforcement placement	0.5 day
	Casting concrete 232m^3	1 day
	Finishing first construction phase	1 day
	Demoulding and curing	0.5 day
	Transport element to B	0.5 day
B	Back-wall	2 days
	Formwork preparation	0.5 day
	Reinforcement placement	0.5 day
	Casting concrete 205m^3	1 day
	Counterforts	2 days
	Formwork preparation	0.5 day
	Reinforcement placement	0.5 day
	Casting concrete 106m^3	1 day
	Base-slab	2 days
	Formwork preparation	0.5 day
	Reinforcement placement	0.5 day
	Casting concrete 183m^3	1 day
	Finishing second construction phase	1 day
	Demoulding and curing	0.5 day
	Transport element to C	0.5 day

Table 7.1. Preliminary caisson construction plan (overturning caisson, 2017)

Formwork preparation and placement of the prefabricated reinforcement are relatively simple tasks which can be repeated weakly. However, these tasks are superfluous when a gantry slipform technique is applied. This technique requires a single formwork erection for all walls. The amount of labour hours is therefore considered to be negligible compared to this traditional formwork method. Also reinforcement can directly be placed on spot. Hence, the proposed method for the construction of overturning caissons is more labour intensive.

7.3. Resource consumption

By application of the prescribed construction method and technologies, it is possible to obtain a workflow which must be repeated every week. The construction of one caisson a week is similar to a rectangular caisson (section 4.1). Due to the reinforcement and formwork prefabrication sites, high labour efficiency can be obtained. The estimated production rates are as follows:

Labour activity	Element type	Labour rate rect. caisson	Labour rate overt. caisson
Reinforcement fixing	all elements	90 kg/h	90 kg/h
Concrete distributing, vibrating, curing	horizontal	2.00 m^3/h	2.00 m^3/h
Concrete distributing, vibrating, curing	vertical	1.00 m^3/h	0.80 m^3/h
Concrete distributing, vibrating, curing	declined	N/A	0.60 m^3/h

Table 7.2. Labour productivity rates for caissons

Normally, reinforcement fixing of walls is more labour intensive than flat slabs[9]. Different working conditions and shapes result in varying productivity ratios in the order of 30 to 150 kg/m^3. The highest productivity rate can thereby be obtained for a low complexity flat plate slab with equal rebar spacing. The shape complexity of the overturning caisson shall thereby result in a reduction of productivity rates. It is therefore estimated that an average labour rate of 90 kg/h shall be achieved at the prefabrication site. This is similar to the labour rate for a gantry slipform technique, where good working conditions are provided. On the other hand, as shown in the table above, shape influences are taken into account for concrete labour working rates.

7.3.1. Overturning caisson labour consumption (12.60m)

The concrete and reinforcement labour works for the overturning caisson are therefore estimated to be:

Activity	Concrete volume	Concrete labour	Reinforce ment	Rebar amount	Labour hours
Front-wall	223 m^3	112 hr	92 kg/m^3	20,500 kg	230 hr
Side-walls and compartment-walls	232 m^3	290 hr	92 kg/m^3	21,300 kg	240 hr
Back-wall	205 m^3	232 hr	92 kg/m^3	18,900 kg	210 hr
Counterforts	106 m^3	133 hr	120 kg/m^3	12,700 kg	140 hr
Base slab	183 m^3	229 hr	120 kg/m^3	22,000 kg	240 hr
Total caisson	949 m^3	996 hr		95,400 kg	1060 hr

Table 7.3. Labour consumption for an overturning caisson (concrete and reinforcement)

These working-hours are directly related to the particular building materials. Due to the presented construction process, these activities can be efficiently executed. However, in addition to these direct activities, reinforcement- and formwork placement must be incorporated. Also the additional labour hours for construction joints must be taken into

[9] Source: The productivity of Steel Reinforcement Placement in Australian Construction (2014)

account. The estimated productivity rates for additional activities of an overturning caisson are:

Labour activity	Labour rates	Amount	Total
Reinforcement placement	8 h / mesh	17	136 hr
Formwork placement	8 h / form	28	224 hr
Formwork demoulding	8 h / form	28	224 hr
Construction joints	40 h / element	5	200 hr
Total labour consumption:			784 hr

Table 7.4. Additional estimated labour hours for an overturning caisson

As presented in the table above, the additional formwork preparation and demoulding working hours are estimated to be 2x8h per formwork-element. With a total of 28 formwork elements, the total labour becomes 16 x 28 = 448 hour. The other activities are considered in a similar manner. It must be noted that these working hours are rather hard to predict. In practice, these working hours could differ drastically. This is therefore a large risk for determining the feasibility of such a construction method. To sum up, the direct labour hours for an overturning caisson are estimated to be:

Total labour consumption overturning caisson	amount
Concrete casting and finishing	996 hr
Reinforcement fixing	1060 hr
Additional labour activities	784 hr
Total labour for one caisson	2840 hr

Table 7.5. Total estimated labour hours for an overturning caisson

The additional labour activities contribute to roughly 25% of the total working hours. These working hours are thereby considered to be the most significant change to the consumption compared to a slipform-constructed caisson.

7.3.2. Rectangular caisson labour consumption (12.60m)

Similar to the previous estimates, the productivity rates shown in table 7.2 are used. The labour consumption for a rectangular caisson is estimated to be:

Activity	Concrete volume	Concrete labour	Reinforce ment	Rebar amount	Labour hours
Walls	952 m^3	952 hr	85 kg/m^3	80,900 kg	899 hr
Base slab	183 m^3	92 hr	120 kg/m^3	22,000 kg	244 hr
Buttress / joints	31 m^3	39 hr	120 kg/m^3	4,000 kg	44 hr
Total caisson	1166 m^3	1083 hr		106,900 kg	1187 hr

Table 7.6. Labour consumption for a rectangular caisson (concrete and reinforcement)

The direct labour hours for a rectangular caisson are thereby estimated to be:

Total labour consumption overturning caisson	amount
Concrete casting and finishing	1083 hr
Reinforcement fixing	1187 hr
Total labour for one caisson	2270 hr

Table 7.7. Total estimated labour hours for a rectangular caisson

Due to the slipforming construction method, the labour hours appear to be approx. 25% lower than the overturning caisson, while more building materials are consumed.

7.4. Conclusions

The shape complexity results in a drastic change of caisson construction method. The applicable construction technologies for an overturning caisson are rather traditional. Investments in just one high quality set of formwork and equipment result in high concrete quality and enables a caisson production rate of 1 each week. This construction speed is similar to regular caissons.

The shape complexity results in more construction tasks. Elements such as the declined back-wall and counterforts, require more labour during construction. Due to the proposed construction method and technologies, no quality and safety setbacks occur. However, in order to obtain the same quality, the concrete mixture must be carefully designed and labour consumption increases.

Five construction activities are planned on 2 different locations (A and B) to obtain an efficient construction process. The total construction takes place on three locations, which is similar to the construction site of large rectangular caissons. The proposed process (table 7.1) is a tight and varying construction schedule compared to a slipform process. As a result, the horizontal construction method is estimated to require more labour. Additionally, casting a floor instead of a wall does not result in less formwork. Due to the possibility of high reuse of formwork systems, casting a horizontal element is not intrinsically beneficial. All in all, it cannot be demonstrated that the construction of an overturning caisson is beneficial compared to a vertically constructed caisson.

8. Economic Feasibility (2017)

8.1. Overview

The overturning design combined with a rubble backfill is considered for an economic feasibility analysis. Although material savings can be obtained for each caisson, other cost contributions, such as labour, influence the marginal costs as well. This results in a dependence on the number of constructed caissons. It is thereby intended to stress the cost differences between concepts rather than an accurate cost estimate for the total construction costs.

8.2. Direct construction costs

Based on historical construction cost data (appendix N), the price of concrete and reinforcement is estimated to be respectively 60 €/m^3 and 0.60 €/kg. The average European price rate for construction labour amounts 40 €/hr[10]. However, African labour rates are generally much lower, and can even decrease to values below 10 €/hr for skilled workers. Since the considered reference project is located in Africa, a collaboration of European and African labourers is assumed. Therefore, an average price rate of 25 €/hr is taken into account for all construction labourers.

Variable direct costs	Quantity	Price per unit	Total cost
Concrete	949 m^3	60 €/m^3	€ 56,900,-
Reinforcement	95,400 kg	0.60 €/kg	€ 57,200,-
Labour	2840 hr	25 €/hr	€ 71,000,-
Variable direct caisson costs:			€ 185,100,-

Table 8.1. Total direct costs for an overturning caisson

As can be seen, the value of construction materials is almost two times higher than the labour costs. It is therefore still significant to invest in material savings. The total variable costs for a rectangular caisson are:

Variable direct costs	Quantity	Price per unit	Total cost
Concrete	1166 m^3	60 €/m^3	€ 69,700,-
Reinforcement	106,900 kg	0.60 €/kg	€ 64,140,-
Labour	2270 hr	25 €/hr	€ 56,750,-
Variable direct caisson costs:			€ 190,600,-

Table 8.2. Total direct costs for a rectangular caisson

Hence, shape and construction differences result in a reduction of marginal construction cost. The construction cost is estimated to be reduced by a small 3%. However, as discussed in section 4.1, learning effects for a repetitive construction process can have a significant impact on the overall costs. Therefore, the reduction can become larger if the labour efficiency turns out to be higher, or if local price rates for labour become lower.

[10]Labour price rate for The Netherlands according to International Construction Cost Survey, Gardiner and Theobald (2011)

8.3. Marginal caisson costs

The total marginal costs of a caisson are not identified since the cost difference is from importance. Cost differences can be found in concrete volumes and labour consumption on the construction site, but also in the following phases:

- Construction → prefabrication of caissons (concrete volume, labour);
- Launching → caissons launched by for instance a FDD;
- Transport → self-floating caissons transported by tugboats;
- Turning → floating crane assistance;
- Compartment fill → caisson compartments filled with sand;
- Backfill material → apply rubble backfill;

As calculated in the previous section, construction costs of the prefabricated caissons are estimated to reduce by 3%. The launching and transport costs are expected to differ insignificantly since the draught and weight of an overturning caisson is just slightly reduced compared to a rectangular caisson (chapter 6). On the other hand, the additional costs of turning the caisson are expected to have a significant effect on the marginal costs. The compartment fill and application of backfill material is expected to be slightly more labour intensive since the caisson geometry reduces accessibility of equipment. Considering these differences, an arbitrary amount of € 5,000,- is added to the cost of each overturning caisson to incorporate increased labour and resource consumption. This makes that the marginal cost difference for both caissons become negligibly small. Economic advantages of constructing overturning caissons can therefore not be demonstrated after a certain number of caissons. Only if temporary equipment costs are lower, the overturning caissons can become economical up to a number of caissons.

8.4. Equipment and depreciation

Since the project duration usually amounts one to two years, used equipment is not necessarily at the end of its technical service life. However, the usefulness of custom designed formwork can for instance be highly reduced when the project is finished. The depreciation for custom designed formwork is therefore considered to be the highest, since its salvage value is estimated to be negligible. Regular equipment, such as lifting equipment, can be useful for other projects or other contractors, which allows higher salvage values. With help of procurement costs provided in appendix N, the following cost estimates (table 8.3) are made for large equipment.

Equipment overturning caisson	quantity	Procurement and mobilization value	depreciation over project	estimated project expenses
Fixed tower cranes (500 t/m)	2	€ 1,500,000.-	15%	€ 225,000.-
Mobile crawler cranes (300T)	1	€ 1,000,000.-	15%	€ 150,000.-
High quality formwork system (3500 m^2)	1	€ 3,500,000.-	90%	€ 3,150,000.-
Temporary foundations prefab yards	45	€ 100,000.-	90%	€ 90,000.-
Transport over land: hydraulic jacks 30x100T	30	€ 1,000,000.-	50%	€ 500,000.-
Launching: 5000T floating dock	1	€ 2,000,000.-	30%	€ 600,000.-
Transport over water: tugboats	2	€ 500,000.-	15%	€ 75,000.-
Floating crane 500T (sheerleg)	1	€ 5,000,000.-	15%	€ 750,000.-
Total expenses:				€ 5,540,000.-

Table 8.3. Estimated expenses on large equipment for an overturning caisson

For a rectangular caisson, the depreciation of a gantry slipform is expected to be drastically lower. The required of formwork area is drastically reduced since the custom designed slipforming shutters may be just 1.00m high. The remaining temporary equipment can be reused for other caisson structures.

Equipment rectangular caisson	quantity	Procurement and mobilization value	depreciation over project	estimated project expenses
Fixed tower cranes (300 t/m)	1	€ 1,500,000.-	15%	€ 75,000.-
Gantry framework incl. slipform (~300m^2)	1	€ 5,000,000.-	50%	€ 2,500,000.-
Transport over land: hydraulic jacks 30x100T	30	€ 1,000,000.-	50%	€ 500,000.-
Launching: 5000T floating dock	1	€ 2,000,000.-	30%	€ 600,000.-
Transport over water: tugboats	2	€ 500,000.-	15%	€ 75,000.-
Total expenses:				€ 3,750,000.-

Table 8.4. Estimated expenses on large equipment for a rectangular caisson

The values presented in the previous tables intend to provide a rough cost estimate of the major differences. The cost and depreciation rates are thereby an indication of temporary equipment cost differences. For the formwork estimate of an overturning caisson, the Port of Botany (appendix A.5) is used as reference project. Here, 2000 tonne temporary steelworks are used for the counterfort wall construction. A patented formwork- tie system was used which was developed during the project. This accentuates the uniqueness of such a project and the scale of temporary works. The value of such a formwork system after finishing the project is therefore assumed to be negligible. Nota bene, the exact same dimensions must be used for the next project for the formwork system to be reusable. For the construction process of a rectangular caisson, a lot of equipment can be reused. It is therefore estimated that the formwork costs are higher for an overturning caisson, while the construction method itself is rather traditional.

8.5. Economic feasibility estimate

As described in the first chapter of the thesis repetition and depreciation are expected to influence the feasibility of the concept. The following scenarios were presented in section 1.5.4:

a. Higher fixed costs and lower variable costs result in an economic design after a certain number of caissons;
b. Lower fixed costs and higher variable costs result in an economic design up to a certain number of caissons;
c. Lower fixed costs and lower variable costs; production costs are always lower.

Considering these three scenarios (a/b/c), the overturning caissons will not be economical after a certain number caissons. The estimated marginal cost difference is too low for an explicit answer to the feasibility after a certain number of caissons. The project expenses on large equipment are estimated to be higher for the overturning caisson. However, this cost aspect is also debatable. Scenario (c) can occur if the salvage value of equipment appears to be lower than expected.

The graph on figure 8.1 shows 2 different construction methods for a normal rectangular caisson and the overturning principle. The cost per caisson is plotted vertically and the number of caissons is shown on the horizontal axis. The cost per caisson is based on values from sections 8.2, 8.3 and 8.4. These cost estimates are not intended to provide total construction costs. The results are focussed on the differences between the concepts. The actual construction costs of a caisson will therefore be higher. It is expected that both lines in the figure would shift vertically, while the point of intersection would remain on the same horizontal position. Hence, the feasibility outcome remains unchanged.

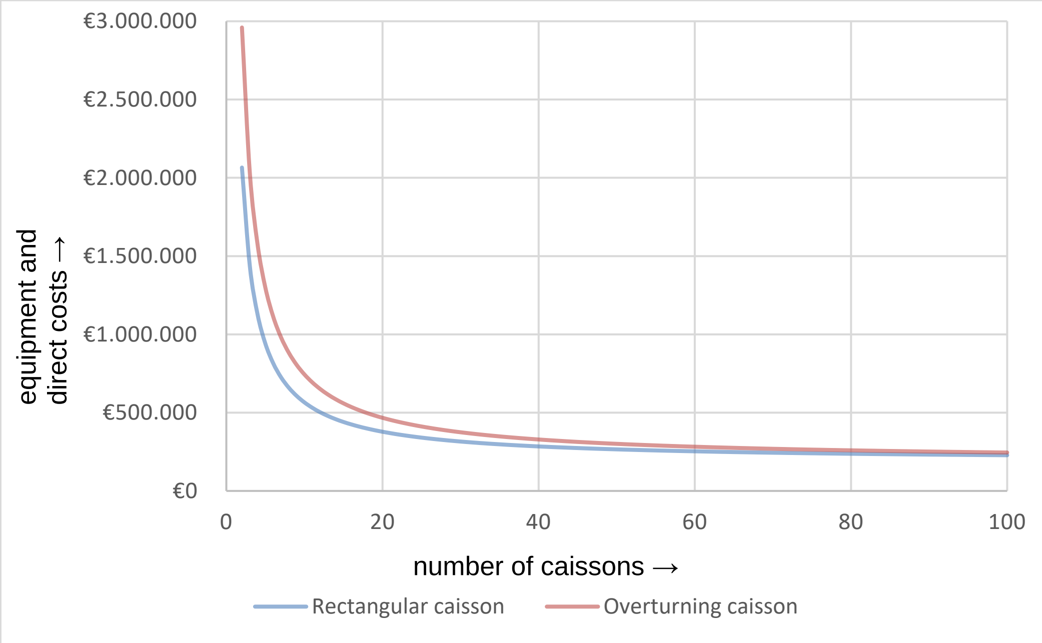


Figure. 8.1. Repetition-feasibility relation of the overturning concept

The graph shows that the costs of a caisson become almost equal after 100 caissons. Theoretically, the price per caisson is still slightly higher than for a rectangular caisson; a feasible region is not found since the direct cost difference is insignificant. However, this is based on many assumptions regarding labour, assembly time, equipment and building material costs. For instance, if the custom designed formwork system can be reused for multiple projects, the economic potential increases. In terms of the reference project, for which approximately 60 caissons are needed, the overturning caissons is not economically feasible.

On top of this, this feasibility analysis is based on the prerequisite that a rubble backfill shall be applied. If this type of backfill is locally not available, or if quay loads are significantly larger, other designs become more favourable.

9. Conclusions

From a technical point of view, the most striking advantages of the caisson geometry are draught and material savings. Compared to a rectangular caisson, the overturning caisson combined with a rubble backfill results in the largest benefits. This is caused by the reduced horizontal quay load and the subsequently increased caisson slenderness. Draught and material use are nowadays significantly higher than for the original concept from 1903. Reasons for these changes in the design are;

- Stability demands → increased width; less caisson slenderness;
- Durability demands → larger concrete cover and wall thickness;

The strengths of the original concept are therefore reduced, but still present. The concept is technically feasible and can be designed according to current standards. Based on current design requirements, the overturning concepts allows 8% to 15% material savings. The largest material savings can be made when a rubble backfill is applied. If quay loads increase, the required caisson width becomes larger which results in a reduction of material savings relative to a rectangular caisson. It is found that rectangular caissons have larger operational stability than overturning caissons with the same width. Therefore, when these structures are compared, it must be noted that additional cost savings can be made by further optimizing material consumption of the considered rectangular caissons.

The L-shaped caissons have a larger metacentric height during its first (horizontal) floating position compared to a rectangular caisson. Due to the increased floating stability, no ballast water is required during transport. As a consequence, the overturning concepts have a lower draught than their rectangular counterparts. The development of caisson designs in relation to draught can be found in table 6.6 to 6.8. It can be seen that technical design changes (e.g. increased width) mitigate draught benefits. The most beneficial overturning caisson concept allows a draught reduction of roughly 25% compared to a ballasted rectangular caisson.

The overturning caisson becomes increasingly feasible when a slender design is considered, while a rectangular caisson becomes increasingly feasible for wider designs. For rectangular caissons, floating stability and draught improve by an increased width, while the conditions for overturning caissons reverse. An underestimation of loads may therefore result in a different optimum design solution. As a result, costs of load reducing measures can influence the degree of feasibility. Since soil actions on quay walls are often dominant (e.g. case study appendix E-H), the costs of a rubble backfill or other soil improvement measures must be considered before evaluating the overturning concept. Instead of the expected increase in design freedom (as in proposed in figure 1.2 and 1.3), the overturning caisson can only efficiently be applied for a certain range of quay loads.

Labour costs are estimated to be higher for an overturning caisson than for a rectangular caisson constructed with the slipforming technique. The construction of an overturning caisson requires larger (and heavier) prefabricated elements. Placement and securing these elements requires additional labour and reduces efficiency.

Equipment costs are estimated to be higher for an overturning caisson. An additional 400 tonne floating sheerleg is required to assist the turning operations. Furthermore, the procurement of job-built formwork is expected to be a cost raising component. The reason for this is that the salvage value is considered to be negligible. However, its actual value depends on the usefulness for other quay wall projects. These components are therefore hard to predict and may change the economic feasibility of the concept. Considering these investments in specialized equipment, a conventional production rate of one caisson each week can be achieved. This is similar to the construction time of a rectangular (slipformed) caisson.

All in all, the economic feasibility cannot be demonstrated by combining the considered aspects of material savings, labour consumption and equipment costs. The economic feasibility is too dependent of various cost parameters and sensitive to changes. When labour and equipment costs are lower, the overturning concept becomes increasingly feasible.

9.1. Answers to research questions

Based on the feasibility study, the following research questions can be answered:

- *Is the caisson shape structurally more efficient?*
  Material savings can be obtained compared to rectangular caissons. The advantages become less significant when loads on the structure increase. For instance, the change of backfill material reduces the amount of material savings from 15% to 8%.

  On the other hand, the caisson shape has lower global stability than a rectangular caisson with an identical width. This is caused by a lower weight, changed centre of gravity and increased soil pressure. Namely, the caisson back-wall is not load reducing (appendix L). On the contrary, the caisson heel results in a “trapped” soil wedge which prevents ground to reach an active pressure state. Destabilizing effects caused by soil are therefore higher for a caisson with heel. However, this effect by itself is not decisive for the feasibility of the concept.

- *Does the concept allow a simplified launching method?*
  The size, weight and draught of the new overturning caisson are similar to rectangular caissons, the launching method considered to be hardly affected. Only when a slender overturning caisson is designed (rubble backfill), the draught decreases significantly. Then, the feasibility of a certain launching system can be influenced. However, as a trade-off, the caisson needs to be turned by a 400 tonne floating crane which requires floating equipment and labour.

- *Do transport conditions become less governing for caisson design?*
  The metacentric height for an overturning caisson in horizontal floating position is sufficient for stable transport without ballast water. This is in contrast to the considered rectangular caissons, which require measures to increase floating stability.

- *Does feasibility depend on the required retaining height?*
  When immersion pressures and quay loads increase, it becomes less economic to design an overturning caisson. Material use and draught benefits become negligible and the turning process becomes more challenging. Increasing slenderness is therefore beneficial, thus lower quay walls with relatively low quay loads are optimal design conditions.

- *Is a horizontal construction method beneficial for caissons?*
  A horizontal construction method is not beneficial for caissons in general. The slipform construction technique allows higher productivity rates and provides convenient concrete quality, inspection and safety.

  The horizontal construction method is however beneficial for the construction of an overturning caisson. The shape results in a drastic change of suitable the construction methods. For this geometry, a horizontal construction is the most economical solution. However, also for the construction of an overturning caisson, no intrinsic benefits are found for this construction method. On the contrary, it is expected that this (formwork) method requires 25% more labour.

- *Does the concept improve safety and environmental aspects?*
  The concept does not improve safety by itself. The average working height is reduced compared to rectangular caissons. Most reinforcement can be cut, bent, and fixed on ground level. However, on the assembly line, working conditions are still hazardous. Sufficient passive fall protective systems must therefore be provided to work in a safe manner. Also, working under hoisted elements must be prevented as much as possible.

  In terms of environmental aspects, a saving can be made since building materials are saved (chapter 6). A reduction of concrete (and cement) consumption lowers the CO_2 production. However, if this were to be the aim, other solutions can be exploited, such as changing the cement type or mixture.

9.2. Recommendations

If the overturning concept would be applied for a future quay wall project, a number of aspects need to be verified or improved before the economic benefits can be quantified. These aspects are;

- Turning and immersion modelling (dynamically);
- Plate analysis side walls;
- Risk analysis;
- Accurate cost estimate;

Dynamic modelling can be performed in order to evaluate the extended sway while the caissons are turned without crane assistance. Also stresses in the caisson can then be evaluated more accurately. An advantage of such a model is that it can reveal whether sufficient fluid dynamic drag occurs to safely turn the caissons.

The side-walls must be considered by a more accurate calculation method. The use of FEM software could be applied to determine the level of safety more precisely.

A risk analysis should be performed in order to evaluate possible economic setbacks. Deviations in labour speed and the construction pace could result in a less economical outcome. Also material aspects are involved, such as risks of cracking due to drying shrinkage. Depending on the chosen construction method and process, a certain (hardening) time difference occurs during the walls and slabs. This can potentially result in concrete cracking. However, if such aspects are incorporated during the design and work-plan, it is not necessarily problematic.

Additionally, a risk assessment for the safety of personnel should be done since the construction method differs from rectangular caisson quay walls. Nevertheless, the construction safety for employees is not expected to differ drastically from other large concrete structures.

If these aspects are all incorporated, a cost estimate can be made with a higher accuracy. After this, a certain band can be considered for which the probability of cost exceedance remains acceptable.

Appendix

A. Existing Concepts

There is a great variety of caisson concepts which all seemed economically feasible at a certain place and time. Locational and technological aspects are majorly contributing to the degree of feasibility of a concept.

The purpose of this section is to recognise advantages and disadvantages of different quay structures. The objective is to find the origin of the particular benefits and apply these (if possible) for a new design. The investigation is focussed on prefabricated quay structures, with the particular overturning caissons as priority. The following five concepts are therefore analysed:

1. Rectangular caissons;
2. Circular caissons;
3. Hybrid floated-in-caissons;
4. Overturning caissons;
5. Prefabricated non-floatable elements;

The first three listed types are categorized by their shape. The fourth concept, the overturning caisson, can basically be designed in any shape, but turning the caisson during its floating phase will not be practical for all shapes.

A.1. Rectangular caissons

Rectangular caissons form the basis for the shape analysis, they usually have a similar cross section as the caisson shown in figure A.1. The caissons are constructed in vertical position. The construction can take place onshore, in a floating or non-floating dry dock. The rectangular shape simplifies the design, execution and transport. This can result in time-savings and it reduces the sensitivity for errors during execution compared to complex caisson shapes.

The main actions are transferred perpendicularly to the structural elements. In contrast to circular and arch shaped structures, this introduces relatively high shear forces and bending moments in the cross sections. The caissons are generally designed with multiple internal walls. Design adjustments are usually required for the transient construction phases. For instance; additions and changes for the floating and immersion phase can be seen as inefficient, since it is not contributing to the desired operational performance. In general, rectangular caissons have the following disadvantages:

- Loads are mainly transferred by shear and bending;
- Compartments may be added to increase floating stability;
- Shape changes to increase buoyancy.

The pictures (A.1 and A.2.) show concrete caisson construction for the Port of Piraeus (Athens, Greece). For this project, 40 caissons were constructed using the slipforming technique. Each caisson was built on a semi-submersible barge or floating dry dock (FDD). When the construction progressed sufficiently, the barge was lowered and the caissons where finished while floating. After finishing, the elements were floated to their final locations.

Figure A.1. Rectangular caissons, Port of Piraeus, Greece (2016) [11]

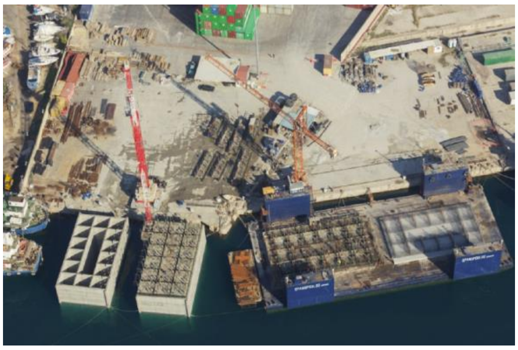


Figure A.2. Construction of rectangular caissons using the slipform technique and a semi-submersible dock, Port of Piraeus, Greece (2016)

The construction efficiency and man-hour consumption for this construction method has been analysed by Panas and Pantouvakis (2013). The overall caisson characteristics are depicted in the table below:

Caisson specifications (Port of Piraeus, 2016)	
Number of caissons	40
Quay length	1,000m
Caisson height	19.50m
Caisson width	13.10m
Caisson length	24.80m
Draught (estimate)	10.00m

Table A.1. Specifications rectangular caissons

[11] Source: *www.slipform.us/piraeus-west-pier-3-caisson-slipforming/*

A.2. Circular caissons

Circular caissons have many similarities to traditional rectangular caissons. In terms of construction methods; the efficient slipforming technique can be used as for rectangular caissons. Often, they have a symmetrical circular cross section, which induce hoop forces in the walls for equally distributed pressures around the structure. Nevertheless, large bending moments and shear forces occur near the wall-to-base connections of the caissons.

Circumferential stresses can result in a significant reduction of wall thickness and a reduction of inner walls. This implies that less material use can be achieved. Since the circular shaped caissons are usually designed with a small number of compartments, high shear forces occur in the bottom slab during transport and immersion. These forces must be transferred by thicker concrete slabs and / or increased reinforcement volumes.

Theoretically, no bending moments occur in the walls when loaded by hydrostatic pressures. Significant material savings can be made, especially for cases where pressure from outside the cylinder is larger than from the inside. Axial forces during transport and immersion can be calculated with the kettle formula (ketelformule). This model is based on the assumption of two half circles which are in equilibrium with the acting forces, shown in figure A.3. Forces would be transferred by a compressive component, which is a particular advantage for concrete structures, since it is characterized by relatively high compressive strengths.

On the other hand, if pressure prevails from inside the structure, tensile forces would be introduced. This would mitigate the advantage of the circular shape of a concrete structure. The schematized pressure distribution would be a possible load case for quay walls, since it is probable that they are filled with completely saturated soil, where the hydraulic pressure from outside is generally lower.

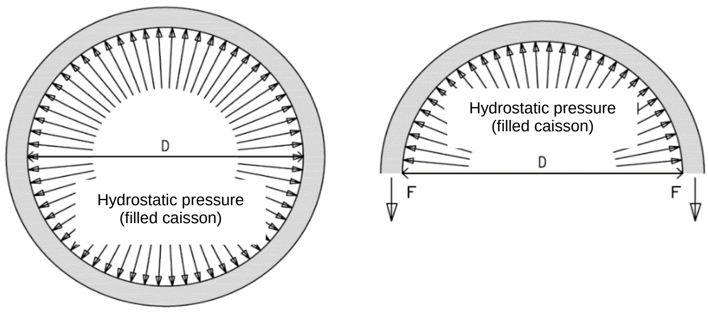


Figure A.3. Load distribution principle of a cylindrical shaped caisson

The joints between circular caissons must be designed properly, since the structures have no large connection surface by themselves. The caissons constructed for the Europaterminal in Antwerp (1990) showed the importance of detailing a proper connection between the circular elements.

Durban (2004)

The construction rate for the caissons in Durban was approximately 4 metres a day per caisson. The application of multiple slipforms resulted in a production speed of approximately 3.5 caissons per week. There was roughly 75,000 m^3 of concrete and 9,000 tonnes of reinforcement required for the 1,200 metre long quay. The caisson yard included two batching plants and a caisson-lift for launching. The concrete operations were carried out continuously over 24 hours a day. General characteristics of the project are presented in table A.2.

Figure A.4. Bi-circular caisson construction for the Port of Durban (South-Africa, 2004)

Figure A.5. Traditional slipforming used for one of the caissons

The caissons for the port of Durban were designed with 2 circular compartments. The radius for the circular sections amounted approximately 7 metres. This resulted in a relatively large load on the base slab during transport and a high slab thickness of 1.50 metres. The concrete volume of the base-slab was thereby almost equal to the volume of the entire walls. Furthermore, a temporary longitudinal compartment wall was added to increase floating stability during immersion[12]

Caisson specifications (Port of Durban, 2004)		
Number of caissons		52
Quay length		1,200m
Caisson height		18.00m
Caisson width		17.00m
Caisson length		24.00m
Draught (approx.)		10.00m
Building materials		
Concrete	per caisson: running metre quay:	1,400 m^3 60 m^3 / m^1
Reinforcement	per caisson: steel / concrete: running metre quay:	168,000 kg 120 kg / m^3 7,200 kg / m^1

Table A.2. Specification circular caissons[13]

[12] Marine Concrete Structures – Design, Durability and Performance(2016)
[13] Source: http://www.maritimejournal.com/news101/marine-civils/port,-harbour-and-marine-construction/first_caissons_floated_out_for_durban_quay_wall_project#sthash.fqehkZou.dpuf

A.3. Hybrid floated-in-caissons

L-shaped quay walls, generally referred to as buttress, cantilever or counterfort walls, are normally non-floatable structures and transported by large vessels or constructed on site. The Kraus caisson can be seen as hybrid form which smartly takes advantage of the economic L-shape for its load transfer and is therefore also shortly discussed.

The application of prefabricated L-shaped wall units is normally limited by the capacity of the available lifting equipment. And in particular, limited by the capacity of floating cranes. The hybrid L-shaped overturning caisson tackles this problem, but requires design adjustments to obtain sufficient buoyancy for floatation.

Hennebique caisson (1900)
Hennebique designed a caisson for a Dutch harbour with a relatively large buttress (front toe) and short heel. The total retaining height of the quay structure amounted approximately 14.00 metre (measured from drawing) and the width equaled 9.75 metre. Besides the relatively large toe, a remarkable aspect is that the quay wall lacks a superstructure.

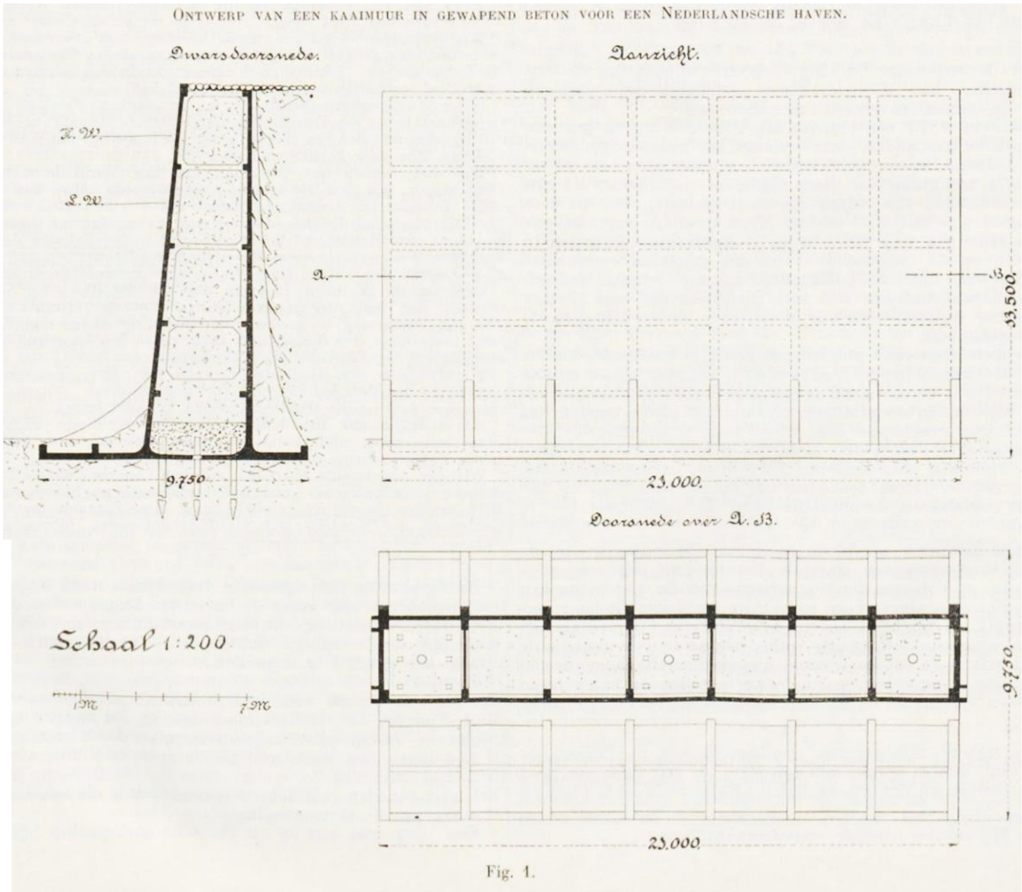


Figure A.6. Hennebique caisson for a Dutch harbour (1900)

According to the date on the drawings, Hennebique made the first design of a reinforced concrete caisson in June 28 in 1900[14]. This is a few years before publication of the Kraus report in 1903. The drawings presented in fig. A.6 are the only evidence found during this historical analysis. The degree in which the design is elaborated remains unclear. It is also found that this particular design is never constructed[14]. There are no objections given in the references, but it is imaginable that transport conditions were not fully considered for this (preliminary) design. However, it is probably the first design of a reinforced concrete caisson quay wall ever made.

[14]De Ingenieur 1902, No 27, page 460 and Technische Lessen en Vraagstukken op het gebied van den Inschischen Havenbouw, page 24

Camilla caisson (1970)
The Camilla caisson shown in figure A.7 was designed and patented in the 1970s by Ballast Nedam Group. It shows great similarities with the first design of professor Kraus. The sloping back-wall and voids in the base plate were designed to reduce the horizontal soil pressure acting on the caisson. This should result in a higher safety factor against sliding and might reduce the required concrete volume.

In the master thesis on this particular caisson, written by F.N. Endtz (1986), retaining heights were studied of 10, 15 and 30 metre. Due to a reduced horizontal thrust and an economic shape, material savings could be realised from 7% up to 30%. The report revealed that the concept could be feasible for larger quay heights (30m), but that its significance depends on the governing failure mechanism. The magnitude of savings was largely depending on a required longitudinal separation wall. However these beneficial characteristics, no applications of this concept have been found.

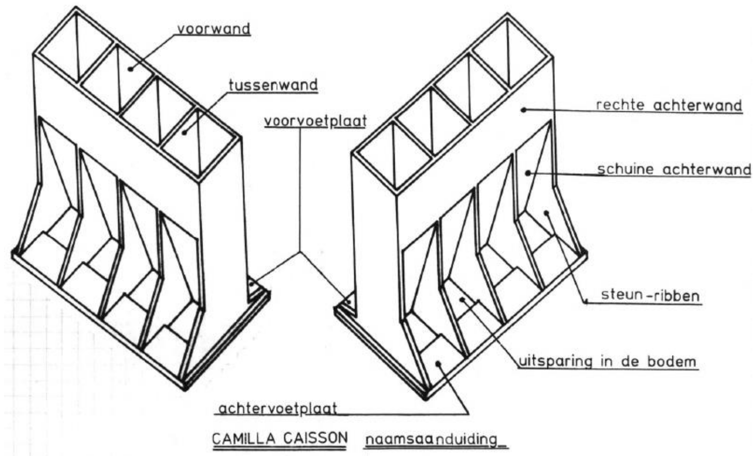
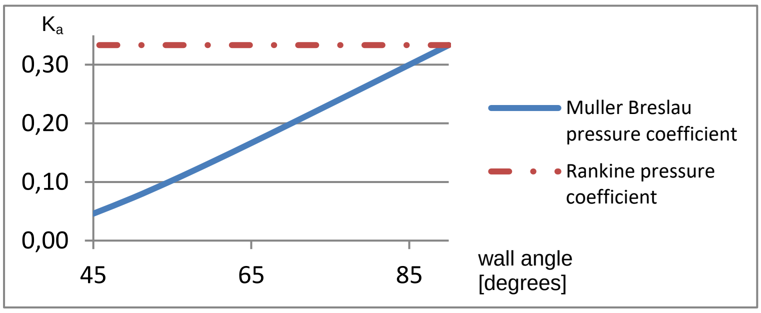


Figure A.7. Drawing of the Camilla caisson (Master thesis, F.N. Endtz (1986))

The declination of the back-wall could reduce the horizontal soil thrust in operational phase considerably. The figure below shows the active soil coefficient relative to the angle of the retaining wall according to Coulombs wedge theory. For this example, an angle of internal friction of $\phi' = 32°$ and $\delta = 2/3\ \phi'$ is used. The graph shows that an angle of 70 degrees reduces the active horizontal soil pressure almost by a factor 2.



This advantage of the declined back-wall can only be used when openings in the bottom plate are present. The original concept of professor Kraus (described in the next section) lacks these openings and would therefore not allow a full active soil pressure state with wall friction in the heel. During failure of the wall, soil within the heel is trapped. The trapped wedge would remain more or less in a neutral pressure state. However the great geometrical similarities to the original overturning caissons, the concept allows great advantages by the openings in the bottom plate. On the other hand, temporary adjustments must be made to allow horizontal transport.

Tsinker (1994)
The use of prefabricated L-shaped walls is limited by the capacity of lifting equipment and floating cranes. On the other hand, cast in situ concrete L-shaped elements involve high construction costs which make both solutions unfeasible in many cases. This is unfortunate, since the L-shape is efficient in terms of material use. In order to cope with this problem, Tsinker (1958, 1994) proposed segmented walls which must be assembled on site (figure A.8). The floating base (bottom slab) and a vertical buoyant component (retaining wall) are connected by several hinges. The compartments are filled at its final position with grout or sand.

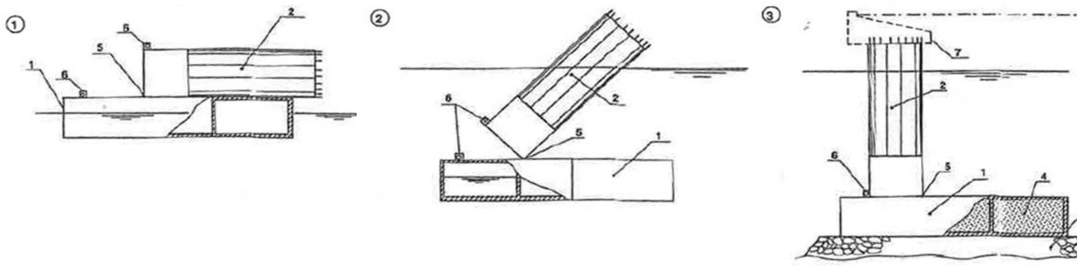


Figure A.8 - Installation phases of an segmented L-shaped caisson (after Tsinker (1958/1994))

A.4. Overturning caissons

The overturning caisson is considered to be a hybrid L-shaped caisson. The most remarkable aspect of this concept is the horizontal construction- and transportation method. This design is referred to as overturning caisson and evaluated in the next sections.

Valparaíso (1903)
Dr. Ir. A.C.C.G. van Hemert, founder of the Hollandsche Beton Maatschappij (HBM) and Dr. Ir. J. Kraus made a sophisticated caisson quay wall design for the port of Valparaíso in Chile. A drawing from the original design is shown in figure A.9. This project has never been executed due to the consequences of an earthquake, but the finished design was preserved. The engineering work was therefore not useless. It formed the basis for future projects at the HBM (later HBG) such as the Talcahuano, Surabaya and Tandjong Priok port expansion projects.

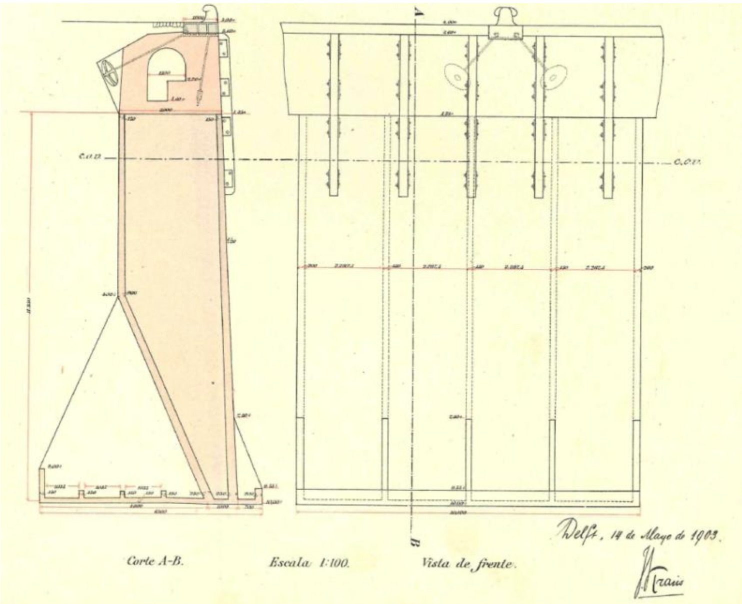


Figure A.9. Drawing of the first caissons for the port of Valparaíso (*Comision Kraus*, 1903)

The Valparaíso design is characterized by the declined back-wall and relatively large base plate. Because of this geometry, the caissons float vertically after ballasting with water. This makes placement and immersing less complex. Although the caissons float almost entirely in vertical position after partial ballasting, a floating crane was used for the final placement.

All the caisson walls were tapered designed. The outer walls started at a thickness of 250mm at the bottom and attenuated to150mm at the top. This saves approximately 20% concrete per wall and is especially advantageous for the overturning principle, since a lower weight at the floating stage simplifies immersion and increases the achievable freeboard after turning.

Remarkable is the fact that the caisson is not a rectangular box, which is currently usual, but more L-shaped. This makes material use more efficient and execution simplified due to the more horizontal nature of the structure.

Talcahuano (1908)
The quay design for the port of Valparaíso was used for the naval port of Talcahuano, which was finished with delay, probably in 1908. The delay was caused by an underestimation of the execution complexity by the Chilean contractor, who had no experience with building reinforced concrete structures. Several photos are found (A.10 - A.12) which were taken during construction.

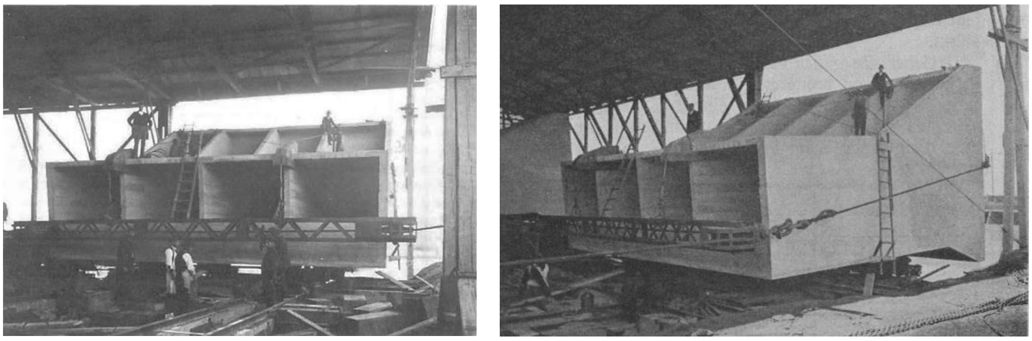


Figure A.10. Caisson construction for Talcahuano, Chile (1908). Source: Wonderen der techniek: Nederlandse ingenieurs en hun kunstwerken : 200 jaar civiele techniek (Dutch Edition), Walburg Pers, Zutphen, 1994

Figure A.11. Transport of the caissons (Talcahuano, Chile (1908))

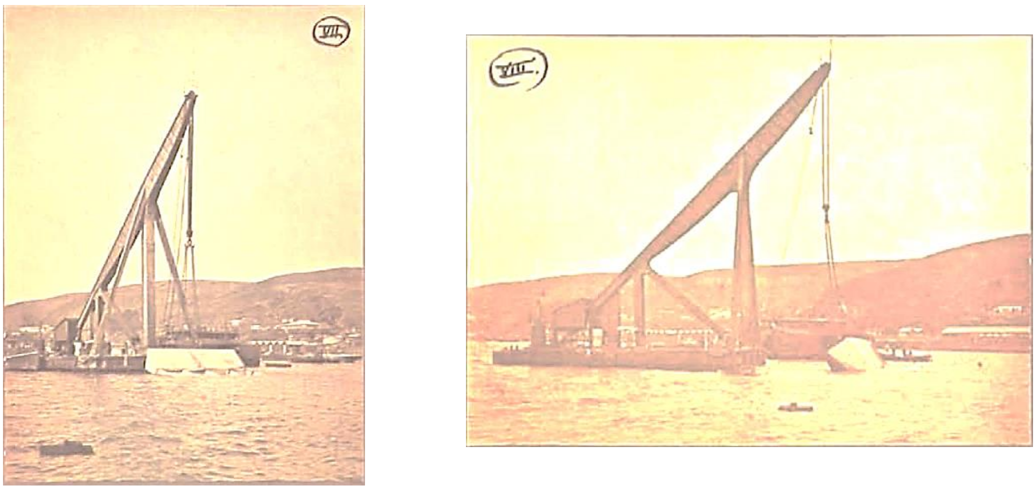


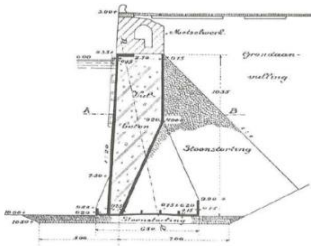
Figure A.12. Turning and immersion of the caissons (Talcahuano, Chile (1908))

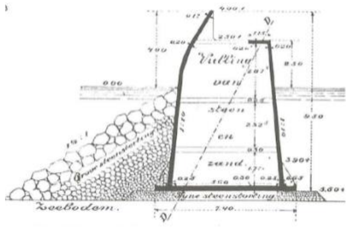
Three different types of (overturning) caisson structures where constructed for this project, namely:

> Figure A.13 - Quay wall (caisson height: 11.35m, superstructure: 2.65m);
> Figure A.14 - Breakwater (caisson height: 9.50m);
> Figure A.15 - Fence wall (caisson height: 11.65m);

The quay wall structure was identical to the original design for Valparaíso in 1903. The breakwater and fence wall were also caisson structures which used the overturning principle. Remark: the fence wall caissons where placed on relatively soft soil. Therefore, there was chosen to fill only the lower part of the caisson (3.00/11.65m) by sand and stones. The rest of the volume filled with water. All caissons were built in dry conditions onshore. There was a temporary structure for launching the concrete boxes.

After transportation to the final location, the caissons were immersed by filling the compartments with water. The structures were placed on the prepared bed and filled with so called “weak concrete” or rubble stones and sand.





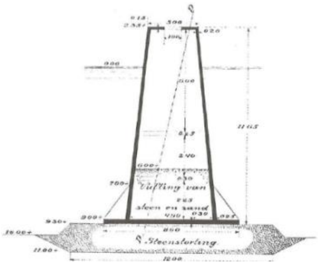


Figure A.13. Quay wall

Figure A.14. Breakwater

Figure A.15. Fence wall

Surabaya (1911)

On the photo (fig. A.16), two almost completed overturning caissons (left) and two caissons under construction (right) are shown. Some notable changes have been made compared to the original Kraus design. Here, the length of a caisson is approximately doubled. The picture clearly shows 10 compartments for each caisson. If the same compartment width remained, the total caisson length amounts (2.50 x 10.00) 25 metre.

Presumably, the enlargement of caissons was previously unfeasible due to the construction site on ground level, which requires heavy transport and launching equipment. The caissons in Surabaya where constructed in dry-docks, which simplified launching.

From the cross-section and the photo below, it can be seen that the outer counterforts decline directly from the top of the caisson. This change is presumably to improve connection between the elements. Furthermore, some minor geometry changes have been made, which increased the slenderness. The bottom slab reduced for instance from 6.50m at Chile to 5.70m at Surabaya. The height increased to 13.00 metre (from the cross-section depicted at fig. A.16).

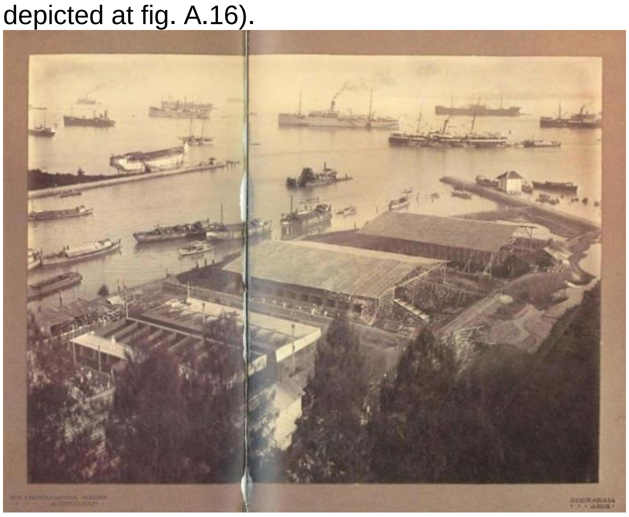


Figure A.16. Left: a photo of the construction side at Surabaya (1911) Right: a typical cross-section of the overturning caisson

A part of rectangular caissons, which were also part of the project, slided over 20 metres when placing the backfill during quay construction. This was caused by a weak mud layer which was present over the full quay width. This project was therefore a reminder of the importance of a proper soil investigation and lead to more conservative design approaches later on.

Tandjong Priok (1914)
The quay wall at the harbour of Tandjong Priok was constructed with four overturning caissons of 31 metre each. This was probably even larger than the ones built in Surabaya. The combined quay length amounts 124 metre. Compared to the Surabaya project, the width of the bottom slab increased to 7.90 metre (+2.20) metre and the height reduced to 11.30 metre (-1.70). The width/height ratio thereby increased to 0.70. Presumably, the increased robustness resulted from lessons learnt at the port of Surabaya.

Two caissons were assembled each time. Execution took place at a very simple construction pit; this was possible because of the relatively low construction height of the elements. The construction pit had to be dredged before the caissons could be floated out. Baseplates and counterforts were casted in the floating stage. A remarkable change to the shape is that the superstructure almost entirely disappeared. At the former projects it was significantly cheaper to build a different structure on top of the caisson at the final stage, but this changed apparently. The first quay wall was finished in 1914.

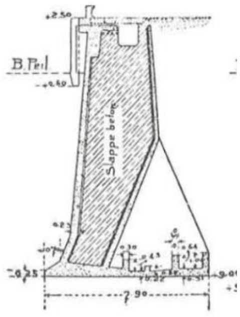


Figure A.17. Caisson for the 1st Binnenhaven Oosterboord (1914)

The following Dutch sentences from the “Technische lessen en vraagstukken op het gebied van den Indischen havenbouw”, written by Ir. Wouter Cool, in Weltevreden (Indonesia,1918) reveal an answer to the question why the concept is abandoned:

> *“Op grond van de te Tandjong-Priok verworven ervaring werd door den Directeur dier haven in samenwerking met de Hollandsche Beton Maatschappij in 1915 voor de caissons het z.g. „Prioktype" ontworpen. Den wanden, die nu vertikaal werden opgetrokken, schonk men meerdere dikte en over de geheele hoogte een constante wapening, terwijl de vulling der vakken uitsluitend met zand geschiedde. Solieder werk, eenvoudiger formeelen, gemakkelijker transport en desondanks gelijke kosten, waren vergeleken bij de vroegere typen de voordeelen.”*

Summarized in English; the text explains why the contractor (HBG) decided to change the overturning principle to the “Prioktype”. It was basically due to easier formwork and transport at equal costs compared to the earlier type. This indicates a well-thought decision for changing the concept. Furthermore, the document reveals that horizontal displacements occurred up to 0.75 metre at the Priok harbour. Disregarding the changed construction method, it was decided that the main dimensions of caissons must be changed and that the calculation approach must be standardized. Reports from former caisson stability calculations lacked proof of horizontal sliding verifications. Nevertheless, new calculation and construction techniques can change the economic feasibility of the overturning caisson.

Gdynia (1927)
This symmetrical caisson (shown in fig. A.18) was built at the Port of Gdynia in Poland. The length of each caisson was 18.15 metre. The height of a caisson was equal to 10.50 metre and its total width was 7.45 metre. The total retaining height was 12.00 metre. The width to height ratio amounted 0.62, which is still relatively slender.

It contained four internal walls which divided the caisson into five compartments. This makes the compartments roughly 3.60 metre wide. Voids where left over at the inner walls in order to reduce the self-weight and improve its floating stability. The caissons where filled and backfilled with sand.

Tunis (pre 1967)
The overturning principle was applied for a quay wall structure at the port of Tunis (La Goulette). The La Goulette caisson is transported in a similar manner as the design of professor Kraus. The exact construction and launching method is unknown due to the scarce available literature regarding this project. The caisson is characterized by its counterforts and inclined back-wall. The front of the caisson was designed with openings.

The caissons were constructed on a slope with the open front on top. Therefore, the closed back-walls could be casted in a relatively simple manner. The La Goulette caisson was completed before launching. From figure A.19, it can be seen that that the caisson was divided into five compartments by four internal walls.

The direction of rotation was different than for the first caissons, since the caisson floated with the front-wall facing upward. The caisson was ballasted by pumping water into the closed bottom compartment. The caisson was partially turned by the added water. The caisson is positioned against a sill structure on the bottom.

Remarkable is that the lower corner of the caisson was placed against a bottom sill. Attention is required to prevent damaging the caisson, sea bed and sill. The procedure is therefore less elegant as the Kraus concept, but its feasibility is not necessarily depreciated.

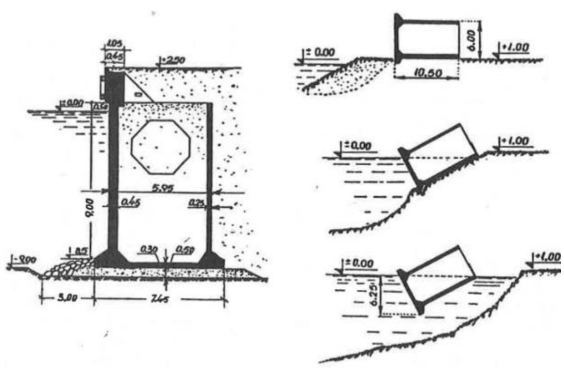


Figure A.18. Gdynia caisson (1927)

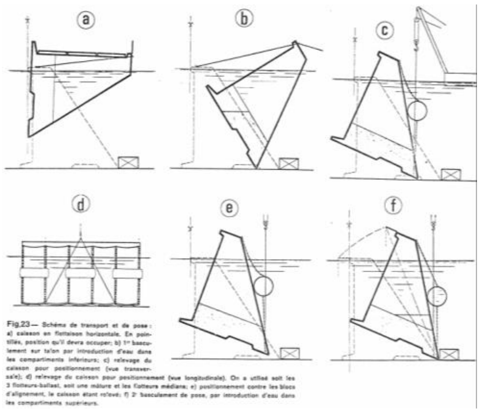


Figure A.19. La Goulette caisson (≤1697)

A.5. Prefabricated non-floatable elements

This type of gravity walls comprises L-shaped concepts with or without counterforts. If the elements are prefabricated on shore, the element length is determined to meet on and offshore lifting requirements. Due to this involvement, the elements length is generally less then floated-in-caissons. Therefore, the total quay wall has a larger number of joints between the elements. Besides, the available contact surface is smaller due to the element geometry. Extra attention is therefore needed to assure a proper and long lasting element connection. Furthermore, since the number of elements is larger and placement must be done by floating lifting equipment, the installation time increases.

The width of cantilever and counterfort walls can be expressed as a function of the total retaining height (H) and generally varies between 0.70 to 0.75H for counterfort and cantilever walls (Tsinker (1997), Smith (2014). While walls made out of prefabricated elements have dimensions varying between 0.75 to 0.85H for internal anchorage and 0.40 to 0.45H for external anchorage. With "internal anchorage" is meant that the wall is connected to the base with a tensile member, where the external anchorage can be similar to a tensile member with anchor plate, which is located behind the soil failure planes of the cantilever wall.

Kitakyushu (1998)
The hybrid L-shaped caisson shown in figure A.20 and A.21 is outfitted with a composite (steel-concrete) slab on the front wall and a steel strut on the back wall. The quay wall is prefabricated at a specialized facility where heavy gantry cranes are available for transport into water.

JFE Engineering Corporation claims that experiments and analyses prove that hybrid L-shaped caissons have equivalent earthquake resistance compared to other gravity type structures, such as caissons. The caissons are relatively low in unit weight and can be applied in deep water. The shown caisson has a height of 17.1 metre, a length of 35 metre, a width of 10.8 metre. This results width-to-height ratio of 0.65, which is rather slender. The total self-weight amounts 1670 ton, from which the amount of steel equals 170 tons. Furthermore, 600m^3 concrete is applied, which equals a volume of approximately 17 m^3 per running metre. This is achieved with a relatively high steel-concrete ratio of 283kg/m^3. The rectangular voids at the front wall are designed to reduce the wave kinetic energy.

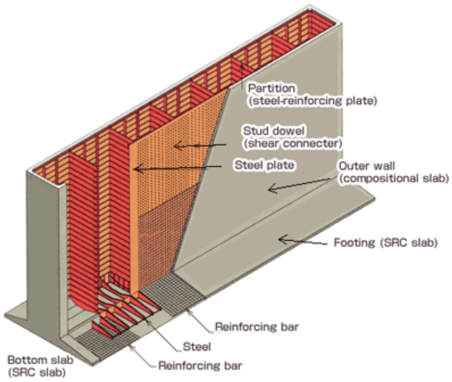


Figure A.20. Kitakyushu composite caisson[15]

Figure A.21 Kitakyushu composite caisson

[15] http://www.nssmc.com/en/product/process/HBC.html

Port of Busan (2008)
A total of 62 caissons have been constructed for a quay wall at the port of Busan (Korea). The caissons are equipped with post-tension steel bars for the lifting the caissons. The weight of the depicted caissons amounts approximately 2,500 tonnes and brought to their final position by a floating crane. The caisson compartments have not been designed to increase their buoyancy, but only to be filled with gravel and sand after placement.

Figure A.22. Prestressed caissons lifted by a 4,000 tonne crane (Port of Busan, Korea, 2008)[16]

Vestebase Kai (2010)
The Vestebase Kai (Norway) has been constructed using ten large concrete caissons. They were designed without buoyancy restrictions since the caissons were lifted into place by an 800 tonnes floating crane. The weight of each caisson amounts approximately 550 tonnes. The relatively small quay length resulted in this unique construction method and caisson shape.

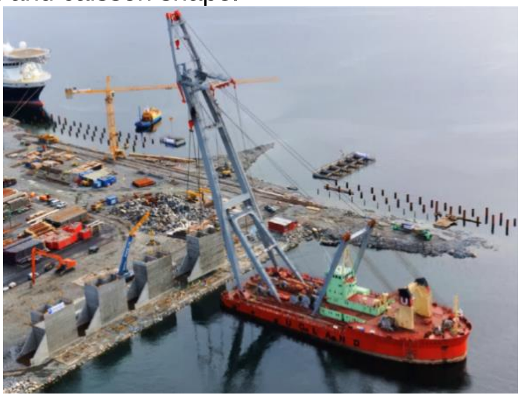


Figure A.23. Vestebase Kai, caissons lifted by a 800 tonnes floating crane (2010), Ugland Marine Services AS, HLV UGLEN [17]

[16] http://www.samhoind.co.kr/catalogue.pdf
[17] http://www.birkenco.no/vestbase-kai-4-5/

Port of Botany, Sydney (1981, 2010)
In 1981, prefabricated counterfort walls have been constructed for a port expansion project in Sydney. These elements were designed with a single counterfort and were 18.65 metre high. The elements weighted 360 tonne to meet requirements of available on- and off-shore lifting capacities. A more recent project (2010) resulted in a design with an almost doubled element weight, which shows that the economies of heavy lifting capacities has shifted over the last decades.

For the most recent project, 200 precasted counterfort units were placed to form the 1,850 m long quay wall. The counterfort units are precast L-shaped unit with a length of 9 m, a height of 20 m and a base length of 15 m, with two triangular counterforts.

Figure A.24. The 460 tonne ringer crane and counterfort elements

Figure A.25. The 700 tonne sheer leg barge lifting a 650 tonne counterfort element

The counterfort units were constructed in three sections, each of roughly equal weight ($80m^3$ / 2,000kN). The formwork systems and lifting equipment required more than 2,000 tonne steelwork. The temporary moulds totalled 5 assembly beds, 4 base-forms and 4 wall-forms. The weight of formwork varied considerably. The "top shed" weight amounted 102 tonne, while the outer form pivots and buttress access towers had a weight of respectively 54 and 35 tonne. Reinforcement was mostly prefabricated, resulting in reinforcement cages of roughly 30 tonne. By using heavy lifting equipment, the casting sequence was planned to be as follows:

1. The base element is casted horizontally;
2. The front wall element is casted horizontally;
3. Wall element is lifted to vertical and placed onto the base element;
4. Two counterforts are casted and a connection between the front-wall land base is formed;

A 460 tonne ringer crane was used to lift the individual sections into place. Due to the application of a large (fixed) ringer crane, multiple construction activities were located around this crane. Three different rings were distinguished, namely:

1. The inner ring → formwork and casting concrete;
2. The median ring → reinforcement cage prefabrication;
3. The outer ring → storage and material handling;

Once a unit was finished, it was transported by a 700 tonne SPMT (Self-Propelled-Modular-Transporter) to a temporary storage facility. Here, the element was cured for a minimum of 28 days. After curing, the element is lifted by a 700 tonne shear leg barge to its final location and lowered into position.

At its peak production level, the precast operation utilised 220 employees, fixing 50 tonne reinforcement, pouring 300 m^3 concrete and lifting 2,000 tonne per day. This resulted in a production rate of seven 640 tonne units per six day cycle. By producing 7 elements of 9 metres a week, over 60 metres of quay wall could be constructed each week.

The concrete mix design had a total binder content of approximately 500 kg/m^3, composed of 50% Portland cement, 25% GGBF slag and 25% fly-ash. A minimum cover of 58 mm was required in the splash zone. A nominal concrete cover of 70 mm was required for the counterforts. Overview of Port of Botany project data:

Counterfort specifications (Port of Botany, 2010)		
Number of elements:		200
Quay length:		1855 metre
Design life:		100 years
Element properties		
Height:		20.00m
Width:		15.00m → 12.00m + 3.00m (diagonal)
Length:		9.00m
Weight:		640 tonnes
Concrete	per element: running metre:	245 m^3 27 m^3/m^1
Reinforcement	per element: steel / concrete: running metre:	50,000 kg 200 kg/m^3 5,400 kg /m^1
Concrete cover to reinforcement		c_{min} = 58 mm c_{nom} = 70 mm

Table A.3. Specifications counterfort walls[18]

Compared to the previously addressed circular caissons for the Port of Durban, the use of concrete reduced over 50%, while the amount of reinforcement reduced by 25% per running metre quay. This, while the counterfort walls are slightly higher.

On the other hand, the reinforcement density per cubic metre concrete increased by 66%. This can lead to execution problems when a low concrete slump mixture is used. Only by paying more attention to detailing and the mix design (which has also been done in practice), good performance can be achieved. These aspects are not free of charge and are expected to have reduced the benefits of material savings to some degree. Furthermore, it has to be addressed that a comparison between these different projects is not entirely fair since design requirements differ.

A.6. Overview of concepts

From the previous analysis and the consideration of alternative concepts, various caisson concepts can be distinguished. The most promising concepts are schematically presented in table 4.1 below. Material saving is the major aspect on which the concepts are evaluated. This is because of its high influence on the direct construction costs. Furthermore, when material savings form the starting point of design, the horizontal execution process will consequently follow from this demand.

Note that an optimization in the vertical plane of the structure is sought. Optimizations in the horizontal plane (e.g. by application of circular sections) are not considered. It is already demonstrated that material savings can be obtained by applying circular sections, but there are also various disadvantages which make these concepts unfeasible or undesired. Therefore an optimization in vertical cross-section is sought, which will hopefully reduce the overall lifecycle costs.

[18] http://www.insideconstruction.com.au/site/news/1018684/port-botanys-precast-production-line

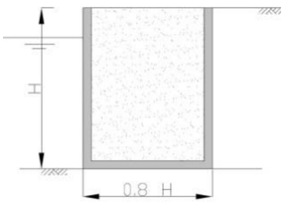
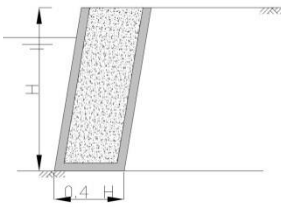
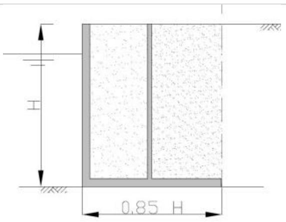
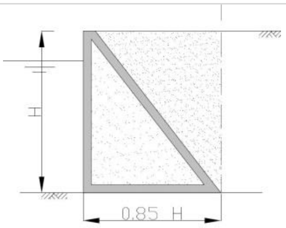
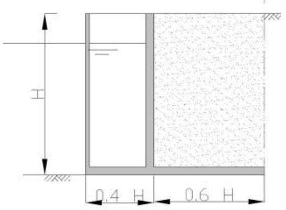
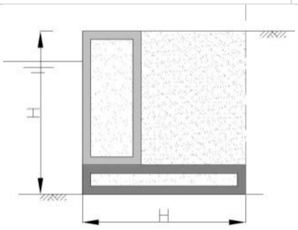
vertical cross-section	**caisson type**	**relative material use (estimated)**	**typical advantage**	**typical disadvantage**
	*Rectangular*	*100%*	*Neutral*	*Neutral*
	U-shaped	50% (excl. backfill)	Lowest amount of reinforced concrete and labour Structural coherence[19]	High amount of cementitious fill required and/or permeable backfill
	L-shaped *(Kraus)*	70%	One primary retaining wall and L-shape result in high material savings	No full soil mobilization on short heel → less stability Equal pressure on backwall must be provided (e.g. by openings)
	Triangular *(La Goulette)*	75%	High torsional stiffness for transport requirements	Soil mobilization on heel depends on friction coefficient → less stability
	T-shaped	80%	Full soil mobilization on heel → similar stability Evenly distributed soil pressure	Larger heel width and less material savings
	Segmented (Tsinker)	80%	No large adjustments for buoyancy required	Complex connection required between elements → increasing risks

Table A.4. Comparison of caisson concepts with reference to material use

[19] Due to the bonded compartment fill which connects the front- and back-wall

B. Analysis: The overturning caisson (1903)

B.1. Calculation approach

The original calculation approach, used in 1903, is unknown. In order to verify if the caisson can currently be applied for a civil engineering work, new calculations are made on basis of the British Standard (BS-6349) and Eurocodes. The chart below shows how the input parameters from the original report are used for stability verification. The calculations are performed in simplified form and based on just one load combination. The calculation results can therefore only be used as indicative values.

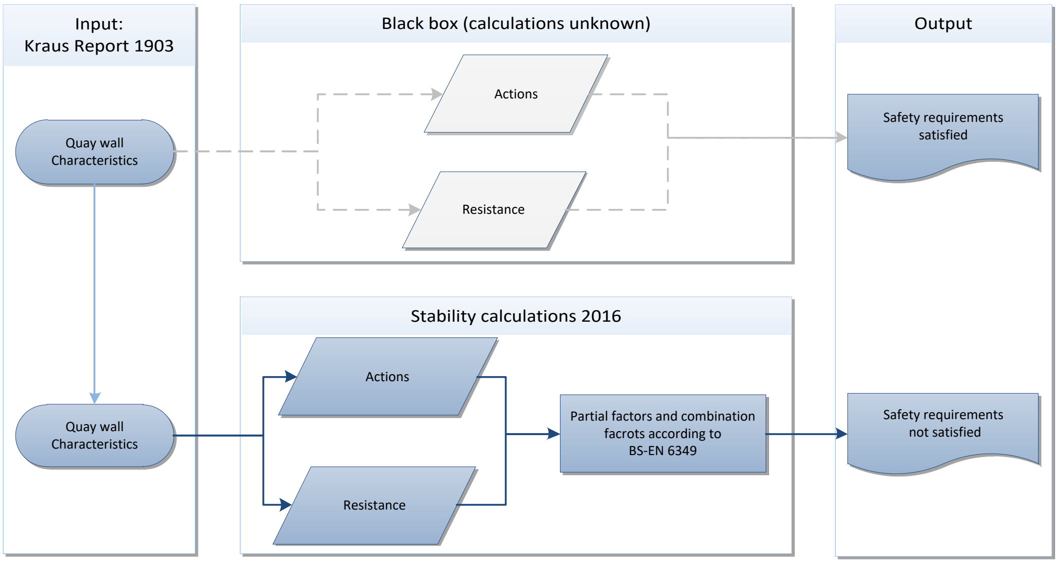


Figure B.1. Approach for analysing the stability calculations

According to the Eurocode 7, the following limit states for gravity based structures must be considered:

a) Overturning: rigid foundation (EQU limit state)
b) Forward sliding (GEO limit state)
c) Overturning: soil foundation (GEO limit state)
d) Bearing failure (GEO limit state)
e) Ground failure (GEO limit state)
f) Structural failure (STR limit state)

The limit states are visualized by the schematizations below.

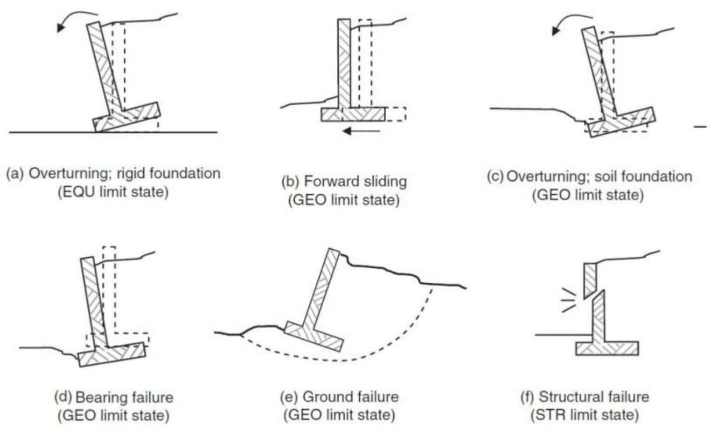


Figure B.2. Failure mechanisms for soil retaining structures

For analysis, the limit state for ground failure (e) is not considered. This failure mode is assumed to be highly depending on local circumstances, and not necessarily influenced by the application of the overturning concept itself.

B.2. Material parameters

Based on the report of professor Kraus, the following situation is considered for the stability calculation in operational conditions:

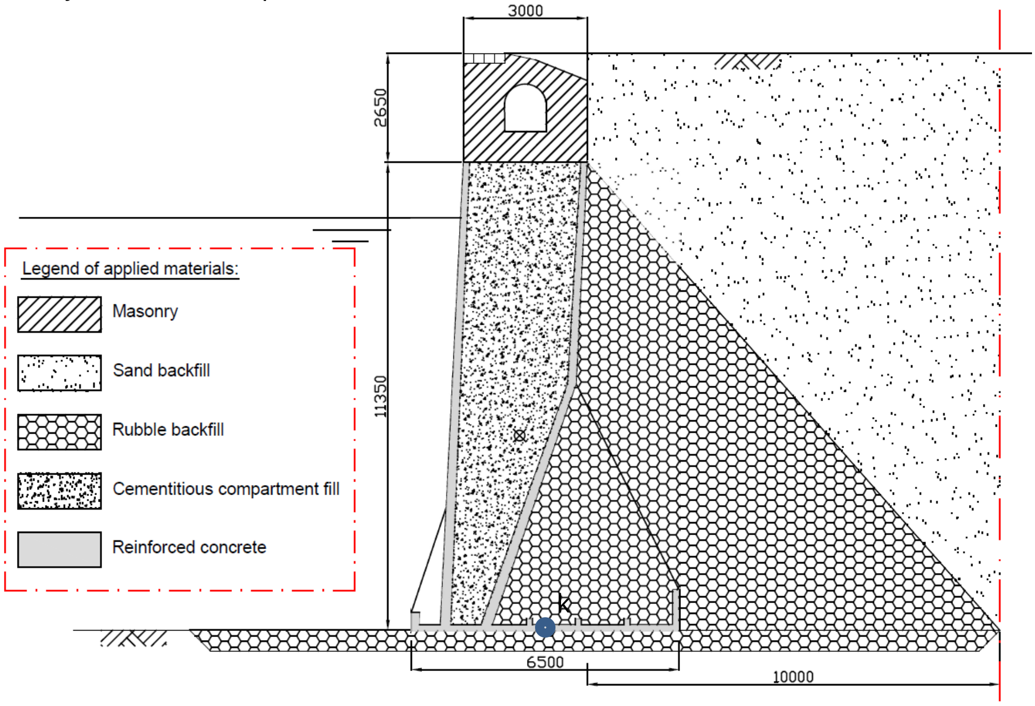


Figure B.3. Operational situation without variable quay loads

In addition to the shape and applied building materials, the following input parameters have been obtained from the report:

Input parameters (1903)	Value
Specific weight of armed concrete	23 kN/m^3
Specific weight of sand concrete and masonry	20 kN/m^3
Specific weight of the rubble behind the wall, counting the empty space left by the stones	18 kN/m^3
Specific weight of the submerged rubble, under the level of +1.00m CD	12 kN/m^3
Charge of the wall and adjacent grounds due to the merchandise and rolling material, answering to a weight uniformly distributed of	60 kN/m^2
Tangent of the natural talus of the ground (45°)	1
Tangent of the angle of friction between the ground and the wall (26° 40’)	½

The report also provides notes on the calculation method and pressures on the foundation. The report states that it is assumed that the weight of the backfill is accounted for up to a vertical virtual plane along the heel. The foundation pressure is calculated to be at most 350 kN/m^2 at the toe and 59 kN/m^2 at the heel of the structure. The average foundation pressure from the report (1903) can thereby be calculated as:

$$P_K = \left(\frac{P_{M,1} - P_{M,2}}{2}\right) = \left(\frac{350 - 59}{2}\right) = 146 \text{ kN/m}^2$$

From this point, the governing vertical resultant force (V) and destabilizing moment (M) can be recalculated with use of the superposition principle. The values below are obtained without knowledge of the original stability calculation. The occurring moment corresponding to the given foundation pressure amounts:

$$M_K = \left(\frac{P_{M,1} - P_{M,2}}{2}\right) \cdot W$$

$$M_K = \left(\frac{350 - 59}{2}\right) \cdot \frac{1}{6} \cdot 1.00 \cdot 6.50^2 = 1025 \text{ kNm}$$

B.3. Verification of operational stability (resistance)

The stability of the quay structure can be calculated with the previously presented parameters and geometry. The original stability calculations are not available, but the outcome in 1903 is probably similar to the current outcome, since the identical input parameters are used.

From all structural elements, only the exact weight of the superstructure is unknown and therefore estimated to be 135 kN/m[1]. This is based on the superstructure dimensions with a service-opening equal to 15% of the cross section. The considered lever arms are rounded values to +/- 5 centimetres.

Description of element	Volume	Specific weight	Weight per running metre quay	Lever arm from mid-point (k)	Moment (from point k)
Masonry superstructure;	6.75m^3	20 kN/m^3	135 kN	-0.50 m	-67.5 kNm
Concrete caisson;	9.60m^3	23 kN/m^3	221 kN	-0.60 m	-132.6 kNm
Cementitious compartment fill;	17.50m^3	20 kN/m^3	350 kN	-0.85 m	-297.5 kNm
Sand backfill (dry);	6.50m^3	18 kN/m^3	117 kN	+2.25 m	263.3 kNm
Rubble backfill (wet);	30.00m^3	22 kN/m^3	660 kN	+1.60 m	1056.0 kNm
Water column above toe	9.00 m^3	10.3 kN/m^3	93 kN	-2.77 m	-257.6 kNm
Total stabilizing effects		***V_R =***	***1,576 kN***	***M_R =***	***565 kNm***
Hydraulic uplift component (rectangular)	67.30 m^3	10.3 kN/m^3	-693 kN	+0.00 m	-0.0 kNm
Hydraulic uplift component (triangular)	3.25 m^3	10.3 kN/m^3	33.5 kN	+1.08 m	-36.6 kNm
Effective stabilizing effects		***$V_{R.eff}$ =***	***849 kN***	***$M_{R,eff}$ =***	***529 kNm***

Based on the weight calculation, the average effective foundation pressure is:

$$P_M = V_R / A_R = 849 / (6.50 \cdot 1.00) = 131 \text{ kN/m}^2$$

In this case, the allowable moment with foundation pressure over the full width is:

$$M_{R,tot} = M_{R,eff} + P_M \cdot W$$

$$M_{R,tot} = 529 + 131 \cdot \frac{1}{6} \cdot 1.00 \cdot 6.50^2 = 1{,}449 \text{ kNm}$$

The calculated moment found from the foundation pressures in the report remained below this value. Therefore, the given pressures from the report remain within the limits of SLS (kern) criteria.

B.4. Verification of representative loads (actions)

For limit state verifications, the wall is assumed to move in such an amount that an active soil pressure state occurs. The active horizontal earth pressure coefficient can nowadays be calculated by the formula proposed by Müller-Breslau (1906). The formulation including wall friction may only be used for the upper half of the caisson, since the lower failure wedge remains trapped in the heel of the caisson. Therefore, Rankine's approach is used for calculating the earth pressure on the lower half of the caisson (see appendix L).

During the design phase of the first caissons, the Müller-Breslau formula was not yet presented. Nevertheless, this formula has been derived on basis of the analytical theory of Coulomb (1776), which was most probably the method for calculating soil pressures in 1903. Therefore, the outcome in terms of active soil pressures is not expected to differ significantly. The active soil pressure coefficient can be expressed as:

$$K_a = \frac{\cos^2(\phi + \alpha)}{\cos^2(\alpha)\left(1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\cos(\alpha - \delta)\cos(\alpha + \beta)}}\right)^2}$$

In which:

Obliqueness of the structure:	α = 0°, 3°, 29°
Angle of the ground level:	β = 0°
Angle between the resultant force exerted on the retaining wall and the normal to this wall:	δ = 2/3 ϕ
Angle of internal friction of rubble:	ϕ = 45°

The sign conventions for the use of the Müller-Breslau formulation are depicted below. The K_a values for the different conditions are shown on the drawing on the next page.

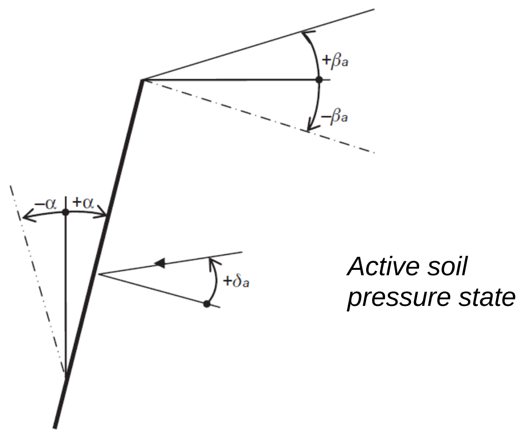


Figure B.4. Inclinations and sign convention Müller-Breslau formula

Fundamental actions

The characteristic combination is used for irreversible limit states. Characteristic design situation is:

$$\sum_{j\geq 1} G_{K,j} + Q_{K,1} + \sum_{i>1} \psi_{0,i} Q_{K,i}$$

The calculated soil pressure on the back of the wall results in a destabilizing moment of M_S = 1,096 kNm. The horizontal soil thrust amounts F_s = 252 kN. An overview of all actions is given in table B.1.

Element	Remarks and starting points	Thrust [kN]	Lever arm [m]	Moment [kNm]
Soil pressure	GW = +1.00m CD Active Rankine zone ½H Active Coulomb zone ½H	252	4.35	1,096
Live load	General cargo: 60 kN/m^2 (report)	124	6.00	744
Tidal lag	1/3 of tidal range; ∆H = 0.50m	55	5.40	298
Bollard load	The BS6349-1-2-2015: bollard load for vessels < 10,000 DWT -> 300 kN With a centre to centre distance of 25m, this results in 300 kN / 25m = 12 kN/m[1]	12	14.00	168

Table B.1. Overview of actions

When one designs a foundation according to the Eurocode 7, unless an adequate drainage system and maintenance plan are ensured, the ground water table should be taken as the maximum possible level. Nevertheless, the maximum groundwater level is considered to be equal to CD +1.00m. This water level difference is considered to be realistic. For instance; according to Furudoi and Katayama (1971)[20], the hydrostatic load generally equals about one-third of the tidal range above the low water level (LWL). Where the difference is less for cases where the quay-wall is placed on a permeable bedding and with a coarse granular backfill. The backfill has a natural drainage capacity, where water can flow through the caisson joints and subsoil.

Combination of actions

For the verification of loads, only one load case is considered. Therefore, this is not a comprehensive validation according to the code. However, it is sufficient for obtaining insight in the level of safety of the original caisson. The considered combination of actions for high water (+0.50m CD) behind the caisson is;

SLS Element	Comb. factor	Design thrust [kN]	Lever arm [m]	Destab. moment [kNm]
Soil pressure	1.00	252	4.35	1,096
Live load	1.00	124	6.00	744
Tidal lag	0.60	33	5.40	178
Mooring load	0.50	6	14.00	84
Total destabilizing effects		***F_S = 415 kN***		***M_S = 2,102 kNm***

The effective destabilizing SLS moment amounts:

ΣM_S = 2,102 kNm

[20]Furudoi, T. and Katayama, T.,1971. "*Field Observation of Residual Water Level*" Technical Note of PHRI, No. 115, Japan.

The representative destabilizing moments, calculated on basis of the BS6349, are almost twice as large as the calculated serviceability limit state actions in the previous section (1,025 kNm). And 1.5 times larger than the limiting value in which complete foundation pressure is present.

A possible explanation for the difference can be found in the large destabilizing moments caused by the live load and tidal lag. Perhaps, these loads were only partially included in stability calculations. A remarkable aspect is that the value for only the active soil pressure is close to the calculated SLS moment (1,025 kNm ↔ 1,096 kNm).

B.5. Serviceability verification

Forward sliding (GEO)
The sliding mechanism would occur in case of insufficient base friction.

$$\sum R_H \ \leq \ \sum V_R \cdot \tan(\delta)$$

$$V_{R,eff} = 849 \text{ kN/m}^1$$

$$R_{H,soil} = 415 \text{ kN/m}^1$$

$$\delta = \frac{2}{3}\phi' = \frac{2}{3} \cdot 45° = 30°$$

$$R_V = V_R \cdot \tan(\delta) = 849 \cdot \tan(30°) = 490 \text{ kN/m}^1$$

Overall factor of safety:

$$\frac{R_V}{R_H} = \frac{490 \text{ kN/m}^1}{415 \text{ kN/m}^1} = 1.20$$

This verification shows that the quay wall is just stable during its service life.

Overturning (EQU)

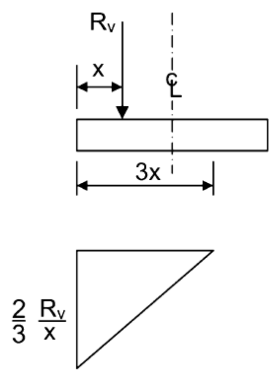
$$M_{R,\text{toe}} = M_{R,eff} + \frac{1}{2}B \cdot V_R = 529 + 0.5 \cdot 6.50 \cdot 849 = 3{,}288 \text{ kNm}$$

$$M_S = 2{,}102 \text{ kNm}$$

Overall factor of savety:

$$\frac{M_{R,\text{toe}}}{M_{S,eff}} = \frac{3{,}288 \text{ kNm}}{2{,}102 \text{ kNm}} = 1.55$$

Foundation pressure (GEO)



The sum of moments at the midpoint of the caisson (k) amounts:

Σ (M$_S$ ***– M***$_R$***)*** = 2,102 – 529 = 1,573 kNm

The eccentricity of the resultant force amounts:

$$e_{\text{kern}} = \frac{1}{6}B = \frac{1}{6} \cdot 6.50 = 1.08m$$

$$e_S = \frac{\Sigma M}{\Sigma V} = \frac{1{,}573 \text{ kNm}}{849 \text{ kN}} = 1.85 \text{ m} \left(> e_{\text{kern}}\right)$$

$$x = 0.5B - e = 0.5 \cdot 6.5 - 1.85 = 1.40 \text{ m}$$

This implies that the back of the caisson lacks foundation pressure, and that high foundation pressures would occur on the front side of the structure.

It can be concluded that the original foundation pressure calculations are the result of low destabilizing effects. Destabilizing actions are small enough when one considers permanent loads only. This could imply that only these loads were included for stability analysis, or that the permanent loads were predicted to be considerably lower than the values obtained by applying the Müller-Breslau formulation.

B.6. Ultimate limit state verification

The ultimate limit state is verified with a similar approach as the serviceability verification. The applied partial factors correspond to the factors provided by the BS-6349.

ULS Element	Partial factor	Comb. factor	Design thrust [kN]	Lever arm [m]	Desta. moment [kNm]
Soil pressure	1.35	1.00	340	4.35	1,479
Live load	1.50	1.00	186	6.00	1,116
Tidal lag	1.50	0.60	50	5.40	267
Mooring load	1.50	0.50	9	14.00	126
Total destabilizing effects			***F_S = 585 kN***		***M_S = 2,988 kNm***

Forward sliding (GEO)
The sliding mechanism would occur in case of insufficient base friction. The maximum calculated horizontal thrust is used in combination with the lowest water level.

$$\sum R_H \;\leq\; \sum V_R \cdot \tan(\delta)$$

$$V_{R,eff} = 849 \text{ kN/m}^1$$

$$R_{H,soil} = 585 \text{ kN/m}^1$$

$$\delta = \frac{2}{3}\phi' = \frac{2}{3}\cdot 45° = 30°$$

$$R_V = V_R \cdot \tan(\delta) = 849 \cdot \tan(30°) = 490 \text{ kN/m}^1$$

Overall factor of safety:

$$\frac{R_V}{R_H} = \frac{490 \text{ kN/m}^1}{585 \text{ kN/m}^1} = 0.85$$

This verification shows that the quay wall fails before reaching the ultimate limit state conditions.

Overturning (EQU)

$$M_{R,\text{toe}} = M_{R,eff} + \frac{1}{2}B \cdot V_R = 529 + 0.5 \cdot 6.50 \cdot 849 = 3{,}288 \text{ kNm}$$

$$M_S = 2{,}988 \text{ kNm}$$

Overall factor of savety:

$$\frac{M_{R,\text{toe}}}{M_{S,eff}} = \frac{3{,}288 \text{ kNm}}{2{,}988 \text{ kNm}} = 1.10$$

Foundation pressure (GEO)

The sum of moments at the midpoint of the caisson (k) amounts:

Σ (M_S – M_R) = 2,988 – 529 = 2,459 kNm

The eccentricity of the resultant force amounts:

$$e_{ULS} = \frac{1}{3}B = \frac{1}{3} \cdot 6.50 = 2.17m$$

$$e_S = \frac{\Sigma M}{\Sigma V} = \frac{2{,}459 \text{ kNm}}{849 \text{ kN}} = 2.90 \text{ m} \left(> e_{ULS}\right)$$

The ULS verification shows that the caisson is just stable for EQU conditions and that the GEO conditions are not satisfied.

B.7. Required stability adjustments

The caisson would be stable if the width is adjusted to 8.50 metre. This corresponds to approximately 75% of the caisson height. NB. 0.75 x 11.35 = 8.50.

For simplicity, the same values for destabilizing effects are taken into account. In practice, these values differ due to the widened heel and thus larger Rankine active state. Also the live load acts somewhat differently onto the quay. Nevertheless, these differences are neglected for this analysis.

Description of element	Volume per running metre	Specific weight	Weight per running metre	Lever arm from mid-point (k)	Moment (from k)
Masonry superstructure	6.75 m^3	20 kN/m^3	135 kN	-1.35 m	-182.3 kNm
Concrete caisson	10.60 m^3	23 kN/m^3	244 kN	-1.45 m	-353.5 kNm
Cementitious compartment fill	17.50 m^3	20 kN/m^3	350 kN	-1.70 m	-595.0 kNm
Sand backfill (dry)	11.40 m^3	18 kN/m^3	205 kN	+2.35 m	482.2 kNm
Rubble backfill (wet)	47.00 m^3	22 kN/m^3	1,034 kN	+1.85 m	1,913 kNm
Water column above toe	14.00 m^3	10.3 kN/m^3	144 kN	-3.80 m	-548.0 kNm
Total stabilizing effects		***V_R* =**	***2,112 kN***	***M_R* =**	***716 kNm***
Hydraulic uplift component (rectangular)	88.00 m^3	10.3 kN/m^3	-906 kN	+0.00 m	-0.0 kNm
Hydraulic uplift component (triangular)	4.25 m^3	10.3 kN/m^3	-44 kN	1.40 m	-62.3 kNm
Effective stabilizing effects		***$V_{R.eff}$* =**	***1,162 kN***	***$M_{R,eff}$* =**	***655 kNm***

Forward sliding (GEO)

The sliding mechanism would occur in case of insufficient base friction. The maximum calculated horizontal thrust is used in combination with the lowest water level.

$$\sum R_H \;\leq\; \sum \mathrm{V}_R \cdot \tan(\delta)$$

$$V_{R,eff} = 1{,}162 \text{ kN/m}^1$$

$$R_{H,soil} = 415 \text{ kN/m}^1$$

$$\delta = \frac{2}{3}\phi' = \frac{2}{3} \cdot 45° = 30°$$

$$R_V = V_R \cdot \tan(\delta) = 1{,}162 \cdot \tan(30°) = 671 \text{ kN/m}^1$$

Overall factor of safety:

$$\frac{R_V}{R_H} = \frac{671 \text{ kN/m}^1}{415 \text{ kN/m}^1} = 1.60$$

Overturning (EQU)

$$M_{R,\mathrm{toe}} = M_{R,eff} + \frac{1}{2} B \cdot V_R = 655 + 0.5 \cdot 8.50 \cdot 1{,}162 = 5{,}594 \text{kNm}$$

$$M_{\mathrm{S}} = 2{,}102 \text{ kNm}$$

Overall factor of savety:

$$\frac{M_{R,\mathrm{toe}}}{M_{\mathrm{S},eff}} = \frac{5{,}594 \text{ kNm}}{2{,}102 \text{ kNm}} = 2.65$$

Foundation pressure (GEO)

The effective moment at the midpoint of the caisson (k) amounts:

Σ (M_S – M_R) = 2,102 – 655= 1,447 kNm

The eccentricity of the resultant force amounts:

$$e_{\mathrm{kern}} = \frac{1}{6} B = \frac{1}{6} \cdot 8.50 = 1.40 m$$

$$e_S = \frac{\Sigma M}{\Sigma V} = \frac{1{,}447 \text{ kNm}}{1{,}162 \text{ kN}} = 1.25 \text{ m}$$

$e_S < e_{\mathrm{kern}} \;\rightarrow$ eccentricity ok

The distance from the toe to the resultant force is:

$$x = 0.5B - e = 0.5 \cdot 8.50 - 1.25 = 3.00 \text{ m}$$

B.8. Structural capacity

The basic principles regarding strength of the caisson will be analysed and roughly calculated. Note that it is not intended to be comprehensive. The original strength parameters for the reinforced caisson are listed in the table below.

Material characteristics (1903)	**Value**
Concrete	
Concrete compressive strength	15.00 N/mm^2
Concrete tensile strength	2.00 N/mm^2
Overall safety factor (during transport)	2.00
Design value of concrete compressive strength	7.50 N/mm^2
Design value of concrete tensile strength	1.00 N/mm^2
Reinforcement	
Iron yield strength	250 N/mm^2
Bar diameter (lower part of caisson)	1/2” ≈ 12.7mm
Bar spacing (lower part of caisson)	70mm
Concrete cover (estimate)	10mm
Wall properties	
Height at critical depth (h)	232 mm
Effective depth ($d \approx h - c - 0.5\phi$)	232 – 10 – 6= 216mm
Internal lever arm ($z_u = 0.85 \times d$)	0.85 x 216 = 184mm

Wall thickness

The walls could be designed with a thickness of 150mm at the top and 250mm at the bottom of the caisson. This limited wall thickness was possible due to the limited hydrostatic pressure on the top of the caisson.

The schematic load case on the front-wall of the caisson is as follows:

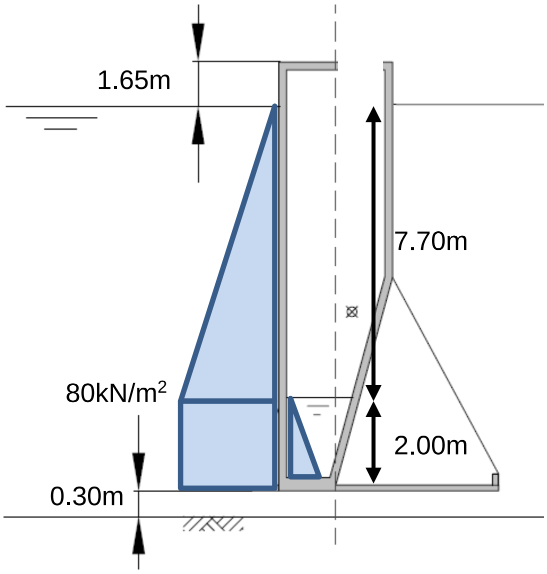


Figure B.5. – Hydrostatic pressure during immersion

The maximum water level difference during immersion would be roughly 7.70 metres. This would result in a maximum hydraulic pressure on the walls of P = 10.30 x 7.70 ≈ 80 kN/m^2. Due to the wall tapering, which varies from 150 to 250mm, the local wall thickness (h_w) at the maximum hydraulic pressure amounts:

$$h_w = 250 - \left(\frac{2.00}{11.35} \cdot 100 \right) = 232 \text{ mm}$$

Bending moment capacity
The bending moments acting on the walls can be calculated with the before mentioned hydrostatic pressure. The sidewalls have a limited span length at the point with the highest hydrostatic pressure due to the declined backwall. Therefore, the front- and back-walls are governing for the design of the caisson.

Considering similar material properties as for the design in 1903; reinforcement steel with a design yield stress of 250 N/mm^2 and the area reinforcement area (A_s) per running metre equal to 15 bars Ø12.7mm (≈1900mm^2). This amount is equal to a reinforcement percentage of approximately 0.8%, which is acceptable for concrete classes of C12/15 (avoiding brittle failure).

The separation walls can be considered as supports for the front- and back-wall of the caisson, which results in a schematization of a beam on multiple supports. The bending moment and the wall capacity can roughly be verified as:

$$M_E \leq M_R$$
$$M_E \approx 1/12 \cdot q \cdot l^2 = 1/12 \cdot 80 \cdot 2.5^2 = 42 \text{ kNm}$$
$$M_R \approx A_s \cdot f_{yd} \cdot z_u = 1900 \cdot 250 \cdot 184 / 10^6 = 87 \text{ kNm}$$
$$F.o.S = \frac{M_R}{M_E} = \frac{87}{42} = 2.08$$

This calculation shows that the desired factor of safety of 2 is satisfied according to current preliminary design rules. A more exact calculation is not expected to deviate significantly.

Shear force capacity
There were probably no formulations for shear force capacity of unreinforced concrete available in time of the design of the first caissons (see also chapter 4). The capacity estimates were made based on specific strength test for this quay project.

Shear reinforcement in walls is generally undesired due to labour and executional aspects. The first caissons were also designed without shear reinforcement, and perhaps it was a governing aspect in determining wall thickness. In order to verify if shear resistance could be decisive for the design, the capacity is analysed. The following verification can be made if the concrete caisson would be designed with the same dimensions as used in 1903, but in accordance with the current Eurocode 2:

$$v_E \leq v_{R,\mathrm{d,c}}$$

The shear force acting on the walls is:

$$V_E = 0.5 \cdot q \cdot l_{eff} = 0.5 \cdot 80 \cdot 2.50 = 100 \text{ kN}$$
$$v_E = V_E / (d \cdot b) = 100 / (216 \cdot 1000) = 0.46 \text{ N/mm}^2$$
$$v_E \leq v_{R,\mathrm{d,c}}$$

Note: the shear capacity near the supports is actually higher, but not included for this analysis.

The design value for the shear resistance is given by:

$$v_{\min} = 0.035 \cdot k^{3/2} \cdot f_{ck}{}^{1/2}$$

$$v_{R,\text{d,c}} = C_{R,\text{d,c}} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}$$

In which:

$$C_{R,\text{d,c}} = 0.18 / \gamma_c = 0.18 / 1.5 = 0.12$$

$$k = 1 + \sqrt{\frac{200}{d}} = 1.96$$

$$\rho_1 = \frac{A_s}{d \cdot b} = \frac{1900}{216 \cdot 1000} = 0.009$$

f_{ck} = characteristic compressive cylinder strength = 12 N/mm^2

$k_1 = 0.15$

$\sigma_{cp} = 0.00$ N/mm^2 (conservative estimate during immersion)

$$v_{R,\text{d,c}} = 0.12 \cdot 1.96 \cdot (100 \cdot 0.009 \cdot 12)^{1/3} = 0.52 \text{ N/mm}^2$$

$$v_{\min} = 0.035 \cdot 1.96^{3/2} \cdot 12^{1/2} = 0.33 \text{ N/mm}^2$$

The overall factor of safety amounts:

$$F.\text{o.S} = \frac{v_{R,\text{d,c}}}{v_E} = \frac{0.52}{0.46} = 1.13$$

This shows that the shear stress is theoretically just sufficient during the immersion process. The originally desired factor of safety of 2 is not satisfied without shear reinforcement.

The value of shear resistance $v_{R,d,c}$ is calculated according to the EN 1992-1-1 eq. 6.2.b. The formula for determining the characteristic shear resistance is actually based on higher steel reinforcement grades (B500). Therefore, the estimated capacity deviates from the actual result, but is seen as a reasonable value. The minimum value of shear resistance (without reinforcement) is calculated according to equation 6.3N. Both calculations are based on an effective depth (d) of 216mm and concrete strength class C12/15.

B.9. Simplified model: floating position

The following calculation is based on rounded values and gives an indication of the floating position and stability of the empty caisson. The caisson is only considered in transversal direction for this moment.

Values

Distance to centre of gravity from bottom of caisson:	5.266 m
Caisson weight:	2600 kN
Displaced water:	260 m^3
Required buoyancy area for 10 metre long caisson = 260/10 =	26 m^2

Equations

1 (eq1):	caisson weight = weight displaced water
2 (eq2):	horizontal distance of metacentric height = distance of centre of gravity

Model of buoyancy

In order to find the buoyant point, a simplified model is used consisting of one rectangle and one triangle. The simplified shape of displaced water is shown in the schematic representation on the next page. Maple output (shown on the next page) gives a total draught of approximately (1.16 + 2.26) 3.42 m.

> $G := 4.74$

$$G := 4.74$$

> $h := 11.35$

$$h := 11.35$$

> $A := 26$

$$A := 26$$

> $A1 := d1 \cdot h$

$$A1 := 11.35\, d1$$

> $A2 := 0.5 \cdot d2 \cdot h$

$$A2 := 5.675\, d2$$

> $area := A1 + A2 = A$

$$area := 11.35\, d1 + 5.675\, d2 = 26$$

> $eq1 := h \cdot d1 + 0.5 \cdot h \cdot d2 = A$

$$eq1 := 11.35\, d1 + 5.675\, d2 = 26$$

> $eq2 := \dfrac{\left(\left(\frac{1}{2}\right) \cdot h \cdot A1 + \left(\frac{1}{3}\right) \cdot h \cdot A2\right)}{A} = G$

$$eq2 := 2.477355769\, d1 + 0.8257852565\, d2 = 4.74$$

> $sol := solve(\{eq1, eq2\}, [d1, d2])$

$$sol := [[d1 = 1.158493276, d2 = 2.264511246]]$$

> $assign(sol);$
> $d1 + d2 :$
> $evalf(\%, 3);$

$$3.42$$

Floating stability
The distance to the metacentric height can be found by approximately:

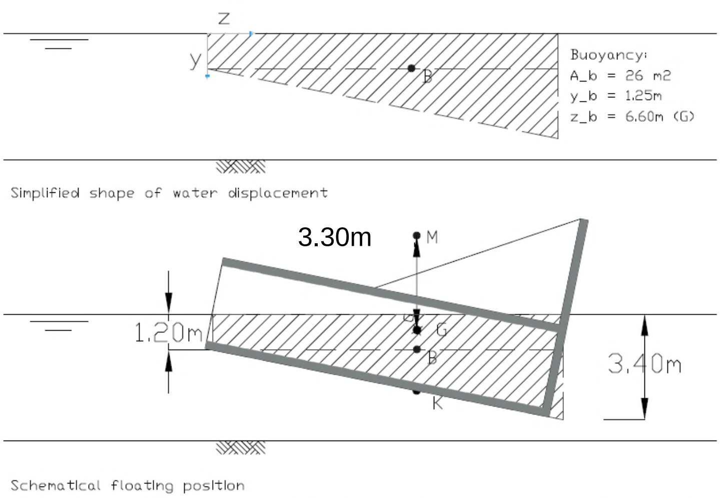


Figure B.6. Schematic floating position of a simplified caisson

$$\overline{BM} = \frac{I_{xx}}{V_w} \approx \frac{\frac{1}{12} l \cdot b^3}{V_w} = \frac{\frac{1}{12} \cdot 10 \cdot 11.35^3}{260} = 4.7\text{m}$$

$$\overline{GM} = \overline{BM} - \overline{BG} \approx 4.7 - 1.4 = 3.3\text{m}$$

In which I_{xx} is not the exact value since the moment of inertia is has changed due to the asymmetrical shape of the cross section. The estimated error is acceptable since it is has only been calculated for preliminary analysis purposes.

Turning

The total weight of the caisson amounts approximately 2600 kN. The total water displacement amounts approximately 355 m^3, which implies a theoretical ballast capacity of 95 m^3. When the compartments are filled with 70m^3 water, the centre of gravity will lower to roughly 3.70 metre. The total water displacement amounts ca. 33 m^2/m. The following floating position will be obtained if no water enters the heel.

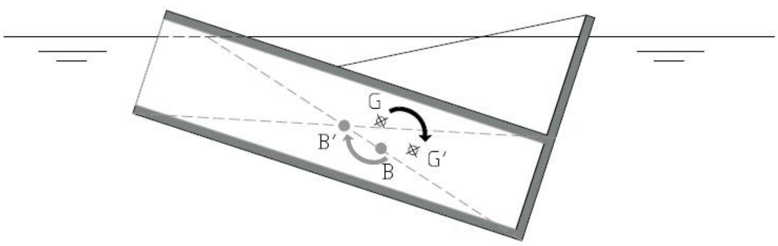


Figure B.7. Change of the buoyancy point (B) and centre of gravity (G) during ballasting

B.10. Design considerations in relation to transport

Maximum eccentricity of the centre of gravity (G) amounts 1/6B and 1/6H for a rectangular floating object with a water displacement (Δ) equal to 50%. The caisson will not tipple over (for hydrostatic conditions) if point G remains within the hatched area. Based on these conditions, the caisson would directly heel over if the centre of gravity is positioned outside 1/6B of the cross-section.

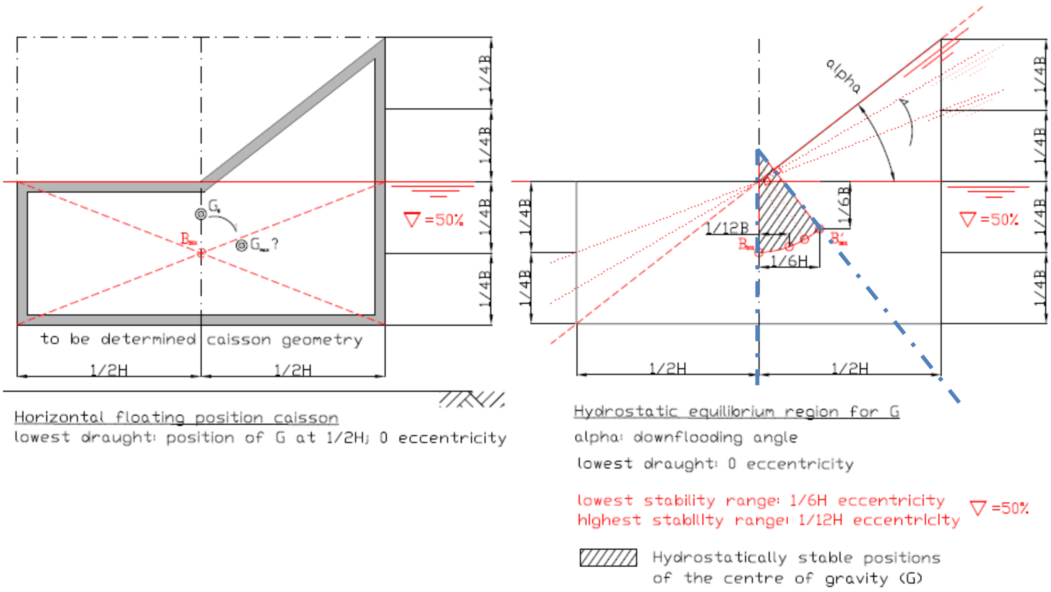


Figure B.8. Displacement (Δ =50%) and possible locations of the centre of gravity (G)

A similar approach can be followed to determine the possible locations of the centre of gravity when (for instance) 25% water is displaced. It can be seen that a low relative weight allows more freedom in possible locations. For this case, the limiting values for G are equal to 1/3B and 1/3H from the middle. However, these eccentric locations still cause significant draught increase.

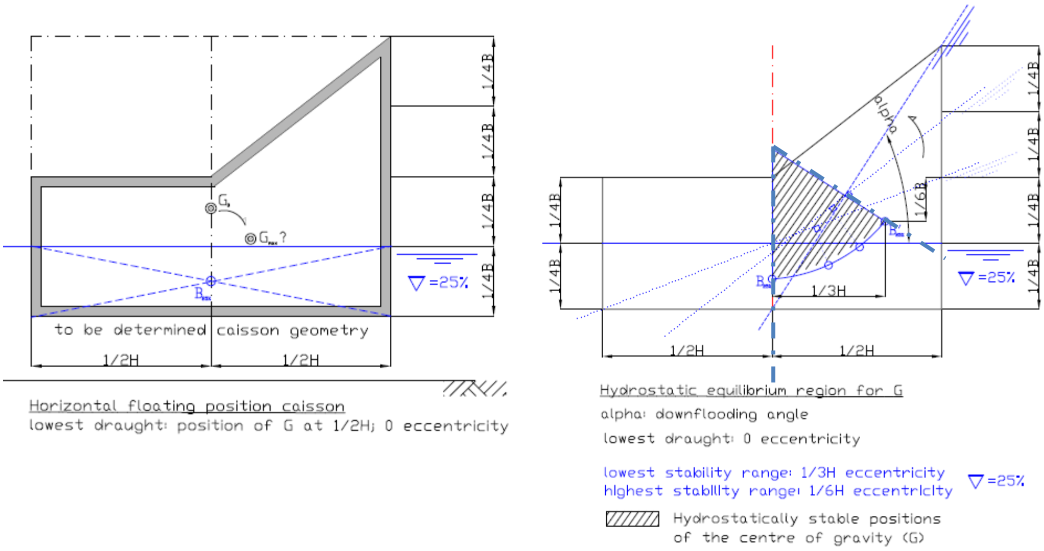


Figure B.9. Displacement (Δ =25%) and possible locations of the centre of gravity (G)

B.11. Significance of horizontal construction (formwork)

A preliminary cost estimate is made in order to quantify the significance of less formwork. Material use and other aspects are kept equal in order to quantify the relative difference. The estimations are based on a caisson structure with a height of 11.35 metre and a width of 6.50 metre. The simplified structure consists of 2 walls and 1 floor in case of vertical execution and 2 floors and 1 wall in case of horizontal execution.

Vertical execution					
Description	Amount	Total	Prize per unit	Total costs	Weight
Concrete C35/45	1	104 m^3	€ 150	€ 15,600,-	0.18
Reinforcement B500	150 kg/m^3	15,600 kg	€ 1	€ 15,600,-	0.18
Formwork slab	1	65 m^2	€ 20	€ 1,300,-	0.02
Formwork walls	2	910 m^2	€ 15	€ 13,650,-	0.16
Preparation formwork slab	1	65 m^2	€ 40	€ 2,600,-	0.03
Preparation formwork walls	2	910 m^2	€ 40	€ 36,400,-	0.43
Total:				€ 85,150,-	1.00

And for the horizontal construction method:

Horizontal execution					
Description	Amount	Amount	Prize per unit	Total costs	Weight
Concrete C40/45	1	104 m^3	€ 150	€ 15,600-	0.20
Reinforcement B500	150 kg/m^3	15,600 kg/m^3	€ 1	€ 15,600,-	0.20
Formwork slabs	2	230 m^2	€ 20	€ 4,600,-	0.06
Formwork walls	1	580 m^2	€ 15	€ 8,700,-	0.11
Preparation formwork slabs	2	230 m^2	€ 40	€ 9,200,-	0.12
Preparation formwork walls	1	580 m^2	€ 40	€23,200,-	0.30
Total:				€ 76,900,-	1.00

By only changing the position of the structure during casting, a cost saving of approximately € 85,150 – € 76,900 = € 8,250 per running metre quay wall can be obtained. This could be a significant saving to the overall construction costs. However, as a direct consequence of the casting position, the caisson needs to be turned before it can fulfil its purpose. The actual cost savings due to savings in formwork costs are therefore not directly clarified by this estimate.

The complete structure, combined with construction technologies and labour costs are therefore considered in chapter 7 and 8 to obtain representative values.

C. Durability aspects

Nowadays, it is known that durability of reinforced concrete is not some given characteristic of the material itself. Many different aspects influence the durability and life time. Only if the structure is designed and built properly, the desired performance can be achieved. Durability aspects are perhaps even more important than the compressive strength, since the majority of problems are associated with degradation, rather than lack of strength.

Durability of concrete can be defined as; the ability to resist attack from environment in which it is placed. The attack can be either physical or chemical. Examples of different attacks are presented in table C.1 below.

Physical attack	Chemical attack
Abrasion	Sulphates
Impact	Chlorides
Ice growth (freeze thaw)	Carbon dioxide
Permeation / diffusion	Alkalis
	Acids

Table C.1. Different forms of attack on a concrete structure

Form the examples of physical attacks, abrasion and (ship) impact are from importance for quay wall design. On the other hand, chlorides and carbon dioxide are from major importance when considering chemical attack. These can influence the concrete quality and induce corrosion of carbon steel reinforcement.

C.1. Historical overview of corrosion protection

The first large reinforced concrete structures have been built in the early twentieth century. During this period, it was assumed that cement and iron would chemically react and form iron-silicate. This would develop a passive layer around the reinforcement and it would prevent corrosion of reinforcement (Verhey, 1912). The passivation was thereby assumed to be irrespective of porosity or cracking of concrete.

The initial purpose of a concrete cover was thereby only to transfer bond forces. During this juvenile period of reinforced concrete applications, the possibility of micro crack formation in existing concrete structures was discussed in relation to durability[21], but its magnitude and significance was still unclear. The discussion on how this could affect the durability for reinforced concrete structures in marine environments resulted in many years of research following.

The first Dutch concrete regulation, the Gewapend Beton Voorschriften (GBV 1912)[22] was published by the Koninklijk Instituut van Ingenieurs (KIVI) as a “permanent appendix” of the KIVI yearbook. The recommended concrete cover varied between 10 and 15mm, depending on the geometry. Remarkable is that only one sentence was assigned to this subject. In the following years, the recommendations by KIVI regarding the concrete cover became more extensive. This was not directly leading to larger cover depths; the Dutch KIVI standard 1930 addresses for instance that one has to be cautious when “very large” cover depths of 50mm in aggressive environments are applied, because of the risk of cracks due to shrinkage.

The increasing knowledge regarding corrosion protection eventually resulted that the minimum cover depth has increased drastically over the years. This is due to awareness that steel reinforcement is not by definition in a passive state if an arbitrary concrete cover is applied. The severity of chloride ingress and carbonation are strongly influenced by the cover depth to the steel reinforcement. In the dissertation of Gaal

[21] Plasscheart, B. F. (1902), Beknopt practisch leerboek der burgerlijke en waterbouwkundige
[22] Gewapend-beton-voorschriften, vastgesteld in de vergadering van 23 maart 1912 van de afdeeling voor bouw- en waterbouwkunde van het Koninklijk Instituut van Ingenieurs / D. Kruyf

(2004), the prescribed concrete cover depths by Dutch standards for wet environments exposed to chlorides are presented of the last century. Based on his work, a graph is plotted of prescribed values of cover depths for structural walls and slabs designed for exposure to marine environments (fig.C.1). The prescribed concrete cover shows a pronounced increase.

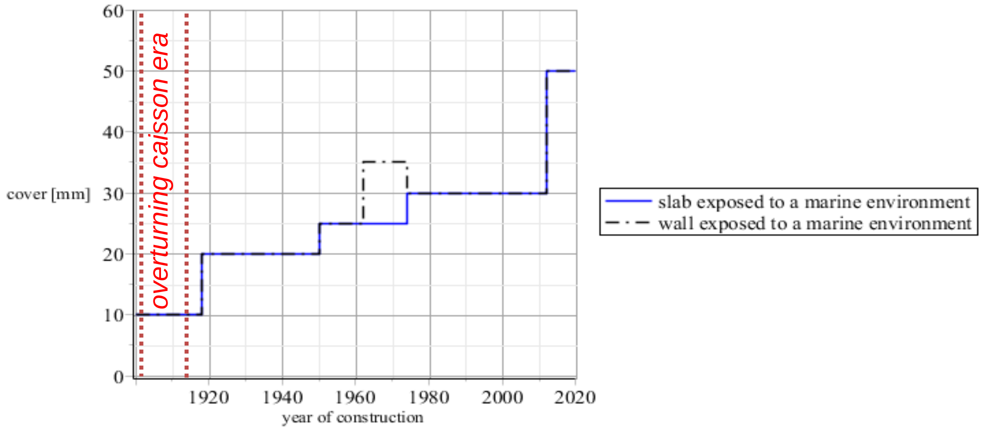


Figure C.1. Concrete cover regulations for concrete in marine environments (1900 -2016)

It is not known which cover depths were applied before 1912 (it could even be have been less than 10mm), since no Dutch concrete design regulations existed. It might be a coincidence, but a remarkable aspect is that the original overturning caisson was designed for several projects between 1903 and 1914, which is a typical period that lacks cover recommendations and durability knowledge.

C.2. Corrosion protection

Protection of steel reinforcement to prevent corrosion is generally done by applying a proper concrete mixture and a certain concrete cover. In addition to cover depth requirements, crack width is controlled by limited allowable steel stress levels in the serviceability limit state, a maximum centre to centre distance and rebar diameter requirements. If these measures are sufficient, possible existing corrosion on the reinforcement will not propagate.

Regular (carbon) steel reinforcement, which is embedded in concrete, will thereby not corrode due to the existence of a protective layer, which passivates the steel in the strong alkaline conditions of the concrete pore water. Passivity can be destroyed by several mechanisms. This occurs for example when chlorides penetrate through concrete and reach steel reinforcement. At this point, corrosion can be initiated.

As a result of the corrosion reaction, rust forms and increases the steel volume by 6 to 7 times. This can generate bursting forces which can exceed the tensile strength of concrete, resulting in cracking and spalling of the concrete. This eventually leads to further corrosion and loss of bond between the concrete and steel.

C.3. Protection measures

Measures to protect reinforcement steel are to apply a concrete cover of sufficient quality (permeability / density), sufficient depth and a minimized crack width. In addition, also the following measures could be taken to improve the life time and/or reduce the concrete cover:

- Apply non-metallic reinforcement (e.g. fibre reinforced polymers);
- Apply alloyed steel types with a higher chloride corrosion threshold values (stainless steel);
- Apply a passive or active protection (cathodic protection);
- Apply coatings to the concrete surface or to the carbon reinforcement;

In relation to the design of the overturning caisson, finding a proper concrete mixture and adjusting the reinforcement for crack width control would not threaten the technical feasibility. By current techniques, a concrete mixture can be obtained which satisfies durability requirements in combination with acceptable permeability.

The consequence of an increased concrete cover threatens the feasibility for the self-floating concrete structure significantly. Namely, the required cover results in a weight increase and/or reduced strength. These aspects influence the design considerations drastically.

C.4. Environmental aggressivity

The corrosion rate of steel reinforcement depends on the environmental aggressivity. Oxygen, water and chlorides (figure C.2.) are majorly responsible for steel corrosion in marine environments. In cases where steel is passivated by concrete, carbon dioxide induces deterioration of the concrete cover by the carbonation process. Carbonation reduces the alkalinity of concrete and could thereby destroy the passive layer which prevents corrosion. In case of chloride ingress, corrosion can be initiated when a certain chloride corrosion threshold value has been reached. Both deterioration mechanisms differ and their combined impact can therefore be larger (e.g. for tidal splash zones).

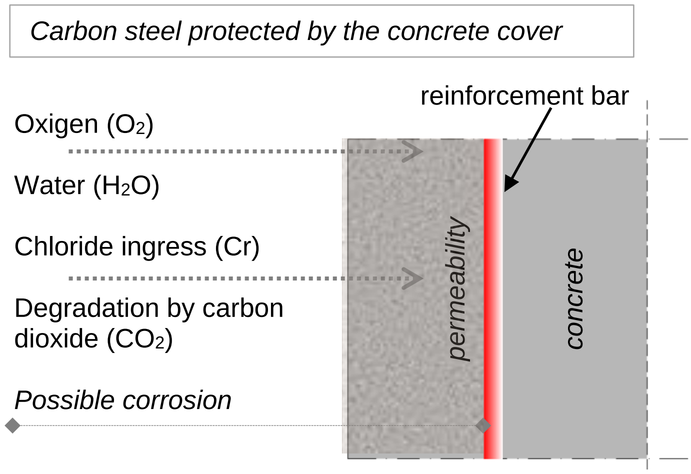


Figure C.2.Corrosion protection by a concrete cover in marine environments

C.5. Mechanical abrasion and impact

Berthing manoeuvres of ships, abrasion from steel mooring ropes and sand abrasion might affect the concrete of the upper part of the quay structure. These aspects result in additional requirements regarding robustness of quay wall structures. For this (marine) quay wall design, robustness in relation to maintenance is from importance.

Regarding this, the Eurocode 2 prescribes for instance that concrete abrasion may be allowed for by increasing the concrete cover which functions as a sacrificial layer. In that case the minimum cover c_{min} should be increased by 5, 10 or 15mm, depending on the severity of abrasion. The corresponding abrasion classes are respectively defined as XM1, XM2 and XM3. A sacrificial layer might therefore be applied at the upper part of the quay wall to take possible abrasion into account.

C.6. Concrete cover requirements and recommendations

Within Europe, durability aspects for marine structures are nowadays addressed by different countries in separate codes, recommendations and guidelines (e.g. the German EAU 2012, British BS 6349 and Norwegian NS 3473). These documents prescribe measures which generally go beyond the minimum requirements laid down in the Eurocode 2 (EN-1992-1-1).

With respect to the design of reinforced concrete structures in marine environments, the additional requirements and guidelines are desired due to the harmful effects of for instance; changing water levels, chlorides in waters and soils, ice loadings, ship impacts and abrasion. Due to the importance of durability requirements to the technical feasibility of the overturning caisson, the following codes are addressed:

Code / recommendation	
EN 1992-1-1: 2005	European Eurocode 2 - Design of concrete structures Part 1-1 general rules and rules for buildings
EAU 2012	German recommendations of the "Committee for Waterfront Structures Harbours an Waterways
BS 6349: 2013	British Standard for Maritime Works
NS 3473: 2003	Norwegian Standard for concrete structures – Design and detailing rules

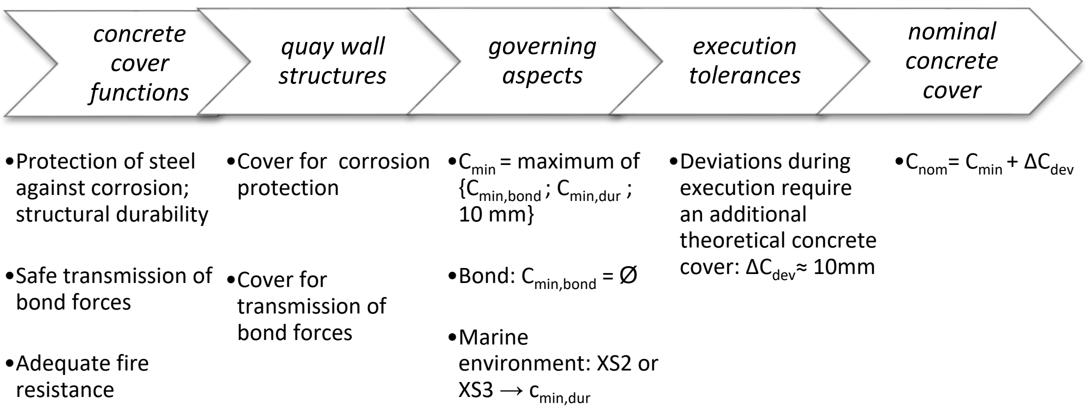
Table C.1. Addressed codes for the durability analysis

The Eurocode 2 is applicable for all the listed recommendations and standards and can be seen as basic framework.

Eurocode 2 – Design of concrete structures
There are two main functions of a concrete cover for quay walls prescribed by the Eurocode 2. The functions and cover notations are as follows:

1. Structural durability: minimum cover value denoted as $c_{min,dur}$;
2. Transfer of bond forces: minimum cover value denoted as $c_{min,bond}$;

The concrete cover could also improve fire resistance, but this effect is seemed to be of negligible influence for quay wall design. Based on the Eurocode 2, a schematized representation of the decision making process for a decent concrete cover for quay wall design is depicted below.



The prescribed minimum durability concrete cover by the Eurocode 2 is presented in table C.2. This table only specifies the minimum cover in terms of durability requirements. There are two main factors which influence the value in this case; the structural class (S1-S6) and exposure class. The recommended class (starting point) is S4, from which can be deviated when a different design life, concrete strength class, slab geometry and quality control is used.

Environmental Requirement for $c_{min,dur}$ [mm]							
	Exposure Class (EN-1992)						
Structural class	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	**50**	**55**

Table C.2. Minimum concrete cover for different exposure classes (Eurocode 2)

For quay walls in marine environments, exposure classes XS2 and XS3 are from major importance. These classes represent environments in which corrosion can be induced by chlorides from sea water.

Besides dependence on the exposure class, the required concrete cover is also affected by the structural class (S1 to S6). The code prescribes class S4 as starting point from which can be deviated when particular design criterion are satisfied. The design criteria, on which the structural class is dependent, are presented in table 2.2.

Criterion	Exposure class XD3 / XS2 / XS3
Design working life of 100 years	S4 +2
Strength class ≥ C45/55	S4 -1
Member with slab geometry (position of reinforcement not affected by construction process)	S4 -1
Special quality control of the concrete production ensured	S4 -1

Table C.3. Structural classes (Eurocode 2)

The most probable concrete cover requirement ($c_{min,dur}$ + Δc_{dev}) for marine structures with a design life of 50 year, would become 40 +10 = 50mm. For structural elements which are cyclic wet and dry, the nominal cover should be at least 55mm (based on 50 year design life).

From the table, it can be seen that the structural class could theoretically be reduced to S1. This is allowed on condition of a design life of 50 years, a strength class ≥C45/55, slab geometry and special quality control of the concrete production is ensured. This could theoretically result in a nominal concrete cover of 30 + 10 = 40mm. However, from research and field practice, such as obtained from Pier Scheveningen (Polder 2005), it is learnt that such low values results in severe structural damage within 50 years of service life.

The concrete strength class, mixture and curing affects the durability significantly. This correlation is for instance assimilated in the Dutch CUR-Leidraad 1. Besides the CUR guideline, the informative annex E provided by the Eurocode 2, recommends a minimum concrete strength class of C35/45 for XS2 and XS3 exposure classes. Altogether, the Eurocode is reserved regarding the concrete cover quality to durability relations.

It is remarkable that these prescribed values for concrete covers are relatively high compared to the Dutch VBC 1995, which is withdrawn just a couple of years ago (2012). Based on this norm, a total concrete cover of just 30mm would be allowed for structures exposed to XS3 and plate geometry.

EAU 2012 – Recommendations of the committee of Waterfront Structures
The EAU 2012 states that, for quay walls, the concrete cover should be larger than that given in DIN EN 1992-1-1 and at least c_{min} = 50mm, with a nominal cover c_{nom} = 60mm. For most conditions, a similar value would be obtained if a quay structure would be

designed according to the EN 1992 only, but the EAU 2012 prescribes this value for as a minimum for all environments, which makes it more stringent in terms of durability requirements.

The minimum thickness of a caisson front-wall recommended by the EAU 2012 is 300mm. The backwall may be reduced to a minimum thickness of 250mm. An overview of the recommended wall thicknesses is given in the table below. Besides thicknesses, the EAU also recommends to adjust shapes of particular members to ensure a durable design. For instance, concrete walls should have a 5 × 5 cm chamfer along their upper edges or be correspondingly rounded and/or protected on the water side by steel angles in the case of transhipment operations.

Member	**Minimum wall thickness [mm]**
Face wall	300
Rear and side walls	250
Internal diaphragm	200

Table C.4. Minimum wall thickness recommended by the EAU 2012

Furthermore, the EAU 2012 prescribes the following exposure classes (figure C.3.) for concrete maritime structures in sea water environments. Where ship contact is allowed for in design, an additional concrete cover of at least 5mm is prescribed as sacrificial concrete cover (XM1).

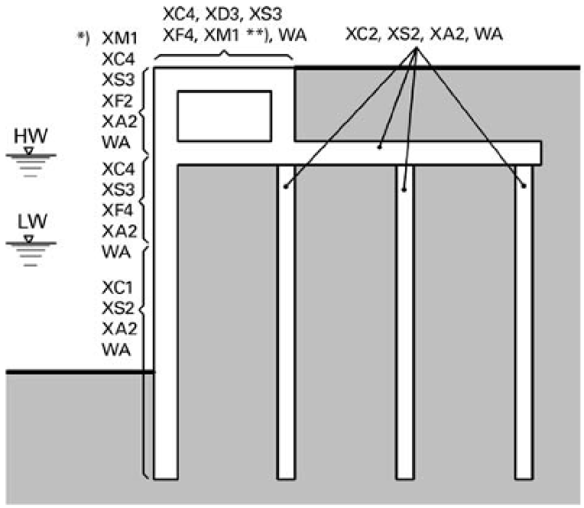


Figure C.3. Exposure classes for quay walls in marine environments (EAU 2012:R72-2)

BS 6349 – Maritime works

The BS 6349 - code of practice for marine works - is part of the British Standard. It gives recommendations for minimum concrete covers depending on design working life, exposure class, concrete strength, water/cement ratio, minimum cement content and specific cement restrictions.

A concrete cover (c_{nom}) up to 80mm can be recommended in the most severe conditions. Similar to the Eurocode 2, a Δc of 10mm has to be added for normal in situ-construction. Table C.5 shows recommended concrete cover values on basis of a design life of 50 years and a tidal splash zone (XS3). The lowest recommended cover can be obtained in combination with concrete class C40/50, a water/cement ratio of 0.35 and a minimum of 380 kg/m^3 cement. Furthermore, the permissible cement combinations would be CEM III/A/B or CEM II/B-V+SR with specified percentages of fly ash and blast furnace slag. For increasing concrete covers, the concrete mixture requirements are more relaxed.

The concrete cover recommendations for a structural design life of 100 years are roughly 15 to 20mm higher, depending on the particular exposure class.

Nominal cover [mm]	45 + ∆c	50 + ∆c	55 + ∆c	60 + ∆c	65 + ∆c
Blast furnace slag: >45% Fly ash content: >25%	C40/50 0.35 380	C35/45 0.45 360	C32/40 0.50 360	C28/35 0.55 340	C25/30 0.55 340
Blast furnace slag: >36% Fly ash content: >21%	--	C40/50 0.35 380	C35/45 0.40 360	C32/40 0.45 360	C28/35 0.50 360

Table C.5. - Limiting values for composition and properties of concrete classes with normal weight aggregates of 20 mm maximum size exposed XS3 (UK seawater conditions) for a required design working life of 50 years.

NS 3473 – Concrete design and detailing rules

Norway has a relatively long coast and numerous marine structures on which the chloride induced failure mechanisms (XS) are from interest. For this reason, many research and experience for this particular environment has been gathered. The Norwegian regulations regarding concrete cover requirements are given in the NS 3473: 2003 “Concrete design and detailing rules”. This code prescribes minimum required concrete covers (c_{min}) for the exposures classes which are defined by the EN 1992.

Table C.6 presents the requirements for minimum concrete cover for various exposure classes by the NS 3473. When a structure is designed for a service life of 100 year in marine environments, the minimal cover would amount 60 to 70mm. On top of this, a certain value for deviation in execution (Δc_{dev}) has to be accounted for. The standard value for cover deviation amounts 10mm, which could result in the largest nominal cover of 80mm.

Exposure class	50 year service life		100 year service life	
	Reinforcement slightly sensitive to corrosion	Reinforcement sensitive to corrosion	Reinforcement slightly sensitive to corrosion	Reinforcement sensitive to corrosion
XC1	15	25	25	35
XC2, XC3, XC4	25	35	35	45
XS1, XS2, XD1, XD2, XD3	40	50	50	60
XS3	50	60	60	70

Table C.6. Minimum concrete cover [mm] with respect to corrosion protection (NS 3473: 2003)

Port designer’s Handbook (Thoresen, 2014)

Among other concrete durability recommendations, the Norwegian engineer and author Thoresen (2014) recommends that the concrete cover to the reinforcement in maritime structures should not be less than:

- 50mm above the berth slab;
- 100mm in the splash zone;
- 120mm in the tidal zone;
- 100mm in the submerged zone;

It is also stated that a minimum cover thickness (c_{min}) of 75mm is commonly used for berth structures in Norway. These concrete cover values are based on a design life of 100 years and recommended for *increased security* against chloride penetration. The handbook is thereby cautious on recommendations regarding the cover depth.

The handbook is not a legal document for quay design, but the recommendations threaten future feasibility of the overturning caisson principle for a design life over 50 years.

Handbook of Port and Harbor Engineering (Tsinker, 1997)
From point of structural longevity Tsinker [4] proposes a minimum wall thickness of 300mm for the face walls and base slabs of caissons in marine environments. Concrete cover is advised to be at least 50mm in splash and atmospheric zones. Also the base slab should have this value. For other components, such as inner walls, the concrete cover could be reduced to 30mm, if allowed by recognized codes and / or recommendations.

Member	**Minimum wall thickness [mm]**
Face wall	300
Rear and side walls	200
Internal diaphragm	150
Base slab	300

Table C.7. Minimum wall thickness for seawater conditions proposed by Tsinker (1997)

C.7. Alternative protection measures

The Eurocode 2 allows a reduction to the durability cover if stainless steel or additional protection (e.g. coating or cathodic protection) is applied. The code notes that the value of $\Delta c_{dur,st}$ and $\Delta c_{dur,add}$ for use in a country may be found in its National Annex, but the recommended value, without further specification, is 0 mm.

Stainless steel (SSR) reinforcement
The concrete cover might therefore be reduced for quay wall application, if it is locally prescribed by the National Annex. For instance, the UK National Annex advises that 0 mm reduction is recommended when stainless steel is applied, unless specialist literature justifies a certain reduction. Such specialist literature states that the cover for durability can be relaxed to 30mm where stainless steel is used irrespective of the concrete quality or exposure condition.

Although the term stainless steel might suggest that corrosion is impossible, the passive film which ensures corrosion resistance can still be broken down with degradation as result. The degradation might be negligible, since the passive layer has the ability of re-passivation in particular environments. However, for environments with relatively high chloride contents, re-passivation becomes impossible on which corrosion can progress. This occurs when chloride concentrations becomes higher than the chloride corrosion threshold value. The threshold value mainly depends on the steel-alloy, alkalinity and the ambient temperature.

The corrosion initiation threshold for regular carbon steel reinforcement B500 varies from 0.2% to 2.0% per mass binder. The chloride corrosion threshold for stainless steel is significantly higher and varies from roughly 1% to 7% per mass binder. In terms of durability requirements, the concrete cover ($c_{min,dur}$) could be reduced significantly. The governing values for applying a concrete cover become more or less transmission of bond forces ($c_{min,bond}$) and execution tolerances ($\Delta c_{dev} \approx$ 10mm). Therefore, the total concrete cover could be reduced to approximately 30 to 40mm.

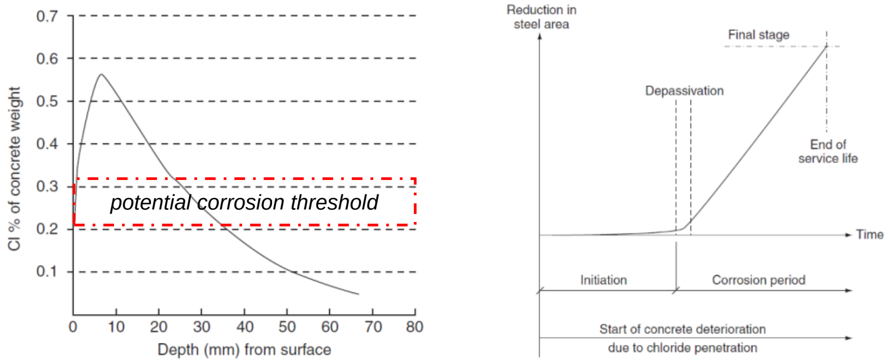


Figure C.4. Typical chloride concentration profiles for marine environments (Thoresen 2003)

These advantages are certainly not free of charge. In comparison with the unit price of regular carbon steel, the price of stainless steel reinforcement is about six to ten times higher (Markeset et al. 2006). Differences are largely depending on bar size and alloy, which can be clearly seen in the table below (Rostam 2000). Besides negative economic aspects, the cover depth is not purely depending on durability requirements. Also the previously discussed mechanical impact and abrasion aspects have to be taken into consideration when designing a reinforced concrete quay structure in a marine environment.

Steel reinforcement type and quality	**Relative cost per unit weight**
Carbon steel B500	1.0
Austenitic 1.4301 / 304	4.5
Austenitic 1.4401 / 316	5.5
Austenitic-Ferritic (Duplex) 1.4462 / 318	5.5 – 6.0

Table C.8. Relative costs of different reinforcement alloys

Fibre reinforced polymer (FRP) reinforcement

The concrete cover could also be reduced by placing fibre reinforced polymer (FRP) reinforcement. FRP reinforcement has generally a similar (or greater) tensile capacity (500 – 900 N/mm^2) compared to regular reinforcement steel. On the other hand, the stiffness is usually lower than that of steel (40.000 – 140.000 N/mm^2), the concrete bond is lower and the material is not ductile[23]. These differences in mechanical properties require drastic changes in the design approach. It is therefore uncertain if the performance of relatively thin walled concrete structures in marine environments can be improved by applying FRP reinforcement. Besides technical obstacles, the initial purchase costs of FRP reinforcement are at least 2 – 3 times higher than carbon steel reinforcement. The relatively high flexible behaviour and higher costs of this material does not seem to result in a durable solution to obtain a relatively thin walled concrete structure.

Durability enhancing measures

Other durability enhancing measures, such as coatings, are not likely to result in a (legally allowed) reduction on the cover. Such measures are more or less an addition to the standard measures and are prescribed if the risk of reinforcement corrosion is likely to be extreme.

At this moment, there is still little information available regarding long-term efficiency of coatings (Thoresen 2014). This, although current experience indicates that the proper application of surface protective coatings can provide valuable advantages. Besides lack of experience, protective coating requires regular maintenance throughout the service life of the structure since there is a probability of de-bonding or peeling off. The combination of little durability knowledge, higher initial costs and higher maintenance costs suggest that a protective coating is economically unfeasible at this moment.

[23]Although new developments might affect the FRP as a material, it does not have yielding characteristics. This does not imply brittle behaviour by definition.

D. Caisson Design Conditions (2017)

D.1. Introduction

This chapter clarifies the design of a quay wall for a sea harbour in the Gulf of Guinea (West-coast of Africa). The quay wall shall function as berthing and mooring facility of a container terminal. The desired quay wall has a length of 1,400 metres and a total retaining height of 21.00m. The proposed quay wall structure concerns a floated-in reinforced concrete caisson.

D.2. Local conditions

D.2.1. Construction site

The construction site is situated approximately 10 kilometre from the desired quay wall. An overview of the locations is given in figure D.1 below.

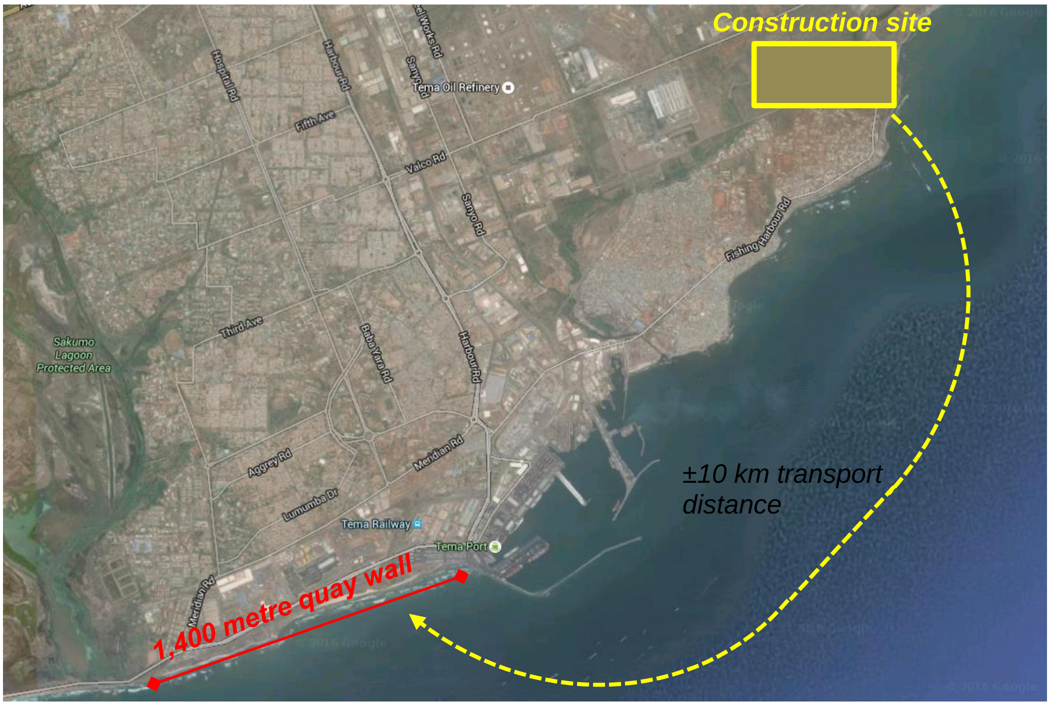


Figure D.1. Distance and transport route from the construction site to the quay wall

D.2.2. Quay geometry

Top of structure:	+4.00m CD
Bottom of structure:	-17.00m CD
Total retaining height (H):	21.00m
Minimum caisson height (h):	18.00m

The cross section of the caisson with backfill is schematically presented below. The superstructure is not indicated and the overall shape might change in order to fulfil strength and/or stability requirements.

D.2.3. Metocean data

The metocean data regarding tidal variations is presented in the table on the next page.

Tides
At the project site, a semi-diurnal M_2 tide prevails. The tidal water levels are as follows:

Abbreviation	Water level	Value
M.H.W.S.	Mean High Water Spring	+1.50 m
M.H.W.N.	Mean High Water Neap	+1.20 m
M.L.W.N.	Mean Low Water Neap	+0.60 m
M.L.W.S.	Mean Low Water Spring	+0.20 m
NLD	National Level Datum	+0.585 m
C.D.	Chart Datum	+0.00 m

D.2.4. Hydraulic conditions

The hydraulic conditions for the project are tabularised below. The high water levels for different return periods are as follows:

High Water Levels (HWL)		
Return period	High Water Level	Remark
< 1 year	1.60m CD	construction level (MHWS + surge)
1 year	2.00m CD	
10 years	2.15m CD	
50 years	2.15m CD	
100 years	2.30m CD	design value; expected sea level rise included

The low water levels are as follows:

Low Water Level (LWL)		
Return period	Low Water Level (LWL)	Remark
100 years	0.00m CD	Chart Datum, design low water level
< 1 years	0.20m CD	MLWS, operational/construction low water level

The significant wave height for various return periods are presented in the tale below.

Significant wave height		
Return period	H_s	Application
< 1 year	0.40m	During transport and immersion
< 5 year	0.70m	During construction
100 years	0.65m	Operational, protected by breakwaters
100 years	1.70m	Extreme case, non-operational

D.2.5. Geotechnical conditions

At surface level, a very loose to loose sand layer is found. These layers have low SPT values, which make the sand liquefiable. Underneath the sand a dense layer of gravel and cobbles is found. The bedrock starts at a level of -14.40 m CD and consists of weathered very weak to weak Gneiss. Locally, the rock transitions into weak to medium strong Gneiss. At -21.00 m CD, weak to medium strong Gneiss is encountered.

The following stratigraphy has been derived from boreholes:

Material	Top of layer [m CD]
(Very) loose sand	-10.30
Dense gravel / cobbles	-13.30
Very weak to weak weathered Gneiss	-14.40
Weak to medium strong Gneiss	-21.00

The soil parameters to be used for the quay wall design are given in the table below.

Material	γ_{Dry} [kN/m^3]	γ_{Sat} [kN/m^3]	φ [°]	C [kPa]	C_u [kPa]
Very loose sand	16	16	27	0	-
Loose sand	16	17	28	0	-
Medium dense sand	18	19	30	0	-
Dense gravel / cobbles	19	20	45	0	-
Hard clay	19	19.5	12	70	250
Very weak to weak weathered Gneiss	19	20	45	0	750
Weak to medium strong Gneiss	20	21	60	250	20,000
Rock fill	16	20	43	0	-

D.3. Operational Requirements

The quay will provide an operational facility for berthing and mooring for the seagoing vessels. The structures shall be designed to provide safe berthing and mooring for the full range of design vessels. A breakwater shall be constructed to minimise downtime corresponding to weather conditions. The proposed port design must be safe to operate and maintain. The marine facilities shall comply with all codes and standards listed under section D.4.

D.3.1. Design life

The design life of the quay wall structure is 50 years. Other components, such as fenders, bollards and scour protection shall be designed for a design life of 25 years.

D.3.2. Design vessels

The port shall be designed to accommodate a range of types and sizes of vessels. The largest ships which must be able to moor are Ultra Large Container Vessels (ULCV) with a weight up to 200,000 DWT and a design draught of 16.00 metres. Other design vessel types and specifications are listed in the table below.

Classification	Early container ships	Panamax	Post Panamax	Mid-size New Panamax	Large New Panamax / ULCV
LOA [m]	100	230	300	350	367
LBP [m]	90	217	285	334	352
Width [m]	18.00	32.00	43.00	50.00	51.00
Draught [m]	6.50	12.50	14.50	15.00	16.00
Displacement [t]	7,700	60,000	120,000	150,000	200,000

D.3.3. Port layout

The finished top level of the quay at cope line shall be at + 4.00 m CD. The cope level shall be constant along its entire length of 1,400m. There shall be no vertical or transverse step in cope line and level between sections of the quay wall.

A Ship to Shore (STS) crane for ultra large container vessels shall be operational at the waterfront of the terminal. The waterside STS crane rail will be positioned at 3.50 metres from the cope line. The quay wall shall be designed for two different rail spans; 18.00 metres and 30.50 metres.

The 600 tonnes mobile crane can be placed randomly between 3.5 m to 50.0 m behind the quay cope line.

From the waterfront crane rail up to a distance of 50 metres from the waterfront, containers and general cargo should be able to be (temporarily) stacked. At a distance of 50 metres, containers shall be stacked up to 4 high.

D.3.4. Serviceability requirements

During quay operations, a maximum vertical displacement of 5 mm is specified. The settlements and displacement criteria only apply to the situation after installation of the crane rails. Settlements which occur prior to the crane rail installation are considered to be recoverable.

D.4. Regulations and verifications

The following guidelines are used for the design of the quay wall structure:

Code / standard	Title
EN 1990 (Eurocode 0)	Basis of structural design
EN 1991 (Eurocode 1)	Action on structures
EN 1992 (Eurocode 2)	Design of concrete structures
EN 1993 (Eurocode 3)	Design of steel structures
EN 1997 (Eurocode 7)	Geotechnical design
EN 1998 (Eurocode 8)*	Design of structures for earthquake resistance
BS 6349-1-1: 2013	Code of practice for planning and design for operations
BS 6349-1-2: 2015	Code of practice for assessment of actions
BS 6349-1-3: 2012	Code of practice for geotechnical design
BS 6349-1-4: 2013	Code of practice for materials
BS 6349 Part 2: 2010	Maritime works –Part 2: Code of practice for the design of quay walls, jetties and dolphins
BS 6349 Part 4: 2014	Maritime structures — Part 4: Code of practice for design of fendering and mooring systems

*Earthquake loads are not (yet) considered

The structure is designed following the Limit State Design approach according to the Eurocode. The Limit states are related to different cases:

- Failure in *transient* situations (construction or transport);
- Failure during operational conditions, the so called *persistent* design situations;
- *Accidental* design situation, which refers to exceptional circumstances;
- *Seismic* design situations, which concerns conditions in which the structure is subjected to seismic events;

The relevant design situations and combinations are selected taking into account the circumstances under which the structure is required to fulfil its function. The considered load cases are based on a level I reliability method, which implies that different partial factors shall be applied for different scenarios. There are several design situations which have to be considered for the caissons.

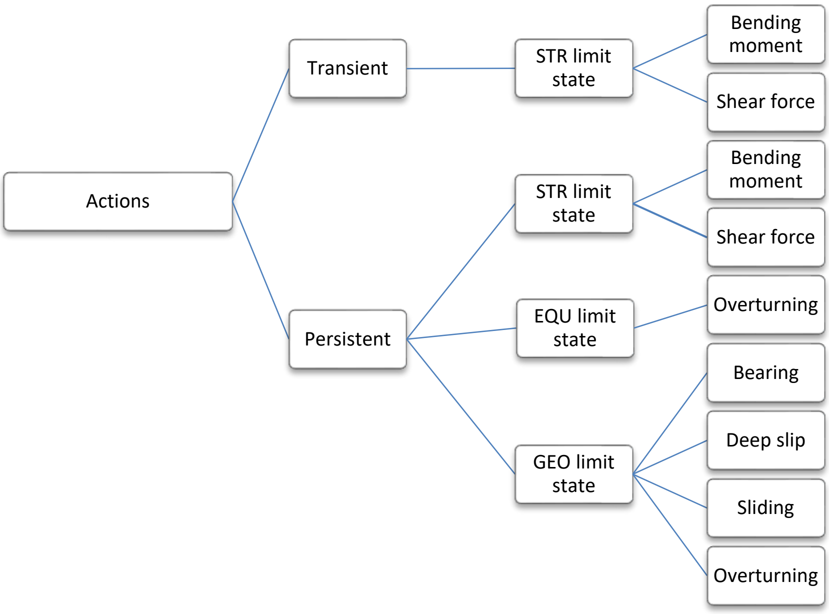


Figure D.2. Characteristic loads and design situations

D.4.1. Transient design situations

The buoyancy capacity of the concrete caissons shall be sufficient during transport and the turning phase. The assistance of any floating case or object is undesired. No bed contact is allowed during the turning operation. Buoyancy loads will include the uplift due to submergence in sea water considering a seawater density of 1030 kg/m^3.

The floated-in-caissons shall have sufficient floating stability during transport without help of sponsons. The metacentric height (GM) shall be at least 1.00 metre, in order to guarantee sufficient transverse floating stability. The range of stability in degrees of heel depends on the geometry of the caisson and self-weight. The downflooding angle must be at least 10 degrees.

Waves during transport are assumed to be at most 0.40m. Considering a partial reflection of 50% of the wave, the minimum freeboard would be 0.40 x 1.50 = 0.60m. The design freeboard is therefore considered to be 1.30 x $H_{refl,d} \approx$ 0.80m.

Floating cranes shall only be used for the assistance of the turning and immersion operation. This is for obtaining a vertical position and gradual immersion.

The hydrostatic pressure will reach its maximum after the turning process. The actual pressure peak depends on the water level inside the compartments. The pressure distribution will be schematized according to the yield line envelope depicted in figure D.3.

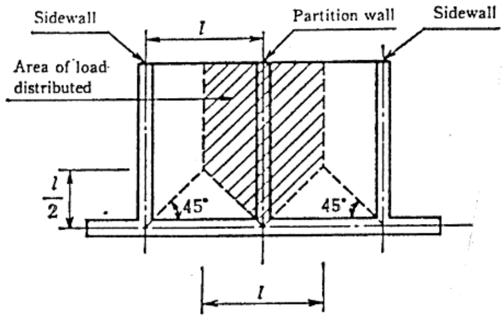


Figure D.3. Pressure distribution of a two way spanning slab (EAU 2012)

Pressures induced during the construction and launching phase are considered to be not governing. It is expected that adjustments can be made in the execution process to mitigate stresses if this becomes necessary.

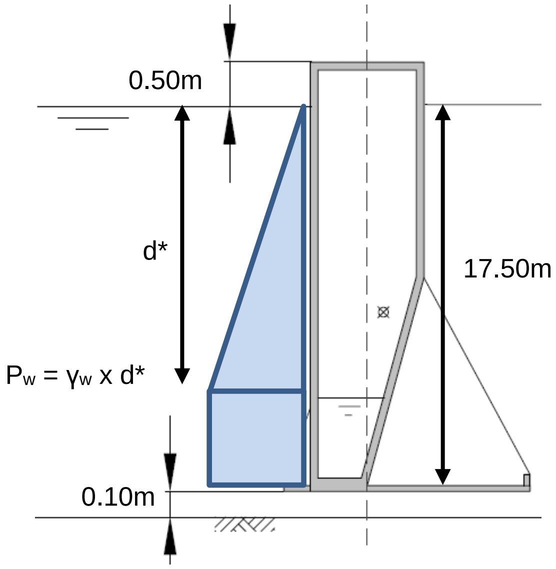


Figure D.4. Pressure on the front wall of the caisson

D.4.2. Persistent design situations

The loads in persistent situations are verified according to the Eurocode 7 (EN-1997), which addresses the following limit states for gravity based structures:

a) Overturning: rigid foundation (EQU)
b) Forward sliding (GEO)
c) Overturning: soil foundation (GEO)
d) Bearing failure (GEO)
e) Ground failure (GEO)
f) Structural failure (STR)

The loads which are from importance for determining the caisson dimensions and cross-sections are:

- Caisson self-weight;
- Superstructure self-weight;
- Water pressure (from outside and inside);
- Soil pressure (external);
- Compartment pressure (Janssen silo pressure);
- Wave loads (modelled with the Sainflou approximation);

The soil pressure around the quay shall be influenced by the following loads on / near the apron:

- Distributed live loads, resulting in additional soil pressure;
- STS crane loads;
- Mobile harbour crane loads;
- Reach stacker and truck loads;
- Foundation pressure;
- Loads induced by an unequal foundation bed;

The considered water level differences are:

Current SLS conditions
- Minimum SLS groundwater = +0.00m CD
- Maximum SLS groundwater = +1.00m CD

SLS conditions after 50 years of sea level rise:
- Minimum SLS groundwater = +0.50m CD
- Maximum SLS groundwater = +1.50m CD

ULS conditions, including 50 years of sea level rise:
- Minimum ULS groundwater = +0.80m CD
- Maximum ULS groundwater = +2.30m CD

The load sensitivity analysis (appendix K) showed that the landside STS-crane loads, loads from the mobile harbour crane, reach stacker and trucks do not influence overall stability significantly. These actions are therefore not considered for stability calculations. The following loads are included for stability verification (GEO and EQU):

- Distributed live loads on top of the quay structure;
- STS-crane loads from the waterside crane track;
- Berthing and mooring loads;
- Hydrostatic water pressure differences;

Load combinations regarding the STS crane track:

- No waterside crane load (decreased downward load);
- Operational waterside crane load (increased destabilizing SLS/ULS comb.);
- Stacked STS crane track load during storm (accidental ULS combination);

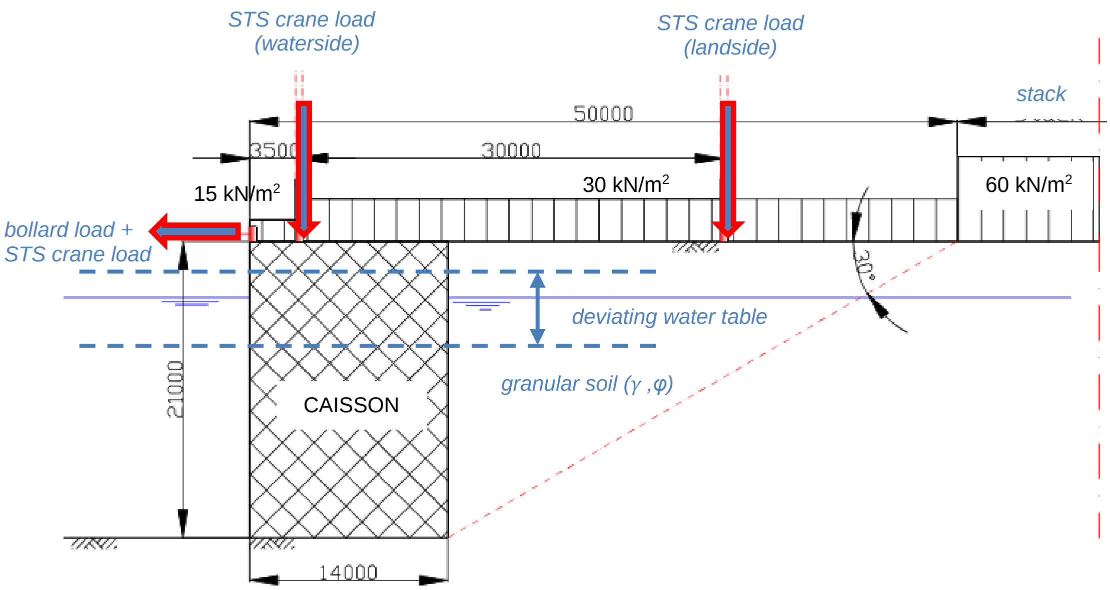


Figure D.5 load overview

For a caisson width of approximately 14 metres, the largest live load (60kN/m^2) from the stack does not influence the stability. The largest live load must only be included for structures wider than 14 meters.

Loads on top of the caisson and its heel have a favourable effect on the stability and are therefore not considered for the overturning failure mechanism. The live load of 30.00kN/m^2 is positioned behind the heel of the caisson for stability calculations.

D.5. Combinations and Factors (BS-6349)

The design loads on the quay wall will be determined in accordance with the Eurocode 0 and 7. In these codes, a distinction has been made between permanent actions (denoted by G) and variable actions (denoted by Q). A partial factor for safety or serviceability is denoted by γ and a combination factor is denoted by ψ.

D.5.1. Combinations of actions (SLS)

The serviceability limit state corresponds to conditions beyond which specified service requirements for the quay structure are no longer met. The Eurocode 0 prescribes that the partial factors for actions should be taken as 1.0 for serviceability limit states, except if differently specified in EN 1991 to EN 1999. The following three design situations shall be considered:

1. Characteristic design situation:

   $$\sum_{j\geq 1} G_{K,j} + Q_{K,1} + \sum_{i>1} \psi_{0,i} Q_{K,i}$$

   The characteristic combination is used for irreversible limit states.

2. Frequent design situation:

   $$\sum_{j\geq 1} G_{K,j} + \psi_{1,1} Q_{K,1} + \sum_{i>1} \psi_{2,i} Q_{K,i}$$

   The frequent combination is used for reversible limit states.

3. Quasi-permanent design situation:

   $$\sum_{j\geq 1} G_{K,j} + \sum_{i\geq 1} \psi_{2,i} Q_{K,i}$$

   The quasi-permanent combination is used for long-term effects and the appearance of the structure.

D.5.2. Combinations of actions (ULS)

The Ultimate Limit State (ULS) is associated with collapse or with other similar forms of structural failure. Regarding this state, the following forms of failure shall be verified:

- Loss of static equilibrium of the structure or any part of it considered as a rigid body (EQU limit state);
- Failure or excessive deformation of the ground (GEO limit state);
- Internal failure or excessive deformation of the structure or structural members (STR limit state);

The design values of actions shall vary for each limit state and shall be in accordance with Eurocode 0. For each critical load case, the design values of actions shall be determined by combining values that are considered to occur simultaneously. The following three design situations shall be considered:

1. Persistent and transient design situation (fundamental combination):

   $$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{K,j} + \gamma_{Q,1} Q_{K,1} + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} Q_{K,i}$$

   For STR and GEO limit states, the fundamental combination of actions can alternatively be the less favourable of the two following expressions:

   $$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{K,j} + \gamma_{Q,1} \psi_{0,1} Q_{K,1} + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} Q_{K,i}$$

   $$\sum_{j\geq 1} \xi_j \gamma_{G,j} \cdot G_{K,j} + \gamma_{Q,1} Q_{K,1} + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} Q_{K,i}$$

Where ξ is a reduction factor for unfavourable permanent actions G and recommended to be equal to 0.85.

2. Accidental design situation:

$$\sum_{j\geq 1} G_{K,j} + A_d + \sum_{i>1} \psi_{2,i} Q_{K,i}$$

In which A_d represents the design value for the accidental action or load.

3. Seismic design situation:

$$\sum_{j\geq 1} G_{K,j} + A_{ed} + \sum_{i>1} \psi_{2,i} Q_{K,i}$$

D.5.3. Combination factors

The partial load factors applicable to the considered limit states are as follows:

Action	Symbol	Description	Partial factor		
			γ_A set A	γ_B set B	γ_C set C
Permanent	G	Dead weight	1.05 / 0.95	1.35 / 0.95	1.00
	G_w	Buoyancy / water pressure	1.05 / 0.95	1.35 / 0.95	1.00
Variable	Q_m	Mooring loads	1.50	1.50	1.30
	Q_B	Berthing loads	1.35	1.35	1.35
	Q_S	Live loads (general cargo)	1.50	1.50	1.15
	$Q_{C,1}$	STS crane loads	1.35	1.35	1.15
	$Q_{C,2}$	Mobile crane loads	1.35	1.35	1.15
	Q_V	Vehicle loads (e.g. reach stackers)	1.35	1.35	1.15
	Q_P	Pedestrian loads	1.35	1.35	1.15
	Q_T	Temperature loads	1.50	1.50	1.20
	Q_{wi}	Wind loads	1.50	1.50	1.30
	Q_{wa}	Wave loads	1.50	1.50	1.30
	Q_{ti}	Tidal lag	1.50	1.50	1.30

The partial factors for set B and set C are used for verifying STR and GEO limit states only. The partial factors are thereby based on the BS 6349-1-2 and deviate from the Eurocode 7. These partial factors are only considered for the fundamental load combination. For accidental and seismic load cases, the partial factors are equal to 1.00.

Similar symbols are used for further calculations. Corresponding subscripts are used when for instance a pressure (P) is considered caused by a vertical load (Q).

The combination factors applicable to the considered limit states are as follows:

Action	Symbol	Description	Combination factor		
			ψ_0	ψ_1	ψ_2
Permanent	G	Dead weight	-	-	-
	G_w	Buoyancy / water pressure	-	-	-
Variable	Q_m	Mooring loads	0.50	0.20	0
	Q_B	Berthing loads	0.75	0.75	0
	Q_S	Live loads (general cargo)	0.70	0.50	0.30
	$Q_{C,1}$	STS crane loads	0.75	0.75	0
	$Q_{C,2}$	Mobile crane loads	0.75	0.75	0
	Q_V	Vehicles (e.g. reach stackers)	0.75	0.75	0
	Q_P	Pedestrians	0.40	0.40	0
	Q_T	Temperature loads	0.60	0.60	0.50
	Q_{wi}	Wind loads	0.77		
	Q_{wa}	Wave loads	0.60	0.20	0
	Q_{ti}	Tidal lag	0.60	0.20	0

In which:
ψ_0 = factor for the combination value of a variable action
ψ_1 = factor for the frequent value of a variable action
ψ_2 = factor for the quasi permanent value of a variable action

Hence;

Design situation	Variable actions	Limit State	Load situation	Example
Characteristic	$\psi_0\ Q_k$	ULS	Persistent	Normal use / operational conditions
		ULS	Transient	During construction, transport or repair
		SLS	Irreversible limit states	Yield stress reinforcement
Frequent	$\psi_1\ Q_k$	ULS	Accidental	Ship impact
		ULS	Seismic	Earthquake
		SLS	Reversible limit states	Crack-width
Quasi permanent	$\psi_2\ Q_k$	ULS	Accidental	Ship impact
		ULS	Seismic	Earthquake loading
		SLS	Long-term-effects	Crack-width

The main focus of this feasibility study is the characteristic load situations and therefore combination factor ψ_0.

D.5.4. Partial factors (EQU)

The partial factors for EQU verification should only be used in combination with the fundamental combination prescribed by the Eurocode 1990, Eq. 6.10, set A. The partial material factors for soil parameters applicable to the EQU limit state are as follows:

Soil parameter	Symbol	Value
Angle of shearing resistance*	γ_{Φ}	1.25
Effective cohesion	$\gamma_{c'}$	1.40
Undrained shear strength	γ_{cu}	1.40
Weight density	γ_{γ}	1.00
*factor applied to tan φ		

D.5.5. Partial factors (STR and GEO)

For the STR and GEO limit states, prescribed design approach 2 (see EC7: annex B), will be used for quay design. It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: A1 “+” M1 “+” R2

In which:
A1: Actions limit state set A1
M1: Material limit state set M1
R2: Resistance limit state set R2
“+”: implies: to be combined with

In this approach, partial factors are applied to actions or to the effects of actions (A1) and to ground resistances (R2). The material parameters (M1) are kept unfactored (γ equal to 1.0).

The considered partial factors on actions are based on the BS 6349-1-2. The resistance factors applicable to the GEO limit states are:

Resistance	Symbol	Value (set R2)
Bearing	$\gamma_{R,v}$	1.40
Sliding	$\gamma_{R,h}$	1.10

The partial resistance factors applicable to the STR limit states are:

Resistance	Symbol	Concrete	Reinforcement
Persistent and transient	$\gamma_{c,1}$ / $\gamma_{s,1}$	1.50	1.15
Accidental	$\gamma_{c,2}$ / $\gamma_{s,2}$	1.20	1.00
Seismic	$\gamma_{c,3}$ / $\gamma_{s,3}$	1.50	1.50

D.6. Building materials

The structural design of the reinforced concrete caisson will be in accordance to the Eurocode 2 (EN-1992).

D.6.1. Reinforced concrete

For durability, maintenance and executional aspects, the following requirements of caisson geometry are specified:

- The total height of the caisson shall be at least 18.00 metres;
- The internal walls have a minimum thickness of 250mm;

These values are not based on structural limit states which have to be considered as well. This could result in higher values than these minima prescribed in this section.

An assessment has been done to find the characteristics of the materials that will be used for the quay wall. During this assessment, the right balance between the required strength, durability, workability and economics has been found. The results and material characteristics are tabularised below.

Material	**Grade**
Concrete quality	C35/45
Concrete weight	25.00 kN/m^3
Cement	CEM II or CEM III
Cement content	360 kg/m^3
Water / cement ratio	≤ 0.45
Reinforcement steel	B500B or B500C
Nominal concrete cover	60 mm

The exposure classes of the quay structure are listed below. The nominal concrete cover shall be at least 60mm, independent of the exposure class.

Exposure classes related to environmental conditions in accordance with EN 206-1	**Class designations**
Front of structure above chart datum	XM1, XA2, XC4, XS3
Front of structure below chart datum	XA2, XC1, XS2
Back of structure above chart datum	XA2, XC4, XS3
Back of structure below chart datum	XA2, XC1, XS2

D.6.2. Compartment fill

The compartments shall be filled by a granular material such as sand. No cementitious mixtures are applied in the compartments. It is assumed that the compartment fill has a dry weight of γ_{Dry}= 18.00 kN/m^3 and a saturated weight of γ_{Sat}= 20.00 kN/m^3. The weight variation within granular fills is considered to be insignificant, unless it is explicitly desired during design.

D.6.3. Backfill materials

The material used as backfill can either be reused from dredging works (sand) or it can especially be procured (rubble stones). The dredged material is therefore much cheaper, but consists of a lower soil shearing angle. The most economical solution can be found by considering different designs. The two considered scenarios are:

1. Caisson quay wall in combination with a sand backfill (ϕ = 30°)
2. Caisson quay wall in combination with rubble stones (ϕ = 45°)

D.6.4. Overview

Design parameters	Symbol / notation	Value
Geometry		
Height	H	21.00 m
Depth	D	17.00 m
Ground level	GL	+4.00m CD
Hydraulic		
Unit weight sea water	γ_w	10.30 kN/m^3
Soil		
Unit weight dry granular soil	$\gamma_{s,dry}$	18.00 kN/m^3
Unit weight of wet soil	$\gamma_{s,wet}$	20.00 kN/m^3
Angle of shearing resistance of sand backfill	ϕ (= ϕ')	30°
Angle of shearing resistance of rubble backfill	ϕ (= ϕ')	45°
Friction angle	δ (= 2/3 ϕ)	23° / 30°
Structural		
Concrete strength class / grade		C35/45
Characteristic compressive strength after 28 days		35 N/mm^2
Concrete weight (incl. reinforcement)		25.00 kN/m^3
Reinforcement		B500B
Reinforcement steel design yield stress	$f_{yd} = f_{yk}/\gamma_s$	500/1.15 = 435 N/mm^2
Durability and maintenance		
Design life		50 years
Nominal concrete cover	c_{nom}	60mm
Exposure classes above CD		XC4, XS3
Exposure classes below CD		XC2, XS2
Dimensions		
Thickness compartments		250mm
Thickness walls		500mm
Thickness base slab		600mm
Compartment dimensions (rect. caissons)		3.50 x 3.50m
Compartment dimensions (overt. caissons)		3.50 x 6.50m

E. Persistent loads

E.1. Distributed live loads

The distributed live loads apply to all free areas of platform or decks. The structure shall be designed to resist (at least) the loads which are indicated in the table below.

Location	**Item**	**Value**
In front of waterside crane rail	Q_{s1}	15 kN/m^2
Quayside operations	Q_{s2}	30 kN/m^2
Storage area	Q_{s3}	60 kN/m^2

The prescribed live load of 60 kN/m^2 acts on a distance (L_{st}) of 50.00 metre from the waterfront. In combination with an internal friction angle of φ = 30 degrees, the live load of 60 kN/m^2 would only be from importance if the caisson width would be larger than:

$$\tan(\varphi) = \frac{H}{L - W_{c,\max}}$$

in which:

$\varphi = 30°$

$H =$ retaining height = 21.00m

$L =$ distance from stack to waterfront = 50.00m

$W_{c,\max} =$ largest caisson width which is not affected by the stack load

$$W_{\mathrm{c,max}} = L - \frac{H}{\tan(\varphi)} = 50.00 - \frac{21.00}{\tan(30)} = 13.60m$$

Quay wall structures smaller than 13.60 metre are therefore outside the influence zone of the largest load. The live load induced by the storage facility is therefore not considered in further design calculations, on the condition that a soil with relatively large internal friction angles will be applied (φ ≥ 30°).

The live load Q_{s1} = 15 kN/m^2 in front of the waterside crane rail acts up to 3.50 metre from the waterfront. The remaining quay structure shall be designed to resist at least the live load Q_{s2} = 30 kN/m^2.

The horizontal soil pressure induced by distributed loads is calculated with the active/neutral and fully neutral soil pressure coefficient. In example, the following pressure would occur on the caisson if a neutral state is considered:

$P_{S,n} = K_n \times Q_v = 0.50 \times 30.00 = 15.00$ kN/m^2

Horizontal thrust due to vertical quay load:
$F_{S,n} = P_{Q,n} \times H = 15.00 \times 21.00 = 315$ kN

Acting moments due to vertical quay load:
$M_{S,n} = ½ \times P_{Q,n} \times H^2 = 0.5 \times 15.00 \times 21.00^2 = 3{,}308$ kNm

The neutral soil pressures originating from live loads are added to the neutral soil pressures from the backwall. Similarly, an active pressure from live loads is added to the active soil pressure from the backwall itself. Partial factors are not applied on the neutral soil pressure state, they are only used in combination with the active state.

The pressures from live loads vary with the geometry of the caisson. Live loads are considered to act from behind the caisson. Due to a heel, the live load acts up to a certain height of the retaining structure. Actions on a particular distance from the retaining wall are presented in figure E.1.

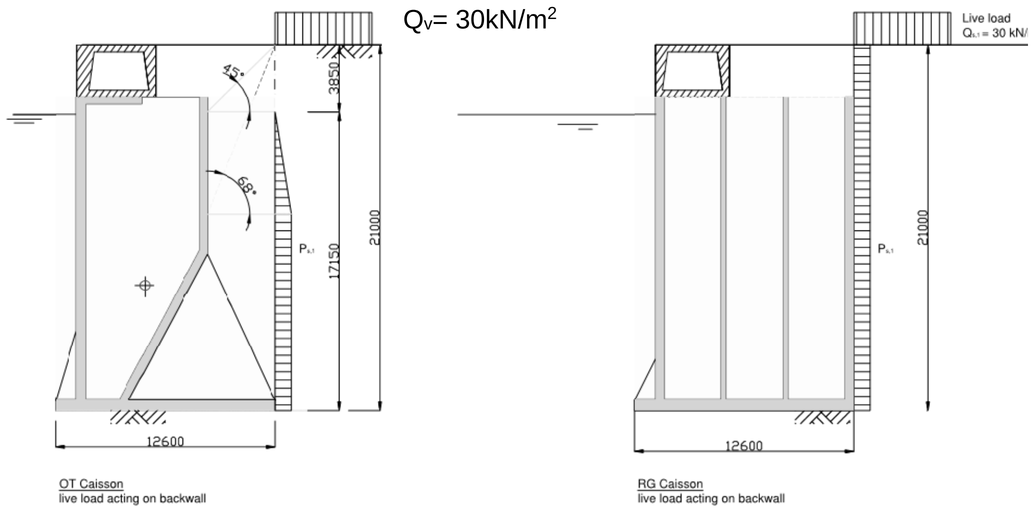


Figure. E.1. Live load actions on retaining wall

The following loads per running metre quay wall are obtained from spreadsheets:

Live loads for all caisson designs					
Soil pressure state	Friction angle	Notation	Horizontal thrust	Notation	Destabilizing moment
Neutral	$\phi = 30°$	$F_{n,Q,1}$	315 kN	$M_{n,Q,1}$	3,308 kNm
Neutral	$\phi = 45°$	$F_{n,Q,2}$	183 kN	$M_{n,Q,2}$	1,918 kNm

Live loads for overturning caissons					
Soil pressure state	Friction angle	Notation	Horizontal thrust	Notation	Destabilizing moment
Active	$\phi = 30°$	$F_{a,Q,1}$	241 kN	$M_{a,Q,1}$	2,000 kNm
Active	$\phi = 45°$	$F_{a,Q,2}$	115 kN	$M_{a,Q,2}$	794 kNm

Live loads for rectangular caissons					
Soil pressure state	Friction angle	Notation	Horizontal thrust	Notation	Destabilizing moment
Active	$\phi = 30°$	$F_{a,Q,1}$	176 kN	$M_{a,Q,1}$	1,851 kNm
Active	$\phi = 45°$	$F_{a,Q,2}$	89 kN	$M_{a,Q,2}$	930 kNm

E.2. Tidal water pressures

Design for SLS conditions is based on tidal variation from highest astronomic tide to lowest astronomic tide, but taking into account tidal lag, drainage issues and the effects of fresh water flow plus any other known contributors[1].

The tidal variations in SLS conditions are assumed to be 1.00 metre. This results in a maximum pressure difference of ($P_{tidal,SLS}$) 10.30 kN/m^2. This is equal to a head difference (Δh) of approximately 2/3 of the tidal range.

- Minimum SLS sea water level = +0.00m CD
- Maximum SLS sea water level = +1.00m CD

This value is rather conservative compared to empirical data and numerical calculations. Field measurements[24] showed that the hydrostatic load generally equals about one-third of the tidal range above the low water level. From numerical calculations based on Darcy's model for groundwater flow[25], it is obtained that the water level variation behind a caisson quay structure can be larger than half the tidal difference.

Design for ULS conditions shall be based on extreme water levels on both sides of the structure, with the astronomic tide level considered as temporary loading and the other elements as transient loads.

- Extreme ULS low sea water level: +0.80m CD
- Extreme ULS high sea water level: +2.30m CD

The extreme low water level is thereby considered to be higher than chart datum. This is because the extreme high water level is based on a model in which sea level rise (+0.50m) and extreme surge (+0.30m) are included.

Groundwater table
The considered groundwater (GW) tables are identical to the extreme water levels at sea, which are:

- Minimum SLS groundwater = +0.00m CD
- Maximum SLS groundwater = +1.00m CD
- Minimum ULS groundwater = +0.80m CD
- Maximum ULS groundwater = +2.30m CD

These groundwater tables (GW) are considered from behind the backwall and from within the compartments of the caisson. The soil pressure is considered differently for each limit state. The maximum water pressure is γ_w times the water level difference.

$P_{ti,SLS} = \gamma_w \times \Delta h = 10.30 \times 1.00\text{m} = 10.30 \text{ kN/m}^2$
$P_{ti,ULS} = \gamma_w \times \Delta h = 10.30 \times 1.50\text{m} = 15.45 \text{ kN/m}^2$

The horizontal thrust is approximately:
$F_{ti,SLS} \approx P_w$ x (depth + groundwater table) = 10.30 x (17.00 + 0.5 x 1.00) = 180 kN
$F_{ti,ULS} = P_w$ x (depth + groundwater table) = 15.50 x (17.00 + 0.5 x 2.30) = 287 kN

The destabilizing moment is:
$M_{ti,SLS} = F_w$ x ½ (depth + groundwater table) = 180 x ½ (17.00 + ⅓ x 1.00) = 1,562 kNm
$M_{ti,ULS} = F_w$ x ½ (depth + groundwater table) = 287 x ½ (17.00 + ⅓ x 2.30) = 2,546 kNm

E.3. Horizontal STS-crane loads

The waterside crane track is positioned at 3.50 metre from the waterfront. Therefore, it is assumed that the waterside crane track is founded on the superstructure of the quay wall and transfers its loads through the structure.

The maximum horizontal load exerted by the STS-crane during operational (SLS) conditions amounts 79.25 kN/m^1. The maximum horizontal load exerted by the STS-crane in stowed ULS conditions amounts 198.50 kN/m^1.

The STS-crane load results in a maximum horizontal force perpendicular to the quay of:

$F_{C,1,,SLS} = 79.25$ kN/m
$F_{C,1,,ULS} = 198.50$ kN/m

[24]Furudoi, T. and Katayama, T.,1971. "*Field Observation of Residual Water Level"* Technical Note of PHRI, No. 115, Japan.
[25] Internal reference: Westerschelde Container Terminal (2001) Ontwerpbasis Zeekade Concept Caissons

The maximum destabilizing moment per running metre quay becomes:

$M_{C,1,SLS} = H \times F_{C,SLS} = 21.00 \times 79.25 = 1{,}664$ kNm
$M_{C,1,ULS} = H \times F_{C,ULS} = 21.00 \times 198.50 = 4{,}169$ kN/m

E.4. Vertical STS-crane loads (line loads)

The design quay STS crane for ultra large container vessels has a weight of 20,000 kN. The considered maximum wheel and rail loads are as shown in table X. These loads correspond to 30.00m rail gauges.

Description	**Item**	**Value**
Crane weight	F_{s1}	20,000 kN
Crane jack up load per corner	F_{s2}	7,000 kN
Maximum wheel load	F_{s3}	1,620 kN
Equivalent vertical line load	$Q_{c,serv,1}$	1,100 kN/m (operational)
	$Q_{c,serv,2}$	1,200 kN/m (stowed)
Horizontal wheel load perpendicular to the rail	$Q_{c,serv,Hq}$	107 kN (normal) 268 kN (storm)
Horizontal wheel load parallel to the rail	$Q_{c,serv,Hr}$	331 kN (normal) 619 kN (storm)

The line loads provided above correspond to the maximum loads on an individual bogie. When two cranes are considered next to each other the combinations of loads will be assumed. The wheel configuration of the crane is depicted below.

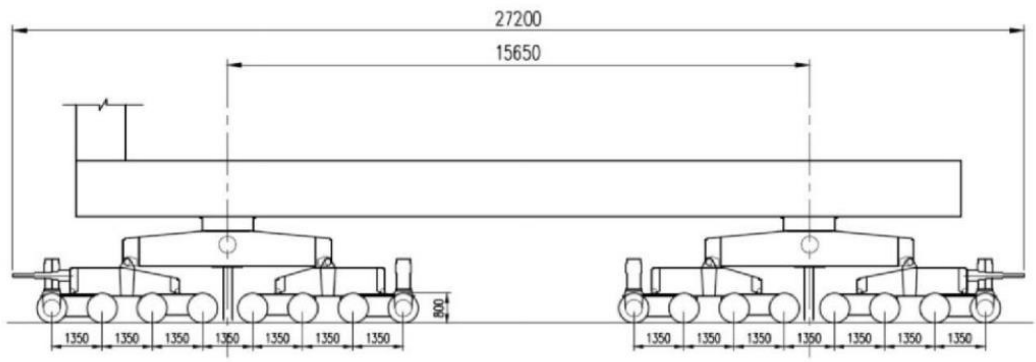


Figure E.2. STS Crane wheel configuration

The design quay STS crane for ultra large container vessels has a weight of 20,000 kN. During operation, the crane bogies induce a maximum line load of 1100 kN/m[1], in combination with a horizontal wheel load of 107 kN. The centre to centre distance of the crane wheels are 1.35 metre. The equivalent horizontal line load therefore amounts: 107 / 1.35 = 79.25 kN/m[1].

During storm conditions, the crane shall be stowed. For this case, the crane bogies induce a maximum line load of 1,200 kN/m[1], in combination with a horizontal load of 268 kN wheel load perpendicular to the quay. The equivalent horizontal line load therefore amounts: 268 / 1.35 = 198.50 kN/m[1]. The horizontal wheel loads under storm conditions are exceptionally high. Therefore these loads under storm conditions are considered as accidental load case.

The line loads provided above correspond to the maximum loads on an individual bogie. When two cranes are considered next to each other the combinations of loads will be assumed. The length of one STS crane bogie amounts approximately 10 metres. If two bogies are positioned next to each other, the maximum total length amounts approximately 20 metres.

Governing vertical line load
The induced ship-to-shore (STS) crane loads are schematized as line loads along the quay. The waterside crane rail is supposed to transfer its load directly to the (super)structure.

The landside crane rail is positioned at a distance of 33.50 metre from the waterside. The crane can thereby induce stresses on the back-wall of the structure. The landside STS-crane load results in a horizontal load on the quay wall.

Based on elastic theory, the vertical and horizontal stresses on an arbitrary distance from the line loads can be calculated with help of the derivation of Flamant (1892). This derivation is equivalent to the Boussinesq problem from 2 dimensional perspective. The basic principle is presented in figure E.3.

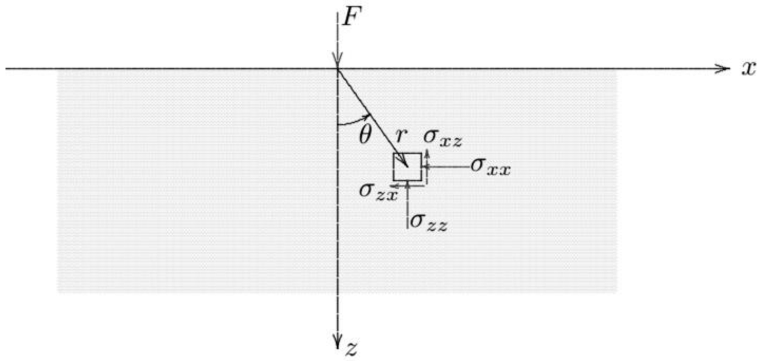


Figure E.3. Boussinesq 2 demensional stress state

The applied quantity F represents a line load with the dimension of a stress. The earth stresses at any point can be found by:

$$\sigma_{zz} = \frac{2F}{\pi}\frac{z^3}{r^4} = \frac{2F}{\pi r}\cos^3\theta$$
$$\sigma_{xx} = \frac{2F}{\pi}\frac{x^2 z}{r^4} = \frac{2F}{\pi r}\sin^2\theta\cos\theta$$
$$\sigma_{xx}(\theta) \propto \sin^2\theta\cos\theta$$

Obviously, the earth pressure acting on the back-wall depends on the position of the wall in relation to the crane track. Assuming a position of the back-wall on 10 metres from the waterfront, the remaining distance from the line load would be 23.50 metres.

Especially the horizontal earth pressure is from interest in terms of the design of the soil retaining element. For this design, the values of r and θ determine the quantity of the load. The proportionality in relation to θ can be expressed as:

$$\sigma_{xx}(\theta) \propto \sin^2\theta\cos\theta$$

Neglecting the effect of radius (r), which could slightly influence the maximum pressure depth, the maximum value can be obtained by setting the derivative equal to zero:

$$\frac{d}{d\theta}\sigma_{xx} = 2\sin\theta\cos^2\theta - \sin^3\theta = 0$$

$$\frac{1}{2}\sin^2\theta = \cos^2\theta$$

$$\frac{1}{2}\tan^2\theta = 1$$

$$\tan\theta = \sqrt{2}$$

$$\theta_{\max} \simeq 60.8°$$

The equivalent triangle dimensions are 1-$\sqrt{2}$-$\sqrt{3}$, from which follows;

$$r = \sqrt{\frac{3}{2}} \cdot x = \sqrt{\frac{3}{2}} \cdot 23.50m$$

The ship-to-shore crane can exert a significant pressure on the quay wall through the landside crane rail. The maximum pressure acts at a depth of roughly 23.50 / √2 ≈ 16.6 metres. The maximum pressure depth is equal to +4.00 – 16.60 = -12.60m CD.

Using the maximum SLS load of 1,100 kN/m^1, a resultant angle of 60.8 degrees and a radius (r) of 23.50 x √3/2, the following horizontal soil pressure can be obtained:

$$\sigma_{xx} = \frac{2F}{\pi}\frac{x^2 z}{r^4} = \frac{2F}{\pi r}\sin^2\theta\cos\theta$$

$$\sigma_{xx} = \frac{2 \cdot 1100}{\pi \cdot \sqrt{\frac{3}{2}} \cdot 23.50}\sin^2(60.8)\cos(60.8) = 9.40 \text{ kN/m}^2$$

Using the maximum ULS load of 1,200 kN/m^1, a resultant angle of 60.8 degrees and a radius (r) of 23.50 x √3/2, the following horizontal soil pressure can be obtained for ULS conditions:

$$\sigma_{xx} = \frac{2F}{\pi}\frac{x^2 z}{r^4} = \frac{2F}{\pi r}\sin^2\theta\cos\theta$$

$$\sigma_{xx} = P_{C,1} = \frac{2 \cdot 1200}{\pi \cdot \sqrt{\frac{3}{2}} \cdot 23.50}\sin^2(60.8)\cos(60.8) = 13.95 kN / \text{m}^2$$

These pressures occur not over the full depth of the quay. Considering the relatively small values, these pressures are neglected for overall stability verifications. These pressures are therefore only taken into account for wall design (STR).

E.5. Vertical point loads

Mobile harbour crane loads

The design mobile harbour crane is a LHM600, GHMK 8410 or equivalent. The design loads for the mobile crane, concerning a LHM600 crane, are presented below. The standard pad dimensions are 5.50 m x 1.80 m.

Load case	**Load value**	
Mobile crane weight	6,000 kN	
Maximum axle load	600 kN	
Equivalent vertical pad load	**Max. pad load**	**Opposite pad load**
Out of service crane pad load	1,390 kN	1,390 kN
Operational load	3,710 kN	465 kN

Reach stacker and truck loads

Various vehicles must be able to operate at any place near the waterfront. The highest axial loads are:

- The axial load of a reach stacker is at most 1,000 kN per axle (4 wheels).
- The axial load of a truck is at most 195 kN per axle (4 wheels).

The effective wheel pressure of both vehicles are at most 1,100 kN/m^2

Governing vertical point loads

The design mobile harbour crane is a 600 tonnes crane. This crane is considered to induce the governing vertical point loads. The maximum outrigger pad load amounts 3,710 kN and the standard pad dimensions are 5.50 m x 1.80 m. The total crane weight equals 6,000 kN.

Other vertical loads, for example originating from reach stackers and truck loads, are significantly smaller. The contact area for these vehicles is also smaller, which can theoretically result in higher pressures. Nevertheless, these small contact areas are assumed to be smeared out by an asphalt or other surface layer. The load distribution of the smaller vehicles is presented in figure E.4.

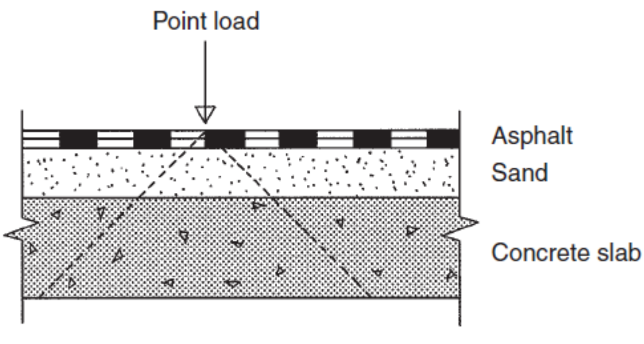


Figure E.4. – Point load distribution for vehicles at the apron

The mobile harbour crane, is schematized as being a small area load which has the following load characteristics:

mobile crane self-weight	max. static outrigger reaction	pad dimensions	max. equally distributed pad pressure ($\sigma = F/A$)
6,000 kN	3,710 kN	5.50 x 1.80m	375 kN/m^2

The horizontal soil pressure induced by this load is calculated by applying equations evolved by Boussinesq (1885) and Fadum (1948). The equations are based on a homogeneous isotropic linear elastic soil and can be used to determine stress components that act at a point below a surface.

In the worst scenario, the crane is operated with its pads perpendicular to the quay wall and just next to the quay wall. In case of a caisson, the back-wall would possibly be subjected to a part of the pad load. The figure below shows points of equal vertical pressure on a cross-section through the foundation.

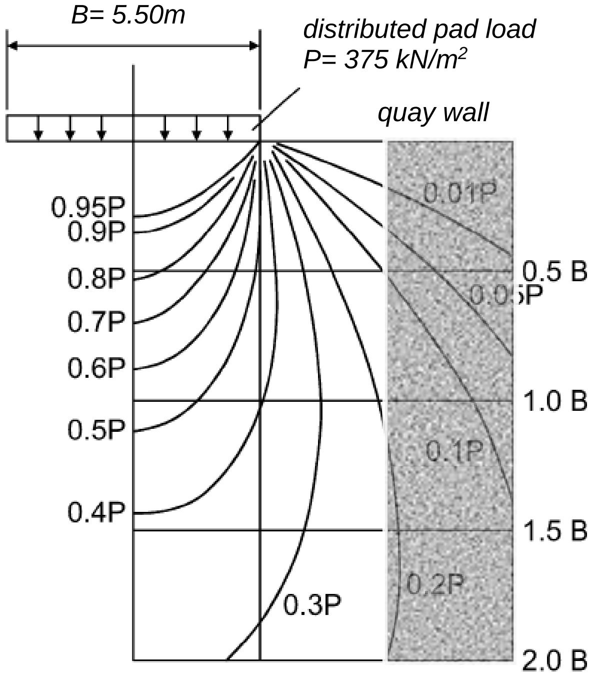


Figure E.5. Pressure bulb

This implies that the largest dimension of the pad, which is 5.50 metre, might induce an earth pressure on the quay. The pressure can be significant up to a depth of roughly 2.0B, which is equal to 11.00 metres.

The pad load could theoretically induce a vertical soil pressure of approximately 0.3P to the retaining structure:

$$\sigma_{xx} = 0.2 \cdot \sigma_{zz} = 0.2 \cdot 375 = 75 \text{ kN/m}^2$$

This pressure is conservative since the pad pressure is smeared out through the asphalt layer. Nevertheless, it indicates the value of a decent superstructure and back-wall.

E.6. Loads from ships

Mooring loads
The mooring load applies within the following minimum angular range for mooring lines:

- vertical angle: -30° to +60° relative to the horizontal plane;
- horizontal angle: -90° to +90° relative to the vertical plane;

The spacing between the bollards within a twin set is 3.0m. The centre to centre distance between twin sets of bollards amounts 25.00 metre. The following loads are considered for the design:

Load case	Mooring load
Operational mooring	2,000 kN
Accidental mooring	2x 1,500 kN

The mooring loads are considered to act at a height of 0.50 metre above the coping beam. The height relative to the chart datum amounts +4.50 metre.

The mooring load shall be schematized as equally distributed force over the quay length. This is an ideal situation which might not be realistic. The calculated forces and moments are therefore only for preliminary design purposes only.

The bollard force results in a maximum horizontal force perpendicular to the quay of:

$F_{m,SLS}$ = 2,000 kN / 25m = 80 kN/m^1
$F_{m,ULS}$ = 3,000 kN / 25m = 120 kN/m^1

Assuming that the mooring lines act on +0.50 metre from ground level. The moment acting from the bottom of the structure becomes:

$M_{m,SLS} = F_B$ x (21.00 + 0.50) = 1,720 kNm
$M_{m,ULS} = F_B$ x (21.00 + 0.50) = 2,580 kNm

Berthing loads
A fender system will transmit a certain force to the structure. The quantity of the reaction force depend on fender and ship characteristics. In order to design a quay with adequate loading capacity to resist typical berthing forces, the structural components shall be designed in such a way that compression load equal to the bollard force can be applied at any point.

E.7. Wave loads

The quay wall is situated in a sheltered environment. Therefore, only non-breaking waves shall be considered. The reflection by non-breaking waves shall be calculated by the Sainflou approximation for pressures over the height of the wall. The waves are thereby assumed to act perpendicular to the wall in combination with 100% reflection. Note that the degree of reflection actually depends on the properties of the reflecting edges (angle, roughness, porosity, etc.) and the water depth in front of the structure.

A Rayleigh distribution shall be assumed for the local wave spectrum. The design wave height (H_d) can therefore be determined from the significant wave height $H_d \approx 1.87\ H_s$.

The pressure of non-breaking waves is determined with the following parameters:

The design wave height: $H_{in} \approx 1.87\ H_s$
The reflected wave height: $H_{refl} \approx 2\ H_d$
The maximum wave pressure: $P_1 = (H_{in} + h_0)\ \times \gamma_w$

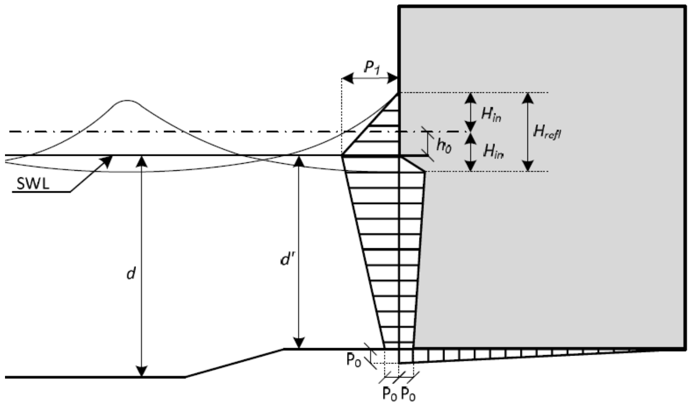


Figure E.6. – Sainflou approximation

The pressure at the bottom of the caisson becomes:

$$P_0 = \frac{\rho \cdot g \cdot H_{in}}{\cosh(k \cdot d')} = \frac{\rho \cdot g \cdot H_{in}}{\cosh((2\pi/L_0) \cdot d')}$$

in which:

$d' =$ water depth abve foundation level (17 metre)

$k =$ wave number incoming wave $(2\pi/L_0)$

$L_0 \approx T \cdot \sqrt{g \cdot d} = 6 \cdot \sqrt{9.81 \cdot 17} = 77.5$m (wave period varies between 6 and 8 seconds)

$$P_{0,SLS} = \frac{1030 \cdot 9.81 \cdot 1.87 \cdot 0.65}{\cosh((2\pi/77.5) \cdot 17.00)} = 5{,}819 \text{ Pa} = 5.82 \text{ kN/m}^2$$

$$P_{0,ULS} = \frac{1030 \cdot 9.81 \cdot 1.87 \cdot 1.70}{\cosh((2\pi/77.5) \cdot 17.00)} = 15{,}219 \text{ Pa} = 15.22 \text{ kN/m}^2$$

The still water level in front of the caisson will increase by water level h_0:

$$h_{0,SLS} = \frac{1}{2} \cdot k \cdot H_{in}^2 \cdot \coth(k \cdot d) = \frac{1}{2} \cdot (2\pi / L_0) \cdot (1.87 \cdot 0.65)^2 \cdot \coth((2\pi / L_0) \cdot 17.00) = 0.13 \text{ m}$$

$$h_{0,ULS} = \frac{1}{2} \cdot k \cdot H_{in}^2 \cdot \coth(k \cdot d) = \frac{1}{2} \cdot (2\pi / L_0) \cdot (1.87 \cdot 1.70)^2 \cdot \coth((2\pi / L_0) \cdot 17.00) = 0.86 \text{ m}$$

The pressure peak from non-breaking waves becomes approximately:

$P_{wa,SLS} = (1.87 \times H_s + h_0) \times \gamma_w = (1.87 \times 0.65 + 0.13) \times 10.30 = 13.86$ kN/m^2

$P_{wa,ULS} = 1.87 \times H_s \times \gamma_w = (1.87 \times 1.70 + 0.86) \times 10.30 \approx 41.60$ kN/m^2

F. Overturning Caisson Design (12.60m)

F.1. Geometry

The geometry of the caisson is depicted below; the upper drawing shows a transverse cross-section and the lower drawing a horizontal cross-section.

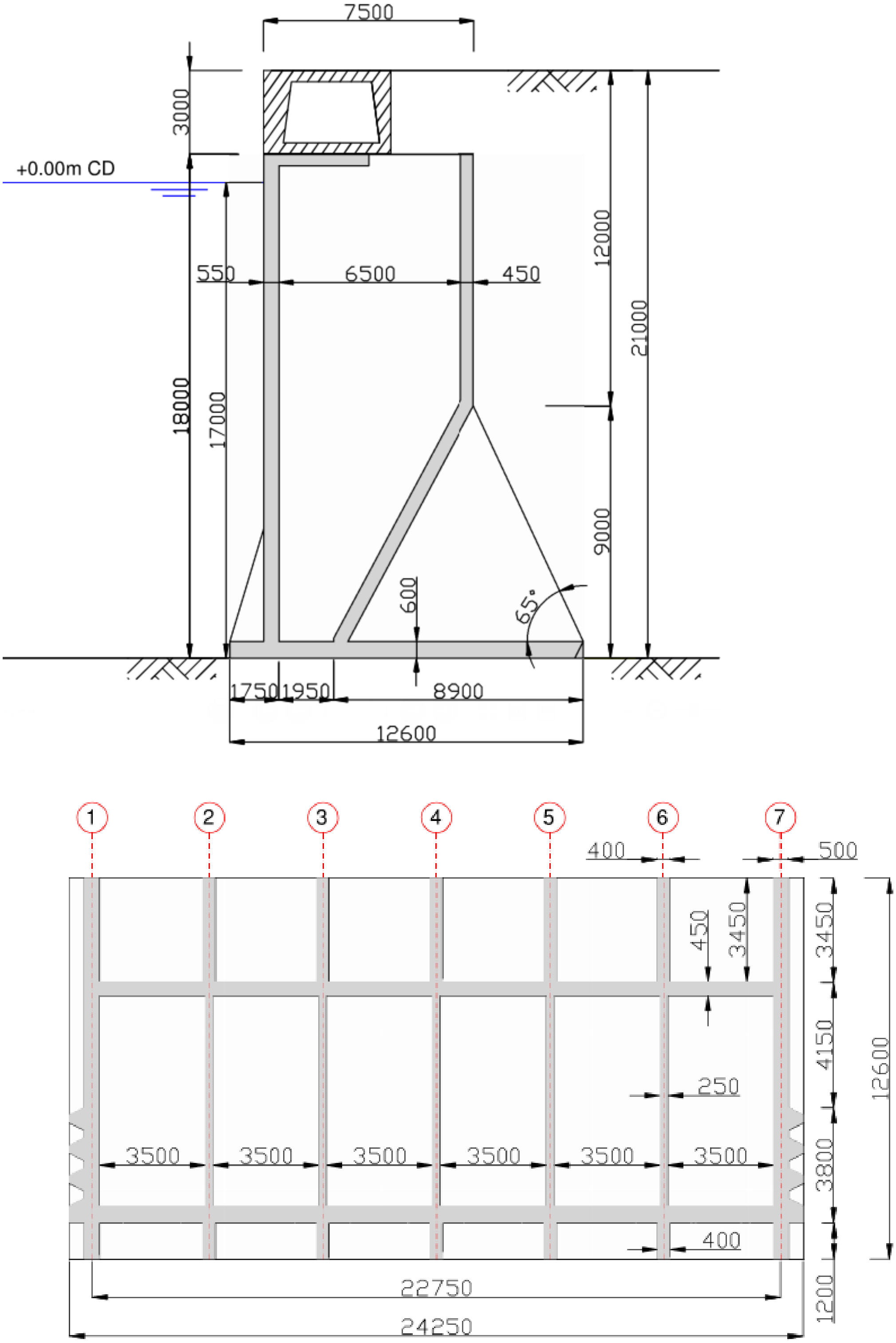


Figure F.1. Cross sections of the overturning caisson (12.60m)

F.2. Weight and Centre of Gravity

The total length of caisson is 24.25 metre. The total amount of concrete is 985 m^3 and the corresponding weight amounts 24,628 kN.

Per running metre quay, this is equal to 41 m^3 and 1016 kN/m^1.

Element	Thickness	Volume	Weight
Frontwall	550mm	223 m^3	5,563 kN
Inner walls	250mm	114 m^3	2,859 kN
Side walls	500mm	92 m^3	2,291 kN
Counterforts	400mm	106 m^3	2,646 kN
Back wall (straight)	450mm	94 m^3	2,354 kN
Back wall (declined)	500mm	111 m^3	2,354 kN
Baseplate	600mm	183 m^3	4,583 kN
Buttress	400mm	5 m^3	126 kN
Top slab	400mm	31 m^3	784 kN
Joints	-	26 m^3	653 kN
Caisson	-	985 m^3	24,628 kN

Centre of gravity

G_x = 5.13 metre (horizontal distance from front of structure)

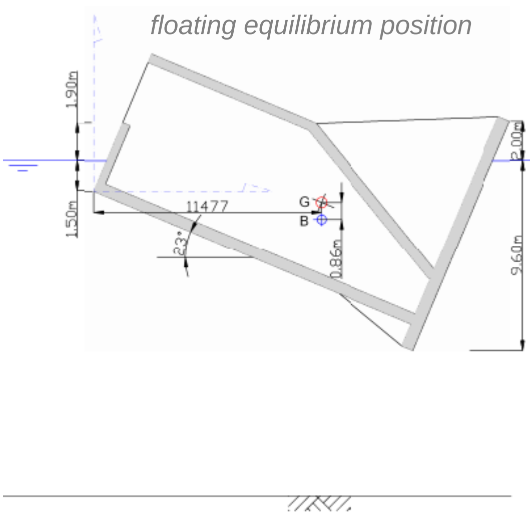
G_y = 7.21 metre (vertical distance from bottom of caisson)

F.3. First floating equilibrium position

The floating position presented below is in equilibrium and satisfies the two major hydrostatic conditions:

1. *Buoyancy:* The weight of displaced water (Δ_c) equals the weight of the caisson (W_c);
2. *Equilibrium:* The upward buoyancy force (B) acts on the same vertical axis as the centre of gravity (G).

It is found that the floating position depicted below is in equilibrium. The corresponding draught is approximately 9.60 metre. The particular floating position is verified in the next section.



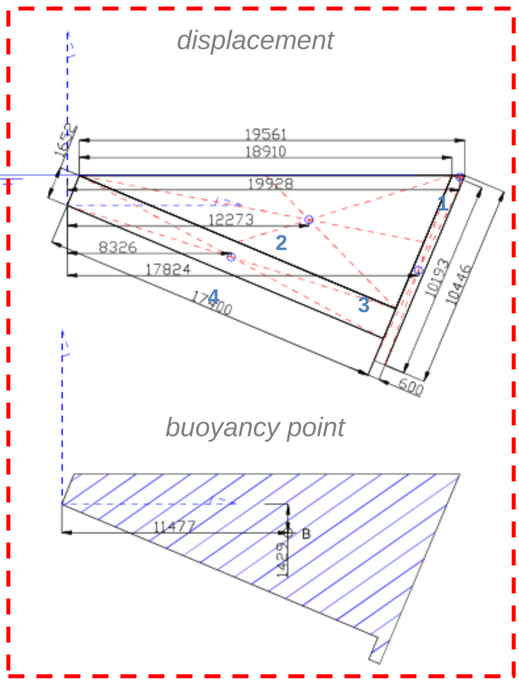


Figure F.2 Initial floating equilibrium position

Floating equilibrium verification

The horizontal distance (x) to the centre of gravity (B) of displaced water can be calculated by dividing the first moment by the total mass (M):

$$x_B = \frac{\sum_{i=1}^{N}(m_i \cdot x_i)}{M}$$

In which the static moment is:

$$M \cdot x_B = \sum_{i=1}^{N}(m_i \cdot x_i)$$

And the total mass:

$$M = \sum_{i=1}^{N} m_i$$

Since the weight of the displaced water is constant, the horizontal distance to point B can be calculated by considering the volume of displaced water. Furthermore, the displacement is almost constant over the length of the caisson, which allows considering planes of the cross section. Only the displacement of counterforts must be adjusted since they do not displace the entire longitudinal section.

In order to calculate the position of the buoyancy point, the displaced water is divided into 4 elements (see figure F.2.). The following properties can be distinguished per element:

Element	Area (A) [mm^2]	Horizontal (x_b) distance [mm]	First moment [mm^3]
1	7.59×10^4	1.99×10^4	1.51×10^9
2	6.39×10^7	1.23×10^4	7.84×10^{11}
3	6.12×10^6	1.78×10^4	1.09×10^{11}
4	2.87×10^7	8.33×10^3	2.39×10^{11}
Total:	$A_{c,1} = 9.88 \times 10^7$		1.13×10^{12}

The weight of displaced (Δ_c) water amounts:

$$\Delta_{c,1} = \nabla_{c,1} \cdot \varphi_w = 98.8\text{m}^3 / \text{m}^1 \cdot 10.30\text{kN/m}^3 = 1018 \text{ kN/m}^1$$

Which shows a minor difference with the calculated weight of the caisson per running metre. This difference insignificant;

$$\Delta_{c,1} \cong W_c$$
$$1018 \text{ kN/m}^1 \cong 1016 \text{ kN/m}^1 \rightarrow 0.2\%$$

From the presented table, the distance of the buoyancy point to the reference plane can be calculated as:

$$x_{B,1} = \frac{\sum_{i=1}^{N}(a_i \cdot x_i)}{A_c} = \frac{1.13 \cdot 10^{12}}{9.88 \cdot 10^7} = 11{,}477 \text{ mm}$$

The distance to the buoyancy point (B) is identical to the position of the centre of gravity (G), when the caisson floats under an angle of approximately 23 degrees. Therefore;

$$x_{B,1} = x_{G,1} = 11{,}477 \text{ mm}$$

The vertical distance of point B relative to the reference plane can be calculated in a similar way and is found to be:

$y_{B,1} = -1429$ mm

And the vertical distance to point G:

$y_{G,1} = -568$ mm

The vertical distance of the centre of gravity (G) is smaller, which implies that a sufficient metacentric height is required for a stable floating position.

Floating stability: metacentric height
The distance between points B and G is:

$$\overline{BG} = |y_{c,1} - y_{G,1}| = |-1429 + 568| = 861\text{mm}$$
$$\overline{BG} = 0.86 \text{ m}$$

The metacentric height can thereby be found by calculating distance BM:

$$\overline{BM} = \frac{I_{yy}}{V_w}$$

In which I_{yy} is the second moment of area of the water plane, which can be calculated with Steiner's rule as:

$$I_{yy} = \left\{\frac{1}{12} \cdot L \cdot H_1^3 + A_1 \cdot a^2\right\} + \left\{\frac{1}{12} \cdot L \cdot H_2^3 + A_2 \cdot a^2\right\}$$
$$I_{yy} = \left\{\frac{1}{12} \cdot 24.25 \cdot 10.83^3 + (24.25 \cdot 10.83) \cdot (10.83 / 2)^2\right\} +$$
$$\left\{\frac{1}{12} \cdot 24.25 \cdot 8.71^3 + (24.25 \cdot 8.71) \cdot (8.71 / 2)^2\right\}$$
$$I_{yy} = 15{,}609 \text{ m}^4$$

From which the distance BM can be obtained:

$$\overline{BM} = \frac{I_{yy}}{V_w} \approx \frac{I_{yy}}{\nabla_{c,1} \cdot L} = \frac{15{,}609}{98.8 \cdot 24.25} = 6.50 \text{ m}$$

The initial metacentric height is:

$$\overline{GM} = \overline{BM} - \overline{BG} \approx 6.50 - 0.86 \cong 5.60\text{m}$$

In conclusion, sufficient floating stability can be obtained without adjustments to the caisson.

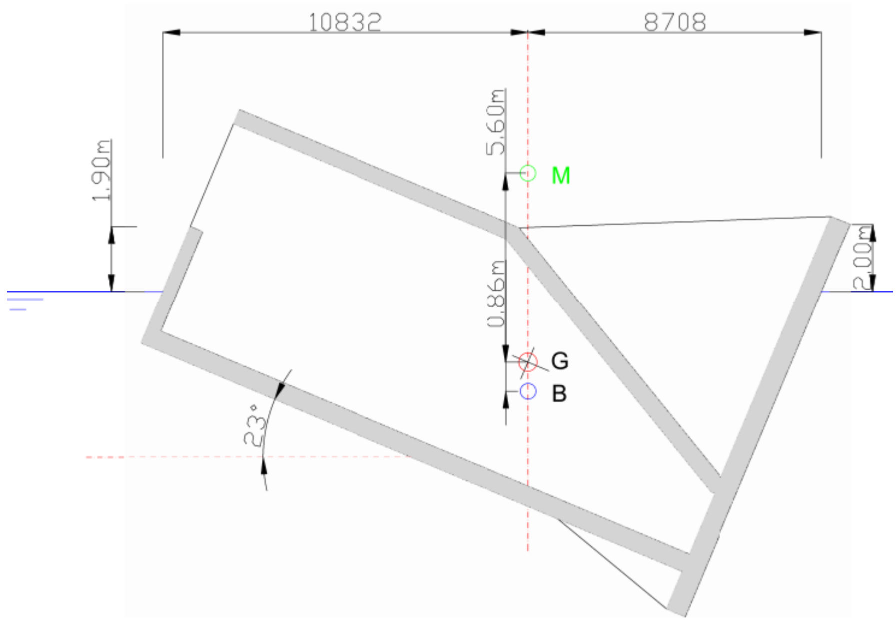


Figure F.3. Floating position, distances to gravity centre and initial metacentric height

F.4. Turning process

The (final) equilibrium position during the turning process can be obtained in a similar manner as the calculation of the first floating equilibrium position. The following floating position and properties are verified:

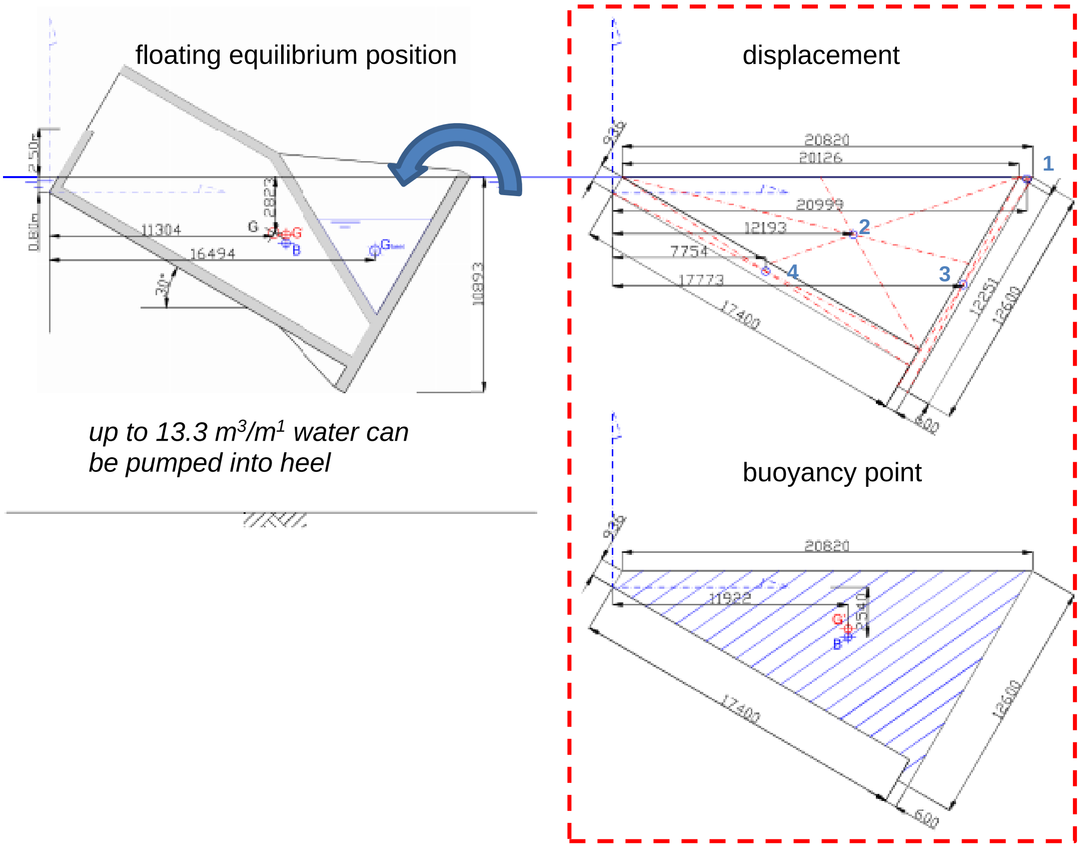


Figure F.4. Floating equilibrium, displacement and buoyancy point with 13.30m^3/m^1 water in heel

Similar to the initial floating position, the shown displacement model is divided into 4 elements, which are presented in the table below:

Element	Area (A) [mm^2]	Horizontal (x_b) distance [mm]	First moment [mm^3]
1	1.05×10^5	2.10×10^4	2.20×10^9
2	8.80×10^7	1.22×10^4	1.07×10^{12}
3	7.35×10^6	1.78×10^4	1.31×10^{11}
4	1.63×10^7	7.75×10^3	1.26×10^{11}
Total:	$A_{c,1} = 1.117 \times 10^8$		1.33×10^{12}

The weight of displaced water ($\Delta_{c,2}$) amounts:

$$\Delta_{c,2} = \nabla_{c,2} \cdot \varphi_w = 111.7\text{m}^3 / \text{m}^1 \cdot 10.30\text{kN/m}^3 = 1{,}151\ \text{kN/m}^1$$

Which shows a difference with the calculated weight of the caisson. The difference between these values is devoted to occupied water in the heel of the caisson.

$$\nabla_{heel} = \nabla_{c,2} - \nabla_{c,1}$$
$$\nabla_{heel} = 111.7\text{m}^3 / \text{m}^1 - 98.4\text{m}^3 / \text{m}^1 = 13.3\text{m}^3 / \text{m}^1$$

From the presented table, the distance of the buoyancy point to the reference plane can be calculated as:

$$x_{B,2} = \frac{\sum_{i=1}^{N}(a_i \cdot x_i)}{A_c} = \frac{1.33 \cdot 10^{12}}{1.117 \cdot 10^8} = 119{,}210\ \text{mm}$$

The distance to the buoyancy point (B) must be identical to the position of the centre of gravity (G), when the caisson floats under an angle of approximately 30 degrees. However, the added water in the heel of the caisson causes the centre of gravity G to shift. This new point, denoted as G', can be calculated as follows:

Element	Area (A) [mm^2]	Horizontal (x_b) distance [mm]	First moment [mm^3]
caisson	9.86×10^7	1.13×10^4	1.11×10^{12}
ballast-water	1.31×10^7	1.65×10^4	2.17×10^{11}
Total:	$A_{c,2} = 1.117 \times 10^8$		1.33×10^{12}

From the presented table, the distance of the gravity centre G' to the reference plane can be calculated with the same values as previously:

$$x_{B,2} = \frac{\sum_{i=1}^{N}(a_i \cdot x_i)}{A_c} = \frac{1.33 \cdot 10^{12}}{1.117 \cdot 10^8} = 119{,}210\ \text{mm}$$

Therefore;

$$x_{B,2} = x_{G,2} = 119{,}210\ \text{mm}$$

The vertical distance of point B relative to the reference plane can be calculated in a similar way and is found to be:

$$y_{B,2} = -2540\ \text{mm}$$

And the vertical distance to point G':

$$y_{G,2} = -2131\ \text{mm}$$

The vertical distance of the centre of gravity (G') is smaller, which implies that a sufficient metacentric height is required for a stable floating position.

Floating stability: metacentric height
The distance between points B and G is:

$$\overline{BG} = |y_{c,1} - y_{G,1}| = |-2540 + 2131| = 409\text{mm}$$

$$\overline{BG} = 0.41 \text{ m}$$

The metacentric height can thereby be found by calculating distance BM:

$$\overline{BM} = \frac{I_{yy}}{V_w}$$

In which I_{yy} is the second moment of area of the water plane, which can be calculated with Steiner's rule as:

$$I_{yy} = \left\{\frac{1}{12} \cdot L \cdot H_1^3 + A_1 \cdot a^2\right\} + \left\{\frac{1}{12} \cdot L \cdot H_2^3 + A_2 \cdot a^2\right\}$$

$$I_{yy} = \left\{\frac{1}{12} \cdot 24.25 \cdot 11.45^3 + (24.25 \cdot 11.45) \cdot (11.45/2)^2\right\} + \left\{\frac{1}{12} \cdot 24.25 \cdot 9.40^3 + (24.25 \cdot 9.40) \cdot (9.40/2)^2\right\}$$

$$I_{yy} = 18,780 \text{ m}^4$$

From which the distance BM can be obtained:

$$\overline{BM} = \frac{I_{yy}}{V_w} \approx \frac{I_{yy}}{\nabla_{c,1} \cdot L} = \frac{18,780}{111.7 \cdot 24.25} = 6.90 \text{ m}$$

The metacentric height is:

$$\overline{GM} = \overline{BM} - \overline{BG} \approx 6.90 - 0.41 \cong 6.50\text{m}$$

The stability is therefore still sufficient just before the heel scoops water.

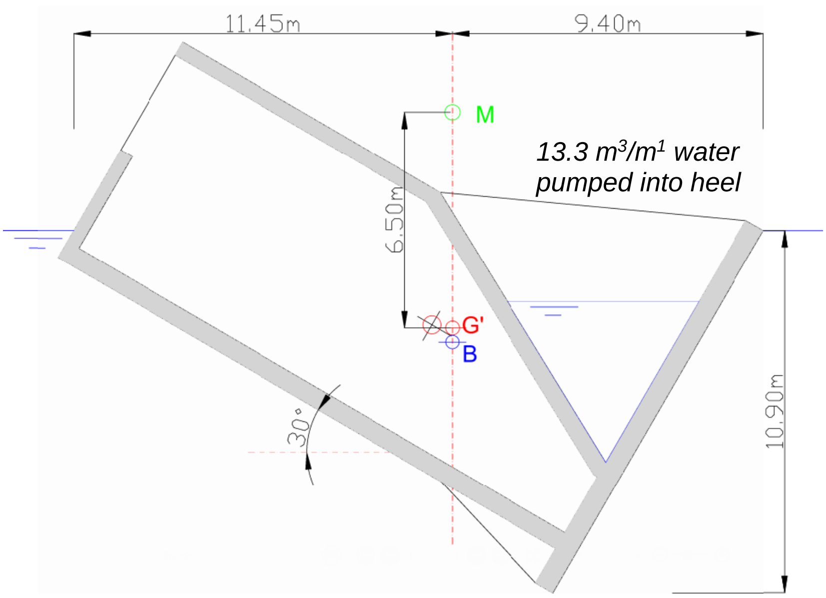


Figure F.5. Floating position, distances to gravity centre and final metacentric height

Heel overflow and turning
A different floating position will be obtained when the heels scoops water. At this point, a similar position is not feasible due to the drastic change of buoyancy.

Turning the caisson involves an unsteady underwater motion of the caisson itself and unsteady flow around it. Both effects can be taken into account and modelled by adding mass to the equation of motion. This added weight is to incorporate the effect of acceleration or deceleration, which requires movement of the surrounding water. Nevertheless, this requires many assumptions a proper calibration which is out of scope of this thesis.

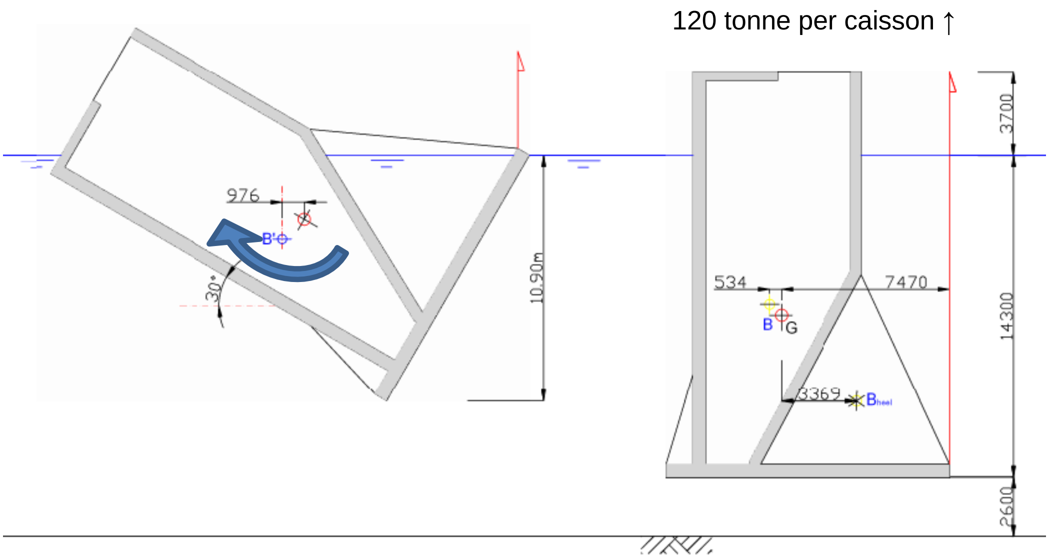


Figure F.6. Change of buoyancy point and turning process

In order to turn the caissons in a controlled manner, a sheerleg or floating crane of at least 120 tonne is required. Further optimization could result in a reduction of crane capacity.

F.5. Second floating equilibrium position (without assistance)

This position can be obtained in a similar manner as the calculation of the first floating equilibrium position. In this case, it is assumed that no crane assistance is provided. In order to calculate the position of the buoyancy point, the displaced water is divided into 6 elements (see figure F.7.). The following properties can be distinguished per element:

Element	Area (A) [mm^2]	Horizontal (x_b) distance [mm]	First moment [mm^3]
1	1.05×10^7	1.08×10^4	1.13×10^{11}
2	2.01×10^7	9.98×10^3	2.00×10^{11}
3	3.86×10^7	4.97×10^3	1.92×10^{11}
4	1.89×10^7	7.51×10^3	1.42×10^{11}
5	3.88×10^6	9.15×10^3	3.55×10^{10}
6	7.56×10^6	6.01×10^3	4.54×10^{10}
Total:	$A_c = 9.94 \times 10^7$		7.28×10^{11}

The weight of displaced (Δ_c) water amounts:

$$\Delta_{c,3} = \nabla_{c,3} \cdot \varphi_w = 99.4 \cdot 10.30 = 1024 \text{ kN/m}^1$$

Which shows a minor difference with the calculated weight of the caisson per running metre. This is considered to be insignificant;

$\Delta_{c,3} \cong W_c$

$1024\ \text{kN/m}^1 \cong 1016\ \text{kN/m}^1 \rightarrow 0.8\%$ difference

From the presented table, the distance of the buoyancy point to the reference plane can be calculated as:

$$x_B = \frac{\sum_{i=1}^{N}(a_i \cdot x_i)}{A_c} = \frac{7.28 \cdot 10^{11}}{9.94 \cdot 10^{7}} = 7{,}323\ \text{mm}$$

The distance to the buoyancy point (B) is identical to the position of the centre of gravity (G), when the caisson floats under and angle of approximately 20 degrees.

$x_B = x_G = 7{,}323\ \text{mm}$

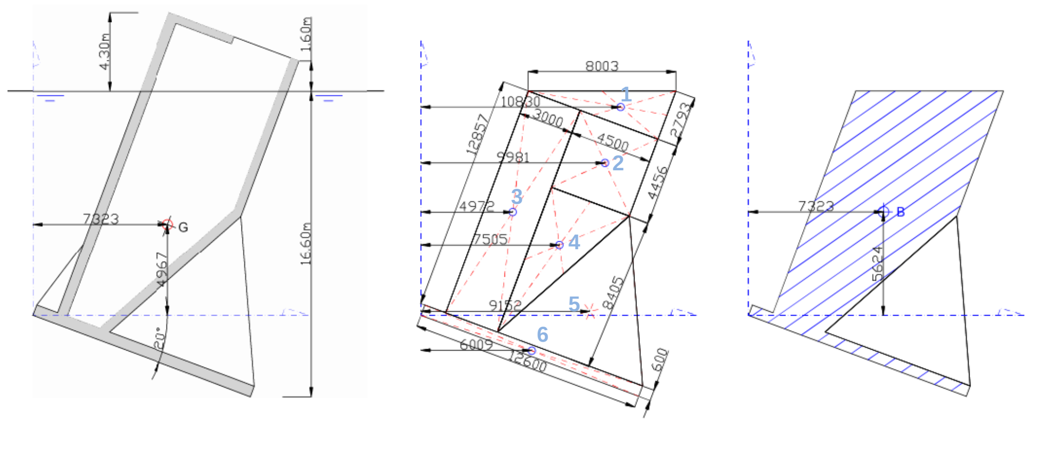
The vertical distance of point B relative to the reference plane can be calculated in a similar way and is found to be:

$y_B = 5{,}624\ \text{mm}$

and:

$y_G = 4{,}967\ \text{mm}$

The vertical distance of the centre of gravity (G) is smaller, which implies a stable equilibrium.



Centre of gravity relative to reference planes	Horizontal distance to elements (1-6)	Shape of displaced water and buoyancy point relative to reference planes

Figure F.7. Floating position after turning

F.6. Operational stability

The load scheme for kern verification is similar to the drawing below. Note that horizontal actions are not drawn, but nevertheless included.

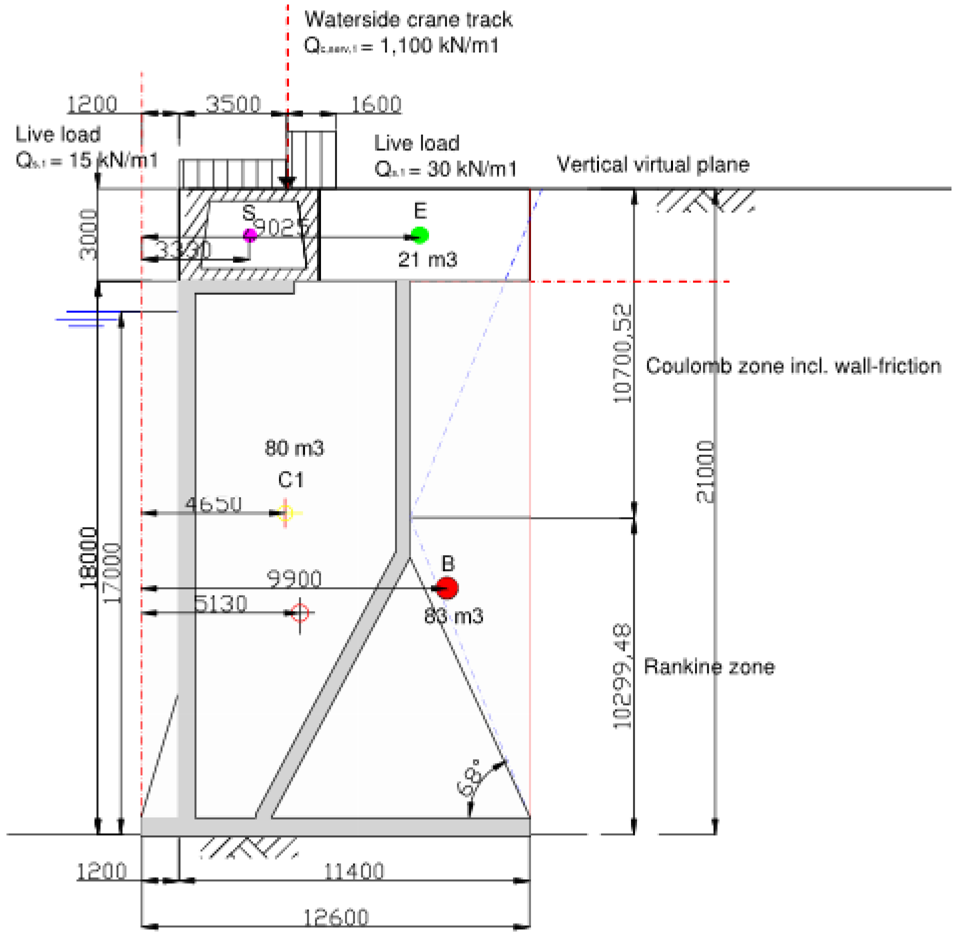
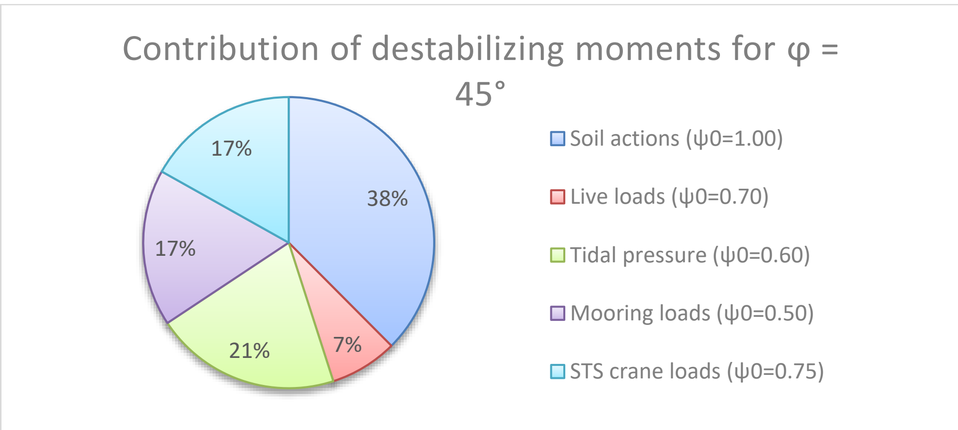


Figure F.8. Schematization for operational stability

The load distribution for GEO stability verification is as follows:



The verification of actions and safety factors are shown in the tables below:

Overturning verification	Situation	Verification	Factor of safety*
Kern verification	High water + vertical STS-crane load	SLS - GEO	1.0
Kern verification	High water	SLS - GEO	1.2
Resultant force within 1/3 of foundation width	Low water	ULS - GEO	1.6
Resultant force within 1/3 of foundation width	High water	ULS - GEO	1.5
Equilibrium condition	Low water	ULS - EQU	1.9
Equilibrium condition	High water	ULS - EQU	1.6

Sliding verification	Situation	Verification	Factor of safety*
Forward sliding	Low water + STS load	ULS - GEO	2.1
Forward sliding	Low water	ULS - GEO	1.7
Forward sliding	High water	ULS - GEO	1.9
Forward sliding	High water + STS load	ULS - GEO	1.4
Forward sliding	High water + STS crane load (storm)	ULS - GEO	1.8

*Factor of safety on top of Eurocode / British Standard requirements. A value of 1.0 is sufficient

F.7. Structural design

The caisson concrete elements are designed with a concrete quality C35/45 and a concrete cover of 60mm. The thickness of each element is determined by considering shear force since shear reinforcement is undesired. Additionally, the thickness is determined in view of floating equilibrium positions. An overview of element thickness, effective depth (estimate) and clear spans is given in the table below.

Element	Thickness	Effective depth (d)	Clear span for immersion pressure
Front-wall	550mm	482mm	3.50m
Side-wall	500mm	432mm	2.00m - 6.50m
Compartment wall	250mm	182mm	6.50m
Back-wall (straight)	450mm	382mm	3.50m
Back-wall (declined)	500mm	432mm	3.50m
Counterforts	400mm	332mm	-
Base-plate	600mm	532mm	2.00m

Furthermore, the next table presents an overview from persistent loads retrieved from appendix E. These pressures do not all act in the same direction. If the loads act in the same direction, pressures can also act on a different height. Therefore these pressure values cannot be combined to obtain one design value. It shows that the values are considerable, but still lower than the pressure during immersion (see next sections). Immersion pressures are therefore considered for further calculations.

On top of the immersion pressure, which increases linearly over the water depth, a pressure of 80 kN/m^2 is considered to cope with operational loads near and above chart datum. This value is determined considering the loads presented the table below.

Pressure on walls	Symbol	SLS value [kN/m^2]	ULS value [kN/m^2]	Partial factor γ_B	Comb. factor ψ_0	Design value [kN/m^2]
Soil pressure neutral, low water	P_G	68.70	68.70	1.35	-	92.75
Live load	P_S	15.00	15.00	1.50	0.50	11.25
Landside crane track	$P_{C,1}$	13.95	13.95	1.35	0.75	14.10
Mobile harbour crane	$P_{C,2}$	75.00	75.00	1.35	0.75	75.95
Compartment pressure	P_J	21.90	36.90	1.35	1.00	49.80
Tidal pressure	P_{ti}	10.30	15.45	1.50	0.60	13.90
Wave loads	P_{wa}	13.86	41.60	1.50	0.60	37.45

F.7.1. Front wall design

The front-wall is mainly loaded by hydraulic pressure. As can be seen in appendix K, the soil pressure due to a sand compartment-fill is limited. The hydraulic pressure during transport, immersion and caused by wave impact and tidal deviations prevail. The thickness of 550mm is larger than other elements and shall therefore not be a governing element. The increased thickness allows a more vertical floating position (section F.5.) and improves durability (appendix C).

As a conservative approach, the maximum bending moments at the top 6.00m of the caisson is assumed to be:

$$M_{comp,d} = \pm \frac{1}{12} \cdot P_{A,tot,d} \cdot l_{eff}^{\ 2}$$

$$M_{comp,d} = \pm \frac{1}{12} \cdot 80.00 \cdot 3.75^2 = \pm 93.75 \text{ kNm}$$

As can be seen in figure F.9, the lower part of the caisson is subjected to at most 170 kN/m^2. The maximum bending moment in the front-wall becomes:

$$M_{comp,d} = \pm \tfrac{1}{12} \cdot P_{A,tot,d} \cdot l_{eff}{}^2$$

$$M_{comp,d} = \pm \tfrac{1}{12} \cdot 170.00 \cdot 3.75^2 = \pm 205 \text{ kNm}$$

With the effective span of:

$$l_{eff} = l_n + a_1 + a_2 = 3.50 + 0.5 \cdot 0.25 + 0.5 \cdot 0.25 = 3.75m$$

Reinforcement for respectively the upper and lower part is;

$$A_{s,comp,1} \cong \frac{M_{comp,d}}{z \cdot f_{yd}} = \frac{93.75 \cdot 10^6}{0.9 \cdot 516 \cdot 435} = 464 \text{ mm}^2 / \text{m}^1$$

$$A_{s,comp,1} \cong \frac{M_{comp,d}}{z \cdot f_{yd}} = \frac{205 \cdot 10^6}{0.9 \cdot 516 \cdot 435} = 1015 \text{ mm}^2 / \text{m}^1$$

The top of the caisson has a reinforcement ratio of 0.09%, which is lower than the required minimum to prevent brittle failure (>0.17% for C35/45). Therefore, In addition to the calculated reinforcement, a base mesh of ϕ16-125 (=1608mm^2) is assumed for preventing brittle failure and incorporating unconsidered aspects such as unequal settlements, shrinkage and crack-width control.

Approximate reinforcement in front-wall:	4 x 1608 mm^2 / m^1
Reinforcement volume per square metre:	0.0064 m^3 / m^2

Reinforcement volume per cubic metre concrete:
V / h_{slab} = 0.0064 m^3 / 0.55m = 0.012m^3

Reinforcement per cubic metre concrete:
W = 0.012m^3 x 7850 kg /m^3 = 92 kg/m^3

F.7.2. Side wall design

Depending on the water level during immersion, the walls shall be loaded to a certain hydrostatic pressure. There are two locations for which shear capacity has to be verified:

- At the start of the declination of the back-wall (1/2H);
- At the bottom of the caisson;

The maximum span is present up to halfway the caisson. At this point, the highest hydrostatic pressure occurs for this span. The bottom of the caisson must be able to resist the largest overall hydrostatic pressure, but has the shortest wall-span. However, the width of 3.00m at the bottom part of the caisson is lower than for the back-wall, and therefore not governing for shear verifications.

The locations of the side-walls are loaded by a pressure of:

P_1 = 8.00 x 10.30 kN/m^2 = 82.40 kN/m^2
P_2 = 17.00 x 10.30 kN/m^2 = 175.10 kN/m^2

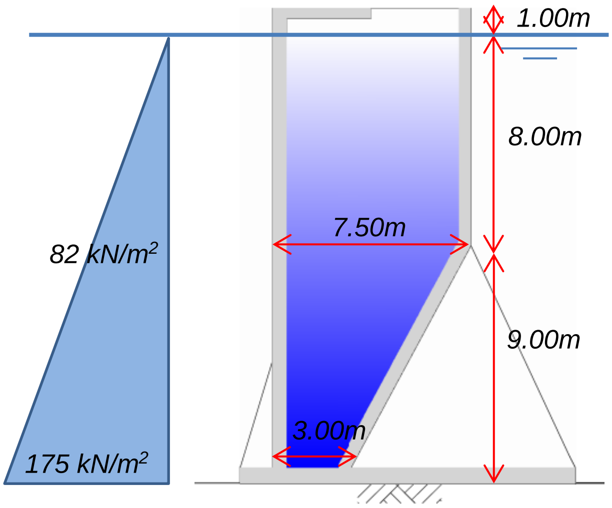


Figure F.9. Hydrostatic pressure on side wall

A load scheme could be similar to the model on the next page (fig. F.10), where the blue rectangles represent the equally distributed water pressure. The figure shows a cross-section and the pressure on the sides of the wall.

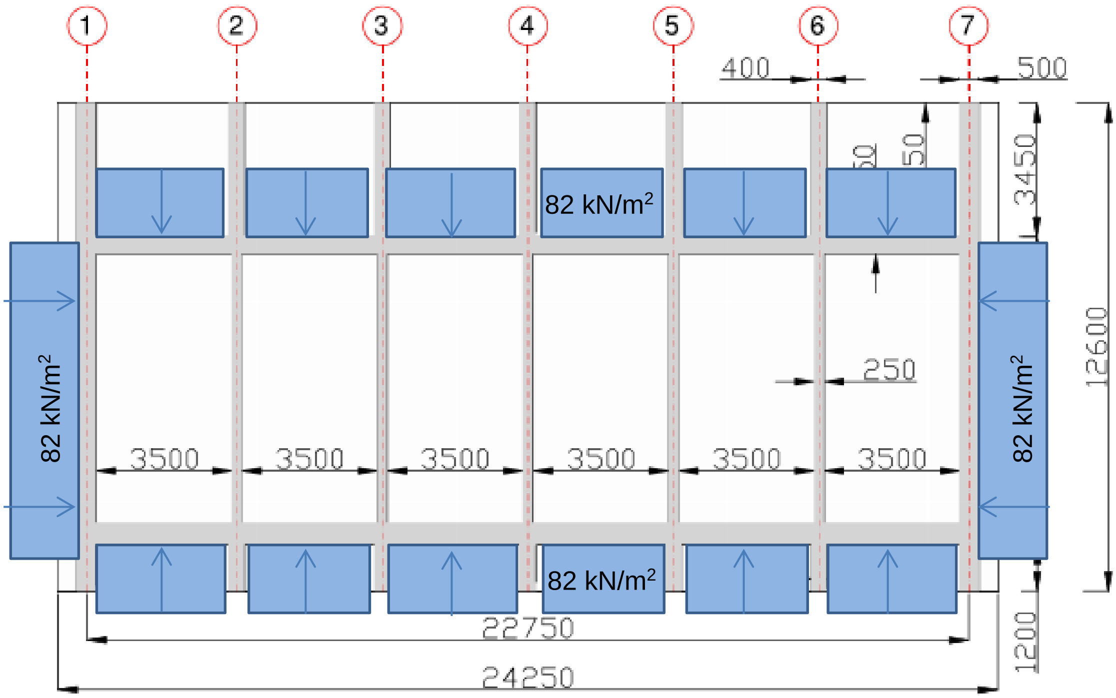


Figure F.10. Hydrostatic pressure on largest side-wall span

The effective span can be reduced for shear verifications since the load is uniform, which is stated by the BS-EN1992-1-1 (section 6, ULS, 6.2 Shear):

> *"(8) For members subject to predominantly uniformly distributed loading the design shear force need not to be checked at a distance less than d from the face of the support. Any shear reinforcement required should continue to the support. In addition it should be verified that the shear at the support does not exceed $V_{Rd,max}$ (see also 6.2.2 (6) and 6.2.3 (8)."*

Therefore, the effective length over which shear must be verified becomes equal to the total width, minus the wall thickness and effective depth on both sides:

$$l_{shear} = \mathrm{l}_{tot} - \left(h_{w,1} + h_{w,2} + d_{h,1} + d_{h,2} \right)$$
$$l_{shear} = 7.50 - \left(0.55 + 0.45 + 0.49 + 0.39\right) = 5.62 \text{ metre}$$

The perimeter within a distance of the effective depth (d) from the supports can be verified with a higher shear capacity ($V_{Rd,max}$).

$$V_{R,d,max} = \frac{\alpha_{cw} \cdot b_w \cdot z \cdot \upsilon_1 \cdot f_{cd}}{\cot\theta + \tan\theta}$$

α_{cw} = coefficient taking into account the state of stress in compression chord
(= 1.0 for non-pressured structures)
b_w = width of section
z = internal lever arm (0.9d)
$\upsilon_1 = 0.60$ (for $f_{ck} \leq 60$Mpa)
f_{cd} = design compressive stress = 23.33 Mpa
$\theta = 21.8° - 45.0°$

$$V_{R,d,max} = \frac{1.00 \cdot 1000 \cdot 0.9 \cdot 432 \cdot 0.6 \cdot 23.33}{2.5 + 0.4} = 1.88 \cdot 10^6 \text{ N}$$
$$v_{R,d,max} = \frac{V_{R,d,max}}{A_c} = \frac{1.88 \cdot 10^6}{1000 \cdot 432} = 4.34 \text{ N/mm}^2$$

The maximum shear force acting on the wall is:
$$V_E = 0.5 \cdot q \cdot (l_{tot} - 0.5(\mathrm{h}_{w1} + \mathrm{h}_{w2}) = 0.5 \cdot 82.40 \cdot (7.50 - 0.5(0.55 + 0.45)) = 268 \text{ kN}$$
$$v_E = V_E \,/\, (d \cdot b) = 268 \,/\, (0.432 \cdot 1.00) = 620 \text{ kN/m}^2$$
$$v_E = 0.62 \text{ N/mm}^2$$
$$v_E < v_{R,d,max} \rightarrow \text{shear capacity near support OK}$$

Shear verification in accordance with the Eurocode 2:
$$v_E \leq v_{R,\mathrm{d,c}}$$

The shear force acting on the wall:
$$V_E = 0.5 \cdot q \cdot l_{eff} = 0.5 \cdot 82.40 \cdot 5.62 = 232 \text{ kN}$$
$$v_E = V_E \,/\, (d \cdot b) = 232 \cdot 10^3 \,/\, (432 \cdot 1000) = 0.54 \text{ N/mm}^2$$
The design value for the shear resistance is given by:
$$v_{R,\mathrm{d,c}} = C_{R,\mathrm{d,c}} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}$$

In which:

$C_{R,\mathrm{d,c}} = 0.18 / \gamma_c = 0.18 / 1.5 = 0.12$

$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{432}} = 1.68$

$\rho_1 = \frac{A_s}{d \cdot b} = 0.60\%$

$f_{ck} =$ 35 N/mm^2

$k_1 = 0.15$

$\sigma_{cp} = 0.00$ N/mm^2 (conservative estimate during immersion)

$v_{R,\mathrm{d,c}} = 0.12 \cdot 1.68 \cdot (100 \cdot 0.008 \cdot 35)^{1/3} = 0.61$ N/mm^2

The overall factor of safety amounts:

$$F.\mathrm{o.S} = \frac{v_{R,\mathrm{d,c}}}{v_E} = \frac{0.61}{0.54} = 1.13$$

The factor of safety is rather low. Partial factors are only applied on material properties, while the loads are un-factored. The withdrawn code of practice BS-6349: Part 6: 1989 (Design of inshore moorings and floating structures) prescribed a partial factor (γ_{FL}) equal to 1.0 for temporary hydrostatic loading during construction and transport. Nevertheless, this low partial factor is not valid according to the Eurocode.

The actual factor of safety is expected to be higher, a more detailed calculation should therefore be performed using the actual plate geometry and force distribution. The compressive force can be included as well, due to the hydrostatic pressure acting on all sides of the caisson.

Furthermore, it could be reasoned to deviate (to a limited extent) from partial factors presented by the Eurocode (Table A1.2.B) Design values of actions. A partial factor of 1.35 is given for transient structural loads. However, in case of immersion of the caissons, there is hardly any risk for loss of human lives. It is expected that a particular failure only results in an economic loss. Also, the time of loading during immersion is relatively short. The start of the immersion could therefore be well planned (mitigating wave and current influences etc.), while the peak loading only occurs for several minutes. However, this reasoning could result in an applicable partial factor of 1.20, which is still higher than the calculated value.

F.7.3. Compartment wall / inner-wall design

The inner walls (250mm) are not loaded perpendicularly since the compartments are equally filled. During immersion and placement, the pressure difference must be limited. This results in primary load transfer through normal forces. Therefore, a reinforcement amount of 65 kg/m^3 is assumed, which is similar to the front wall.

F.7.4.Backwall design

The back-wall is loaded by horizontal soil- and water- pressure. On the other hand, compartment pressure reduces the resultant actions on the back-wall during operational conditions. During operational conditions, an entirely neutral pressure state results in a soil pressure of approximately 120 kN/m^2. This value, combined with hydraulic pressures and silo pressures, is lower than the considered immersion pressure of 175 kN/m^2, depicted in figure F.9. The most critical situation therefore appeared to be the immersion phase. This situation is analysed for determining wall thickness.

Due to the yield line envelope, such as schematized in figure D.3, the lowest wall-section shall not be governing. At a distance of half the span (0.5x 3.50m) from the base-plate, shear forces are also transferred to the bottom slab. This lowers the shear stress in the wall.

Therefore, the considered hydrostatic pressure on a one way spanning wall element becomes:

$P_{2,wall}$ = (17.00 - 0.60 - 0.50x 3.50) x 10.30 kN/m^2 = 151 kN/m^2

Shear verification in accordance with the Eurocode 2:

$v_{E,wall,d} \leq v_{R,\mathrm{d,c}}$

The shear force acting on the wall is:

$V_{E,wall,d} = 0.5 \cdot q \cdot l_n = 0.5 \cdot 151 \cdot (3.50 - 2x0.18) = 237$ kN

$v_{E,wall,d} = V_{E,wall,d} \,/\, (d \cdot b) = 237 \cdot 10^3 \,/\, (432 \cdot 1000) = 0.55$ N/mm^2

The design value for the shear resistance is given by:

$v_{R,\mathrm{d,c}} = C_{R,\mathrm{d,c}} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}$

In which:

$C_{R,\mathrm{d,c}} = 0.18 \,/\, \gamma_c = 0.18 / 1.5 = 0.12$

$k = 1 + \sqrt{\dfrac{200}{d}} = 1 + \sqrt{\dfrac{200}{432}} = 1.68$

$\rho_1 = \dfrac{A_s}{d \cdot b} = 0.80\%$

$f_{ck} =$ 35 N/mm^2

$k_1 = 0.15$

$\sigma_{cp} = 0.00$ N/mm^2 (conservative estimate during immersion)

$v_{R,\mathrm{d,c}} = 0.12 \cdot 1.68 \cdot (100 \cdot 0.008 \cdot 35)^{1/3} = 0.61$ N/mm^2

The overall factor of safety amounts:

$F.\mathrm{o.S} = \dfrac{v_{R,\mathrm{d,c}}}{v_E} = \dfrac{0.61}{0.55} = 1.11$

A similar conclusion holds as for the previously calculated side-walls. The factor of safety is rather low. The straight part of the back-wall (upper part) is designed with a thickness of 450mm, while the hydrostatic pressure is almost half the pressure acting on the lower part of the wall. Therefore, the upper section shall not be governing during immersion.

F.7.5. Counterfort design

The counterforts are subjected to a destabilizing moment. This causes tension in the outer zone of the counterforts. The lever arm is measured perpendicular from the counterforts to the front-wall. The front-wall itself is subjected to a compressive force.

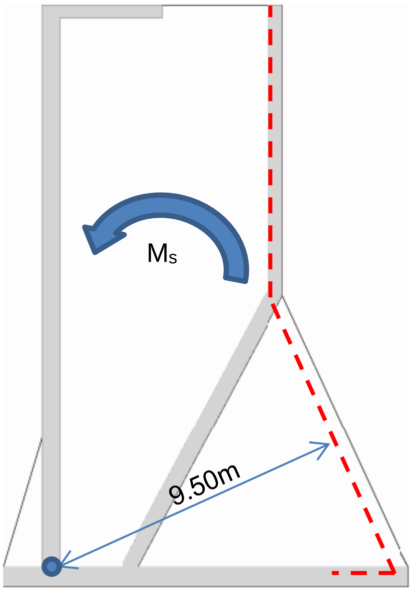


Figure F.11. Counterfort reinforcement

From the presented actions in appendix E, the effective destabilizing moment at the toe of the caisson is calculated to be:

$$M_{E,d} \approx 10{,}500 \text{ kNm/m}^1$$

The centre to centre distance of the counterforts amounts 3.75 metre. The counterforts must be able to transfer the total load of:

$$M_{E,\mathrm{tot,d}} \approx 3.75 \times 10{,}500 \text{ kNm/m}^1 = 39{,}375 \text{ kNm / counterfort}$$

Which results in a force of:

$$F_{E,tot,\mathrm{d}} = 39{,}375 \;/\; 9.50 = 4{,}145 \text{ kN / counterfort}$$

The total reinforcement amounts:

$$A_{s,count} = \frac{F_{tot}}{f_{yd}} = \frac{4{,}145 \times 10^3 \text{N}}{435 \text{ N/mm}^2} = 9{,}528 \text{ mm}^2 \text{/ counterfort}$$

20ϕ25 in every counterfort required = 9,817mm^2

F.7.6. Base-slab design

The base-slab is loaded by hydraulic- and soil pressures. The front-section of the caisson is loaded with the highest pressure and is therefore critical for the design. Due to horizontal loads, the slab is loaded in tension. This effect shall however not be considered for this preliminary analysis. The slab can be schematized as an element with multiple line-supports (inner walls). Different situations must be considered for the design of the base-slab:

- Hydraulic pressure during transport;
- Hydraulic pressure during immersion;
- Foundation pressure in operational conditions;

In contrast to the previous wall design, the loads in operational conditions can be considerably higher than the hydraulic pressure. The following aspects are therefore be considered:

- Vertical foundation pressure due to self-weight;
- Additional foundation pressure due to destabilizing actions;
- Moment transfer in wall-to-base connection;

The moment transfer in the wall-to-base connections is not considered for this preliminary study. The other aspects will be further addressed in the following paragraphs.

Based on figure F.8, a spreadsheet with different loads and combinations is made (in accordance to appendix D). The spreadsheet provides the following data regarding the governing design values for ULS foundation pressure:

Hydraulic conditions				
Caisson height	18.00	m		
Water level in front of quay	17.00	m		
Water level behind quay	17.00	m		
L-shaped caisson phi= 45 deg				
Destabilizing moment	9237	kNm		
Caisson specifications				
Caisson width	12.60	m		
Toe width	1.20	m		
Superstructure width	4.25	m		
Vertical actions	lever arm [m]	Force [kN]	Moment [kNm]	
STS crane load	4.70	1100	5170	
Q_1 = 15 kN/m^2	2.95	52.5	155	
Q_2 = 30 kN/m^2	5.50	48	264	
Superstructure	3.33	255.00	847.88	
Dry earth	9.03	386.10	3484.55	
Caisson weight (G)	5.13	1016.00	5212.08	
Water column above toe	0.60	210.12	126.07	
Compartment fill	4.53	1700.00	7706.67	
Backfill	9.65	1660.00	16019.00	
Hydraulic uplift	6.30	-2206.26	-13899.44	
	R_v=	4,221 kN	25,086	kNm
Sum of moments (M)	15,849	kNm		
Distance x (M / R_v):	3.75	m		
Eccentricity of resultant:	2.55	m		

F.7.6.1. Mean effective foundation pressure

The mean effective foundation pressure can be calculated from the sum of vertical forces, divided over the total width of the caisson:

$$P_{F,mean} = \frac{R_v}{B_{caisson}} = \frac{4{,}221\ \text{kN}}{12.60\ \text{m}} = 335\ \text{kN/m}^2$$

F.7.6.2. Foundation pressure due to eccentricity of resultant

The eccentricity (2.55m) is larger than 1/6x the width of the caisson (2.10m). According to the Eurocode, this is allowed for ultimate limit states. Due to a decreased effective width, the foundation pressure increases. The effective width (b_{eff}) amounts 3x3.75 = 11.25m.

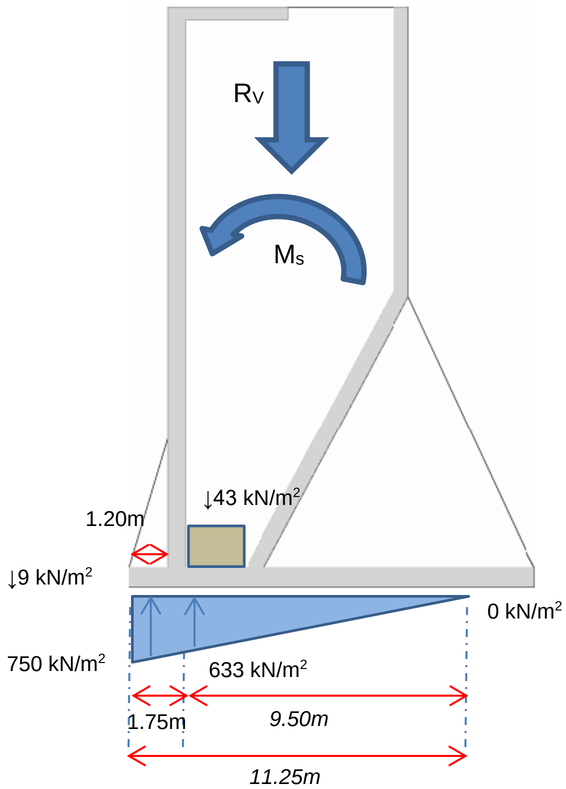


Figure F.13. Effective pressure on caisson slab

Considering this, the foundation pressure is calculated by the scheme shown in figure F.12. The maximum pressure at the toe becomes;

$$P_{F,max,d} = \tfrac{2}{3} \cdot \frac{R_v}{x}$$

$$P_{F,max,d} = \tfrac{2}{3} \cdot \frac{4{,}221\ \text{kN}}{3.75\ \text{m}} = 750\ \text{kN/m}^2 \qquad \uparrow$$

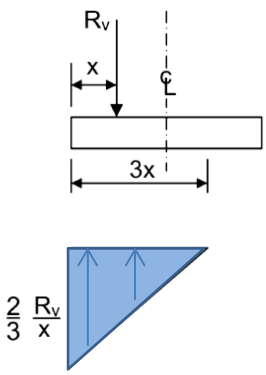


Figure F.12. Foundation pressure scheme for large eccentricities

The compartment walls (and buttresses) are schematized as line-supports for the bottom-slab. The clear span of the slab is therefore 3.50 metre and the upward effective pressure is equal to 750 kN/m².

On the other hand, the foundation pressure is slightly reduced by the self-weight of the slab:

$$P_{G,slab} = h_{slab} \cdot (\gamma_c - \gamma_w)$$

$$P_{G,slab} = 0.60 \cdot (25.00 - 10.30) = 8.82\ \text{kN/m}^2 \qquad \downarrow$$

Foundation pressure (toe)
The resulting pressure under the toe of the caisson becomes:

$P_{F,toe,d}$ = 750 - 9 = 741 kN/m² ↑

The toe can conservatively be schematized as a cantilever element (1.20m). This neglects the possible load transfer to the buttresses, which are designed to extent the counterforts and compartment-walls (≈3.60m).

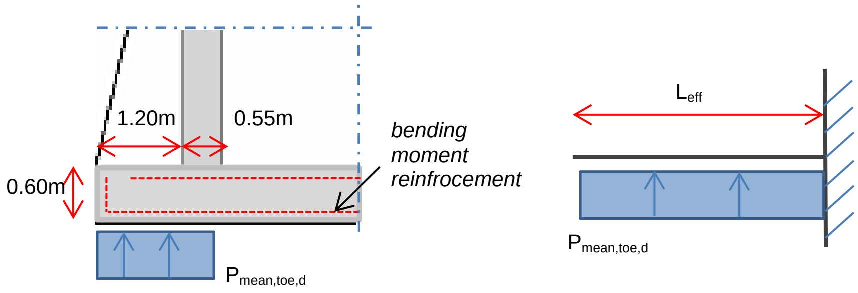


Figure F.14. Load scheme and schematization of toe

The effective span is:

$$l_{eff,toe} = l_n + a_1$$

l_n = clear distance
a_1 = ½ h_{wall}; ½ h_{slab}= 0.5 x 550 =275mm

$$l_{eff,toe} = 1.20 + 0.28 \cong 1.50\text{m}$$

Shear reinforcement (toe)
The mean shear stress on the toe is:

$$P_{mean,toe,d} = \frac{P_{F,toe,d} + \left(\frac{b_{eff} - b_{toe}}{b_{eff}}\right) \cdot P_{F,toe,d}}{2}$$

$$P_{mean,toe,d} = \frac{750 + \left(\frac{11.25 - 1.50}{11.25}\right) \cdot 750}{2} = \frac{750 + 650}{2} = 700 \text{ kN/m}^2 \quad \uparrow$$

The shear stress on the intersection of the wall-to-base is:

$$V_{F,toe,d} = P_{mean,toe,d} \cdot l_{eff,toe} = 700 \text{ kN/m}^2 \cdot 1.50 \text{ m} = 1050 \text{ kN/m}^1$$

$$v_{F,toe,d} = V_{F,toe,d} / (d \cdot b) = 1050 \cdot 10^3 / (532 \cdot 1000) = 1.97 \;\; \text{N/mm}^2$$

This is higher than the shear capacity without shear reinforcement (see for instance fig. J.2.). Similar as for the side walls, the maximum shear stress for a section including vertical shear reinforcement can be calculated as;

$$V_{R,d,max} = \frac{\alpha_{cw} \cdot b_w \cdot z \cdot \upsilon_1 \cdot f_{cd}}{\cot\theta + \tan\theta}$$

$$V_{R,d,max} = \frac{1.00 \cdot 1000 \cdot 0.9 \cdot 532 \cdot 0.6 \cdot 23.33}{2.5 + 0.4} = 2.31 \cdot 10^6 \text{ N}$$

$$v_{R,d,max} = \frac{V_{R,d,max}}{A_c} = \frac{2.31 \cdot 10^6}{1000 \cdot 532} = 4.34 \text{ N/mm}^2$$

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot \cot\theta$$

$$\frac{A_{sw}}{s} = \frac{V_{F,\text{toe,d}}}{z \cdot f_{ywd} \cdot \cot\theta} = \frac{1050 \cdot 10^3}{0.9 \cdot 532 \cdot 0.8 \cdot 500 \cdot 2.5}$$

$$\frac{A_{sw}}{s} = 2.19 \text{ mm}^2 / \text{mm}^1$$

$$A_{sw} = 2190 \text{ mm}^2 / \text{m}$$

This is an equivalent of 2ϕ16-180 (=2234mm^2). Although shear reinforcement in slabs is generally labour intensive, it is applied locally.

Bending moment reinforcement (toe)
Since shear reinforcement ϕ16 is applied in the toe, the effective depth (d) becomes smaller than previously assumed. For further calculations, the effective depth is reduced to:

d = h_{slab} - c – ϕ_V – 1/2 ϕ_M = 600 – 60 – 16 – 8 = 516mm

The bending moment of the cantilever slab (toe):

$$M_{toe,d} = 0.5 \cdot P_{mean,toe.d} \cdot l_{eff,\text{toe}}{}^2$$

$$M_{toe,d} = 0.5 \cdot 700 \cdot 1.50^2 = 788 \text{ kNm /m}^1$$

The required reinforcement:

$$A_{s,\text{toe,d}} \cong \frac{M_{toe,d}}{z \cdot f_{yd}} = \frac{788 \cdot 10^6}{0.9 \cdot 516 \cdot 435} = 3898 \text{ mm}^2 / \text{m}^1$$

This is an equivalent of ϕ20-80 (3927mm^2/m^1).

Reinforcement estimate (toe)
The amount of reinforcement in the front of the bottom-slab is estimated on the previous preliminary reinforcement calculations. In addition to the calculated reinforcement, a base mesh of ϕ16-125 (=1608mm^2) is assumed for unequal settlements, crack-width control and other unconsidered aspects.

Shear reinforcement:	2234 mm^2 / m^1
Bending moment reinforcement:	3927 mm^2 / m^1
Lower longitudinal reinforcement:	1608 mm^2 / m^1
Upper transverse reinforcement:	1608 mm^2 / m^1
Upper longitudinal reinforcement:	1608 mm^2 / m^1
Approximate total reinforcement in slab:	10,985 mm^2 / m^1
Reinforcement volume per metre:	0.011 m^3

Reinforcement volume per cubic metre concrete:
V / h_{slab} = 0.011 m^3/m^1 / 0.60m = 0.018m^3

Weight of reinforcement per cubic metre concrete (toe):
W = 0.018m^3 x 7850 kg /m^3 = 144 kg/m^3

Foundation pressure (below compartments)

The pressure below compartments is determined according to the same triangular pressure distribution over the width of the caisson (fig. F13). Furthermore, compartment pressure can be subtracted, as derived in appendix K, the vertical soil pressure can be calculated as:

$\sigma_z(z) = z_0 \gamma_s (1 - e^{-z/z_0})$

$z_0 \cong$ 4.40 m

$\gamma'_s = 20.00 - 10.30 = 9.70$ kN/m^2

$z = 17.40$ m

$\sigma_z(z) = z_0 \gamma_s (1 - e^{-z/z_0}) \cong z_0 \gamma_s$

$\sigma_z(z) \cong 4.40 \times 9.70 = 43$ kN/m^2

hence;

$P_{J,eff}$ = 43 kN/m^2 ↓

This calculated value, according to the Janssen theory is conservative. A detailed analysis might result in higher counteracting pressures. Nevertheless, a downward pressure of at least 9 + 43 = 52 kN/m^2 can be subtracted from the effective foundation pressure. The resulting pressure becomes:

$P_{F,\text{tot,d}} = \left(\frac{9.50}{11.25} \cdot 750 \right) - 9 - 43 = 581$ kN/m^2 ↑

Bending moment reinforcement (below compartments)

The free spans of the bottom-slab are 2.00 metre x 3.50 metre. In which 2.00 metre corresponds to the distance between the front- and back-wall, and 3.50 metre represents the distance between separation walls.

The effective span of this section is:

$$l_{eff,x} = l_n + a_1 + a_2 = 2.00 + 0.5 \cdot 0.55 + 0.5 \cdot 0.50 = 2.53\text{m}$$

$$l_{eff,y} = l_n + a_1 + a_2 = 3.50 + 0.5 \cdot 0.25 + 0.5 \cdot 0.25 = 3.75\text{m}$$

l_y / l_x = 3.75m / 2.53 = 1.50

The slab section is considered as a clamped element on all sides. Therefore, positive and negative bending moments occur in directions x and y. As a conservative approach, the maximum bending moments in all directions is assumed to be:

$$M_{comp,d} = \pm 1/12 \cdot P_{F,tot,d} \cdot l_x^2$$

$$M_{comp,d} = \pm 1/12 \cdot 581 \cdot 2.53^2 = \pm 310 \text{ kNm}$$

$$A_{s,\text{comp,M}} \cong \frac{M_{comp,d}}{z \cdot f_{yd}} = \frac{310 \cdot 10^6}{0.9 \cdot 516 \cdot 435} = 1534 \text{ mm}^2/\text{m}^1$$

Apply from front-wall to back-wall; ϕ16-125 (=1608mm^2).

Shear reinforcement (below compartments)
Conservatively, the shear stress can be calculated as:

$$V_{comp,d} = 1/2 \cdot P_{F,tot,d} \cdot l_x$$

$$V_{comp,d} = 1/2 \cdot 581 \cdot 2.53 = 735 \text{ kN/m}^1$$

$$v_{comp,d} = V_{comp,d} / (d \cdot b) = 735 \cdot 10^3 / (516 \cdot 1000) = 1.42 \ \text{ N/mm}^2$$

This is higher than the shear capacity without shear reinforcement (see for instance fig. J.2.). Similar as for the side walls, the maximum shear stress for a section including vertical shear reinforcement can be calculated as;

$$\frac{A_{sw}}{s} = \frac{V_{F,\text{comp,d}}}{z \cdot f_{ywd} \cdot \cot\theta} = \frac{735 \cdot 10^3}{0.9 \cdot 532 \cdot 0.8 \cdot 500 \cdot 2.5}$$

$$\frac{A_{sw}}{s} = 1.54 \text{ mm}^2 / \text{mm}^1$$

$$A_{sw} = 1535 \text{ mm}^2 / \text{m}$$

Apply from front-wall to back-wall; 2ϕ16-250 (=1608mm^2).

Reinforcement estimate (below compartments)
The amount of reinforcement in the front of the bottom-slab is estimated on the previous preliminary reinforcement calculations. In addition to the calculated reinforcement, a base mesh of ϕ16-125 (=1608mm^2) is assumed for unequal settlements, crack-width control and other unconsidered aspects.

Shear reinforcement:	1608 mm^2 / m^1
Lower transverse reinforcement:	1608 mm^2 / m^1
Lower longitudinal reinforcement:	1608 mm^2 / m^1
Upper transverse reinforcement:	1608 mm^2 / m^1
Upper longitudinal reinforcement:	1608 mm^2 / m^1
Approximate total reinforcement in slab:	8040 mm^2 / m^1
Reinforcement volume per metre:	0.008 m^3

Reinforcement volume per cubic metre concrete:
V / h_{slab} = 0.008 m^3/m^1 / 0.60m = 0.013m^3

Weight of reinforcement per cubic metre concrete:
W = 0.013m^3 x 7850 kg /m^3 = 105 kg/m^3

Foundation pressure and reinforcement (heel)
In ULS situations, the back of the base-slab lacks upward pressure. On the other hand, a downward soil pressure is present (fig. F15.). The downward soil pressure is approximately equal to the height of the soil column above the heel and an added live load:

$$P_{back,\text{d}} = \gamma_B \left\{ d_{wet} \cdot \gamma'_s + d_{dry} \cdot \gamma_s \right\} + \gamma_B Q_S$$

$$P_{back,\text{d}} = 1.35 \cdot \left\{ 17.00 \cdot 10.00 + 4.00 \cdot 20.00 \right\} + 1.50 \cdot 30.00 = 383 \text{ kN/m}^2$$

$$M_{\text{heel},d} = \pm 1/12 \cdot P_{F,tot,d} \cdot l_{eff}^{\ 2}$$

$$M_{heel,d} = \pm 1/12 \cdot 383 \cdot 3.90^2 = \pm 485 \text{ kNm}$$

$$A_{s,\text{comp,M}} \cong \frac{M_{comp,d}}{z \cdot f_{yd}} = \frac{485 \cdot 10^6}{0.9 \cdot 516 \cdot 435} = 2401 \text{ mm}^2 / \text{m}^1$$

Apply in heel slab; ϕ20-125 (=2513mm^2).

And shear reinforcement:

$$V_{\text{heel},d} = 1/2 \cdot P_{\text{back},d} \cdot l_{eff}$$

$$V_{\text{heel},d} = 1/2 \cdot 383 \cdot 3.90 = 747 \text{ kN/m}^1$$

$$v_{heel,d} = V_{\text{heel},d} / (d \cdot b) = 747 \cdot 10^3 / (516 \cdot 1000) = 1.45 \text{ N/mm}^2$$

Similar stress as found below compartments; apply from front-wall to back-wall; 2ϕ16-250 (=1608mm^2).

Reinforcement estimate (heel-slab)

The amount of reinforcement in the heel of the bottom-slab is estimated on the previous preliminary reinforcement calculations. In addition to the calculated reinforcement, a base mesh of 0.19% is assumed for unequal settlements, crack-width control and other unconsidered aspects.

Shear reinforcement:	1608 mm^2 / m^1
Lower transverse reinforcement:	1608 mm^2 / m^1
Lower longitudinal reinforcement:	2513 mm^2 / m^1
Upper transverse reinforcement:	1608 mm^2 / m^1
Upper longitudinal reinforcement:	1608 mm^2 / m^1
Approximate total reinforcement in slab:	8945 mm^2 / m^1
Reinforcement volume per metre:	0.009 m^3

Reinforcement volume per cubic metre concrete:
V / h_{slab} = 0.009 m^3/m^1 / 0.60m = 0.015m^3

Weight of reinforcement per cubic metre concrete:
W = 0.015m^3 x 7850 kg /m^3 = 117 kg/m^3

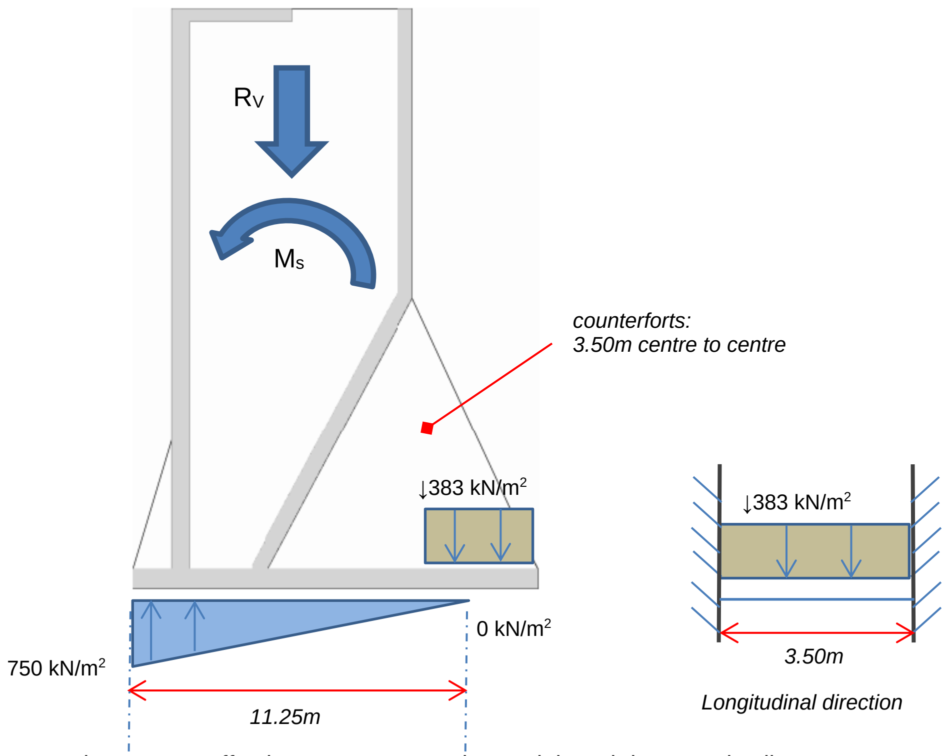


Figure F.15. Effective pressure on caisson slab and downward soil pressure

Estimate of reinforcement in elements
Based on the previous reinforcement calculations, the structural elements are roughly divided into high reinforced sections (120kg/m^3) and low reinforced sections (92kg/m^3). This is not a lean quantitative estimate since more detailed calculations will increase the amount of steel. Aspects such as, crack-width control, unequal settlements, thermal shrinkage, auxiliary reinforcement shall reasonably result in an increase of steel use.

Based on these aspects, the following estimate is made for the amount of reinforcement required for one caisson:

Element	Reinforcement B500 [kg/m^3]	Concrete volume [m^3]	Reinforcement amount [kg]
Front-wall	92	223	20,500
Side- and compartment-walls	92	206	18,950
Back-wall (top)	92	94	8,650
Back-wall (declined)	92	111	10,200
Counterforts	120	106	12,700
Bottom-slab	120	183	22,000
Additional parts (joints, buttress, etc.)	120	62	7,400
Total caisson	-	985 m^3	100,420 kg

G. Rectangular Caisson Design (12.60m)

G.1. Geometry

The geometry of the rectangular caisson is depicted below; the upper drawing represents a transverse cross-section and the lower drawing a horizontal cross-section.

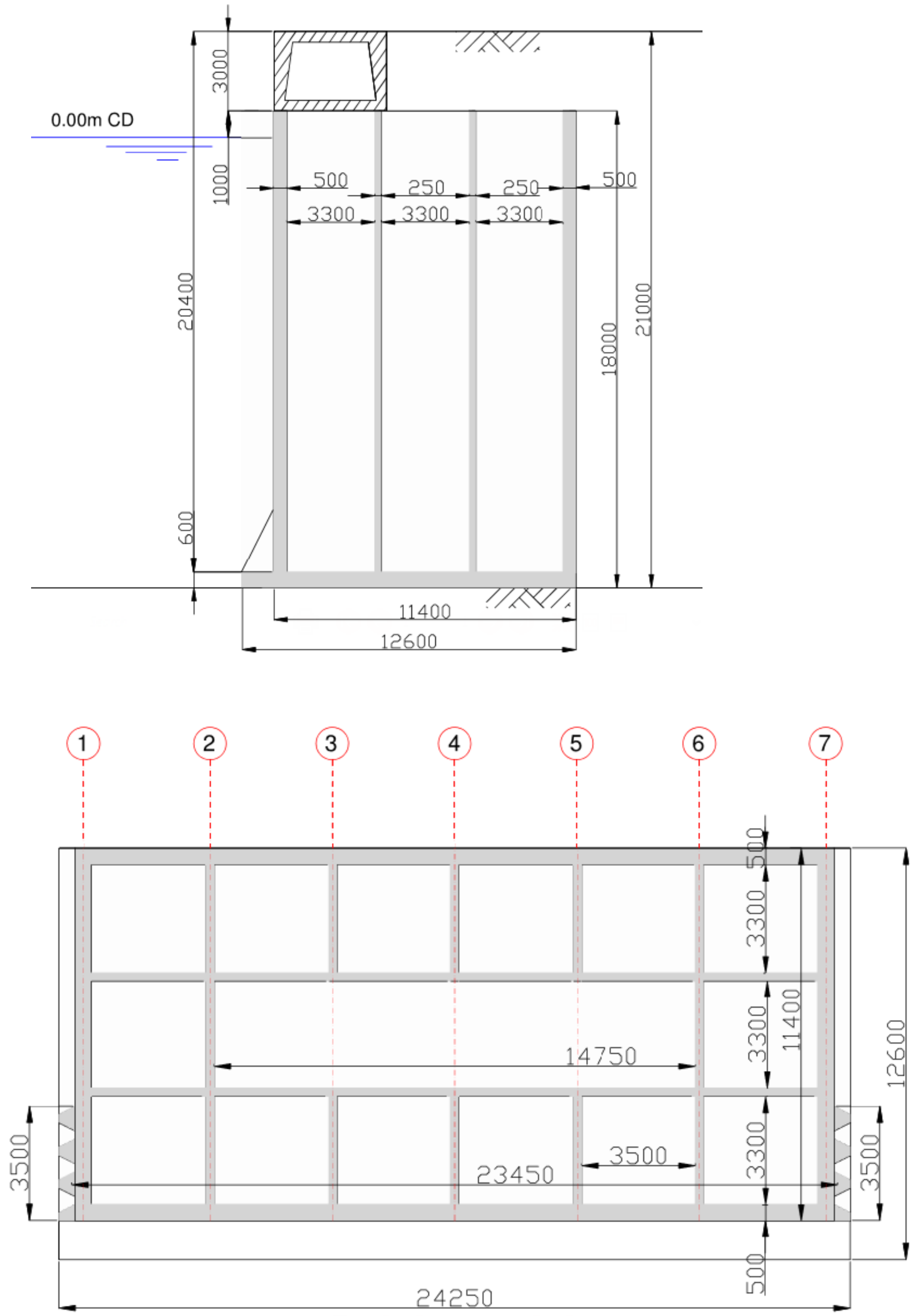


Figure G.1. Cross-sections of the rectangular caisson (12.60m)

G.2. Weight and Centre of Gravity

The total length of caisson is 24.25 metre. The total amount of concrete is 1,166 m^3 and the corresponding weight amounts 29,150 kN.

Per running metre quay, this is equal to a volume of 48 m^3/m^1 and 1,202 kN/m^1.

Element	Thickness	Volume	Weight
Frontwall	500mm	194 m^3	4850 kN
Backwall	500mm	194 m^3	4850 kN
Side walls	500mm	198 m^3	4950 kN
Inner walls	250mm	366 m^3	9150 kN
Baseplate	600mm	183 m^3	4575 kN
Buttress	500mm	5 m^3	125 kN
Joints	-	26 m^3	650 kN
Caisson	-	1166m^3	29,150 kN

Centre of gravity

G_x = 6.80 metre (horizontal distance from front of structure)

G_y = 7.90 metre (vertical distance from bottom caisson = KG)

Note that these values are slightly off centre due to the toe structure which extends the bottom plate. Ballast water can be applied in order to obtain a straight floating position.

G.3. Floating equilibrium position

The draught of the caisson is approximately:

$$d \cong \frac{W_c}{\varphi_w \cdot A_c} = \frac{29{,}150}{10.30 \cdot (23.25 * 11.40)} = 10.70 \text{ m}$$

The distance from the bottom of the caisson (K) to the buoyancy centre is approximately:

$$\overline{KB} \cong d / 2 = 5.35 \text{ m}$$

The distance between points B and G is:

$$\overline{BG} = \overline{KG} - \overline{KB}$$

$$\overline{BG} = 7.90 - 5.35 = 2.55 \text{ m}$$

The metacentric height can be found by calculating distance BM:

$$\overline{BM} = \frac{I_{yy}}{V_w} \cong \frac{\frac{1}{12} \cdot L \cdot B^3}{\nabla_{slab} + \nabla_{comp}} = \frac{\frac{1}{12} \cdot L \cdot B^3}{\{\nabla_{slab}\} + \{(\text{d} - \text{d}_{slab}) \cdot (L_{comp} \cdot B_{comp})\}}$$

$$= \frac{\frac{1}{12} \cdot 24.25 \cdot 12.60^3}{\{183\} + \{(10.70 - 0.60) \cdot (23.25 \cdot 11.40)\}} = 1.40 \text{ m}$$

$$h_{metacentre} = \overline{BM} - \overline{BG} = 1.40 - 2.55 = -1.15 \text{ m} \rightarrow \text{negative metacentric height}$$

The metacentric height must be at least 0.50 metre to provide sufficient floating stability. There are two obvious measures which can be taken;

1. Width increase, which results in a larger area moment of inertia;
2. Weight increase, adding ballast water into the compartments or a different (floor) design. This weight increase will result in a decrease of distance BG.

Option 2 seems to be the most economical solution to increase floating stability. However, this option could be restricted in practice due to local constrains. The following calculation is including 500 m^3 ballast water in the 14 compartments (3.50 x 3.30m^2). The largest middle compartment (14.75 x 3.30m^2) is kept empty in order to reduce the free surface effect of ballast water. This corresponds to an internal water level of approximately 2.80m.

Including ballast, the new distance from the bottom of the caisson (K) to its centre of gravity (G) becomes:

$\overline{KG} \cong 7.00$ m

The draught (d) would increase to approximately:

$$d \cong \frac{W_c + W_{ballast}}{\varphi_w \cdot A_c} = \frac{29{,}150 + (500 \cdot 10.30)}{10.30 \cdot (23.25 * 11.40)} = 12.60 \text{ m}$$

The distance to the buoyancy centre amounts:

$\overline{KB} \cong d / 2 = 6.30$m

The distance from the buoyancy point (B) to the metacentre (M) can be found by:

$$\overline{BM} \cong \frac{I_{yy}}{V_w}$$

$$\overline{BM} = \frac{I_{yy}}{V_w} \cong \frac{\frac{1}{12} \cdot L \cdot B^3}{\nabla_{slab} + \nabla_{comp}} = \frac{\frac{1}{12} \cdot L \cdot B^3}{\{\nabla_{slab}\} + \{(\text{d} - \text{d}_{slab}) \cdot (L_{comp} \cdot B_{comp})\}}$$

$$= \frac{\frac{1}{12} \cdot 24.25 \cdot 12.60^3}{\{183\} + \{(12.60 - 0.60) \cdot (23.25 \cdot 11.40)\}} = 1.20 \text{ m}$$

The metacentric height becomes:

$$h_{metacentre} = \overline{KB} + \overline{BM} - \overline{KG} = 6.30 + 1.20 - 7.00 = 0.50 \text{ m} \rightarrow \text{ sufficient height}$$

At this point, no free surface effect has been considered for calculating the metacentric height. Unfortunately, free water in the compartments has a destabilising effect on the stability of the caisson. This can simply be explained by the additional shift of the centre of gravity of ballast water when the caisson turns. This shifts results in an additional moment which amplifies the rotation.

The unfavourable influence of ballast water on stability can be incorporated by subtracting the area moment of inertia of compartment water from the original moment of inertia:

$$I_{stab} = I_{caisson} - \Sigma I_{comp,i}$$

Therefore, the shift of the centre of gravity (G), due to the free surface effect can be calculated as:

$$\overline{GG'} = \frac{I_{comp}}{V_{water}} = \frac{\Sigma I_{comp,i}}{V_{water}} = \frac{n_{comp}\left\{\frac{1}{12} \cdot l_{comp} \cdot b_{comp}{}^3\right\}}{\{\nabla_{slab}\} + \{(\text{d} - \text{d}_{slab}) \cdot (L_{comp} \cdot B_{comp})\}}$$

$$= \frac{14\left\{\frac{1}{12} \cdot 3.50 \cdot 3.30^3\right\}}{\{183\} + \{(12.60 - 0.60) \cdot (23.25 \cdot 11.40)\}} = 0.04 \text{ m}$$

The free surface effect reduces the metacentric height slightly. The new height becomes 0.46m, which is slightly below the desired minimum of 0.50m.

In overview, the following cross section can be drawn from the calculation results:

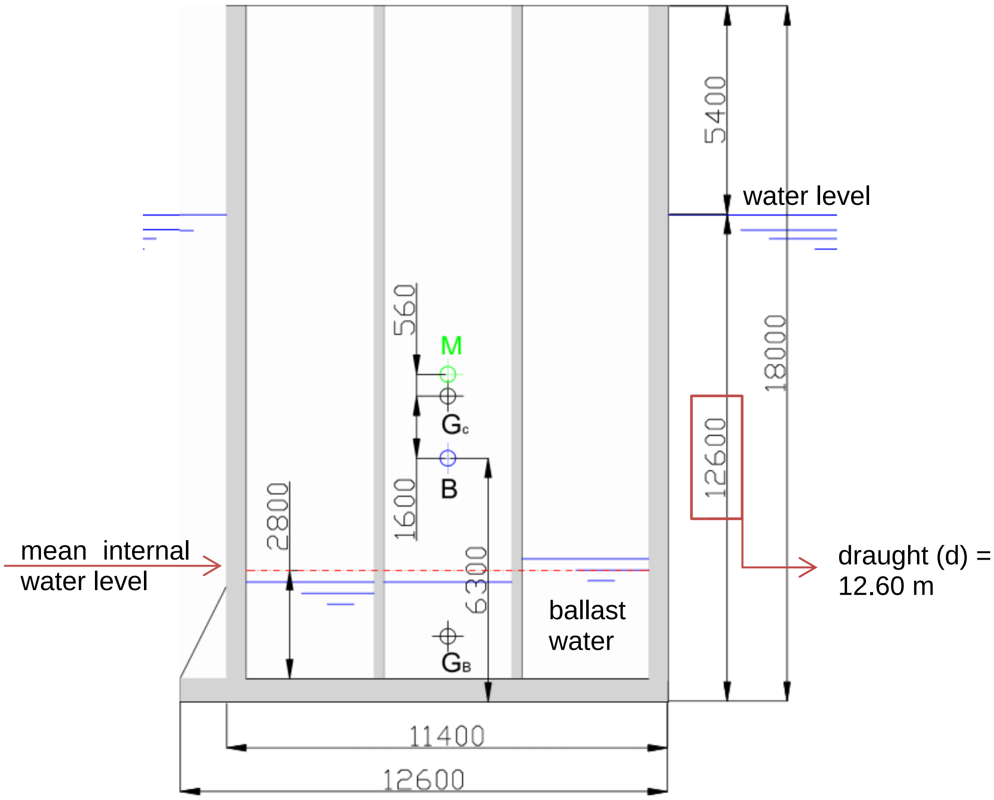


Figure G.2. Floating position of caisson with ballast water

G.4. Operational stability

The verification of actions and safety factors are shown in the tables below:

Overturning verification	Situation	Verification	Factor of safety*
Kern verification	High water + vertical STS-crane load	SLS - GEO	1.5
Kern verification	High water	SLS - GEO	2.0
Resultant force within 1/3 of foundation width	Low water	ULS - GEO	2.4
Resultant force within 1/3 of foundation width	High water	ULS - GEO	2.2
Equilibrium condition	Low water	ULS - EQU	2.8
Equilibrium condition	High water	ULS - EQU	2.4

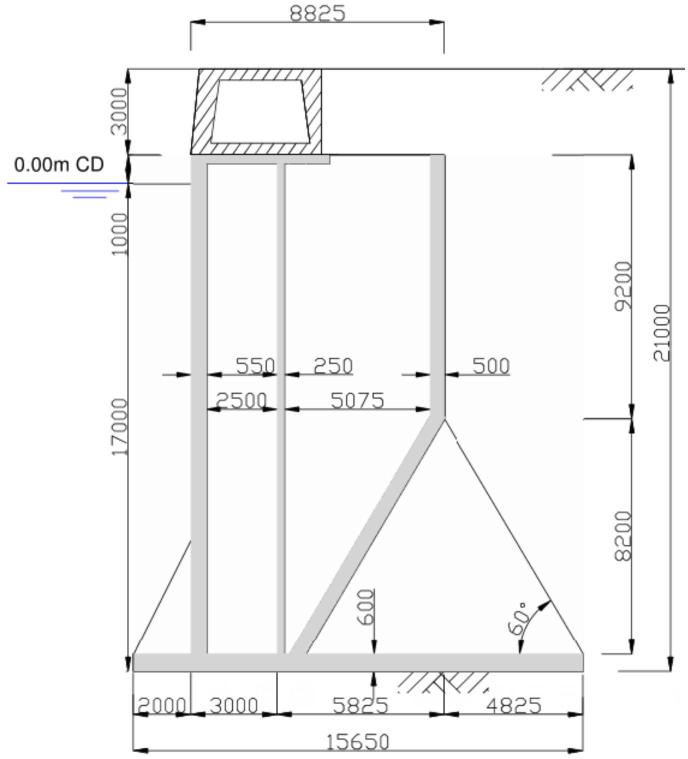
Sliding verification	Situation	Verification	Factor of safety*
Forward sliding	Low water + STS load	ULS - GEO	2.5
Forward sliding	Low water	ULS - GEO	2.0
Forward sliding	High water	ULS - GEO	2.2
Forward sliding	High water + STS load	ULS - GEO	1.8
Forward sliding	High water + STS crane load (storm)	ULS - GEO	2.2

*Factor of safety on top of Eurocode / British Standard requirements. A value of 1.0 is sufficient

H. Overturning Caisson Design (15.65m)

H.1. Geometry

The geometry of the overturning caisson, designed for a sand backfill, is depicted below; the upper drawing represents a transverse cross-section and the lower drawing a horizontal cross-section.



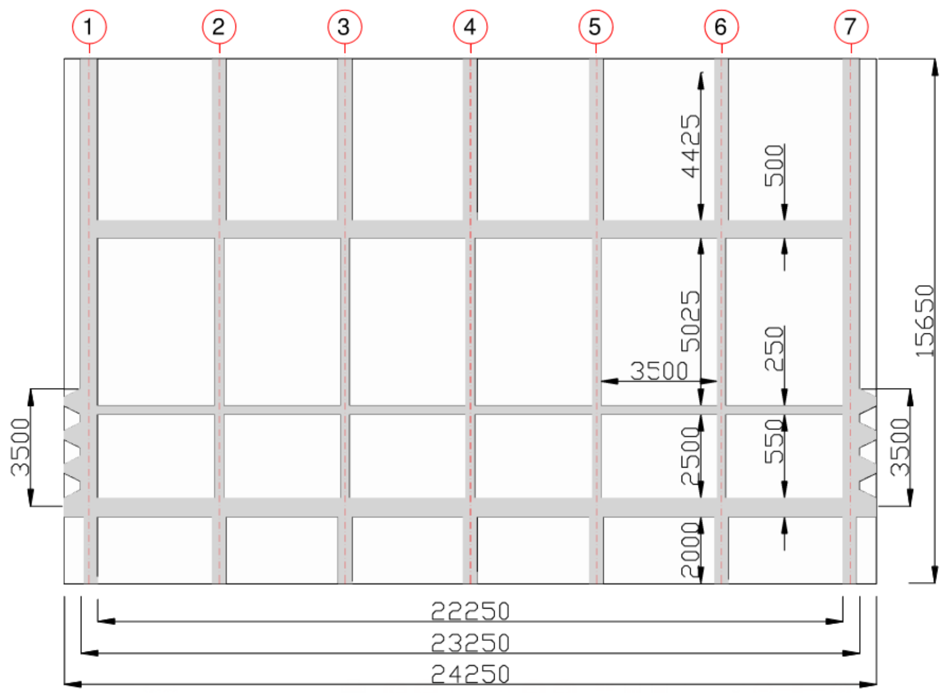


Figure H.1. Cross-sections of the overturning caisson (15.65m)

H.2. Weight and Centre of Gravity

The total length of caisson is 24.25 metre. The total amount of concrete is 985 m^3 and the corresponding weight amounts 24,628 kN.

Per running metre quay, this is equal to 41 m^3 and 1016 kN/m^1.

Element	Thickness	Volume	Weight
Frontwall	550mm	223 m^3	5,563 kN
Inner walls - longitudinal	250mm	92 m^3	2,859 kN
Inner walls – transverse	250mm	101 m^3	2,520 kN
Side walls	500mm	135 m^3	3360 kN
Counterforts	400mm	55 m^3	1,385 kN
Back wall (straight)	500mm	102 m^3	2,547 kN
Back wall (declined)	500mm	121 m^3	3,037 kN
Baseplate	600mm	218 m^3	5,458 kN
Buttress	500mm	14 m^3	350 kN
Top slab	400mm	28 m^3	698 kN
Joints	-	26 m^3	653 kN
Caisson	-	1,115 m^3	27,880 kN

Centre of gravity

G_x = 6.45 metre (horizontal distance from front of structure)

G_y = 7.21 metre (vertical distance from bottom of caisson)

H.3. Floating equilibrium position

The draught of the wide overturning caisson amounts approximately 11.40m (fig. H2).

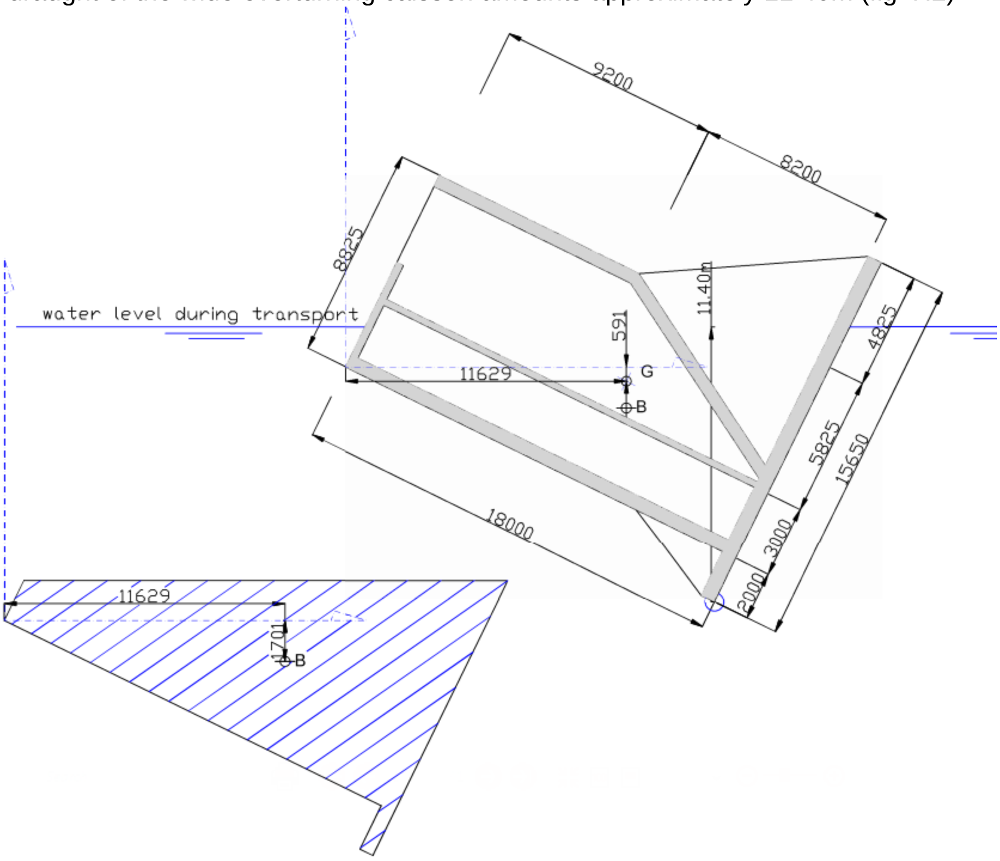


Figure H.2. Initial floating equilibrium position

H.4. Turning process

A vertical position can be obtained with approximately 9 m^3/m^1 ballast water in the front compartment. This situation is shown in figure H.3 below.

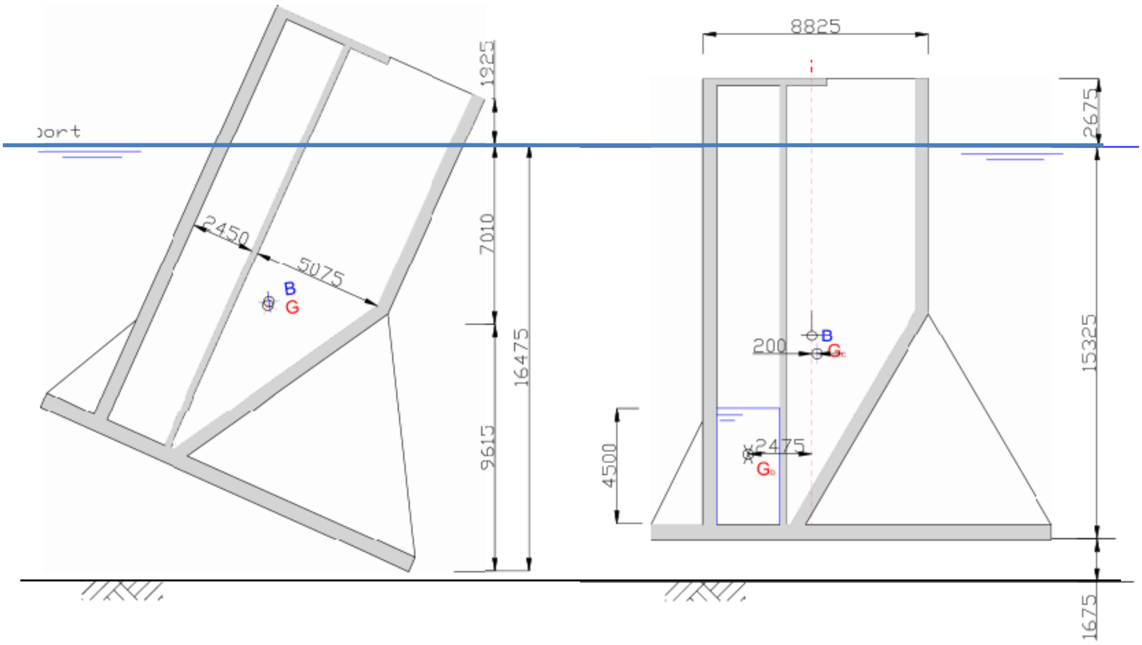


Figure H.3. Floating positon after turning (second equilibrium position)

The displaced water after turning is shown in H.4. Here, the counterforts are also responsible for a part of the displacement. The hatched (blue) area contributes over the full length of the caisson.

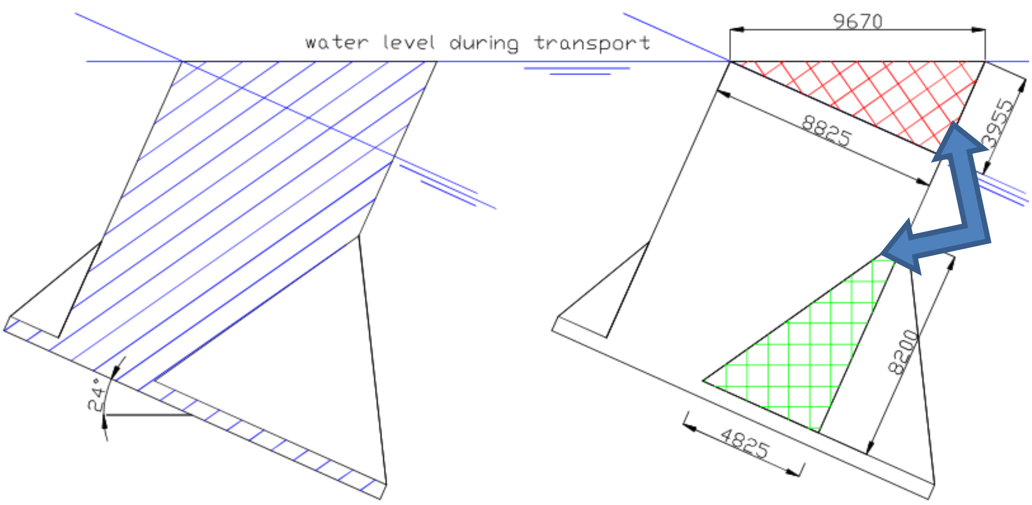


Figure H.4. Displacement after turning (second equilibrium position)

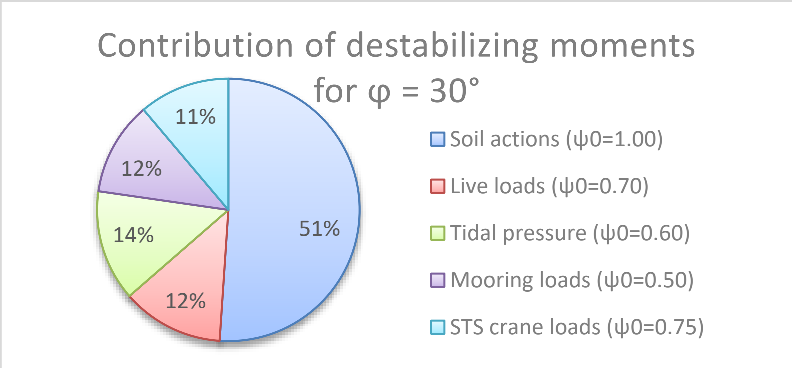
The position after turning deviates more than 30 degrees from vertical. This can be compensated by ballast water, or the caisson can be (partially) lifted by a floating crane.

The metacentric height becomes approximately:

$$\overline{BM} = \frac{I_{yy}}{V_w} \cong \frac{\frac{1}{12} \cdot L \cdot B^3}{V_w} = \frac{\frac{1}{12} \cdot 24.35 \cdot 10.388^3}{2{,}683 \text{ m}^3} = 0.85 \text{ m}$$

H.5. Operational stability

The load distribution for GEO stability verification is as follows:



The verification of actions and safety factors are shown in the tables below:

Overturning verification	Situation	Verification	Factor of safety*
Kern verification	High water + vertical STS-crane load	SLS - GEO	1.1
Kern verification excl. vertical STS-crane load	High water	SLS - GEO	1.3
Resultant force within 1/3 of foundation width	Low water	ULS - GEO	1.8
Resultant force within 1/3 of foundation width	High water	ULS - GEO	1.6
Equilibrium condition	Low water	ULS - EQU	1.9
Equilibrium condition	High water	ULS - EQU	1.8

Sliding verification	Situation	Verification	Factor of safety*
Forward sliding	Low water + STS load	ULS - GEO	1.4
Forward sliding	Low water	ULS - GEO	1.1
Forward sliding	High water + STS load	ULS - GEO	1.3
Forward sliding	High water	ULS - GEO	1.0
Forward sliding	High water + STS crane load (storm)	ULS - GEO	1.2

*Factor of safety on top of Eurocode / British Standard requirements. A value of 1.0 is sufficient

I. Rectangular Caisson Design (15.65m)

I.1. Geometry

The geometry of the rectangular caisson, designed for a sand backfill is depicted below; the upper drawing represents a transverse cross-section and the lower drawing a horizontal cross-section.

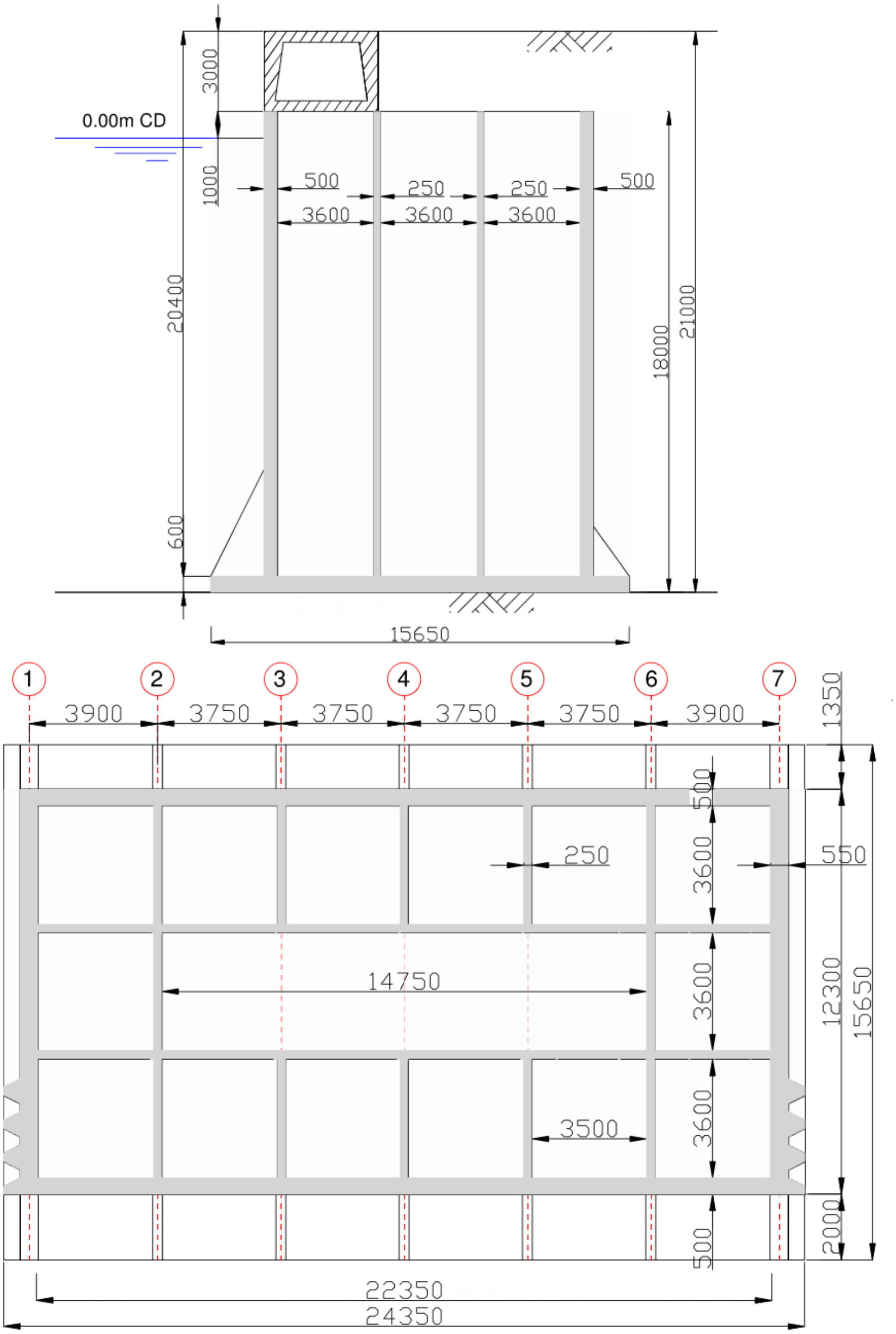


Figure I.1. Cross-sections of the rectangular caisson (15.65m)

I.2. Weight and Centre of Gravity

The total length of caisson is 24.35 metre. The total amount of concrete is 1,275 m^3 and the corresponding weight amounts 31,883 kN.

Per running metre quay, this is equal to a concrete volume of 52 m^3/m^1 and a weight of 1,309 kN/m^1.

Element	Thickness	Volume	Weight
Frontwall	500mm	204 m^3	5100 kN
Backwall	500mm	204 m^3	5,100 kN
Side walls	550mm	216 m^3	5,407 kN
Inner walls	250mm	382 m^3	9,559 kN
Baseplate	600mm	229 m^3	5,716 kN
Buttress	300mm	14 m^3	350 kN
Joints	-	26 m^3	650 kN
Caisson	-	1275m^3	31,883 kN

Centre of gravity

G_x = 7.98 metre (horizontal distance from front of structure)
G_y = 7.46 metre (vertical distance from bottom caisson, denoted as KG)

Note that these values are slightly off centre due to the toe structure which extends the bottom plate. Ballast water can be applied in order to obtain a straight floating position.

I.3. Floating equilibrium position

The draught of the caisson is approximately:

$$W_c = \varphi_w \cdot A_c \cdot d_1 + \varphi_w \cdot A_{slab} \cdot d_{slab}$$

$$d_1 \cong \frac{W_c - \{\varphi_w \cdot A_{slab} \cdot d_{slab}\}}{\{\varphi_w \cdot A_c\}} = \frac{31{,}883 - \{10.30 \cdot 24.35 \cdot 15.65 \cdot 0.60\}}{\{10.30 \cdot 23.25 \cdot 12.30\}} = 10.02 \text{ m}$$

$$d = d_1 + d_{slab} = 10.02 + 0.60 = 10.62 m$$

The distance from the bottom of the caisson (k) to the buoyancy centre is approximately:

$$\overline{KB} \cong d / 2 = 10.62 / 2 = 5.31 \text{ m}$$

The distance between points B and G is:

$$\overline{BG} = \overline{KG} - \overline{KB}$$

$$\overline{BG} = 7.46 - 5.31 = 2.15 \text{ m}$$

The metacentric height can be found by calculating distance BM:

$$\overline{BM} = \frac{I_{yy}}{V_w} \cong \frac{\frac{1}{12} \cdot L \cdot B^3}{V_w} = \frac{\frac{1}{12} \cdot 23.25 \cdot 15.65^3}{31{,}883 / 10.30} = 2.40 \text{ m}$$

$$h_{metacentre} = \overline{BM} - \overline{BG} = 2.40 - 2.15 = 0.25\text{m} \rightarrow \text{insufficient metacentric height}$$

In order to increase the floating stability, 500m^3 ballast water is added to the compartments. The distance from the bottom of the caisson (K) to its centre of gravity (G) reduces to:

$\overline{KG} \cong 7.07$ m

The draught (d) would increase to approximately:

$$W_c + W_{ballast} = \varphi_w \cdot A_c \cdot d_1 + \varphi_w \cdot A_{slab} \cdot d_{slab}$$

$$d_1 \cong \frac{W_c + W_{ballast} - \{\varphi_w \cdot A_{slab} \cdot d_{slab}\}}{\{\varphi_w \cdot A_c\}} =$$

$$\frac{31{,}883 + \{500 \cdot 10.30\} - \{10.30 \cdot 24.35 \cdot 15.65 \cdot 0.60\}}{\{10.30 \cdot 23.25 \cdot 12.30\}} = 11.77 \text{ m}$$

$$d = d_1 + d_{slab} = 11.77 + 0.60 = 12.37m$$

The distance to the buoyancy centre amounts:

$\overline{KB} \cong d / 2 = 12.37 / 2 = 6.19$m

The distance from the buoyancy point (B) to the metacentre (M) can be found by:

$$\overline{BM} \cong \frac{I_{yy}}{V_w}$$

$$\overline{BM} = \frac{I_{yy}}{V_w} \cong \frac{\frac{1}{12} \cdot L \cdot B^3}{\nabla_{slab} + \nabla_{comp}} = \frac{\frac{1}{12} \cdot L \cdot B^3}{\{\nabla_{slab}\} + \{(\text{d}-\text{d}_{slab}) \cdot (L_{comp} \cdot B_{comp})\}}$$

$$= \frac{\frac{1}{12} \cdot 24.35 \cdot 15.65^3}{\{229\} + \{(15.65 - 0.60) \cdot (23.45 \cdot 12.30)\}} = 1.70 \text{ m}$$

The metacentric height becomes:

$$h_{metacentre} = \overline{KB} + \overline{BM} - \overline{KG} = 6.18 + 1.70 - 7.07 = 0.81 \text{ m} \rightarrow \text{ larger than } 0.50\text{m}$$

At this point, no free surface effect has been considered for calculating the metacentric height. Free water in the compartments has a destabilising effect on the stability of the caisson, but the decrease of the metacentric height shall be less than 0.30 metre. Therefore, sufficient metacentric height shall remain, also when the free surface effects is included.

I.4. Operational stability

The verification of actions and safety factors are shown in the tables below:

Overturning verification	Situation	Verification	Factor of safety*
Kern verification eccentricity max. 1/6 of foundation width	High water + vertical STS-crane load	SLS - GEO	1.4
Kern verification eccentricity max. 1/6 of foundation width	High water	SLS - GEO	1.9
eccentricity max. 1/3 of foundation width	Low water	ULS - GEO	2.2
eccentricity max. 1/3 of foundation width	High water	ULS - GEO	2.2
Equilibrium condition	Low water	ULS - EQU	2.5
Equilibrium condition	High water	ULS - EQU	2.3

Sliding verification	Situation	Verification	Factor of safety*
Forward sliding	Low water + STS load	ULS - GEO	1.7
Forward sliding	Low water	ULS - GEO	1.4
Forward sliding	High water	ULS - GEO	1.6
Forward sliding	High water + STS load	ULS - GEO	1.3
Forward sliding	High water + STS crane load (storm)	ULS - GEO	1.6

*Factor of safety on top of Eurocode / British Standard requirements. A value of 1.0 is sufficient

J. Size and scaling aspects

J.1. Compartment scaling

The first caissons were designed to be immersed to -10.35m CD, while it is nowadays common to reach twice this depth. Increasing dimensions of a design is not a matter of increasing the height and width of all elements. It can be inefficient to further increase compartment dimensions. Reason for this is that shear can be transferred more efficiently by intermediate walls (J.2) and the shear capacity decreases for larger cross sections (J.3).

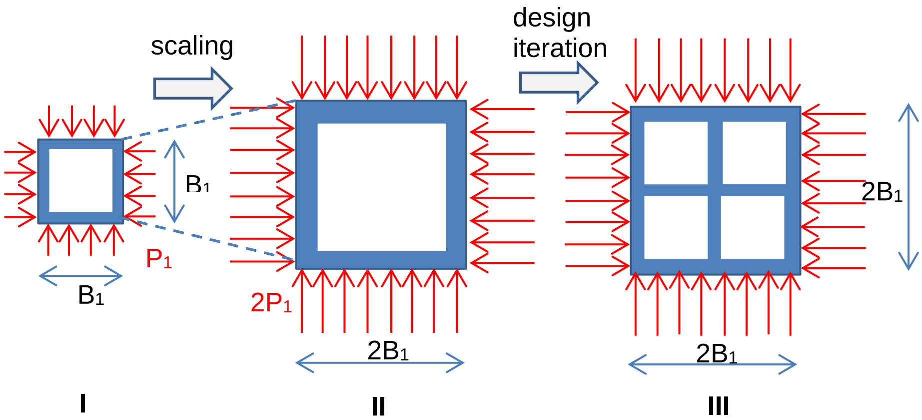


Figure J.1. Scaling a compartment to larger dimensions and an iteration

J.2. Shear stress

Considering the compartment elements depicted in figure J.1, the following simplified calculations can be made to evaluate the shear stress:

P_1 = 10.30 x 7.70 ≈ 80 kN/m^2
d_1= 250mm
B_1 = 2.50 metre
V_1 = 0.5 x 80 x 2.50 = 100 kN
v_1 = 100 x 10^3 / (250 x 1000) = 0.40 N/mm^2

The circumference is: 4B_1
Weight of a one metre high section ≈ 4 x 2.50 x 0.25 x 25 x 1.00 = 62.50 kN/m
Displacement of a one metre high section: B^2 x H = 2.50 x 2.50 x 1.00 = 6.25 m^3
Weight / displacement ratio: 62.50 / 6.25 = 10

When the compartment spans and immersion pressure are doubled, the effective compartment wall thickness must be increased by a factor four to obtain the same shear stress:

P_2 = 2P_1 =160 kN/m^2
d_2 = 1,000 mm
B_2 = 2B_1 = 5.00 metre
V_2 = 0.5 x 160 x 5.00 = 400 kN
v_2 = 400 x 10^3 / (1,000 x 1000) = 0.40 N/mm^2

The circumference is: 4x 2B_2
Weight of a one metre high section ≈ 4 x 5.00 x 1.00 x 25 x 1.00 = 500 kN/m
Displacement of a one metre high section: B^2 x 1.00 = 5.00 x 5.00 x 1.00 = 25.00 m^3
Weight / displacement ratio: 500 / 25 = 20

As can be seen, material consumption and weight increases by a factor 8, while the displacement increases by a factor 4.

When the compartments are subdivided by internal walls, materials can be saved. This can be seen by considering the following situation (III):

$P_3 = P_2 = 160$ kN/m^2
$d_3 = 500$ mm
$B_3 = 2B_1 = 5.00$ metre

Adding separation walls reduces the spans to half the compartment width (B). The shear force and stress remains unchanged:

$V_3 = 0.5 \times 160 \times 2.50 = 200$ kN
$v_3 = 200 \times 10^3 / (500 \times 1000) = 0.40$ N/mm^2

Weight of a one metre high outer walls ≈ 4 x 5.00 x 0.50 x 25 x 1.00 = 250 kN/m
Weight of one metre high inner walls: 2 x 5.00 x 0.25 x 25 x 1.00 = 62.50 kN/m
Total weight: 312.50 kN/m

Displacement of a one metre high section: B^2 x 1.00 = 5.00 x 5.00 x 1.00 = 25.00 m^3
Weight / displacement ratio: 312.5 / 25 = 12.50

Therefore, the material consumption can be reduced by adding separation walls.

J.3. Shear capacity

Following the regulations provided by the EN-1992, a minimum shear capacity can be calculated. The minimum depends on the applied concrete quality and effective depth. The capacity can be increased by including the dowel function of regular reinforcement bars, thus the values below provide conservative values.

$$v_{\min} = 0.035 \cdot k^{3/2} \cdot f_{ck}^{\ 1/2}$$

$$k_1 = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{250}} = 1.90$$

$$k_2 = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{1000}} = 1.44$$

$$f_{ck} = 35 \text{ N/mm}^2$$

$$v_{\min,1} = 0.035 \cdot 1.90^{3/2} \cdot 35^{1/2} = 0.54 \text{ N/mm}^2$$

$$v_{\min,2} = 0.035 \cdot 1.63^{3/2} \cdot 35^{1/2} = 0.36 \text{ N/mm}^2$$

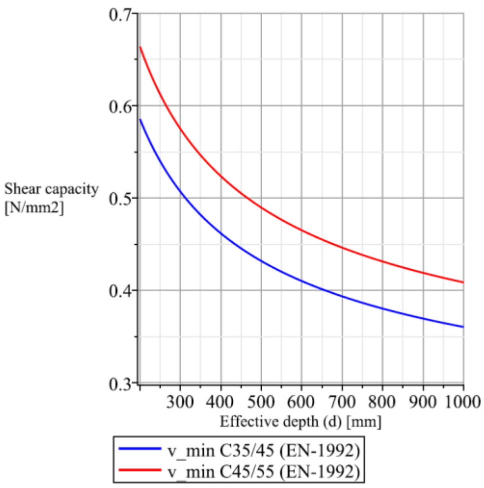


Figure J.2. Minimum shear strength concrete cross-sections (EN-1992)

J.4. Floating stability

The draught of a floating object in horizontal position increases by width (b) increments, while the draught (d) reduces when the same object is considered in vertical position. The comparison between floating positions is schematized in figure 6.9. Besides draught considerations, floating transport of light-weight slender objects can be limited by stability requirements.

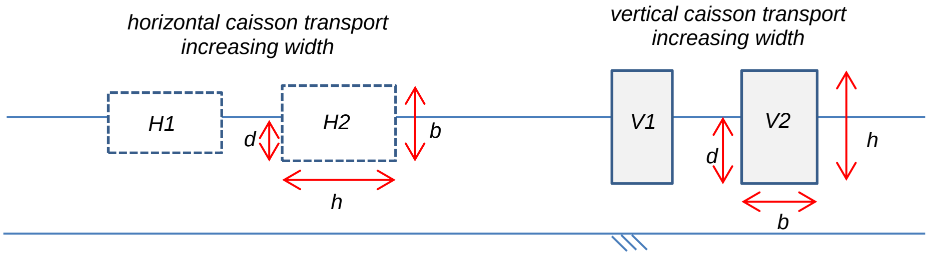


Figure 6.9. Caisson transport-shape relation

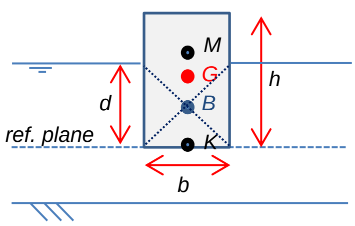
When rectangular floating objects are considered, such as presented in figure 6.9, the limiting width for intrinsic floating stability can be calculated. The point for which a vertical floating element, having generalized dimensions, is considered in the next sections. The relevant properties (appendix G and I) of the analysed rectangular caissons are:

Caisson properties	height (h)	width (b)	excluding ballast		including ballast	
			draught (d)	dist. KG	draught (d)	dist. KG
Slender rectangular caisson (section 6.3)	18.00m	12.60m	10.70m	7.90m	12.60m	7.00m
Wide rectangular caisson (section 6.2)	18.00m	15.65m	10.60m	7.50m	12.40m	7.10m

Table 6.6. Relevant caisson properties for floating stability

The objective of the following analysis is to clarify the stability region for rectangular floating objects. This region defines the required relative weight and width of a caisson for stable transport. Note however that the analysis is performed with averaged and rounded values, which makes the presented outcome applicable for preliminary purposes only.

A generalized rectangular floating object (caisson) is considered using the notations shown in figure 6.10. Dimensional parameters are denoted by lower case letters (h,d, and b), while the stability parameters are denoted by upper case letters (K, B, G, M). Distances from the bottom of the caisson (K) are denoted as for instance KG, which implies the distance from keel to gravity centre.



Notation	Description
h	Height of caisson
d	Draught of caisson
b	Width of caisson
K	Keel (bottom of caisson)
B	Buoyancy point
G	Centre of gravity
M	Metacentric height

Figure 6.10. Notations for floating stability analysis

The primary requirement for floating stability is a positive metacentric height (M above G). This height is influenced by the width (b), draught (d) and height of point G. The height of the centre of gravity varies when ballast water is added. The essential variables are therefore:

(4) draught;
(5) width;
(6) height of centre of gravity.

This allows us to define relations between these elements which results in a clarified stability region for rectangular floating objects. The draught and width are considered to be most important design aspects and therefore taken as variables. The height of the centre of gravity is kept as a constant and considered for the unballasted and ballasted situations.

J.4.1. Intrinsic stability of a rectangular floating object

The stability can be analysed for generalized objects by defining dimensionless parameters. The parameters are chosen to be related to the total height of the floating object in order to obtain an outcome which is interpretable for different caisson dimensions.

The relative draught of the caissons varies between:

$$\left.\begin{array}{l}\text{slender caisson: } d/h = 10.70/18.00 = 0.59 \\ \text{wide caisson: } d/h = 10.60/18.00 = 0.59\end{array}\right\} \rightarrow \text{mean value } 0.59$$

The relative width of the caissons varies between:

$$\left.\begin{array}{l}\text{slender caisson: } b/h = 12.60/18.00 = 0.70 \\ \text{wide caisson: } b/h = 15.65/18.00 = 0.87\end{array}\right\} \rightarrow \text{significant difference, not combined}$$

The relative draught parameter is denoted as x (horizontal axis), while the relative width parameter is denoted as y (vertical axis). Based on these notations, a stability graph can be plotted.

The location of the centre of gravity (G) differs for the considered rectangular caissons. The relative position of the centre of gravity initially varies between:

$$\left.\begin{array}{l}\text{slender caisson: } \overline{KG}/h = 7.90/18.00 = 0.44 \\ \text{wide caisson: } \overline{KG}/h = 7.50/18.00 = 0.42\end{array}\right\} \rightarrow \text{mean value } 0.43$$

Stability conditions

The floating object is stable when:

$$\overline{KM} > \overline{KG}$$

And the defined distance KG is:

$$\overline{KG} = 0.43h$$

The centre of buoyancy (B) from the keel of the caisson can be described as:

$$\overline{KB} = 0.5d$$

The metacentric height (BM) is:

$$\overline{BM} = \frac{I_c}{V}$$

In which:

$$V = b \cdot d \cdot l$$

$$I_c = \frac{1}{12} \cdot l \cdot b^3$$

Thus, distance KM is:

$$\overline{KM} = \overline{KB} + \overline{BM}$$

$$\overline{KM} = 0.5d + \frac{1/12\ l\ b^3}{b\ d\ l}$$

$$\overline{KM} = 0.5d + \frac{b^2}{12d}$$

When the parameters are combined, the stability can be verified by:

$$\overline{KM} > \overline{KG}$$

$$\frac{1}{2}d + \frac{b^2}{12d} > 0.43h$$

$$\frac{1}{2}d + \frac{b^2}{12d} - 0.43h > 0$$

Stability formulation

The critical stability condition can be found when the inequality is changed to an equality. The formulation can then be rewritten in terms of x and y by applying the following steps:

$$\frac{1}{2}d + \frac{b^2}{12d} - 0.43h = 0$$

$$y = \frac{d}{h}$$

$$x = \frac{b}{h}$$

$$\frac{x}{y} = \frac{b}{d}$$

$$\frac{1}{2}y + \frac{1}{12}\frac{x^2}{y} - 0.43 = 0$$

The formulation can now be plotted as the following elliptical curve (fig. 6.12):

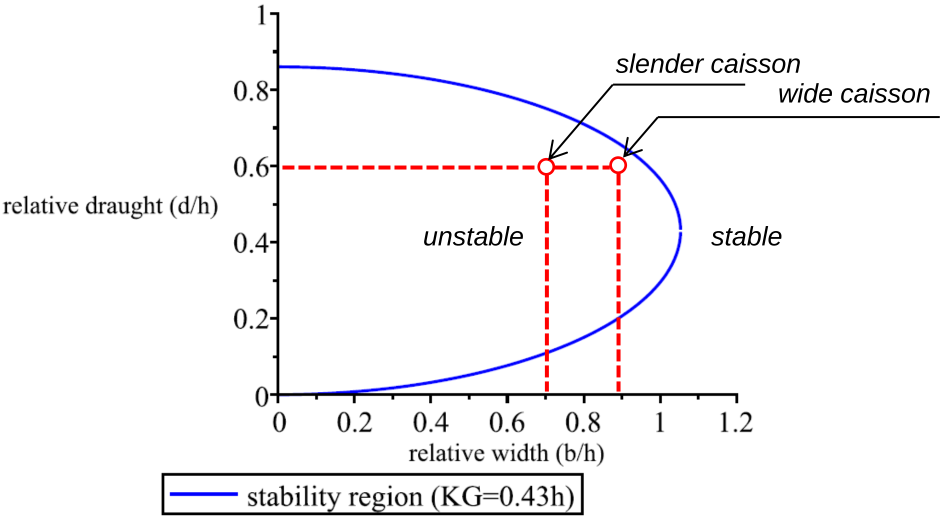


Figure 6.12. Stability of the considered rectangular caissons (unballasted)

The relative draught of the caissons (0.59) and relative width (0.70 and 0.87) can be found from the particular intersection points. These points are indicated by red dotted lines in the stability region of figure 6.12. It can be seen that both caissons are instable without adjustments. However, the wide caisson is almost stable by itself. This point is already located near the blue boundary.

J.4.2. Ballasted stability of a rectangular floating object

A similar approach can be used for the analysis of the floating stability of ballasted caissons. Due to the weight increments, the centre of gravity reduces and the draught increases. This results in the following change of parameters:

$$\left.\begin{array}{l}\text{slender caisson: } \overline{KG} / h = 7.00/18.00 = 0.39 \\ \text{wider caisson: } \overline{KG} / h = 7.10/18.00 = 0.39\end{array}\right\} \rightarrow \text{mean value } 0.39$$

The relative draught of the caissons varies between:

$$\left.\begin{array}{l}\text{slender caisson: } d / h = 12.60/18.00 = 0.70 \\ \text{wide caisson: } d / h = 12.40/18.00 = 0.69\end{array}\right\} \rightarrow \text{mean value } 0.70$$

The stability region for these values is presented in figure 6.13. It can be seen that the slender caisson is just outside the boundary and therefore has a positive metacentric height. However, the wide caisson has considerably more stability and does therefore require less adjustments for transport.

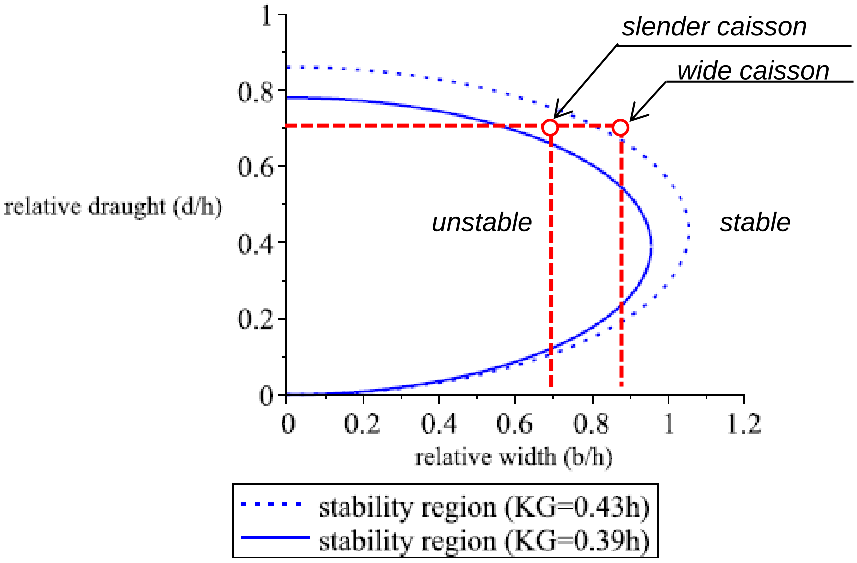


Figure 6.13. Stability of the considered rectangular caissons (ballasted)

K. Silo pressure

K.1. Janssen pressure theory

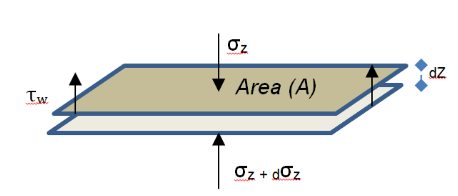
The Janssen (1895) theory is generally applied for calculating pressures of bulk solid materials within silos. The Janssen's theory is derived under the assumption that two parallel, rigid vertical walls retain granular soil and that the settlement of the soil is large enough to fully induce friction between the walls and the soil. It follows that the weight of the element is partially supported by the frictional resistances at the walls. In addition, the following assumptions and simplifications are made in order to derive the expression:

- Symmetrical shape of the horizontal cross section;
- The volumetric weight of the soil / bulk material is constant over the depth and width;
- Full wall friction is developed against the wall at every point. The mean shear stress is related to pressure (σ_y) through the friction coefficient (μ). This results in the relation:
  $\tau_w = \mu\ \sigma_y$;
- Pressure (p) is related to the mean vertical stress (σ_z) by a lateral pressure relation k (Rankine's theory). This results in the relation p=k σ_y;

A = area, U = perimeter, k = neutral soil pressure coefficient, μ = friction coefficient (equal to tan(δ)), γ_s = volumetric weight of soil.

K.2. Derivation of the Janssen pressure theory

Considering these aspects, vertical equilibrium on a slice of soil in a compartment results in:



Vertical equilibrium scheme for the derivation of the Janssen theory

$$(\sigma_z + d\sigma_z)A + U\tau dz = \sigma_z A + \gamma_s A dz$$

Which can be simplified to:

$$\frac{d\sigma_z}{dz} + \frac{U}{A}\tau = \gamma_s$$

$$\frac{d\sigma_z}{dz} + \frac{U \cdot \mu \cdot k}{A} \cdot \sigma_z = \gamma_s$$

Boundary condition:

$$\sigma_z(0) = 0$$

Integrating factor lambda:

$$\lambda = \frac{U \cdot \mu \cdot k}{A}$$

Multiplication by exp{lambda z} results in:

$$e^{\lambda z}\frac{d\sigma_z}{dz} + e^{\lambda z}\lambda\sigma_z = \gamma_s e^{\lambda z}$$

$$\frac{d}{dz}(e^{\lambda z}\sigma_z) = \gamma_s e^{\lambda z}$$

$$e^{\lambda z}\sigma_z = \int \gamma_s e^{\lambda z} dz = \frac{1}{\lambda}\gamma_s e^{\lambda z} + C_0$$

$$\sigma_z(z) = \frac{1}{\lambda}\gamma_s + C_0 e^{-\lambda z}$$

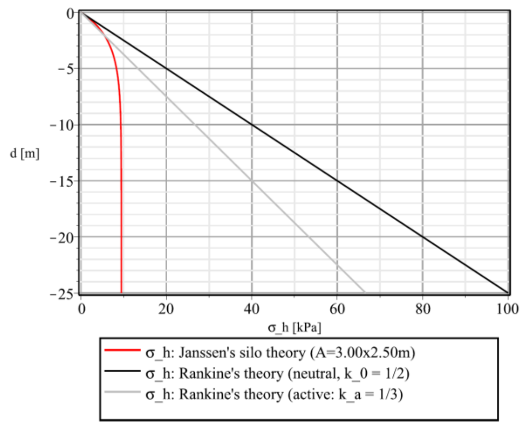


Figure K.1. Example of a horizontal soil pressure distribution in a compartment according to Janssen's theory compared to neutral and active soil pressure

$$\sigma_z(0) = \frac{1}{\lambda}\gamma_s + C_0 e^{-\lambda 0} = 0$$

$$\frac{1}{\lambda}\gamma_s + C_0 = 0$$

$$C_0 = -\frac{1}{\lambda}\gamma_s$$

$$\sigma_z(z) = \frac{1}{\lambda}\gamma_s - \frac{1}{\lambda}\gamma_s e^{-\lambda z} = \frac{\gamma_s}{\lambda}(1-e^{-\lambda z})$$

Which can be further simplified by redefining lambda to z_0:

$$z_0 = \frac{1}{\lambda} = \frac{A}{\mu \cdot k \cdot U}$$ *(the Janssen reference depth)*

$$\sigma_z(z) = z_0 \gamma_s (1 - e^{-z/z_0})$$ *(general expression for vertical soil pressure in silo's)*

Hence, the effective horizontal soil pressure σ'_z can be expressed by the Janssen theory:

$$\sigma'_h(z) = k_n \cdot \gamma'_s \, z_0 (1 - e^{-z/z_0})$$

$$z_0 = \frac{A}{\mu \cdot k_n \cdot U}$$

And including a vertical live load, the formula becomes:

$$\sigma'_h(z) = k \cdot \gamma'_s \, z_0 (1 - e^{-z/z_0}) + q \cdot e^{-z/z_0}$$

In which:

Z_0 = reference depth (Janssen (1895))
A = compartment area
U = perimeter of the compartment
k_n = neutral soil pressure coefficient (0.5)
μ = friction coefficient (equal to tan(δ) ≈ 0.4)
γ'_s = effective weight of soil (10 kN/m^2)

The compartments are schematized as rectangular cells with a particular width B. Also a perimeter (U) and area (A) are defined as depicted below.

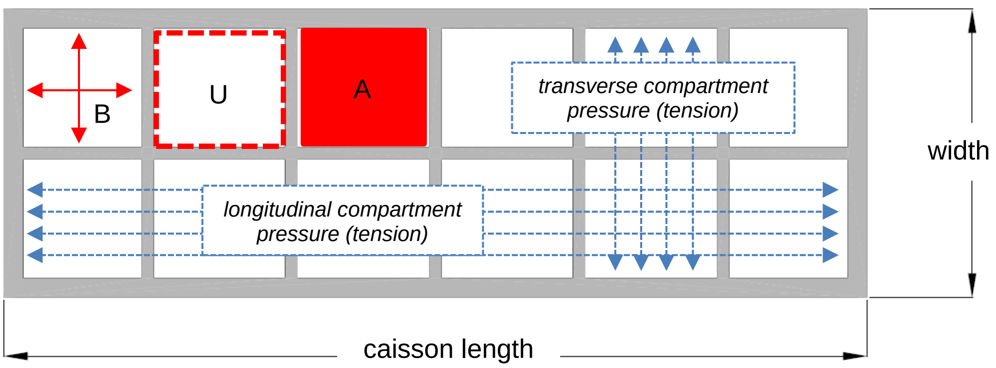


Figure K.2. Caisson compartments and parameters

K.3. Alternative silo pressure theories

The Janssen pressure can be seen as lower bound value due to the assumption of full friction. The graph below (fig. A6) shows the Janssen pressure (red) compared to other theories and regulations. Under specific conditions, the German DIN 1055-6 is even more conservative than the original Janssen expression, while it is actually based on the same principle. This difference is considered to be negligible for caisson design. The graph below also shows dynamic (filling / emptying) pressures which can be significantly higher. This is not from importance for caisson design since the soil is considered static during its service live. The dynamic effects which occur during the filling phase of the compartments shall be considered independently.

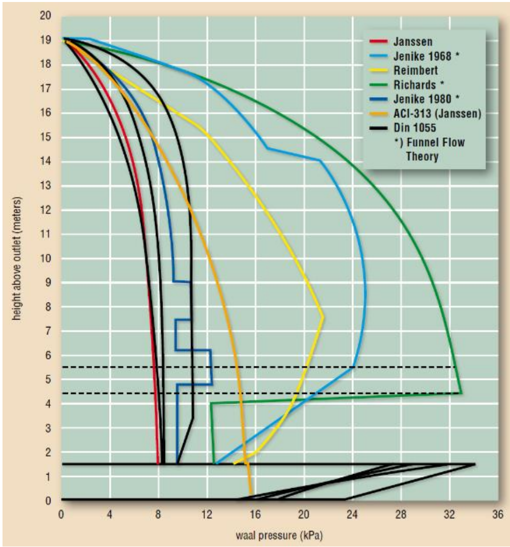


Figure K.3. Silo pressures according to different theories and standards, ref [A11]

K.4. Caisson compartment pressure

The Janssen pressure is calculated for various compartment dimensions. The minimum pressure is without live load (e.g. for cases in which the superstructure transfers load), the maximum pressure is calculated with a live load of 30.00 kN/m^2 (which is prescribed on the apron side).

soil pressure at bottom of structure	reference depth	excl. live load	incl. live load
	$z_0 = \dfrac{A}{\mu \cdot k \cdot U}$	$\lim_{z\to\infty} \sigma'_h(\mathrm{z}) = k_n \cdot \gamma_s z_0$	$\lim_{z\to\infty} \sigma'_h(\mathrm{z}) = k_n \left(\gamma_s z_0 + \mathrm{q}\right)$
A = 2.50 x 2.50m	3.13 m	15.63 kN/m^2	30.63 kN/m^2
A = 3.00 x 3.00m	3.75 m	18.75 kN/m^2	33.75 kN/m^2
A = 3.50 x 3.50m	4.40 m	21.90 kN/m^2	36.90 kN/m^2
A = 4.00 x 4.00m	5.00 m	25.00 kN/m^2	40.00 kN/m^2

Serviceability Limit State
Two compartment pressure states are considered for SLS conditions; a low water table and a high water table. These are assumed to be identical to the hydrostatic pressure difference on the front- and backwall. Using the expression, the compartment pressure for a 3.50 x 3.50m compartment including live load becomes:

$P_{J,max}$ = 36.90 kN/m^2.

The maximum water pressure for 1.00 metre head difference is calculated as:

$P_{ti,1} = \gamma_w \times \Delta H = 10.30 \times 1.00 = 10.30$ kN/m^2

Hence, the total compartment pressure for this geometry amounts:

$P_{J,SLS} = P_{J,max} + P_{ti} = 36.90 + 10.30 = 47.20$ kN/m^2

Ultimate Limit State
Similar to the SLS compartment pressure calculation, two pressure states are considered; a low water table and a high water table. The compartment water levels are:

The maximum water pressure of the 1.50 metre head difference is calculated as:

$P_{ti,2} = \gamma_w \times \Delta H = 10.30 \times 1.50 = 15.45$ kN/m^2

The total compartment pressure for this particular geometry therefore amounts:

$P_{J,ULS} = \gamma_G P_{J,} + \gamma_s Q_S + \gamma_{ti} P_{w,c} = 1.35 \times 21.90 + 1.50 \times 0.5 \times 30 + 1.50 \times 15.45 = 75.24$ kN/m^2

L. Soil pressure states and models

L.1.Soil pressure states

The smallest horizontal earth stress value occurs in active state. Its limiting lower bound value can be calculated by the Mohr-Coulomb failure criterion. However, the active state only occurs when the element is moving away from the soil. From a purely scientific point of view, the lateral stress against a rigid retaining wall remains unknown until deformation has been considered.

If the horizontal displacements are practically zero, a neutral stress state occurs. In a linear elastic material and under the assumption that the horizontal stresses σ_{xx} and σ_{yy} are equal, the following ratio between vertical and horizontal stresses can be found:

$$K_e = \frac{\nu}{1-\nu}$$

Where ν stands for the Poisson's ratio and can vary between 0.15 and 0.45 for granular soils[26]. The ratio varies significantly among different soils and various aspects play a role for quantification. Nevertheless, a value of 0.30 seems to be appropriate for medium dense sand and gravel. Point loads and line loads (SLS and ULS) shall be considered using elastic soil theories of Boussinesq (1885), Flamant (1892) and Fadum (1948), which are described in the sections regarding point and line loads.

A linear elastic model is not the best estimate since soil is not an elastic material and the history of stress development in soil can affect the stress state more dominantly (Verruijt 2012). Nevertheless, the upper bound value for a neutral stress state can be found to be 1. In practice, the neutral stress state seems to be largely depending on the friction angle of soil (ϕ). Without having a well-substantiated scientific basis, the K_0 value can be estimated by the formula proposed from Jaky (1948), which is:

$$K_0 = 1 - \sin\phi$$

Following this reasoning, a neutral stress state is considered for the serviceability limit state design. At this state, no large deformations are allowed. The caisson itself is thereby expected to be rigid. The active soil pressure state is only considered when designing for ultimate limit state. In this case, large deformations are allowed. In terms of a caisson being a gravity based structure, movement is likely to occur before failing.

L.2. Global stability: Rankine's theory

Besides the well-known Coulomb theory, Rankine developed a different approach in 1857. He extended earth pressure theory by deriving a solution for a complete soil mass in a state of failure. It can be used as a rather simple method to verify equilibrium of an L-wall and therefore commonly used.

The theory can only be used for cohesionless (granular) soils and stiff soils. Also, a complete failure wedge must be formed and the resulting force must be parallel to the ground surface. The required conditions regarding soil properties can be satisfied for particular projects. The other conditions are affected by the shape of the structure.

For reinforced concrete L-walls, there will be almost no movement of soil relative to the back of the wall. A virtual plane can then be considered and Rankine's theory can be applied properly. A full soil wedge can only be formed, when the heel width satisfies inequality:

[26] The Civil Engineering Handbook, second edition, Chen, W.F., Richard Liew, J.Y., CRC Press, 2003

$$B \geq H \tan\left(45° - \frac{\phi'}{2}\right)$$

For example, for soils with an angle of shearing resistance ϕ' of 30 degrees, the inequality reduces to approximately B > 0.6H. If this inequality is not satisfied, the thrust wedge (fig. 4.5. triangle ACD) is interrupted by the retaining wall itself. This causes Rankine's theory to be invalid for retaining structures with short heels.

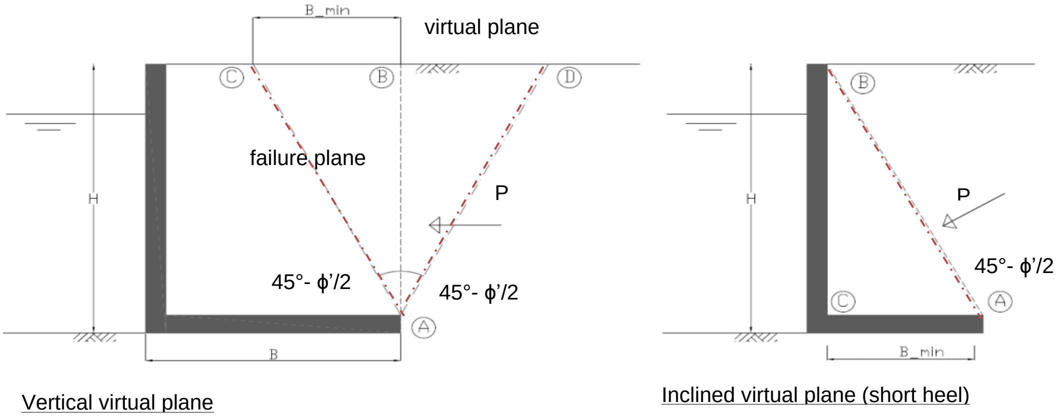


Figure L.1. Soil wedge for cantilever walls (left: Rankine situation, right: short heel)

L.3. Global stability: hybrid soil pressure model

Since Rankine's method is invalid for structures with short heels, other methods are developed to calculate soil pressures and thrust. The correct soil pressure can be found by using Coulomb's approach in terms of limit equilibrium[27]. This is however a relatively complex iterative process. A more simplistic method is described by Vandepitte[28], who divides counterfort walls with a short heel into two sections. The uninterrupted section can be calculated according to Rankine's theory, where the interrupted zone can be calculated as separate action on the wall.

The method described by Vandepitte is used in order to calculate the horizontal thrust on structures with a short heel. The lower zone is assumed to be a *trapped* soil wedge in neutral soil pressure state. Here, a Rankine pressure state is assumed to prevail. The higher region is assumed to be an active soil pressure state in which wall friction can be included.

Failure of the overturning caisson with s short heel is presented in the following drawings.

[27] Active earth thrust on cantilever walls with short heel, Greco (2001)
[28] Berekenen van constructies – Bouwkunde en Civiele Techniek, D. Vandepitte (1979)

A = active pressure zone
N = neutral pressure zone

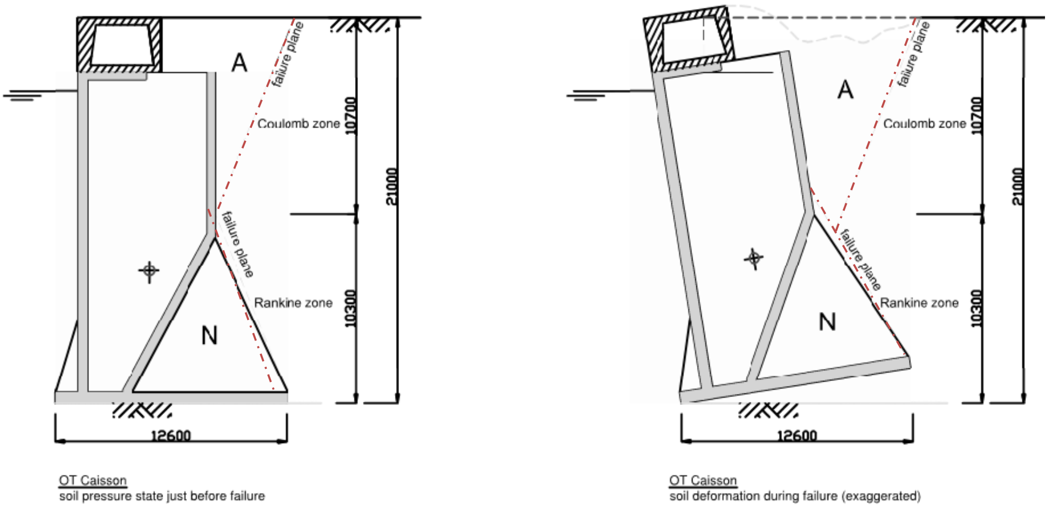


Figure L.2. Soil pressure states for an overturning caisson

The soil pressure on an overturning caisson can therefore be larger than the maximum pressure on a rectangular caisson. In case of deformation of a rectangular caisson, a full active pressure state can be formed.

To overcome the pressure increase due to a trapped wedge, an opening can be made in the baseplate. This is basically the principle of a Camilla caisson (1970), which is also addressed in appendix A. This type of caisson would experience the least amount of pressure, since the declination is beneficial in lowering the Coulomb stress.

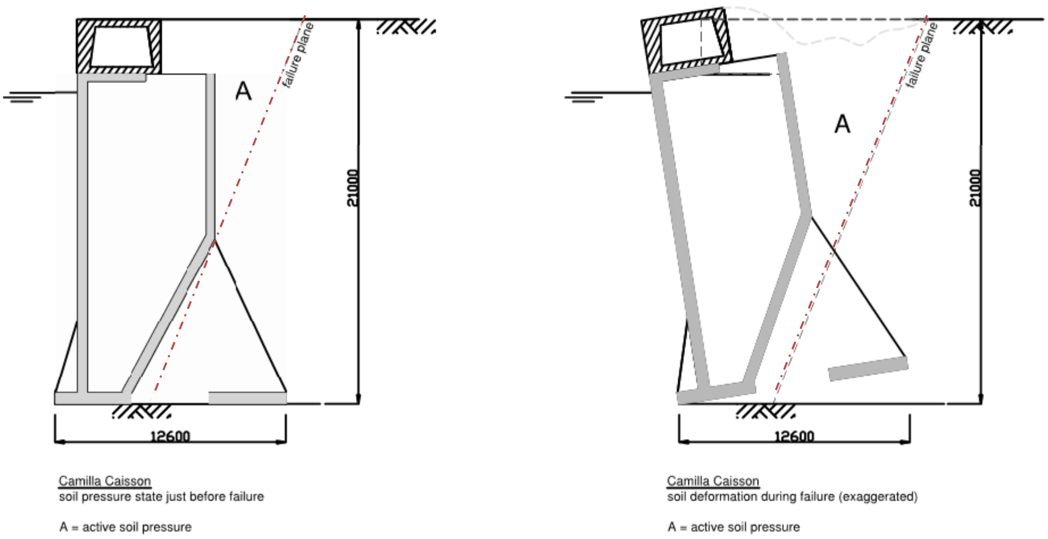


Figure L.3. Soil pressure states for a Camilla caisson

L.4. Behaviour of soil retaining walls

The behaviour of soil retaining walls depends on many different aspects. The previously described hybrid soil pressure model is intended to be a proper representation of reality, however, experiments indicate that also the stiffness of the foundation bed influences the horizontal soil pressure. Huang and Luo[29] found that the K factor increases significantly when the subgrade stiffness decreases. The measured lateral thrust was in some cases even greater than the soil pressure state at rest (K_0). However, if the subgrade is non-yielding ($k_v = \infty$), the found lateral pressure is similar to the active pressure state.

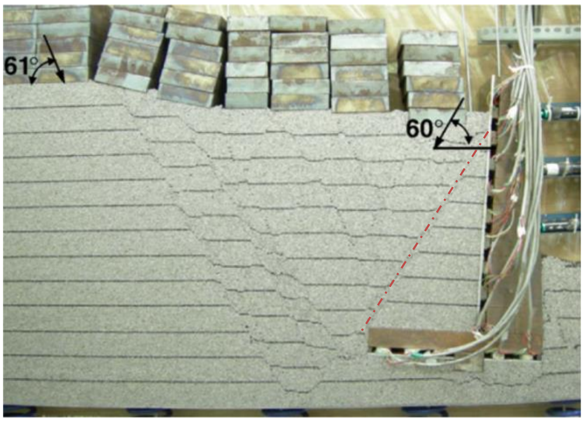


Figure L.4. behaviour of a cantilever wall at failure

L.5. Overview of design methods

The following K values are obtained for different soil pressure states and models:

Pressure state	Formula for the lateral earth pressure coefficient (K)	Value for ϕ = 30°	Value for ϕ = 45°
Elastic	$K_e = \frac{\nu}{1-\nu}$	0.30	0.30
Neutral	$K_0 = 1 - \sin\phi$	0.50	0.29
Active (without friction)	$K_a = \frac{1-\sin\phi}{1+\sin\phi}$	0.33	0.17
Active (with friction)	$K_{a,f} = \frac{\cos^2(\phi+\alpha)}{\cos^2(\alpha)\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\alpha-\delta)\cos(\alpha+\beta)}}\right)^2}$	0.28	0.14

[29] Behavior of soil retaining walls on deformable foundations, Huang and Luo (2009)

For ULS verification, a fully neutral pressure state and the hybrid pressure state are verified. From these states, the highest value is considered for design. The active state is considered including all prescribed partial factors. The verification which comprises a neutral soil pressure state does not include partial factors. This failure mechanism is included due to the desire of low deformations. It is categorized as an ultimate limit state (ULS), if the loads appear to be higher than the active soil pressure state including partial factors.

One could argue that a load combination without partial factors must be categorized as a serviceability limit state. However, disregarding the terminology, the calculation outcome shall be identical.

Serviceability limit states

In terms of overturning stability for the serviceability limit state, in which the line of the resultant force may only be positioned within the kern of the section, an active soil pressure state is considered. The thrust on the lower part of the retaining wall is considered according to Rankine’s approach in case of the overturning concept.

The essence of kern verification is guaranteeing bearing pressure over the complete foundation and thereby avoiding a gap between the foundation to occur. Loss of foundation pressure can only occur if the soil pressure state is active. A gap caused by overturning failure cannot occur simultaneously with a neutral soil pressure state, since rotation of the structure is required.

Ultimate limit states

Two pressure states are considered for the ultimate limit state verification. The first pressure state is partly active and includes partial factors. The second pressure state is fully neutral and does not include partial factors. For ULS verification it is assumed that the heel “traps” the soil, which implies that an active state cannot occur

M. Design and safety

M.1. Change of working height

Less working height might indicate an improved level of safety and thus less risk for personnel. The original caissons had a maximum compartment width of 3.00 metres. The compartments could therefore be constructed with a maximum working height of approximately 4.00 metres above ground level. This relatively low height and the lower safety standards in 1903 probably resulted in no (or limited) fall-protection for the labourers during the construction of the first caissons. From figure M.1., fall heights of less than 5 metres show a significant reduction in the probability of death. This was the case for the original overturning design.

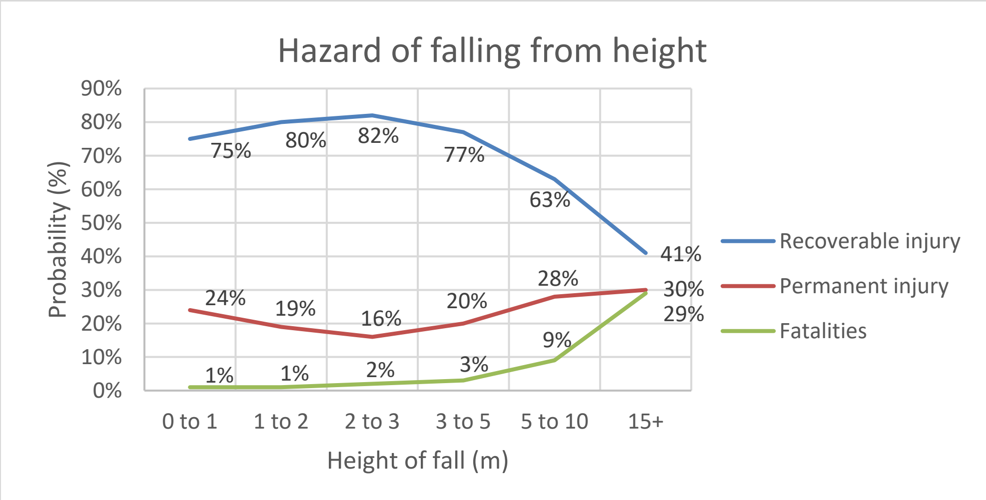


Figure M.1. Relation between fall-height [m] and the probability of recoverable injury (blue), permanent injury (red) and death (green) from labour accident data in the Netherlands (2003-2012)[30].

The largest factor which affects the degree of injury is thereby the height of the fall. This can theoretically be explained by the increasing kinetic energy (since the terminal velocity is generally not reached) of a person during a fall, which is transferred to the body when it touches a surface. Besides the theoretical background, also various studies of historical data show clear correlations. Height is not the only influencing factor for the degree of falling risks. It is among others affected by: fall-height, the surface of impact, fall-position, age, gender and body mass.

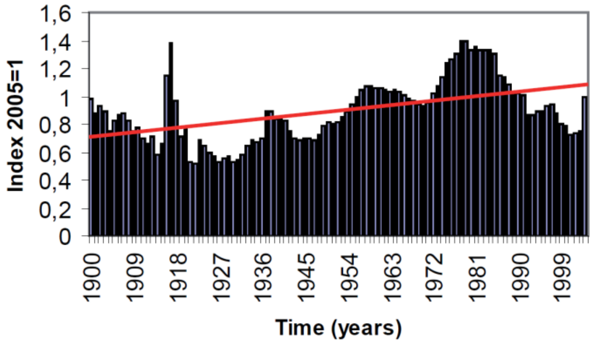
Concrete caissons are nowadays much larger than a century ago. Irrespective of the construction method (horizontally or vertically), the working height would be far above acceptable safety limits. A horizontal caisson construction method is therefore not likely to increase the level of safety intrinsically.

Unfortunately, fall hazard cannot be engineered out by applying the overturning concept. Measures must therefore be taken in order to keep the probability of a fall from height acceptably low. This can be in the form of a passive fall protection or active fall protection, on which the passive fall protection method is the most desired option. Passive systems, such as fencing and catching platforms, do not require special participation of the worker and does not hinder the freedom of movement. In case of the horizontal construction method, the length of the passive protection system must be increased since the perimeter of the work area is larger. Therefore, the lowered height of the construction method is not necessarily an advantage for safety of personnel.

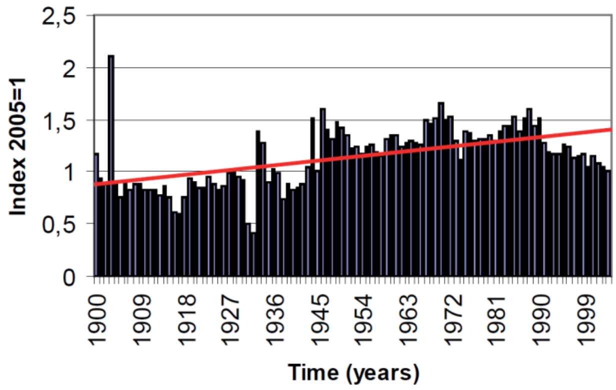
[30] Source: Health council of the Netherlands; falls from height (2013)

N. Cost deviations

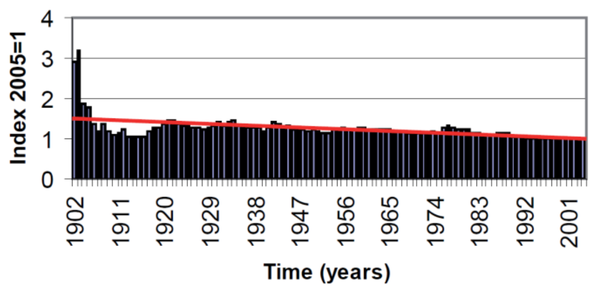
N.1. Historical cost deviations of building materials (1900-2005)



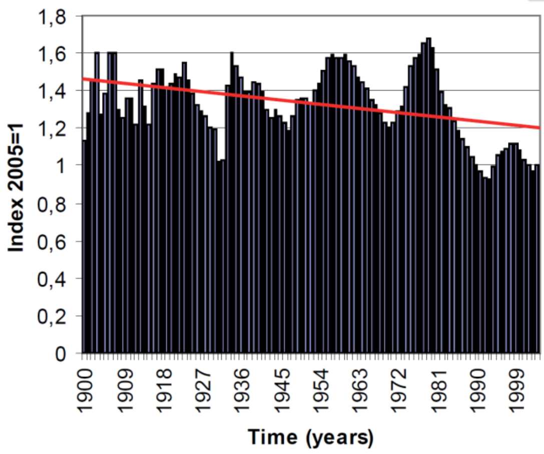
Steel index versus time (De Gijt, 2010)



Stone and riprap index versus time (De Gijt, 2010)



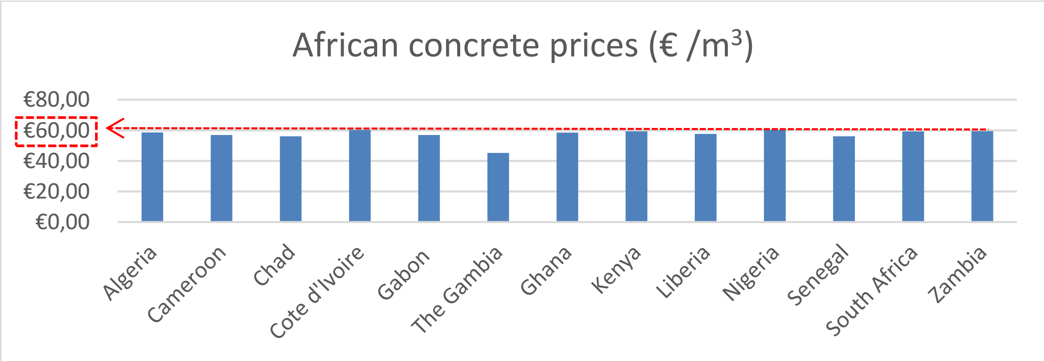
Sand and gravel index versus time (De Gijt, 2010)



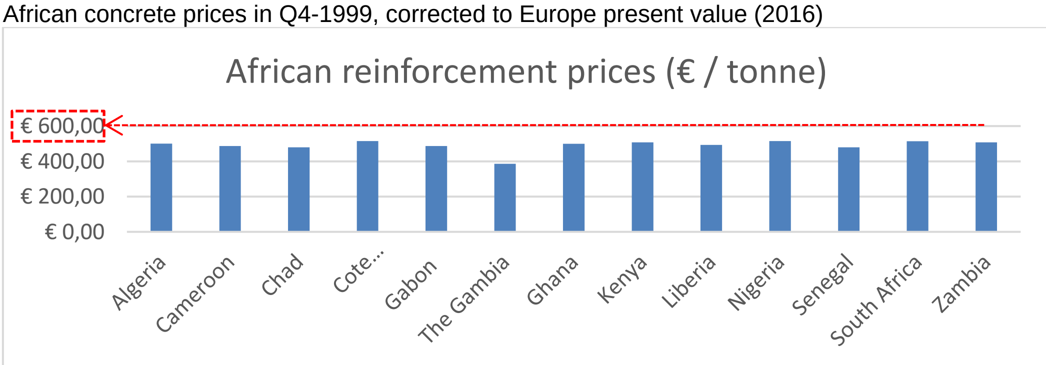
Cement index versus time (De Gijt, 2010)

N.2. Geographical cost deviations of building materials

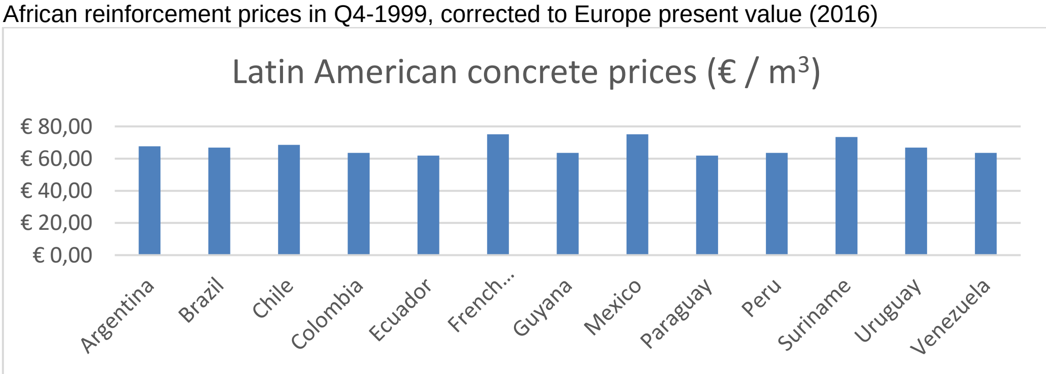
The cost deviations in the bar charts below are retrieved from Spon's African and Latin American Construction Cost Handbooks. The price rates include all necessary labour, plant and material costs for carrying out the operations. Price rates from the handbooks (1999) are corrected to 2016 values. The geographical cost deviations of building materials seem to be little and make the exact location of the project less interesting. Due to the relatively constant concrete and reinforcement prices, the feasibility of the overturning caisson shall not depend on a particular country.



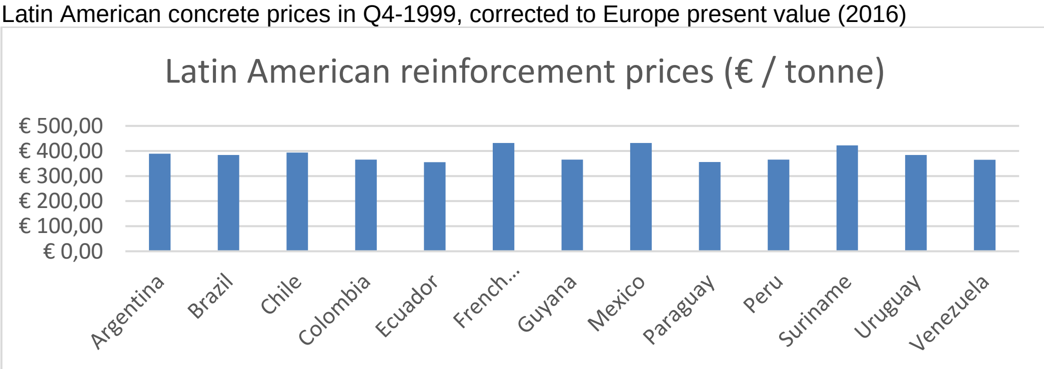
African concrete prices in Q4-1999, corrected to Europe present value (2016)



African reinforcement prices in Q4-1999, corrected to Europe present value (2016)



Latin American concrete prices in Q4-1999, corrected to Europe present value (2016)



Latin American concrete prices in Q4-1999, corrected to Europe present value (2016)

N.3. Cost deviations of heavy lifting equipment

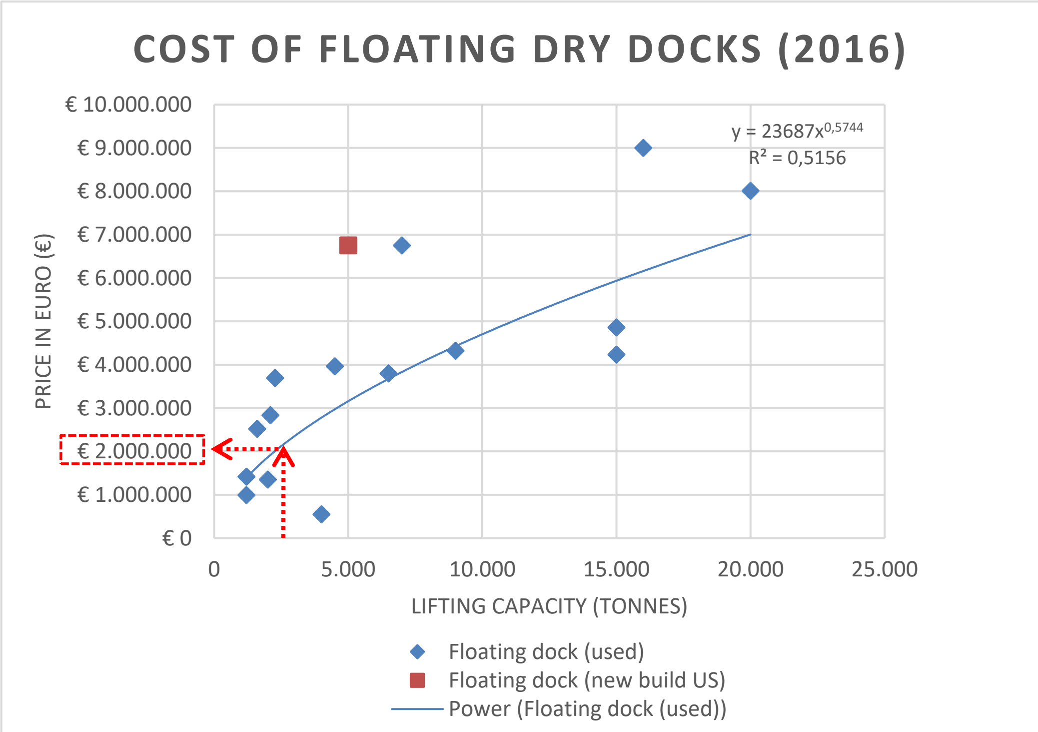
Weekly average rental rates in the UK and Ireland. The annual rental rate survey (2016) performed by www.vertikal.net.

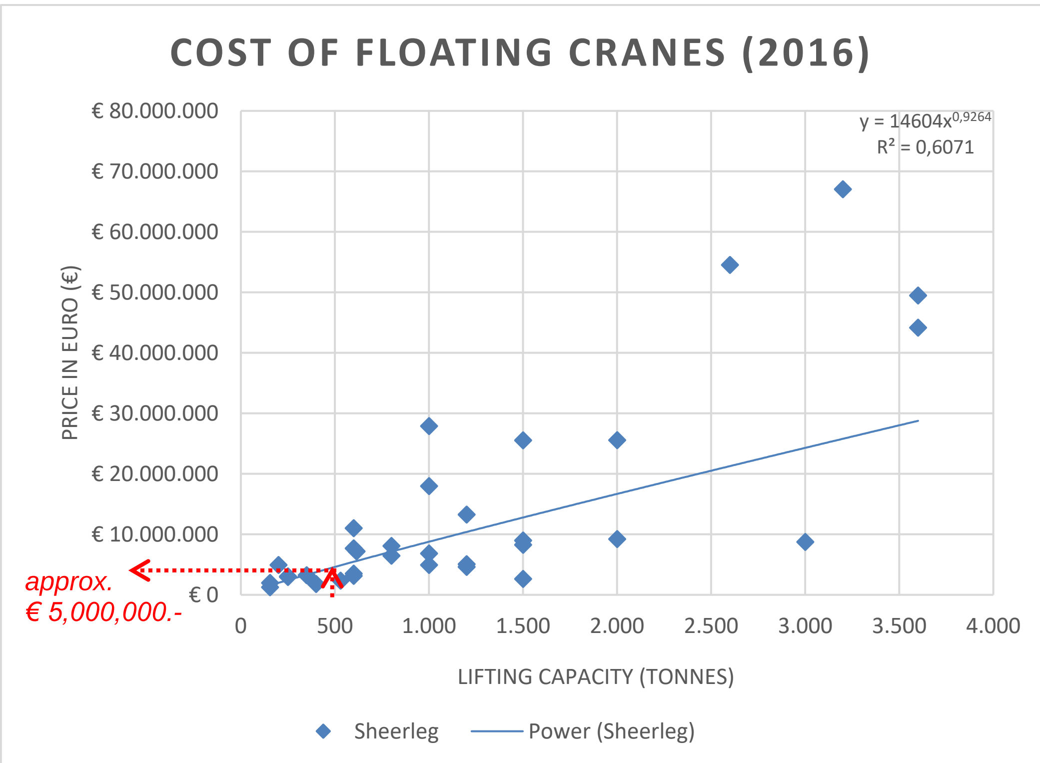
Crane type	Average rental cost per week
Tower crane <70 tm	€ 800.-
Tower crane <120 tm	€ 1,200.-
Tower crane <200 tm	€ 1,750.-
Tower crane <300 tm	€ 2,250.-
Crawler crane <50 t	€ 1,825.-
Crawler crane 50 - 60 t	€ 1,525.-
Crawler crane 70 - 80 t	€ 2,875.-
Crawler crane 90 - 100 t	€ 2,850.-
Crawler crane 120 - 150 t	€ 3,200.-
Crawler crane 180 - 250 t	€ 5,500.-

Estimate of procurement cost of tower cranes:

Crane type	Price (used)	Source / website (2017)
Tower crane Kroll K-1400 (40m reach)	€535,000,-	cranenetwork.com/crane/tower-cranes/kroll/k1400-1800/222107
Tower crane Liebherr 630EC-H 40 Litronic (80m reach)	€ 1,037,000,-	cranenetwork.com/crane/tower-cranes/liebherr/630-ec-h-40-litronic/211178?sc=3
Tower crane Kroll K-10000 (80m reach)	€ 3,220,000,-	de.machinerypark.com/obendreher-kroll-kroll-k-10000-gebraucht-lu-6686
Crawler crane Liebherr LR1250 (275t)	€ 658,000,-	cranenetwork.com/crane/crawler-lattice-boom-cranes/liebherr/lr1250/223101?sc=3
Crawler crane Liebherr LR1300SX (330t)	€1,468,000,-	cranenetwork.com/crane/crawler-lattice-boom-cranes/liebherr/lr-1300/209913?sc=3

Costs of floating dry-docks (FDD) and sheerlegs are determined by asking prices of online brokers, horizonship.com and workbargebrokers.com. Prices are retrieved from the websites in august 2016. Required prices for the overturning caisson are indicated by the red arrows.





Report Kraus citations

Report Kraus citations

The original report of prof. Kraus introduced the use of reinforced concrete as follows:

> *"Before treating of the manner of constructing these blocks, we will devote a few words to this material, which has not been employed in Chili, judging from the data at our disposal, but in the foundations of the work-yard of the dock of Talcahuano, but in Europe and the United States it has been used for works of all kinds, as: aqueducts, bridges, revetments, buildings, etc.*
>
> *The last Paris exposition, as also the one that has just taken place in Düsseldorf, have again demonstrated most plainly the great advantages of this material for construction which, as is well known, is simply a happy combination of iron and concrete, whereby it unites the supreme conditions of resistance, duration and incombustibility. Different systems for the construction of this mass are in existence, being known by the names of their inventors: Monier, Wayss, Rabitz, Matrai, Hennebique, Coignet and also many others, but they all resemble each other mutually in so far as that they have as principle an iron frame enveloped in concrete carefully made from materials of superior quality."*

It is interesting to notice that in this period of time, reinforced concrete was not commonly known. It was a highly innovative composite material which was even patented by their inventors. The different construction technology "systems" could only be used under licence of the involved firm or inventor.

As written in the original report:

> "*The quaywalls of this dock will be of the same type as those of the western side of the bay, that is to say, that they are formed of great cases of armed concrete with a superstructure of masonry work. The wall of the northern side of the enlarged part of the point will consist of great floating blocks analogous to those of breakwaters."*

The following is written in the report from the Commission Kraus:

> *"On launching the cases of armed concrete in a more or less horizontal position, their floating line will answer to the line Y-Y of drawing No 131, and their careening centre will be found at the point Q. Another position of stable equilibrium, and more or less vertical, answers to a submersion of the case to a depth of 7.20m."*

As in paragraph 102, Sheltered Piers is written:

> *"In view of the great length of these quaywalls and of the considerable costs entailed by their construction, many types of walls have been studied and mutually compared, in order to choose from among them the one offering the greatest advantages. After this preliminary work, the following type was adopted, same satisfying not only the conditions of resistance and of easy and safe execution, but also economical exigencies."*

This accentuates the advantages which had been obtained by application of the concept.

The report also provides notes on the calculation method and pressures on the foundation:

> *"It is supposed that the vertical pression of the rubble behind the wall, on the bottom-plate of the case of armed concrete, limits itself exclusively to the weight of the cubic comprised between the interior side of said wall and the vertical plane which answers to the inside edge of the plate.*"

And:

> *"Being distributed, according to the lineal law, the total vertical pression exercised by the base of the wall on the bottom, it results that said pression will be of 0.59 kg/cm^2 at the interior edge of the plate and of 3.50 kg/cm^2 at the exterior edge."*

Apparently, it is assumed that the weight of the backfill is accounted for up to a vertical virtual plane along the heel. The foundation pressure is calculated to be at most 350 kN/m^2 at the toe and 59 kN/m^2 at the heel of the structure.

Definitions

Definitions

Caisson A prefabricated floated-in quay wall structure with undefined shape. One can assume that a reinforced concrete caisson is implied, if no particular material is prescribed.

Box caisson or rectangular caisson A generalized term for caissons which have a rectangular shape. A box caisson is constructed vertically; in the same position as required for operational conditions.

Overturning caisson A generalized term for caissons which are constructed and floated horizontally. This type of caisson is, after transportation, turned at/near its final location. The term "*overturning caisson*" is literally translated from the Dutch word *"kantelcaisson"*. The word *overturning*, not to be confused with the overturning limit state which must be considered at final position, originates from the turning process in floating stage.

Horizontal construction The execution method on which the caisson is built or assembled with the front- or back wall in horizontal position.

Horizontal floatation The first floating position of an overturning caisson without ballast weight. In practice, the caisson might float in more or less diagonal position due to its asymmetrical shape.

Kraus caisson The original overturning caisson concept, designed by professor Kraus in 1903. Also referred to as overturning caisson. The economical L-shaped (counterfort) caisson which has been designed for sheltered quay walls.

Comision Kraus The commission who is responsible for the realization of the report "*Proyecto de Mejoramiento del Puerto de Valparaíso*" and therefore, but not exclusively, the establishment of the Kraus concept itself. Note that this particular design was one among many other state of the art concepts, which are extensively described in the report by Comision Kraus.

Technical feasibility The capability of building the concept according to current standards, bearing the influence of design changes with relation to costs in mind. The technical feasibility mainly focusses on the evaluation of opportunities and threats of the concept.

Economic feasibility The costs of the concept in relation to other quay structures, without necessarily all prerequisite knowledge of technical execution. The economic feasibility mainly addresses the quantification of (known) strength and weaknesses.

EQU limit state Loss of equilibrium of the structure or the supporting ground, considered as a rigid body. The internal strengths of the structure and the ground do not provide resistance.

GEO limit state Failure or excessive deformation of the ground, where the soil or rock is significant in providing resistance.

STR limit state Failure or excessive deformation of the structure, where the strength of the structural material is significant in providing resistance.

UPL limit state The loss of equilibrium of the structure by vertical uplift due to water pressures (buoyancy).

HYD limit state Hydraulic heave, internal erosion and piping in the ground as might be experienced.

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