# THE REUSE OF CAISSONS FROM THE PORT OF ROTTERDAM

Gemeente Rotterdam Havenbedrijf Rotterdam



# Port of Rotterdam Reuse of caissons

Ву

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in partial fulfilment of the requirements for the degree of

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## Preface

This report is the result of the research performed for my master Structural Engineering at the faculty of Civil Engineering and Geosciences at Delft University of Technology. The research is done in cooperation with the engineering department of Stadsontwikkeling Rotterdam and Port of Rotterdam Authority.

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### Abstract

The port of Rotterdam placed between 1950 and 1961, particularly in the Waalhaven and Merwehaven, caissons to serve as quay walls. The Waalhaven has 44 caisson quay walls with a total length of 1.921 meter. The Port of Rotterdam and the city have grown through the years. Due to the continuously increasing need for space from both the city and the port, they are grew towards each other. Some ports are now enclosed by urban areas of the city of Rotterdam. There is a need for a research on the opportunities to redevelop the older ports in the heart of the city, and therefore in the reuse of the caisson quay walls. The advantage of caissons, in general, is the wide-ranging applicability of those structures. This study was done to qualify the potential functions to reuse and provide the caissons of the Port of Rotterdam a second-life. For the caissons in the Waalhaven there are 10 functions discussed, including the null alternative, namely recycling the concrete material and reinforcement steel.

A literature study is done to obtain information on the types and history of caissons, the transportation of caissons over water and the related calculation method. Furthermore, the properties and long-time effects of the concrete and the condition of the caissons are considered. It was found that the compressive strength of the concrete is now a factor 3.84 times higher after 55 years than the (assumed) design strength. The reinforcement is assumed to have the same strength properties, but one should note that the reinforcement in some parts of the caissons, especially in the submerged and splash zones, are in a critical condition due to corrosion. This is partly due to the small concrete cover of 20 mm.

The caissons were not constructed with the idea to be uplifted and transported after they were placed. This is apparent from the connection between the caissons which is consists of vertical slots filled with concrete mortar and using steel hatches for inflow of water. However, the uplifting and transportation of the caissons are feasible by sawing the connection, using water pumps and protecting the surrounded area by sheet piles. For this operation the most likely risks are described. The rusted hatches for inflow of water are a serious point of concern because the water can flow freely into the caisson what makes uplifting impossible without closing the gaps. Also it is a very likely that the steel hatches are corroded due to the (salt) water.

One of the possibilities is to uplift and demolish the caissons to reuse the materials. The costs to uplift and demolish the caisson are estimated at  $\leq$  4.9 M. The demolition costs account for the largest part of the total costs, namely 35% of the total costs.



For the caisson in the Waalhaven the function as building foundation became, out of the Multi-criteria decision analysis, the most favourable option. This option is from economic point of view interesting, because the land value of the piers (and therefore the Waalhaven itself) also increases when buildings are built on the caissons. The maximum height of the building is determined by strength and stability calculations of the structure, which resulted in 12 floors including the ground floor. To counteract the splitting forces caused by the point loads of the columns, the wall needs to be strengthen by applying external reinforcement.

From this study it can be concluded that the caissons, at a first view, can be uplifted and transported to other locations and be reused for wide varying purposes. The condition of the caissons, having regard to the age and conditions, are in a relative good state. However, the corrosion of the reinforcement is critical, especially when exposed to an oxygen-rich environment. This study shows it is feasible to reuse the caissons as building foundation for a 12-floor building and in addition it improves also the land value of the area. It is recommended to do further research on the elaboration of the caissons as building foundation. Furthermore, research in the other potential possibilities could lead to more attractive solutions (i.e. economical, sustainable or innovative) or the applicability to combine functions.



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# Symbols

| $ ho_c$         | Concrete density   | [kN/m³] |
|-----------------|--|---------|
| $ ho_w$         | Water density  | [kN/m³] |
| $\sigma_c$      | Concrete stress  | [N/mm²] |
| f <sub>ck</sub> | Characteristic concrete cylinder<br>compressive strength | [N/mm²] |
| f <sub>cd</sub> | Design stress concrete                                   | [N/mm²] |
| $f_{yk}$        | Characteristic yield strength                            | [N/mm²] |
| $f_{yd}$        | Design stress reinforcement                              | [N/mm²] |
| γ <sub>c</sub>  | Material safety factor concrete                          | [-]     |
| $\gamma_s$      | Material safety factor steel<br>reinforcement            | [-]     |
| V               | Volume   | [m³]    |
| M <sub>Rd</sub> | Moment of resistance                                     | [kNm]   |
| V <sub>Rd</sub> | Shear force resistance                                   | [kN]    |
| $A_s$           | Reinforcement surface area                               | [mm²]   |
| $\gamma_Q$      | Partial safety factor live load                          | [-]     |
| $\gamma_G$      | Partial safety factor permanent load                     | [-]     |
| M <sub>ms</sub> | Moment at midspan  | [kNm]   |
| $M_{ms}$        | Moment at support  | [kNm]   |



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## 1. Introduction

#### 1.1 Case description

Nowadays, ports are in a continuous phase of change, overwhelmed with changing markets and demands. Emerging countries develop new maritime ports or expand and renew existing ones, offering high services and demanded requirements. Ships are getting larger, cargo tonnage increases and harbours need to adapt the services and infrastructure. Larger ships and increased tonnage demands adaptation of several structures, including quay walls. Larger keel clearance, by excavation of waterways and mooring places, is needed due to increase of ship dimensions. Heavier equipment to unload cargo requires a replacement of quay walls and further redevelopment of harbours.



Figure 1 Maasvlakte 2 in development (vanoord.com)

In the Port of Rotterdam quay walls made of caissons have been built between 1900 and 1950 and used for mooring vessels. The port of Rotterdam had grown through the history, leading to different expansions, both in size and demands. The older port areas are in an unremitting stage of redevelopment and adaptations to new requirements and guidelines.

The reuse of (old) caissons can be advantageous, especially when they can serve on temporarily base in projects. Two studies showed the usefulness of caissons as temporarily breakwaters (Mann, 1999; Spanjers, 1997). In the past caissons have been temporarily, like the example of Mulberry harbour. In this harbour the Allied Forces used in 1944, after the invasion in Normandy, huge concrete caissons for construction of breakwaters and piers to facilitate rapid offloading of cargo.

From an economic point of view, the reuse of structures can be interesting. When construction time is considered, it can give positive results to reuse existing structures, instead of building new ones.



Caissons can have a 'second life' in temporarily projects or as integral part of a larger structure that needs to be redeveloped or can be demolished.

In this respect, a study on the reuse of caissons is very useful and interesting to investigate feasibility and the possibilities. The central question in this study will focus on the feasibility to reuse the relevant caissons, and if so, the need of adaptations and transportation to a desired location. An investigation of the current situation as well as the current condition of the caissons will be part of this study.

Aspects that also have to be taken into consideration are the remaining service and technical lifetime and the costs that are involved. Those caissons date from the late 1950s and were designed according to the guidelines of that time. The service and technical lifetime of the caissons could have a decisive influence on the feasibility and reuse possibilities.

#### 1.2 Objectives and goals

The main goal of this study is to examine the feasibility to reuse caissons for other purposes than they were built for and the need of adaptations in order to meet requirements.

The objectives of this case study are:

- To investigate the possibilities to reuse old caissons that were mainly used as quay walls in the Port of Rotterdam for other purposes.
- To consider also the uplifting of the caissons on the current location and transportation to a suggested location if necessary.
- To determine the critical points of the structure for transportation and reuse.
- To provide design calculations for the reuse of the caisson quay walls.

The central question is if the caissons in the Port of Rotterdam can be reused for other purposes, and if so, what are the most promising functions. Before the central question can be answered, the answers on the following additional questions are essential.

#### **Additional questions**

- Which types of caissons are located in the Waalhaven and Merwehaven in the Port of Rotterdam? Where are they exactly situated?
- What characteristics possess these caissons in terms of concrete strength, remaining lifetime and current condition of the structure?
- What are the critical points of the caissons both structural and during uplifting?



- What are the costs for demolition of the caissons?
- Are there adjustments or repairs needed?
- What functions are appropriate for these caissons?
- What is the capacity of the caissons with respect to the selected function?

#### 1.3 Report structure

The outline of this report and approach of this study will be discussed in this section. For each chapter a brief summary will be described including its goal. The report consists of 9 chapters and the appendices.

Chapter 1 contains an introduction to the case, the scope and objectives of this study and its goals.

The second chapter outlines the history of the Port of Rotterdam, the harbours Waalhaven and Merwehaven and the history and position of the caissons in these harbours.

Chapter 3 provides a literature study to gain knowledge and understanding about the subject. The literature study is divided in four parts. The first part of the chapter begins with general information about caissons. The second part of the chapter discusses possible functions and examples of reuse of civil structures, with the focus on caissons. In the third part the focus lies on specific materials in relation to ageing. This to understand how materials, like concrete, behave after years and if the properties change in time.

Chapter 4 gives the hydraulic and geotechnical boundary conditions specified by the environment. It deals also with the elaboration on the structural aspects of one caisson. The goal is to verify the strength and capacity of the structure by calculations. To the fact that those caissons are built more than fifty years ago, impact of ageing of materials will also be considered.

Chapter 5 discusses the uplifting of the caisson in the Waalhaven in case when uplifting is necessary to fulfil a new function. The aim is to study the possibility to uplift and transport the caissons to another location and discuss possible bottlenecks during this operation.

Chapter 6 is linked to chapter 5 by treating the risks during uplifting of the caissons. The objective is to give insight in risks and oppose those risks by preventive measures.

Chapter 7 starts with the reuse possibilities of the caissons located in the Waalhaven, including removal costs as part of the null alternative. This part is followed by the possibilities that can be taken and limitations that can restrict the reuse of the caissons for some functions and/or locations.

In chapter 8 a defined multi-criteria analysis will give one function which is according the criteria the most suitable purpose. Moreover, the stability of the caisson and the concrete wall bearing capacity of



the caisson will be checked on critical issues in order to discuss the feasibility and potential maximum capacity.

The last chapter, chapter 9, the conclusion of this report will be given and will also offer recommendations for further studies and use of the caissons.





## 2. History of the Port of Rotterdam

#### 2.1 The Port of Rotterdam

Till 2002, the Port of Rotterdam was well-known as the busiest seaport of the world (measured in annual cargo tonnage). Anno 2013 it is still the largest and main entrance of Europe for shipping, and one of the key ports of the world. The Port of Rotterdam has recently expanded with the major project of Maasvlakte 2, approximately reclaim 2000 hectares for harbour activities and related industries (Barker, 2010).

#### 2.1.1 Brief history of the Port of Rotterdam

The port of Rotterdam has an extensive history, going back to the 14th-century. As the city of Rotterdam has developed from a small town into a large commercial city, the port needed to be expanded. In earlier centuries, docks were constructed on the banks of the *Nieuwe Maas, Buizengat, Haringvliet, Leuvehaven, Wijnhaven*, where among the first harbours of Rotterdam. In the 19th century, Rotterdam was poorly accessible from the North Sea, with a large estuary/delta area with numerous small waterways between them.



Figure 2 Old drawing of Delfshaven (Rotterdam.nl)

To solve this problem, a direct connection with the North Sea and the rivers, Rhine and Meuse, was designed in 1866, called *NieuweWaterweg* (New Waterway), improving the harbour activities.

#### 2.1.2 Outline of the port

The harbour is composed of several parts, among those parts are the city centre's historic harbour area, including Delfshaven; the Maashaven/Rijnhaven/Feijenoord complex and the Waalhaven; and the reclaimed Maasvlakte area, which is constructed into the North Sea. In the figure below (see Figure 3), the Waalhaven and the Merwehaven are shown in a map of the Port of Rotterdam.





Figure 4 Overview Waalhaven and Merwehaven (Havenbedrijf Rotterdam)

#### 2.1.3 Vision of the Port of Rotterdam

Rotterdam has set goals for 2030, still being as Europe's major port and trade complex, leading both in adaptability and sustainability. Older parts of the port are in a continuous phase of redevelopment and adaptations to provide the most modern facilities and to meet requirements of leading corporations. Adaptability is the keyword to be a vital cornerstone for the well-being of the Netherlands, Europe and especially the region of Rotterdam.



Spread over several ports, the Port of Rotterdam has over a length of 13 kilometres caissons used as quay walls. One of the first caisson quay walls was built by HBM (*Hollandsche Beton Maatschappij*) in 1913 in the Lekhaven and later also in the Merwehaven and Waalhaven.

In the Waalhaven and Merwehaven there are several caissons used as quay walls or foundations of new quay walls (after damage in the World War II). This case study will focus on three quay-wall caissons that are situated in two mentioned harbours of the older part of the Port of Rotterdam. In the following sections, a brief history of the harbours will be given and a description of the specific caissons, dimensions and position in the harbour supported with drawings.

### 2.2 History of quay walls in Waalhaven

The Waalhaven is situated on the left bank of the *Nieuwe Maas* (New Meuse) and was excavated in 1907, making it the largest excavated harbour basin in the world with a surface area of 310 ha. In the years that followed the Waalhaven expanded several times, with the last large expansion in 1930. In Figure 5 the harbour layout is shown.



Figure 5 Overview Waalhaven

In the Second-World War, quay walls in the Waalhaven were damaged by bombardment on the airport, an airport which not exists anymore, and was meant for air cargo service to England. The bombardment damaged the Waalhaven area too, destroying several quay walls of the bellowing type (see Figure 6). After the





war the quay walls were rebuilt and the piers extended, including with a new type of caisson quay wall built between 1954 and 1960 (see Figure 7).





Figure 7 Caisson with straight walls (Caissonbouw)

Till the 90's the Waalhaven served predominantly as transhipment port of bulk containers transported mostly by lighter abroad ship (LASH) carriers. Some LASH barges are still in the harbour, being used as storage. The harbour changed gradually from transhipment port to a more provider of service related activities.

The Waalhaven consists of seven piers; three on the east side of the harbour and four on the west side with varying lengths. Figure 5 depicts an overview of the Waalhaven indicating the location of the caisson quay walls. The caisson quay walls are (partly) built along *pier 2, pier 7* and *pier 9*. All these caissons have a general shape, with little variation in dimensions and partition walls.



#### Caisson 1 (Type P- Waalhaven) 1958-1960

Along the south side of pier 2, over almost the whole length of the quay, caissons were placed serving as quay walls. The total length of the quay wall, existing of 17 caissons, is 742 meters caissons have the same shape.

| Caisson 1                   | Pier 2 south side  |
|-----------------------------|--------------------|
| Concrete density            | (2500 kg/m³)       |
| Concrete volume per caisson | 1021.80 m³         |
| Height                      | 12.50 – 13.40 [m]  |
| Total width                 | 12.20 [m]          |
| Total length                | 43.60 [m]          |
| Thickness front wall        | 0.30 [m]           |
| Thickness partition wall    | 0.20 [m]           |
| Thickness back wall         | 0.30 [m]           |
| Thickness floor (varying)   | 0.65 [m] (maximum) |
| Thickness floor (varying)   | 0.40 [m] (minimum) |

Table 1 Properties of caisson P

Through the last decades the load on the quay wall has increased due to changes in the market demand. The caisson quay-wall was designed for a load of 25 kN/m<sup>2</sup> for the first 20 meters from the waterside, but has now a permissible load of 10 kN/m<sup>2</sup> for the first 6 meters from the waterside. This because the surcharge loads caused a horizontal sliding towards the water. Between 6 and 12 meters the caisson can handle a load of 30 kN/m<sup>2</sup> and from 12 meters a load of 60 kN/m<sup>2</sup>.





Figure 8 Cross-section caisson in Waalhaven

#### Caisson 2 (Type Q - Waalhaven) 1955

This caisson is located in the south-western part of the port (pier 9) with a quay wall length of 175 meters. This structure, in contrast with the other caissons, has no superstructure, but an upper deck. The total allowable load on the caisson is  $30 \text{ kN/m}^2$  and at the back of the quay  $60 \text{ kN/m}^2$ .

| Caisson 2                      | Pier 9 (Kolenpier) |
|--------------------------------|--------------------|
| Concrete density               | (2500 kg/m³)       |
| Height                         | 15.60 [m]          |
| Total width                    | 15 [m]             |
| Total length                   | 43.60 [m]          |
| Thickness front wall           | 0.35 [m]           |
| Thickness partition walls (2x) | 0.20 [m]           |
| Thickness back wall            | 0.30 [m]           |
| Thickness floor (varying)      | 0.65 [m] (maximum) |
| Thickness floor (varying)      | 0.35 [m] (minimum) |

Table 2 Properties caisson Pier 9





Figure 9 Cross-section caisson Q

#### 2.3 History of quay walls in Merwehaven

The Merwehaven is situated on the right bank of the *Nieuwe Maas* (New-Meuse), not far from the Waalhaven. The construction of the port started in 1923 and was completed after nearly 8 years in 1931. Compared with the Waalhaven (310 ha), the Merwehaven is much smaller in surface area (46 ha). The Merwehaven is also known as the *Fruitport* when in 1971 fruit warehouses were opened for the storage and processing of fresh fruits and vegetables.



Figure 10 Overview Merwehaven



The port has 2 piers which are accessible by water from both sides. The quay walls of the Merwehaven mainly consist of caissons. The caisson have different forms, varying in dimensions and slightly in shape. The figures below (see Table 3) are the caissons along pier 2 (the black line in Figure 10) where caissons function as quay-walls. In the figures below an overview of the Merwehaven and a cross section of the caisson quay wall are shown.

#### Caisson 3 (Type E - Merwehaven) 1956-1959

Table 3 Caisson E

| Caisson E                      | Pier 2 south side |
|--------------------------------|-------------------|
| Concrete density               | 2500 kg/m³        |
| Height                         | 17 [m]            |
| Total width                    | 14.85 [m]         |
| Total length                   | 43.65 [m]         |
| Thickness front wall           | 0.30 [m]          |
| Thickness partition walls (2x) | 0.20 [m]          |
| Thickness back wall            | 0.30 [m]          |
| Thickness floor                | 0.55 [m]          |
| (excluding toe and heel)       |                   |





Figure 11 Cross-section caisson E (Stadsontwikkeling Rotterdam)



Figure 12 This picture shows the backside of the caisson. Clearly one can see the height difference of the front wall and the back wall (Caissonbouw)



### 3. Literature study

#### 3.1 Caissons

#### 3.1.1 Introduction

The word caisson comes from the French word *caisson*, which literally means box. The etymology of the French word caisson is derived from the Italian word *cassone*, which is a dowry chest, referring to the regular shape of caissons. From the point of view of civil engineering, caissons could be defined as a retaining watertight box, used both for temporarily as permanent structure (Voorendt, Molenaar & Bezuyen, 2011). Often a caisson is part of a larger structure and can serve numerous purposes as, but not limited to:

- (Floating) breakwaters;
- Storage of liquids like oil or solid raw materials like sand;
- Freshwater storage of desalinated water;
- Foundation of a bridge;
- Lock head;
- Quay walls.

Caissons are mainly constructed in a dock or (special) construction place and transported to the final place. Before and after replacement of the caisson, it is necessary to protect the bed against erosion due to water flows. Erosion can cause uneven settlement of the elements (caissons) leading to leaking and possible failure of the structure.

#### 3.1.2 Types of caissons

In civil engineering three main types of caissons can be distinguished; box caisson, open caisson, pneumatic caisson and monolith caisson. All these caissons have specific qualities, applications and construction methods. In the next sections, the three types will be discussed.

#### **Box caisson**

A box caisson, open at the top and closed at the bottom, is commonly fabricated on land, then launched, transported to desired location, and immersed on top of a prepared foundation, leaving its upper edge above water level. It functions as an appropriate shell for a pier, dyke, breakwater, quay wall, or comparable work. After (partial) immersion the top part, if needed, can be constructed or placed prefabricated.





Figure 13 Box caisson (therefeetank.com)

The box caisson is a hollow concrete structure, where the phenomenon floating should be considered during transportation and sinking of the element. Caissons have often floating capacity, an advantage during transportation, but a concern during immersion and keeping the caisson on position. Anchoring and/or ballast are necessary to hold the caisson on position in all possible situations.

#### **Open caisson**

The open caisson is comparable to the box caisson, except that it has no bottom face but a top face. This type of caisson is suitable for use in soft soil where no large obstructions are present in the ground and where open trench excavations are impractical. The open caisson is a practical solution to set up deep manholes and inspection chambers pump stations, launch pits for micro tunnelling, pipe jacking and other operations.



Figure 14 Cutting shoe of a caisson with dredging walls (John Stamets)



By self-weight, hydraulic jacks or extra ballast, in the form of concrete or water, the caisson will sunk into the ground. During sinking the soil from the space within the caissons is excavated by use of clamshell. To improve the sinking process, the cutting edge, usually a cutting *steel shoe* (Figure 14), is sloped out at a sharp angle. For reducing friction the steel shoe is generally wider than the caisson itself, and supported with bentonite slurry to fill voids and depressions. When the caisson is sunk to the required depth, the top face can be constructed if desired. Due to the increasing soil friction during sinking, the caisson will not sink after a certain depth. Yet, additional measurements should be considered when friction is not sufficient to stop sinking. To counteract this problem, piles from surface level could act as load-bearing walls and anchors.

#### **Pneumatic caisson**

The name of the third type of caisson originated from the technique that is used during execution. Pneumatic caisson is executed according the 'diving bell principle', where the water is forced out of the caisson by compressed air during digging. The enclosed space underneath or in the caisson, the working chamber, is compressed to create a dry working space, free of water and mud. Traditionally workforces move mud and rock debris from the edge of the workspace to a water-filled pit, connected by a pipe (called the muck pipe) to pump it upwards out of the caisson. A cutting edge needs to provide a smooth subsidence into the soil. After the caisson is on depth, the reinforced concrete bottom face is constructed.

This is also the main advantage of pneumatic caissons, creating a dry work environment using compressed air and therefore supervacaneous of dewatering pumps. In environments where space around the caisson is small and scarce, the pneumatic caisson could be preferred over other types. An example are the caissons placed in the city centre of Amsterdam as integral part of a new metro line, considering the accessibility of traffic and not too spacious working environment, pneumatic caissons seemed to be the most preferred type.





Figure 15 A pneumatic caisson (Cyclopedia Of Architecture, Carpentry, And Building)

#### 3.1.3 History of caissons

Caissons have a long history, stretching back into the era of the Roman Empire, where caissons have been used for several purposes (Voorendt et al., 2011). The first application of caissons is in about 250 years BC, in Alexandria, Egypt, where impermeable caissons have been used to construct quay walls along the river banks. A timber mould was constructed as part of a timber caisson and mortar blocks were cast in this mould. With help of the floating caisson, this mould was then located at the required site for the quay wall (de Gijt, 2010).

During the centuries, caissons were made for a wide-range of objectives, for both civil as military purposes. Also construction methods were improved or carried out in a new manner, like the pneumatic construction method. Caissons were no longer only used as quay wall, as the ancient pioneers did, but served an extensive range of purposes, for instance bridge foundations as first constructed in Vichy (France) and the famous Firth of Forth bridge (1890) in Scotland.



Figure 16 Principle of a pneumatic caisson



Nowadays, ports and harbours are a main field for the application of caissons, as in the ancient world as quay walls. Port of Rotterdam has used pneumatic caissons during its expanding in the twentieth century. Most of these caissons were placed to serve as new quay walls for the fast growing port. Also upgrading of existing caissons was part of the redevelopment during the first part of the twentieth century, using standard caissons. Drawback of those caissons is the uncommon shape, as can been seen in Figure 18.



Figure 18 Quay wall with trapezoidal shape, in Rotterdam (de Gijt, 2010)



Figure 17 Reinforced concrete quay wall port of Valparaiso 1903 (de Gijt, 2010)

Caissons proofed also to be valuable as temporarily structures. An example can be drawn from the North Sea flood of 1946 and 1953, where caissons were used to protect parts of the Netherlands for further damage. At the night on the first of February 1953, springtide in combination with a strong north-western wind caused breakthrough of dikes in the southwest part of the country (mainly province Zeeland). Closing the gaps was priority to stop flooding and reduce damage. For that reason, the so-called Phoenix caissons were reused to close gaps in the dikes.



Figure 19 Phoenix caisson closes gap, in 1946 (Caissonbouw)



Phoenix caissons were built for the purpose of the Second World War and offered as leftovers by the allied forces (Heijkoop, 2002). Figure 19 below shows one of the Phoenix caissons during the repair of the dikes in 1946, positioned by tug boats, after inundation by the allied forces during World War II.

#### 3.1.4 Transportation of caissons

Caissons are often constructed on a site or in a dock and then transported to the location. Transportation of caissons is possible over water as well as over land, however over water is more advantageous in many cases. Those type of structures are often used nearby rivers or seas, making transportation over water advantageous. Also dimensions and weight of a caisson leaves road transportations for exceptional cases.

Transportation over water demands that caissons should have sufficient buoyancy to be moved. This can be obtained by the floating capacity of the structure itself or by additional measurements, such as drift bodies. Besides floating capacity, one should also consider static and dynamic stability issues during transportation and immersion.

#### **Equilibrium of vertical forces**

Archimedes' principle is the single most important law for floating objects. Archimedes' principle states that an object immersed in a fluid will exerts a upward buoyant force, either fully or partially submerged, is equal to the weight of the fluid that the object displaces (Nortier & de Koning, 1994).

#### **Equilibrium of moments**

During transport and immersion, static stability is the second concern to be managed. Static stability of a floating object is the ability to counteract forces for overturning of the object. Tilting during transportation and immersion should be restricted since it could cause serious damage to the caisson. Tilting could be distinguished in two directions, around the x-axis (surging) and around the y-axis (pitching) (see Figure 20).





Figure 20 Surging and pitching of an object (Voorendt et al., 2011)

Tilting can be initialised by several (external) forces on the caissons, thinking for example mooring forces, wave motions, forces during tugging and water inlet. Caissons need to resist those forces by a righting moment, to return the structure to its initial position and avoid tilting. Resistance to tilting is expressed in the term of metacentric height as will be explained in the next part.



Figure 21 Buoyancy and gravity forces

#### Metacentre

Usually to check the stability of caissons, calculation of the metacentre is necessary, which is the distance between the centre of gravity **[G]** and the metacentre **[M]** of the caisson (see Figure 21). The metacentre is the intersection point of the *z*-axis and the action line through the buoyant force. Generally caissons could be considered as secure when the metacentre is at least 0.5 meter above the gravity centre (Voorendt, Molenaar & Bezuyen, 2011). In the Figure 22, an illustration of this principle is shown.





Figure 22 Principle of stability

For stability, three imaginary points are of importance to assess whether the caisson is stable. In Figure 22 those points are illustrated, namely centre of buoyancy **[B]**, centre of gravity **[G]**, and the metacentre **[M]**.

- The centre of buoyancy is a point where the buoyancy force  $F_b$  applies in case of equilibrium. This centre stands for the gravity centre of the displaced water and moves when the caisson rotates due to change of the geometry of displaced volume. In the illustration the reallocated centre of buoyancy is specified with **B** [ $\phi$ ] and the horizontal shift with distance **a** [m]. The buoyancy force will contribute to the righting moment, as well as the self-weight of the caisson.
- The buoyancy point **[B]** is the centre of the displaced water; the gravity centre **[G]** represents the centre of the caisson. It is worth to notice that if the caisson is filled with any material, this should be taken into account for calculating the position of the gravity centre. Any ballast will influence the gravity centre by lowering it due to increase of draught. The centre of gravity is considered as rotation point of the caisson and fixed, assuming in the case there is any ballast, it will rotate along with the caisson.
- The metacentre point is the intersection between the z-axis (a rigid symmetry axis) and action line of the buoyancy force. When the caisson is symmetric and upright (φ is zero), the action line and z-axis will coincide. The metacentre should be positive and above the centre of gravity in order to cause a righting moment.




Figure 23 Tilted element (Voorendt et al., 2011)

This distance between the buoyancy point and metacentre can be calculated. Consider Figure 23, where element dx comes under the waterline by rotation of the caisson. This particle undergoes an upward force:

$$dF_b = \varphi \, l \, dx \, \rho_w g \tag{1.1}$$

This equation has a restriction and is merely valid for rotations smaller than 10° in which case we consider  $\tan \phi \approx \phi$  [rad]. Considering **[G]** as rotation point, the force gives a moment:

$$M = xdF = \varphi x^2 l dx \rho g$$
(1.2)

in which x is the distance between the centre of gravity and centre of the particle. Integration of equation 1.2 over the entire width obtains for the righting moment:

$$M = \varphi \rho g I \tag{1.3}$$

in which I is the moment of inertia relative to the y-axis.

The buoyancy force, *F*<sub>b</sub>, will shift when the caisson is not in equilibrium. Due to rotation of the caisson, the buoyancy point [B] will shift over a distance which can be calculated with the following equation using equilibrium of moments:

$$a = \frac{M}{F_b} = \frac{\varphi \rho g I}{\rho g V} = \frac{\varphi I}{V}$$
(1.4)



The **V** in this formula is the volume of the displaced water (or in other words, the volume of the immersed part of the caisson). From equation **1.4** the distance between the metacentre **[M]** and the buoyancy point **[B]** can be determined:

$$\overline{BM} = \frac{a}{\varphi} = \frac{l}{V}$$
(1.5)

When looking to small rotations ( $\varphi < 10^{\circ}$ ), the position of [M] can be considered as fixed. However, the buoyancy point will change of position and will cause an increase of  $\overline{BM}$ . When rotations are more significant, the position of the metacentre will shift upwards and in the opposite horizontal direction of the rotation and will not coincide anymore with the vertical y-axis. The moment that cause a stabilizing position, when [M] is situated above [G], can be written as

$$F_b h_m \varphi = \rho g V h_m \varphi$$

For caissons,  $h_m$  should be at least 0,5 meters to consider the caissons as stable, assuming that **[M]** is positioned above **[G]**, otherwise the caisson will tilt and becomes unstable. The distance between those two points, the metacentre and centre of gravity, can be calculated as follow:

$$h_m = \overline{KB} + \overline{BM} - \overline{KG}$$

where  $\overline{\text{KB}}$  is equal to the distance from buoyancy point to the bottom line of the caissons and  $\overline{\text{KG}}$  the distance from bottom line to centre of gravity. A positive  $h_{\text{m}}$  is considered as stable, however,  $h_{m} \ge 0.5 \text{ m}$  is recommend. When  $h_{\text{m}}$  is below this threshold additional measures are needed to ensure the stability.

There are two main types of additional measures that can be taken to ensure the stability of the caisson. The first measure is more focused on adaptations of the structure, a disadvantage when the structure already is built, as in this thesis is the case. The second types of measures are temporarily measures during uplift and transportation. Examples of these measures are:

- Adding additional ballast below the centre of gravity, to increase KB, the distance between the bottom line of the caisson and buoyancy point.
- To increase the polar moment of inertia, which has a positive influence on  $\overline{BM}$  through the formula  $\overline{BM} = \frac{I}{V}$ . To temporarily increase the moment of inertia for the benefit of the stability during transportation, stabilizing pontoons or vessels can be used to link them to the caisson.



Pontoons could also be used to enhance the floating capacity of heavy caissons with insufficient floating capacity. In case when several caissons need to be transported, they can serve as stabilizing elements, by linking those caissons to each other (side by side) and therefore increase the second moment of inertia.

#### Ballast

Using additional weight can be necessary in two situations:

1) When stability of the caissons is not ensured during transportation over water (temporarily);

2) During immersion of the caisson on final location (permanently).

The most often used ballast is water or soil, which depends on the situation, is usually sand. The advantage of water ballast over other types is the use of water ballast tanks that can be easily mounted/dismounted and are relatively cheap. Without ballast tanks it is harder to control undesirable movements, due to free movement of the water, and can cause tilting, especially in the lengthways. Soil, on the other hand, has less effects on tilting, providing it is evenly spread over the caisson and sliding of soil is restricted.

#### **Dynamic stability**

In the previous section the static stability of caissons has been discussed, but dynamic stability should also be taken into account. The dynamic stability is related to waves and swell that can cause the caisson to sway. Oscillations of the water surface have influence on the stability of the caisson, where navigability and clearance can be in hazard when the fluctuations are large.

Waves have influence on structures that are floating, submerged or have any contact with open water. Especially floating structures are more likely to endure influence of waves, for the structure is not fixed for transportation purposes. To check whether a structure is dynamic stable or additional measure should be taken, some rules of thumbs are available.

Sway of structures is caused by waves and swells by fluctuation of the water surface. Wave length is a crucial factor that affects the dynamic stability and the rule of thumb is therefore based on this factor. The dimensions of the structure (length or width) should have a certain dimension compared with the wavelength or swell. The following rules reflect this:



| $L_w$ | $< 0.7 \cdot l_e$ | (1.6) |
|-------|-------------------|-------|
| ~~    | U                 |       |

 $L_w < 0.7 \cdot l_b$  (1.7)

where:

| $L_w$ = The wavelength             | [m] |
|------------------------------------|-----|
| $l_e$ = The length of the caisson  | [m] |
| $l_{b}$ = The width of the caisson | [m] |

Depending on the direction of the waves relative to the caisson, rule 1.6 or 1.7 should be checked for dynamic stability. This is a quick check whether dynamic instability, and so sway, can be expected.

The second instability that can occur is related to the natural oscillation period of the floating caisson, that if comes too close to the natural period of water movements, can result in (heavy) uncontrolled movements. Resonance is a tendency of a system to oscillate with larger amplitudes at some frequencies than at others. When the natural period of water is close to the period of the caisson, measurements or adjustments of the design can offer a solution. However, the costs rise whether by the former solutions or by postponement of the execution.

The natural oscillation period of the relevant structure can be calculated as follow:

$$T_d = \frac{2\pi j}{\sqrt{h_m \cdot g}} \tag{1.8}$$

in which:

| $T_d$ = natural oscillation period        | [s]    |
|---|--------|
| j = polar inertia radius of the structure | [m]    |
| $h_m$ = metacentric height                | [m]    |
| g = gravitational constant                | [m/s²] |

The polar inertia radius can be obtained as follow:

$$j = \sqrt{\frac{I_p}{A}}$$
, with  $I_p = I_{xx} + I_{zz}$ , the polar moment of inertia or second moment of inertia.

When the natural oscillation period of the structure is not close to the natural frequency of waves, dynamic stability is not in danger and is considered as stable.



# 3.2 Reuse and sustainability

Reuse of existing buildings is widely applied phenomena, where vacant premises are adapted to new standards and requires conformance to recent code requirements for example fire safety and healthy work –or life environment. Structures can also be used at a new location, depending on the structure, mainly the demountability. Steel structures, which have uniform shapes and are relative easy to disassemble, are often used for reuse. The elements can be reused separately or the whole structure can be relocated. An example is Honda's warehouse in Swindon (UK) that was dismantled and build up from the same elements on a new site. The same was the case of a parking garage in Munich constructed from steel elements that was moved to a new location in the city.

Compared to office and residential constructions sustainability and reuse of civil structures is still not common practice. Still there are some structures that are served for other purposes then the initial use. In the Netherlands some civil structures were reused after they were no more needed for different reason. One of these structures was a caisson structure built as temporality breakwater in the Second World War. More recently are some steel bridges in the Netherlands that have been reused as bridge or in a single case for a total different function.

Below two specific examples are given in the context of reuse of structures. The first example is a steel bridge situated in the Netherlands, as the second example focusses on the caissons used in the Second World War. Some of those caissons are still used and in operation for a different purpose.

#### **Bridges**

The idea to reuse structures for other functions or on other locations is not a recent phenomenon. By widening the lock at Spaarndam (NL), the just 7 year old iron bridge was sold by the contractor to be reused at a new location. Rijkswaterstaat relocated in 2004 two drawbridges near Harderwijk that became superfluous after construction of an aqueduct. Inspection of the bridges indicated that the service lifetime was not reached and reuse was possible. One of the drawbridges replaced a concrete bridge to increase accessibility of a nearby ship repair wharf. That the initial function is not always the final destination of the structure can be illustrated the reuse of an old pivot bridge as carrier of a restaurant (see Figure 24).





Figure 24 An old pivot bridge as restaurant (Maarten Helle)

### Caissons

The above mentioned caissons were once used by the British army as breakwaters during the allied invasion of Normandy in 1944. The breakwaters were needed to construct a temporarily harbour for mooring ships in the so-called Mulberry Harbour (see Figure 25). At the time of the invasion 147 caissons were build but production continued until October 1944. In total 213 caissons were constructed, divided into 8 types. After the war, the caissons were reused to close the bombed dikes near Walcheren.



Figure 25 Art impression of Mulberry Harbour (Burnham on sea)

The same type of caissons was bought in 1953 to close the gaps in dikes after the catastrophic flood in Zeeland. The caissons have a length of 62 meters, 19 meters in the width and a height of 18 meters. The caissons were constructed in a very short period with unskilled labour, low quality material and without any consideration for durability, since they were built for a limited time span (Melchers & Pape, 2012). However, the Phoenix caissons that were reused to close gaps in the dikes are now serves as museum aimed at the flood in 1953.





Figure 26 Waternoodsmuseum in Zeeland (waternoodsmuseum.nl)

## 3.2.1 Possible functions

This subparagraph describes some possibilities to give an impression for use or reuse of caissons. Chapter 8 will elaborates on the possibilities that are specifically applicable to the caissons in the Waalhaven.

#### **Caisson as fruit storage**

Daily tons of goods are transported, stored and distributed in the Port of Rotterdam, including fruit, vegetables and fresh products. The processing of these goods takes place in the Fruitport Rotterdam (FPR), located in the Merwehaven. This harbour section controls the handling, processing and distribution of fresh fruit and vegetables. Fruits and vegetables are sensitive goods that need proper storage areas, where cooling, ripening and packaging can take place. Fruitport Rotterdam processed daily tons of fruits and vegetables in refrigerated warehouses with a total area of 50 000 m<sup>2</sup>.

In the context of expansion for the storage of refrigerated fruit, there is a design study carried out to store the fruit in underground facilities (Knibbe, 1997). Using caissons as storage, the fruit terminal can also be located above the ground on a different location. The fruit and vegetables are transported in pallets. A maximum of four pallets can be stored on each other, so it is reachable for forklifts. However, present the so-called reefer containers are more usual in transporting cooled fruit and vegetables. These reefer containers are fitted with refrigeration units and an engine for cooling.

For temporarily underground storage of fruit, caissons can be an option to use, especially when standard units are used. Possible bottlenecks can be the researchable of specific containers in underground storages and likely to transport the caissons to another location.



#### **Caissons as oil storage**

One of the most transported liquids over water is oil (or a derivative of oil) which are transported to harbours for storage, trading and transfer. Storage tanks are mostly constructed by steel or concrete, mostly in a cylindrical form. However, other forms of storage tanks are also possible. Caissons are used in concrete offshore structures as buoyancy and foundation, but act also as storage of liquids.

#### **Caissons for freshwater production**

Caissons can be deployed for producing fresh water by evaporating sea water. The idea is to boil seawater under pressure with evaporators which are situated near the coast. For these installations a floating body, a caisson, is needed to store the produced fresh water and act as foundation for several installations needed for the fresh water production.

#### **Caissons as foundation for buildings**

Offices, storages, and residential buildings need all foundations for bearing capacity to the subsoil. Foundations can consist of piles, but also caissons could function as foundations for (residential) buildings. Taking into account that in the future the Merwehaven and Waalhaven will probably be redeveloped as an area for living working and recreation, caissons can be used as foundations for buildings along the waterside.



Figure 27 Art impression of Merwehaven (NHTV Breda)

For this purpose, the caissons probably do not have to be transported and can stay at the current location. The technical lifetime of the caissons, after necessary renovation, should be equal to the service lifetime of the buildings on top of it.

## **Caisson for generating energy**

Generating power from tides, tidal energy, is a form of hydropower that transforms energy from tides into electrical energy. To use tides to generate power, one can use tidal barrages to generate energy in



the difference in height between high and low tides (potential energy). For this reason, such tidal barrages needs to be placed in places with a high tide differences to generate efficiently energy.



Figure 28 Art impression of caisson for tidal energy (Power DTP)

Caissons, with built-in generators, can function as tidal barrages. Main disadvantage is the (large) adaptations that are necessary to existing caissons in order to make them function as power plant.

### **Caissons as breakwater**

Breakwaters are used to protect structures, harbours, shorelines and/or worksites from waves by reducing the intensity of wave actions. Breakwaters provide safe harbourage for ships and reduce wave loads on quay walls in harbours and overtopping. These structures are also used to protect coast lines against erosion by decreasing the impact on the coast. Besides permanent use, breakwaters can be used on temporarily base during construction of a harbour, windmill farm, or any other project. The advantage of caisson breakwaters over other types of breakwaters, is the relative simple execution, since the caissons can be constructed on a more favourable location and then transported to the final location.

When caissons will be reused as a breakwater structure, it has often not the desired or optimal shape for a breakwater, unless the caisson was already designed as a breakwater. However, caissons have the capacity to float and therefore they can be used as floating breakwater. This type of breakwater are suitable and preferred over in cases of deep water or poor foundations possibilities.



#### **Caissons as emergency dike reinforcement**

Several examples in the past have showed the value of using caissons for dike reinforcement and closing dams. Caissons are used as closure dams as part of the Delta Works to protect land against flood.

Besides, the reason for building the Delta Works was the large flood in 1953. The dikes broke through during the storm, closed by caissons that had served before as breakwater.

Caissons are ideally suited as flood defences by the large dimensions, impermeable behaviour and held in place by gravity. The relative rapid construction time in combination with the above advantages provide those structures an interesting alternative.

In case of emergency the caissons need be available in a very short time span to use as dike



Figure 29 Caissons as emergency solution at Ouwerkerk, 1953 (Beeldbank Rijkswaterstaat)

reinforcement. In the ideal situation, the caissons should be stored on several spots spread across the coast. One of the spots to store the caissons is the *Tweede Maasvlakte*.



## 3.3 Concrete

Over the last 100 years the development of concrete had made enormous leaps in concrete composition, concrete casting and new techniques. Also the ratio labour –and material costs was a crucial factor in building with concrete. Where labour costs were relative low after the Second World War, today labour costs are an important key to consider. In that time, Portland-cement was the standard used cement for constructing concrete structures.

Concrete properties change gradually after pouring of the concrete influenced by time, hydration and temperature. The strength development is a time-dependent process, which even after years still can develop. Another aspect that should be considered is that concrete is not endlessly durable due to environmental impacts and the condition of concrete itself. Permeability and porosity of concrete are the main factors that influence durability of concrete structures and affecting the steel reinforcement.

In this paragraph the properties and durability of old concrete will be discussed and the relevant environments and its impact on concrete structures.

## 3.3.1 Concrete in marine environments

In general, cracks in concrete interconnect flow paths and increase concrete permeability. The increase in concrete permeability due to the development of cracks permits more water or aggressive chemical ions to penetrate into the concrete, initiating deterioration and decrease the service lifetime of concrete structures. Cracking is a typically behaviour of concrete and a normal phenomenon in concrete structures. Cracks are actually not a hazard, as long as the crack widths do not exceed a certain value. Essential in crack width control are the environmental conditions to which concrete is exposed, e.g. chlorides, aggressive gasses and frost.

Severe corrosion of steel reinforcement takes place when humidity, oxygen and chlorides in the concrete interact with the embedded reinforcement. Humidity and oxygen from the environment can penetrate to the steel rebar through pores or cracks in the concrete. Chlorides from (marine) environments can also find their way through pores and cracks to reach the steel rebar. When steel reinforcement starts to corrode, the surrounding concrete will have no bond any longer with the reinforcement by the expansion of reinforcing steel. The reinforcement steel expands by corrosion, causing the concrete cover will crack by this pressure.

Hydraulic structures have in general direct contact with (sea) water, where some of the structures are permanently submerged. Structures near or on the coast are attacked by chlorides from seawater (saltwater). Saltwater induces earlier deterioration of concrete structures as a result of corrosion of



reinforcement in concrete and degradation of mechanical properties caused by the reaction of hydration products of concrete with saltwater (Mohammed, Hamada & Yamaji, 2004). The resistance of the concrete against chlorides depends mainly on permeability, porosity of the concrete and the applied concrete cover. To reduce the impact on the reinforcement embedded in concrete, a minimum concrete cover should be applied depending on the situation and environment. According to the recent requirements (NBN EN 199211:2005), hydraulic structures exposed to chlorides from seawater are classified in the XS2 class. A concrete cover with a minimum of 30 mm is required to enhance the durability of the structure.

Permeability is a measure of the ability of a porous material to let liquids and gases to pass through it by the pores [in m/s]. Concrete is a porous material that has a certain permeability. A low permeability of concrete is positive for durability, for harmful substances infiltrate more slowly and less deep in the concrete. Permeability is affected by the curing period (hydration) and water-cement ratio. By hydration the pores volume will decrease causing a closure of capillary pores and therefore has a lower permeability.

## 3.3.2 Concrete-strength over time

The development of concrete strength is a time-dependent process which regularly increases rapidly during the first few years and thereafter more gradually as it moves towards a nearly uniform level. After the characteristic value of 90 days, the concrete will increase gradually. Tests with 28-year-old concrete indicate that the compressive facture strength of concrete increases by 29% to 39% from the 28-days characteristic value to the age of 28 years (Prassianakis & Giokas, 2008). However, the concrete strength depends on some factors, among them concrete grade, cement type, water-cement ratio and the environment. The compressive strength of concrete specimens submerged in soil at coastal areas (chlorides of seawater) was more than those cured in laboratories, caused by the accelerated hydration of cement. The data also show that the highest strength gain was in the concrete specimens with a water-cement ratio of 0.40 (Bader, 2003).

The water-cement ratio is an important factor concerning permeability of concrete and therefore the durability of concrete. A high water-cement factor results in a material that is more susceptible to chlorides due to its larger permeability. Till 1962, before the CUR 1962 was published, water-cement ratios were often determined by contractors and ranged between 0.45 and 0.8. This was restricted to a maximum of 0.6 by the CUR 1962. Water-cement factors play a major role in development of concrete properties as strength and permeability and therefore durability. Higher water-cement ratios, or structures that are submerged under water, will experience a higher level of hydration attributable to



the amount of available water in order to react with the contained cement. After years this cause a gradual increase of strength. High water-cement ratios (> 0.6) have also a downside. When a high water-cement ratio is applied, the relative strength (28-days concrete) will decrease.

The hydration process itself needs a specific amount of water. Concrete is essentially mixed with more water than is required for the hydration processes. This extra water is added to give concrete sufficient workability. The water that is not used in the hydration process will stay in the microstructure pore space. These pores make the concrete weaker by the lack of strength-forming calcium silicate hydrate bonds.

The low relative strength (28-days concrete) of early concrete compared with modern concrete is also caused by using a coarser composition of cement than the current fine cement. By the coarser structure of the cement hydration will develop slower, making the relative strength after 28 days in comparison to 50-year old concrete significantly larger. Studies have shown that the compressive concrete strength had increased during lifetime of structures. The magnification factor of concrete strength varies among the researches. In an investigation of concrete bridges (built during 1931-1962) an increase of 71% was obtained of the compressive strength (Thun et al., 2006). Specifically for lower strength concrete, 25 years' strength approaching 240% of the 28-day strengths appeared from research (Washa & Wendt, 1975).

Additionally, storing concrete in dry or wet conditions obtains different strength developments (Baykof & Syglof, 1976). They found that concrete in wet conditions (at 15°C) considerably increased (see Figure 30). On the other hand, the strength of concrete in dry conditions showed no significant difference. Concrete exposed to a wet environment could increase in strength up to a factor 1.63 in 10 years.



Figure 30 Concrete strength in dry and wet conditions (Baykof & Syglof, 1976)



## 3.3.3 Field inspection of caisson Waalhaven

A field investigation of the caisson concrete structures of pier 7 (Waalhaven) is performed (concrete specification and age equal to caissons structures of pier 2). The aim of the investigation is to evaluate the concrete characterization and service life time evaluation. This section will cover the following subjects;

- Compressive strength
- Carbonation-induced corrosion
- Chloride content

### **Compressive strength**

The compressive strength of the concrete is tested with drilled cores from several sections, specifically 5 tests. The tested cores showed that the compressive strength of the concrete is much higher as estimated before. The average compressive strength of the concrete is 58.7 N/mm<sup>2</sup>, where before the estimation was around 24 N/mm<sup>2</sup>.

### Carbonation-induced corrosion

Carbonation is a chemical process whereby calcium hydroxide, of the concrete, reacts with carbon dioxide from the air or water. The reaction of this two elements forms calcium carbonate. Laboratory tests showed a minimal risk of carbonation, with no carbonation depth at all.

#### **Chloride-induced corrosion**

A passive oxygen layer around the steel reinforcement protects the reinforcement bars against corrosion. Decrease of pH values or chloride ions can negatively affect the oxygen layer. The degradation of the passive oxygen layer take place when a certain amount of chloride is reached at the steel level.

From tests it has been found that the caissons, especially the submerged part, has high levels of chloride contamination caused by the salty of seawater which differs over the water level in the harbour.

#### Conclusion

Based on chemical degradation, chloride-induced corrosion and carbonation-induced corrosion, the caissons are expected to be in a good condition, except the submerged part which contains high levels of chloride. This part is in a dire need of repair. However, when this part stays under water where the oxygen-level is limited, corrosion will propagate slowly.

Therefore it is important to mention, that when the submerged part of the caisson in the future will be exposed to oxygen, reparation interventions should be proposed.





#### Figure 31 Formation of corrosion

## 3.3.4 Summary

Comparing the findings from theory and the empirical investigation, one obtains a significant difference in results. This can partly be clarified by the difference in concrete mixture and cement characteristics used in the literature and of the caissons in the Port of Rotterdam. Moreover, the environmental conditions as well the conditions of testing could have significant influence on the concrete strength. Some tests are performed after 10 years, while other researchers using 50-year old concrete.

The table below shows the difference in compressive strengths, including the ratio between the design strength and current (estimated) compressive strength.

|                            | Design compressive strength ( $f_{ck}$ ) | (Estimated)<br>compressive strength | Ratio |
|----------------------------|--|-------------------------------------|-------|
| Baykof and Syglof,<br>1976 | 15.3 N/mm²                               | 25 N/mm²                            | 1.63  |
| Washa and Wendt,<br>1975   | 15.3 N/mm²                               | 36.72 N/mm²                         | 2.40  |
| Empirical investigation    | 15.3 N/mm²                               | 58.70 N/mm²                         | 3.84  |

Table 4 Comparison compressive strength over time

The ratio between the empirical test from the caissons, and which literature provides, is 135% respectively 60% larger. Besides the prior reasons, it might be underestimating the design strength and the high factored safety factors by engineers at that time that leads to such high ratio.



For the steel reinforcement, the same tensile strength is assumed to have the same tensile force as the design tensile stress. However, corrosion can have a negative influence on the reinforcement and bond between the concrete and reinforcement.



# 4. Strength calculations for caisson P (Waalhaven)

Most caissons in the Port of Rotterdam are built in the late fifties of the past century. Through the decades, guidelines and requirements have changed. Besides the guidelines that have changed, the current conditions can also be different compared with the design conditions (e.g. changed water levels, increased or decreased loads, different load class). Therefore, a hand calculation will be made to check the structure according to recent guideline Eurocode (EN-NEN 1992-1-1+C2). As mentioned in the literature study, concrete develops its strength properties in time and will obtain gradually a higher strength over a certain period. Especially concrete made in that period (before 1960) has a significant compressive strength increase after the usually measured 28 days strength due to slower hydration of cement. For the calculations, a magnification factor is used for the (compressive) strength of concrete. Because several (empirical) researches found different factors, there will be two magnification factors used to compare their impact on the compressive strength in relation with the unity checks.

In this first part of this chapter the hydraulic and geotechnical boundary conditions will be given. The second part will discuss the strength of the caisson structure which are situated at pier 2 in the Waalhaven. The aim of these calculations is to gain insight into what extent the caissons satisfy the Eurocode in the current load conditions.



# 4.1 Boundary conditions

Boundary conditions are conditions that are constituted by the environment. In this part a list of boundary conditions will be specified for hydraulic and geotechnical conditions. Furthermore, basic information on the quay walls in the Waalhaven of pier 2 and 9 and Merwehaven pier 2 is provided. Drawings of the quay walls are given in Appendix A.

Information on boundary conditions is primarily obtained on the engineering department of Public Works Rotterdam and Port of Authority Rotterdam. In this section the hydraulic boundary conditions in the Waalhaven and around this harbour (in New-Meuse) are listed below together with the geotechnical boundary conditions around the quay walls.

## 4.1.1 Hydraulic boundary conditions

The govern water levels are determined by using the measured water levels in the Waalhaven. The governing water levels are given in the tables below (Table 5). Because the Waalhaven has no permanent monitoring of water levels, the water levels originate from the adjacent 1<sup>st</sup> Eemhaven.

| Water level                  | Relative to NAP |
|------------------------------|-----------------|
| Mean High Water Level (MHWL) | +1.36 m         |
| Mean Water Level (MWL)       | -0.44 m         |
| Mean Low Water Level (MLWL)  | -0.70 m         |

 Table 5 Water levels in 1<sup>st</sup> Eemhaven (Hydrometeo Informatiebundel 3, 2004)

Table 6 Probability of water levels for 1<sup>st</sup> Eemhaven (Hydrometeo Informatiebundel 3, 2004)

|                                | 1%      | 5 %     | 50 %    | 90 %    | 99 %    |
|--------------------------------|---------|---------|---------|---------|---------|
| Probability of exceedance      | +2.14 m | +1.67 m | +1.35 m | +1.03 m | +0.75 m |
| Prob. water level smaller then | -0.95 m | -0.75 m | -0.47 m | -0.13 m | +0.38 m |

## 4.1.2 Geotechnical boundary conditions

To determine soil properties and geotechnical conditions in the area of Waalhaven, the engineering department of Public Works Rotterdam has executed several cone penetration tests during the spring of 2003. From this tests the soil properties and soil layers can be determined. The soil layers are not an



exact representation of the real profile, because only the soil conditions of some points are examined. The cone penetration tests for relevant piers of the Waalhaven are attached in Appendix B.

In the next figure the profile is depicted.



Figure 32 Soil profile (KG414)



# 4.2 Hand calculations

To verify/check the caisson structure according the Eurocode, some hand calculations are made in order to indicate the condition of the caisson (see Appendix D). These conservative calculations are based on governing situations during uplifting or transportation and are a hand calculation to illustrate the strength of the walls and bottom of the structure.

## 4.2.1 Values, assumptions and premises

The caissons are built in a period when guidelines, concerning calculating concrete and concrete mixing, differed from modern ones. For this reason, some assumptions have to been done to convert out-dated values to modern standards.

The permissible concrete stress was the basis for the concrete calculations at that time. This stress is a known value. To obtain the corresponding characteristic strength, the following equation is used:

 $\sigma_c = 0.45 \cdot f_{ck} = 0.45 \cdot 15.3 = 6.9 \, N/mm^2$ 



In the following table, a summation of properties is shown.

Table 7 Concrete properties K225

|  | Values     |
|--|------------|
| Concrete class   | K225       |
| Concrete density (p)   | 25 kg/m³   |
| Permissible concrete compressive stress                          | 6.9 N/mm²  |
| Characteristic concrete cylinder compressive strength $(f_{ck})$ | 15.3 N/mm² |
| Design stress concrete $(f_{cd})$                                | 10 N/mm²   |

For the reinforcement steel, an allowable stress of 1400 kg/cm<sup>2</sup> (137 N/mm<sup>2</sup>) is given. On basis of a technical report of Rijkswaterstaat<sup>1</sup>, this reinforcement steel, at that time named steel quality QR24, has the properties shown in Table 8.

Table 8 Reinforcement properties QR24

|                               |            | Values                |
|-------------------------------|------------|-----------------------|
| Steel quality                 |            | QR24                  |
| Permissible steel stress      |            | 137 N/mm²             |
| Characteristic yield strength | $(f_{yk})$ | 240 N/mm <sup>2</sup> |
| Design stress reinforcement   | $(f_{yd})$ | 209 N/mm²             |
| Ductility class               |            | В                     |

# 4.3 Calculation method (Eurocode)

Eurocode guidelines are used to verify the concrete structure. The strength of the structure as well as the loads (partial safety factors, moments and shear forces) are determined according Eurocode and VBC 1990 guidelines. In the following section, a description and summation is specified of the used equations, factors, and the obtained results.

## Safety factors

In order to deal with uncertainties, safety factors are included in calculations for both loads as materials. This (partial) safety factors are determined by probabilistic based theories, since the strength of the

<sup>&</sup>lt;sup>1</sup> From "Richtlijnen Beoordeling Kunstwerken" bij Rijkswaterstaat, 2013



material as well as the loads have a variance in values. The guidelines offer constant safety factors for different load situations and materials. In the following table a summation of partial safety factors is given for a permanent situation.

**Table 9 Partial safety factors for materials** 

|  | Values |
|--|--------|
| Material factor concrete $(\gamma_c)$            | 1.5    |
| Material factor steel reinforcement $(\gamma_s)$ | 1.15   |

Former safety factors (see Table 9) are applied in situations where structures are in use and loaded by the design loads. However, understanding of the origin of partial safety factors is necessary for determining those factors in sporadic situations. Partial safety factors for civil structures are mainly based on:

- Loss of human lives;
- Economic damage;
- Time period of loading;
- Environmental damage.

Considering those aspects the partial safety factors can be adjusted to a temporarily situation where the caisson is replaced to a new location. In this situation the safety factors are less strict, for the next reasons:

1) The risk of losing human lives is tremendously reduced, because the caisson is not used as quay wall anymore during transportation;

2) Economic damage is expressed in monetary value of material losses. Removing all the structures on the quay wall before uplifting, leaves the caisson as only most obvious economic loss;

3) With extending the time of loading on a structure, uncertainties and risks increase towards failure. When considering a limited time frame risks can be determined more accurately and therefore reduce failure. Floating of the caissons should be considered in the time of optimal weather conditions and a minimum of wave impact in combination with accurate monitoring.

For this reasons, partial safety factors during floating (and transportation) are chosen different as for permanent and variable loading during service lifetime. For the outside water pressure a safety factor of 1.2 will be adopted and for inside water pressure, both as ballast and cooperating force, a factor of 0.9 will be used.





Figure 33 Safety factors during uplift and transport

## **Determination of loads & stability**

The walls and floor of the caisson can be represented schematically as slabs. For the moments and shear forces are not specified in the Eurocode, the determination of moments and shear forces are determined by use of VBC 1990<sup>2</sup> (see Appendix C).

Loads on the caisson differ for each situation and therefore a governing situation needs to be determined and examined. A possible situation that likely can occur, is the uplifting and transportation of the caisson to another location. In this case the ballast inside the caisson will be removed to a certain height so that the caisson will be lifted up in controlled manner. As shown by the quick calculations the uplifting is governing for the strength of the concrete structure.

First the draught of the caisson will be calculated. The starting point is without any ballast, after which the caisson is loaded with a half meter of ballast (water) per step. The initial draught, with the data from Table 10, is calculated as follow.

<sup>&</sup>lt;sup>2</sup> Voorschriften Beton en Constructieve eisen en rekenmethoden, part of NEN 6720 the Dutch guidelines.





Figure 34 Essential terms for uplifting and transports

| Table 3 | 10 Va | lues fo | or draugh | nt |
|---------|-------|---------|-----------|----|
|---------|-------|---------|-----------|----|

|                                | Values        |      |
|--------------------------------|---------------|------|
| Length                         | 43.65         | [m]  |
| Width bottom                   | 12.20         | [m]  |
| Width top                      | 9             | [m]  |
| Height                         | 12.50 - 13.40 | [m]  |
| Volume of concrete per caisson | 1062.3        | [m³] |
| Volume inside caisson          | 4000          | [m³] |
| Surface of ballast             | 336.2         | [m²] |

The gravity force of the caisson:  $F_c = V \cdot \rho = 1062.29 \cdot 25 = 26557 \ kN$ 

The draught of the caisson without ballast is:



$$d = \frac{F_c}{b \cdot l \cdot \gamma_w} = \frac{26557}{9 \cdot 43.65 \cdot 10} = 6.76 \, m$$

That means that  $\overline{KB} = \frac{1}{2} \cdot d = 3.39 m$ 

For the static stability the centre of gravity is calculated using AutoCAD, giving relative to the outer left corner of the bottom the following values:

x = 5.05 m and y = 4.55 m

Therefore  $\overline{KG}$  is equal to the x-position of the gravity centre.

For calculating  $\overline{BM}$ , the moment of inertia first will be determined:

$$I = \frac{1}{12} \cdot l \cdot b^3 = \frac{1}{12} \cdot 43.65 \cdot 9^3 = 2651.74 \, m^4$$

The displaced volume by the caisson:  $V = l \cdot b \cdot d = 43.65 \cdot 9 \cdot 6.76 = 2656 m^3$ 

$$\overline{BM} = \frac{I}{V} = \frac{2651.74}{2656} = 1.0 m$$

The criterion for static stability is assured when  $h_m > 0.5 m$ :

$$h_m = \overline{KB} + \overline{BM} - \overline{KG} = 3.39 + 1 - 4.55 = -0.16 m$$

So the criterion for static stability does not hold. Therefore, additional ballast is necessary in order to have a static stable caisson. From Figure 35, the minimum ballast is determined in order to meet the requirement of static stability. When an additional mass is added, in form of water ballast, the requirement is met when the water height in the caisson is 2.30 meter in all compartments.





**Figure 35 Determination metacentre** 

By adding mass, in this case water is used as ballast, the caisson will sink. With the same formulas, the draught is graphically displayed in Figure 36. The starting point of the graph is 6.76 meters, the initial draught as calculated above.



Figure 36 Draught of the caisson per half meter ballast

Before the caisson is fully filled, the draught will reach the total height of the caisson, specifically the back wall of the caisson with a height of 12.50 m. This point will be reached when all the compartments are filled up to 6.5 meters with water. In this case, the mean water depth of the (Waalhaven) harbour is 10.50 meters. The draught will be equal to the water depth when the caisson is filled with 4.50 meters water ballast. For the transportation of the caisson over water, there should be at least 1.00 m keel clearance below the caisson. Therefore a balance should be found between static stability of the caisson and keel clearance. To meet both requirements the water level in the Waalhaven should be equal to



MHWL (Mean High Water Level), which gives almost 1.00 m keel clearance and gives a metacentric height of 0.5 m in combination with 2.30 meter ballast.

Table 11 Critical draughts

|   | Water ballast/meter |
|---|---------------------|
| Draught equal to height of caisson              | 6.50 [m]            |
| Draught equal to water depth (10.50 m)          | 4.35 [m]            |
| Draught with respect to keel clearance (1.00 m) | 3.20 [m]            |
| Static stable draught (8.73 m)                  | 2.30 [m]            |

Figure 37 shows the calculated moments of resistance of the concrete and the acting moments during uplift of the caissons. As can see is the acting midspan moment on the wall (blue line) lower than the moment of resistance of the midspan of the wall (green line). The same holds for the support moments (red and violet line). Both the acting support as the midspan moments are smaller than the moment of resistance of the concrete wall, so uplifting is possible with respect to the concrete structure.



Figure 37 Resistance and design bending moment of outside wall for both support and midspan

## 4.3.1 Increased strength factors/proven strength

As earlier mentioned in the literature study, the strength of concrete increases over time. Especially a low strength concrete, containing coarse cement, has a higher increment compared with high strength concrete. Results from research studies have indicated an increase of the compressive concrete strength with a factor 1.63 and for low concrete strengths, as in this case, an increase of 2.40 is obtained. On the



other hand, the composition of the concrete mixture and the conditions vary for each situation and therefore destructive testing is needed to give a decisive answer on the concrete behaviour under different loads. Core drill tests have shown that the compressive strength of concrete has increased to an average of 58.70 N/mm<sup>2</sup>. This leads to a small increase of the moment of resistance. The shear force resistance on the other hand benefits clearly from the increase of the compressive strength.

The moment of resistance (midspan) and shear force resistance of the concrete outer wall (thickness 300 mm) have the following values, based on a compressive strength of 58.7 N/mm<sup>2</sup>.

$$M_{Rd} = A_s \cdot f_{yd} \cdot d \cdot \left(1 - 0.52 \cdot \rho \cdot \frac{f_{yd}}{f_{cd}}\right)$$

with:

$$\rho = \frac{A_s}{b \cdot d} = \frac{1340}{1000 \cdot 300} = 0.52 \cdot 10^{-2}$$
$$M_{Rd} = 1340 \cdot 209 \cdot 300 \cdot \left(1 - 0.52 \cdot 0.52 \cdot 10^{-2} \cdot \frac{209}{58.7}\right) \cdot 10^{-6} = 83.20 \ kNm/m$$

For the shear force resistance:

$$V_{Rd,c} = \frac{0.18}{\gamma} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} \cdot b_w \cdot d$$

with:

$$\rho_1 = \frac{A_s}{b \cdot d} = \frac{1586}{1000 \cdot 300} = 0.53 \cdot 10^{-2}$$

$$V_{Rd.c} = \frac{0.18}{1.5} \cdot (1 + \sqrt{\frac{200}{300}}) \cdot (100 \cdot 0.53 \cdot 10^{-2} \cdot 58.7)^{\frac{1}{3}} \cdot 1000 \cdot 300 \cdot 10^{-3} = 205.7 \ kN/m$$



The following table gives the different resistance values.

| $f_{ck}$ increase (factor) | Moment of resistance<br>midspan [kNm/m] | Moment of resistance<br>support [kNm/m] | Shear force resistance<br>[kN/m] |
|----------------------------|---|---|----------------------------------|
| 1.0                        | 80                                      | 94                                      | 114                              |
| 1.63                       | 81                                      | 96                                      | 134                              |
| 2.40                       | 82                                      | 97                                      | 153                              |
| 3.84                       | 00                                      | 08                                      | 206                              |
| (Empirical result)         | 00                                      | 50                                      | 200                              |

Table 12 Moment of resistance and shear force resistance

# 4.4 Conclusion

From the field-investigation report (see 3.3.3) it appeared that the current concrete strength is a factor 3.8 higher as the design concrete strength. This is a larger increase of strength compared with the provided factors in (experimental) researches. This means that the moment of resistance of the concrete is slightly increased with a factor 1.04. On the other hand, the shear force resistance benefits much more of the higher concrete strength with a factor 1.81. The hand-calculations showed, when the concrete strength is multiplied with a factor 3.8, the structure meets the requirements with respect to moment –and shear force resistance.

Due to the increase in concrete strength, the stricter requirements of the Eurocode are partly counterbalanced by this increase. However, one of the critical parts of the caisson that do not meet the Eurocode is the concrete cover. The 20 mm concrete cover is determined during design and is measured from the most outer reinforcement to the concrete surface. According to the Eurocode guidelines a minimum of 45 mm is required for concrete structures in class XS3.



# 5. Uplift & transport of caissons

# 5.1 Introduction

One of the possibilities that have to be taken into account, is when one or more of the caissons in the harbour will be transported to another location. Before drawing a construction plan, investigation of the construction method at that time will give insights which could be beneficial for setting up a construction plan. In this respect, the method how those caissons are built, transported and placed at the current location will be part of this chapter. The second part of this chapter will consist of the construction plan to lift up and transport the caissons and solutions for bottlenecks will be proposed in the final construction plan. Furthermore, costs of demolition will be included as zero alternative.

# 5.2 Construction method of old caissons

The caissons quay-walls in the Waalhaven are built in 1960 and placed along the south-side of the Pier 2 with a total length of 742 meters. The bottom and lower part of the caisson was built in a floating dock, where after the resting upperpart is floated outside the dock to be finished. Starting in meanwhile a new caissons in the floating dock. Due to the height and concrete development, the caissons are constructed in 6 phases of 3 meter. After finishing, the caissons are transported to their final position in the harbour.



Figure 38 From topside: vertical slots

Before situating the caissons on the bed, soil improvement was necessary to prevent sliding. The top of the soil improvement, approximately one meter, exists of very coarse gravel. By the supply of sludge, which endangers the stability, dredging the sludge right before placing the structure on the bed could prevent sliding. After placing the caisson by the inflow of

water through steel hatches, the rear is filled up

with a mix of clean fine, coarse and normal sea sand to avoid high (water) pressures on the back wall. Additional measurements to keep the caissons in line with a minimum displacement are so-called vertical slots (see Figure 38). Those vertical slots are filled up with gravel and concrete mortar and



prevent large singly movements. First the outer two slots are filled up with gravel, whereupon the compartments are filled with sand. During this operation movements of the caissons can occur. Concrete mortar is poured in the second vertical slot after which no movements can occur anymore. Those vertical slots, filled with concrete, are not flexible nor desirable and suitable for caissons that needs to be uplifted after a certain time.

## 5.2.1 Bottlenecks during uplift

According the calculations at that time which proceeded on the assumption that the caissons will not be lifted anymore after sinking, so therefore no account is taken of during design with this option.

Before and during uplifting and transportation some aspects should be taken into consideration.

- First of all the adhesive force does has a negative effect on uplifting force. This adhesive force is caused by the interaction between the caisson and soil of the bed, magnified by long time period. Depending on the soil properties of the bed, this should be counted for during uplift of the caissons.
- Secondly, the connection between the caissons, as described above, should be removed to unlock the caissons from each other. The gravel in the two outer slots prevent only movement in the horizontal direction. The concrete mortar restraints both horizontal as vertical movements.
- Thirdly, when sinking the caisson on the final position, water is used as ballast. Water flowed in the compartments through steel hatches which were closed after the right amount of ballast.
   Over the years, those steel hatches could be corroded and not function properly anymore.

# 5.3 Uplifting of the caisson

In this section the construction plan of the caissons at pier 2 will be elaborated. This construction plan is largely applicable for the other caissons built during the same period in the Waalhaven. The phases of construction are explained and supported where necessary with sketches.

The first phase is work preparation that needs to be done before the preparation of uplifting can take place. The second phase is uncovering the caissons and prepare the actual uplift.

## Phase 1.1: Removing surcharge loads

Quay walls have surcharge loads caused by cranes, buildings, storage goods, paving and etcetera. The first phase exist therefore from removing all the facilities and structures on top of the quay wall. In Figure 39 shows a cross section of the caisson with an upper structure to which element fenders and



bollards are fastened. The purlins are also attached to the caissons at a level of -0.18 m NAP which is above the MWL (-0.44 m NAP).

Important to notice, during this phase equipment loads should not exceed strength capacity of the caisson nor cause an unstable quay wall. Deconstruction can be executed both from landside and waterside which emerge another advantage; there is a possibility for shipment over water.





#### Phase 1.2: Placing sheet piles

The second stage of phase 1 consist of the protection of surrounding quay walls during the removal of the caissons from soil sliding and decrease of groundwater level and ensure the stability. Monitoring is desirable at this stage, and for the future phases, to anticipate on any movement of the caissons or adjacent quay walls. When on both side of the pier caissons are used as quay walls, additional protection could be reduced since those caissons have (almost) equivalent sides.

For the above reason it is worth to consider the sequence of removal, when other type of quay walls (sheet piles, diaphragm walls) are planned to be removed. When applicable, an appropriate sequence of removal of the different quay walls will reduce costs, additional measurements and save time.

#### Phase 2.1: Excavation of soil

In order to uncover the caissons the above soil layer and the soil behind the caissons should be excavated. The excavation of area B (see Figure 40) must be at least excavated to -12.35 m NAP to unload the top of the heel from soil pressure, which is favourable for uplifting.





Figure 40 Area A and area B with sheet pile

Table 13 Soil excavation

|   | Amount of soil       |
|---|----------------------|
| m <sup>3</sup> of soil to be excavated (Area A) | 31700 m <sup>3</sup> |
| m <sup>3</sup> of soil to be excavated (Area B) | 46375 m³             |
| Total m³  | 78075 m³             |

Excavation of area A can be carried out on the pier by an excavator and transport the removed soil by trucks. Disadvantage are the many trucks needed and facilitation of a work path for both the excavators as the trucks. Another opportunity to disposal the soil is by ship, which means large quantities can be transported at once and where excavators can operate from the caissons. The advantage of this option is that no equipment is stored at the quay wall (except excavators) and no work roads have to be created for trucks. Excavators will run on dragline mats for a stabile work ground and deposit the soil directly in the waiting barges. In the next table lists the most common type of barges with the main features.



| CEMT-class | Type barge  | Length [m] | Breadth [m] | Draught [m]  | Tonnage [t] |
|------------|-------------|------------|-------------|--------------|-------------|
|            |             |            |             | (when laden) |             |
| IV         | Europa I    | 70.00      | 9.50        | 3.0          | 1450        |
| Va         | Europa II   | 76.50      | 11.40       | 3.5          | 2450        |
| Va         | Europa II a | 76.50      | 11.40       | 4.0          | 2780        |
| Va         | Europa II a | 90.00      | 11.40       | 4.0          | 3220        |
|            | extended    |            |             |              |             |

Table 14 Characteristics of common barges (Waterway Guidelines Rijkswaterstaat, 2011)

#### Phase 2.2: Caisson separation

After the caissons are uncovered, the next step is to separate the caissons from each other. In Figure 38 one can see the interlock connection filled with concrete. Before uplifting of the caissons can begin, first the connections between the caissons should be broken. The unreinforced concrete mortar could have loss strength, by creep of concrete, influence of chlorides and movements of the caisson. Therefore the possibility exist the concrete mortar will breach if one of the caissons will move vertically. As this is an uncertainty, the connection will be sawed through once over the entire height, or only the upper part when resistance of the concrete mortar appears to be weaker than expected.

#### Phase 2.3: Uplift of caisson

After the caissons are separated from each other, the actual uplifting can start by removing the sand ballast inside the compartments and eventual refill it simultaneously with water. Water can better be controlled while pumping it out the compartments than sand. Also due to its liquidity water has the ability to distribute equally over the area in the compartments. The current sand ballast inside the caissons can be removed by sucking the sand out the compartments through a Toyopump.

The caissons are designed with steel hatches for controlling water inside the compartments. By opening and closing the hatches water can flow in or out the caisson, depending on the situation. However, after 60 years it is highly likely that the hatches, made of steel, are rusted and cannot be opened or closed appropriately or in the worst scenario rusted away. To prevent this risk, water pumps are needed to control the water ballast inside the caissons instead of the steel hatches after divers inspected the steel hatches.

The threshold for uplifting is calculated, in paragraph 4.3 to be approximately 6.5 metres in all compartments (see Figure 41). When the water height in the caisson drops below this value of 6.5



metres, uplift is instigated. In the same paragraph 4.3 (see Figure 36) the desired ballast is calculated to fulfil the requirement of static stability. For the static stability of the caissons, a minimum (water) ballast of 4.5 metres is desired in all compartments. To meet also the requirement of 1.00 meter keel clearance, the caisson should be uplifted during Mean High Water Level.



Figure 41 Essential terms for uplifting and transports

The bed on which the caissons rest consists of very coarse gravel where water easily pass through. In contrast of sand, very coarse gravel as in this case does not cause an adhesive force during uplifting of the caisson.

## **Phase 3: Transportation of caissons**

After the caissons is uplifted and the static and dynamic stability is secured, the caissons can be towed to another location in the vicinity or attainable location. Tugboats can escort and tow the caissons out the harbour. Each caisson should have at least two tugboats to pull one of the front corners.

## **Important notes**

 The (water) ballast should be equally distributed over all compartments in order to ensure the stability of the caissons during uplifting.



- The uplifting of the caissons should be done during Mean High Water Level (MHWL) so that the keel clearance of 1 meter can be satisfied.
- It is very likely that the steel hatches of the caissons, used for inflow of water in the compartments, are rusted. Therefore water pumps are needed for in –and outflow of water in the caissons and controlling the ballast inside the caissons for uplifiting.


# 6. Risk Management

Risk is everywhere. From crossing the road to parachuting, risk is inherent in the actions we select. Within a project, risks are unforeseen events or circumstances that can have a positive or negative effect on its realisation. Not all risks are bad, but almost all are seen as a threat.

As any other civil project, risk is an element which not should be underestimated. A thorough risk analysis can therefore prevent unpleasant surprises and save time and costs. Moreover, a risk analysis should also cover the prevention measurements to lower potential risks and clarifies who is liable for the prevention and consequences of any risk. Risk analysis can also be a handy tool for choosing between alternatives where risks differ in probability and impact. Therefore, for this reasons, this chapter will address the risks associated with the uplifting of the caisson and propose potential measurements to reduce risk.

# 6.1 Qualitative risk analysis

# 6.1.1 Risk process analysis

Risk analysis is the process of defining, analysing and developing of strategies to prevent or reduce impact of hazards. The risk analysis process has the following steps:

- 1. Identify the risk
  - First of all the risks should be identified which might harm the structure, environment, financial position, process, human lives and so on. Risks needs to be identified from different perspectives.
- 2. Asses the risk
  - Second step is to assess what or who might be harmed by the risk(s) and what the consequences will be if the hazard occurs.
- 3. Develop countermeasures to the risk
  - Risk can be managed in two ways; eliminating the hazard or introduce measurements to ensure a hazard becomes unlikely. If the latter occurs, the question arise what to do to manage the risk?
- 4. Contingency plan
  - If one or more of the risks befall, a contingency plan contains the information how to manage and response to risks.



# 6.1.2 Identify and asses risks

First of all risk has many sources and can be divided in predicable and unpredictable risks. Examples of unpredictable risks are natural disasters, sabotage and unforeseen governing requirements. Predictable risks be classified in different categories of risk depending on origin of the hazard. Among the categories are legal risks, project management risks and technological risks. This section will focus mainly on the technological and environmental risks that can befall during uplifting and transportation of the caissons.

# Risks

The following table shows risks that likely can occur, their consequence and the probability. In this part the focus will lie mostly on technical and operational risk before and during uplifting. Each event is coupled to a risk and its consequence. The severity of the events are determined qualitatively according Table 16 on basis of impact and probability of occurrence, and classified in *high, medium* or *low* risk. The risks are in the tables 17 to 20 sorted from high to low.

|             |             | Trivial | Minor  | Impac<br>Moderate | t<br>Major | Extreme |
|-------------|-------------|---------|--------|-------------------|------------|---------|
| Probability | Rare        | Low     | Low    | Low               | Medium     | Medium  |
|             | Unlikely    | Low     | Low    | Medium            | Medium     | Medium  |
|             | Moderate    | Low     | Medium | Medium            | Medium     | High    |
|             | Likely      | Medium  | Medium | Medium            | High       | High    |
|             | Very likely | Medium  | Medium | High              | High       | High    |

### Table 15 Qualification of risk (based on ISO31000:2009)



Table 16 Operational risk events

| Category    | Event  | Risk   | Consequence                                      | Impact -<br>Probability   |
|-------------|--|--|--|---------------------------|
| Operational | Steel hatches are<br>rusted                            | Steel hatch does not open or close                                   | Water inside compartments cannot be controlled   | Moderate – Very<br>Likely |
| Operational | Steel hatches are rusted away                          | Water can flow in  | Water inside compartments cannot be controlled   | Moderate – Likely         |
| Operational | Ballast not equally<br>distributed                     | Static stability<br>requirement is not<br>fulfilled                  | Caisson will tilt and fill up with water         | Major - Unlikely          |
| Operational | Ship comes to close<br>to caissons                     | Collision of ship<br>against caissons                                | Caissons and/or ship<br>damaged                  | Major - Unlikely          |
| Operational | Defective<br>equipment's and/or<br>materials           | Operations are<br>suspended  | Delays for 1-3 days and additional costs         | Moderate -<br>Unlikely    |
| Operational | Removing surcharge,<br>starting from quay<br>wall side | Caisson will slide<br>away   | Structure failure, loss of utility and equipment | Moderate - Rare           |
| Operational | Soil adheres to<br>bottom of caisson                   | Uplift force not<br>sufficient to<br>counteract<br>additional weight | The caisson cannot be uplifted                   | Minor - Unlikely          |

## Table 17 Technical risk events

| Category  | Event                           | Risk   | Consequence  | Impact -<br>Probability |
|-----------|---------------------------------|--|--|-------------------------|
| Technical | Overestimated concrete strength | Structure capacity not<br>sufficient (at a specific<br>moment) | Damage of structure or structure failure                           | Extreme - unlikely      |
| Technical | Ballast not equally distributed | Structure capacity is not sufficient                           | Walls/bottom will be<br>damaged or structure<br>failure will occur | Major - Moderate        |



#### Table 18 Design risk events

| Category                   | Event   | Risk                                      | Consequence                               | Impact -<br>Probability |
|----------------------------|---|---|---|-------------------------|
| Design                     | Incomplete and/or missing drawings            | Renew execution plan<br>or specific steps | Delays for 1-2 weeks and additional costs | Moderate - Likely       |
| Design                     | Unknown material specifications               |   | Delays for 1-2 weeks and additional costs | Moderate - Likely       |
| Design/work<br>preparation | Unsuitable<br>equipment's and/or<br>materials | Operations are<br>suspended               | Delays for 1-3 days                       | Moderate - Minor        |

#### Table 19 Unforeseen risk events

| Category                    | Event   | Risk                                       | Consequence                               | Impact -<br>Probability |
|-----------------------------|---|--|---|-------------------------|
| Unforeseen<br>circumstances | Finding of obstacles into<br>the soil (cables, old<br>foundation piles) | Identifying obstacles<br>and devise a plan | Delays for 1-3 weeks and additional costs | Minor - unlikely        |
| Unforeseen<br>circumstances | Extreme and unusual weather condition                                   | Uplifting not executable                   | Delays                                    | Moderate - rare         |

Besides the technical and operational risks mentioned above, several of other risks can occur before, during and even after a project. Risks can emerge on any level of the project, for example, during or owning to project management, design process or political factors. Those risks will be described without specifying associated risks.

#### Organizational/project management risks

- Inexperienced staff members
- Estimating and planning errors
- Stagnation of supply of materials and equipment

# **Financial risks**

- Bankruptcy contractor
- Large delays by any event

## **External risks**

• Legal risks to demolish the caissons



- (Abrupt) change in legalisation
- o No provision of permits
- Social risks
  - o Vandalism of material
- Environmental risks
  - o (Ground)water pollution due to (fine dust) of the concrete during demolition
  - o Air pollution due to emissions of machines and fine dust
  - o Contaminated soil caused by concrete particles
  - o Degradation of flora on the old quay walls

# 6.2 Mitigation & prevention measurements

Once the identified risks have been prioritised, promising measurements should be considered. To avoid a risk, reduce the likelihood of a risk or mitigate the impact of it, measurements should be introduced on forehand. This section will introduce measurements to manage the risks of paragraph 6.1.2. The most promising measurements are listed in Table 21-23. For any risk the proposed measurement needs to reduce the impact and/or probability of occurrence of the risks, or even eliminate the risks.

| Event  | Risk   | Consequence  | Prevention - mitigation   |
|--|--|--|---|
| Steel hatches are<br>rusted                            | Steel hatch does<br>not open or close                                | Water inside<br>compartments cannot<br>be controlled | Use of water pumps to control<br>ballast inside instead of steel<br>hatches |
| Ballast not equally<br>distributed                     | Static stability<br>requirement is<br>not fulfilled                  | Caisson will tilt and fill up with water             | Supervision of (contracting)<br>engineer                                    |
| Ship comes to close<br>to caissons                     | Collision of ship<br>against caissons                                | Caissons and/or ship<br>damaged                      | Placing fenders to reduce impact collision and clear visibility of caissons |
| Defective<br>equipment's and/or<br>materials           | Operations are suspended   | Delays and additional costs                          | Having backup pieces and repairers in case of defects                       |
| Removing<br>surcharge, starting<br>from quay wall side | Caisson will slide<br>away   | Structure failure, loss of utility and equipment     | Start from backside to minimize sliding of caisson                          |
| Soil adheres to<br>bottom of caisson                   | Uplift force not<br>sufficient to<br>counteract<br>additional weight | The caisson cannot be uplifted                       | Remove soil by jet pipe underneath the caisson                              |

### Table 20 Measurements for operational risks





# Table 21 Measurements for technical risks

| Event                              | Risk  | Consequence  | Prevention - mitigation                  |
|------------------------------------|---|--|--|
| Overestimated concrete strength    | Structure capacity<br>not sufficient (at<br>a specific<br>moment) | Damage of structure or structure failure                           | Accurate testing by core drill           |
| Ballast not equally<br>distributed | Structure capacity is not sufficient                              | Walls/bottom will be<br>damaged or structure<br>failure will occur | Supervision of (contracting)<br>engineer |

Table 22 Measurements for design risks

| Event   | Risk   | Consequence                 | Prevention - mitigation                       |
|---|--|-----------------------------|---|
| Incomplete and/or<br>incorrect data           | Renew execution<br>plan or specific<br>steps | Delays and additional costs | In case of no date, well-founded assumptions. |
| Unsuitable<br>equipment's and/or<br>materials | Operations are suspended                     | Delays                      | Factoring in extra time for this operations   |



# 7. Alternative functions & demolition costs

The aim of this chapter is to propose the most suitable functions for the caisson (type P) from the Waalhaven and check the structural elements according to the current guidelines. First the focus will be on limitations and possibilities of the caissons and, in addition, measures to expand potential functions. Also a range of potential functions as discussed during the first section of this chapter. For each function the advantages and drawbacks will be mentioned and explained.

# 7.1 Reuse possibilities

Caissons can be designed and built for a variety of purposes due to their almost uniform structure. For this reason the caisson quay walls could be reused and serve another function. Furthermore, the range of possibilities can be enlarged, if necessary and beneficial, by several additional measures and adaptations of the caisson. This will be discussed in the subsequent section, followed by interesting and promising functions in the second section. In chapter 8 one of the functions will be chosen using a multi-criteria decision matrix and elaborated.

# 7.1.1 Functions

In the literature study (see 3.2.1) some general functions of caissons were discussed. Some of those functions are applicable for the relevant caissons in the Waalhaven and are described also in this section. Besides those functions, some other promising purposes of the caissons will be described including the advantages and disadvantages.

# 1. Caisson as fruit storage

Daily tons of goods are transported, stored and distributed in the Port of Rotterdam, including fruit, vegetables and fresh products. The processing of these goods takes place in the Fruitport Rotterdam (FPR), located in the Merwehaven. This harbour section controls the handling, processing and distribution of fresh fruit and vegetables. Fruits and vegetables are sensitive goods that need proper storage areas, where cooling, ripening and packaging can take place. Fruitport Rotterdam processed daily tons of fruits and vegetables in refrigerated warehouses with a total area of 50 000 m<sup>2</sup>.

In the context of expansion for the storage of refrigerated fruit, there is a design study carried out to store the fruit in underground facilities (Knibbe, 1997). Using caissons as storage, the fruit terminal can also be located above the ground on a different location. The fruit and vegetables are transported in pallets, with a surface of approximately 1 m<sup>2</sup> per pallet and have a height of 2 meters. A maximum of four pallets can be stored on each other, so it is reachable for forklifts.





Figure 42 A reefer container

However, present the so-called reefer containers (refrigerated container) are more usual in transporting cooled fruit and vegetables. These reefer containers have typical dimensions ( $5.46 \times 2.30 \times 2.27 \text{ m}^3$ ) or ( $11.59 \times 2.29 \times 2.57 \text{ m}^3$ ) and are fitted with refrigeration units and an engine for cooling. With this

dimensions the compartments are too small. The containers (5.46 x 2.30 x 2.27 m<sup>3</sup>) could be stored in different positions (see Figure 43 and 44). A critical point when using the above mentioned containers, is the fact that the compartment walls need to be removed. It does not matter in which position the containers are placed, the compartment walls should mainly be removed which actually makes underground storage not possible without The taking measures.



Figure 43 Plan view of possibility 1 of reefer containers



Figure 44 Plan vie of possibility 2 of reefer containers compartment walls support the outer walls against external forces. In this case the external forces exist of soil and hydrostatic pressure. Removing those walls cause decrease of the bearing capacity of the concrete structure. To increase the bearing capacity, struts can be placed between outer walls to replace the compartment walls. However, struts impacts the movability when hoisting containers in and out of the caissons.



For temporarily underground storage of fruit, caissons can be an option to use, especially when standard units are used. Possible bottlenecks can be the researchable of specific containers in underground storages and likely to transport the caissons to another location. A point of concern is the ventilation to remove the generated heat, so a ventilation shaft is required for an appropriate ventilation. Some reefer containers are using water cooling systems, in spaces without no adequate ventilation. Nevertheless, the use of the water cooling systems declines because of the high costs of those systems.



Figure 45 Underground storage of containers

The critical points of this functions are listed below:

- Removing compartments is necessary for underground storage of conventional containers;
- Replacement for compartment walls when removing is necessary;
- Additional space should be reserved for ventilation shafts when refrigerated containers are placed;



## 2. Caissons for LNG storage

LNG stands for Liquefied Natural Gas that has been converted, for transportations and storage purposes, from gas to liquid. LNG becomes liquid at a temperature of minus 162 degree Celsius. In comparison with diesel and fuel oil, which is the usual fuel for ships and barges, the emissions are many times higher than when using LNG. The maritime and inland waterway vessels are highly polluting and therefore very undesirable, especially in urban areas. Hence, LNG is introduced to be used for inland vessels and in the future maritime vessels. Comparing Liquefied Natural Gas (LNG) to the current fuel, emissions of vessels (mainly particulates, sulphur oxides and nitrogen oxides) will be reduce by 80 to even 95 % of current levels and  $CO_2$  levels by 20 %. Besides air pollution, also noise nuisance will reduce which is advantageous, especially in ports, which are in or in vicinity of urban areas.

Concrete silos and steel tanks are commonly used for storage of LNG, though caissons in general are suitable for storage of LNG. The inner wall, where LNG is stored, is made of high-nickel steel. An important issue is the extremely isolation needed to minimize evaporation due to large differences between temperature of the gas (-162 °C) and outside temperatures. For this functions, as LNG storage, a top deck needs to be constructed for protecting LNG, but also for installations for pumping and keeping the gas liquefied. These caissons are used as LNG storage and gas station for inland waterways on strategic locations such as ports and along the river banks. Vessels can refuel during their trip along the rivers and reduce emissions.



Figure 46 Cutaway section of an ordinary LNG tank



Before the caissons can be used as LNG storage, the structures should meet the safety and environmental requirements. Guidelines for storage of LNG are set out in PGS 33-1 and Pressure Equipment Directive (97/23/EG).

To see whether it is constructive feasible, some hand calculations will check the structure. Consider the caisson is totally filled with LNG and no external loads are present by soil or water. The mass density of LNG is 450 kg/m<sup>3</sup>, equivalent to 4.5 kN/m<sup>3</sup>. The moment of resistance and shear force resistance of the concrete have the following values, based on a compressive strength of 58.7 N/mm<sup>2</sup>.

**Table 23 Concrete resistance values** 

| Moment of resistance | Moment of resistance | Shear force resistance |  |
|----------------------|----------------------|------------------------|--|
| midspan              | support              |                        |  |
| 83 [kNm/m]           | 98 [kNm/m]           | 206 [kN/m]             |  |

When the caisson is totally filled with LNG, this gives a load of:

 $\gamma_0 \cdot \rho \cdot h = 1.5 \cdot 4.5 \cdot 12 = 81 \ kN/m^2$ 

At midspan the moment this gives a moment of:

$$M_{ms} = 0.001 \cdot C_1 \cdot p_d \cdot l_x = 0.001 \cdot 71 \cdot 81 \cdot 3,25^2 = 61 \ kNm/m$$

And for at the support:

$$M_{sp} = 0.001 \cdot C_2 \cdot p_d \cdot l_x = 0.001 \cdot 85 \cdot 81 \cdot 3,25^2 = 73 \ kNm/m$$

The calculation meets the Eurocode requirements. Placing the caisson in water or surrounded by soil, both cases with an empty caisson, requires a calculation check. According the same method as above, choosing a density of 1000 kg/m<sup>3</sup> for water and 2000 kg/m<sup>3</sup> for soil (wet sand), it gives the following results.

The caissons can be submerged up to 9 meter under water and for soil, a height of approximately 4 meter is the limit.

Critical points:

- The adaptations costs for those existing structures could be more expensive as for a new structure;
- LNG storage should meet safety and environmental requirements;
- The caisson should be submerged or repaired due to corrosion of the walls.



## 3. Caissons as foundation for buildings

Offices, storages, and residential buildings need all foundations for bearing capacity to the subsoil. Foundations can consist of piles, but also caissons could function as foundations for (residential) buildings. Taking into account that in the future the Merwehaven and Waalhaven will probably be redeveloped as an area for living working and recreation, caissons can be used as foundations for buildings along the waterside. Furthermore, a part of the caissons (the upper part) can be used as basement. For using it as foundation and basement, a top deck on the caissons is necessary.



Figure 47 Art impression of Merwehaven (NHTV Breda)

From a constructive viewpoint, the column pattern of the buildings should coincide with the walls of the caissons. This to minimize tensile stresses in the concrete structure and stability issues. Therefore the width will be approximately 12 meter from quay wall side (see Figure 48).





For this purpose, the caissons probably do not have to be transported and can stay at the current location. The Merwehaven and Waalhaven are located close to residential areas. For construction of new buildings, especially when it comes to foundation works, local residents encounter (noise) disturbance. Therefore, the main advantage of using caissons as foundations compared traditional to foundations, the massive is reduction in noise and much faster construction time on the spot.

Figure 48 Cross-section caisson as foundation for buildings

Unlike in temporary buildings, the service lifetime of the caissons, should be equal to the service lifetime of the buildings on top of it. Besides using the caissons as foundation for permanent buildings, it also can serve as foundation for temporary buildings or structures or temporary in function. For example, congress hall, circus, temporary school and offices.

Table 24 Advantages and disadvantages Building foundation

| Advantages                                       | Disadvantages   |
|--|---|
| No transportation needed                         | Reaming lifetime caissons should be equal to the lifetime of the buildings on top |
| Low redevelopment costs of foundation            | Uneven settlements can occur between caissons                                     |
| No nuisance compared to conventional foundations |   |



Another option is to use caissons as elements for floating houses. Advantage to the prior idea is the flexibility of location which also can be on temporarily bases. Not all locations are suitable to use it for houses. The location should be free from high waves and large difference in water levels, and well accessible from landside. Floating houses could arrange an optimal use of space in densely populated areas such as cities with a relatively large water surface. Beside the optimal use, people may even



Vermeer)

prefer a house on water over a normal house by capabilities like mooring places for boats and recreation.

Because water levels can fluctuate, mooring places and land access roads should be designed such to stay accessible at any time. Extreme weather conditions are a point of concern, such as extreme high (or low) water levels, storm and eventually evacuation. But also ship collision is a conceivable risk, particularly on shipping lanes.

| Advantages                            | Disadvantages                                    |
|---------------------------------------|--|
| No nuisance compared to conventional  | Reaming lifetime caissons should be equal to the |
| foundations                           | lifetime of the buildings on top                 |
| Ontimal use in densely populated area | Flood risk can cause damage and undesired        |
| optimal use in densely populated area | floating   |
| Relocating is possible                | Ship collision                                   |

Table 25 Advantages and disadvantages floating houses

For floating buildings, weight of the houses is an extremely important load on the caissons. Floating foundations have a limited bearing capacity, because the buoyant force remains almost the same but the weight increases. Therefore lightweight materials are an alternative to conventional building materials like concrete, steel and timber to reduce weight.





Figure 50 Flow chart possibilities for building foundation

When reusing the caissons as building foundation in the Waalhaven and therefore transform the area from an industrial to a residential area, the land value per square meter can increase tremendously. The total area that is available for commercial and/or residential buildings can be simple calculated by considering Figure 48. Assume the total width of the building is 8 meter with a length 742 meter (17 caissons) and a land value of  $\leq 600/m^2$ :

*Groundvalue* =  $(742 \cdot 8) \cdot 600 = \text{€}3.561.600, -$ 



### 4. Caissons as breakwater

Breakwaters are used to protect structures, harbours, shorelines and/or worksites from waves by reducing the intensity of wave actions. Breakwaters provide safe harbourage for ships and reduce wave

loads on quay walls in harbours and overtopping. These structures are also used to protect coast lines against erosion by decreasing the impact on the coast. Besides permanent use, breakwaters can be used on temporarily base during construction of a harbour, windmill farm, or any other project. The advantage of Figure 51 Floating breakwater (Baltic Floating Structures) caisson breakwaters over other types of



breakwaters, is the relative simple execution, since the caissons can be constructed on a more favourable location and then transported to the final location.



Figure 52 Caisson as breakwater (European Coasts, figure 76)

Caissons have the capacity to float and therefore they can be used as floating breakwater. This type of breakwater are suitable and preferred over other breakwaters in cases of deep water or poor foundations possibilities. Also there is no need for constructing an (expensive) foundation. (Floating) breakwaters can be used for or in combination with:

- To protect ships and quay walls in harbours
- To protect a beach
- As quay wall for mooring (at inner face of the breakwater)



Or similar hydraulic functions as:

- Groyne
- Pier

However, optimal shapes of floating breakwaters differs from breakwaters founded on a sill, especially in long wave regions. When floating breakwaters are used in regions where long waves occur, like in deep seas, it will follow the wave motion as a small object. Waves impact also the dynamic stability of structures like caissons. The maximum wave length could be determined from the next requirements, with the minimum as the governing permissible wave length:

$$L_w < 0.7 \cdot l_e \rightarrow L_w < 0.7 \cdot 43.65 = 30.56 \, m$$

or

 $L_w < 0.7 \cdot l_b \rightarrow L_w < 0.7 \cdot 12 = 8.4 m$ 

As for the permissible wave height, the following rule of thumb can be applied presuming the linear wave theory for non-breaking waves:

 $F_{max} = \frac{1}{2} \cdot \rho \cdot g \cdot H_i^2 + \rho \cdot g \cdot H_i$   $F_{max} = 206 \, kN \, (Shear force resistance)$  $H_i = the wave height of an incoming wave [m]$ 

With  $H_i$  as two times the amplitude H, in case of total reflection of the wave.

$$206 = \frac{1}{2} \cdot 10 \cdot 10 \cdot H_i^2 + 10 \cdot 10 \cdot H_i$$

Solving this equation obtains:  $H_i = 1,26 \text{ m and } H = 2 \cdot H_i = 2,52 \text{ m}.$ 

This rule of thumb gives a quick estimation of the upper bound of the wave height, which is 2.52 meter primarily based on the shear force resistance of the concrete. From other criteria and requirements, the estimated values can differ.

 Table 26 Estimated wave boundaries

| Maximum wave length | 8.40 m |
|---------------------|--------|
| Maximum wave height | 2.52 m |



Table 27 Advantages and disadvantages

| Advantages                                       | Disadvantages  |
|--|--|
| Breakwaters can be used temporarily as permanent | Could not be used in any situation (high and long waves) |
| Are relatively fast to remove                    | Relatively thin walls                                    |
|  | Splash zone sensitive for steel corrosion                |



## 5. Caissons as (floating) light traffic road

Connecting two parts of land can be done in several ways. One of the possibilities is to use the caissons as a (floating) bridge. By linking the caissons to each other at the front side, a corridor can be formed through water. The (floating) road is intended for light traffic, as pedestrians and bicycles, but could also be used for motor vehicles. Whether the caissons should float or be positioned on the bottom, depends mainly on the local water depth and requirements for passage of shipping. When the water depth is large enough, the design can be such that shipping can pass through. To achieve this, a moveable bridge needs to be constructed on the most suitable location. In this case it will be a floating and moveable bridge to maintain the waterway accessible. In general, there are two possibilities. Possibility one is that the caisson is sinking down (see Figure 54) and possibility two is that the caissons slides horizontal two create a gap for shipping (see Figure 55)



Figure 53 Floating road (Bayards)

Floating roads can provide access in areas where water is frequently allowed to overflow as control mechanism or could be a permanent road in areas with unsuitable foundation soil. It would also be particularly useful in case of road works near a river or canal, minimizing disruption. However, it requires quick assemblage and easy relocation. Concrete caissons, especially of this format, are less easy and economical to relocate on short-terms. Use of such caissons should be primarily aimed for longer periods.



### Figure 54 Side view sinking caisson for floating bridge





Figure 55 Plan view sliding caisson for floating bridge

In the service state, it is required that waves has no impact on the floating road by means of swell and undesired vibrations, causing unsafety, damage to the structure as well as reducing driving comfort. A crucial element are the joints between the caissons. Those connections should be stiff enough to prevent that each caisson moves separately, but also flexible to reduce forces on the joints. For linking caissons to each other, one can construct a continuous top deck and/or fill up the interlock (see Figure 38) with concrete.

Measurements and calculations for the structures should involve movements of vehicles for comfort (in six directions) and loads on the structure and displacement of the structure (by vehicles, waves, currents, wind and ship collision). This movements are unfavourable for the floating caissons. By placing piles that are fixed into the bed, movements can be reduced, leading the forces to the bed.

Table 28 Advantages and disadvantages Floating road

| Advantages   | Disadvantages                                  |
|--|--|
| No expensive foundation needed compared to conventional alternatives | Moveable bridge needed for accessible waterway |
| Demountable and moveable   |  |
|  |  |

Flexibility to move with water level



## 6. Floating swimming pool

Converting caissons to a swimming pool is one of the simplest option when it comes to adjustments. Caissons can be adapted to simple swim pools with basic facilities to advanced pools for training purposes or luxurious amenities. Possible locations for the swimming pools ranges from along a river in the city to the sea coast during summertime. Figure 54 shows a concept idea of a floating swimming pool.



Figure 56 Floating swimming pool (Sculp(IT) Architecten)

The depth of a swimming pool depends on its purposes, and whether it is used as a private or public pool. An average depth of 2 meters is commonly used for public pools, whether for children this depth is varying between 0.8 meter and 1.2 meter. For training purposes, for example diving and rescue courses, the depths can be up to 6 meters. Extending the use of swimming pools for different purposes, some facilities can be provided. One of the main functions to enhance the possibilities of the swimming pool, is a moveable pool bottom. Varying the water depth gives opportunities for the above mentioned purposes. Also an indoor pool can be realized for protecting the pool from weather conditions.



Figure 57 Design floating pool for training purposes (Royal HaskoningDHV)

In order to make the caissons suitable as swimming pool, the upper part of the compartment walls (amount of height depending on required pool depth) needs to be removed and a waterproof floor to be constructed. Moreover, a mooring place for small boats is an option to create an additional opportunity to reach the floating swimming pool.



The above mentioned modifications to the concrete structure mainly depend on freeboard and the desirable water depth. Removing a part of the compartment walls have impact on the bearing capacity of the outer walls. Two situations can occur, that can be governing for the walls. The first is when the swimming pool is empty of water and the second case when the pool is in use and filled with water. The last case is also of interest for determining the thickness of the concrete floor.

Considering a floating swimming pool totally filled with water (assuming a pool depth of 3 meter). The draught of the caisson, without additional ballast and estimating the floor thickness (300 mm), results in 9 meter. The freeboard is therefore 3.50 meter, which means an inside water load of 3 meter on the outer walls (see Figure 56).



Figure 58 Cross-section swimming pool

The floor thickness of the swimming pool, can be estimated with the table of VBC 1990 (see Appendix C) by assuming the floor as concrete slab. When the loads in Table 30 are considered, along with the determined figures in Table 31, the moments on the floor can be estimated.

Table 29 Loads on pool floor

|               | Load      | Load factor |
|---------------|-----------|-------------|
| Static load   | 30 kN/m²  | 1.2         |
| Variable load | 2.5 kN/m² | 1.5         |



Table 30 Figures from VBC 1990 for slab II

| $l_y/l_x$                 | 1.26 |
|---------------------------|------|
| C-coefficient for midspan | 28   |
| C-coefficient for support | 66   |

$$\begin{split} m_{vx} &= 0.001 \cdot C \cdot p_d \cdot {l_x}^2 = 0.001 \cdot 28 \cdot (1.2 \cdot 30 + 1.5 \cdot 2.5) \cdot 3.25^2 = 10.5 \, kNm \\ m_{sx} &= -0.001 \cdot C \cdot p_d \cdot {l_x}^2 = -0.001 \cdot 66 \cdot (1.2 \cdot 30 + 1.5 \cdot 2.5) \cdot 3.25^2 = 27.7 \, kNm \end{split}$$

The acting moments are small due to the low loads acting on the floor and relative small distances to the supports. Therefore a floor thickness of 200 mm is sufficient for this function as swimming pool.



## 7. Floating quay wall

The current function of the caissons can be continued, but in in a floating variant. Floating quay walls are advantageous in cases of land scarcity, which most city ports are, to offer additional mooring places, higher transhipment capacity and flexibility within ports. Increase of container ships leads to increased water depths in ports, and rising of seawater levels gives exactly the opposite problem. Floating quay walls handle both issues and hence are a promising solution for further problems.

Floating quay walls, in the form of caissons, have in the port multiple application possibilities and due to the flexibility of the caissons, they can each time used for another mode or simultaneously. To increase

the surface area of quay walls the caissons can linked to each other and offer more surface to unload containers. Depending on the amount of containers, caissons can linked to each other to meet the demanded surface area. It can even done in such a way to provide unloading of two ships at the same time, each on one side (see Figure 57). This principle is also applicable for a fixed and floating quay wall. Enhancing the unloading time and so decreasing mooring time, offers a competitive opportunity over other ports.



Figure 59 Unloading floating quay wall

Besides using caissons as floating quay walls within ports, multiple caissons may forming a floating harbour in –or outside the port. Maintenance work, expansion, temporary increase of incoming ships, extreme weather conditions, are examples of cases that temporary floating ports could be useful. Both the floating quay walls as breakwaters could be constructed from caissons.



Figure 60 Floating port (Baltic Floating Structures)



Mulberry Harbour, constructed by the Allies in the Second World War as temporary harbour, was built of caissons which served as breakwaters and quay walls. Advantage of this construction method, is the time-saving construction time, the easy mountable elements and the economic value of temporary harbour. Those type of harbours can be used for different functions, for example as fishing harbour, container terminal or leisure port.

For container terminal, the floating port should expect large container handling. Available space on quay walls, or maybe even storage of containers in quay walls, is an important requirement for container handling. Besides the available space, cranes need to be present to load and unload containers. Floating quay walls that are not adjacent to landside have a minimum space for (large) cranes. For that reason, it is convenient to place small quay walls adjacent to land and wider quay walls into the water (see Figure 59).



Figure 61 Plan view of floating port

Fishing ports are require other demands than large container ports. Fishing boats are much smaller, usually less than 30 meters and a draught of a few meters for the largest vessels. For fishing boats the quay walls needs to be low, with a maximum of 2 meters above the waterline to remain accessible. Most fishing vessels are for commercial purposes and must be located near fish areas, like seas and oceans. In the Netherlands, it are mainly the regions, the North Sea, Wadden Sea and IJsselmeer. Fish is a fresh product which demands appropriate storage in the port, so space should be available for refrigerated storing. Fishing ports should have, especially when situated near seas or large lakes, breakwaters to protect the small vessels against (wind) waves which are sensitive to waves.



Recreational ships have almost the same dimensions as fishing vessels, only the fact that the former ships also could be much smaller than fishing vessels. Quay walls are primarily used for mooring only. Primary objective of a leisure ports could be mooring places or combined with other functions such as hospitality venues and fishing port.



### 8. Floating stadium

Large cities all have a common problem, and that's limited space, particularly in the city centre. Therefore often sports facilities are also located on the outskirts of the city, such as soccer fields and

tennis courts. These fields, which take up a lot of space, have in general an economic low value. Floating sport fields could be located near river banks on floating caissons for public or commercial use. The sport fields can be expanded by tribunes on landside and connected through a footbridge. Besides the function for sport purposes, it confers excellent for events.



Figure 62 Floating sport field in Marina Bay, Singapore

In areas with waves, currents and shipping, piles fixed into the bed can act as foundation for the caissons and minimize undesired movements. Underneath the field, constructed on a concrete deck, space is reserved for power supply and a drainage system. This space should be accessible for maintenance and repair.

Dimensions of sport fields vary by sport. The most common sports are soccer and tennis, both with different field dimensions. Those dimensions are governing to determine the minimum required caissons to realize a sport field. In Table 32 the dimensions are given. The second column shows the actual dimensions of the field and the second column the total surface of the caissons taking into account some additional space for facilities and aisles.

Table 31 Sport field dimensions

| Type of field     | Dimensions field (I x w) | Surface caissons (l x w) | Amount of caissons |
|-------------------|--------------------------|--------------------------|--------------------|
|                   |                          |                          | One field per 15   |
| One soccer field  | 10 x 46 [m]              | 9 x 43.65 [m]            | caisson            |
|                   |                          |                          | 4 fields per 3     |
| Four tennis field | 24 x 11 [m]              | 9 x 43.65 [m]            | caissons           |



The figure below shows an example of the configuration of the caissons to achieve four tennis fields.



Figure 63 Plan view of tennis courts

As can see, is the division of the fields a challenge but optimization can divide the fields efficiently eventually by using the free space for accommodation. Moreover, a critical point is to place and hold the caissons horizontal during use, especially when floating.



## 9. Car parking

Car parking is in most large cities an issue due to lack of space and higher demand for parking spots. Large rivers, canals and lakes provide suitable space for floating car parking, on condition that there is demand for car parking but not the space for it. Floating parking spots can also be used for a redevelopment plan to create more space in the streets and enhance low-traffic areas. At events such as sporting events and concerts, where parking spaces are required, the floating caissons could provide temporary parking spots to the visitors. The cars can single park on the top-deck of the caisson. When only the latter case is considered, it will for economic reasons be advantageous to combine it with another function.

To determine the feasibility and capacity when the caisson are used as parking places. The space requirements for car parking are not totally fixed, but there are guidelines recommending necessities to meet for safe and comfort parking. For the design of the parking spaces on top of the caissons, the recommendations of NEN 2443 will be applied for perpendicular parking. Table 33 gives the dimensions which are divided in Figure 62. On each caisson, including parking road, fits 36 parking lanes.

**Table 32 Parking dimensions** 

| Length of single<br>parking lane | Length of double<br>parking lane | Width parking<br>road | Width parking lane |
|----------------------------------|----------------------------------|-----------------------|--------------------|
| (P1)                             | (P2)                             | (W1)                  | (W2)               |
| 5.00 m                           | 10.00 m                          | 6.35 m                | 2.30 m             |



Figure 64 Plan view of parking places on two caissons



#### **10.** Demolition

Demolition of the caissons is considered as null alternative in this list of functions. This null alternative means that the caissons, which are now functioning as quay walls, are demolished and removed. The concrete can be crushed and used for example as road foundations. The reinforcement could be melted and used in steel products.

A more obvious null alternative would be leaving the caissons as quay walls, but since this is not the main objective of this thesis, this alternative is not taken into consideration.

Broadly, there are two demolition methods that can be applied in the case of the caissons quay walls. Each method has his strengths and weaknesses which will be clarified in the next section after a short explanation of the two methods. The excavating, uplift and floating of the caissons is described in Chapter 6.

#### Method 1

The first method focuses on demolition of the caissons in an existing or new dock. The caissons will be

excavated and floated to a dry dock. Docks are suitable for constructing caissons, but also highly appropriate for the demolition of large concrete structures. Advantages of this method lies in the controlled conditions in the dock, independency of water and partly weather conditions and no interference with activities in and around the

harbour. This minimizes the disturbance of shipping Figure 65 Dry dock (Pacific Marine) in the two harbours. Downside of using a dock for



demolition are the costs for the dock and the availability of a suitable dock.

### Method 2

An alternative option, is demolition of the concrete caissons in the harbour. After excavation and controlled uplifting, each part that protrudes above the water is removed by sawing, leaving a small freeboard to prevent overflow of water. In combination with the weight decrease and reducing ballast the caisson will appear step-by-step above the waterline. Figure 64 shows in a schematic way the process.





#### **Figure 66 Demolition process**

This alternative provides demolition in the harbour itself or at another location after transportation without the need of a dry dock or even transportation (only for the debris). This work can also be conducted along a quay wall, so that not all work need to be done on the water. Compared with a dry dock, water and weather conditions are crucial during the demolition process. Heavy rain fall, (ship) waves and swell lead to unsafe work conditions. Therefore work activities only can start with good weather expectations.

# 7.2 Demolition costs

Costs play in a decisive role in most decision-making processes. This is also valid when considering the null alternative, which is demolishing the caisson quay walls. The demolition costs can outweigh the (financial) benefits of reuse of the same caissons and are therefore an essential aspect to consider in the decision-making. This section will estimate, on the previous clarification, the costs that are involved for demolition (i.e. as described in the previous paragraph) of the caissons at pier 2 of the Waalhaven.

The first part of this section consists of a specification and description of the relevant costs, followed by estimating amounts and, finally, costs in the second part. In Appendix H an overview of all costs is given.

## 7.2.1 Which costs?

First the components that will form the total costs will be specified. For estimating the costs, the caissons will be removed globally in the same manner as described in paragraph 5.3. Assuming that the caissons will be demolished in an existing dock, no building costs of a dock will be calculated. The total costs are composed of the following components;

- Preliminary work
- Excavation costs
- Demolition costs
- Uplifting costs
- Transport costs



- Engineering costs
- Other costs

# 7.2.2 Estimation of amounts and costs

Before the costs can be calculated, amounts of work, based on early provided information, will be estimated. The amount of work will be divided into previously mentioned components. Afterwards the costs, based on information supplied by Stadsontwikkeling Rotterdam and Archidat Bouwkosten, will be assigned to each work activity. Addition to the defined costs, a percentage of *still to be specified* costs will be part of the total direct costs. These are mostly costs that come besides with the costs of the main works that should still be defined. This cost estimation is mainly based on the SSK-systematics (SSK-2010).

The total costs of a project consists of different cost items which roughly can be divided in three parts; namely direct costs, indirect costs and other costs. The direct costs are derived from, among others, the labour costs and material costs. The indirect costs include non-recurrent costs and execution costs. In the next scheme the build-up of the direct and indirect costs are shown.

For indirect costs an estimation, expressed in percentage, is given for each component. This percentage is derived from indicators supplied by Stadsontwikkeling Rotterdam and Archidat Bouwkosten. The direct costs are expressed in united prices, which includes the total direct costs (mentioned as 'A' in Table 34).



## Table 33 Specification of costs (CROW)

| Specification                    | Costs |
|----------------------------------|-------|
| Labour costs                     |       |
| Material costs                   |       |
| Equipment costs                  |       |
| Subcontractors                   |       |
| Inspection costs                 | +     |
| Total direct costs (A)           |       |
|                                  |       |
| Non-recurrent costs              |       |
| Execution costs, 7% of A         |       |
| General operation costs, 8% of A |       |
| Profit and risk, 4% of A         | +     |
| Total indirect costs (B)         |       |
|                                  |       |
| Total estimated costs (A+B)      |       |

# 7.2.3 Amounts & costs

The quay wall at the pier 2 of the Waalhaven consists of 17 caissons with a total length of 742 meter. The first elements that need to be removed are (nautical) facilities (fenders, dolphins). Assuming those facilities are along the entire quay wall, the facilities should be removed over a length of 742 meter.

**Table 34 Costs remove facilities** 

|                                 | Amount | Unit price | Total costs |
|---------------------------------|--------|------------|-------------|
| Remove (nautical)<br>facilities | 742 m  | € 100 / m  | € 74.200,-  |

After the facilities on and around the quay walls are removed, the excavation activities are the following cost item. The excavation consist of three parts;

- Excavation of the top layer;
- Excavation of the soil behind the caissons;
- Removing sand from compartments.



In the next table the total estimated amount of soil that needs to be excavated is shown. The unit price is including transport of the excavated soil and labour costs.

Table 35 Costs excavation

|                                    | Amount     | Unit price  | Subtotal costs |
|------------------------------------|------------|-------------|----------------|
| Excavation of the top<br>layer     | 22.300 m³  | € 4 ,- / m³ | € 88.200       |
| Excavation behind the caissons     | 55.700 m³  | € 4,- / m³  | € 228.000      |
| Removing sand from<br>compartments | 67.000 m³  | € 10 / m³   | € 670.000      |
| Subtotal                           | 145.000 m³ |             | € 987.200      |

After removing sand, the caisson must be uplifted by outflow of water from the compartments. This can be done by using water pumps to pump water in and out the compartments. The steel hatches are highly likely rusted, and so water pumps are needed for controlling ballast. Assuming dry sand in the compartments and requiring controlled uplift, the compartments should be filled with at least 7 meter of water ballast to let the caisson on its place. For safety the required amount of ballast will be set on 8 meter, equivalent to 2600 m<sup>3</sup> water. After that, the ballast should be pumped out till the required ballast for uplifting and static stability. Because the caissons are rest on a layer of coarse gravel

Table 36 Uplifting costs

|                                     | Amount  | Unit price                     | Subtotal costs           |
|-------------------------------------|---------|--------------------------------|--------------------------|
| Water pumping into the compartments | 2600 m³ | €685 / week, excluding<br>fuel | €200 / day / per caisson |
| Water pumping out the compartments  | 750 m³  | €685 / week, excluding<br>fuel | Included in above costs  |



Demolition costs are the fourth component that make part of the total costs. Beside the demolition of the caissons, also account should be taken for the demolition of the (concrete) substructures and, in particular, the joints between the caissons.

**Table 37 Costs demolition** 

|                               | Amount               | Unit price | Subtotal costs |
|-------------------------------|----------------------|------------|----------------|
| Demolition substructures      | 4.650 m <sup>3</sup> | € 35 / m³  | € 162.750      |
| Demolition caissons           | 17.370 m³            | € 70 / m³  | € 1.215.900    |
| Demolition/removing<br>joints | 208 m³               | € 25 / m³  | € 5.200        |
| Subtotal                      |                      |            | €1.458.050     |

In case the caissons will be demolished in an existing dock, the caissons need to be transported to the dock. When it is supposed that the caissons will be transported through waterways to reach the dock, transport costs take a part of the costs. It is assumed that each caisson needs 3 tugboats to be navigated them through the waterways.

**Table 38 Transport costs** 

|                               | Amount | Unit price | Subtotal costs |
|-------------------------------|--------|------------|----------------|
| Tugboats (3)                  | 90     | € 1000     | € 90.000       |
| Employers (3)                 | 90     | € 320      | € 28.800       |
| Coordination & traffic safety | 1      | €9.000     | € 9.000        |
| Subtotal                      |        |            | € 127.800      |

In Figure 65 the distribution of the three above costs is depicted in a pie chart. The major part of the direct costs is the demolition of the substructures and caissons, including removal of (nautical) facilities, followed by excavation of the soil in and around the caissons. Just a small percentage of these costs are composed of transport costs, to be exactly 1%. This means demolition of the caissons in an existing dock or on the spot is, in financial terms, not a sensitive issue.




Figure 67 Distribution of direct costs

Figure 68 Distribution of indirect costs

The indirect costs consist of the site costs, site foreman and other costs. The indirect costs are derived by calculating 6.9% of the direct costs. The remaining costs are other costs such as general costs and insurance. An overview of the other costs are given in Table 40. Figure 68 depicts the indirect costs included with other costs.

**Table 39 Structure of Other costs** 

|                         | Estimated percentage | Class according Figure 67 |
|-------------------------|----------------------|---------------------------|
| General costs           | 4 %                  | 1                         |
| Profit & risk           | 4 %                  | 1                         |
| Contributions (RAW FCO) | 0.15 %               | 4                         |
| Insurance CAR           | 0.75 %               | 3                         |

The above costs are determined by taking a percentage of the costs defined in Figure 66. The number in the righter column of Table 40 corresponds to one of the costs of Figure 67.





Figure 67 Cost structure

### 7.2.4 Total costs of uplift and demolition

Cost estimations may contain uncertainties that may affect the overall outcome. The total costs are all costs that are directly or indirectly related to the work activities for demolition of the caissons. Table 41 shows the total demolitions costs for the entire caisson quay walls, with a length of 742 meters, including the most probable upper and lower limit of the total costs.

Table 40 Dispersion of total costs

|             | Lower limit (25%) | Mean    | Upper limit (25%) |
|-------------|-------------------|---------|-------------------|
| Total costs | € 3.1 M           | € 4.2 M | € 5.2 M           |



# 7.3 Limitations & possibilities

This part will focus on the general limitations that the caissons could have for some functions. On the other hand, measurements can be taken to increase the possibilities of reusing the caissons. Therefore, the limitations and possibilities will be discussed in this section.

When the caissons need to be transported to new locations, dimensions are an issue of concern. Draught, as well as width and height, could be a limitation for caissons to reach a desired location. Constraints, during transportation, may be caused by (temporarily) insufficient depth of waterways – depending on discharge, tides, etc. – bridges, tunnels or small waterways.

The second limitation that arises, due to the age and quality of the concrete, is the porosity and permeability of the concrete. This property most probably restricts the use of liquids and gasses in the caisson. Liquids, and also gasses, can penetrate through the concrete and cracks and may leak out. Moreover, the concrete and reinforcement bars degrades when coming in contact with (aggressive) liquids and gasses. Hence, additional measurements and adaptations are essential when dealing, in the future, with content like liquids and gasses in those caissons.

Beside the porosity, brittleness of the concrete is increased after years causing less deformation capacity. Hoisting the caissons is therefore too risky if only small deformations are allowed. This applies also to forces that require large deformations.

In order to extend possibilities for reusing the caissons dimensions can be changed, specifically the height. By sawing the top of the caisson to a certain extent, the caisson might be an interesting option where it was not in first instance by the height. The limitation for height could, in combination with a suitable function, be nullified. If the function is promising and requires reduction of caisson height, either for transportation purposes or for the function itself, the limitation of dimensions are partially cancelled out. Table 42 gives the results of the draughts after sawing. The calculation procedures are found in Appendix D and section 5.2.



| Total height | of caisson | Drau | ught | Ва  | llast | <i>h<sub>m</sub></i> (≥ 0.5 m) |
|--------------|------------|------|------|-----|-------|--------------------------------|
| 12.50        | [m]        | 8.73 | [m]  | 2.3 | [m]   | 0.53 [m]                       |
| 11.50        | [m]        | 6.83 | [m]  | 0.5 | [m]   | 0.56 [m]                       |
| 10.50        | [m]        | 6.05 | [m]  | 0   | [m]   | 0.81 [m]                       |
| 9.50         | [m]        | 5.70 | [m]  | 0   | [m]   | 1.12 [m]                       |
| 8.50         | [m]        | 5.34 | [m]  | 0   | [m]   | 1.41 [m]                       |
| 7.50         | [m]        | 4.99 | [m]  | 0   | [m]   | 1.72 [m]                       |
| 6.50         | [m]        | 4.63 | [m]  | 0   | [m]   | 2.02 [m]                       |
| 5.50         | [m]        | 4.28 | [m]  | 0   | [m]   | 2.31 [m]                       |

#### Table 41 Draught per reduced meter height

An alternative option, when draught cannot be decreased by weight, is to enhance the buoyancy by using drift bodies. Drift bodies can me imagined as large bags filled by air and are fixed to the caisson. Figure 68 shows an example of drift bodies on both sides.



Figure 68 Drift bodies for additional buoyancy (Eversafe Marine)

Reducing the height of the caissons will lead to decrease of the weight and the draught of the caissons and therefore transportation through sallow waterways is possible. However when static stability is taken into account, draught will scarcely decrease by adding additional ballast to secure the static stability of the caisson. To reduce the draught when water depths are not sufficient, additional measures can provide solutions.



One of the methods to do so, is by increasing the moment of inertia by linking the caissons to each other (Figure 71). The connection should be rigid enough to interact with each other, but should have at the same time some flexibility to avoid unacceptable forces in the connections between the caissons (by avoiding very stiff connections between the bar and caisson). The connections will endure large moments, therefore a truss is needed to let the two caissons form one entirely structure. The bar could exists of a steel tube which is connected to the caisson with a steel plate and bolts. Caution is advised due to the relative thin walls of the caissons. In this case the caissons are stable enough during transportation, so there is no need for the mentioned measure. However, this method can be used when the functions requires such a construction (e.g. floating quay walls).



Figure 69 Linked caissons

After transport to a location, the second adaptation for expanding the options is to construct a deck at the top of the caisson. A deck, customized to the function, will enhance the possibilities. The top deck could serve even a second function besides the chosen function. Furthermore, caissons can placed next to each other to increase surface when a larger top deck is needed.

Table 42 Water depths of rivers (Rijkswaterstaat)

| River           | Water depth relative to NAP (average) |
|-----------------|---------------------------------------|
| Lek             | -5.30 [m]                             |
| Beneden Merwede | -5.90 [m]                             |
| Noord           | Between -10.00 and -4.80 [m]          |
| Oude Maas       | -9.45 [m]                             |
| Waal            | -3.00 [m]                             |
| Boven-Rijn      | -2.80 [m]                             |
| Hollands Diep   | -6.80 [m]                             |



The reach of the caissons when transported on water, for the large rivers in the Netherlands, is listed in Table 43. The caissons can be transported to the North Sea. Here, the draught, width and height of the caissons, as mentioned earlier, affect negatively the possibilities. The minimum water depth along the waterways should be at least 11.60 meter including keel clearance to safely, with respect to draught, transport the caisson. As for the width a clearance of 1.50 meter at each side must be maintained for accurately manoeuvring through rivers and between obstacles, such as river banks, bridges, and groynes.

| Table 43 | Dimensions | including | required | clearance |
|----------|------------|-----------|----------|-----------|
|          |            |           |          |           |

|                                  | Dimension             | Clearance |
|----------------------------------|-----------------------|-----------|
| Draught                          | 10.60 [m]             | 1.00 [m]  |
| Width (lower part)<br>Upper part | 12.20 [m]<br>9.00 [m] | 1.50 [m]  |
| Free board:                      |                       |           |
| Front wall                       | 2.80 [m]              | 1.00 [m]  |
| Back wall                        | 1.90 [m]              | 1.00 [m]  |

Height is also of interest, but to a lesser extent because of the relatively small free board that is required (see Table 44). Therefore major problems hardly arise with respect to the height of the caisson above the waterline (free board).

#### **Concrete deck**

Most caissons used as quay walls in the Port of Rotterdam are box caissons, which are closed at the bottom but open at the top. To enhance possibilities, some functions requires a closed top which can be realized by constructing a top deck on it. Prefab floors or a combination with in situ, are the most favourable choices in this situation by the repetitive work and difficult formwork to construct in the caissons. Prefab floors are available in different forms, construction, lengths and permitted load.





Figure 70 Placing a prefab floor (Henk Wind)

Figures 71 and 72 show two frequently used prefab flooring systems, but almost any floor system can be constructed on demand. For easy placement and assembly of the prefabricated floors, the floor elements should fit on the compartments cells. In Figure 73 the cell dimensions are illustrated.



Figure 71 Core floor slab (Betonson)



Figure 72 Lattice girder floor (Betson)

# **″u**Delft

Using prefabricated elements is less labour-intensive than in situ concrete floors. Pouring of concrete on the spot is requires formwork and labour-intensive work as reinforcement work. Lattice





Figure 73 Top view cell dimensions of compartments

Figure 74 Lattice girder floor connected to the existing caisson wall girder floors are lifted and placed on the right positions and then poured over it a layer of concrete. The prefabricated part functions also as formwork. The prefab elements are connected to each other by filling up the cut out by concrete. The advantages of prefabricated concrete elements compared with in situ concrete, are the rapid placing and assembly of the elements, reduced labour-intensive and the repetitive elements. Especially reducing labour on the site for safety reasons and almost impossible placing of formwork are the most important aspects to choose for prefabricated elements.

The elements could be shipped and directly placed, having the large capacity of ships and easy accessibility over water in favour.



# 8. Decision & design

One of the described functions in chapter 8 will be verified according current guidelines. Before doing this, a multi-criteria decision analysis (MCDA) is introduced, including defined criteria. On the basis of this multi-criteria analysis, the most preferred option, chosen from 7.1.1., will be elaborated and verified.

### 8.1.1 Criteria of decision

The criteria, which are part of the multi-criteria decision analysis, are explained in this section. Criteria are important and significant aspects about environmental impact, costs and revenues, construction time, etcetera. The decision of one of the functions is coherent with the impact of each function on one of the defined criteria: structural condition, economic, adaptations, environment impact and flexibility.

#### **Structural condition**

Over time, structures could have deficiencies created by interference with the environment. This deficiencies weaken the structure in several ways, limiting some functions. For the case of the caissons, Appendix F describes the condition of the structure and critical points.

Safety is also part of the structural condition. Some functions are been exposed to fire and explosion hazards, directly by its function or by the environment. To serve a function of this type, the structure should meet the requirements concerning fire safety.

#### **Adaptations**

When the caisson quay walls required to be adjusted in means of additional constructive elements, reduction of height or adding non constructive elements, this should be minimised in order to reduce caissons in the original condition costs and reuse the as much as possible. The more adjustments are required to a caisson, the lower the rating will be for this criterion.

#### Disruption

Disruption to the environment are one of the major annoyances of surrounding residents. Disruption could be noise disturbances, vibrations, diversions of roads and visual pollution. This is often an important criteria in urban areas, particularly residential areas and high populated areas. Also in areas with historic buildings, one should take the impact of vibrations into account.

#### Environment

Any construction project needs to minimize polluting the environment with respect to air, ground and water pollution. The new function should be designed in such a way that it has a low environmental impact, especially in long-term. Another aspect that contributes to a positive impact on the



environmental is the function itself. When the functions of the caissons contribute to the environment in a positive way, this will also be in the multi-criteria decision analysis.

#### Flexibility

The caissons can serve a function for a long period or a shorter one, depending on the situation. Flexibility is a criterion that measure the easiness to switch from function to another function, both at a new location or the same location.

#### **Traditional solutions**

The caissons can fulfil a function instead of designing and constructing a new structure. To what extent are the caissons a good replacement of traditional structures with regard to construction time, completeness of the function and disruption?

#### Economic

This criterion exists of two aspects: the construction costs and potential revenues. General costs play most of the time an important role in decisions between alternatives. But some solutions can also earn revenues. And therefore, if the construction costs are high, it can be an economic interesting option.

#### 8.1.2 Multi-criteria decision analysis

This section focuses on the decision for the most promising function of the caissons. The criteria for this decision are outlined and described in the previous section. Each criterion will have a certain weighting, depending on the priorities given to the criteria. Table 45 shows the multi-criteria decision analysis, including the weight of each criterion and the given ratings.

The ratings are based on a 5-point scale (5 = very positive, 4 = positive, 3 =neutral, 2 = negative, 1 = very negative). For the weighting, 10 points are divided and assigned to each criterion appropriate to the priority of the criterion. The score of the criterion is multiplied by this assigned value. The much higher the weight value, the higher the priority.

In most decisions the economic criterion is the most influential factor and outcomes are therefore largely dependent on economic reasons. For that reason the first multi-criteria decision analysis (MCDA) contains all mentioned criteria, excluding the economic criterion. The second MCDA assesses all the described functions on the economic criterion, resulting in an outcome without the economic impact and an outcome only based on the economic criterion. The third table includes all criteria.



#### Table 44 Multi-criteria decision analysis

|                      | Weighting | Fruit storage | LNG storage | Building<br>foundation | Breakwater | Floating road | Floating<br>swimming pool | Floating quay<br>wall | Floating<br>stadium | Car parking |
|----------------------|-----------|---------------|-------------|------------------------|------------|---------------|---------------------------|-----------------------|---------------------|-------------|
| Structural condition | 3         | 4             | 4           | 5                      | 2          | 3             | 4                         | 3                     | 5                   | 4           |
| Adaptations          | 2         | 2             | 1           | 5                      | 5          | 3             | 3                         | 4                     | 3                   | 4           |
| Disruption           | 1.5       | 4             | 3           | 4                      | 3          | 3             | 3                         | 3                     | 2                   | 3           |
| Environment          | 1         | 3             | 5           | 4                      | 3          | 2             | 3                         | 3                     | 3                   | 2           |
| Flexibility          | 1         | 3             | 2           | 2                      | 5          | 3             | 3                         | 5                     | 4                   | 5           |
| Traditional designs  | 1.5       | 2             | 2           | 4                      | 4          | 4             | 5                         | 5                     | 4                   | 4           |
| Total                | 10        | 31            | 28.5        | 43                     | 34.5       | 30.5          | 36                        | 35                    | 37                  | 35.5        |

#### Table 45 MCDA for the economic criteria

|          | Weighting | Fruit storage | LNG storage | Building<br>foundation | Breakwater | Floating<br>road | Floating<br>swimming<br>pool | Floating<br>quay wall | <b>Floating</b><br>stadium | Car parking |
|----------|-----------|---------------|-------------|------------------------|------------|------------------|------------------------------|-----------------------|----------------------------|-------------|
| Economic | -         | 3             | 1           | 5                      | 5          | 3                | 4                            | 5                     | 4                          | 5           |



#### Table 46 Complete MCDA

|                        | Weighting | Fruit storage | LNG storage | Building<br>foundation | Breakwater | Floating road | Floating<br>swimming pool | Floating quay<br>wall | Floating<br>stadium | Car parking |
|------------------------|-----------|---------------|-------------|------------------------|------------|---------------|---------------------------|-----------------------|---------------------|-------------|
| Structural condition   | 3         | 4             | 4           | 5                      | 2          | 3             | 4                         | 3                     | 5                   | 4           |
| Adaptations            | 2         | 2             | 1           | 5                      | 5          | 3             | 3                         | 4                     | 3                   | 4           |
| Disruption             | 1.5       | 4             | 3           | 4                      | 3          | 3             | 3                         | 3                     | 2                   | 3           |
| Environment            | 1         | 3             | 5           | 4                      | 3          | 2             | 3                         | 3                     | 3                   | 2           |
| Flexibility            | 1         | 3             | 2           | 2                      | 5          | 3             | 3                         | 5                     | 4                   | 5           |
| Traditional<br>designs | 1.5       | 2             | 2           | 4                      | 4          | 4             | 5                         | 5                     | 4                   | 4           |
| Economic               | 2.5       | 3             | 1           | 5                      | 5          | 3             | 4                         | 5                     | 4                   | 5           |
| Total                  | 12.5      | 38.5          | 31.5        | 55.5                   | 47         | 38            | 46                        | 42.5                  | 48                  | 48          |

In Table 47 the economic criterion is included with a weight factor of 2.5. It appears that also, when the economic criterion is included in the MCDA, the foundation for buildings is the best option according those criteria. From the first and third matrix-criteria decision analysis (see Table 45 and Table 47) it appears the caissons are best to use as foundation for buildings on basis of the defined criteria. The buildings could be offices, residential buildings, commercial areas, or a combination of those.



# 8.2 Design caisson as building foundation

The caisson is an existing structure built in the late '50s of the last century. During decades, harbour activities, loads and especially concrete properties and guidelines have changed. To reuse those caissons for the above chosen function, caissons as building foundation, the structure will be verified according to the current guidelines.

### 8.2.1 Calculations building foundation

For the compressive strength different values are defined. Two values are determined along theoretical papers and one from a core drill test of one of the caissons. Those different compressive strengths will be compared to each other by unity checks. Before the calculations, first all conditions and parameters are determined.

The aim is not to design to above buildings, but to assess how many floors can be built with the caissons as foundation, therefore a calculation is made of a building with one floor and a ground floor (see Appendix J).

#### Assumptions

The caissons are connected to each other by pouring concrete mortar in the lock spaces (see Figure 38). Because those locks are weakest point, due to low quality concrete and no reinforcement is applied, and also the impact of chlorides. Therefore, for the calculations, the connections between the caissons will not be taken into account and assuming the concrete mortar has cracked.

#### **Partial factors**

Partial (safety) factors are determined, both for the materials as the loads, according Eurocode 1 & 2. The table below gives an overview of the most relevant safety factors.

#### Table 47 Partial safety factors for loads

| Partial safety factors | Values |
|------------------------|--------|
| Favourable loads       | 1.0    |
| Permanent loads        | 1.2    |
| Live loads             | 1.5    |

#### **Material specification**

The material specifications of concrete and reinforcement would be based on the former calculation report, destructive tests, literature research and assumptions. Table 48 shows the concrete specifications as taken into account by the engineers during the design. Table 49 shows the assumed



concrete properties of the caissons and Table 50 the specifications of the concrete class (C50/60) with a similar compressive strength in the current Eurocode 2.

Table 48 Concrete specifications in 1958

|  | Values     |
|--|------------|
| Concrete class   | K225       |
| Concrete density ( $ ho$ )                                       | 25 kg/m³   |
| Permissible concrete compressive stress                          | 6.9 N/mm²  |
| Characteristic concrete cylinder compressive strength $(f_{ck})$ | 15.3 N/mm² |
| Design stress concrete $(f_{cd})$                                | 10 N/mm²   |

#### Table 49 Concrete specifications concrete caisson (K225)

| Concrete properties                            | Values                 |
|--|------------------------|
| f <sub>ck</sub>                                | 58.7 N/mm <sup>2</sup> |
| $f_{\mathit{ctk};0.05}$ (According Eurocode 2) | 2.9 N/mm²              |
| $f_{\mathit{ctk};0.95}$ (According Eurocode 2) | 5.3 N/mm²              |
| ρ  | 25 kN/m³               |

#### Table 50 Concrete specifications C50/60 (Eurocode 2)

| Concrete properties          | Values               |
|------------------------------|----------------------|
| f <sub>ck</sub>              | 50 N/mm <sup>2</sup> |
| f <sub>ck;cube</sub>         | 60 N/mm²             |
| $f_{cm}$                     | 58 N/mm²             |
| $f_{ctk;0.05}$               | 2.9 N/mm²            |
| <i>f</i> <sub>ctk;0.95</sub> | 5.3 N/mm²            |
| E <sub>c</sub>               | 37 GPa               |
| ρ                            | 24 kN/m³             |

#### Loads

For the building, a calculation is made in Appendix J. For the building, load combination 2 is governing. Besides this load, there are other external forces present. One of the external loads is the fluctuating water level in the harbour. The governing situation occurs when the Mean Low Water Level (MLWL) is reached in the harbour basin (see Table 51). When this point is reached the load on the front wall is to



the utmost. The maximum difference height between the top of the front wall (+1.36 NAP) and MLWL is 2.06 meter.

| Table 51 Water levels in 1 <sup>st</sup> Eemhaven (Hydrometer | o Informatiebundel 3, 2004) |
|---|-----------------------------|
|---|-----------------------------|

| Water level                  | Relative to NAP |
|------------------------------|-----------------|
| Mean High Water Level (MHWL) | +1.36 m         |
| Mean Water Level (MWL)       | -0.44 m         |
| Mean Low Water Level (MLWL)  | -0.70 m         |

On the other side of the front wall, the wall is loaded by soil over the entire height of the wall. The soil will be assumed as wet dense sand with parameters mentioned in Table 52. The caisson is founded on a one meter high gravel bed.

Table 52 Soil parameters

|            | $\gamma_{dry}$ | Ysat    | $q_c$ | $oldsymbol{arphi}'$ | <i>c</i> ′ |
|------------|----------------|---------|-------|---------------------|------------|
|            | [kN/m³]        | [kN/m³] | [Mpa] | [°]                 | [kPa]      |
| Dense sand | 18             | 20      | 25    | 35                  | -          |
| Gravel     | 19             | 21      | 30    | 39                  | -          |

 $\gamma_{dry}$  = weight dry sand [kN/m<sup>3</sup>]

 $\gamma_{sat} = weight \ saturated \ sand \ [kN/m^3]$ 

 $q_c = sounding value [Mpa]$ 

 $\varphi' = internal\ friction\ angle\ [^{\circ}]$ 

c' = cohesion [kPa]

#### Calculations

The hand calculations are showing the feasibility of one of the possibilities that is chosen by the MCDA. This section will be divided in two parts; the calculations of the stability of the caisson and concrete bearing capacity. First the stability of the caisson will be checked for governing situations. Secondly, the concrete structure will be checked for the function as building foundation.



#### Stability

During transformation of the area from a harbour to a living/residential area a different load situation occurs. This means attention should be given to failure mechanisms that may occur. The following failure mechanisms will be considered:

- Shear criterion
- Rotational stability
- Vertical bearing capacity
- Scour

The failure mechanisms will be considered during the transformation –and the final phase. The situation for the stability of the caisson during the transformation phase, including the forces, is schematized in Figure 75. This situation can occur during transformation of the harbour to living area. During this period practically no surcharge is present which makes stability, for some failure mechanisms, more critical.

First the loads on the caisson will be determined then the stability calculations will be performed for the shear criterion, rotational stability and vertical bearing capacity.



Figure 75 Forces acting on the caisson in final situation

#### Horizontal loads

In Figure 76 the horizontal forces on the caisson are schematized. The present loads at the right handside of the caisson are the soil –and water pressure. Besides those loads, we assume an additional surcharge load of 10 kN/m. This load includes paving and light traffic next to the buildings.





Figure 76 Horizontal forces acting on the caisson

Table 53 Parameters for horizontal loads

| Parameter                                | Unit     |
|--|----------|
| Saturated sand density                   | 20 kN/m³ |
| Internal friction angle                  | 35°      |
| Active Earth Pressure Coefficient $K_a$  | 0.33     |
| Passive Earth Pressure Coefficient $K_p$ | 3        |
| Neutral Earth Pressure Coefficient Ka    | 0.50     |

The lateral earth pressure per unit length:

$$P_{s} = \frac{1}{2} \cdot K_{a} \cdot \gamma_{s} \cdot h^{2} = \frac{1}{2} \cdot 0.33 \cdot 20 \cdot 12.35^{2} = 503.3 \ kN/m$$

$$P_{w;1} = \frac{1}{2} \cdot K_{a} \cdot \gamma_{w} \cdot h^{2} = \frac{1}{2} \cdot 0.33 \cdot 10 \cdot 12.35^{2} = 251.7 \ kN/m$$

$$P_{q} = q \cdot K_{0} \cdot h = 10 \cdot 0.50 \cdot 12.35 = 61.8 \ kN/m$$

On the left hand-side of the caisson the load consist only of the hydrostatic pressure of the water from the port. The governing situation occurs when there is a low water level in the harbour (MLWL: -0.70 m NAP), which results in a water height of 9.95 meter.

$$P_{w;2} = \frac{1}{2} \cdot \rho \cdot g \cdot h^2 = \frac{1}{2} \cdot 1000 \cdot 10 \cdot 9.95^2 = 495013 \, N/m = 495 \, kN/m$$

Wind pressure is an external static force that the wind exerts on the building. This wind load affects mainly the rotational stability of the caisson and the connection between the columns of the building and the caisson walls.

The wind load is determined according Table 68 (Appendix J). Because the building will be built in a city, we will assume a cultivated area. The distance between the columns is 6 meter. This results in a wind pressure of 0.64 kN/m<sup>2</sup>, including safety factors the design wind load is:



 $q_{wind} = 1.5 \cdot 6 \cdot 0.64 = 5.76 \ kN/m.$ 

With a simple model (see Figure 77) the maximum moment at the foot of the building can be calculated as follow:



Figure 77 Model for wind load

#### Vertical loads

Surcharge on the caisson are mostly vertical loads, but also the self-weight of the structure and sand in the compartments can be counted as vertical load. The vertical load by the self-weight of the caisson is assumed to be equally disturbed over the bed. The forces of the building are transferred through the outer walls to the bottom of the caisson and underlying soil bed.

The following vertical loads are present: The building ( $F_{building}$ ), self-weight caisson ( $q_1$ ), ballast (sand) in compartments ( $q_2$ ) and surcharge next to the building ( $q_3$ ).

 $F_{building} = 1198 \ kN$  (See calculation in Appendix J).

$$q_1 = \frac{V \cdot \rho_{concrete}}{l} = \frac{25 \cdot 1062.3}{43.65} = 608 \ kN/m$$

 $q_2 = h \cdot \rho_{sand} = 12 \cdot 20 = 240 \ kN/m$  (Unfavourable situation)  $q_3 = 10 \ kN/m$ 

#### Shear criterion

The caisson is from both sides exposed to horizontal forces that are introduced by external forces like water and sand. Those forces are transferred to the subsoil and resisted by the friction force of the



subsoil. The friction between the structure and subsoil is expressed in friction coefficient (f). The shear criterion is like as follow:

$$\sum V > \frac{\sum H}{f}$$

Where:

 $\sum V = sum of total vertical forces$  $\sum H = sum of total horizontal forces$ f = friction coefficient



Figure 78 Principle of sliding off

The largest loads are on the right-hand side of the caisson, i.e. the landside. The governing situation occurs when the horizontal loads from the hand-right side are at a maximum and the horizontal forces from the left-hand side (the waterside) are at a minimum. The minimum load is when the water level in the harbour reach the Mean Low Water Level (MLWL) with a total water height of 9.95 m.

The two materials between which friction occurs depends on the two materials specifications and is expressed in the friction factor. For concrete on gravel the coefficient is 0.55.

| Table 54 Parameters shear criterion |             |  |
|-------------------------------------|-------------|--|
|                                     | Values      |  |
| MLWL                                | -0.70 m NAP |  |
| Water height harbour                | 9.95 m      |  |
| Friction coefficient                | 0.55        |  |



The situation without surcharge and a maximum of horizontal forces is governing for the shear criterion. Therefore, the load of the building is not taken into account.

 $F_{v;c} = vertical force due to selfweight$  $F_{v;b} = vertical force due to ballast in compartments$  $F_{v;w;3} = hydrostatic pressure underneath the caisson$ 

$$F_{v;c} = V_c \cdot \rho_{concrete} = 1062.3 \cdot 25 = 26\ 558\ kN$$

$$F_{v;b} = V_{emty} \cdot \rho_{sand} = 4000 \cdot 20 = 80\ 000\ kN$$

$$F_{v;w;1} = \frac{1}{2} \cdot \rho_w \cdot g \cdot h \cdot A = \frac{1}{2} \cdot 1000 \cdot 10 \cdot 9.95 \cdot (9.95 \cdot 43.65) = 21\ 607\ 296\ N = 21\ 607\ kN$$

$$F_v = 26\ 558 + 80\ 000 - 21\ 607 = 84\ 951\ kN$$

The horizontal forces from the hand-right side:

$$\begin{split} F_{h;1} &= Eart \ Pressure \ due \ to \ soil \ (F_s) + Pore \ Pressure \ (F_{w;3}) + Surcharge \ (F_q) \\ F_s &= \frac{1}{2} \cdot K_a \cdot \gamma_s \cdot h^2 \cdot b = \frac{1}{2} \cdot 0.33 \cdot 20 \cdot 12.35^2 \cdot 43.65 = 21\,969 \ kN \\ F_{w;3} &= \frac{1}{2} \cdot K_a \cdot \gamma_w \cdot h^2 \cdot b = \frac{1}{2} \cdot 0.33 \cdot 10 \cdot 12.35^2 \cdot 43.65 = 10\,987 \ kN \\ F_q &= q \cdot K_0 \cdot h \cdot b = 10 \cdot 0.50 \cdot 12.35 \cdot 43.65 = 2698 \ kN \\ F_{h;1} &= 21\,969 + 10\,987 + 2698 = 35\,654 \ kN \end{split}$$

The only horizontal force from the hand-left side is the hydrostatic water pressure:

$$F_{h;2} = P_{w;2} = \frac{1}{2} \cdot \rho \cdot g \cdot h \cdot A = \frac{1}{2} \cdot 1000 \cdot 10 \cdot 9.95 \cdot (9.95 \cdot 43.65) = 21\,607\,296\,N$$
$$= 21\,607\,kN$$

It can now be verified that:

$$\sum V > \frac{\sum H}{f} \to 84\,951 > \frac{(35\,654 - 21\,607)}{0.55} = 25\,540\,kN$$



#### **Rotational stability**

Instead of sliding, horizontal forces can also rotate a structure by introducing moments about a certain point. The moments are counteracted by the opposing forces which are the vertical forces exposed on the caisson, but also the self-weight of the structure. Rotational stability can be checked by the following rule-of-thumb:

$$e_r = \frac{\sum M}{\sum V} \le \frac{1}{6} b$$

Where:

 $e_R$  = distance from moment centre to intersection point of resulting forces at the bottom line [m]  $\sum V$  = sum of total vertical forces [kN]  $\sum M$  = sum of total moments [kNm] b = width of caisson [m]



Figure 79 Principle of rotation stability (Hydraulic structures - caissons, 2011)

A maximum height difference between the right and left side of the caisson provides the largest moment and so the governing situation for the rotational stability. The forces and rotation point 'r' are defined in Figure 79. The moments are calculated by the forces multiplied by the distance to the rotation point 'r'.





Figure 80 Forces for the rotational stability

$$F_{s;1} + F_{w;2} = 35\ 654\ kN$$

$$F_{w;3} = 21\ 607\ kN$$

$$F_{w;4} = \frac{7}{6} \cdot \frac{(33\ 288 + 21\ 607)}{2} = 32\ 022\ kN$$

$$F_{wind} = \gamma \cdot q_{wind} \cdot l_{building} \cdot h_{building} = 1.5 \cdot 0.64 \cdot 43.65 \cdot 11 = 461\ kN$$

$$a_{1} = \frac{1}{3} \cdot h_{1} = \frac{1}{3} \cdot 12.35 = 4.1 m$$

$$a_{2} = \frac{1}{3} \cdot h_{2} = \frac{1}{3} \cdot 9.95 = 3.3 m$$

$$a_{3} = \frac{1}{6} \cdot b = \frac{1}{6} \cdot 12 = 2 m$$

$$a_{4} = \frac{1}{3} \cdot 11 + 12.35 = 16 m$$

With the above forces and distances, the moments can be determined. Moments to the left are considered as positive moments.

Moment due to lateral soil and<br/>water pressure: $M_1 = 35\ 654 \cdot 4.1 = +\ 146\ 181\ kNm$ Moment due to hydrostatic<br/>pressure: $M_2 = 21\ 607 \cdot 3.3 = -\ 71\ 303\ kNm$ Moment due to hydrostatic<br/>pressure underneath caisson: $M_3 = 32\ 022 \cdot 2 = +\ 64\ 044\ kNm$ 



Moment due to wind load:

 $M_4 = 461 \cdot 16 = +7376 \ kNm$ 

$$\sum M = 146\ 181 + 64\ 044 + 7376 - 71\ 303 = 146\ 298\ kNm$$

For the vertical forces, the load of the building is included because the wind load is taken into account in the moment equilibrium. The building is founded with 14 columns (7 at each side) on each caisson with a force of 1147 kN per column. The total vertical forces are formed by the building, self-weight of the caisson, ballast in the compartments and an additional surcharge (see 'vertical loads'):

 $F_{v;building} = 14 \cdot 1198 = 16\,772\,kN$  $F_{v} = 84\,951\,kN\,(see\,'vertical\,loads')$ 

The total vertical loads are:

 $F_{v;total} = 16\ 772 + 84\ 951 = 101\ 723\ kN$ 

$$e_r = \frac{\sum M}{\sum V} \le \frac{1}{6} b$$

$$e_r = \frac{146\ 298}{101\ 723} = 1.44\ m\ < \frac{1}{6} \cdot 12 = 2\ m$$

The rotational stability of the caisson satisfy the turn-over criterion.

The question arise to which level the building can be constructed when considering this criterion. Raising the building will lead to an increased vertical force, but also a larger moment (a larger distance to the rotation point) due to the wind on the façade. Table 55 and Figure 81 show the results of the calculations made in Excel and based on the above method.

When only considering the rotational stability, the maximum number of floors that can be constructed are 11 floors, excluding the ground floor which corresponds to a 39 meter high building (see Table 55 and Figure 81).



Table 55 Maximum floors according by rotational stability



Figure 81 Maximum floors by rotational stability



#### Vertical bearing capacity subsoil

Structures directly founded on soil should be checked if the vertical effective soil stress does not exceed the maximum bearing capacity of the soil. The maximum effective soil stress can be calculated by the following equation:

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

Where:

 $\sigma_{k;max} = maximum \ acting \ stress \ [N/mm^2]$   $F = vertical \ force \ [kN]$   $A = area \ bottom \ of \ caisson \ [m^2]$   $M = acting \ moment \ [kNm]$   $W = section \ modulus \ [m^3]$   $\sum V = sum \ of \ total \ vertical \ forces \ [kN]$   $\sum M = sum \ of \ total \ moments \ around \ point \ K \ [kNm]$   $b = width \ of \ structure \ element \ [m]$   $l = length \ of \ structure \ element \ [m]$ 

The maximum stress is more critical if the total vertical loads are present, so including the load of the building. The sum of total moments is already determined in the previous criterion. Now, the partial safety factors will be included.

 $F_{v;caisson} = 1.2 \cdot 26\ 558 = 31\ 870\ kN$   $F_{v;ballast} = 80\ 000 = 80\ 000\ kN$   $F_{v;w3} = 1.5 \cdot 21\ 607 = 32\ 411\ kN$   $F_{v;building} = 14 \cdot 1198 = 16\ 772\ kN$   $q_3 = 1.5 \cdot 10 = 15\ kN/m$ 

 $F_{d:total} = 31\,870 + 80\,000 + 16\,772 - 32\,411 = 96\,231\,kN$ 

and



| Moment due to lateral soil and water pressure:         | $M_1 = 1.2 \cdot 146\ 181 = +175\ 417 kNm$ |
|--|--|
| Moment due to hydrostatic pressure:                    | $M_2 = 1.5 \cdot 71303 = -106955kNm$       |
| Moment due to hydrostatic pressure underneath caisson: | $M_3 = 1.5 \cdot 64\ 044 = +96\ 066 kNm$   |
| Moment due to wind load:                               | $M_4 = +7376 \ kNm$                        |

 $\sum M = 175\,417 + 96\,066 + 7376 - 106\,955 = 171\,904\,kNm$ 

$$\sigma_{k;max} = \frac{\sum V}{b \cdot l} + \frac{\sum M}{\frac{1}{6} \cdot l \cdot b^2} + q_3 = \frac{96\,231}{12 \cdot 43.65} + \frac{171\,904}{\frac{1}{6} \cdot 43.65 \cdot 12^2} + 15 = 184 + 164 + 15$$
$$= 363 \, kN/m^2$$

The maximum bearing capacity can be approximated by Prandtl and Birch Hansen:

$$p'_{max} = 0.5 \cdot \gamma' \cdot B \cdot N_{\gamma} \cdot s_{\gamma} \cdot i_{\gamma}$$

The soil underneath the caisson is classified as drained soil. We assume solid gravel and sand with an internal friction angle of 35°. On basis of the internal friction angle, the bearing force factors of the above equation are expressed in Table 56.

**Table 56 Bearing force factors** 

$$\gamma' = \gamma_s - \gamma_w = 20 - 10 = 10 \ kN/m^2$$

The shape factor for the foundation:

$$s_{\gamma} = 1 - 0.3 \cdot \frac{b}{l} = 1 - 0.3 \cdot \frac{12}{43.65} = 0.92$$



$$m = \frac{2 + \frac{b'}{l}}{1 + \frac{b'}{l}} = \frac{2 + \frac{12}{43.65}}{1 + \frac{12}{43.65}} = 1.78$$
$$i_q = \left[1 - \frac{H}{V + A'c' \cdot \cot\theta'}\right]^m = \left[1 - \frac{14.047}{131.145 + 0}\right]^{1.78} = 0.82$$
$$i_\gamma = i_q^{\left(\frac{m+1}{m}\right)} = 0.82^{1.56} = 0.73$$

With the above parameters the equation can be filled in and gives:

 $p'_{max} = 0.5 \cdot \gamma' \cdot b \cdot N_{\gamma} \cdot s_{\gamma} \cdot i_{\gamma} = 0.5 \cdot 10 \cdot 12 \cdot 46 \cdot 0.92 \cdot 0.73 = 1854 \ kN/m^2$ 

The bearing capacity can now be verified and gives:

$$\sigma_{k:max} < p'_{max} \rightarrow 363 < 1854$$

The bearing capacity of the soil is much larger than the loads and this condition satisfied. From the following figure (Figure 82) one can see that the soil, according Prandtl, has a large bearing capacity mainly due to the distribution over the surface of the caisson and therefore the possible maximum floors exceeds the maximum floors with respect to rotational stability.



Figure 82 Maximum floors with respect to soil bearing capacity

#### <u>Scour</u>

One of the failure mechanisms for a caisson is scour. Scour is erosion of soil around a structure caused by the structure itself or may be a consequence of a natural process. This process can also be initiated



by purpose, as it is not always a negative process. However, for structures scour can be a serious threat and hence considered as a failure mechanism.

In this case, the most likely area where scour can occur, is at the toe of the caisson. This problem can be caused by waves but also (local) turbulence affected the propeller of mooring ships. Scour is already a problem that occurred in the past at the toes of some caissons. To improve the soil conditions and prevent further scour, some soil improvements are carried out by grout injections.



#### **Concrete structure**

The concrete caisson structure is used as foundation for buildings. All forces of the building are directed to the caisson and led to the subsoil. This section will deal with the verifying of the concrete wall capacity by a simplified strut-and-tie model. The strut-and-tie model offers a rational method by representing a complex structural element with a simplified truss model.

Also the connection between the columns of the building and the concrete wall will be considered. The columns of the building should be connected to the existing walls of the caissons to transfer the loads from the top-structure to the foundation (i.e. caissons).

#### Strut-and-tie model

Table 57 Parameters for strut-and-tie model

For this model (see Figure 83) we consider a concrete wall of the caisson with a column at the middle and two columns on each side. The two columns on the outside bear half of the load to the adjacent wall, and therefore the half of the vertical forces is taken into account.

| F       | b    | h    | $\sigma_s$   | A <sub>c</sub>                            |
|---------|------|------|--------------|---|
| 1147 kN | 12 m | 12 m | $209 N/mm^2$ | $300 \cdot 12000 = 3600 \cdot 10^3  mm^2$ |



Figure 83 Strut-and-tie model concrete wall



The forces causes stresses in the concrete wall. In an idealized model, the system exist of a node, strut and tie. At 0.5h tension in the concrete wall occurs. The required reinforcement can be determined by the following equation:

$$A_{spl} = \frac{N_{spl}}{\sigma_s} = \frac{F}{4 \cdot \sigma_s} \to \frac{1198 \cdot 10^3}{4 \cdot 209} = 1433 \ mm^2$$

The stress distribution at h is determined by:

$$\sigma_c = \frac{2 \cdot F}{A_c} = \frac{2 \cdot 1198 \cdot 10^3}{3600 \cdot 10^3} = 0.67 \, N/mm^2 < 32.6 \, N/mm^2$$

The present reinforcement at 0.5h is 1237 mm<sup>2</sup>, which is lower than the required reinforcement to counter the local splitting forces. The value of design stress of the used reinforcement (QR24) is much lower as the current reinforcements.

This means that the forces need to be adapted in order to not exceed the allowable stresses. In other words, the maximum load of the building, and therefore the maximum height of the building, is dependent on this criterion.

The maximum allowed force on top of the caisson can now be determined on basis the above equation:

$$F = 4 \cdot A_{spl} \cdot \sigma_s = 4 \cdot 1237 \cdot 209 = 1\,034\,132\,N = 1034\,kN$$

Assume a doubling of the applied reinforcement, then the allowed acting force would be:

$$F = A_{spl} \cdot 4 \cdot \sigma_s = 2 \cdot 1237 \cdot 4 \cdot 209 = 2\ 068\ 264\ N = 2068\ kN$$

There are three options to meet the requirement of the tensile splitting force:

- 1. By optimization of the building loads. Decreasing loads, or self-weight of the building can decrease the total forces on the foundation;
- 2. Instead of column distance of 6 meter, one can choose a distance of 3 meter between the columns. The latter distance is based on the distances between the compartments walls;
- 3. Applying, whether or not with the above options, external reinforcement.



# 9. Conclusions & recommendations

# 9.1 Conclusions

In the early '30 and end of the '50's many caissons have been constructed as quay walls for the Port of Rotterdam. The caissons that are built before the Second World War are all damaged, demolished and/or replaced by new quay walls. At this moment, the most caissons quay walls are located in the Waalhaven and the Merwehaven. All those caissons are built between 1950 and 1961 and are still operational as quay walls in the Port of Rotterdam. Even though the dimensions of the caissons are different, the shape is almost uniform. Various second-life possibilities are discussed for the caissons at pier 2 of the Waalhaven are considered and elaborated.

The caissons, used for the quay wall in Waalhaven, can still function as quay wall and be used for other purposes. In the past structures have been successful allocated a second-life and also caissons have been reused (e.g. as emergency dikes in Zeeland). Instead of building new constructions, it is profitable, time-effective and/or sustainable to use an existing structure even though it was initially designed for another purpose.

During design of the caisson quay walls, a possible removal of the quay wall was not taken into account. This led to the following undesirable situations for the deconstruction of the quay walls: concrete mortar and gravel in the connections between caissons and, most probably, corroded steel hatches. Though, removing of the caissons seems to be possible when measures are taken. Reusing caisson quay walls is possible, both on the location as transportation to another location. When transportation is desirable, one should pay attention to the condition of connection between the caissons and protect the surrounding quay walls on the pier with for instance sheet piles.

There are many risks during the uplifting and transportation of the caissons. The risks with large/moderate impact and large probability are: rusted steel hatches, which leads to no control over the water inside the compartments, and overestimated concrete strength, which lead to damage of structure or structure failure. Therefore, water pumps need to be used in case of rusted steel hatches.

The deconstruction and demolition of the quay wall at pier 2 of the Waalhaven, including the removal of the facilities, were considered. The pier consists of 17 caissons with a total length of 742 meters. The total costs for the deconstruction and demolition are  $\notin$  4.2 million of which 35% consist of excavation costs. When using the caissons as foundation for buildings, the land can have a value of  $\notin$ 3.56 million.



The current value is very low due to industrial use, so that means an upgrading of the land value when transforming that area to residential (and commercial) use.

Essential is to verify the condition of the caissons and the concrete strength. In case of reusing the caissons, it is important to remark that the submerged parts of the caissons are in a critical condition with respect to corrosion. The oxygen-level under water is limited, but when transporting the caisson to a location where the submerged parts are exposed to air, the corrosion process will be speeded up. Only when reparations are carried out, then caissons will have a larger range of new functions to use for. Especially since the compressive strength of the concrete increases in time; core drill tests even show an increase of the compressive strength to an average of 58.79 N/mm<sup>2</sup>.

In this thesis various second life possibilities for the caissons are considered, many of which require (costly) adaptions. Nine possibilities were considered, the demolition of the caisson (null alternative) excluded. According to the multi-criteria decision analysis, the building foundation alternative has by far the highest score and the LNG storage alternative the lowest. When only considering the economic criterion, it gives the same results. This is due to the fact that for the LNG storage the adaption costs are relatively high. Since the building foundation is also constructible, it is feasible to reuse the caissons.

When the caissons are used as foundation for building, the question is how many storeys are possible? For this case, a building of one storey and a ground floor is taken into account. Considering the overall stability of the caisson and stresses in the concrete wall, the latter is become decisive. The tensile forces occurring in the concrete wall, due to concentrated force of the columns, appears to be critical. The low design stress of the reinforcement is the critical factor of the structure of the caissons.

The acting splitting force is larger than the reinforcement capacity in the concrete wall. Therefore, a higher building than can only be possible if the loads will be reduced, for example by using a different building material (with a lower self-weight). Another alternative, which can be combined with the latter option, is using external applied reinforcement to counteract the splitting force on local points. The rotational stability of the caissons allows a 11 floor-building which eventually can be stretched by using anchors to counteract the moment.

The caissons are in a good condition to be used as foundation for buildings and have the advantage that no foundation has to be constructed in a densely populated area near the waterfront. Furthermore, the use of caissons is profitable because the land value can be upgraded when turning those areas to residential and/or commercial areas, using the caissons as foundations.



## 9.2 Recommendations

This section will give, as a result of this study, recommendations for further research to reuse of caissons. The following is recommend to a successive reuse of the caissons.

The available information on the caissons was not always present to investigate all the details of the caisson. Not all information (drawing and reports) was present in the available sources, for example the location of the steel hatches. The location, amount and condition of the steel hatches is important to know when transportation is needed. It is recommended to investigate the condition of the caissons, in particular the steel hatches and the condition of the connections between the caissons when considering transportation.

The strength of some caissons are based on three core drilled tests. With the amount of caissons that are situated in the Waalhaven and Merwehaven, it is recommended to carry out more compressive strength tests. The strength mainly determines the function possibilities, especially when heavy loads are part of the function. Besides the concrete strength, the condition and strength of the reinforcement must be tested and assessed as part of the bearing capacity of the structure.

In addition to the suggested possibilities, one can offer more possibilities to reuse caissons. One of the opportunities could be to use the caissons for multiple functions at once or over time, and therefore enhancing the efficiency of the caissons.

From the investigation report, the concern raised about the condition of the submerged parts of the structure. It is advisable to keep those critical areas submerged when possible or repair those parts when exposed to air in order to retard the corrosion process. Research is needed in the rate of the corrosion process and the consequences of this corrosion process.

For further research of the possibilities, the bearing capacity of the caissons needs to be determined and verified with computational modelling. By using those methods, designs can be optimize the height of the building. Moreover, it is recommendable to elaborate improvements in order to increase the maximum floors of the building. External applied reinforcement to strengthen the structure and applying anchors or other options to meet the rotational criterion with the aim to increase the building height.

Further study on reuse of caissons of the Port of Rotterdam has a high potential for the reason that there are 44 caissons only in the Waalhaven. The Merwehaven has also a large number of caissons which are similar to the caissons in the Waalhaven. Due to the relatively high amounts of caissons in the Port of Rotterdam, especially the Waalhaven and Merwehaven, an additional study is recommended. It



is advisable to start an early and further study to the reuse of the caissons in the Port of Rotterdam before decisions are taken to remove the caissons.



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## **Appendix A**

This appendix shows the different types of caissons in the Waalhaven and Merwehaven and their position at the piers. Most caisson types are used on multiple positions in both the Waalhaven and Merwehaven. Drawings of the following quay walls are belonging to Public Works Rotterdam and Port of Authority Rotterdam.

#### Waalhaven



Table 58 Figures caisson quay wall pier 2

| Amount of caissons | Length caisson | Total length of caisson<br>quay wall | Year of construction |
|--------------------|----------------|--------------------------------------|----------------------|
| 17                 | 43.65 m        | 742 m                                | 1958 - 1960          |





Figure 85 Caisson quay wall (pier 6 & 7)

#### Table 59 Figures caisson quay wall pier 7

| Amount of caissons | Length caisson | Total length of caisson<br>quay wall | Year of construction |
|--------------------|----------------|--------------------------------------|----------------------|
| 17                 | 43.65 m        | 742 m                                | 1956 - 1959          |

#### Table 60 Figures caisson quay wall pier 6

| Amount of caissons | Length caisson | Total length of caisson<br>quay wall | Year of construction |
|--------------------|----------------|--------------------------------------|----------------------|
| 6                  | 43.65 m        | 262 m                                | 1956 - 1959          |





Figure 86 Caisson quay wall (pier 9)

Table 61 Figures caisson quay wall pier 9

| Amount of caissons | Length caisson | Total length of caisson<br>quay wall | Year of construction |
|--------------------|----------------|--------------------------------------|----------------------|
| 4                  | 43.65 m        | 174.70 m                             | 1955                 |



#### Merwehaven







Figure 89 Caisson quay wall (Pier 4)



ł

## Appendix B





# Appendix C

|   | l, / l.  | 1.0  | 1,2 1,4   | 1,6  | 1,8                                   | 2,0                                   | 2,5                                   | 3,0                                   |
|---|--|--|---|--|---------------------------------------|---------------------------------------|---------------------------------------|---------------------------------------|
|   | $m_{rs} = 0.001 p_d l_s^2 \times m_{ry} = 0.001 p_d l_s^2 \times$  | 41<br>41   | 54 67<br>35 31  | 79<br>28   | 87<br>26                              | 97<br>25                              | 110<br>24                             | 117<br>23                             |
|   | $\begin{array}{rcl} m_{**} &=& 0.001 \ p_{0}  l_{*}^{2} & \times \\ m_{*y} &=& 0.001 \ p_{0}  l_{*}^{2} & \times \\ m_{*s} &=& -0.001 \ p_{0}  l_{*}^{2} & \times \\ m_{*y} &=& -0.001 \ p_{0}  l_{*}^{2} & \times \\ a_{*}  l_{*} &=& \\ a_{y}  l_{y} &=& \end{array}$  | 18<br>18<br>51<br>51<br>0,18<br>0,18                             | 26         32           16         12           63         72           54         55           0.19         0.15           0.16         0.15 | 36<br>10<br>78<br>54<br>0,19<br>0,13   | 39<br>10<br>81<br>54<br>0,19<br>0,11  | 41<br>10<br>82<br>53<br>0,19<br>0,10  | 42<br>10<br>83<br>51<br>0,19<br>0,07  | 43<br>10<br>83<br>49<br>0,19<br>0,06  |
|   | $\begin{array}{ll} m_{\rm rst} &= 0.001 \; p_{\rm d}  l_{\star}^{ 2} \; \times \\ m_{\rm ry} &= 0.001 \; p_{\rm d}  l_{\star}^{ 2} \; \times \\ m_{\rm st} &= -0.001 \; p_{\rm d}  l_{\star}^{ 2} \; \times \\ m_{\rm ry} &= -0.001 \; p_{\rm d}  l_{\star}^{ 2} \; \times \\ a_{\star}  l_{\star} &= \\ a_{\star}  l_{\star} &= \\ \end{array}$ | 25<br>25<br>68<br>68<br>0,20<br>0,21                             | 36         45           23         20           84         97           74         77           0,22         0,22           0,19         0,17 | 53<br>19<br>106<br>77<br>0,22<br>0,16  | 58<br>18<br>113<br>77<br>0,23<br>0,13 | 62<br>17<br>117<br>76<br>0,23<br>0,12 | 67<br>17<br>122<br>73<br>0,24<br>0,09 | 69<br>17<br>124<br>71<br>0,24<br>0,09 |
|   | $\begin{array}{ll} m_{\rm vx} &= 0.001 \; p_d  l_x^2 \; \times \\ m_{\rm vy} &= 0.001 \; p_d  l_x^2 \; \times \\ m_{\rm vy} &= -0.001 \; p_d  l_x^2 \; \times \\ a_{\rm y}  l_{\rm y} &= \; \times \end{array}$  | 16<br>29<br>69<br>0,19   | 28 42<br>32 32<br>85 97<br>0,19 0,17  | 56<br>30<br>105<br>0,17  | 69<br>27<br>110<br>0,16               | 80<br>24<br>112<br>0,15               | 100<br>20<br>112<br>0,12              | 112<br>18<br>112<br>0,11              |
|   | $\begin{array}{l} m_{\rm rs} &= 0.001 \ p_0 \ l_*^2 \ \times \\ m_{\rm ry} &= 0.001 \ p_0 \ l_*^2 \ \times \\ m_{\rm rs} &= -0.001 \ p_0 \ l_*^2 \ \times \\ a_* \ l_* &= \qquad \times \end{array}$   | 29<br>16<br>69<br>0,19 0   | 34 38<br>14 13<br>76 80<br>0,20 0,20  | 40<br>13<br>82<br>0,21   | 42<br>13<br>83<br>0,21                | 42<br>13<br>83<br>0,21                | 42<br>13<br>83<br>0,21                | 42<br>13<br>83<br>0,21                |
|   | $\begin{array}{rcl} m_{\rm rst} &=& 0.001 \; p_d  l_s^{-2} \; \times \\ m_{\rm ry} &=& 0.001 \; p_d  l_s^{-2} \; \times \\ m_{\rm ry} &=& -0.001 \; p_d  l_s^{-2} \; \times \\ a_y  l  l_y &=& \times \end{array}$   | 27<br>38<br>91<br>0,21<br>0                                      | 41 54<br>37 34<br>102 108<br>0,21 0,20  | 67<br>30<br>111<br>0,18  | 78<br>27<br>113<br>0,17               | 89<br>25<br>114<br>0,15               | 105<br>24<br>114<br>0,13              | 115<br>23<br>114<br>0,10              |
|   | $\begin{array}{rcl} m_{\star\star} &=& 0.001 \ \rho_{d} \ l_{\star}^{2} &\times \\ m_{\star y} &=& 0.001 \ \rho_{d} \ l_{\star}^{2} &\times \\ m_{\star\star} &=& -0.001 \ \rho_{d} \ l_{\star}^{2} &\times \\ a_{\star} \ l_{\star} &=& \times \end{array}$   | 38<br>27<br>91<br>0,21<br>0                                      | 44 52<br>21 19<br>98 107<br>0,21 0,22   | 58<br>18<br>113<br>0,23  | 62<br>17<br>118<br>0,24               | 65<br>17<br>120<br>0,24               | 68<br>17<br>124<br>0,24               | 70<br>17<br>124<br>0,25               |
|   | $\begin{array}{ll} m_{sk} &= 0.001 \ p_{d} \ l_{s}^{2} &\times \\ m_{sy} &= 0.001 \ p_{d} \ l_{s}^{2} &\times \\ m_{sx} &= -0.001 \ p_{d} \ l_{s}^{2} &\times \\ m_{sy} &= -0.001 \ p_{d} \ l_{s}^{2} &\times \\ \sigma_{s} \ l_{s} &= &\times \\ \sigma_{y} \ l_{y} &= &\times \end{array}$   | 18<br>23<br>54<br>60<br>0,19<br>0,19<br>0,19<br>0,               | 29         39           23         20           72         88           69         74           .21         0.23           .18         0.17   | 47<br>17<br>100<br>76<br>0,23<br>0,15  | 54<br>15<br>108<br>76<br>0.23<br>0,14 | 59<br>14<br>114<br>76<br>0,24<br>0,14 | 66<br>13<br>121<br>73<br>0,24<br>0,10 | 69<br>13<br>124<br>71<br>0,24<br>0,08 |
|   | $\begin{array}{rcl} m_{xx} &= 0.001 \ p_d  l_x^2 &\times \\ m_{xy} &= 0.001 \ p_d  l_x^2 &\times \\ m_{xx} &= -0.001 \ p_d  l_x^2 &\times \\ m_{xy} &= -0.001 \ p_d  l_x^2 &\times \\ a_x  l_x &= & \times \\ a_y  l_y &= & \times \end{array}$  | 23 3<br>18 1<br>60 7<br>54 5<br>0,18 0,<br>0,19 0,               | 30         35           15         14           70         76           55         55           ,19         0,19           ,18         0,14   | 38<br>13<br>80<br>54<br>0,19<br>0,13   | 40<br>13<br>82<br>53<br>0,20<br>0,13  | 41<br>13<br>83<br>53<br>0,21<br>0,10  | 42<br>13<br>83<br>51<br>0,21<br>0,08  | 43<br>13<br>83<br>49<br>0,21<br>0,07  |
| = vrij opgelegd<br>= volledig ingeklemd | $m_{vy}$ is hel positieve moment pe<br>$m_{vy}$ is hel positieve moment pe<br>$m_{vy}$ is hel negatieve moment pe<br>$m_{vy}$ is hel negatieve moment pe   | r lengte in o<br>r lengte in o<br>er lengte lan<br>er lengte lan | de middende<br>de middende<br>ngs een lang<br>ngs een kon   | orsnede e<br>orsnede e<br>e zijde (I <sub>r</sub><br>e zijde (I <sub>s</sub> ) | venwijdi<br>venwijdi<br>)             | g ann de<br>g ann de                  | lange zij<br>korte zij                | de (/,)<br>jde (/,)                   |



# Appendix D

# Appendix E













## Appendix F

A field investigation of the caisson concrete structures of pier 7 (Waalhaven) is performed. The aim of the investigation is to evaluate the concrete characterization and service life time evaluation. This section will cover the following subjects;

- Compressive strength
- Carbonation-induced corrosion
- Chloride content

#### **Compressive strength**

The compressive strength of the concrete is tested with drilled cores from several sections, specifically 5 tests. The tested cores showed that the compressive strength of the concrete is much higher as estimated before. The average compressive strength of the concrete is 58.7 N/mm<sup>2</sup>, where before the estimation was around 24 N/mm<sup>2</sup>.

#### **Carbonation-induced corrosion**

Carbonation is a chemical process whereby calcium hydroxide, of the concrete, reacts with carbon dioxide from the air or water. The reaction of this two elements forms calcium carbonate. Laboratory tests showed a minimal risk of carbonation, with no carbonation depth at all.

#### **Chloride-induced corrosion**

A passive oxygen layer around the steel reinforcement protects the reinforcement bars against corrosion. Decrease of pH values or chloride ions can negatively affect the oxygen layer. The degradation of the passive oxygen layer take place when a certain amount of chloride is reached at the steel level.

From tests it has been found that the caissons, especially the submerged part, has high levels of chloride contamination caused by the salty of seawater which differs over the water level in the harbour.

#### Conclusion

Based on chemical degradation, chloride-induced corrosion and carbonation-induced corrosion, the caissons are expected to be in a good condition, except the submerged part which is contain high levels of chloride contamination. This part is in a dire need of reparation, however when this part stays under water where oxygen is limited, corrosion will propagate slowly.

Therefore it is important to mention, when the submerged part of the caisson in the further will be exposed to oxygenate area, reparation interventions should be proposed.



## **Appendix G**

#### **Monte Carlo**

The Monte Carlo method is a process for that rely on developing numerical results through randomly repeating samples. Simulations will run for many times over to find a distribution of an unknown probabilistic entity. It is commonly used for complications involving a random variable with an identified or presumed probability distribution function. The (arbitrarily) chosen values relate to their probability of occurrence as defined by the probability distribution function.

When estimating the costs of a (building) project, it is very common that to choose a triangular probability distribution function. The triangular distribution is chosen out of simplicity instead of a more advanced probability distribution function. The triangle probability distribution function contains of a conservative (i.e. largest value), a most likely (i.e. the mode itself) and an optimistic (i.e. smallest value) value. The probability distribution function of a triangular distribution is given by:

$$f(x|a,b,c) = \begin{cases} 0 & \text{for } x < a, \\ \frac{2(x-a)}{(b-a)(c-a)} & \text{for } a \le x \le c, \\ \frac{2(b-x)}{(b-a)(b-c)} & \text{for } c < x \le b \\ 0 & \text{for } b < x \end{cases}$$



**Figure 90 Triangular distribution** 

In a Monte Carlo simulation, the complete system is simulated a large number of times (iterations). For each single uncertain parameter, a value is selected from the chosen probability distribution function. Simulation of this system will result in a large number of independent outcomes, representing a possible outcome of the system that are gathered into probability distributions. Normally a Monte Carlo simulation should be carried out of 10.000 or more iterations for an ordinary project to ensure that the



outcomes are ruled out from most statistical biases (Vrijling, 2010). In Figure 93 one can see a simplified yet complete flowchart of the Monte Carlo technique.



Figure 91 Flowchart Monte Carlo method



## Appendix H

#### **Direct costs**

Г

| 10% $\notin$ 74.200 $\notin$ 7.420,00         10% $\notin$ 162.750 $\notin$ 16.275,00         15% $\notin$ 5.200 $\notin$ 7.80,00 | Total costs     NDC costs       10% $\in$ 74.200 $\in$ 7.420,00       10% $\in$ 162.750 $\in$ 16.275,00       15% $\in$ 5.200 $\in$ 7.80,00 | Total costs     NDC costs       10% $\in$ 74.200 $\in$ 7.420,00       10% $\in$ 162.750 $\in$ 16.275,00       15% $\in$ 5.200 $\in$ 780,00 | Total costsNDC costs10% $\notin$ 74.200 $\notin$ 7.420,0010% $\notin$ 162.750 $\notin$ 16.275,0015% $\notin$ 5.200 $\notin$ 780,00 | Total costs     NDC costs       10%     € 74.200     €7.420,00       10%     € 162.750     €16.275,00       15%     € 5.200     €780,00 | € 987.200         € 165.720           Total costs         NDC costs           10%         € 74.200         €7.420,00           10%         € 162.750         €16.275,00           15%         € 5.200         €7.80,00 | ZUNA         E 987.200         € 165.720 $\epsilon$ 987.200 $\epsilon$ 165.720           Total costs         NDC costs           10% $\epsilon$ 74.200 $\epsilon$ 7.420,00           10% $\epsilon$ 162.750 $\epsilon$ 16.275,00           15% $\epsilon$ 5.200 $\epsilon$ 780,00 | 20% $\epsilon$ 670.000 $\epsilon$ 134.000 $\epsilon$ 987.200 $\epsilon$ 145.720 $\epsilon$ 987.200 $\epsilon$ 145.720           Total costs         NDC costs           10% $\epsilon$ 74.200 $\epsilon$ 7.420,0C           10% $\epsilon$ 162.750 $\epsilon$ 7.420,0C           15% $\epsilon$ 5.200 $\epsilon$ 780,0C | 20% $\varepsilon$ 670.000 $\varepsilon$ 134.000 $\epsilon$ 987.200 $\varepsilon$ 145.720 $\epsilon$ 987.200 $\varepsilon$ 145.720           10% $\varepsilon$ 74.200 $\varepsilon$ 7.420,00           10% $\varepsilon$ 162.750 $\varepsilon$ 716.275,00           15% $\varepsilon$ 5.200 $\varepsilon$ 780,00 | $ \begin{array}{c ccccccccccccccccccccccccccccccccccc$  | $ \begin{array}{c ccccccccccccccccccccccccccccccccccc$  | $ \begin{array}{c ccccccccccccccccccccccccccccccccccc$  | $ \begin{array}{c 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€ 162.750         €16.275,00           15%         € 5.200         €780,00 | € 987.200         € 165.720           Total costs         NDC costs           10% $\in$ 74.200 $\in$ 7.420,00           10% $\notin$ 162.750 $\notin$ 16.275,00           15% $\notin$ 5.200 $\notin$ 7.80,00 | € 987.200         € 165.720           Total costs         NDC costs           10%         € 74.200         €7.420,00           10%         € 162.750         €16.275,00           15%         € 5.200         €780,00 | € 987.200         € 165.720           Total costs         NDC costs           10% $\notin$ 74.200 $\notin$ 7.420,00           10% $\notin$ 162.750 $\notin$ 16.275,00           15% $\notin$ 5.200 $\notin$ 7.80,00 | € 987.200         € 165.720           Total costs         NDC costs           10% $\in$ 74.200 $\in$ 7.420,00           10% $\in$ 162.750 $\in$ 16.275,00           15% $\in$ 5.200 $\in$ 7.80,00 | Total costs         NDC costs           10%         € 74.200         €7.420,00           10%         € 162.750         €16.275,00           15%         € 5.200         €780,00 | Total costs     NDC costs       10% $\in$ 74.200 $\in$ 7.420,00       10% $\in$ 162.750 $\in$ 16.275,00       15% $\in$ 5.200 $\in$ 780,00 | Total costsNDC costs10% $\notin$ 74.200 $\notin$ 7.420,0010% $\notin$ 162.750 $\notin$ 16.275,0015% $\notin$ 5.200 $\notin$ 780,00 | Total costs     NDC costs       10% $\notin$ 74.200 $\notin$ 7.420,00       10% $\notin$ 162.750 $\notin$ 16.275,00       15% $\notin$ 5.200 $\notin$ 780,00 | 10% $\notin$ 74.200 $\notin$ 74.20/00         10% $\notin$ 162.750 $\notin$ 16.275,00         15% $\notin$ 5.200 $\notin$ 780,00 | 10% $\notin$ 74.200 $\notin$ 7.420,00       10% $\notin$ 162.750 $\notin$ 16.275,00       15% $\notin$ 5.200 $\notin$ 7.80,00 | 10% $\in$ 162.750 $\in$ 16.275,00         15% $\in$ 5.200 $\in$ 780,00 | 10%     € 162.750     €16.275,00       15%     € 5.200     €780,00 | 15% € 5.200 €780,00      | 15% E 5.200 E /80,00 | _     | 25% € 1.215.900 €303.975,00 |       | € 1.458.050 € 328.450 | Total costs NDC costs | 25% € 90.000 €22.500,00 | 25% € 28.800 €7.200,00 | 25% € 9.000 €2.250,00         | € 37,800 € 31,950 |   | Total direct costs | € 2.483.050         | Total NDC costs | F 574 130 | E 320.12U | Total | 02100053 | € 3.009.170 |
|---|---|--|--|---|--|---|---|---|---|---|---|---|--|---|--|--|---|---|---|---|---|---|--|--|--|--|---|--|--|--------------------------|----------------------|-------|-----------------------------|-------|-----------------------|-----------------------|-------------------------|------------------------|-------------------------------|-------------------|---|--------------------|---------------------|-----------------|-----------|-----------|-------|----------|-------------|
| 1 €100,00<br>3 €35,00<br>3 €25,00   | Unit Price       1     €100,00       3     €35,00       3     €25,00  | Unit Price           1         €100,00           3         €35,00           3         €25,00   | Unit Price       1     €100,00       3     €35,00  | Unit Price  | Unit Price<br>€100,00<br>€15,00<br>€25,00  | <ul> <li>CO,OO</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul>  | <ul> <li>€10,00</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul>   | <ul> <li>€10,00</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul>   | <ul> <li>€10,00</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul> | <ul> <li>€10,00</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul> | <ul> <li>3 €10,00</li> <li>€10,00</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul> | <ul> <li>€10,00</li> <li>E10,00</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul> | <ul> <li>€10,00</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul>  | <ul> <li>€10,00</li> <li>Unit Price</li> <li>€100,00</li> <li>€35,00</li> <li>€25,00</li> </ul> | Unit Price       €100,00       €35,00       €25,00   | Unit Price<br>€100,00<br>€15,00<br>€25,00  | Unit Price  | Unit Price<br>€100,00<br>€35,00<br>€25,00   | Unit Price  | Unit Price<br>€100,00<br>€35,00<br>€25,00   | Unit Price<br>€100,00<br>€35,00<br>€25,00   | Unit Price  | Unit Price       1     €100,00       3     €35,00       3     €25,00   | Unit Price           1         €100,00           3         €35,00           3         €25,00                                       | Unit Price           1         €100,00           3         €35,00           3         €25,00   | 1 €100,00<br>3 €35,00<br>3 €25,00  | 1 €100,00<br>3 €35,00<br>3 €25,00   | 3 €35,00<br>3 €25,00   | <sup>3</sup> €35,00 €25,00   | 3 €25,00                 | ° €∠5,UU             |       | 3 €70,00                    |       | € 230                 | Linit Drico           | s €1.000,00             | s €320,00              | 00,000                        | € 10.320          | 1 |                    | € 300.917           |                 |           |           |       |          |             |
| 742 n<br>4 650 m  | Estimated amounts<br>742 n<br>4.650 m   | Estimated amounts<br>742 n<br>4.650 m  | Estimated amounts<br>742 n<br>4.650 m  | Estimated amounts<br>742 n<br>4.650 m   | Estimated amounts<br>742 n   | Estimated amounts<br>742 m  | 67.000 m<br>Estimated amounts<br>742 n<br>4.650 m   | 67.000 m<br>Estimated amounts<br>742 m  | 67.000 m<br>Estimated amounts<br>742 m  | 67.000 m<br>Estimated amounts<br>742 m  | Estimated amounts<br>4 650 m  | Estimated amounts<br>4 650 m  | 67.000 m<br>Estimated amounts<br>742 m   | 67.000 m<br>Estimated amounts<br>742 m  | Estimated amounts<br>742 n   | Estimated amounts<br>742 n   | Estimated amounts<br>742 n<br>4.650 m   | Estimated amounts<br>742 n<br>4.650 m   | Estimated amounts<br>742 n<br>4.650 m   | Estimated amounts<br>742 n  | Estimated amounts<br>742 n<br>4.650 m   | Estimated amounts<br>742 n<br>4.650 m   | Estimated amounts<br>742 n<br>4.650 m  | Estimated amounts<br>742 n<br>4.650 m  | Estimated amounts<br>742 n<br>4.650 m  | 742 n<br>4.650 m   | 742 n<br>4 650 m  | 4 650 m  | 4 650 m  |                          |                      | Z08 m | 17.370 m                    | -     |                       | Ectimated amounte     | 90 day                  | 90 day                 | 1 Eur                         | -                 |   |                    | 10%                 |                 |           |           |       |          |             |
| Remove of nautical facilities   | Demolition costs<br>Remove of nautical facilities   | Demolition costs<br>Remove of nautical facilities  | Demolition costs<br>Remove of nautical facilities  | Demolition costs<br>Remove of nautical facilities   | Total<br>Demolition costs<br>Remove of nautical facilities   | Total<br>Demolition costs<br>Remove of nautical facilities  | Remove soil in compartments<br>Total<br>Demolition costs<br>Remove of nautical facilities   | Remove soil in compartments<br>Total<br>Demolition costs<br>Remove of nautical facilities   | Remove soil in compartments<br>Total<br>Demolition costs<br>Remove of nautical facilities       | Remove soil in compartments<br>Total<br>Demolition costs<br>Remove of nautical facilities       | Remove soil in compartments<br>Total<br>Demolition costs<br>Remove of nautical facilities                         | Remove soil in compartments<br>Total<br>Demolition costs<br>Remove of nautical facilities                       | Remove soil in compartments<br>Total<br>Demolition costs<br>Remove of nautical facilities  | Remove soil in compartments<br>Total<br>Demolition costs<br>Remove of nautical facilities       | Total<br>Demolition costs<br>Remove of nautical facilities   | Total<br>Demolition costs<br>Remove of nautical facilities   | Total<br>Demolition costs<br>Remove of nautical facilities  | Total<br>Demolition costs<br>Remove of nautical facilities  | Total<br>Demolition costs<br>Remove of nautical facilities  | Total<br>Demolition costs<br>Remove of nautical facilities  | Total<br>Demolition costs<br>Remove of nautical facilities  | Demolition costs<br>Remove of nautical facilities   | Demolition costs<br>Remove of nautical facilities  | Demolition costs<br>Remove of nautical facilities  | Demolition costs<br>Remove of nautical facilities  | Remove of nautical facilities  | Remove of nautical facilities   |  |  | Demolition substructures |                      |       | Demolition caissons         | Total | lotal                 | Transnort costs       | Tugboats (3)            | Employers (3)          | Coordination & traffic safety | Total             |   | Unforeseen costs   | 10 % of total costs |                 |           |           |       |          |             |

| NDC costs               |             |            | NDC costs               | I           |             | NDC costs              | I         |           |             | ı             | I             | I                     | I             |             |
|-------------------------|-------------|------------|-------------------------|-------------|-------------|------------------------|-----------|-----------|-------------|---------------|---------------|-----------------------|---------------|-------------|
| Total costs             | €79.551,48  | €79.551,48 | Total costs             | €123.268,50 | €123.268,50 | Total costs            | €4.812,75 | €4.812,75 | Total costs | €99.322,00    | €99.322,00    | € 4.811,72            | € 24.374,28   | €203.455,72 |
|                         | 6'9%        |            |                         | 6,9%        |             |                        | 6,9%      |           |             | 4%            | 4%            | 0,15%                 | 0,75%         |             |
| Actual                  | € 1.152.920 |            | Actual                  | € 1.786.500 |             | Actual                 | € 69.750  |           | Actual      | €2.483.050,00 | €2.483.050,00 | €3.207.814,00         | €3.249.903,60 |             |
| nated                   | %           |            | nated                   | %           |             | nated                  | %         |           | nated       | %             | %             | %                     | %             |             |
| Estin                   |             |            | Estin                   |             |             | Estin                  |           |           | Estin       |               |               |                       |               |             |
| cavation indirect costs | e costs     | al         | molition indirect costs | e costs     | al          | ansport indirect costs | e costs   | al        | her costs   | neral costs   | fit & risk    | ntributions (RAW FCO) | urance CAR    | al          |

#### Indirect costs





### Summary of the costs

| Construction costs | Costs       | Unforeseen  |
|--------------------|-------------|-------------|
| Direct costs       | € 3.009.170 | -           |
| Indirect costs     | € 411.088   | -           |
| Unforeseen         | -           | € 300.917   |
| Subtotal           |             | € 3.721.175 |

| Engineering costs |           |           |
|-------------------|-----------|-----------|
| Direct costs      | € 240.734 | -         |
| Indirect costs    | -         | -         |
| Unforeseen        | -         | -         |
| Subtotal          |           | € 240.734 |

| Other additional costs |           |           |
|------------------------|-----------|-----------|
| Direct costs           | € 203.456 | -         |
| Indirect costs         | -         | -         |
| Unforeseen             | -         | -         |
| Subtotal               |           | € 203.456 |

| Total costs                       | +           | +                                 |
|-----------------------------------|-------------|-----------------------------------|
|                                   | € 3.864.448 | € 300.917                         |
| Lower bound<br>25%<br>€ 3.124.024 | € 4.165.365 | Upper bound<br>25%<br>€ 5.206.706 |



## **Appendix I**

One way of decreasing the draught is by reducing weight of the caisson. This can be done by sawing the top of the walls which lead to weight reduce. Besides reducing of weight, it can be promising for a function to reduce the height. For calculating the draught, the height is reduced each time with 1 meter. Each meter reduction is equivalent to a reduction of weight of 55.73 m<sup>3</sup> of concrete.

Actually by changing the shape of the structure, the centre of gravity moves more downwards with respect to the bottom. For each meter reduction the centre of gravity is determined by the equation below (approximately) or more accurate by AutoCAD.

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

The centre of gravities (only with respect to x-direction) are showed in Table 63 for each reduced meter height.

| Height [m] | Y-coordinate of gravity centre [m] | Ballast [m] |
|------------|------------------------------------|-------------|
| 12.50      | 4.61                               | 2.30        |
| 11.50      | 3.84                               | 0.5         |
| 10.50      | 3.33                               | 0           |
| 9.50       | 2.91                               | 0           |
| 8.50       | 2.52                               | 0           |
| 7.50       | 2.12                               | 0           |
| 6.50       | 1.75                               | 0           |
| 5.50       | 1.41                               | 0           |

Table 62 Gravity centres

On basis of the equations in paragraph 5.2, the results in Table 63 are obtained. For all draught the static stability is checked. By reducing height of the walls, and therefore weight, there is also less ballast needed.





Figure 92 Position of gravity centre, including coordinate system.



## **Appendix J**

Using the caissons as building foundation gives the question to which extent the buildings can rise. The basement floor and upper floors should not exceed the bearing capacity of the concrete structure. The calculations are made with the software MatrixFrame. The following parameters and loads are used:

Table 63 Parameter building structure

| Parameters       | Values                |
|------------------|-----------------------|
| Concrete class   | C20/25                |
| Concrete density | 25 kN/m³              |
| Reinforcement    | 435 N/mm <sup>2</sup> |

To verify the caisson structure as building foundation, a building is constructed in order to determine the foundation forces. The building consist of three parts; the ground floor, the roof and the floors. The number of floors depends on the concrete strength, failure mechanisms of the caissons and rules provided by the guidelines.

Figure 94 and Figure 95 shows the building with the most important dimensions. The columns of the building coincides with the centre-to-centre distance of both outer walls of the caisson to have favourable force transfer between the building and the caisson and most favourable position for the stability of the walls.



Figure 93 Cross-section caisson and building







The following loads are considered as governing and crucial for the foundation:

- Self-weight of floor, beam and column
- Partition walls
- Screed
- Façade
- Wind

| Screed:          | 1.0 kN/m² |
|------------------|-----------|
| Roof:            | 0.7 kN/m² |
| Partition walls: | 0.5 kN/m² |
| Façade:          | 5.0 kN/m² |

#### Permanent loads

| Ground floor       | Self-weight     | $0.2 \cdot 24 =$ | 4.8 kN/m²        |   |
|--------------------|-----------------|------------------|------------------|---|
|                    | Screed          |                  | 1.0 kN/m²        |   |
|                    | Partition walls |                  | <u>0.5 kN/m²</u> | + |
|                    |                 |                  | 6.3 kN/m²        |   |
|                    |                 |                  |                  |   |
| Intermediate floor | Self-weight     | 0.2 · 24 =       | 4.8 kN/m²        |   |
|                    | Screed          |                  | 1.0 kN/m²        |   |
|                    | Ceiling         |                  | 0.3 kN/m³        |   |



|                     | Partition walls |                                     | <u>0.5 kN/m²</u> | + |
|---------------------|-----------------|-------------------------------------|------------------|---|
|                     |                 |                                     | 6.6 kN/m²        |   |
|                     |                 |                                     |                  |   |
| Roof                | Self-weight     | $0.2 \cdot 24 =$                    | 4.8 kN/m²        |   |
|                     | Roofing         |                                     | 0.5 kN/m²        |   |
|                     | Ceiling         |                                     | <u>0.3 kN/m²</u> | + |
|                     |                 |                                     | 5.6 kN/m²        |   |
|                     |                 |                                     |                  |   |
| Beam (ground) floor | Self-weight     | $(0.55 - 0.2) \cdot 0.3 \cdot 24 =$ | 2.52 kN/m        |   |
|                     | Partition walls | $0.2 \cdot 3 \cdot 18 =$            | 10.8 kN/m        |   |
|                     | Floor           | 6.6 · 6 =                           | <u>39.6 kN/m</u> | + |
|                     |                 |                                     | 52.9 kN/m        |   |
|                     |                 |                                     |                  |   |
| Beam roof           | Self-weight     | $(0.55 - 0.2) \cdot 0.3 \cdot 24 =$ | 2.52 kN/m        |   |
|                     | Floor roof      | $5.5 \cdot 6 =$                     | <u>33 kN/m</u>   | + |
|                     |                 |                                     | 35.5 kN/m        |   |
| Column facade       | Self-weight     | $5 \cdot 6 =$                       | 2.52 kN/m        |   |
|                     | Column          | $5.5 \cdot 6 =$                     | <u>33 kN/m</u>   | + |
|                     |                 |                                     | 35.5 kN/m        |   |

## Live loads

| Persons and goods (residential): | 2.5 kN/m²  |
|----------------------------------|------------|
| Persons and goods<br>(shop):     | 4 kN/m²    |
| Snow:                            | 0.56 kN/m² |
| Wind:                            | 0.91 kN/m² |

| Beam ground floor    | $1.5 \cdot 4 \cdot 6 =$              | 36 kN/m      |
|----------------------|--------------------------------------|--------------|
| Beam floor           | $1.5 \cdot 2.5 \cdot 6 =$            | 22.5 kN/m    |
| Beam roof            | $1.5 \cdot 0.56 \cdot 6 =$           | 22.5 kN/m    |
| Façade wind pressure | $1.5 \cdot 0.8 \cdot 0.64 \cdot 6 =$ | $4.6 \ kN/m$ |



#### Load combinations

Table 64 Load combination 1

| Beam ground floor       | $q_d = 1.35 \cdot 52.9 = 71.4 \ kN/m$ |
|-------------------------|---------------------------------------|
| Beam intermediate floor | $q_d = 1.35 \cdot 52.9 = 71.4 \ kN/m$ |
| Beam roof               | $q_d = 1.35 \cdot 35.5 = 47.9 \ kN/m$ |
| Column facade           | $F_d = 1.35 \cdot 40.8 = 55.1  kN/m$  |

| Table 65 Load combination 2 |   |
|-----------------------------|---|
| Beam ground floor           | $q_d = 1.2 \cdot 52.9 + 1.5 \cdot 4 \cdot 6 = 99.5 \ kN/m$    |
| Beam intermediate floor     | $q_d = 1.2 \cdot 52.9 + 1.5 \cdot 2.5 \cdot 6 = 86 \ kN/m$    |
| Beam roof                   | $q_d = 1.2 \cdot 35.5 + 1.5 \cdot 0.56 \cdot 6 = 52.9 \ kN/m$ |
| Column facade               | $F_d = 1.2 \cdot 40.8 + 1.5 \cdot 0.64 \cdot 6 = 53.6  kN/m$  |

Load combination 2 is the governing load combination, because the load is larger than of load combination 1. Load combination 2 will be used as input for calculating the forces.

#### MatrixFrame results

2 floor building



#### Figure 95 Reaction forces on the foundation (2 floors, ground floor and roof)



Figure 96 Moment diagram ground floor (2 floors, ground floor and roof)





Figure 97 Reaction forces of foundation (1 floor, ground floor and roof)



Figure 98 Moment diagram ground floor (1 floor, ground floor and roof)



### Wind load



Figure 99 Wind regions in Netherlands



#### Table 66 Wind load

| L   | Windstuwdruk in kN/m <sup>2</sup> |         |                   |      |           |         |
|-----|-----------------------------------|---------|-------------------|------|-----------|---------|
| (m) | gebie                             | ed I    | gebied II         |      | gebie     | d III   |
| . , | onbebouwd                         | bebouwd | onbebouwd bebouwd |      | onbebouwd | bebouwd |
| ≤2  | 0,64                              | 0,64    | 0,54              | 0,54 | 0,46      | 0,46    |
| 3   | 0,70                              | 0,64    | 0,54              | 0,54 | 0,46      | 0,46    |
| 4   | 0,78                              | 0,64    | 0,62              | 0,54 | 0,49      | 0,46    |
| 5   | 0,84                              | 0,64    | 0,68              | 0,54 | 0,55      | 0,46    |
| 6   | 0,90                              | 0,64    | 0,73              | 0,54 | 0,59      | 0,46    |
| 7   | 0,95                              | 0,64    | 0,78              | 0,54 | 0,63      | 0,46    |
| 8   | 0,99                              | 0,64    | 0,81              | 0,54 | 0,67      | 0,46    |
| 9   | 1,02                              | 0,64    | 0,85              | 0,54 | 0,70      | 0,46    |
| 10  | 1,06                              | 0,70    | 0,88              | 0,59 | 0,73      | 0,50    |
| 11  | 1,09                              | 0,76    | 0,91              | 0.64 | 0,76      | 0,54    |
| 12  | 1,12                              | 0,81    | 0,94              | 0,68 | 0,78      | 0,58    |
| 13  | 1,14                              | 0,86    | 0,96              | 0,72 | 0,80      | 0,61    |
| 14  | 1,17                              | 0,90    | 0,99              | 0,76 | 0,82      | 0,64    |
| 15  | 1,19                              | 0,94    | 1,01              | 0,79 | 0,84      | 0,67    |
| 16  | 1,21                              | 0,98    | 1,03              | 0,82 | 0,86      | 0,70    |
| 1/  | 1,23                              | 1,02    | 1,05              | 0,85 | 0,88      | 0,72    |
| 18  | 1,25                              | 1,05    | 1,07              | 0,88 | 0,90      | 0,75    |
| 19  | 1,27                              | 1,08    | 1,09              | 0,90 | 0,91      | 0,77    |
| 20  | 1,29                              | 1,11    | 1,10              | 0,93 | 0,93      | 0,79    |
| 25  | 1,37                              | 1,23    | 1,18              | 1,03 | 1,00      | 0,88    |
| 30  | 1,43                              | 1,34    | 1,24              | 1,12 | 1,06      | 0,95    |
| 35  | 1,49                              | 1,43    | 1,30              | 1,20 | 1,11      | 1,02    |
| 40  | 1,54                              | 1,50    | 1,35              | 1,20 | 1,15      | 1,07    |
| 40  | 1,08                              | 1,57    | 1,39              | 1,32 | 1,19      | 1,12    |
| 50  | 1,02                              | 1,02    | 1,43              | 1,37 | 1,23      | 1,10    |
| 20  | 1,00                              | 1,00    | 1,40              | 1,42 | 1,20      | 1,20    |
| 60  | 1,09                              | 1,69    | 1,50              | 1,40 | 1,29      | 1,24    |
| 70  | 1,73                              | 1,73    | 1,03              | 1,50 | 1,32      | 1,27    |
| 70  | 1,70                              | 1,70    | 1,00              | 1,04 | 1,34      | 1,31    |
| 10  | 1,70                              | 1,70    | 1,00              | 1,57 | 1,37      | 1,33    |
| 00  | 1,01                              | 1,01    | 1,01              | 1,00 | 1,39      | 1,30    |
| 00  | 1,03                              | 1,03    | 1,05              | 1,05 | 1,41      | 1,39    |
| 90  | 1,00                              | 1,00    | 1,00              | 1,05 | 1,45      | 1,41    |
| 100 | 1,00                              | 1,00    | 1,00              | 1,00 | 1,45      | 1.44    |
| 110 | 1.90                              | 1,90    | 1.74              | 1.74 | 1.51      | 1,40    |
| 120 | 1,94                              | 1,94    | 1,74              | 1,74 | 1.51      | 1.50    |
| 120 | 2.01                              | 2.01    | 1.90              | 1.90 | 1,04      | 1,54    |
| 140 | 2,01                              | 2,01    | 1.83              | 1,00 | 1,57      | 1,57    |
| 150 | 2,04                              | 2,04    | 1.86              | 1.86 | 1.62      | 1,00    |
| 150 | 2,01                              | 2,07    | 1,00              | 1,00 | 1,02      | 1,02    |