Offshore Pumped Storage Hydropower

Design, Planning and Cost Assessment

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Offshore Pumped Storage Hydropower: Design, Planning and Cost Assessment

by

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Cover: Boskalis "BOKA VANGUARD" at the outer port of A Coruña, Spain (May, 2022). (Modified)

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Preface

This thesis concludes the Master Hydraulic Engineering and specialisation Coastal Engineering at the Delft University of Technology, the Netherlands. This research is a feasibility study of an offshore Pumped Storage Hydropower (PSH) facility, with the goal to create more insight in the effects of sheltering and diffraction on the construction costs and cost-effectiveness.

I would like to thank the members of the graduation committee for their comments, suggestions and passion throughout the entire research. Their input and insight has been inestimable value to me. Individually, I would like to thank Cong Mai Van for his constructive and valuable feedback regarding risk and research outline, Roelof Moll for being a very helpful person and his extensive notes, George Lavidas for his extensive expertise regarding offshore engineering, and last but not least, Lucas de Vilder for his support and coffee breaks throughout the entire research.

Furthermore, I owe special thanks to Stefan van de Sande for his involvement throughout the research, Casper Mudde and Philipp Bangen for sharing their experience and knowledge regarding workability and Remy Traversier for this enthusiasm and expertise regarding the planning aspects.

Finally, I would like to thank my friends, family and girlfriend for their continuous support throughout my study.

Sicco Folkert Nessen
Delft, November 2023

Summary

Over the past few years, climate change has been combated by reducing the emission of greenhouse gasses, also known as deep decarbonization. In particular, fossil fuel energy resources have been substituted by renewable alternatives (especially wind and solar energy). Since these resources are fully dependent on the highly variable weather, the reliability and the continuity forms the next challenge: how to provide a constant, continuous and sufficient energy supply that meets the demand at all times?

Back in 1981, the Dutch Hydraulic Engineer Luc Lievense introduced Plan Lievense: a conceptual plan that includes a pumped storage hydropower solution suitable for low-lying countries like the Netherlands. This concept works by imposing a water level difference between two reservoirs, which is regulated by pumps and turbines. During periods of energy abundance, water will be pumped out, thereby charging the reservoir. When energy demand increases, water will be discharged into the reservoir, generating electricity with turbines.

This study focuses on an offshore pumped storage hydropower plant located in the North Sea. The facility consists of a round caisson dam supported by an inner berm. With a diameter of 5 km and a local water depth of 29 m, the total gross capacity of the reservoir is 23 GWh with an installed capacity of 2 GW.

More specifically, this research aims to improve the current knowledge and insight into the construction process, logistics, and construction costs associated with the offshore pumped storage hydropower plant. Constructing an offshore mega structure, particularly one that requires very specific geotechnical conditions, poses significant challenges. During the construction of the almost 16 km long round caisson dam, the dam will partly block and diffract incoming waves, thereby improving local wave conditions at the leeward side of the dam. The main objective is to enhance the construction costs by optimizing the work method based on these sheltering effects.

To determine the potential locations in the North Sea, a geo-technical assessment, GIS analysis, and wave data statistics have been applied. After thoroughly evaluation, a suitable area 50 km off the coast of Texel has been identified, meeting all requirements. The design of the caisson dam has been developed following Dutch and Spanish design manuals and has been assessed under various governing conditions.

For the construction planning, two approaches have been considered. The deterministic planning computed activity durations neglecting weather conditions. A probabilistic construction planning has been composed by combining this deterministic approach with wave data statistics. For a conservative approach, an exceedance probability of 70% has been applied to the wave data.

Ultimately, the construction costs have been determined and the corresponding cost-effectiveness of the project has been calculated by applying the Levelised Cost of Storage (LCOS). The LCOS is an important metric used to assess the economic viability of energy storage projects.

The effect of diffraction of the partly constructed caisson dam, has been analysed with the Goda diffraction tables. Various scenarios, among them are those that exclude or include sheltering effects and subsequent optimization of the work method and orientation of the dam, have been assessed.

Including sheltering effects and corresponding optimization of the work method substantially improves the cost-effectiveness of the offshore pumped storage hydropower plant. For instance, the construction time decreases from 9 years and 2 months to only 3 years and 6 months, thereby enhancing the LCOS by 15%, from € 212/MWh to € 180/MWh.

In summary, this research addresses the promising potentials of work method optimization for offshore caisson projects regarding the local wave climate. For a case study in the North Sea, it has not only highlighted the relevance and importance of including wave sheltering, but also shows the competitiveness of the offshore pumped storage hydropower concept once again. Alternative energy storage methods such as lithium-ion or hydrogen are in the range of € 200-400/MWh for lithium-ion and € 200-1900/MWh for hydrogen storage. Therefore, offshore pumped storage hydropower (PSH) can be a favorable solution enhancing the energy transition.

Contents

Nomenclature

Hydraulic Characteristics

Φ	Stability parameter	–
Ψ	Shields parameter	–
h	Water depth	m
h'	Reduced water depth above foundation level of the structure	m
h_0	Wave induced water set-up	m
H_{in}	Incoming wave height	m
h_{sill}	Height of the sill or foundation layer of the caisson	m
k	Wave number	m^{-1}
K_h	Depth factor	–
K_s	Steepness factor	–
K_t	Turbulence factor	–
L	Wave length	m
L_0	Deep water wave length	m
p_0	Wave induced water pressure	kPa
p_1	Wave induced water pressure	kPa
T	Wave period	s
u	Flow velocity	$m\,s^{-1}$

Soil Characteristics

Δ	Relative density	$kg\,m^{-3}$
δ	External friction angle	20°
γ_c	Specific weight of concrete	$25.0\,kN/m^3$
γ_s	Specific weight of sand	$20.0\,kN/m^3$
γ_s'	Effective specific weight of sand	$10\,kN/m^3$
γ_w	Specific weight of water	$10.1\,kN/m^3$
ϕ	Internal friction angle	30°
ρ_c	Density of concrete	$2550\,kg\,m^{-3}$
ρ_s	Density of sand	$2000\,kg\,m^{-3}$
ρ_w	Density of water	$1025\,kg\,m^{-3}$
$\sigma_{k,max}$	Maximum acting vertical stresses on the subsoil	kPa
$\sigma'_{k,max}$	Reduced maximum acting vertical stresses on the subsoil due to increase in surface area	kPa
b'	Increased acting width due to the presence of the sill	m
c'	Cohesion sand	0 kPa
d	Grain diameter	m

d_{50}	Median of grain size distribution	cm
$d_{n,50}$	Nominal median grain size	cm
f	Friction coefficient	–
i_γ	Factor for horizontal load	–
N_γ	Factor for the subsoil	–
N_q	Factor for surcharge	–
p'_{max}	Maximum soil bearing capacity	kPa
s_γ	Shape factor for foundations	–
u_{10}	Wind speed measured at 10 m above the ground.	m/s

Other Symbols

d_i	Consequence of the event	€
$E(d)$	Risk associated with an event	€/year
e_R	Distance from the moment centre (K) to the intersection point of the resulting force with the bottom line	m
f	Frequency of occurrence	year^{-1}
F_b	Buoyant force	kN
F_w	Weight force	kN
g	Gravitational acceleration	$9.81\,\text{m}\,\text{s}^{-2}$
h	Caisson height	m
i	Scenario event	–
l	Caisson length	m
P	Probability of failure	year^{-1}
p_i	Probability of occurrence	1/year
R	Return period	years
T	Lifespan of the structure	years
t_b	Caisson bottom thickness	m
t_w	Caisson wall thickness	m
V_{uw}	Submerged volume of floating element	m^3
w	Caisson width	m

List of Abbreviations

PSH Pumped Storage Hydropower

CSD Cutter Suction Dredge

TSHD Trailing Suction Hopper Dredge

SSDV Side Stone Dumping Vessel

HLB Heavy Lifting Barge

AHT Anchor Handling Tug

FPV Fall Pipe Vessel

HTV Heavy Transport Vessel

BHD Back Hoe Dredge

HAT Highest Astronomical Tide

HMW High Mean Water

MSL Mean Sea Level

LMW Low Mean Water

LAT Lowest Astronomical Tide

LCOS Levelised Cost of Storage

NPV Net Present Value

PV Present Value

FV Future Value

CAPEX Capital Expenditures

OPEX Operational Expenditures

DoD Depth of Discharge

List of Figures

List of Tables

1 Introduction

The energy transition towards renewable and sustainable energy resources requires a multi-disciplinary collaboration in terms of innovation, investment, policies and strategies, in order to achieve the goals set in the Paris Climate Agreement (United Nations, 2015). One of the hurdles to achieve deep decarbonization is the storage of (renewable) energy (Schill, 2020), which may offer a solution to overcome the intermittency of renewable energy resources. Currently Pumped Storage Hydropower (PSH) is the most used energy storage technology, however for low-lying countries often inapplicable due to the dependence of mountainous terrain. To that end, this thesis will look into offshore pumped hydropower storage.

Based on *Plan Lievense*, after the Dutch engineer L.W. Lievense, the European Horizon 2020 research program Alpheus considers a ring dam of caissons in the North-sea, which creates a circular reservoir. During energy abundant periods water will be pumped out of the reservoir, lowering the water level inside the reservoir with respect to the sea level, creating a head difference. When energy deficits build up, the water can be directed back into the reservoir levelling out the established head difference and will generate energy. An artist impression of project Alpheus is shown in Figure 1.1, besides the reservoir other utilities likes a power house, (working) harbor and adjacent wind farms are likely to be realised.

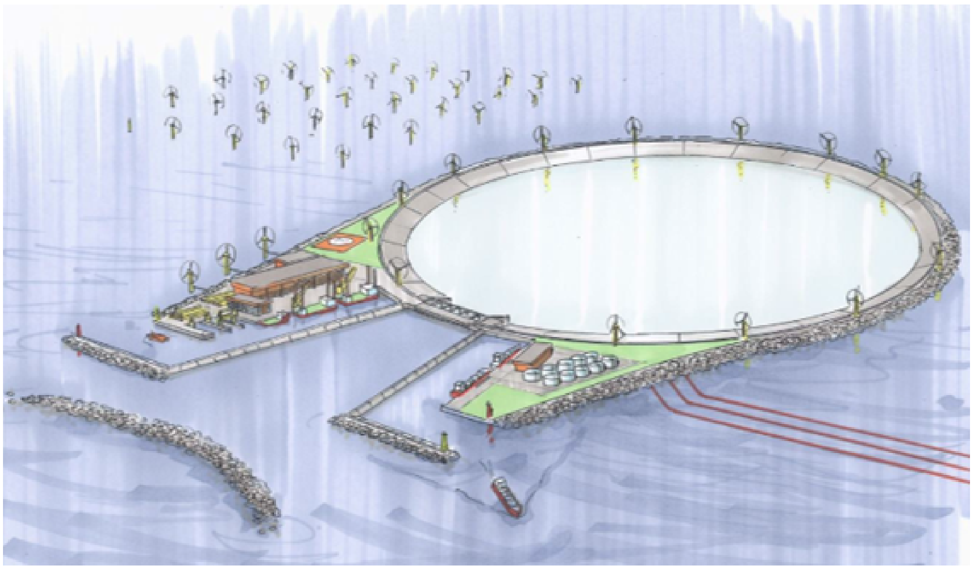


Figure 1.1: Artist impression of Project Alpheus.

After previous studies, like several BSc theses and the MSc thesis of de Vilder (2017), the conceptual design of the dam has already been finished, and is schematically shown in Figure 1.2. The design consists of a caisson dam with a gentle sloped inner berm at the reservoir side, to resist the hydraulic loads when the reservoir is entirely emptied. With an estimated energy storage capacity of 20 GWh, the diameter will be roughly 5 km, a perimeter of 16 km and a water depth of 20 m resulting in a surface area of 20 km^2 and total reservoir volume of 3.9·10^8 m^3.

As can be seen in the conceptual design in Figure 1.2, the caisson dam must be constructed on an impermeable layer (i.e. clay) to prevent bursting, piping and (extreme) seepage to happen. Hence, the location must contain a substantially clay layer to prevent the latter mechanisms and must be relatively

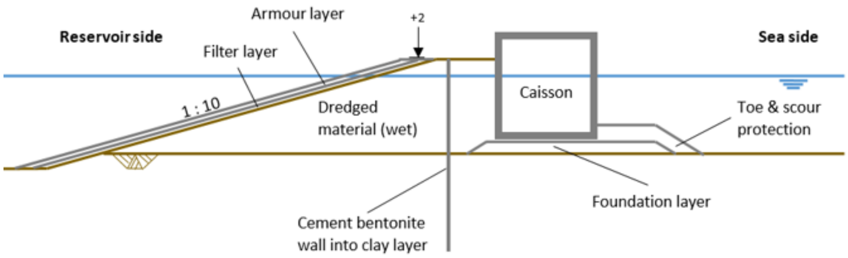


Figure 1.2: Schematic cross section of the caisson dam.

shallow, as a water depth of 20-30 m is desired. To that end, GIS-based site identifications have already been executed and has reduced the possible location to two options that would be viable. In Figure 1.3 the potential offshore sites are shown, and are located 45 km and 90 km of aerial distance from the Dutch shore.

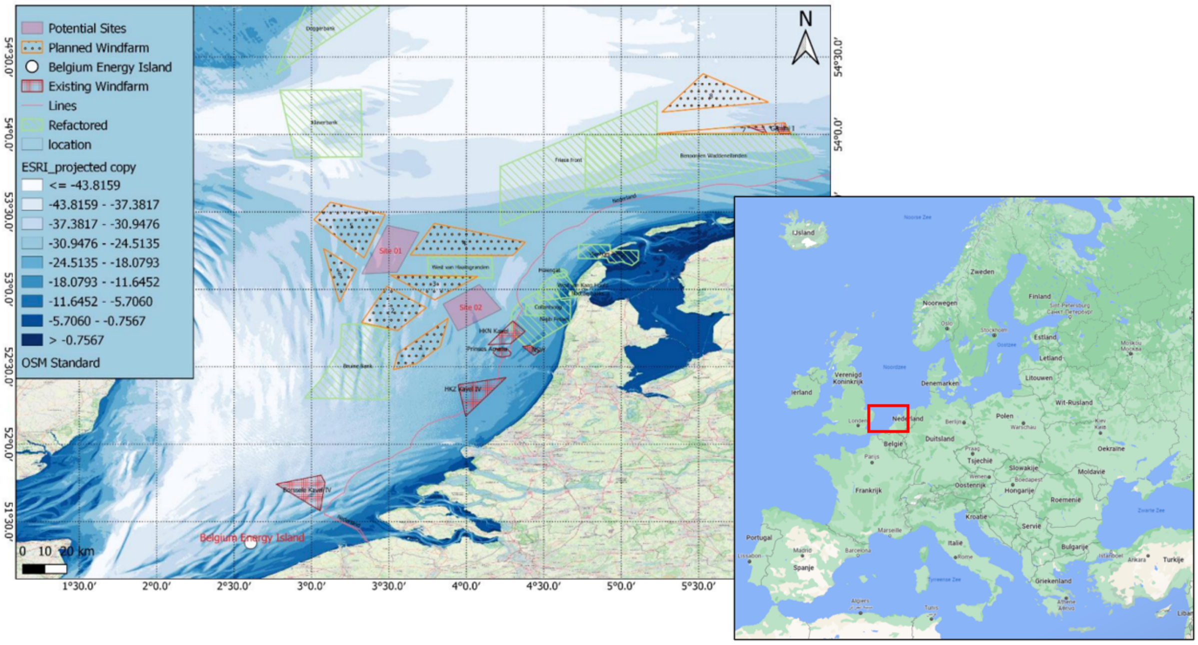


Figure 1.3: Site identification for potential locations for Project Alpheus, including the current and already planned infrastructure and windfarms.

1.1. Background

During the Industrial Revolution, around the 1850s, the utilization of oil rapidly increased due to the enormous energy potential characteristic of the fossil fuel. As a result, numerous inventions were made that changed the world, among them the combustion engine which offered limitless options for the world: since then globalization took a flight. One of the effects of burning fossil fuels is the emission of greenhouse gasses like carbon dioxide (CO_2), nitrous oxide (N_20) or methane (CH_4). The rapid increase of emission of CO_2 due to the use of fossil fuels is shown in Figure 1.4. In this figure the economical growth during the post-war period (around 1950s) can be seen clearly as the steady linear increase in emission.

An important effect of the emission of Greenhouse gasses (GHG) is the so called Greenhouse effect: the presence of the Greenhouse gasses act like a blanket for the earth, incoming sunlight is absorbed and stored, heating up the entire earth. In Figure 1.5 the global temperature since the pre-Industrial conditions is shown. Remarkable is that the period in which the rate of change in emission of GHG took a flight (around 1970s, Figure 1.4) the global temperature subsequently increased, as can be seen in Figure 1.5. Subsequently, since the pre-Industrial period the global temperature has already increased with 1.1 °C (IPCC, 2021).

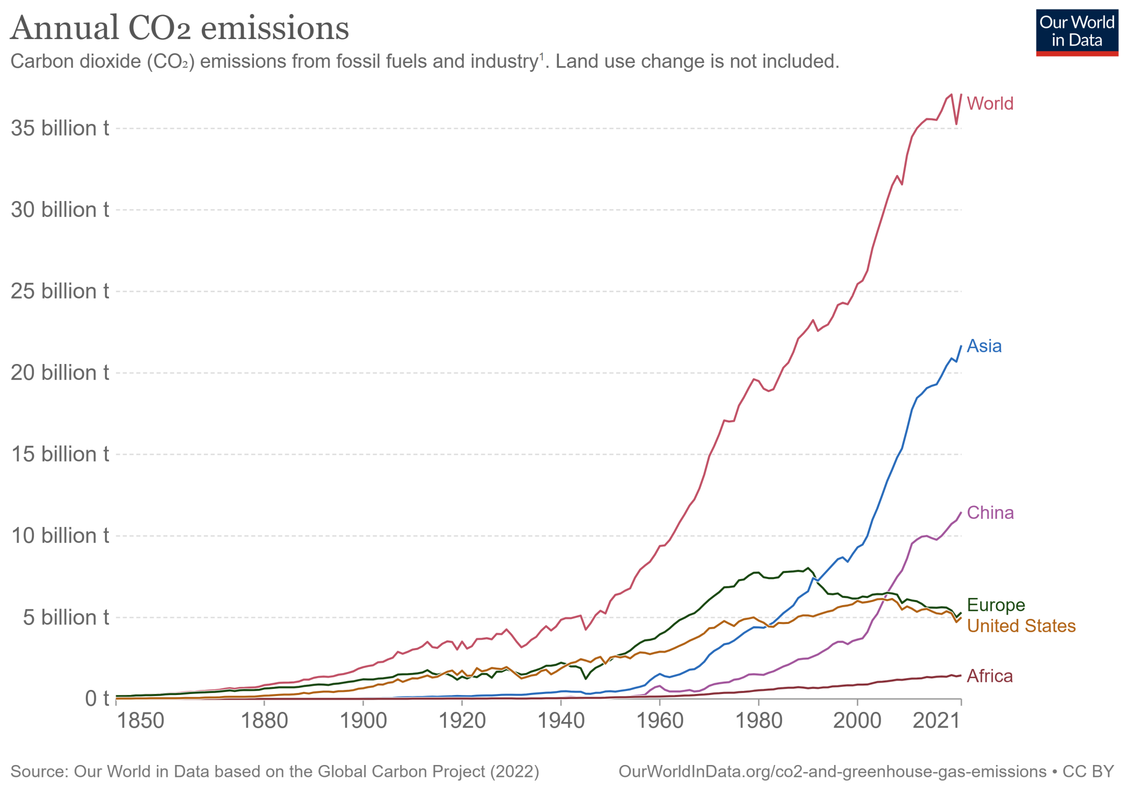


Figure 1.4: Annual emission of carbon dioxide (CO_2) per region/country over time. From Ritchie (2022).

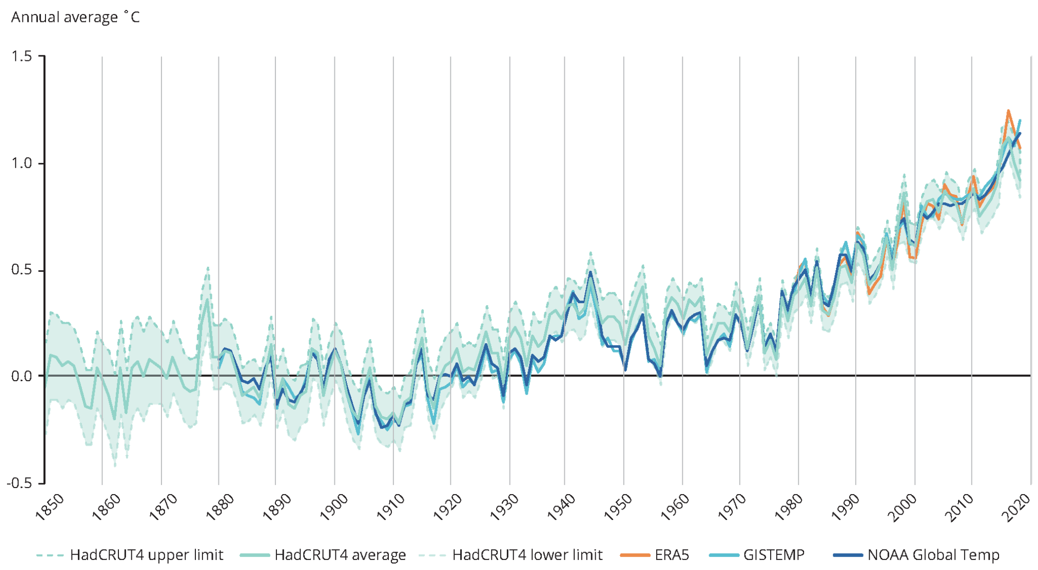


Figure 1.5: Time series of the annual average global temperature since pre-Industrial period. From European Environment Agency (EEA).

Due to the rising temperatures many global systems and natural processes are distorted (United Nations, 2022), which entail an increase in occurrence or severeness of events such as droughts and heat waves, changes in precipitation patterns, warming and rising ocean, loss of species, lack of food, health risks and poverty and displacements (United Nations, 2022; NASA, 2022). In addition, it is expected that several tipping points will be encountered, implying that a certain system balances between survival and total failure. An example of these tipping points is the existence of Amazon rain-forest or the melting of the polar ice sheets (United In Science, 2019).

One of the measures to prevent the worst to occur or at least to reduce the severeness of the consequences, several treaties have been signed with main goal to slow down the heating of our earth and minimize the maximum global temperature increase. For instance, the Paris Climate Agreement, in which 194 countries (including all leading countries) have agreed upon the threatening consequences of climate change and signed the biggest climate agreements in 2015 (UNFCCC, 2022; United Nations, 2015). The overreaching goal of this multidisciplinary agreement is to hold "the increase in the global average temperature to well below 2°C above pre-industrial levels" and pursue efforts "to limit the temperature increase to 1.5°C above pre-industrial levels." Exceeding the threshold of 1.5°C will unleash far more severe climate change impacts, including more frequent and severe droughts, heatwaves and rainfall (UNFCCC, 2022).

One of the milestones stated in the Paris Climate Agreement is reducing greenhouse gas emissions, in order to reach the peak of emission before 2025 at least and a total decline of emission of 43% by 2030. As such, the urge to abandon fossil energy resources and utilize renewable alternatives grow rapidly as the window of opportunity gets smaller. Hence, the transition from fossil fuels to renewables is more relevant than it was ever before: implementation of organic solar panels into our daily life (King, 2020), constructing the world's largest wind turbine (H260-18MW) (Blain, 2023) and the rapid increase in solar energy production all contributes to the reduction of the emission of GHG.

Although the energy transition can be considered as a necessary and critical development, due to the daily variations in sun and wind energy the reliance of the current renewable energy is limited. Looking into the energy demand and supply of Germany, shown in Figure 1.6, this problem becomes clear. During the intermittent nature of renewable energy, the energy supply mainly originates from fossil fuels like coal (brown and hard) and gas. For instance, during sun set when solar energy yield decreases, fossil energy plants will be activated. In terms of the clean energy transition and the deep decarbonization, the ability to overcome the intermittency of renewables is one of the hurdles (Schill, 2020).

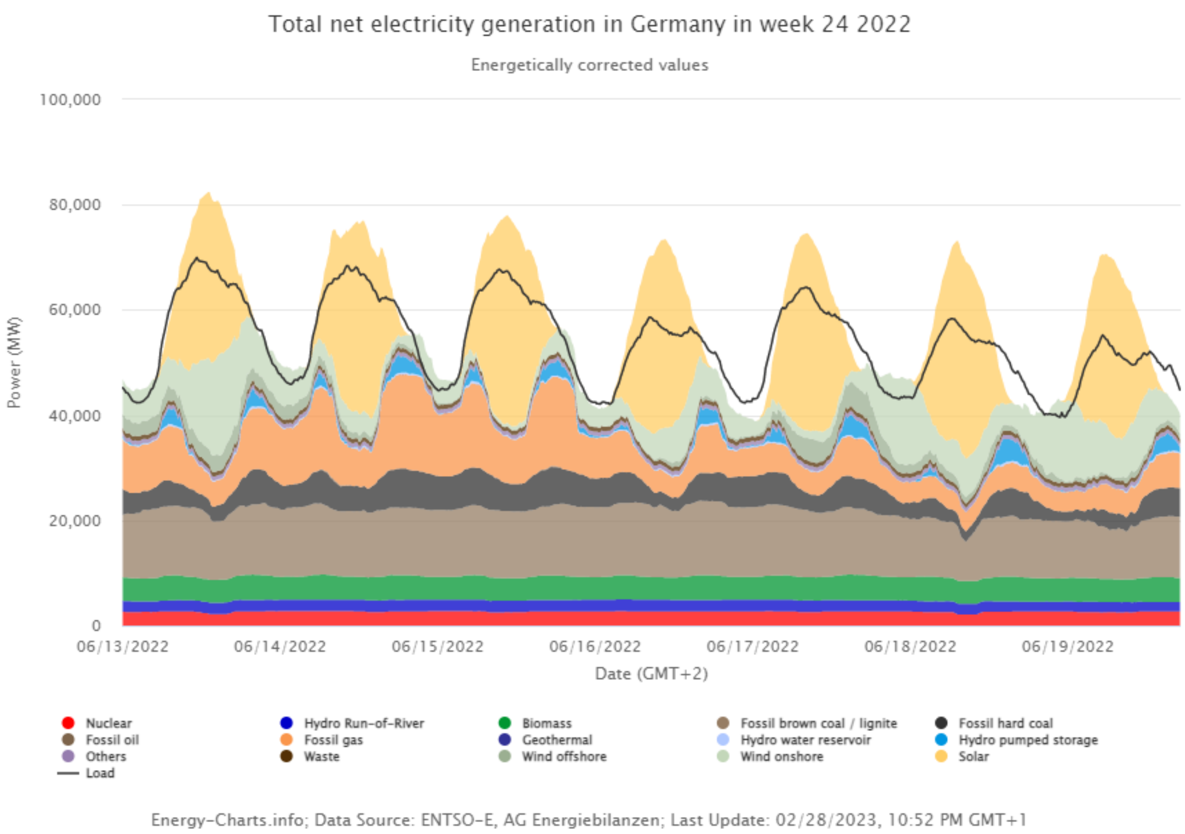


Figure 1.6: Energy demand and various supplies of one week in Germany during summer. From *Energy-Charts*

One solution for the intermittency of renewable energy resources might be the application of bulk energy storage: during energy abundant periods energy can be stored temporarily and can later be utilized during energy absent periods, Nowadays, the options for energy storage are limited mainly to three types: i). batteries, ii). hydrogen storage and iii). pumped storage hydropower (PSH).

Batteries play a major role in the energy transition, as electricity can nowadays be used to substitute fossil fuels, for instance in the transportation sector. However, when we look into the capabilities for bulk energy storage, which basically means the supply of electricity for thousands of households during a longer period, the battery turns out to be relatively expensive. According to a cost analysis of Zakeri and Syri (2015), it turns out that bulk energy storage with batteries is 2-3x as expensive compared to the pumped storage hydropower (PSH). An overview of costs per storage type is shown in Figure 1.7. Since the batteries in general are relatively expensive, this is not the most desired solution regarding bulk energy storage.

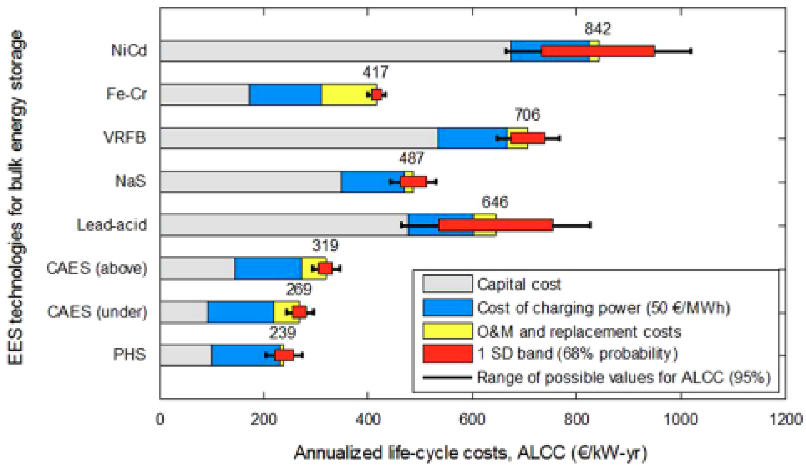


Figure 1.7: The annualized life cycle costs (ALCC) of EES systems in bulk energy storage and related uncertainties, considering 250 cycles per year, 8% interest rate and 8 h discharge time. The average values are shown above each bar. After Zakeri and Syri (2015).

Although the potentials of hydrogen storage are great, one of the main disadvantages of hydrogen energy storage is the relatively low efficiency: according to Yue et al. (2021) the round-trip efficiency of hydrogen energy storage is roughly 30-50%. As such, this alternative requires an abundance of energy to overcome this low efficiency, and therefore this would not be an optimal storage considering the short term innovations.

The other alternative is pumped storage hydropower (PSH), which has already been applied in mountainous terrain like the Alps or parts of Norway for many years. The working principle of a PSH is during periods of energy abundance water is pumped into a reservoir upstream via turbines, and subsequently during periods of energy deficits the water will generate energy by flowing downstream, turning turbines that will generate energy. With a approximate cycle efficiency of 80% this would be an economical viable business case and an effective measure in terms of energy storage.

However, due to the lack of mountainous terrain in the Netherlands a regular PSH is not feasible, as a head difference of 100-200m is considered to be perfect for this option. During the 1980s, the Dutch engineer L.W. Lievense came up with *Plan Lievense*: a conceptual design for a PSH that could be constructed in flat countries like the Netherlands. Partly based on that idea the European funded Project Alpheus was established. So far insufficient research into offshore energy islands has been done to get a good understanding of the construction of such a mega project.

To that end, this research focuses on an offshore pumped hydroelectric energy storage which will be constructed in the North Sea, providing bulk energy storage for the adjacent wind farms. This could be a solution to overcome the daily variation and the intermittency of the wind and solar energy, providing electricity to millions of households during low production periods, e.g. during the night, cloudy or windless days.

1.2. Problem analysis & statement

Although pumped hydroelectric energy storage has been around for many years, Project Alpheus is a revolutionary initiative in terms of energy storage and hence many topics have been described concise and on a fairly low level of detail. Especially due the size of the project many unforeseen consequences increases the complexity substantially: since the project can not be finished within one working season, the seasonal variability must be taken into account as some sections of the project are fully exposed to storm conditions (yearly storm conditions during winter in the North Sea imply waves with a significant wave height of $H_s \approx 6$ m). As a result, due to the harsh and relatively unpredictable environment in combination with strict workability limits the current planning is vague and unsophisticated.

The construction sequence of similar projects (e.g. the Belgium energy island ELIA) consists of many interconnected steps, each with their own dedicated equipment restricted by their workability limits and wave conditions. This implies that when work A is delayed due to exceedance of the workability limits, the execution of work B must be postponed, delaying the entire sequence. An example related to this project could be as follows: if the dredging activities for the caisson foundation are delayed, the placement of the caisson must be postponed, subsequently delaying also the placement of the scour protection and inner-berm. So, the high dependency of the construction sequence in combination with the unpredictable harsh offshore environment, makes it very complicated to make a sophisticated planning.

In order to improve the current knowledge and contribute to this topic, this thesis focuses on the logistic, planning and cost-effectiveness aspects of an offshore PSH and answers the question:

"To what extent can the construction costs be optimized by alternating the construction phasing of the caisson dam, not considering the powerhouse and related infrastructure?"

In order to answer the main research question, this thesis will address the following sub-questions:

1. What are the most critical/sensitive phases in the construction of the caisson dam, considering a deterministic approach?
2. How does the planning change using a probabilistic approach, including wave data statistics and workability limits of equipment?
3. How can the construction phasing and choice of equipment be optimised to minimize the project costs?
4. To what extent are sheltering effects due to the partly constructed/finished dam relevant for the remaining part, and what are the effects on the construction costs?

1.3. Approach

The objectives described above will be achieved by following the approach visualised in Figure 1.8, which is further elaborated below.

1. **Preliminary design of the caisson unit and stability check of rock works**, based on the hydraulic boundary conditions such as the Extreme Value Analysis (EVA) of the wave height to determine return values, tidal levels, expected sea level rise and storm surge levels. This includes almost all stability analyses regarding the caisson unit described in Voorendt et al. (2020), Voorendt and Molenaar (2021) and ROM (2006). Moreover, this also includes the stability check of some rock layers during high tidal currents and wave attack.
2. **Determine a first construction phasing of the caisson dam**, based on the wave statistics in the North-Sea, (i.e. considering the governing wave direction, this would probably be the location where the construction will start), Based on the preliminary wave statistics, the order of construction will follow from the governing wave conditions. This planning will form the baseline scenario for the rest of the research; any beneficial adjustment regarding a change in construction sequence will be calculated with respect to this plan.
3. **Make a first deterministic planning**. Inventory the construction phases/actions and volumes, i.e. which steps are necessary for the construction of the caisson dam and how much material is needed, for instance leveling dredging of the sea bed, apply foundation of caisson, ... After that, gather information about the required equipment for construction that are suitable for the job, with the volumes, the capacity and production rates of the vessels, a first planning can be made.

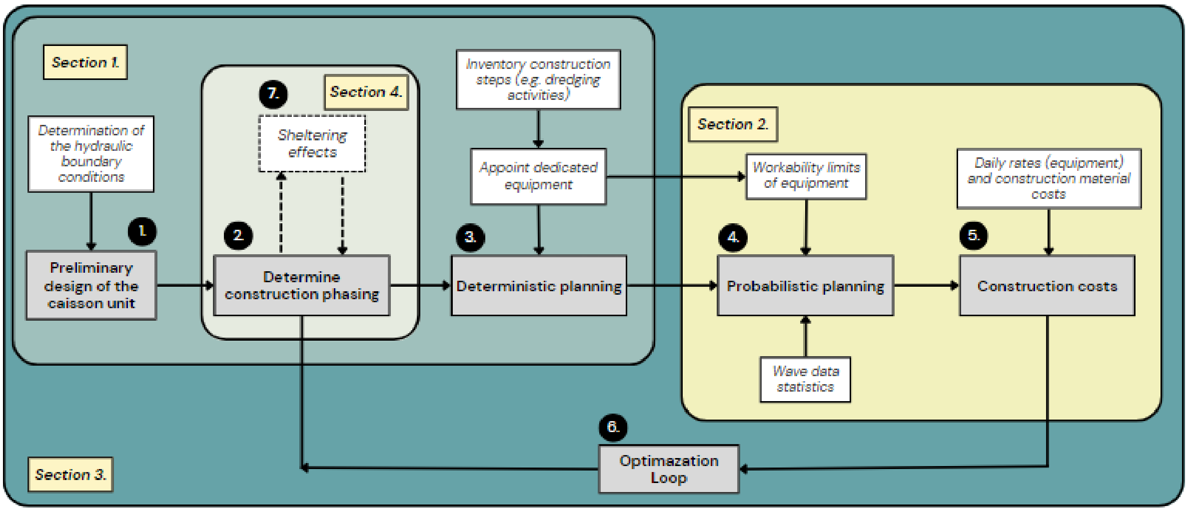


Figure 1.8: Visualisation of the thesis approach, with the distinct sections indicated by the coloured rectangles, their relevant topics and the necessary input.

4. **Make a probabilistic planning**. Include the vessel information (i.e. workability), apply probabilistic wave (and wind) conditions data to find a probabilistic project duration and obtain e.g. information about the deployment per vessel. The probabilistic planning is based on wave data retrieved from an observation platform in the North-Sea, which must be close to the potential project site. In general, waves are heterogeneous and hence can not be simply extrapolated from the point of measurement to the point of interest. Therefore, the wave buoy of observation platform must be located close to the project site, to exclude additional uncertainty regarding homogeneity of waves.
5. **Incorporate costs** With the probabilistic planning the deployment per vessel is known. Combined with the daily rate of the vessels and costs for construction materials per volume, an estimate for the construction costs can be made.
6. **Optimize the construction costs** By tweaking the construction phasing and/or work method of the caisson dam, i.e. different deployment of the vessels, different sequence, construction costs can be minimized. Early access to the facility by finishing the structure sooner than the baseline scenario, will also be taken into account for calculating the project costs. The earlier utilization of the facility means that production can also commence earlier, starting the payback period sooner, As such, this will have a positive effect on the overall cost-benefit analysis of the project. The Levelised Cost of Storage (LCOS) is a financial assessment metric that will be used to determine the cost-effectiveness.
7. **To what extent does sheltering effects of the finished dam play a role in the project**, and can it be used in the construction phasing to reduce the costs of the project? The sheltering effects due to the (partly) constructed caisson dam will influence the wave conditions on the leeward area of the structure, reducing the transmitted wave height and hence improving the workability. As it is expected that this would have a huge impact on the overall project duration, this will be one of the important topics of this research. Possible methods to include the sheltering effects are: Goda's diffraction figures and wave simulating software (SWAN).

To gain insights in the methodology of this thesis, a schematic overview of the approach is shown in Figure 1.8. In this figure the aspects that are relevant to answer the main question of this thesis are indicated by the numbers in the black circles and the necessary input to determine these aspects are shown in the smaller white boxes. Furthermore, the big coloured rectangles coincide with the sub-questions that must be answered.

Report outline

First, in Section 2 an extensive site description will be given for the potential project sites. This includes many crucial facets that determine the success of the project, among them are a geo-technical and geographical analysis for the desired location of the Pumped Storage Hydropower (PSH) plant. Furthermore, the hydraulic and geo-technical design conditions will be determined. This refers to the design water level, currents and wave heights.

Section 3 will discuss the design in detail, starting with the design of the rock works that provides protection to the structure and the caisson unit. For both elements the failure mechanisms will be inventoried and briefly elaborated. From this list, the most governing or relevant failure mechanisms for each element will be chosen and worked out in detail, resulting in stability checks. In case the element turns out to be insufficient for the design conditions, an improvement of the design will be made in order to ensure overall stability for the entire lifespan of the structure.

In Section 4 a deterministic planning will be made based on a simplified construction phasing, neglecting wave conditions. First, the work method must be determined for the construction of the caisson dam. Such a work method contains all construction works, estimates for all volumes and type of equipment that must be deployed for the corresponding sequence. After this has been inventoried, a deterministic planning will be made based on the chronologically of the actions. During this step, wave conditions will be neglected, resulting in a more favorable planning than expected.

Section 5 will discuss the probabilistic planning, which improves the deterministic planning by including the hindcast wave data. By considering wave data that corresponds to an exceedance probability of 70%, a more realistic planning can be made. This includes the wave limiting threshold of the dedicated equipment (Appendix D and E). Among other results, the probabilistic planning will result in a distribution of deployment per vessel, and more importantly a comprehensive probabilistic construction planning.

Section 6 will address the cost-effectiveness of the project, by taking into account the expenditures such as material costs, operational costs and risk costs. Various scenarios will be evaluated to gain insight in the sensitivity of multiple parameters. Moreover, the cost-effectiveness metric, the Levelised Cost of Storage, will be compared to existing energy storage technologies.

Section 7 discusses the sheltering effects and corresponding optimization process with regard to the work method and phasing. This refers to including hydraulic physics such as wave diffraction, which improves local wave conditions. This section will address to what extent this affects the cost-effectiveness of the project.

Section 8 thoroughly elaborates the results, findings and limitation that have been faced throughout this research. Subsequently, the main conclusions and research questions will be discussed in Section 9. Finally, Section 10 highlights shortcomings and corresponding recommendations for further research regarding the construction of an offshore energy island.

2 Site description

Prior to the main research, it is important to compose an extensive site description. This refers to the geographical location, geo-technical analysis and hydraulic boundary conditions. In this section the most relevant facets with regard to the project site will be discussed. The main goal of this section is to obtain a location for the facility that meets all requirements and conditions. The second goal is to determine the hydraulic boundary conditions that will be the design conditions for the caisson unit and rock works.

The most sophisticated existing map - shown in Figure 1.3 - was at a low level of detail and did not contain all the relevant details when considering the construction of the dam. For instance, it only showed the potential sites for the offshore PSH, nature conservation areas and wind farms, both existing and planned. In this section, a high detailed study will be dedicated to investigate the potential sites, based on the already existing map, adding parameters such as maritime traffic and infrastructure (pipelines, power cables and telecom cables) but also incorporate existing knowledge of van der Wel (2021).

2.1. Geo-technical analysis

Soil characteristics

The most strict requirements for the location is the expected head difference between the ocean and the reservoir. This depends on the soil characteristics under the reservoir: the thicker the clay layer, the greater the water level difference can be. In Appendix A a full analysis is presented, which includes the background, literature research and the results. In the following paragraph only a concise summary will be given.

According to the assessment, multiple locations in the North Sea are suitable for a PSH plant (with achievable water level differences of >30 m). The conclusions that have been drawn: i). for Site 02, proposed by Project Alpheus, sufficient head difference can be achieved but lacks space for the desired PSH, ii). a new location near Texel has been found which is enriched with thick clay layers, allowing for large head differences.

Bathymetry

Typical sand dunes in the North-Sea can reach an absolute height difference of 40 m, depending on the exact location. Using the EMODnet Map Viewer of the European Marine Observation and Data Network (2023), the exact bathymetry can be analysed, which will lead to potential areas for the construction of the PSH. For the desired location nearby Texel, the sea bed is, in absence of any major sand dunes, relatively flat. This can also been seen in Figure A.6, where the bathymetry of the structure's perimeter is plotted with respect to mean sea level, starting at the northern part of the structure. The lowest elevation in the perimeter is MSL -29 m.

Lithography

The lithography of the desired location is one of the leading aspects for the site investigation, as it was the driving parameter behind the potential locations: which location contains a sufficient impermeable layer that can withstand a significant head difference? The results of van der Wel (2021) have been leading in this assessment, as it provided a clear overview of the possible locations. After the location has been chosen, it is important to know what the lithography of the proposed location is, also to take the subsoil into account during the design phase.

2.2. Existing infrastructure

One of the criteria for a successful location is that existing infrastructure can not interfere with the proposed location. Two types will be studied: i). maritime shipping routes and ii). power cables and pipelines. An overview of the North Sea is shown in Figure 2.1.

Maritime shipping routes

Maritime routes are essential areas to ensure safe passage for vessels, avoiding physical constrains such as coasts, reefs and man-made structures like wind farms. Part of the global infrastructure, maritime shipping routes can not be changed easily. Therefore, the location can not be located in a shipping route. Appendix A analyses all existing routes in the North Sea thoroughly. The conclusions that can be drawn based on this are: i). the proposed Site 01 interferes with the shipping route, ii). Site 02 only partly lies inside the shipping route and iii). the new proposed location near Texel does not overlap with routes.

Power cables, pipelines and wind farms

Power cables, pipelines and wind farms may hinder the construction of the facility or even increase the risk of failure. Therefore, it is worth to study the existing and planned infrastructure. An extensive overview of the infrastructure is shown in Figure 2.1 and is analysed further in Appendix A.

The conclusion that follows from this assessment: the proposed location near Texel interferes with two decommissioned pipelines, which do not pose any risk on first sights. Since the pipelines are decommissioned, before to construction commences, they can be removed.

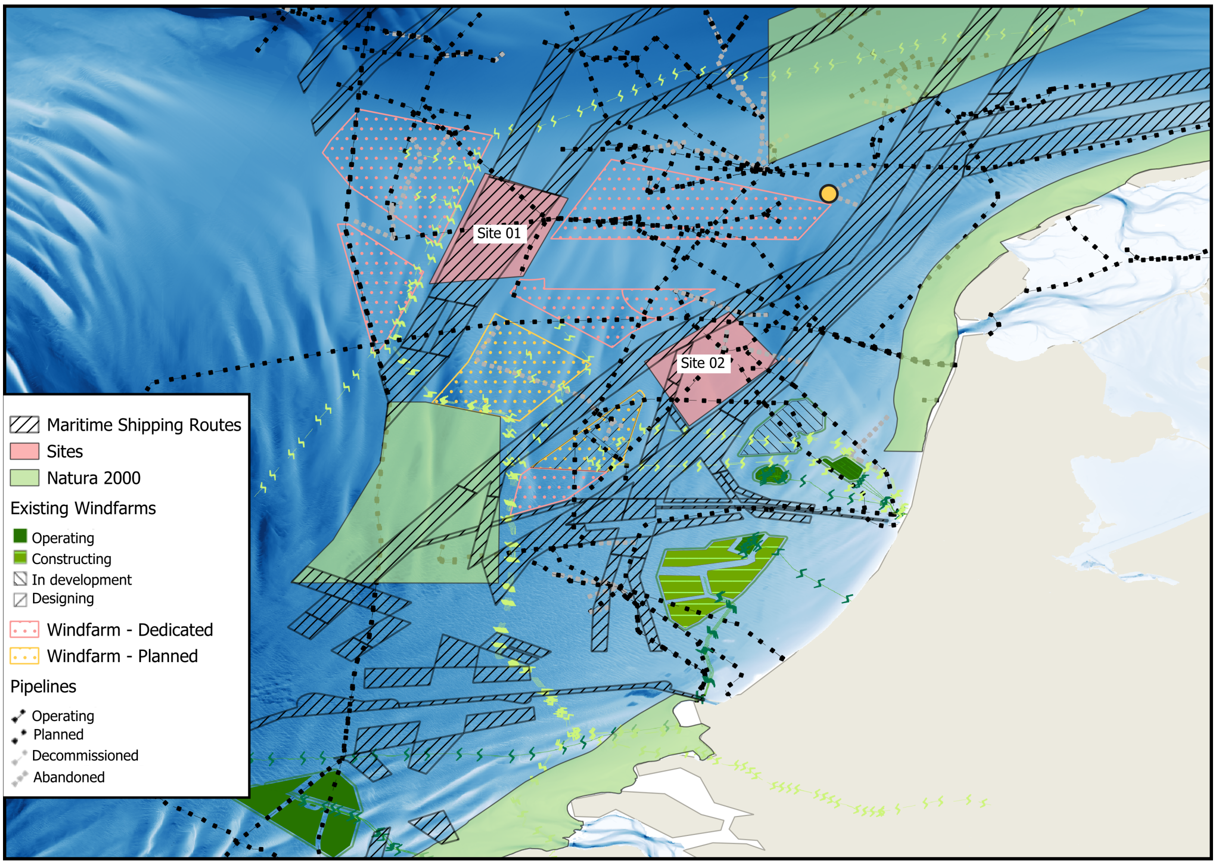


Figure 2.1: Map of the North Sea including maritime shipping routes, nature conservation areas, windfarms, proposed sites by Project Alpheus and existing infrastructure such as power- and pipelines. The newly proposed location in the research is 50 km off the coast of Texel (53.3356°N, 4.3771°E).

2.3. Hydraulic boundary conditions

Hydraulic boundary conditions are an essential aspect of any design phase. They include various parameters, among them are the wave characteristics, design water level and prevailing currents near the facility. An Extreme Value Analysis (EVA) will be conducted to determine the significant wave height

Figure 2.2: Omnidirectional windrose for the desired location, consisting of 42 years of data.

at the design water level, consisting of the storm surge level and sea level rise. Finally, the prevailing currents are essential with regard to the toe and scour protection, during the execution phase as well as at the final stage.

Wave data

The significant wave height plays a vital role in nearly every failure mechanism, for instance overtopping, shear-criterion or rational stability. Therefore, an Extreme Value Analysis must be executed to find the corresponding wave heights per directional spread.

In Appendix A an extensive determination of the hydraulic boundary conditions can be found. Here, only a concise overview will be provided. Starting with a brief methodology, followed by the results and the conclusion.

The Metocean Data Portal of (DHI, n.d.) contains over 40 years of hourly weather and hydraulic hindcast data. The desired location for the PSH plant is North West of Texel (shown in Figure 2.1), with the following coordinates: 53.3356°N, 4.3771°E. With a Peak-Over-Threshold (POT), the extreme values for the wave height can be obtained. Afterwards, a Generalized Pareto Distribution (GPD) has been fitted over the extreme values to find the return period and their values. Table 2.1 only shows the highest wave height per return period (neglecting the corresponding direction).

Table 2.1: Summary of the EVA using Generalized Pareto Distribution (GPD). Significant wave height is given for 95% confidence interval.

Return period [years]	H_s [m]
1	6.16
2	6.65
5	7.23
10	7.64
50	8.45
100	8.85
500	9.75

In addition to the EVA of the wave height, the wave direction has also been analysed and visualised. In Figure 2.2 the windrose that corresponds to the desired location is shown. From these results, two conclusions can be drawn. First, the dominant wave directions are North North West to North and South West. Furthermore, the highest waves originates from the region between North and South West direction.

Design water level
The design water level is an essential consideration during the design phase of hydraulic structures. It is defined as the governing water level that a structure will e exposed to during extreme weather conditions. For many design considerations, the design water level will be the maximum water level, however in some cases the minimum water level can be normative. Therefore, one must be sure to include all possible hydraulic combinations, i.e. design water level and wave height, in the design phase.

The design water level consists of many unfavorable conditions that occur simultaneously, among them are the astronomical tide, wind effects (setup or set-down) and atmospheric pressure effects, while taking into account the possible effects of local sea level rise.

Sea level rise One of the consequences of the rising global temperature is the increase of the average sea level, due to the melting ice caps and other polar regions. The emission of greenhouse gasses contributes to this change, which makes the prediction of sea level rise in the future complicated: to what extent will the emission decrease, and how will the global system react to this change.

Climate models of Intergovernmental Panel on Climate Change (IPCC) use several input parameters, among them the emission of greenhouse gasses, are able to predict the evolution of the climate, including the sea level rise. The newest report, from December 2022, shows the most up-to-date results for three scenarios that represent the change in greenhouse gas emission, SSP5 8.5 considers an increase emission whereas SSP1 2.6 considers a decrease of the greenhouse gas emission.

In general, the results of IPCC (2021) are global results. Due to the spatial effects regarding sea level rise, the site specific sea level rise may differ from the global sea level rise. To that end, KNMI (n.d.) have made predictions for the Netherlands based on the latter report until 2100. Since for this study the local sea level rise must be known for the year 2150, linear extrapolation from 2100 will be applied for both the mean and confidence interval values. The results are shown in Table 2.2.

Year	Mean Sea level [cm NAP]	Lower bound 95%	Upper bound 95%
2000	5	-	-
2020	12	-	-
2050	32	20	51
2100	84	58	125
2150	136	96	199

Table 2.2: Observed sea level (2020) and expected sea level rise in cm according to scenario SSP5 8.5 (IPCC, 2021). Sea levels for 2150, including the confidence interval, follow directly from linear extrapolation.

Storm surge levels Storm surges are defined as coastal flood phenomena of rising water consisting of the highest astronomical tide and wind setup in combination with low-pressure weather systems. The severity of the storm surge levels depend, similar to other hydrodynamic processes, on the return period. According to the EVA of the MetOcean Data Portal of DHI (n.d.) for the dedicated location of the PSH plant (53.3356^{o}N, 4.3771^{o}E), the estimated storm surge levels are presented in Table 2.3.

Variable	Extreme value (omni) - Return period [year]						
	1	2	5	10	50	100	1000
Storm surge (H) [mMSL]	2.0	2.2	2.3	2.5	2.7	2.8	3.2
Storm surge (L) [mMSL]	-1.6	-1.7	-1.8	-1.9	-2.1	-2.2	-2.5
Residual, high [m]	1.5	1.7	1.9	2.0	2.3	2.4	2.8
Residual, low [m]	-0.9	-1.0	-1.2	-1.3	-1.4	-1.5	-1.8

Table 2.3: Return values for the storm surge, set down and the residual influence.

Currents
Currents can cause excessive erosion of the sea bed, leading to the destabilization of bed protections such as the rock foundation layer, or even at the bigger scour protections. Hence, accurate knowledge of the

prevailing currents in the North Sea is necessary to develop effective bed protections.

The damage mechanism where currents cause excessive erosion of the sea bed is not considered in this research, as it requires an in-depth study of the structure. For this research the destabilization of a single grain/rock embedded within the protection layer is studied.

To that end, an EVA of the depth-averaged current velocity has been obtained from the MetOcean Data Portal of DHI (n.d.) for desired location (53.336^{o}N, 4.3771^{o}E). This analysis is shown in Table 2.4, where the current speed is listed for several return values.

Variable	Extreme value (omni) - Return period [year]						
	1	2	5	10	50	100	1000
Current speed, depth-averaged [m/s]	1.1	1.1	1.1	1.2	1.2	1.3	1.4

Table 2.4: Return values for the depth-averaged current speed [m/s].

2.4. Wind data

Wind data plays a vital role in the design and planning for an offshore structure, especially when executing complex maneuvers like caisson installation or lifting heavy and large objects like a wave return wall. Moreover, considering the design of the structure, the stone revetment constructed on the inner berm (to prevent excessive erosion of the berm) will be subjected to wind waves that originate from the reservoir. In Appendix A a wind analysis for various return events has been presented.

2.5. Design return periods

During the first phase of the project the structure's elements will be designed based on the governing design conditions. The robustness of those elements, in turn, depend on the consequences and its severity when the structure would fail, which is often expressed in a certain return period. The corresponding return event can be derived by applying the Poisson equation in combination with the known parameters such as failure probability and the lifetime.

The full determination of the return periods, depending on the lifetime, is presented in Appendix A. The structure is considered as a breakwater which requirements are given by ROM (2006). In summary, three distinct return periods have been determined. Table 2.5 lists the lifetime, failure probability and phase for the considered scenarios.

Phase	Lifetime	Return period	Example
Execution	1 year	5 years	Temporary situation for rock works or unsupported caisson unit.
Execution	5 years	25 years	Longer temporary situation for the inner berm.
Operational	100 years	500 years	Structure's lifetime; caisson design.

Table 2.5: Summary of various return periods for execution and operational phases. The failure probability of 20% corresponds to a breakwater case, according to ROM (2006).

2.6. Conclusion

The aim of this section was to determine the desired location for the offshore PSH, taking into account geo-technecial requirements, maritime shipping routes, existing and planned wind parks, power cables and pipelines and the bathymetry.

The two proposed locations, which followed from previous studies, have been assessed based on these criteria. It turned out that the locations were not suitable due to the overlap with the maritime shipping routes and the limited space and subsequently have been rejected in this research.

To that end, a new location had to be found according to the requirements. The outcome of this analysis is a location approximately 40 km north-west of Texel (one of the northern Dutch islands), which contains excellent soil and does not coincide with other land- or sea-usages. The corresponding coordinates of this location are 53.3356°N, 4.3771°E.

The second goal of this section was to determine the relevant hydraulic boundary conditions. Various return events have been defined corresponding to certain phases in the construction period. A brief summary of this section has been visualised in Figure 2.3, including the hydraulic boundary conditions, preliminary design of the structure and soil properties of the proposed location.

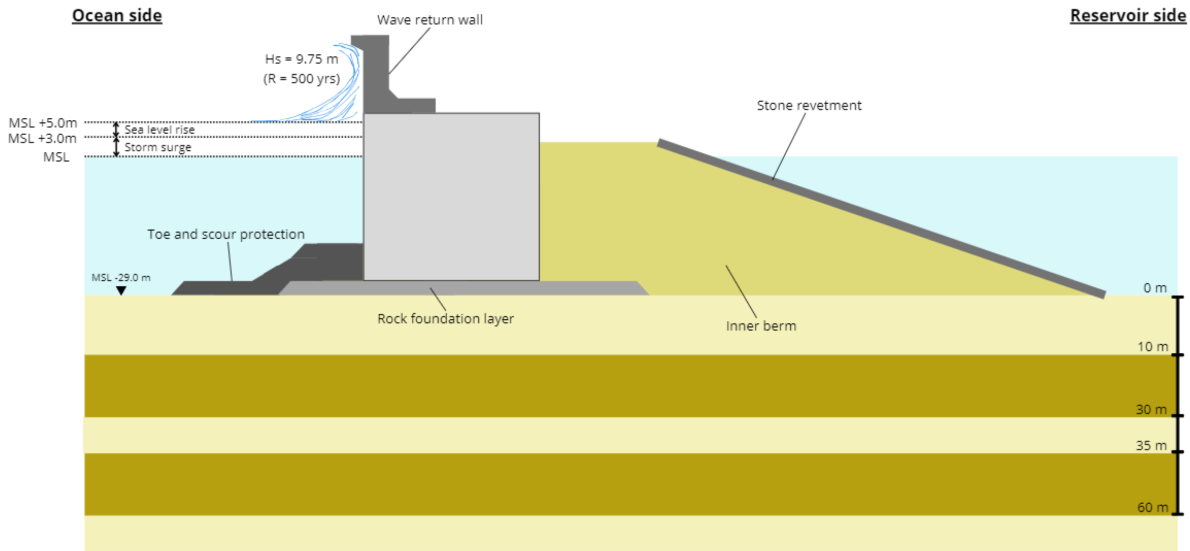


Figure 2.3: Cross section overview of the hydraulic boundary conditions, geo-technical properties of the proposed location and the conceptual design of the structure. The yellow layers represent sandy material, the brown layers represent clay.

3

Structural assessment

Structural assessment is an important process of the design and operation of structures in general. It evaluates the strength and robustness of the structure and its elements, considering several load combinations that are specifically defined per situation. In this section, the design of the entire caisson dam will be discussed in detail and follows entirely from design rules that are sufficient for preliminary designs. In addition, the relevant failure mechanisms will be discussed extensively per work sequence and subsequently also the related stability checks will be executed to assess the safety of the construction during and after construction.

3.1. Design

In general, the design of the caisson unit mainly depends on the local water depth of the dedicated construction site in combination with the design water level, and must meet the requirement that the top of the unit must be above the water surface. Therefore, the height will be the starting point for the design phase, after which, with some rules of thumb, the width and length will be determined. However, these are just an indication and can be adjusted to the governing stability checks in a later stage.

According to the site investigation of Section 2, the maximum elevation of the sea bed of the desired location is located at MSL -29.0 m, on which a rock foundation layer with a height of 2.5 m will be installed. The bottom of the caisson will therefore be placed at MSL -26.5 m. The crest of the unit (excluding the additional height of the required wave return wall) must be located above the water surface during design conditions, taking into account sea level rise. The design water level in 100 years - which consists of the estimated sea level rise in 100 years (2.0 m) in addition to the storm surge level with a return period of 500 years (approximately 3.2 m) - is roughly at MSL +5.2 m, resulting in a required height of 32 m.

According to Voorendt and Molenaar (2021), the length can be estimated by 2 to 3 times the height. Taking into account the production limits of the caisson fabrication facility, the length of the caisson is assumed to be 60 m, with a width of 34 m resulting in the following dimensions: 60 x 32 x 34 m (l x h x w). A cross section view of the dam is shown in Figure 3.1, including the necessary scour protection layers, rock foundation layer and inner berm which are shown in Table 3.1.

Layer	Grading	$d_{50}[cm]$	d_{n50} [cm]	Legend
Rock Foundation	45 - 180 mm	6.3 - 9.0	6.4	
Scour filter A	45 - 180 mm	6.3 - 9.0	6.4	
Filter (inner)	60 - 300 kg	42 - 49	38	
Scour Filter B	60 - 300 kg	42 - 49	38	
Scour Filter C	300 - 1000 kg	65 - 75	59	
Scour protection	3 - 6 ton	168 - 144	118	
Toe filter	45 - 180 mm	6.3 - 9.0	6.4	
Toe protection	300 - 1000 kg	65 - 75	59	
Inner berm	Dredged material			

Table 3.1: Volume estimations for the scour protection and filter layers.

Failure mechanisms

Failure mechanisms involve the processes by which physical forces and stresses cause an element or the entire structure to stop function as intended. Important is that failure mechanisms depend on the type of

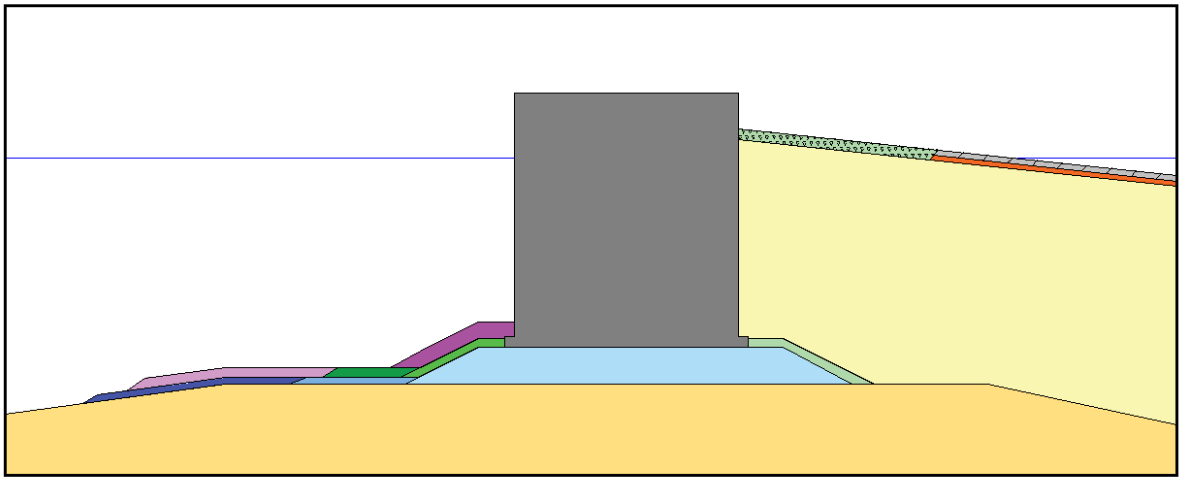


Figure 3.1: Design of the caisson elements and all related protection layers.

loading and hence the way of failing. Considering the PSH plant, two distinct failure mechanisms can be distinguished: failure of the caisson unit and failure of the rock works. The failure description for rock works is the full mobilization of a rock grain according to Pilarczyk, similar to the design principles for bed, bank and shore protections. Regarding the failure mechanisms for the caisson unit, failure is considered when the structure can exceeds a load limit beyond which the structure is no longer able to fulfil the relevant design criteria. For instance, this could be an exceedance of overtopping of the wave return wall, resulting in erosion of the inner berm.

3.2. Theory - Rock works

The stability of the rocks is determined by applying the method of Pilarczyk, which has been widely applied for bed, bank and shore protections. Pilarczyk's formula is derived for many disciplines and includes parameters such as type of construction, Shields parameter, flow turbulence and water depth, and is shown in Equation 3.1.

$$\Delta d = 0.035 \frac{\Phi}{\Psi} \frac{K_t K_h}{K_s} \frac{u^2}{2g} \tag{3.1}$$

in which:

d [m] = Grain diameter
Δ = relative density, $\Delta = (\rho_s - \rho_w)/\rho_w$
Φ = stability parameter (Φ = 1.25 at the end of a single layer of rock, 1.0 at the end of a bed protection, 0.75 for a two layer rock layer, 0.50 for a continuous bed protection),
Ψ = Shields parameter ($\Psi = 0.035$ for dumped rock and 0.05 for placed rock),
K_t = turbulence factor (0.67 for low-turbulent flow, 1.0 for normal turbulence, 1.5 for river bends, 2.0 - 2.5 in sharp river bends, 3.0 for high turbulence flow (e.g. propeller race) and 4.0 for extreme high turbulence)
K_h = depth factor (for a fully developed velocity profile: $K_h = 2/\log(12h/Nd)^2$, where h is the water depth [m], N is 1 to 3 and d is the grain diameter [m])
K_s = steepness factor, given by Figure 3.2
u [m/s] = flow velocity

Due to the fact that the Pilarczyk formula depends on the depth factor, which contains the grain diameter, the expression becomes an implicit formula and should therefore be solved iterative.

3.3. Theory - Caisson unit

The following Section will address the relevant mechanisms for the caisson unit, among them are the internal stability of the unit, the interaction between structure and subsoil and macro slope failure. However, this section is considered to check the stability in a simplified 'hand calculation' manner, only

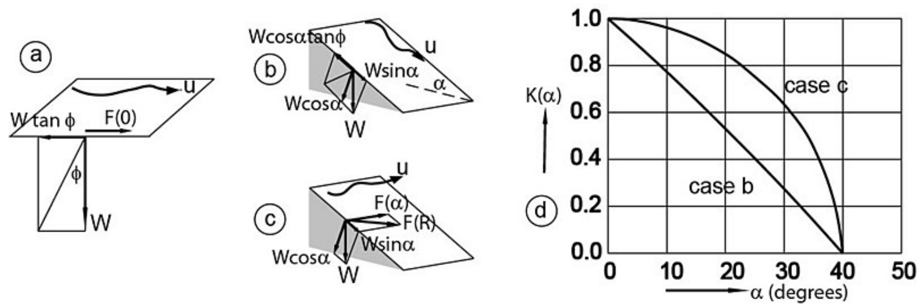


Figure 3.2: Definition of the steepness parameter K_s.

to determine the safety of the preliminary design of the caisson unit and will not go into detail of the caisson design and some failure mechanisms. For example, in general all caisson units are fabricated with partition walls to improve the structural strength and characteristics of the element. However, in this section the caisson is simplified to a basic hollow element. Despite the fact that this will affect the outcome of the stability checks, the scope of this research is not the design optimization of the caisson unit, but optimization of the construction sequence.

As described in Section 3.1, as a starting point the preliminary dimensions of the caisson unit are 60 x 32 x 34 m (length x height x width). The dimension are shown in Table 3.2. After the stability analysis, the dimensions can be adjusted to improve either the stability or material use.

Table 3.2: Characteristics of the caisson unit.

Parameter	Symbol	Value
Length	l	60 m
Width	w	34 m
Height	h	32 m
Wall thickness	t_w	1 m
Floor thickness	t_b	1 m
Crest thickness	t_c	1 m
Draught	d	17 m
Weight	W	19 500 ton

Table 3.3: Soil characteristics and material properties.

Parameter	Symbol	Value
Specific weight water	γ_w	10.05 kN/m^3
Specific weight concrete	γ_c	25.0 kN/m^3
Specific weight sand (unsat.)	$\gamma_{s,unsat}$	14.7 kN/m^3
Specific weight sand (sat.)	$\gamma_{s,sat}$	17.7 kN/m^3
Internal friction angle (rock)	φ	40^o
Internal friction angle (sand)	φ	30^o
Cohesion (sand)	c'	0 kPa

According to Voorendt et al. (2020), the relevant failure mechanisms for a caisson unit during transportation, execution and installation phase are as follows:

- Static stability (during floating transport and immersion)
- Dynamic stability
- Shear criterion caisson-subsoil
- Rotational stability
- Vertical bearing capacity
- Scour
- Strength of the concrete structure
- Overtopping

From this list of failure mechanisms, scour and the strength of the concrete structure is considered to be outside the scope of this research and will therefore not be discussed in the following section.

Static stability of caissons

Floating elements, especially caisson units that are not built for sailing performances, can react undesirable, for instance moving, pitching or rolling, on (very) little disturbances or even without any interaction due to internal stability issues, To avoid undesirable and hence unsafe behavior, the caisson must be designed depending on its stability, which depends on the forces and moments acting on the element, but also the shape and additional structural elements or ballasting material.

One of the requirements for a floating element is an equilibrium for the vertical forces, i.e. the buoyant force and the weight of the element, Obviously, without this requirement the object would either sink when the weight of the element is larger than the buoyant force, or would (fully) immerse until either a new equilibrium will be established or instability causes the element to tilt over due to unbalanced moments.

The vertical equilibrium and hence the draft of a floating element can be determined by the sum of the vertical forces. The present forces are the buoyant force and the weight including possible ballast are shown in Figure 3.3 and are given by:

$$F_b = V_{uw} \cdot \gamma_w = l \cdot w \cdot d \cdot \gamma_w \tag{3.2}$$

$$F_w = \Big(l \cdot w \cdot h - (l - 2t_w) \cdot (w - 2t_w) \cdot (h - t_b) \Big) \cdot \gamma_c \tag{3.3}$$

in which:

V_{uw} [m^3] = submerged volume of floating element
γ_w [kN/m^3] = specific weight of (fresh/salt) water
t_w [m] = thickness of caisson walls
t_b [m] = thickness of caisson floor/bottom
γ_c [kN/m^3] = specific weight of concrete

The static stability analysis of a caisson is shown in Figure 3.3: in the left sketch the caisson is shown while in rest and hence no moments around the centre are present, while in the right sketch the caisson is slightly tilted resulting in moments that may cause stability issues. Points of interest in this Figure are as follows:

- B: centre of buoyancy, at this point the buoyant force F_b applies to the element. Assuming a rectangular caisson (container) this point is halfway from the bottom of the caisson to the waterline, and is the centre of displaced volume by the caisson.
- G: centre of gravity of the caisson, the point of application of the gravitational force F_w. During ballasting or increasing filling the centre of gravity will be lowered. Rotational movement of the caisson has no effect on the position, considering the situation without ballast or fixed ballast.
- M: position of the metacentre, the point of intersection between the action line of buoyant force and the axis of symmetry (z-axis) considering the tilted position. It is assumed that for small rotations, $\phi < 10^o$, the metacentre is a fixed point.

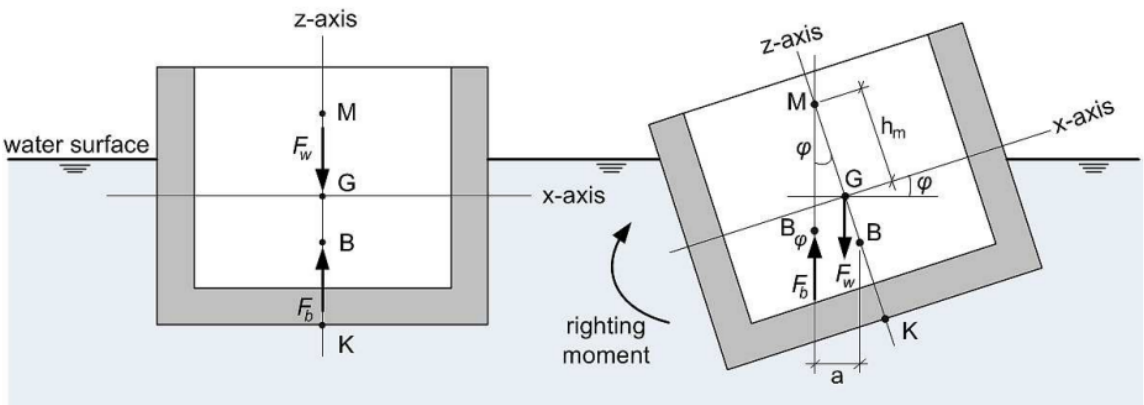


Figure 3.3: Static stability of a floating element. After Voorendt and Molenaar (2021)

According to Voorendt and Molenaar (2021) the metacentric height must be at least 0.5 m above the centre of gravity ($h_m > 0.5$ m) considering small vessels and caissons, and for large vessels this must be at least

1.1 m ($h_m > 1.1$ m).

The static stability of the floating element can be divided into several steps:

Step 1. Calculate the weight of the structure F_w and the corresponding centre of gravity G, with respect to point K ($\overline{KG}$), which is defined as the bottom of the caisson floor. The distance $\overline{KG}$ can be calculated by:

$$\overline{KG} = \frac{\sum V_i \cdot e_i \cdot \gamma_i}{\sum V_i \cdot \gamma_i} \tag{3.4}$$

in which:

V_i [m^3] = volume of element i
γ_i [kN/m^3] = specific weight of element i
e_i [m] = distance from gravity centre of element i and reference level

Step 2. Calculate the draft d of the floating element, $d = \frac{F_w}{w \cdot l \cdot \gamma_w}$

Step 3. Determine the centre of buoyancy B and its position with respect to the bottom of the caisson, which is in this case, $\overline{KB} = \frac{1}{2}d$.

Step 4. Determine the most unstable area at the fluid surface and calculate the smallest area moment of intertia, $I_{yy} = \frac{1}{12} \cdot l \cdot w^3$.

Step 5. Calculate the displaced water volume, $V_{dw} = l \cdot w \cdot d$.

Step 6. Compute $\overline{BM} = \frac{I_{yy}}{V_{dw}}$

Step 7. Determine $h_m = \overline{GM} = \overline{KB} + \overline{BM} - \overline{KG}$

Step 8. Check whether $h_m > 0.5$ m.

Dynamic stability of floating caissons
The dimension, i.e. length and width, of the caisson while floating must be sufficient to prevent any negative interaction between waves or swell and the caisson, otherwise it could happen that the structure will start swinging (rolling or pitching) on the waves. In practice, a rule of thumb is used:

$$L_0 < 0.7 \cdot l \text{ and } L_0 < 0.7 \cdot b \tag{3.5}$$

in which L_0 denotes the deep water wave length [m], l the length of the caisson [m] and b the width of the caisson [m]. Note that this expression is dependent on the the direction of the waves relative to the caisson. If this requirement is not fulfilled, problems due to swinging and pitching of the element can be expected.

Shear criterion caisson-soil
The sum of the horizontal forces acting on the caisson after it has been installed, must be transferred to the subsoil. Hence, this requires the friction between the element and the subsoil to be larger than the total of horizontal forces, as is indicated in Figure 3.4.
The friction coefficient f in this figure takes several mechanisms into account, of which the most critical of the following must be used:

1. Friction between structure and subsoil, $f = \tan(\delta)$, where δ is the friction angle between structure and soil. Typically this is 0.5 for caisson-rubble interaction.
2. Internal friction of the subsoil, $f = \tan(\phi)$, where ϕ is the internal friction angle of the subsoil.
3. A deeper soil layer with a low sliding resistance.

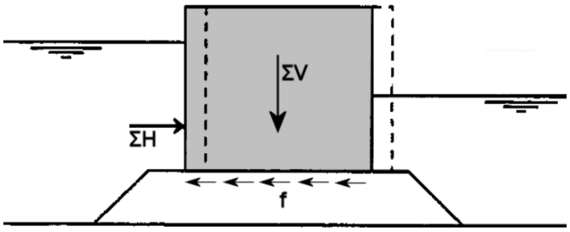


Figure 3.4: Horizontal sliding principle of a caisson. After Voorendt et al. (2020)

For the shear criterion the most critical situation will be chosen, while neglecting the deeper soil properties, thereby resulting in two options: the friction between structure and subsoil ($f = \tan(\delta)$) for caisson-rubble interaction) and the internal friction of the subsoil ($f = \tan(\phi)$). It turns out that the latter situation is the most critical, and hence will be governing for the shear criterion of the caisson. According to the Rock Manual (CIRIA, CUR, & CETMEF, 2007), quarried rock - the material for the rock foundation layer - has a friction angle between 40-55^o.

Now to find the required vertical force to retain horizontal forces, the friction coefficient discussed in the previous paragraph determines the magnitude. For a high friction coefficient, a less strong dependency between vertical and horizontal force will evoke. The downward force necessary to prevent sliding, or shear criterion to happen, can be calculated with:

$$F_{req} = \frac{\sum H}{f} \tag{3.6}$$

Besides the difference in hydraulic pressure after installation due to the water level difference between both sides of the caisson, the impact of waves must also be considered in the force and moment equilibriums. According to Voorendt and Molenaar (2021), the impact of non-breaking waves can be estimated by the Sainflou method, that has been used as an approximation to calculate the total force on a structure or wall. This approach is based on Stokes' second order wave theory. Due to interference of reflected waves, peaks in front of the caisson will reach an amplitude with the magnitude of the incoming wave height. Due to this phenomena, the increase in water level in front of the structure is given by:

$$h_0 = \frac{1}{2} \cdot k \cdot H_{in}^2 \cdot \coth(k \cdot d) \tag{3.7}$$

where:

h_0 [m] = the increase in water level in front of the structure
H_{in} [m] = initial incoming wave height, not influenced by the caisson
d [m] = water depth in front of the structure, 2 or 3 wave lengths away from the wall
k [m^{-1}] = wave number of the incoming waves, $k = 2\pi/L$
L [m] = wave length

Combining Sainflou's approach and Stokes' second order wave theory lead to the following maximum water pressures at mean sea level and at the sea bed respectively:

$$p_0 = \frac{\rho_w \cdot g \cdot H_{in}}{\cosh(k \cdot d')} \tag{3.8}$$

$$p_1 = \rho_w \cdot g \cdot (H_{in} + h_0) \tag{3.9}$$

$$p_2 = \gamma_w \cdot (H_{in} - h_0) \tag{3.10}$$

in which:

ρ_w [kg/m^3] = density of sea water
d' [m] = the water depth above the foundation level of the structure

All parameters introduced above are also shown in Figure 3.5. The corresponding forces acting on the caisson, induced by the water pressure of the waves, are as follows:

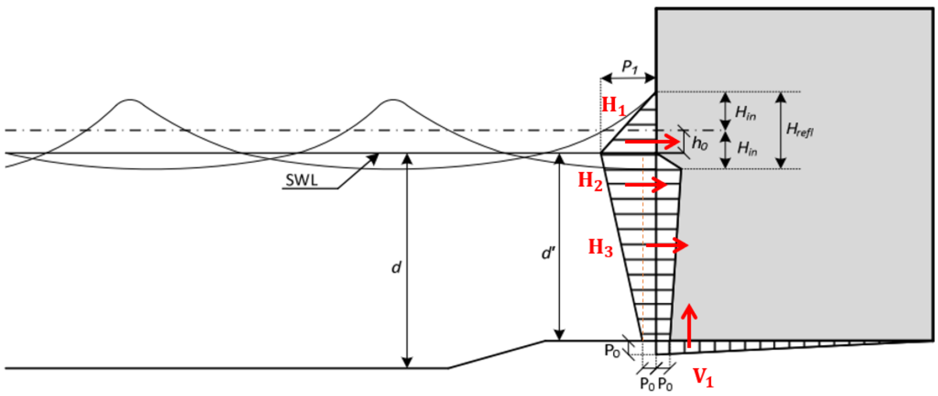


Figure 3.5: Wave pressure according to Sainflou and resulting forces acting on the caisson. After Voorendt and Molenaar (2021).

$$H_1 = \frac{1}{2} \cdot p_1 \cdot (h_0 + H_{in} \cdot l) \tag{3.11}$$

$$H_2 = \frac{1}{2} \cdot (p_1 - p_0) \cdot d' \cdot l \tag{3.12}$$

$$H_3 = p_0 \cdot d' \cdot l \tag{3.13}$$

$$V_1 = \frac{1}{2} \cdot p_0 \cdot w \cdot l \tag{3.14}$$

Intermezzo - Partial safety factors

Safety factors are used to ensure the reliability for uncertainties in the design process and execution phase and include uncertainties in parameters such as material properties, design loads and construction methods. With the application of partial safety factors, the structures failure probability is reduced and hence is able to withstand unfavourable load conditions.

For civil structures in general, the well-known EU Eurocodes (European Commission, n.d.) are often appropriate and suitable to be used, and offer a wide and comprehensive pallet of design rules for buildings and agricultural structures. However, these design rules do not capture offshore structures like breakwaters, caisson structures, quay walls and closure dams. A well-suited alternative to this common design manual is the Spanish design and construction manual, specialised for harbour structures constructed with reinforced floating caissons (ROM, 2006).

In order to determine the stability of the caisson, a factor of safety must be taken into account. The magnitude of the factors depend on the positive or negative effect on the overall stability and whether the load/strength can be considered to be a variable or constant parameter.

According to the Spanish guideline for the construction of caissons and breakwaters, partial safety factors for the SLS and ULS are given, which are summarized per category in Figure C.1. For example, the self weight of the structure is a constant positive load: the weight will not change over time and has a positive effect on the stability of the structure, resulting in a factor of safety of 1. Whereas the variable wave loads have a partial safety factor of 1.5.

According to the partial safety factors for reinforced concrete caisson units (ROM, 2006), negative variable loads, in this case the wave induced pressures and resulting forces, will be multiplied with a factor 1.5, whereas the positive constant loads, i.e. the weight of the structure, ballast force and the buoyant force will be constant, and hence have a safety factor of 1.0. This results in the following sum of forces:

$$\sum H = 1.5 \cdot \Big(H_1 + H_2 + H_3\Big) \tag{3.15}$$

$$\sum V = V_1 \cdot 1.5 + F_b \cdot 1.0 + (G_{caisson} + W_{ballast}) \cdot 1.0 \tag{3.16}$$

Horizontal soil stresses With the presence of the berm of the caisson dam, horizontal acting and retaining soil stresses emerge - that are relevant for the shear criterion - when sand is placed at the inner side. Active soil pressure considers the situation when the soil acts on the structure, while passive soil pressure evoke when the structure acts on the retaining soil. Depending on the acting or retaining function of the soil, the corresponding horizontal soil pressures can be calculated with Equations 3.17 and 3.18.

$$\sigma'_{h,min} = K_a \sigma'_v - 2c\sqrt{K_a} \tag{3.17}$$

$$\sigma'_{h,max} = K_p \sigma'_v + 2c\sqrt{K_p} \tag{3.18}$$

where:

$$K_a = \frac{1 - \sin(\phi')}{1 + \sin(\phi')}$$

$$K_p = \frac{1 + \sin(\phi')}{1 - \sin(\phi')}$$

In case the soil is non-cohesive (c = 0), for instance sand, the horizontal soil stress equation simplify. The forces resulting of the active and passive soil stresses are presented in Figure 3.6, which have been divided into three individual components, S_1, S_2 and S_3. These forces can be determined with the following expressions:

$$S_1 = \frac{1}{2} \cdot \sigma_{h1} \cdot (d' - h_{berm}) \cdot l \tag{3.19}$$

$$S_2 = \sigma_{h1} \cdot h_{berm} \cdot l \tag{3.20}$$

$$S_3 = \frac{1}{2}(\sigma_{h2} - \sigma_{h1}) \cdot h_{berm} \cdot l \tag{3.21}$$

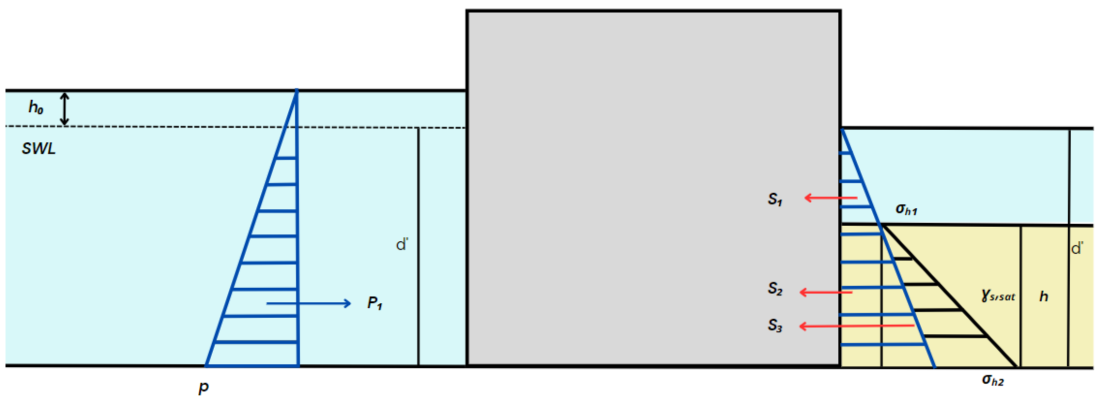


Figure 3.6: Horizontal soil forces (active or passive resisting) of the inner berm for the case when the inner berm has not been finished yet and hence is still partly immersed.

The horizontal stresses can be calculated with Equations 3.17 and 3.18, which depend on the vertical stresses. Two situations can be distinguished regarding the position of the top of the berm: when the berm has not heightened to the design height and is partly submerged and the situation when the top

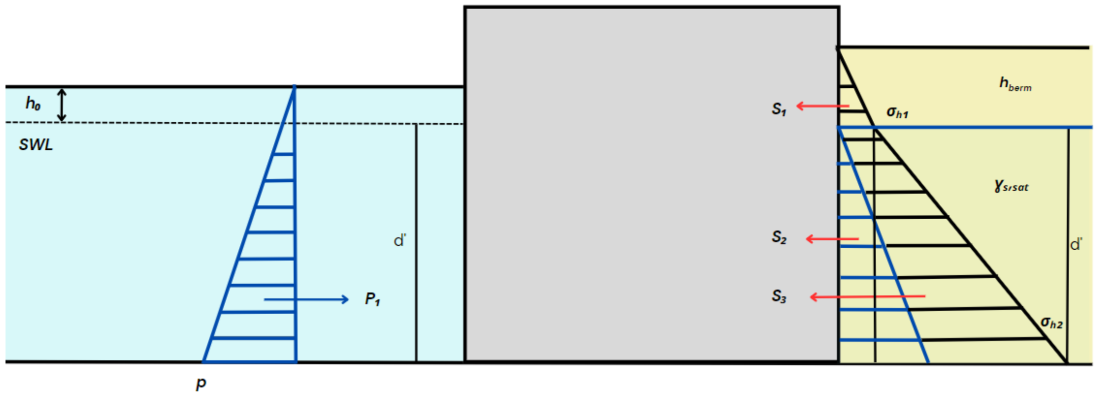


Figure 3.7: Horizontal soil forces (active or passive resisting) of the inner berm, for the phase where the construction of the inner berm has been finished.

of the berm is higher than the water surface. As such, the vertical stresses are given by the following expressions:

if $h_{berm} < d'$:	if $h_{berm} > d'$:
$\sigma_{v1} = \gamma_w \cdot (d' - h_{berm})$	$\sigma_{v1} = \gamma_{s,unsat} \cdot (h_{berm} - d')$
$\sigma_{v2} = \sigma_{v1} + h_{berm} \cdot \gamma_{s,sat}$	$\sigma_{v2} = \sigma_{v1} + h_{berm} \cdot \gamma_{s.sat}$

Rotational stability

Rotational stability issues of the caisson unit occur when tensile stresses develop perpendicular to the bottom of the caisson, due to dis-balance between horizontal and vertical forces acting on the caisson. To prevent these tensile stresses from developing, the line representing the resulting force must intersect the core of the element, which is defined as the area extending to $1/6 \cdot w$ on both sides of the gravity centre line. This can also be seen in Figure 3.8.

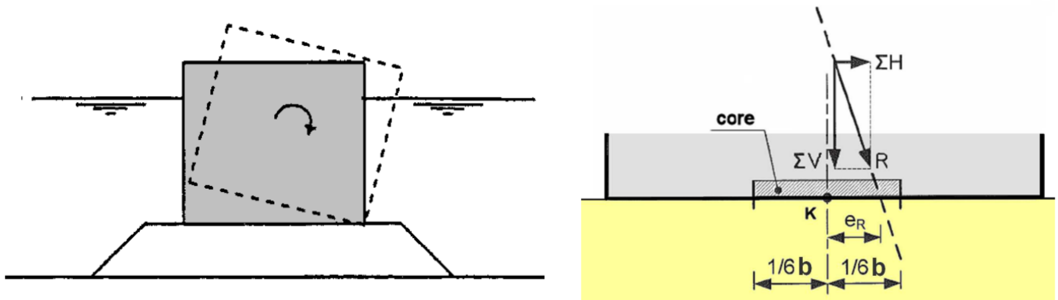


Figure 3.8: Rotational stability of the caisson element. The line of resulting force should intersect the core of the structure to prevent rotational issues. After Voorendt et al. (2020)

The quantitative requirement to ensure rotational stability is:

$$e_R = \frac{\sum M}{\sum V} \leq \frac{1}{6} w \tag{3.22}$$

where:

e_R = distance from the moment centre (K) to the intersection point of the resulting force with the bottom line [m]
$\sum V$ = total of the vertical forces acting on the element [kN]
$\sum M$ = total of moment around the centre of gravity [kNm]
w = width of the structural element [m]

A schematic overview of the forces and the distances to the centre of the caisson bottom is shown in Figure 3.9.

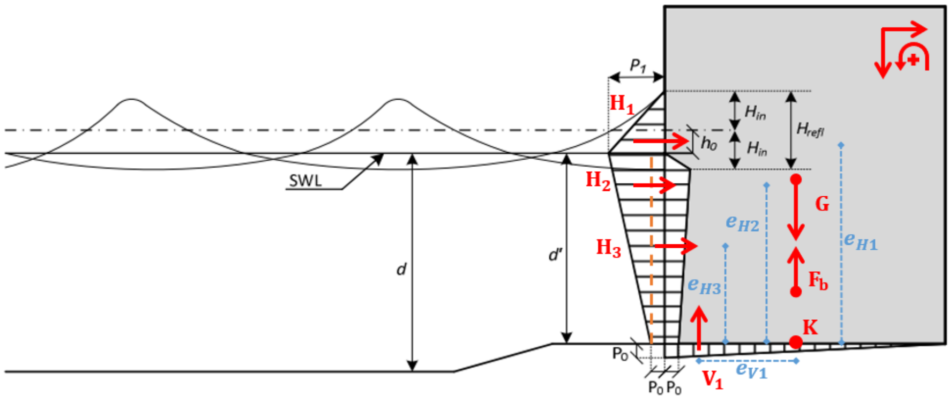


Figure 3.9: Forces and corresponding distances to an arbitrary rotational centre, in this case point K.

Vertical bearing capacity

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

$$\sigma_{k,max} = \frac{F}{A} + \frac{M}{W} = \frac{\sum V}{w \cdot l} + \frac{\sum M}{\frac{1}{6} l w^2} \tag{3.23}$$

where:

F = normal force [kN]
A = area perpendicular to the normal force [m^2]
M = acting moment [kNm]
W = section modulus [m^3]
$\sum V$ = total of the acting vertical forces [N]
w = width of the structural element [m]
l = length of the structural element [m]
$\sum M$ = total of the moment, preferably around centre of gravity [kNm]

However, since the structure will be placed on top of a sill, the resulting soil stress will decrease subsequently, due to the distribution of loads which has been visualised in Figure 3.10. To that end, the effective soil stresses that will act onto the subsoil can be determined by:

$$\sigma'_{k,max} = \frac{w}{w'} \cdot \sigma_{h,max} + \gamma'_{sand} \cdot h_{sill} \tag{3.24}$$

where w' is the increased width of the foundation, and can determined with:

w' [m] = $w + h_{sill} \cdot \tan(45)$

According to Eurocode NEN-EN 9997, the maximum bearing capacity can be determined with the Prandtl & Brinch Hansen method:

$$p'_{max} = c' N_c s_c i_c + q' N_q s_q i_q + \frac{1}{2} \gamma' b' N_\gamma s_\gamma i_\gamma \tag{3.25}$$

For sand the cohesion can be neglected (i.e. $c' = 0$ kPa) hence the first term in the expression will vanish. Moreover, it can be assumed that the effective soil stress near the caisson is negligible, resulting in a bearing capacity expression with only one term left:

$$p'_{max} = \frac{1}{2} \gamma' b' N_\gamma s_\gamma i_\gamma \tag{3.26}$$

where:

p'_{max} [kPa] = maximum soil bearing capacity
γ' [kN/m^2] = the relative effective specific weight, $\gamma' = \gamma_s - \gamma_w$
N_q [-] = factor for surcharge, $N_q = \frac{1+\sin(\phi')}{1-\sin(\phi')} \cdot \exp(\pi \tan(\phi'))$
N_γ [-] = factor for the subsoil, $N_\gamma = (N_q - 1) \cdot \cot(\phi')$
ϕ' [^{o}] = angle of internal friction
s_γ [-] = shape factor for foundations, $s_\gamma = 1 - 0.3 \cdot \frac{w}{l}$
i_γ [-] = factor for horizontal load, $i_\gamma = \left(1 - \frac{\sum H}{\sum V + A \cdot c' \cdot \cot(\phi')}\right)^3$

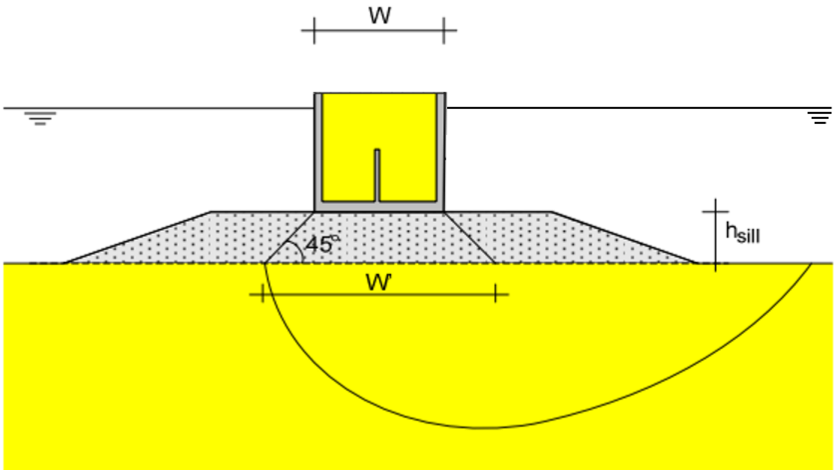


Figure 3.10: Macro stability problem for the installed caisson while fully ballasted with sand. Soil pressures at the subsoil are slightly reduced due to the presence of the sill, which is beneficial for the maximum soil stresses. Image: Voorendt et al. (2020).

Piping and scour

Water flow under or around a structure is driven by a head difference between two locations. Seepage and heave can develop into the phenomena called piping, which is the removal of soil particles from around or under the soil or water retaining structure as can be seen in Figure 3.11. Obviously, the removal of soil particles from the subsoil is considered as a failure, potentially leading to a collapse of the entire structure.

A widely applied method that describe the critical situations in which piping can occur are the empirical formulas of Bligh and Lane, which relate the head difference and the seepage length to a critical limit state where piping will occur. Both method are presented in Figure 3.12, and corresponding length scales are shown in Figure 3.13.

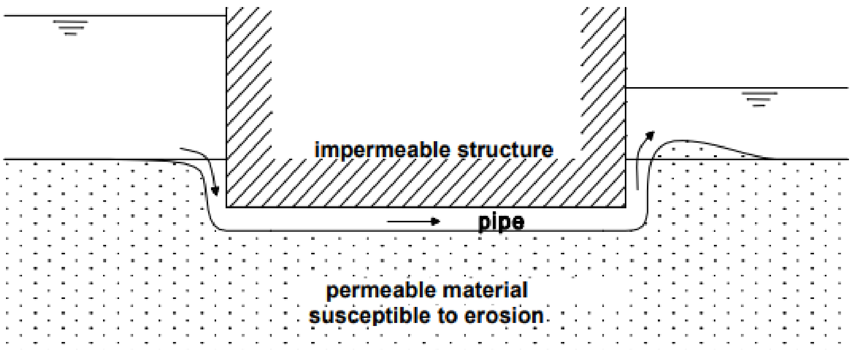


Figure 3.11: Fully developed piping path. After Voorendt and Molenaar (2021).

Piping method:	**Bligh**		**Lane**	
criterion	$L \geq \gamma \cdot C_B \cdot \Delta H$		$L \geq \gamma \cdot C_L \cdot \Delta H$	
used seepage length	$L = \sum L_{vert} + \sum L_{hor}$		$L = \sum L_{vert} + \sum \frac{1}{3} L_{hor}$	
	C_B	i_{max}	C_L	i_{max}
Soil type:				
Very fine sand / silt /sludge	18	5,6 %	8,5	11,8 %
Fine sand	15	6,7 %	7,0	14,3 %
Middle fine sand	-	-	6,0	16,7 %
Coarse sand	12	8,3 %	5,0	20,0 %
(fine) gravel (+sand)	5-9	11,1 – 20,0 %	4,0	25,0 %

Figure 3.12: Safe seepage distance for piping. After Voorendt and Molenaar (2021).

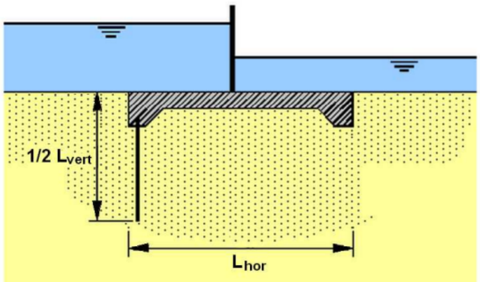


Figure 3.13: Definition of piping lengths. After Voorendt and Molenaar (2021).

Strength of the reinforced concrete structure

The strength of the caisson unit itself depends mainly on the shape and dimensions of the structure and the strength properties of the construction materials. Despite the fact that the dimensions of the caisson have been determined based on other stability design checks, it should be checked whether all loads actually can be resisted by the structure.

Overtopping

Lastly, severe wave overtopping the caisson unit may cause damage to structures on the crest or even at the rear side of the dam, for instance the inner berm. Therefore, the caisson dam must meet the overtopping criterion. This Section will discuss the relevant aspects, overtopping limits and expressions considering overtopping of a vertical wall with bullnose with absence of a foreshore and mound, according to EurOtop (2018).

The latter simplifications are based on the deep water conditions and the relative limited sill height. Depending on the wave characteristics, impulsive and non-impulsive wave impact may occur, which effects the rate of overtopping significantly. Whether impulsive wave conditions are expected to occur, can be determined with Equations 3.27 and 3.28.

$$\frac{h^2}{H_{m0} L_{m-1,0}} > 0.23 \qquad \text{treat as non-impulsive conditions} \tag{3.27}$$

$$\frac{h^2}{H_{m0} L_{m-1,0}} \leq 0.23 \qquad \text{treat as impulsive conditions} \tag{3.28}$$

Non-impulsive wave conditions Due to the absence of a foreshore and mound, the overtopping equations can be calculated by Equation 3.29 for non-impulsive conditions

$$\frac{q}{\sqrt{g H_{m0}^3}} = 0.062 \exp\Big(-2.61 \cdot \frac{R_c}{H_{m0}} \Big) \tag{3.29}$$

Impulsive wave conditions In case impulsive wave conditions prevail, the wave overtopping increases significantly due to the different wave structure interaction and subsequently, the latter expression cannot be applied. When impulsive wave conditions prevail, the ratio R_c/H_{m0} determines which expression

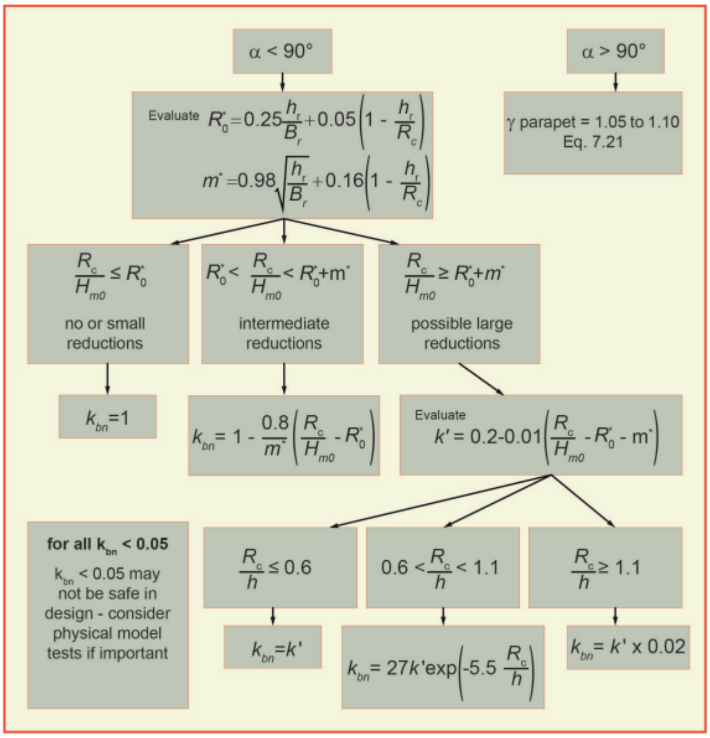


Figure 3.14: Decision chart summarising methodology for seaward overhanging bullnose/wave return wall. (EurOtop, 2018)

should be applied: for $0.1 < R_c/H_{m0} < 1.35$, Equation 3.30 should be used and if $R_c/H_{m0} \geq 1.35$, Equation 3.31 should be applied.

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0155\Big(\frac{H_{m0}}{hs_{m-1,0}}\Big)^{0.5}\exp\Big(-2.2\frac{R_c}{H_{m0}}\Big) \qquad \text{valid for } 0.1 < R_c/H_{m0} < 1.35 \tag{3.30}$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0020\Big(\frac{H_{m0}}{hs_{m-1,0}}\Big)^{0.5}\Big(\frac{R_c}{H_{m0}}\Big)^{-3} \qquad \text{valid for } R_c/H_{m0} \geq 1.35 \tag{3.31}$$

The addition of a seaward overhanging wall on top of the vertical wall may reduce the overtopping discharge significantly, depending on several parameters such as the width and height of the wave return wall and the wave characteristics. The detailed procedure to calculate the influence of the bullnose is shown in Figure 3.14, at which the influence factor k_{bn} is determined. This factor can also be expressed as:

$$k_{bn} = \frac{q_{\text{with bullnose}}}{q_{\text{without bullnose}}}$$

where q [l/s/m] or [m^3/s/m] denotes the overtopping discharge over the vertical structure. According to EurOtop (2018), well protected hinterland structures can withstand overtopping discharges of 100 - 200 l/s/m.

Wind waves

The Bretschneider equations can be used to determine the significant wave height and corresponding wave period for wind waves, generated over a given fetch F with a wind speed u_{10} measured at 10 m above the ground. In this case, this will be relevant to determine the wave heights that can be expected inside the reservoir, which will lead to direct wave attack of the inner berm. To ensure the inner berm will be stable over time, a stone revetment will probable necessary to prevent severe transport of sediment.

The principle behind the Bretschneider equations is the fact that the wind blowing over a stretch of water will cause the water to move, due to the presence of shear stresses at the water surface. If wind conditions persist for a sufficient period (assuming one direction), waves will be generated parallel to the wind direction. The wave height and wave period depends on parameters such as the wind speed (measured at 10 m above the surface), the fetch length and the water depth.

The significant wave height and corresponding wave period can be calculated with Equation 3.32 and 3.33, respectively. In these equations, non-dimensional parameters are used, denoted with a bar (e.g. $\bar{F}$).

$$\bar{H} = 0.283 \cdot \tanh\left(0.530\bar{d}^{0.75}\right) \tanh\left[\frac{0.0125\bar{F}^{0.42}}{\tanh(0.53\bar{d})^{0.75}}\right] \tag{3.32}$$

$$\bar{T} = 2.4\pi \cdot \tanh\left(0.833\bar{d}^{0.375}\right) \tanh\left[\frac{0.077\bar{F}^{0.25}}{\tanh(0.833\bar{d})^{0.375}}\right] \tag{3.33}$$

in which the parameters can be calculated with the following expressions:

$$\bar{d} = \frac{dg}{u^2}$$
$$\bar{F} = \frac{Fg}{u^2}$$
$$\bar{H} = \frac{H_s g}{u^2}$$
$$\bar{T} = \frac{T_s g}{u}$$

Where:
- g [m/s] = gravitational acceleration
- u [m/s] = wind speed at 10m height
- d [m] = water depth
- F [m] = fetch length
- H_s [m] = significant wave height
- T_s [s] = significant wave period

3.4. Stability Checks - Calculations

Stability checks are important for a caisson unit during various installation phases. In the pre-installation phase, factors that determine the caisson unit's buoyancy and weight must be considered, and in particular their relevance with regard to the static and dynamic behavior. For the installation, the caisson unit will be ballasted with water in order to place the unit on the foundation layer. During this process the caisson is subjected to changing forces induced by wind, wave and current. The stability of the caisson shortly after is has been installed and when it has been ballasted with sand, will be further assessed. Once installed completely, the caisson unit and its inner berm must withstand dynamic loads and forces caused by waves, currents and wind.

This Section will evaluate the stability of the caisson unit and scour protection according to the hydraulic boundary conditions that are presented in Table 3.4. The first column shows the various construction phases that are considered, the second column outlines the design water level consisting of the storm surge (and its return period) and the estimated sea level rise, the third column lists the return period and corresponding return value for the significant wave height. The following phases are considered to be relevant for the stability checks:

Phase I - Installation of rock foundation layer
- Stability calculation of armourstone - Currents with R = 100 years and set-down with R = 100 years

Phase II - Transportation of floating caisson (Figure 3.16a):
- Static stability
- Dynamic stability

Phase III - Caisson coupling to HLB and AHT and positioning (Figure 3.16b)

Phase IVa - Caisson fully ballasted with water (Figure 3.16c):
- Shear criterion
- Rotational stability

Phase IVb - Caisson fully ballasted with sand (Figure 3.16d):
- Shear criterion
- Rotational stability
- Vertical bearing capacity

Phase V - Scour protection

Phase VI - Construction inner berm
- Shear criterion - required berm height for storm conditions with R = 25 years

Phase VII - Installation of bank protection

Phase VIII - Facility in operation
- Shear criterion - Active soil pressure with governing wave conditions
- Exceedance of the bearing capacity with conditions of R = 500 years
- Overtopping - ULS with R = 500 years
- Overtopping - SLS with R $\leq$ 1 year

For each construction step, the relevant failure mechanisms from the list above are considered. A visual representation is given in Figure 3.15. For each phase a brief summary will be given, summarizing the corresponding extensive assessment which can be found in Appendix C.

Phase I - Installation of rock foundation

The rock foundation layer, consisting of a 45 - 180 mm grading, will be stable for the design conditions with a maximum current of 1.3 m/s. Despite this fact, the foundation layer might be affected or polluted by morphological changes in the sea bed in case the layer will be installed in one go. Consequently, the foundation layer will be installed in sections of 10 caisson lengths and will be extended when the current foundation is smaller than 5 caisson lengths.

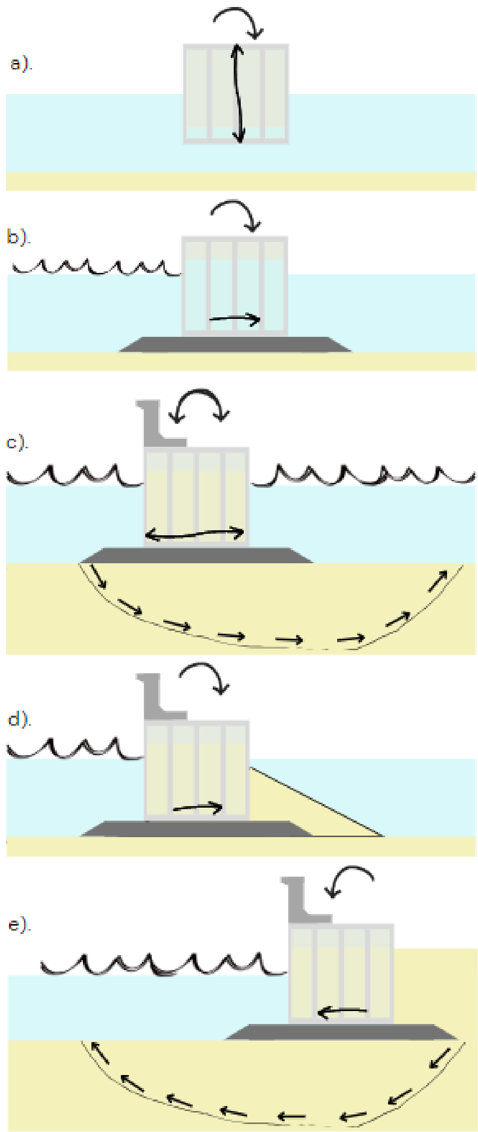


Figure 3.15: An overview of the work method with the relevant failure mechanisms.

Phase	Design Water Level			Significant wave height		
	Sea level rise	Storm surge	**Total**	Return period	Direction	Return value
Phase IVa	0.5 m (2050)	0 m (-)	0.5 mMSL	<< 1 year	omni	< 6.2 m
Phase IVb	0.5 m (2050)	2.2 m (2 yr)	2.7 mMSL	2 years	omni	6.7 m
Phase VI	0.5 m (2050)	2.6 m (25 yr)	3.1 mMSL	25 years	omni	8.12 m
Phase VII (SLS)	2.0 m (2150)	2.0 m (1 yr)	4.0 mMSL	0.1 year	omni	4.1 m
Phase VII (ULS)	2.0 m (2150)	3.0 m (500 yr)	5.0 mMSL	500 years	225^o-270^o	9.75 m
Phase VII (set down)	0 m	-2.5 m (500 yr)	-2.5 mMSL	500 years	225^o-270^o	9.75 m

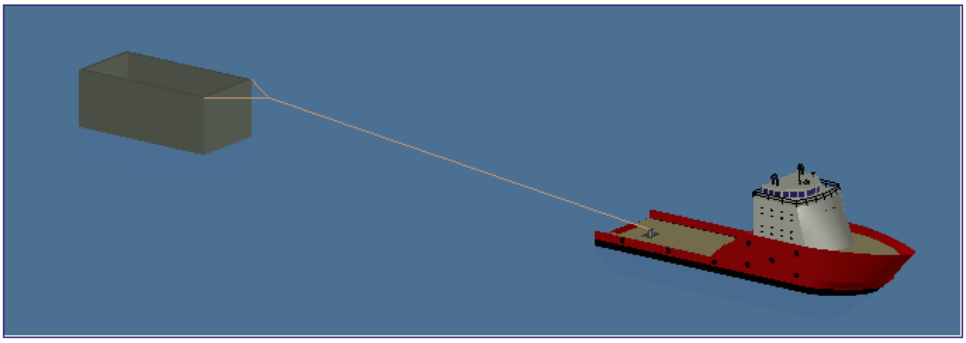
Table 3.4: Hydraulic boundary conditions per construction phase with corresponding return periods.

Phase II - Transportation of floating caisson

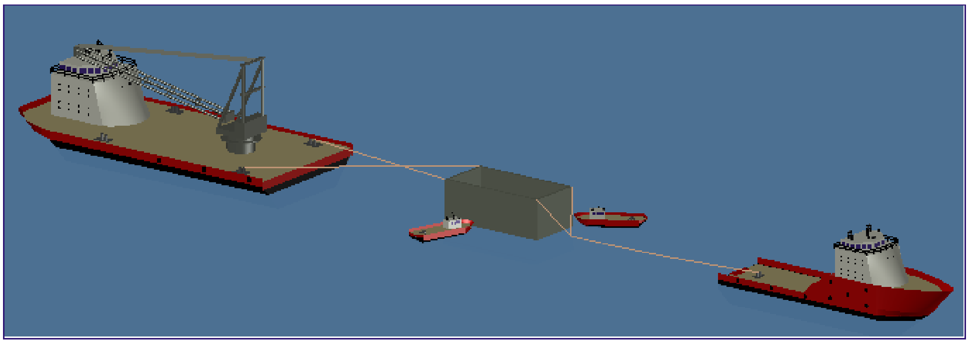
The dynamic and static stability of the caisson unit has been assessed, an important parameter for the transportation to the project site. It turns out that the caisson unit is statically stable. Due to the dynamic stability restrictions, caisson transport is only possible when wave lengths are below L_w = 42 m.

Phase VIII - Caisson fully ballasted with water

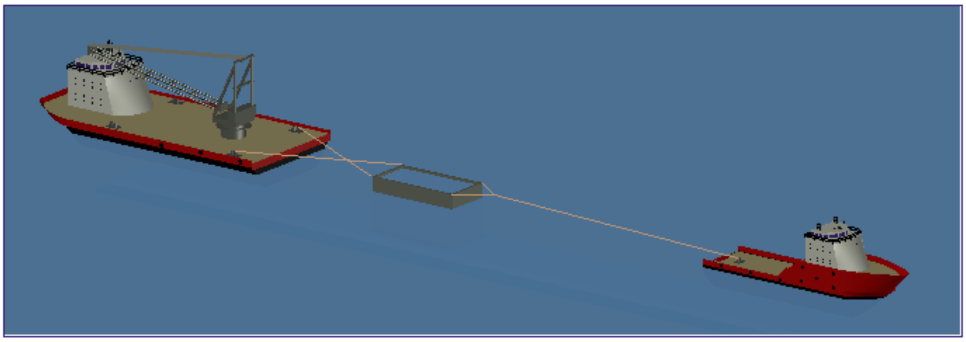
Shortly after installation, the caisson unit is only ballasted with water. It turns out that the element is most vulnerable for exceedance of the shear criterion and rotational stability: the caisson unit becomes unstable for wave heights exceeding H_s = 2.5 m. However, this stability limit is sufficient for the entire



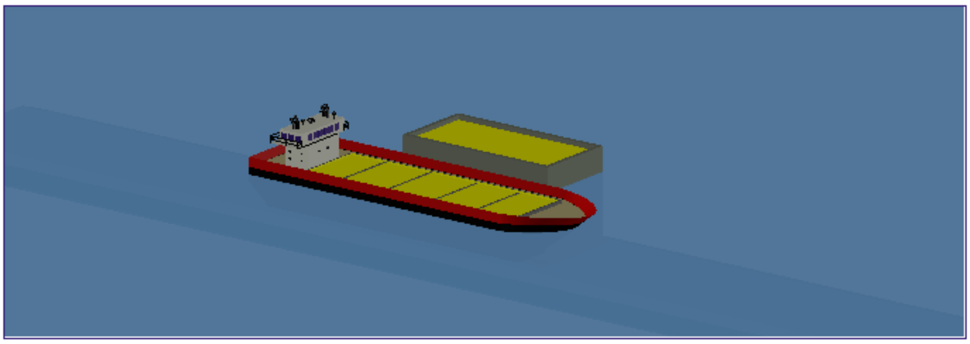
(a)



(b)



(c)



(d)

Figure 3.16: (a) Caisson transportation from intermediate port to project by an Anchor Handling Tug. (b) Coupling of caisson to HLB and AHT for positioning prior to the ballasting. (c) Ballasting of the caisson with water. (d) Filling of caisson unit with locally dredged material by the TSHD.

installation process, since once the unit will be sunken, the unit will be filled with sand immediately.

Phase IVb - Caisson fully ballasted with sand
Once the element has been filled with sand, the stability improves substantially. The duration of this phase, where the caisson will not be supported by an inner berm, will be in the order of several months. Therefore, the element must be stable during storm conditions. According to the assessment, the caisson unit will be stable for wave conditions of H_s = 6.7 m, corresponding to a return period of 2 years. This includes horizontal stability (shear criterion), rotational stability and vertical bearing capacity.

Phase VI - Construction inner berm
In this phase the inner berm must be installed to a certain height, to ensure stability during the winter storm conditions. For design conditions of H_s = 8.1 m (return period of 25 years), an inner berm with a height of 4 m is required. However, to incorporate uncertainties such as unforeseen morphological changes, the height must be 10 m before winter.

Phase VIII - Facility in operation
When the facility is in operation, it will be subjected to harsh offshore conditions, but also to large water level differences. The structure passes the structural assessment with regard to the horizontal and rotational stability and vertical bearing capacity for storm conditions with wave heights of H_s = 9.75 m.

Furthermore, overtopping discharges must be reduced with a wave return wall including a bullnose. Finally, the inner berm must be extended to prevent piping to establish. In addition, the inner berm must be protected with a LM_A 10 - 60 kg to withstand wind generated waves of H_s = 1.4 m. The scour protection of the powerhouse section, which facilitates the turbines, consists of a LM_A 5 - 40 kg grading.

3.5. Conclusion

The goal of this section was to assess and design the structural characteristics of the superstructure, including the caisson unit and wave return wall, scour protection and inner berm with corresponding bank protection. This assessment has considered various critical phases, starting from the transportation of the caisson to the project site until the moment when the facility is fully operating. Both Ultimate Limit State (ULS) and Serviceability Limit State (SLS) are considered for the caisson unit.

After the assessment, it has been determined that the design of the caisson unit will be a rectangular structure with dimensions of 60 meters in length, 34 meters in width, and 32 meters in height. To prevent excessive wave overtopping, the wave return wall will be 9.5 meters high with an additional bullnose. To prevent piping, the inner berm will have a slope of 1:13.5, resulting in a length of 400 meters.

The moment of transportation of the caisson has to be perfectly timed with regard to the weather conditions, to ensure the dynamic behavior of the caisson does not reach an undesirable level. Ballasting of the caisson can help to improve the stability of the caisson unit during towage, but has not been considered in this assessment since this is would be a high detailed analysis.

During placement and installation of the caisson, when the caisson is ballasted with water, the structure will be rotational unstable when the significant wave height exceeds 2.5 m. Once completely filled with sand, the structural stability of the dam improves significantly, resulting in stability even at significant wave heights of up to 6.4 meters. Additionally, an inner berm with a height of 10 m must be installed before winter to create sufficient stability during the storm season, after which it will be heightened slowly over time to a final height of 30 meters.

The powerhouse section is beyond the scope of this research, and generally, will be provided by additional research. Furthermore, an crucial aspect that has not been mentioned nor discussed so far, are the mitigation measures to prevent devastating effects to the flora and fauna near the project. High in- and outflow velocities are expected near the turbines at the powerhouse section. A common mitigation measure to prevent fish entering the turbines is the installation of fish racks in front of the turbines.

4 Deterministic planning

A deterministic planning is critical for ensuring that projects are completed on time and within budget. It involves developing a detailed schedule that takes into account all relevant factors, such as available resources, learning curves, and possible delays. By following a deterministic approach, i.e. a constant construction duration per sequence over time, vulnerabilities and unexpected delays and failures can be revealed, which will answer the question:

"What are the most critical/sensitive phases in the construction of the caisson dam, considering a deterministic approach?"

This section will start with a general introduction in which the planning conditions will be defined, such as the working season. Afterwards, the activities related to the sequence - this also includes the bunkering of vessels, replenish provisions and the loading of vessels - will be inventoried and defined following the work method described in Appendix E. Finally, using the predefined conditions and activities the planning can be composed using MS Projects, gaining insight in the project duration.

An important note prior to the general introduction is the fact that the construction of the powerhouse section will not be analysed in this research. However, it is assumed that the installation of the powerhouse units will take 2 years, regardless of any optimization.

4.1. General introduction

Prior to the elaboration on every aspects and parameters that contribute to the activity duration per sequence, the planning in general should be introduced. Eventually the project duration consists of several aspects. each contributing their share to the total duration. The most obvious is the actual activity duration, which can be expressed in some sort of productivity, that includes how long a vessel will be occupied for one specific task. This will be discussed in the next section. A minor contribution to the overall project duration, but an aspect that can have effects on the available resources and hence cannot be neglected, is the learning curve. Lastly, the definition of the work season, when does work activities can commence and when must they be finished, will have the largest consequences.

Work season

As mentioned, the definition of the work season will have a large effect on the overall duration, especially since the project involves exclusively vessel-based equipment, subjected to all weather conditions that change over the year. Extending the work season will linearly increase the available construction time per year, since the weather conditions are neglected[1]. In line with this reasoning, it would be inappropriate to extend the work season to the entire year for the deterministic, since it is known that offshore construction works cannot continue during the winter months.

To that end, the characteristics of the work season will be based on an educated guess, while taking the wave conditions into account. Taking the season variability analysis from Section 2 into consideration, early construction works and preparations can commence in March (i.e. sea bed preparation, rock foundation layer and other port works) while the structural construction works, i.e. installation of the caisson units, will commence in April and last until the end of September, at which point the latest

[1]Note that the linear increase in available construction time only applies for the deterministic planning, as the weather conditions are neglected. In reality, which also will be discussed in the probabilistic planning, extending the work season will often result in a longer period with worse conditions (seasonal variability). Subsequently, extending the work season will therefore result in a minor increase in available time, however with a significant increase in uncertainty, which often is not worth it.

construction works (scour protection works and construction of the inner berm) must have been finished. This can be visualised in a timeline, which is shown in Figure 4.1.

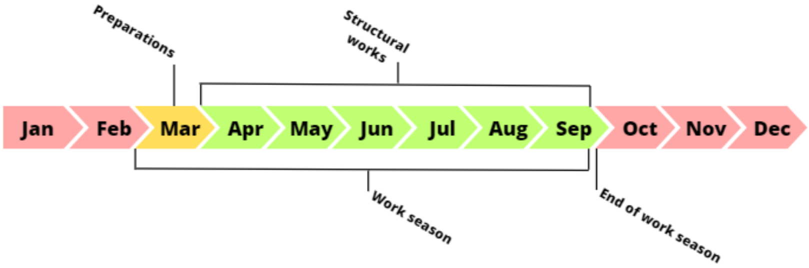


Figure 4.1: Visualisation of the proposed work season, applied to the deterministic planning. Red period indicates off-season months where no construction work is planned nor allowed, Orange period allows preparations and early construction works and Green indicates the work season, without any restrictions.

4.2. Activity duration

The key elements with regard to the construction planning are shown in Table G.2 and G.2. The first column outlines the various phases of the project, including preparations works, rock works and caisson stream. These phases represent the broad stage of the project, as the planning is considered to be preliminary and hence coarse, neglecting detailed micro activities which would be included in the final planning. The second column lists the type of activity associated with each phase, such as the positioning and ballasting of the caisson. The third and fourth column outline the duration and learning curve for each activity, which serve as tangible outputs that help to compose an accurate planning that will create more insight in the vulnerabilities and potential gains of activities. These deliverables provide a useful starting point for constructing a comprehensive plan that addresses key planning activities across all construction phases, which is presented in the next section.

4.3. Planning

The following section will discuss the planning, which is based on the essential aspects and necessary conditions, and the key elements of the planning presented in Table G.2. The detailed planning, consisting of all individual construction steps, is presented in Appendix G.5. An snapshot of the general timeline that followed from the planning is shown in Figure 4.2. In the next section the main takeaways, vulnerabilities and potentials and conclusions of the current planning will be addressed, which are all relevant for composing the probabilistic planning later on.

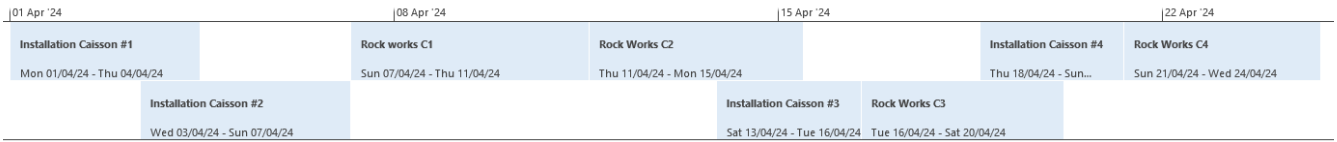


Figure 4.2: Snapshot of the summary tasks, i.e. installation of the caisson and additional rock works per caisson.

Takeaways

From the snapshot in Figure 4.2 the following aspects can be addressed:

1. Learning curve
2. Total construction time (per sequence)
3. Order of sequence

First of all, the learning curve in the installation of the caisson unit can clearly be seen by the reduction of the construction duration that is minimized after the fifth execution. The time reduction for the caisson units is 35 hours, beginning at a installation duration of 83 hours for the first unit, reducing to 47 hours for the fifth unit and following units. For the rock works the improvement of efficiency is 20 hours, reducing the duration from 104 hours to 84 hours.

The total construction duration for one caisson, i.e. placement and scour protection, of approximately 5.5 days (or 131 hours), resulting in the installation of roughly 5 - 6 caissons per month. Considering a work season of 6 months, this would imply that a total of 33 caisson could be placed in one year. When neglecting wave conditions, the total project would take approximately 8 years to construct a total of 262 caissons (length of 60 m, perimeter of 15.7 km) according to the current planning.

However, what has been neglected when considering the deterministic approach is the order of sequence. In Figure 4.2 the caisson and rock works are constructed chronologically - first the caisson and afterwards its scour protection - however in practise this could be different. Due to the prioritization of the installation of the caisson unit, it may happen that the next caisson will be installed before the last scour protection has been finished. This effect - including the wave conditions and difference in workability - will have a substantial effect on the planning, and will be included in the probabilistic planning.

Vulnerabilities and potentials
Identifying the vulnerabilities and potentials or opportunities of the deterministic planning will be beneficial when converting to the probabilistic approach. First, what can be seen in the deterministic planning is that the placement of the caisson unit may restrict the further construction activities. A delay in the installation of the caisson will subsequently cause a delay in the entire scour protection works. For the probabilistic planning, first a prioritization must be made in which the activities will be ranked based on their importance and impact on the rest of sequence; in this case that would be the installation of the caisson unit.

Furthermore, the installation of one caisson and its scour protection will approximately take one week, whereas the fabrication of the caisson itself costs 10 - 11 days. So, this implies that the outflow of caissons will be larger than the inflow, possibly causing a disruption in the logistic chain. To that end, a proper planning should be made with regard to the in- and outflow of caissons, to prevent both a lack of caisson and an exceedance of the storage capacity.

One advantage is that the same vessel can handle the scour protection for the caisson unit. Given the estimated volumes of the scour protection, the loading capacity of the SSDV is enough to install all the layers in one trip. This eliminates the need for multiple round trips to the Port of Rotterdam, saving time and resources.

4.4. Conclusion

The goal of the deterministic planning was to identify the vulnerabilities and opportunities in the current construction sequence. However, it is important to note that this planning approach assumes that wave conditions do not impact the activities and can continue uninterrupted 24 hours a day.

Based on the current planning, it is evident that there is a lack of prioritization of construction activities in terms of their impact on the overall sequence and workability. This means that activities with significant consequences if delayed or with limited windows of opportunity will be given higher priority over activities that can be executed at any time. It is important to address this issue and ensure that construction activities are prioritized based on their impact and feasibility to optimize the overall project timeline and efficiency.

In general, the deterministic planning shows that an entire caisson unit including the scour protection can be installed in approximately one week, which will further reduce due to the learning curve. Neglecting weather conditions, following only the deterministic planning, this would imply that roughly 30 caissons could be installed during one working season (from April until September). In line with this reasoning, the total project would last then approximately 7 years, assuming 212 caisson units required to construct the caisson dam. This excludes the powerhouse section with a length of 3 km which accommodates the turbines.

5 Probabilistic Planning

Disclosure - probabilistic analysis

In this research, a probabilistic approach is being utilized to incorporate wave data statistics and corresponding exceedance probabilities. While there are various stochastic methodologies available, this study focuses on applying probability through the analysis of wave conditions. In contrast, the deterministic planning assumed a fixed construction duration per activity, neglecting any influences from weather conditions and assuming continuous work for 24 hours a day.

In the upcoming section, wave data will be integrated to examine the impact of wave conditions on the project. Conservative wave conditions will be used to calculate the workability and persistency of all activities. By considering a conservative value, such as an exceedance probability of 70%, uncertainty stemming from less favorable weather conditions can be incorporated into the planning process. This, in turn, allows for a more comprehensive assessment of the effects of wave conditions on the project planning, potentially leading to the identification of delays in construction activities.

Probabilistic planning involves analyzing and predicting the outcomes of various construction sequences, based on multiple parameters such as weather conditions and available resources, rather than a deterministic planning for which the activity duration is the only parameter. By modelling different possible scenarios, vulnerabilities and opportunities can be identified, in turn resulting in a more sophisticated and robust planning that is less sensitive to disruptions and hence beneficial with regard to risk and reward balance.

In the previous section all activities were considered to continue 24 hours per day, whereas in reality wind and waves will disrupt construction works as it becomes unsafe for personnel and equipment to operate during some conditions. To include this phenomena - and hence also the seasonal variability - the workability for each distinct activity will determine how many workable hours each month contains. This results in tailored calendar per activity, that is based on the activity and the corresponding workability. Then, all calendars will be applied to the planning shown in Section 4 obtaining a probabilistic planning containing weather conditions in combination with the execution limits of the vessels and equipment.

5.1. Workability

Workability is a term that quantifies how much of the available time a specific operation can be executed, often expressed in percentages. In general, the workability depends mainly on two major components: i). working conditions and ii). workable limits. Firstly, working conditions involves weather conditions such as wind speed and wave height. Secondly, the workable limits represent the threshold beyond which the execution of the activity is considered to be dangerous to personnel, equipment and/or its surrounding. A workability rate of 100% indicates that during all scheduled working hours, a certain activity can be executed, whereas a workability of 50% shows that only half of the scheduled working hours an operation can conducted.

In order to include the differences in sensitivity with regard to the workable threshold, per activity the limiting weather conditions will be defined. Then, using 42 years of hindcast data from DHI (n.d.), the workable hours will be shown, from which the workability per month can be determined. Furthermore, the workability (in percentages or workable windows in case for the caisson installation) will be given for several exceedance probabilities, indicating the probabilistic aspect with regard to the activity.

As has been discussed earlier, in general offshore operations are often restricted by wave and wind conditions, especially during the winter months when wind speeds and wave heights become more extreme during storm season. This can also been seen in the workability tables: generally, during summer the workability is higher than during winter.

Caisson installation

Limiting wave conditions The caisson installation is the process used in the construction of various hydraulic structures, such as breakwaters and quay walls, often constructed in large water depths or narrow construction space. The caisson unit, a watertight concrete element, is towed from an intermediate wet storage to the desired location, where it will be ballasted with water in order to place it on the sea bed or foundation layer. Afterwards, to improve the overall stability of the caisson, it will be weighted with additional ballast material, often sand or quarry run.

The installation process of the caisson unit involves several steps. Starting with the preparation of the caisson in the intermediate storage prior to the journey towards site. The caisson will be provided with navigation lights and winches that will be used during towage and installation, after which it is ready for its journey to the project site. On site, the caisson will be connected with four winches to two tugs boats on one side and the two anchor points on the other side (this could be either two points on the previous installed caisson, or on the crane vessel for the first caisson).

With precise and accurate movement with the winches, the caisson will be positioned to its desired location. Pumps will ballast the caisson with water, subsequently sinking it to sea bed. Once fully ballasted, a trailing suction hopper dredge will connect its discharge system to the caisson inlet system, and will empty the hopper in the caisson. Only after the caisson has been entirely filled with ballast material (in this case dredged material), its stability is ensured and the installation can be considered as finished.

It turns out that the caisson units are very sensitive to wave and wind conditions. As the structure is floating, it reacts almost immediately on the wave conditions (depending on the wave length). This means that with severe waves, the structure will roll, yaw and pitch severely beyond a point where it will be uncontrollable, potentially hitting a vessel or the sea bed resulting in damage or uncontrolled placement.

A sensitivity analysis for a caisson unit (behavior in offshore conditions; roll, yaw and pitch in several wave climates with different wave heights and wave periods) is beyond the scope of this research, as it could easily fill an entire MSc Thesis. In stead of that, suitable limitations will be used, found by case studies, physical modelling or computational modelling.

Multiple papers were found that have investigated the workability limitations with regard to the installation process and are discussed in Appendix E. Ultimately, after an extensive analysis of the different methods, the installation limits of Costa Azul (Hibbs et al., 2010) will be used for this project and are shown in Figure 5.1.

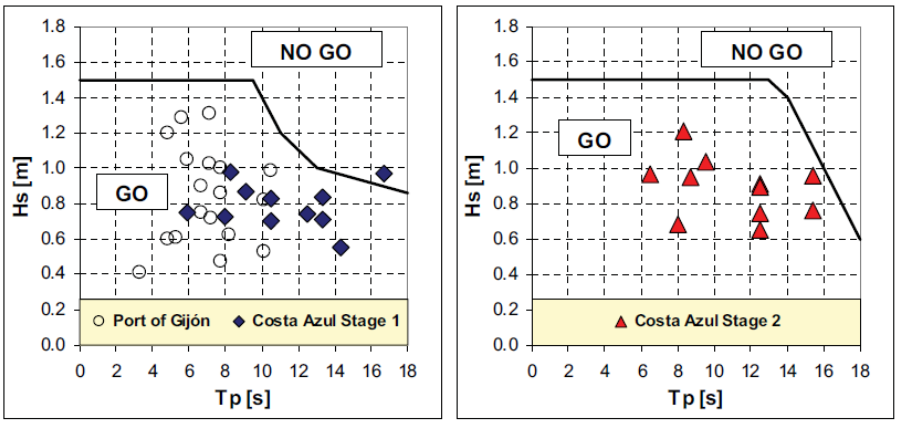


Figure 5.1: Limiting wave conditions for the caisson installation. Source: Hibbs et al. (2010)

Workability From the threshold used by Hibbs et al. (2010), the workability with a weather window of 12 hours can be analysed based on 42 years of historical wave conditions. In Figure 5.2 the workability scatter is presented, which shows the workable and unworkable windows based on the wave and wind requirements.

According to the analysis, the workability for the caisson installation has a mean value of 32 workable windows considering a work season of April - September.

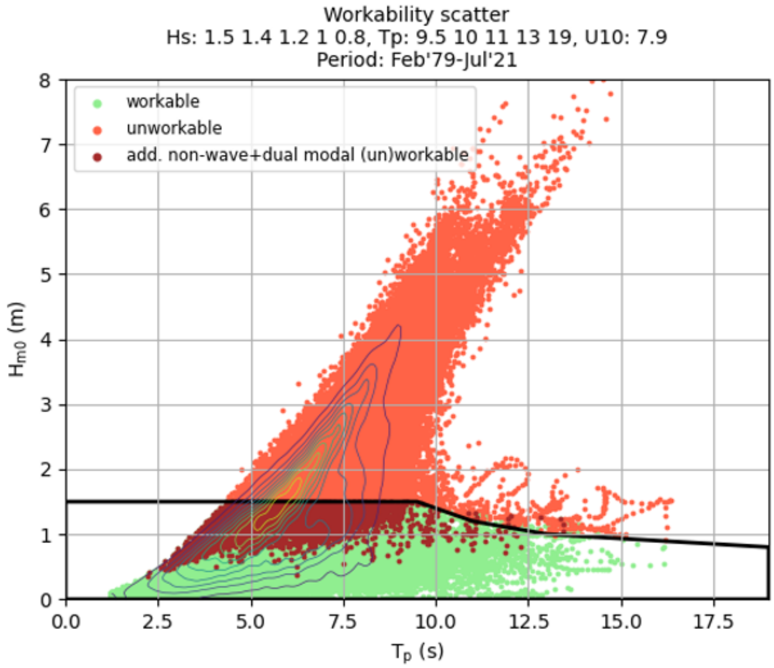


Figure 5.2: Workability scatter for caisson installation considering wave and wind limiting conditions for a window of 12 hours.

Table 5.1: Number of workable windows (12 hours of favorable conditions) for the caisson installation based on 42 years of data, determined for multiple probability of exceedance listed per month. The summation of workable windows has been determined for the favorable work period, i.e. work season from April to September.

Base	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Sum
Average	15	17	24	31	35	37	39	36	26	18	14	14	26	204
P10	25	25	37	40	44	45	49	43	35	27	23	25	28	255
P20	21	24	30	37	40	44	44	41	34	23	21	21	27	239
P30	17	21	27	35	37	40	42	39	30	21	19	19	27	223
P40	15	19	25	33	36	38	41	39	29	19	17	17	26	216
P50	14	17	23	31	35	37	39	36	27	18	15	12	25	205
P60	12	15	22	30	33	35	37	33	25	15	12	10	25	193
P70	11	12	19	28	32	34	34	32	23	14	10	9	24	183
P80	7	11	17	26	29	32	33	30	20	13	9	7	24	171
P90	4	6	11	24	27	28	28	29	16	10	6	4	23	152

Success rate The success rate is a a measure that is often used to evaluate the effectiveness of a particular goal or execution. It refers to the ratio or proportion of successful outcomes to the total number of attempts or potential attempts.

With regard to a vital construction step, for instance many tasks are dependent on this specific task (also defined as successors), taking into account the success rate will decrease the uncertainty of the overall project duration. In particular, this can be helpful for complex operations, with many requirements and predecessors, with an unknown number of windows of opportunity or workable windows. Not every workable window implies that the desired operation can be executed, as many other conditions also must allow this operation.

For example, for the next 48 hours good weather has been predicted. The first caisson has been installed within the first 12 hours, but when preparing the second caisson, the weather conditions are just below the limit, subsequently the project leader cancels the second operation. In theory, four caissons could have been installed, however only one caisson has been placed, resulting in a success rate of 25%.

Since the offshore installation of caisson units is uncommon practice, the success rate is still a vague term to include into the project planning. Therefore, a conservative success rate of 10% will be assumed for the probabilistic planning. Consequently, the distribution of numbers of installed caissons per month is listed in Table 5.2.

Table 5.2: Number of installed caissons determined for multiple probability of exceedance per month. The installed caissons includes the number of workable windows per month and the success rate of installation of 10%. The summation of installations has been determined for the favorable work period, i.e. work season from April to September.

Base	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Sum
Average	2	2	3	4	4	4	5	4	3	3	2	2	3	25
P10	3	3	5	5	5	5	6	5	4	4	3	3	4	30
P20	3	3	4	5	5	5	5	5	4	3	3	3	3	28
P30	2	3	3	4	5	5	5	5	4	3	3	3	3	27
P40	2	3	3	4	5	5	5	5	4	3	2	2	3	26
P50	2	3	3	4	4	4	5	4	3	3	2	2	3	25
P60	2	2	3	4	4	4	5	4	3	2	2	2	3	24
P70	2	2	3	3	4	4	4	4	3	2	2	2	3	23
P80	1	1	2	3	4	4	4	4	3	2	1	1	3	21
P90	1	1	2	3	4	4	4	4	2	1	1	1	3	19

Trailing Suction Hopper Dredge

Limiting wave conditions Much easier to determine are the limiting wave conditions for the different type of vessels deployed during the project. However, in general, the limiting wave conditions per vessel is sensitive information and hence should not be shared publicly. In order to quantify the workable limits, generic wave limits will be used in stead.

Considering the operational activities of the trailing suction hopper dredge, i.e. dredging by trailing its dragheads over the sea bed, it is assumed that this activity is sensitive to severe wave conditions. Despite some active compensation mechanisms installed at the vessel, a rather conservative limit of $H_s = 2.5$ m will be imposed.

Workability The workability for the trailing suction hopper dredge can now be calculated, taking into account the limitation of $H_s = 2.5$ m. Table 5.3 summarizes the workable percentages per month.

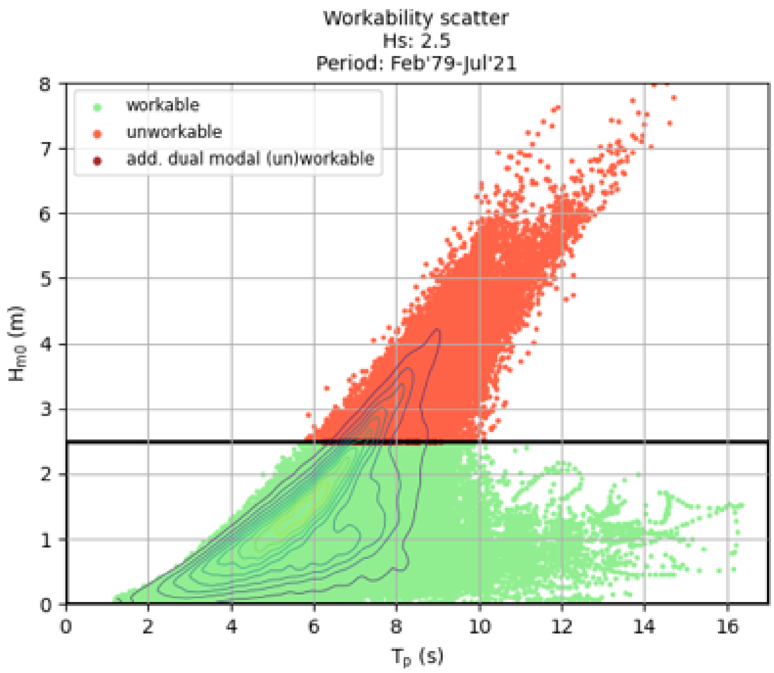


Figure 5.3: Workability scatter for the trailing suction hopper dredge deployment and activities, limited by a significant wave height of $H_s = 2.5$ m.

Table 5.3: Workability table for the trailing suction hopper dredge, given for several exceedance probabilities.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	71%	78%	84%	93%	96%	97%	97%	94%	88%	81%	77%	72%	86%
P10	90%	91%	95%	99%	100%	100%	100%	99%	96%	95%	91%	89%	89%
P20	85%	90%	94%	98%	99%	99%	100%	98%	95%	91%	86%	84%	88%
P30	80%	86%	90%	98%	98%	98%	100%	97%	93%	84%	84%	79%	87%
P40	78%	85%	88%	97%	97%	98%	99%	96%	91%	82%	81%	75%	87%
P50	73%	84%	85%	94%	97%	97%	98%	95%	89%	81%	78%	72%	86%
P60	68%	81%	83%	93%	96%	96%	97%	93%	88%	80%	75%	70%	85%
P70	63%	74%	80%	91%	96%	95%	95%	92%	86%	78%	70%	65%	84%
P80	61%	67%	75%	89%	95%	94%	94%	91%	84%	75%	63%	62%	83%
P90	51%	60%	71%	84%	93%	94%	92%	89%	78%	69%	61%	57%	83%

Fall Pipe Vessel

Limiting wave conditions Contrary to the trailing suction hopper dredge, the fall pipe vessels has no trailing parts that are vulnerable to severe wave conditions. The fall pipe hovers a substantial distance above the bed, which allows some more severe conditions than the TSHD. However, beyond a certain point the installation accuracy may drop significantly, resulting in inefficient deployment. Therefore, the limiting wave condition for the fall pipe vessel has been imposed at $H_s = 3.0$ m.

Workability Limited by a significant wave height of 3.0 m and no restriction on the wave period, the workable and unworkable moments in the hindcast data are shown in Figure 5.4. From this, the workability for the fall pipe vessel can be determined, which is listed per month in Table 5.4.

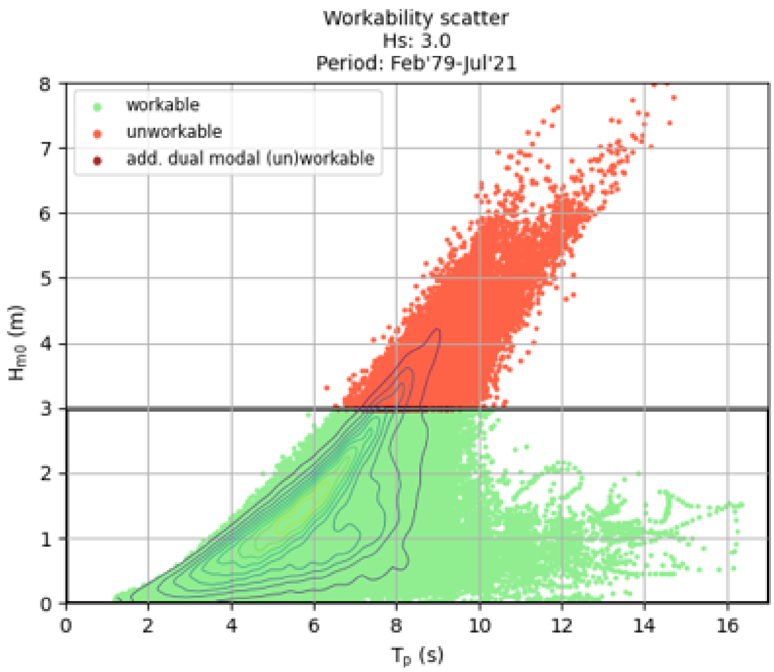


Figure 5.4: Workability scatter for the fall pipe vessel considering wave limiting conditions of $H_s = 3$ m.

Table 5.4: Workability table for the fall pipe vessel, given for several exceedance probabilities.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	83%	87%	92%	97%	99%	99%	99%	98%	95%	90%	87%	84%	92%
P10	95%	97%	99%	100%	100%	100%	100%	100%	99%	99%	97%	98%	94%
P20	93%	95%	98%	100%	100%	100%	100%	100%	98%	97%	95%	91%	94%
P30	90%	94%	95%	100%	100%	100%	100%	100%	98%	93%	93%	89%	94%
P40	88%	93%	94%	99%	100%	100%	100%	99%	97%	91%	90%	87%	93%
P50	85%	92%	92%	98%	99%	100%	100%	99%	96%	90%	89%	86%	92%
P60	83%	88%	91%	97%	99%	99%	99%	98%	95%	88%	86%	81%	92%
P70	78%	84%	89%	96%	99%	98%	98%	96%	94%	87%	81%	78%	91%
P80	73%	82%	86%	96%	98%	97%	97%	96%	91%	86%	80%	75%	91%
P90	65%	73%	83%	93%	97%	96%	96%	95%	88%	83%	75%	73%	90%

Side Stone Dumping Vessel

Limiting wave conditions In the current market, side stone dumping vessels are not used very often due to the fact that not many projects involve these sort of activities. That makes the estimation of the limiting wave conditions for the side stone dumping vessel harder. Considering the fact that these vessels are offshore vessels, there will be (almost) no limiting factor when sailing free. However, when installing the scour protection, the vessel dumps rock by pushing it overboard. When executing this operation during severe wave conditions, the accuracy of placement will drop substantially, resulting in an increase in losses. To that end, an installation limit has been imposed of $H_s = 2.5$ m.

Workability The limiting wave conditions of $H_s = 2.5$ m has been visualised based on the hindcast data in Figure 5.5. From this the workability for the side stone dumping vessel can be determined, and is listed in Table 5.5.

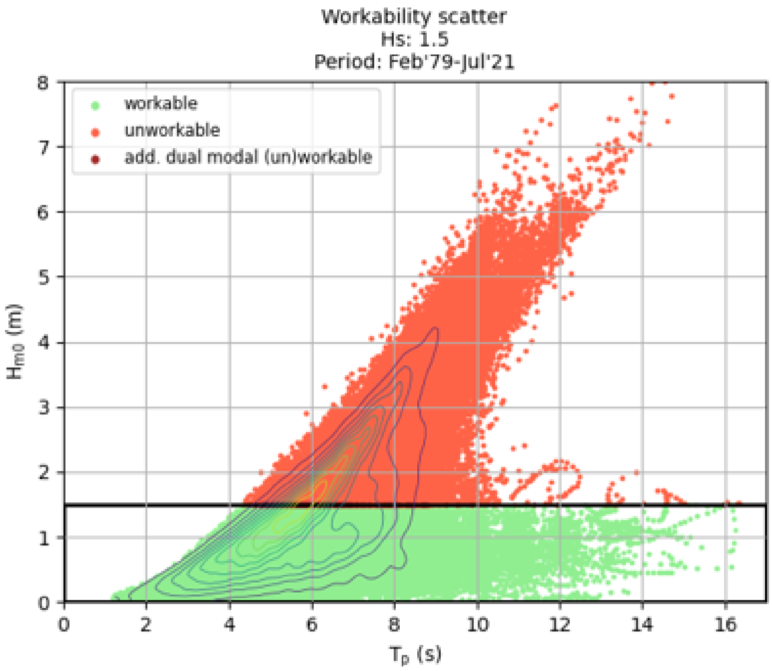


Figure 5.5: Workability scatter for the side stone dumping vessel, with a limiting significant wave height of $H_s = 1.5$ m.

Table 5.5: Workability table for the side stone dumping vessel, given for several exceedance probabilities.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	71%	78%	84%	93%	96%	97%	97%	94%	88%	81%	77%	72%	86%
P10	90%	91%	95%	99%	100%	100%	100%	99%	96%	95%	91%	89%	89%
P20	85%	90%	94%	98%	99%	99%	100%	98%	95%	91%	86%	84%	88%
P30	80%	86%	90%	98%	98%	98%	100%	97%	93%	84%	84%	79%	87%
P40	78%	85%	88%	97%	97%	98%	99%	96%	91%	82%	81%	75%	87%
P50	73%	84%	85%	94%	97%	97%	98%	95%	89%	81%	78%	72%	86%
P60	68%	81%	83%	93%	96%	96%	97%	93%	88%	80%	75%	70%	85%
P70	63%	74%	80%	91%	96%	95%	95%	92%	86%	78%	70%	65%	84%
P80	61%	67%	75%	89%	95%	94%	94%	91%	84%	75%	63%	62%	83%
P90	51%	60%	71%	84%	93%	94%	92%	89%	78%	69%	61%	57%	83%

Crane Vessel

Limiting wave conditions Lastly, the crane vessel that will be used to install the L-walls (or wave return walls) on top of the caisson units. During crane activities the wave conditions will make the vessel move, for instance yaw, pitch, roll and heave. Consequently, the load of the crane will directly or indirectly follow the movement of the vessel. This could potentially result in uncontrollable and undesired movement of the load. Therefore, a very strict limit of $H_s = 1.0$ m has been imposed as a wave limiting condition that must prevail for at least 12 hours, which is the time it takes to install the wave return wall for one caisson.

Workability The workability scatter for the crane vessel is shown in Figure 5.6 for a limiting wave height of $H_s = 1$ m and a duration of 12 hours. The corresponding workable windows are shown in table 5.6. Including a success rate of 50%, similar to the windows for the caisson installation, the window of opportunity reduces substantially, as can been seen in Table 5.7. Fortunately, considering the conservative P80 value, on average 23 wave return walls can be installed assuming a success rate of 50%, which would perfectly suits the caisson installation rate of 22 caissons per year.

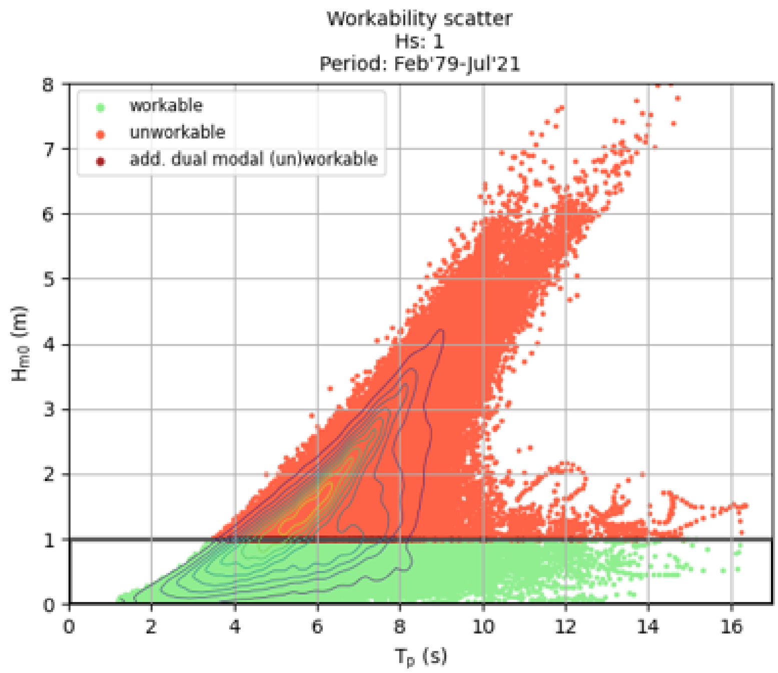


Figure 5.6: Workability scatter for the crane vessel, with a limiting significant wave height of $H_s = 1$ m.

Table 5.6: Number of total workable windows (12 hours) for the crane vessel (HLB) for the installation of the wave return wall with a limiting wave conditions of $H_s = 1$ m. The success rate has not yet been implemented.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Sum
Average	4	5	7	10	11	12	13	13	9	5	4	4	8	67
P10	9	9	14	13	15	16	20	18	14	9	7	8	10	96
P20	7	7	10	13	14	15	17	16	12	8	6	8	9	86
P30	5	6	8	12	13	14	15	14	11	6	5	5	9	79
P40	4	6	7	10	11	13	13	14	10	5	5	4	8	71
P50	4	4	7	9	10	12	12	13	10	4	3	3	8	66
P60	3	4	6	8	10	12	11	12	8	4	3	2	7	61
P70	2	3	5	8	9	11	11	10	6	3	2	2	7	55
P80	1	2	4	7	8	9	9	8	5	3	1	1	7	47
P90	0	1	3	6	5	6	7	7	5	1	1	0	7	37

Table 5.7: Expected number of installed wave return walls based on the workable windows for the crane vessel and including the success rate.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Sum
Average	2	2	4	5	5	6	7	6	5	2	2	2	4	34
P10	5	4	7	7	8	8	10	9	7	5	4	4	5	48
P20	4	4	5	6	7	8	8	8	6	4	3	4	4	43
P30	3	3	4	6	7	7	8	7	6	3	3	3	4	40
P40	2	3	4	5	6	7	7	7	5	3	2	2	4	36
P50	2	2	4	5	5	6	6	7	5	2	2	2	4	33
P60	2	2	3	4	5	6	6	6	4	2	2	1	4	31
P70	1	2	3	4	5	6	6	5	3	2	1	1	4	27
P80	1	1	2	4	4	5	5	4	3	2	1	1	4	23
P90	0	1	2	3	3	3	4	4	3	1	1	0	3	18

5.2. Conclusion

The aim of this section was to include probabilistic in the form of weather conditions in combination with the limiting conditions (either wind or wave) for the construction activities, thereby composing a more realistic and comprehensive planning which results in more accurate determination of the construction costs.

First, the limiting conditions per vessel category has been defined after which the workable hours and subsequently the workability could be determined (using the Viktor workability tool). This was then the starting point for the probabilistic planning.

From this analysis, it turns out that the most vital construction task, the caisson installation, varies between 18 and 35 caissons per year - considering the P90 and P10 respectively - whereas previous it was assumed to be 33 caissons per year. Generally, a conservative value of P70 will be assumed, to take into account unfavorable years, resulting in 23 caisson installations per year.

According to the deterministic planning 33 caissons could have been installed every year, but when including weather conditions and the limiting characteristics of the offshore vessels, a reduction of 33% has been found, resulting in an installation rate of 23 caissons per year. This means that, assuming a constant installation rate of 23 caissons per year, the total project duration would increase from 8 years (based on the 33 caissons per year) to 12 years.

The workability of the other vessels do not have a large impact on the planning compared to the installation of the caisson, as these percentages vary between roughly 80% - 100%. Therefore, the downtime for these activities is minor with respect to the time lost by the caisson installation, i.e. these activities can easily be executed during the down time of the caisson installation.

6 Cost-effectiveness assessment

The cost-effectiveness analysis is a decision-making tool assessment used in the business and project management sector to evaluate the potential costs and benefits of a particular project or action. It involves the identification and quantification of the costs and benefits regarding a specific project, while also taking into account various aspects such as interest rate and the lifetime of the structure. The aim of such an assessment is to determine whether the benefits of a particular decision outweigh the associated costs and hence is economically feasible. This tool is useful in making rational, well-informed decisions that maximize value and minimize risks.

Especially for an offshore structure a cost-effectiveness analysis is important because it involves significant capital expenditures that require large investments in equipment, materials and labour. Moreover, the risk involved in offshore projects can be substantial, especially when considering a brand new concept like an offshore PSH plant, subsequently raising the expected losses and therefore the costs. To that end, the cost-effectiveness assessment should provide guidance and necessary information to make confident decisions related to the construction sequence and its optimization.

A cost-effectiveness assessments related to energy storage can be challenging due to the wide variety of available energy storing technologies - possessing different cost and performance characteristics and the varying requirements of storage applications. Schmidt, Melchior, Hawkes, and Staffell (2019) suggests that the application of the levelised cost of storage (LCOS) for energy storage plants is a suitable and appropriate methodology that includes the differences in storage technology and associated requirements. Moreover, by applying LCOS to the offshore PSH plant the outcome of the cost-effectiveness assessment can be compared to alternative energy storage techniques.

6.1. Levelised cost of storage (LCOS)

The Levelised Cost of Storage (LCOS) is a financial metric used to evaluate the cost-effectiveness of various electricity and energy storage technologies, based on the total costs and production during the total ownership. It measures the total cost of an energy storage system with respect to the total generated energy, including costs related to capital investment, operation and maintenance, charging and end-of-life. Moreover, the expenditures are expressed in their Net Present Value (NPV), which involves the interest rate and the lifetime of the structure. The expression for LCOS is given by Equation 6.1.

$$\text{LCOS}\left[\frac{€}{MWh}\right] = \frac{\sum_n^N \frac{\text{Investment cost}}{(1+r)^n} + \sum_n^N \frac{\text{Risk cost}}{(1+r)^n} + \sum_n^N \frac{\text{O\&M cost}}{(1+r)^n} + \sum_n^N \frac{\text{Charging cost}}{(1+r)^n} + \sum_n^N \frac{\text{End-of-life cost}}{(1+r)^n}}{\sum_n^N \frac{\text{Elec}_{Discharged}}{(1+r)^n}} \tag{6.1}$$

where:

- LCOS [€/MWh] = Levelised cost of storage.
- Investment cost [€] = Investment cost for the structure including all technologies.
- Operation and Maintenance cost (O&M) [€] = Costs related to operational services and maintenance.
- Risk cost [€] = Expenditures associated with the risks during execution and operational phase of the structure.
- Charging cost [€] = Expenses related to storing electricity.
- End-of-life cost [€] = Costs related to the end-of-life of the structure, e.g. demolition, recycling, reuse, etc.
- $\text{Elec}_{Discharged}$ [MWh] = Electricity discharged during the investment period.
- n [years] = Sum of expenditures are given for a given year.

- N [years] = Lifetime of the structure.
- r [-] = Discount rate.

Although this project would involve many other costs, it's impossible to capture all expenditures and hence the most important or relevant are considered, to come up with a comprehensive and realistic cost indexation. It is worth mentioning that the following costs are not included in the levelised cost of storage approach:

- General costs, among them are bathymetry surveys, EXO clearance, soil investigation, environmental and social impact assessments, administration, quality control, construction management and many others.
- Taxes and fees.
- Permit costs.

Intermezzo - Future Value (FV) and Present Value (PV)

Future Value (FV) and Present Value (PV) are two financial concepts that refer to the future and current worth of a future cash flow or expenditure, respectively. The future value is the value of a current asset or cash flow at a future date, while taking into account the accumulation of value due to compounding aspect at a specific interest or growth rate. For example, this could refer to the future value of your current personal savings on a bank account, given a guaranteed interest rate. The Future Value (FV) given an investment I, interest rate r and period T, can be determined with:

$$FV = I(1+r)^T$$

The present value is the value that refer to the current worth of a future cash flow or asset discounted at a specific rate of return, also known as the discount rate. Contrary to the future value, the present value is based on the principle that the value of money decreases over time, for instance due to inflation. For example, the present value of an expenditure at a future date will be less when taking into account inflation. The Present Value (PV) of an asset given the Future Value (FV), interest rate r and period T, can be determined with:

$$PV = \frac{FV}{(1+r)^T}$$

By calculating the present value of projects, the real values can be expressed in their current worth, thereby making it possible to compare projects with different duration. The concept of present and future value reflects on the current and future value of money, and should convince that today's money is worth the investment in a certain project.

Investment cost

Investment costs refers to the initial expensive associated with acquiring, construction, installation and commissioning of the new project. In general, these expenses include cost related to construction materials, equipment, labour, permits, additional site investigations, design and other expenses dedicated to the development of the project. Depending on the scale and complexity of the project, the investment cost can widely vary.

Especially for offshore projects the investment costs can be substantially due to the added complexity and challenges associated with offshore operations. Additionally, mobilizing equipment and personnel to offshore sites increase the costs significantly, as well as the fact that the workability of vessel-based equipment is substantially less than onshore equipment, therefore extending the overall project duration.

The investment costs are divided into the following expenditures:

- Material costs for the caisson dam
- Turbine costs
- Powerhouse costs

- Equipment costs

Material costs for the caisson dam
The material costs capture all expenditures that are related to the construction materials, for instance the costs related to concrete and reinforcement bars for the caisson unit and wave return wall but also the labour required to fabricate the elements. Despite the fact that the real material and labour costs are unknown, a qualitative estimate can be derived based on the required quantity per category (concrete, reinforcement bars and labour) and the price per unit.

A cost estimate has been made based on the used volume per product type (i.e. concrete, steel and labor). This calculation is shown in Appendix I. Summarizing, the total cost for one caisson unit is determined at € 2.9 million, with an additional € 0.9 million for the wave return wall.

Furthermore, the expenditures related to the rock works are estimated on € 3.4 million per caisson unit length. This only includes the material costs and transportation to site. An extensive overview is given in Appendix I.

Turbine costs
Since the concept of a low-head pumped storage hydropower plant is relatively new, the turbines that will be used for this sort of hydropower plants has not been extensively designed and developed compared to the traditional hydropower turbines. Normally, in case of a high-head pumped storage hydropower plant, the high water pressure or high flow velocity in turbine causes the blades or buckets to spin the turbine's shaft, thereby converting the potential energy into kinetic energy.

An extensive analysis of the turbine's costs can be found in Appendix I. From this assessment, the costs are estimated on € 1 million/MW. This means that for the proposed turbines (10 MW) the cost per turbine will be € 10 million each, with a total sum of € 2 billion for all turbines.

Powerhouse costs
The powerhouse costs are after the material costs the largest expenditure regarding the capital investment and hence should be taken into account for the levelised cost of storage assessment. As has been discussed earlier, the powerhouse is the civil structure in the dam that accommodates the hydropower turbines, a possible control and operation room, additional repair storage and facilities but also all electrical infrastructure that is necessary for proper operation.

Appendix I presents an overview of the costs calculation for the powerhouse section. This has resulted in a total material cost of € 2.8 million per powerhouse unit.

Total material costs
Including all material costs mentioned in this section, the total expenditures would sum up to € 4.2 billion. For construction projects in general, it is rare to account for all expenditures with regard to material cost in the first year, instead these costs are spread out over difference phases or period of the project, depending on the project size, construction timeline and payment terms. To simplify the exact payment timeline, it is assumed that the material costs are spread out over the entire project duration evenly, resulting in a constant cash flow (regarding the material costs) over time. Table 6.1 summarizes all material costs for the project, including the caisson dam, powerhouse and turbine costs.

Equipment costs
Equipment costs refer to all expenditures associated with the dedicated equipment required for the preparation, execution, operation and maintenance of the structure. Examples of these type of expenditures are the chartering of specialized machinery such as heavy lifting cranes or diving systems. Generally, in construction projects equipment costs are a substantial part of the overall cost, particularly for offshore projects for which specialized equipment is required.

Since the dredging and marine market is a very competitive market, actual charter costs for dredging equipment is classified and hence sensitive information for the public. Although the actual daily rates can not be used, the cost of dredging equipment has also been estimated by CIRIA (2009) and provides an

Element	Units	Cost per unit	Total cost	Cash flow
Caisson unit		€ 2 864 000	€ 607 million	€ 60 million
Wave return wall	212	€ 852 000	€ 180 million	€ 18 million
Rock works		€ 3 414 600	€ 723 million	€ 72 million
Powerhouse elements	200	€ 2 808 500	€ 561 million	€ 56 million
Scour/Bed protection	N/A	N/A	€ 86 million	€ 9 million
Turbine	200	€ 10 million	€ 2 billion	€ 40 million
Total			€ 4.2 billion	€ 420 million

Table 6.1: Total material cost for the concrete and rock elements of the structure and the associated cash flows spread out over a payment period of 10 years.

indication of the expected costs.

Appendix I extensively elaborates the determination of the operational and stand-by rates per vessel type. Here, a concise summary will be presented, and is shown in Table 6.2.

Vessel type	Installed power	Capacity	D + i	M + R
Trailing Suction Hopper Dredge (TSHD)	23 000 hp	17 000 m^3	€ 503 490	€ 503 490
Fall Pipe Vessel (FPV)	19 000 hp	15 000 ton	€ 528 000	€ 528 000
Side Stone Dumping Vessel (SSDV)	10 000 hp	5000 ton	€ 364 795	€ 364 795
Heavy Lifting Barge (HLB)	15 000 hp	600 / 1000 mT	€ 264 000	€ 264 000
Anchor Handling Tug (AHT)	7000 hp	116 t bollard pull	€ 38 928	€ 38 928
Towing tugs	6000 hp	91 t bollard pull	€ 33 367	€ 33 367
Dock workers	20 workers	N/A	€ 70 000	€ 70 000
Auxiliary equipment	N/A	N/A	€ 100 000	€ 100 000

Table 6.2: Characteristics for different vessel types, with an indication of the weekly operational and stand-by rates based on the expenses of the TSHD. Rates are expressed in euros per week. Abbreviations: D = Depreciation of the vessel, i = Interest costs, M = Maintenance costs and R = Repair costs. It is assumed that D + i are the fixed costs and M + R are incremental operational costs.

Construction risk

Moreover, the investment costs also include the expenditures related to the possible risks that are present. Generally, for onshore projects this contribution is not significant compared to the total investment costs. However, for an offshore project of this scale, the costs associated to the risk can be substantially. Therefore it might be beneficial to adjust the construction sequence or design to lower the probability of occurrence or the consequences when failing. In general, risk is defined as the likely hood of an occurrence times the severity of the occurrence, and can be formulated as follows (Kaplan & Garrick, 1981):

> Risk is a set of scenarios (i_s), each of which has a probability (p_i) and a consequence (d_i)

The unit risk depends on the units probability and consequence. Generally, the probability of an occurrence is often expressed as the probability per unit time, for example per year. The consequence of an event is often complex to determine, as it includes tangible and intangible assets, for instance loss of material, project delay, ecological damage, injuries and fatalities. The determination of these consequences contain a high uncertainty since the impact of an event is hard to estimate. Considering engineering consequences, the consequences are often expressed in terms of monetary value (€). Risk can then be expressed as follows:

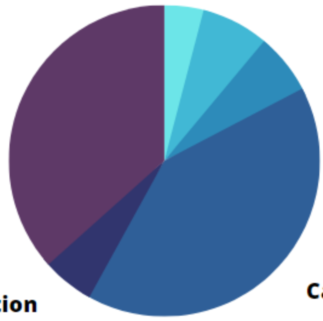
$$E(d) = p_i \cdot d_i \tag{6.2}$$

where:
- $E(d)$ [€/year] = Risk associated with an event.
- p_i [1/year] = Probability of occurrence.
- d_i [€] = Consequence of the event.
- i [-] = Scenario event.

For this project different type of scenarios are considered that could occur during construction works, and are related to incidents and accidents, also known as operational risks. This means that other risks such as an unforeseen substantial increase in prices of steel, fuel or concrete, legal risks, environmental risks and strategic risks are assumed to be beyond the scope of this research, and hence are not considered.

Appendix I contains a comprehensive outline of the considered events and occurrences prior, during and after the construction of the caisson dam. In this appendix, all risks that are considered to be relevant are described for every distinct construction phase, starting with the sea bed preparation and ending with the placed stone revetment on the inner berm. For every event the consequence has been determined, consisting of a monetary value that is associated with potential extra work (salvage, repair and others) and the value that has been lost.

Figure 6.1: Visualisation of the risk costs composition.



Furthermore, besides the direct costs associated with repair works, replacement and extra deployment of vessels and personnel, an incident might also influence the project planning substantially. Depending on the event, it is assumed that the project can be delayed by maximum 4 weeks in case of total loss of a caisson unit while installing at the project site. This delay takes into account the extra salvage works. In total, the final costs per event can be determined based on the direct and indirect costs (i.e. material and delay costs) and is summarized in Table 6.3. This Table also includes the probability of occurrence of the considered event and the corresponding risk expressed in € per caisson unit. It turns out that an additional € 0.6 million is required per caisson unit when incorporating the relevant risks.

Besides the direct installation and transport risks, other relevant risk cost for a construction project are related to permits, environmental, delay, unforeseen conditions, insurance expenditures. It is assumed that these risk costs add up to € 0.35 million per caisson, which leads to a total risk cost of € 265 million for the total project. Neglecting the turbine expenditures, this is roughly 12% of the investment costs.

Operation and Maintenance cost

The operation and maintenance costs of a pumped stored hydropower plant refer to the ongoing expenses associated with running and maintaining the plant. These costs include salaries, equipment repair and replacement, site upkeep, and other expenses. Factors such as plant size, complexity, labor costs, and electricity demand can influence these costs, but also the accessibility of the plant itself. Due to the offshore characteristic of this project, the O&M costs may be higher compared to conventional PSH plants.

Hydropower plant utilities The estimation of O&M expenditures for hydropower plants and pumped storage hydropower plants turns out to provide a proper and reliable indication of the expected costs. So far, many studies focusing on the costs associated with hydropower and PSH, among them the operation and maintenance costs, have been conducted, finding similar results. According to International Renewable Energy Agency (IRENA) (2012) hydropower plants can be divided into three categories: large hydro plant, small hydro plant and refurbished/upgraded plant. The corresponding O&M costs vary between 1% - 6% per year of the total installed costs, i.e. the turbine and powerhouse costs, neglecting the caisson dam costs. In line with these findings, a study conducted by IEA-ETSAP (2010) finds that the O&M costs are within 1.5% and 2.5% of the investment costs per year.

Since none of the studied hydropower and PSH plants is offshore, the actual expenditures will probably be higher due to external factors like the harsh ocean environment and accessibility. On the other hand, these studies form a basis for the estimated O&M costs for the offshore PSH plant. So, taking everything

Category	Event	Total costs	Probability	Risk costs
Sea bed preparation	Unforeseen downtime	€ 0.25 mil	5%	15 000
	UXO clearance	€ 100 000	10%	10 000
Rock foundation layer	Unforeseen downtime	€ 0.2 mil	5%	10 000
	Sediment spill	€ 0.33 mil	10%	33 000
Caisson towage	Winch breaks	€ 0.15 mil	2%	2500
	Winch foundation failure	€ 0.55 mil	1%	5000
	Caisson sinks	€ 5.5 mil	0.5%	26 000
	Other	€ 0.1 mil	5%	5000
Caisson installation	Damage foundation / caisson	€ 1.1 mil	5%	55 000
	Foundation erosion	€ 0.55 mil	5%	50 000
	Capsizing	€ 1.9 mil	1%	20 000
	Sliding	€ 1.9 mil	5%	100 000
	Joint construction	€ 0.5 mil	5%	25 000
L-wall installation	Wind breaks	€ 0.7 mil	2%	15 000
	Anchor failure	€ 1 mil	1%	10 000
	Total loss	€ 3 mill	0.1%	3000
	Other	€ 0.5 mil	1%	5000
Inner berm	Unforeseen losses	€ 0.25 mil	10%	25 000
	Erosion	€ 0.6 mil	30%	150 000
	Other	€ 0.5 mil	10%	50 000
Total risk costs				€ 0.6 million

Table 6.3: Costs associated with risk during construction, transportation and other sequences during construction, expressed per caisson unit. The costs related to the project delay has been determined at per activity based on the impact on the planning and subsequently the number of vessels that must be postponed.

into consideration, the O&M costs are estimated on 4% per year of the total investment costs of the plant's turbine and powerhouse (€ 2.8 billion), resulting in nearly € 55 million per year.

Caisson dam Additionally, these studies consider the O&M costs with regard to the hydropower plant and its electrical infrastructure (turbines, power lines, etc.), whereas these studies only focuses on the utility costs, not considering the caisson dam itself. To that end, the O&M for the caisson dam must be added to the O&M expenditure for the hydropower plant. Although the maintenance costs with regard to marine civil structures are not made public easily, John et al. (2014) suggests that for maritime structures a maintenance and operation cost of 1% of the construction costs is appropriate.

Moreover, after research of Wagner (2004) into the costs of armour unit breakwaters, the constructed breakwaters have used a value of 2% of the initial construction costs for the annual maintenance costs. Considering the fact that a caisson type breakwater will probably be less maintenance intensive, the previous suggested 1% of the construction costs is appropriate. The total costs of the caisson dam, neglecting the powerhouse elements, are € 1.6 billion. resulting in € 16 million per year.

Electrical discharge

The electrical discharge of the PSH plant refers to the energy yield by discharging sea water into the reservoir. In the LCOS metric, the cost of storage is expressed per unit energy (MWh), thereby the method offers the opportunity to compare different energy storage methods.

Appendix I elaborates on the exact determination of the gross and net capacity of the reservoir, taking into account important aspects such as water level differences and Depth of Discharge (DoD). The following paragraphs will only briefly present the required parameters that are necessary for the LCOS.

Although the total energy storage capacity of the reservoir is 18.5 GWh (including the depth of discharge), the daily yield will be considerably less. This is due to working principle and the goal of this facility. The PSH plant aims to overcome the daily energy gap which is evoked by intermittency of renewable energies. This implies that the facility will generate energy during a small window: only when renewable energy resources could not answer the demand.

Therefore, it is assumed that the utility rate, i.e. the fraction during which the facility generates electricity, is only 20% or 5 hours per day (which may be slightly high compared to other PSH plants according to Kougias and Szabó (2017)). To estimate the daily electric discharge, Equation 6.3 has been applied.

$$E_{dis,daily} = S_{net} \cdot \eta_t \cdot \eta_p \cdot DoD \cdot T \tag{6.3}$$

in which:
- $E_{dis,daily}$ [MWh/year] = Daily electrical discharge
- S_{tot} [MWh] = Total reservoir storage capacity
- η_t [%] = Turbine efficiency
- η_p [%] = Facility's utility rate
- DoD [%] = Depth of discharge
- T [hours] = Number of hours in one day

Based on the latter equation, the electrical discharge can be estimated. Table 6.4 lists all relevant parameters for the daily electrical discharge. Following this approach, the daily electrical discharge has been determined at 6.8 GWh/day. To put this number into perspective, this would be the equivalent of the power consumption of 900 000 households with an average consumption of 7.6 kWh per day (City Centre Retreat, 2022).

Category	Symbol	Value	
Total storage	S_{tot}	23	GWh
Turbine efficiency	η_t	70	%
Facility's utility rate	η_p	20	%
Depth of Discharge	DoD	80	%
Daily electrical discharge	$E_{dis,daily}$	6.8	GWh/day

Table 6.4: Reservoir's parameters to determine the daily and annual electrical discharge.

Charging cost and Annual yield

The charging cost of a pumped stored hydropower plant is the cost associated with pumping water from the reservoir to the ocean, where it is stored as potential energy until it is released to generate electricity. The cost related to pumping water depends on the actual price of electricity, which is used in charging the PSH plant, as well as turbine efficiency and storage processes.

Typically, a PSH plant charges during periods of low electricity demand when the cost of electricity is low and releases the stored water during periods of high demand when the cost of electricity is high. In line with the renewable energy market, charging would occur during periods with high solar or wind energy yield. The charging cost of a PSH plant is often less than that of other storage technologies for long storage periods, making it an attractive option for governments and energy companies which need a reliable, cost-effective, and environmentally friendly energy source.

Generally, the turbine, market and plant's characteristics determine the charging cost and annual yield (MWh per year) of a (pumped storage) hydropower plant. Although the plant is still in the conceptual phase and therefore many parameters are unknown or in a low level of detail, a first estimate of the costs, yield and hence the feasibility or levelised cost can be determined based on coarse approximations or assumptions.

Turbine efficiency In pumped storage hydropower plants, turbine efficiency plays a significant role in the overall efficiency of the system, as the turbines must not only extract the potential energy from the water but also pump the water to a higher reservoir (from the reservoir to the sea). Regularly, PSH plants store water with a head difference > 100 m (especially in mountainous terrain this could easily be heightened to 200 - 300 m) using special turbines that are specially tailored for these conditions.

For the considered offshore PSH, the maximum height difference is a lean 30 m, only a small fraction of the potentials found in the Alps. Due to this difference, the working principle of energy capture for the turbines is different than seen for the high head differences, implying that the turbines used in these terrains would not be suitable for these low-head situations. Therefore, a low-head turbine must be researched and developed in order to capture potential energy efficient.

The Alpheus project does extensive research into low-head turbines, as this is the major hurdle to overcome within the research project. One of the concepts investigated in the Alpheus project is an axial flow, shaft-driven, reversible contra-rotating pump-turbine (CRPT). A study (not related to project Alpheus) dedicated to the blade design optimization of the reversible contra-rotating pump-turbine, found by experimental study that the evaluated pump-turbine efficiencies are 77.03% and 81.26% in the pump and turbine modes respectively (Kim et al., 2018). Simplifying this aspects, but also to incorporate some uncertainty due to the lack in development, the efficiency is assumed to be 70% for both the turbine and pump function.

Electricity prices Charging of the reservoir or imposing the head difference over the dam, thereby creating the energy potential in the reservoir, requires electricity to pump water from the lower to the higher reservoir. Consequently, electricity prices determine the charging cost of the PSH plant. Moreover, considering the functionalities of the plant (that is to overcome the intermittent energy gap and flatten-out the small peaks in supply and demand to stabilize the electricity grid) the moment of charging also plays a key role with regard to the charging costs.

Nowadays, due to the drastic increase in renewable energy resources (with high intermittency), electricity prices have a high volatility, implying that the difference between the lowest and highest daily price is substantially and can even be larger than € 100/MWh. Furthermore, during favorable weather conditions for either wind turbines or PV cells, the lowest daily price is often less than € 0/MWh, which would imply that charging the PSH plant would be free of cost.

After extensive literature research, which can be found in Appendix I, the future electricity price can be estimated. For this estimate, it is assumed that the reservoir can be charged during the 50% cheapest hours per day. This means that 50% of the time, the electricity price would be higher. According to this reasoning, the electricity price would be € 10/MWh.

End-of-life cost

The end-of-life cost of a pumped stored hydropower plant is the cost associated with decommissioning and removing the plant from its location at the end of its useful lifespan. This cost includes the removal of all equipment, structures, and materials associated with the plant, as well as additional costs related to demolition, reuse or recycling of the materials. Considering the scale of the PSH, the end-of-life cost can be significant and depends on factors such as the plant's location and size, the materials used in its construction, and the environmental impact of the plant's decommissioning. Proper planning and timely decommissioning can help to minimize the end-of-life cost and environmental impact of a pumped stored hydropower plant. It is important to note that the end-of-life cost is a consideration in the overall life cycle cost of the project, which also includes the cost of construction, operation, and maintenance.

The removal and re-use or demolition of the caissons is considered to be the major expenditure regarding the end-of-life cost. Although many parameters determine the complexity of the decommissioning of the units - among them the end-of-life state of the structural strength of the concrete and reinforcement steel of the caissons itself - it is hard to estimate the associated costs. Especially when considering the fact that caissons in general are not designed with the idea to be uplifted and transported after their design lifetime.

A study dedicated to find an alternative purpose or destination for the caissons that are part of an existing quay wall in the Port of Rotterdam, constructed between 1950 and 1961, also had to incorporate the decommissioning costs for the caisson units (Danad, 2014). For the deconstruction and demolition costs, many parameters have been included, among them are expenditures associated with the removal of auxiliary utilities (fenders, railings, navigation aids, etc.), removal of the ballast material and the transportation costs to a discharge area, excavation of the inner area, the demolition of the caissons and relevant risks during this process. The total decommissioning costs for the quay wall (approximately 750 m long) was estimated on € 4.2 million (€ 5.6 million per km), of which 35% consists of the excavation of the backfill located at the inner side of the caisson.

Relating the latter research to this project, the conditions are completely different. Whereas the Port of Rotterdam is a workable sheltered port, offshore weather conditions prevail for the PSH, a harsh working environment. However, as a first estimate the decommissioning costs found in the study (Danad, 2014) will be applied to this conceptual project, additionally compensated for the fact that offshore conditions prevail.

The offshore location of the decommissioning entails that i). offshore equipment is necessary, whereas for the Port of Rotterdam mostly land equipment could be used and ii). due to the offshore conditions, the workability is significantly less than working on land. Taking all this into consideration, as a first estimate an increase in decommissioning costs of 100% is assumed. Subsequently, the decommissioning costs will be € 11.2 million per km caisson dam, which results in the total decommissioning costs of € 174 million for the entire structure.

Social Discount Rate (SDR)
The present value (PV) concept captures the time value of money by making compounding or discounting adjustments to the cash flows, to reflect on the increase in cost due to inflation. To include the time value of money, a discount rate is applied. The discount rate is defined as the percentage rate that is required to calculate the present value of future cash flow, which can be positive and negative, and is applied to convert future costs to a comparable time base. Hence, the discount rate can be seen as a factor that reflects the time value of money. For project that are associated with society in general (social project), the Social Discount Rate (SDR) can be applied.

The social discount rate can be used in case a project is (partly) dedicated to the society, subsequently improving the overall welfare. Often these projects are expensive as they involve many parties, have a large impact on the society and are mostly supported or funded by the government or municipality. For instance, projects associated with societal improvement are infrastructural projects, service provisions (healthcare) or energy supply projects.

Since the construction of an offshore pumped storage hydropower plant will provide energy, this project can be considered as a societal project. The value of the social discount rate depends on the risk that is involved in the project, resulting in high discount rates for high risk projects and low discount rates for low risk projects.

As the social discount rate depends on many project specific parameters, literature only can provide a range of the discount rate. Therefore, it is assumed that the social discount rate will be 5%.

6.2. Results

Based on all expenditures discussed above the LCOS for the offshore PSH can be determined. Assuming a payment period of 10 years for the Capital Expenditures (CAPEX), consisting of the material costs, turbine costs and powerhouse costs (i.e. the total costs of these expenditures will be spread out over a period of 10 years), the annual cash flow associated with these CAPEX costs is approximately € 400 million. The yearly cash flow related to the equipment costs are roughly € 46 million and € 17 million for the stand-by costs and incremental operation costs respectively. Finally the risk costs that include all risks associated with the caisson and scour protection result in € 13.8 million annual.

All expenditures and cash flows (CAPEX and Operational Expenditures (OPEX)) are listed in the comprehensive overview - which lists all costs categories for the first 10 years - given in Table N.1 (Note that this Table must be updated with the most recent expenditures). A more concise overview, which only summarizes the total present value for each category, is shown in Table 6.5, which lists both the sum of Future Value (FV) and Present Value (PV) per category. Assuming a discount rate of 5% (which is in line with a general interest rate for large scale social projects) and a lifetime of 100 years, the levelised cost of storage for the offshore pumped storage hydropower plant is only € 212 MWh^{-1}.

Only considering the construction costs, thereby neglecting future operational expenditures (OPEX) related to O&M, charging and end-of-life costs, the major contributor to the total cost are the turbine costs (almost 40%), after which the powerhouse costs follow (approximately 11%). The total cost distribution is shown in Figure 6.2.

Category	Total cost	Present Value (PV)
Caisson costs	€ 607 million	€ 3.8 billion
Rock works	€ 723 million	
Wave return wall	€ 180 million	
Turbine costs	€ 2 billion	
Powerhouse costs	€ 561 million	
Scour protection	€ 86 million	
Stand-by costs	€ 461 million	
Operational costs	€ 159 million	
Subtotal	€ 4.8 billion	
Risk costs	€ 127 million	€ 105 million
O&M costs	€ 10.3 billion	€ 1.3 billion
Charging costs	€ 4.5 billion	€ 0.56 billion
End-of-life costs	€ 120 million	€ 1 million
Subtotal	€ 15.0 billion	€ 2.0 billion
Total costs	€ 19.8 billion	€ 5.8 billion
Electrical discharge	2.2×10^8 MWh	2.73×10^7 MWh
LCOS	€ 212 MWh^{-1}	

Table 6.5: Concise overview for all categories considered in the LCOS, expressed in their Future Value (FV). Taking into account the year of expenditure and the discount rate, the LCOS can be determined.

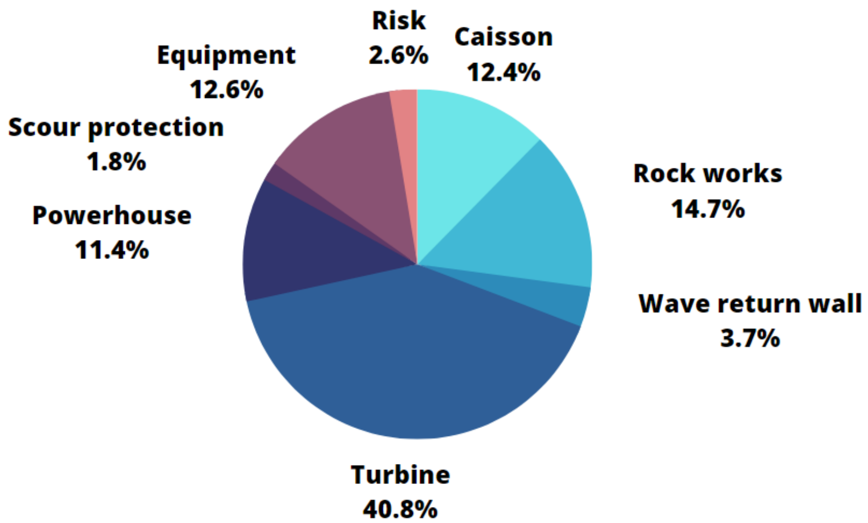


Figure 6.2: Cost distribution of the construction costs (ignoring future operational, maintenance and charging expenditures) for the offshore PSH, expressed in its Future Value (FV) (not discounted yet).

6.3. Comparison

The levelised cost of storage is a financial metric often used to indicate the cost effectiveness of an energy storage technology or facility. Therefore, in order to qualitatively compare the LCOS of the conceptual offshore PSH plant with other methods, the generic costs for other facilities must be known.

The development of LCOS for 9 distinct energy storage technologies, applied in 12 power systems, has been analysed and predicted by Schmidt et al. (2019). In this research, the expected cost effectiveness of the investment has been determined for currently common energy storage techniques such as pumped storage hydropower, but also considers rather new and non-implemented methods like compressed air. It distinguishes applications based on the annual cycles of the plant, the installed power and the energy discharge time.

The development of the LCOS over time for currently competitive energy storage technologies have been determined by Schmidt et al. (2019). For four technologies (pumped hydro, flywheel, lithium-ion and vanadium redox-flow) the expected LCOS has been visualised and is shown in Figure 6.3. Unfortunately, hydrogen storage has not been explicitly analysed.

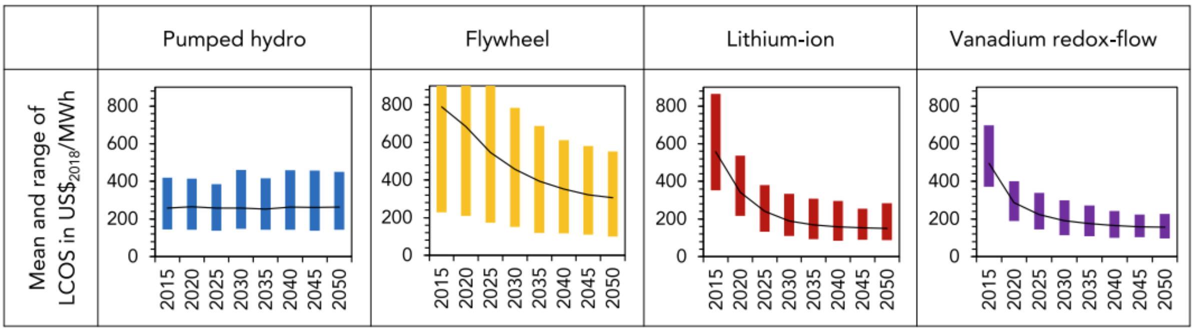


Figure 6.3: Results of a Monte-Carlo analysis for the development of levelised cost of storage for four energy storage technologies, for the period 2015 - 2050.

It is expected that the LCOS of pumped hydro will be more or less stable over the years, as only little improvements in this technology can increase the productivity or decrease the costs. The LCOS for pumped hydro varies between 180 - 400 US$/MWh for the entire modelling period. For the lithium-ion technology, where a major improvement with regard to the cost-effectiveness is expected in the coming years, reducing the LCOS from more than 600 US$/MWh in 2015 to only 200 US$/MWh in 2050.

On the other hand, Jülch (2016) found a LCOS of approximately 80 $/MWh for conventional pumped hydropower storage, which contradict the higher modeled value mentioned in the previous paragraph. In addition, the LCOS for lithium-ion storage has been estimated at 230 - 370 $/MWh, however since battery storage facilities have not been built on this scale, this value has been based on future values.

A fairly new candidate in the electricity (storage) market is the hydrogen storage. Due to the new appearance of this technology, it lacks extensive developments of the electrolyzers and fuel cells (necessary for the convertion of electricity to hydrogen and vice versa). Currently, hydrogen is often used as a conceptual technique, mostly used for comparison with others. However, the production of green hydrogen looks still promising but not yet feasible.

One of the problems is that the technology is rather expensive compared to other energy generating and storage methods. A study to the levelised cost of storage was executed by Urs, Chadly, Al Sumaiti, and Mayyas (2023), which made a conceptual design and business plan for a hydrogen power plant in the United Arab Emirates (UAE), a perfect location considering the high solar radiation and many sun hours. The estimate for the levelised cost of electricity was 0.248 US$/kWh, equivalent of 248 US$/MWh. with a range between 188 $/MWh and 288 $/MWh. However, this value is considered to be very favorable compared to other analyses.

For instance, Mayyas, Wei, and Levis (2020) conducted an extensive research into the LCOS of hydrogen storage for several scenarios in the US. Their results show that the LCOS can be as large as 1890 US$/MWh, depending on the developments in the electricity market. This shows that although the CAPEX for the offshore PSH are substantial, the LCOS are relatively low compared to hydrogen.

Comparing the conceptual offshore PSH, with a LCOS of approximately € 210 /MWh, with the existing energy storing technologies, it would be a relatively competitive energy storage plant in the current market. However, considering the large CAPEX costs of almost € 5 billion (ignoring the OPEX worth € 15 billion), in combination with high uncertainties involved in many aspects (varying from geotechnical and design parameters to workability windows and facility operation parameters), the offshore PSH may not be as competitive as the LCOS would suggest.

6.4. Sensitivity analysis

The Levelised Cost of Storage (LCOS) depends on many parameters, among them are expenditures like CAPEX and OPEX, but also more subtle factors like the (social) discount rate and future electricity price. To create more insight in the dependencies and the system's response on changes of these elements, this subsection will discuss multiple simple sensitivity analyses.

The sensitivity of the LCOS with regard to the Operational Expenses (OPEX) and Capital Expenditures (CAPEX) is shown in Table 6.6. This shows that the LCOS is relatively stable for large changes in expenditures, only in case the operational and maintenance costs change this effects the LCOS, and varies between € 204 and € 219 per MWh.

Table 6.7 lists the remaining parameters that determine the LCOS: utility rate, turbine efficiency, discount rate and future electricity price. The LCOS responses quite volatile to changes in these parameters, in particular for the discount rate. Here, only a change of 1% in discount rate changes the LCOS with over 10%.

Table 6.6: Sensitivity analysis for the Captical Expenditures (CAPEX) regarding to the cost-effectiveness or Levelised Cost of Storage (LCOS) with r = 5%, n = 70%, DoD = 80%, P = 10 / MWh and utility = 20%.

Category	-15%	-10%	-5%	0	5%	10%	15%
Caisson / L - wall							
Rock works	€ 204	€ 207	€ 209		€ 214	€ 217	€ 219
Powerhouse				€ 212			
Turbine	€ 208	€ 209	€ 210		€ 213	€ 214	€ 216
Equipment	€ 211	€ 212	€ 212		€ 212	€ 212	€ 212
End-of-Life	€ 212	€ 212	€ 212		€ 212	€ 212	€ 212

Table 6.7: Sensitivity analysis for the remaining parameters of the offshore pumped storage hydropower plant that determine the levelised cost of storage (LCOS), for the utility, turbine efficiency, discount rate and future electricity prices. The initial parameters for this analysis: social discount rate r = 5%, turbine efficiency η = 70%, Operation and Maintenance (O&M) rate of 1% for the caisson dam and 4% for the powerhouse section and its turbines, Depth of Discharge DoD = 80%, Future electricity price P = € 10 / MWh, utility rate = 20%, construction time of 10 years for only the caisson dam (23 caissons per year), excluding the powerhouse section.

Utility	30%	25%	20%	15%	10%
	€ 179	€ 192	€ 212	€ 244	€ 309
Turbine efficiency	80%	75%	70%	65%	60%
	€ 177	€ 193	€ 212	€ 234	€ 262
O&M Rate - Dam	0.50%	0.75%	1.00%	1.50%	1.50%
	€ 209	€ 210	€ 212	€ 213	€ 215
O&M Rate - Powerhouse	2.00%	3.00%	4.00%	5.00%	6.00%
	€ 191	€ 201	€ 212	€ 222	€ 232
Discount rate	3%	4%	5%	6%	7%
	€ 147	€ 177	€ 212	€ 253	€ 300
Electric price	€ 0 / MWh	€ 5 / MWh	€ 10 / MWh	€ 15 / MWh	€ 20 / MWh
	€ 191	€ 202	€ 212	€ 222	€ 232

6.5. Conclusion

The goal of this section was to inventory the construction costs that are related to the construction of the offshore PSH, and thereby assess the cost-effectiveness of the energy storage technology, in terms of the Levelised Cost of Storage (LCOS).

The total material costs add up to € 4.2 billion. Expenditures related to equipment costs add up to € 620 million, of which € 461 million is related to stand-by costs and € 159 million is related to incremental operational costs. Summarizing, the Capital Expenditures (CAPEX) add up to € 4.8 billion.

The Operational Expenditures (OPEX) include Operational and Maintenance costs (€ 10.3 billion) and charging costs (€ 4.5 billion), resulting in a total of € 15.0 billion.

In conclusion, after discounting, the present value of the project is € 5.8 billion, resulting in a LCOS of € 212/MWh. Compared to existing energy storing techniques, such as conventional pumped storage hydropower, this concept is expensive in its category but still affordable. Furthermore, new technologies like the Lithium-ion batteries or hydrogen storage are far more expensive, subsequently showing that the proposed technique is competitive.

7 Sheltering effects and optimization

Although the Levelised Cost of Storage (LCOS) for the offshore pumped storage hydropower shows its competitiveness with alternative energy storage techniques, for instance lithium-ion and hydrogen storage, this Section will address and discuss the optimization steps regarding the construction process, subsequently reducing the LCOS. During this research many topics have come along that are suitable for the further research and could potentially be beneficial in terms of construction cost reduction.

This section starts by inventory the topics that are relevant for in-depth studies and optimization process, and evaluate the relevance of the further research per item. After addressing the possible topics in Appendix J, one topic will be chosen for the in-depth study and all relevant theory and information will be discussed.

7.1. Sheltering effects

From the evaluation of the in-depth studies, shown in Appendix J, it followed that the sheltering effects of the caisson dam is the most suitable and appropriate research based on the formulated categories. This Section will first introduce and define the problem, after which the theory and background will be elaborated in order to understand the research. Then, the methodology and approach will be discussed and finally the results will be presented with the corresponding effects on the projects together with the conclusion and discussion.

Theory and background

Known from the port's breakwaters, emerged marine structures block direct wave propagation into a bay or port basin, creating a desired sheltered zone behind the structure where wave heights are decreased and hence wave conditions improve. Particular for port basins this is favorable, as a sheltered zone is necessary for safe mooring and loading logistics for all vessels.

In line with this reasoning, it is expected that the caisson dam - for the offshore pumped storage hydropower plant - will have a similar effect on the conditions directly behind the structure. Especially for the scenario that the structure has been completed for the most part, the sheltering characteristics of the dam will play a substantial role.

The relevant processes that are crucial for the understanding of the sheltering effects consist of wave diffraction and wave reflection that originates from the presence of the caisson dam. In the following paragraphs both concepts will be discussed thoroughly.

Diffraction Diffraction refers to the phenomenon that applies to all sorts of waves, including ocean waves. When waves encounter an obstacle or pass through an opening, for instance a harbor entrance or the end of a break water, they exhibit diffraction. This occurs when waves bend around the edges of the structure or opening, spreading out and changing direction, thereby propagating into the sheltered zone directly behind the obstacle.

The amount of diffraction depends on various conditions and characteristics, among them are the wave length, the size of the obstacle and the location of the observer. An example of diffraction of ocean wave, due to the encounter of a breakwater, is shown in Figure 7.1. As a result of the spreading, the wave characteristics also change after the encounter of the structure: the wave height and the wave period decreases.

Figure 7.1: Diffraction phenomenon of ocean waves encountering a break water, after which waves are spread out and changing direction in the basin.

This phenomenon has been studied thoroughly by many scientists and engineers, among them is the Japanese civil engineer Yoshimi Goda, who made significant contributions to coastal engineering. Many studies were focused on coastal engineering problems, for example wave diffraction through a breakwater gap for obliquely and normal incident waves.

In one of his publications, *Random Seas and Design of Maritime Structures* (Goda, 2000), the diffraction coefficient - i.e. the ratio of diffracted to incident height - is related to wave, obstacle and location parameters. This relation is given by Equation 7.1.

$$(K_d)_{eff} = \left[\frac{1}{m_0}\int_0^{\infty}\int_{\theta min}^{\theta max} S(f,\theta)K_d^2(f,\theta)d\theta df\right]^{1/2} \tag{7.1}$$

where:

- $(K_d)_{eff}$ = the diffraction coefficient of random sea waves (i.e. the ratio of diffracted to incident heights of significant or other representative waves),
- $S(f,\theta)$ = the directional wave spectrum,
- $K_d(f,\theta)$ = the diffraction coefficient of component (regular) waves with frequency f and direction θ,
- f = wave frequency,
- θ = wave direction,
- m_0 = the integral of the directional spectrum.

With Equation 7.1 the diffraction of random waves can be determined, which has been validated through simultaneous wave observation. Additionally, multiple schemes were provided to visualise the phenomenon and to quickly estimate the effects of the presence of a breakwater. These schemes are shown in Figure 7.2, and represent the initial and new wave field, consisting of an initial wave height and wave period and adjusted wave height and period after encountering the structure. Based on the location of the observer, the diffraction coefficient varies between 1 (no sheltering) and 0 (full sheltering).

Methodology and approach

The hypothesis with the current knowledge and background of this research, is that due to the construction of the caisson dam incident waves will be directly blocked and partly transmitted in the form of diffracted waves. In most parts of the sheltered area, transmitted waves will be substantially reduce compared with the initial wave height. In turn, this results in improved workability in the corresponding area. The following Section will discuss the methodology and approach of this in-depth study.

Methodology The first step is to schematise the actual situation of the circular dam to a system which can be applied to the Goda tables. Since Goda only studied straight, one-end finite breakwaters, the caisson dam in this study does not match the conditions of their breakwaters. To that end, the actual situation must be simplified into several scenarios, depending on the wave direction to determine the

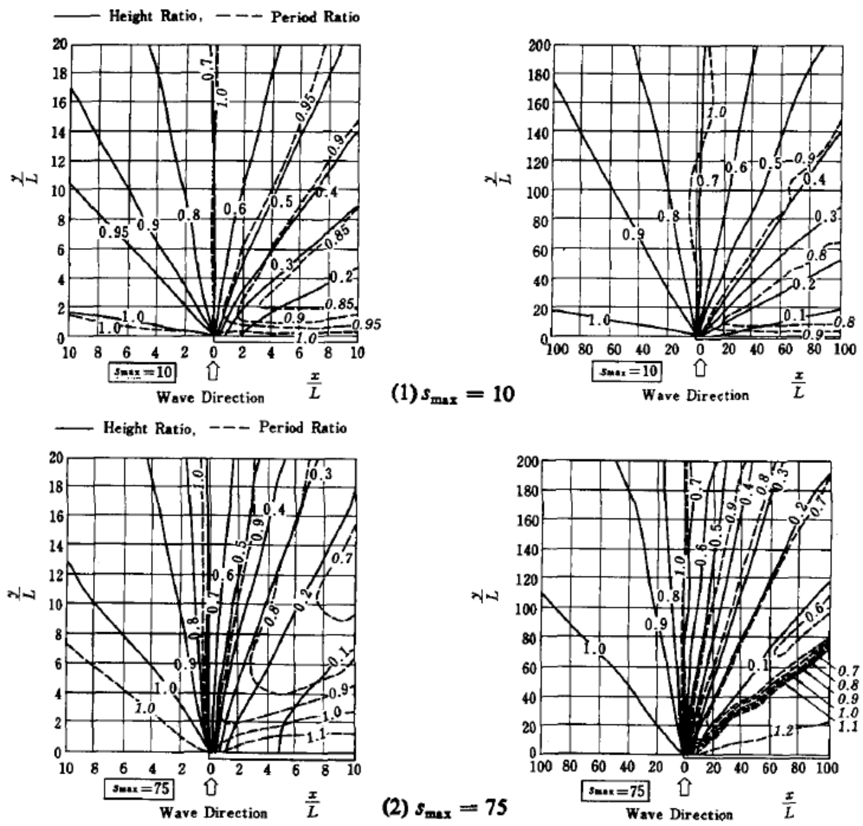


Figure 7.2: Diffraction diagrams of a semi-infinite breakwater for random sea waves of normal incidence (solid lines for wave height ratio and dash lines for wave period ratio. Source: Goda (2000).

orientation and the length of the breakwater.

The in-house software allows the determination of diffraction for three types of structures: i). finite breakwater with a left-side opening, ii). two breakwaters with a grap in between and iii). finite breakwater with a right-side opening. These structures are schematically shown in Figure 7.3, together with a visualisation of the sheltering effect of the breakwater by plotting indicative diffraction coefficients.

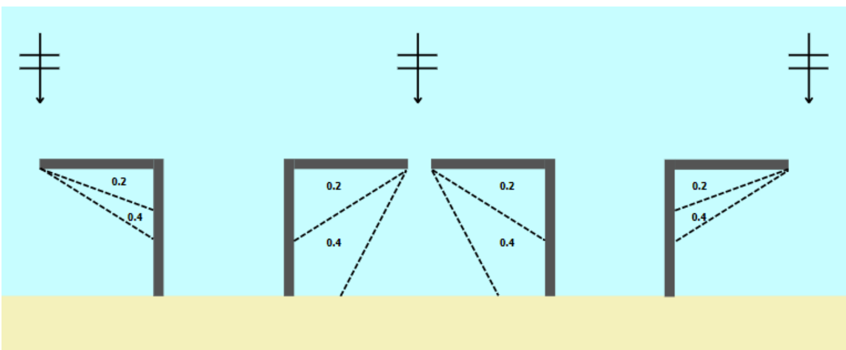


Figure 7.3: Supported structure types by the in-house diffraction software based on the Goda tables. Diffraction coefficients are indicative values.

In order to apply the Goda diffraction software, the round caisson dam must represented by a composition of the three types of structures, while taking the incident wave direction into account. The approach has been visualised in a flow chart shown in Figure 7.5, and will be elaborated in more detail in the following steps.

1. For all considered caisson dam progressions (i.e. 33% and 50%), the following steps will be repeated:

2. The diffraction software calculates the diffraction coefficient for a time-series of waves based on a specific breakwater configuration. The configuration and orientation of the breakwater vary depending on the wave direction. To account for this, the software analyzes eight distinct wave directions. Figure 7.4a represents the schematized situation, including the incoming wave front, finished caisson dam, and observer location for each wave direction (e.g., north, north-east). Sheltering effects are estimated for each direction to determine if the area is fully exposed to the waves or partially sheltered by the dam. If fully exposed, the undisturbed wave conditions are used. If sheltered, the dam is included in the schematization using predefined breakwaters.

3. This is done in the model setup: the red lines in Figure 7.4 shows how the caisson dam can be represented by a combination of (straight breakwater) structure types. The length and the orientation of the breakwater will vary depending on the wave direction. Figure 7.4b represents the situation where waves diffract from the left (eastern) end of the dam, and is enclosed at the north. The same method will be applied for the north-passing waves, shown in Figure 7.4c, where the breakwater consists of the northern part of the dam and is enclosed at the eastern part.

4. Then, the original data will be pre-processed to comply with the capabilities of the diffraction software: the breakwater composition that applies for northern waves must only be calculated for that type of waves. To that end, the metocean data will be divided into 8 wave direction categories. This will be done with a simple Python script that creates an unique data set per wave direction.

5. During the construction phase, the analysis considers two (or four for later analyses) points of interest: 1) at the end of the caisson dam and 2) outside of the caisson dam. These points were chosen based on the difference in shelter provided by the dam and the variation in work method between the outer area of the dam and the caisson installation at the end of the dam. For each scenario, the diffraction coefficient is determined at both the outside and end of the breakwater. The corresponding breakwater configuration is applied to the relevant wave data category. If the incident wave diffracts at both ends, similar to Figure 7.4, two separate scenarios are created to represent the situation. Both diffracted components contribute to the waves observed in the sheltered area.

6. The result is eight different time series - consisting of new wave conditions with (diffracted) wave height and wave period - for inside and outside the reservoir, divided based on the wave direction. Merging the distinct time series leads to the diffracted time series that can be used for activities inside and outside the caisson dam. Again, this will be done with Python.

7. Subsequently, this data can then be used to determine the workability based on the location of the activity or operation.

8. With the new workability a more detailed probabilistic planning can be made including the potential beneficial sheltering effects. For the 'continuous' construction works - such as the installation of the scour protection and all dredging related activities - the new workability will be in terms of percentages, whereas for the installation of the caisson units windows of opportunity of workable windows will be determined, based on the limiting threshold discussed in Section 5.

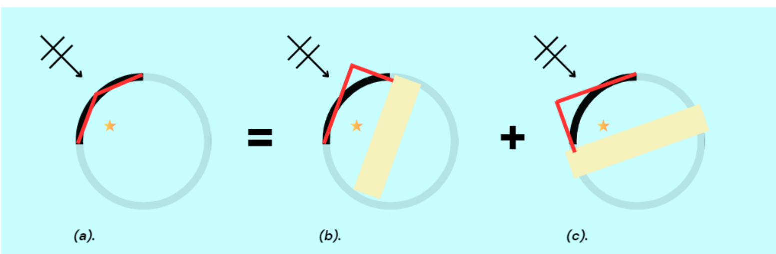


Figure 7.4: Example of a representation for the caisson dam and NW incident waves. Superposition principle applies to the diffracted waves at the observer, which results in a bi-directional wave field directly behind the breakwater.

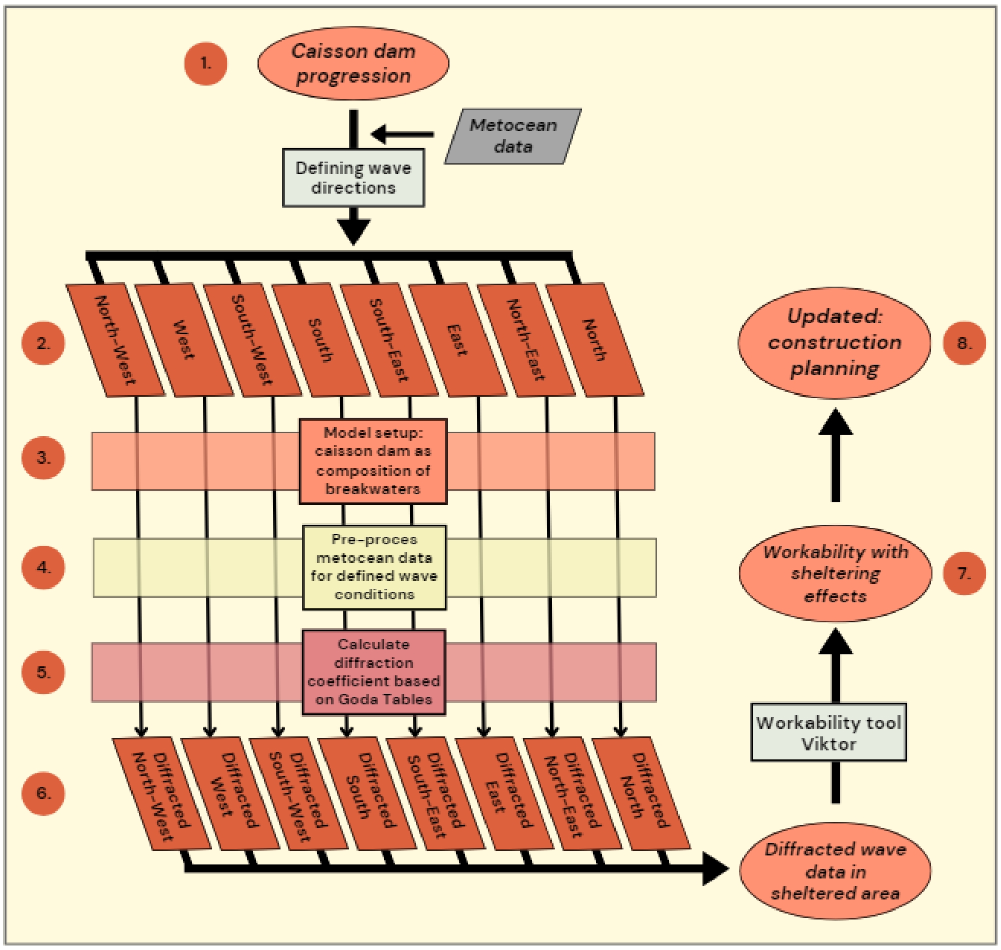


Figure 7.5: Methodology of the determination of the sheltering effects based on the caisson dam progression, taking into account various wave directions and whether the activity is executed inside or outside of the reservoir. Step number corresponds to the approach shown on the previous page.

Point of interest The diffraction software allows the determination of the wave conditions of one coordinate (X,Y) relative to the tip of the breakwater. Thus, choosing the location of the point of interest(s) will be important for the effects of the diffraction. This point of interest must be chosen properly, as it should be interesting regarding the project planning and execution. For instance, choosing the a point in the middle and at the inner side of the caisson dam will not be interesting for this study, since at this location only the inner berm will be constructed by the Trailing Suction Hopper Dredge (TSHD). As the workability of the TSHD is relatively high (compared to the caisson installation), the improvement in terms of cost or time reduction will not be substantial.

For this analysis a total of two types of locations has carefully been chosen based on their potential beneficial effects on the project. During this process an inventory of construction activities has been made, which was governing for choosing the appropriate locations. Eventually a distinction between the 'work front', i.e. the area where new caissons will be installed, and the outer area, i.e. the area where the scour protection will be installed, has been made. Figure 7.6 summarizes the scenarios (33% and 50%) and the points of interest that will be relevant for calculating the sheltering effects.

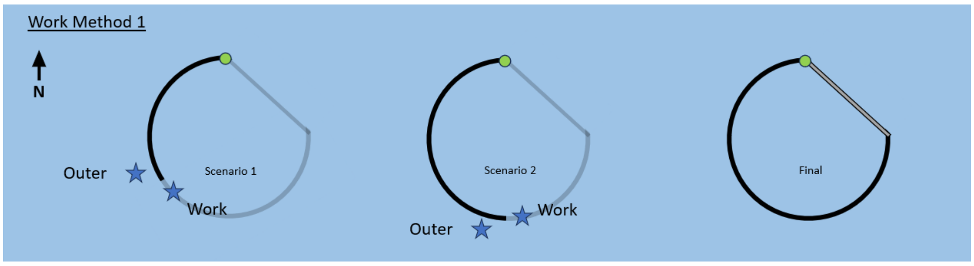
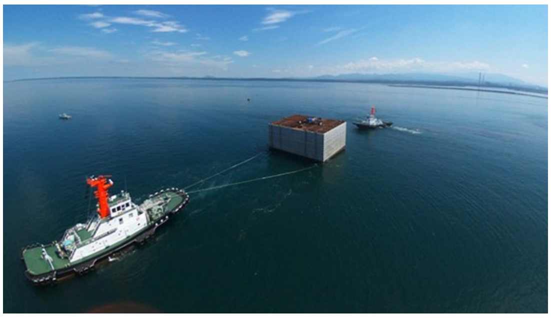


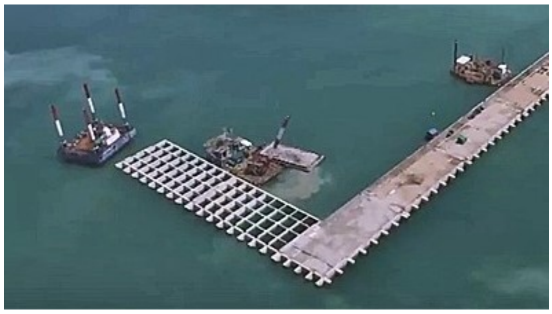
Figure 7.6: Model set up for the various scenarios and their point of interests for the initial work method, divided into the work activities at the outer area and the caisson installation site, indicated by the blue stars. The green dot indicates the starting point of the installation activities. Left figure considers a dam progression of 33%, the middle a dam progression of 50% and the right shows the final situation.

(a) Example of the installation of the scour protection at the *Outer* locations, indicated by the blue stars in Figure 7.6. Source: *Shipshub*

(b) Example of the (more precise) installation of the scour protection at the *Outer* locations, indicated by the blue stars in Figure 7.6. Source: *Van Tunen*



(c) Example of the towage and positioning of the caisson units at the *Work* locations, indicated by the orange stars in Figure 7.6. Source: *AOMI*



(d) Example of the installation of the caisson units at the *Work* locations, indicated by the orange stars in Figure 7.6. Source: *Dutch Water Sector*

Figure 7.7: Examples of the construction works that will be executed in the Outer (a). and b).) and Work (c). and d).) area indicated in Figure 7.6, showing the difference in work method.

Diffraction coefficient The software determines, based on the coordinates of the observer relative to the tip of the breakwater as shown in Figure 7.8, the diffraction coefficient by reading the Goda diagrams shown in Figure K.3 and K.4. After that, the wave height will be corrected for the diffraction by multiplying the initial wave height with the diffraction coefficient.

Rock works installation The installation of rock works at the outer region will mostly be done by a Side Stone Dumping Vessel (SSDV), only a minor part (the toe protection) will be installed with the Fall Pipe Vessel (FPV). To study the effects of the caisson dam on the workability of the SSDV located at the outer area of the dam, the same wave limiting conditions will be applied which has been introduced in Section 5. This implies a simplified single wave condition will be normative for the activity, and is given by a significant wave height of H_s = 2.5 m, without taking the wave period into account.

Caisson installation The limiting wave and wind conditions that are considered for the persistency of the caisson installation are slightly different than the caisson project Costa Azul in Mexico (which have been introduced in Section 5). Although the limits are the same, the required duration of the 'good weather' windows has been decreased from 48 hours to 12 hours, due to a different work method. For the workability assessment, two considerations of the thresholds will be applied: one including wind restrictions and one neglecting them.

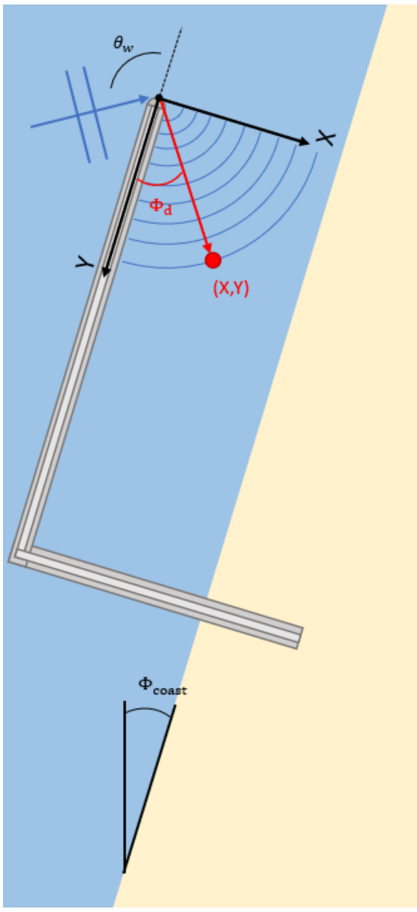


Figure 7.8: Input parameters for the diffraction model, containing the length of the breakwater parallel and perpendicular to the coast, the angle of the coast (ϕ_{coast}), the location of the observer defined as a coordinate relative to the tip (x,y) for a certain wave field with an angle of incident (Θ_w).

First, the same limits will be applied that have been used for the previous assessment in this project and originate from the caisson project in Costa Azul. Here, a 48 hours window has been used to assess the feasibility of the caisson transport to project site as well as the installation of the caisson. Furthermore, during these 48 hours the wave limiting conditions consists of a combination of wave height and wave period. In addition, the maximum tolerable wind speed during these 48 hours was u_{10} = 7.9 m/s.

Additionally, to study only the influence of the caisson dam on the wave conditions observed at a particular location, the workability assessment has also been conducted with the assumption that wind has no limiting role in the installation process. With the reasoning that wind will be normative only for activities which are directly impacted by the wind speed or direction (e.g. lifting/hoisting operations of a crane vessel), it can be argued that the caisson installation - the positioning and ballasting of the unit - can be done while neglecting wind conditions.

During the process, wind will be important as it blows against the side of the caisson, which will act as a huge sail. Without any ballast the caisson will be fully emerged, resulting in a sail surface of approximately 900 m^2. However, before the caisson leaves the intermediate wet storage (Port of Rotterdam) the caisson will be slightly ballasted to improve the dynamic stability of the caisson and subsequently reduce the emerged surface area. Once the caisson will be prepared for installation on site (after it has been temporarily been anchored) additional ballast will be applied to even further increase the draught of the unit, to reduce the installation time and to reduce the surface area.

Considering the wind velocity threshold of u_{10} = 7.9 m/s used for the project in Costa Azul (Mexico), the caisson that was used in this project was different than the rectangular unit that is used in the offshore energy island. The major difference is the shape of the caisson: in Costa Azul a L-shaped unit was used, which mainly was determined based on their installation method and the weather climate, which was slightly harsh compared to the North-Sea. The L-shaped caisson might be the reason that a wind limiting threshold was imposed at Costa Azul, since the stability of this particular unit will be substantially different than a rectangular box.

Generally, for the installation of a regular caisson unit (i.e. rectangular shaped) the wind limiting factor can always be overcome by means of applying a different engineering strategy or work method. However, wind may be an important parameter during caisson transport from the Port of Rotterdam to the project site. To that end, to study both effects on the project planning both scenarios will be considered, the first for which the wind will be included and hence is considered as a limiting factor and the second scenario where the wind can be neglected.

Results

In this Section the most relevant results will be presented concisely, based on the consideration which results are governing for the project planning or are assumed to be of great value based on other aspects. In addition to this summary, Appendix L contains an extensive overview of all results for different scenarios and other wave parameters that are not discussed here.

In total there are four locations for which calculations and determinations have been conducted, subsequently resulting in new wave conditions, workability or persistency (workable windows) per location. For each location, first the new time series will be presented (42 years of hindcast data while taking diffraction of the caisson dam into account), after which the mean workability (P50) for the Side Stone Dumping Vessel (SSDV) activities or persistency for the caisson installation will be given, depending on the location (i.e. Outer or Work location).

Outer site The first location is the outer area located near the caisson installation site. This implies that as construction progresses, this outer area moves along and hence changes over time. At the start of the construction, initial wave conditions will prevail, after 33% has been constructed the area is only exposed to waves from north to south but will be sheltered for eastern waves. After 50% the area will be partially sheltered from waves from the north but fully exposed to waves from the south..

Figure 7.10a shows the probability density function for the corresponding area. A clear decrease in wave height can be seen between the initial conditions and scenario 2, which indicates that northern waves are strongly reduced. Furthermore, it appears that for scenario 3 the contribution of smaller waves from the north is stronger than the reduction in southern waves, as the probability of lower waves (0.5 m - 2 m) decreases. On the other hand, it also shows that the probability of higher waves (>2 m) decreases, which indicates that either these waves are reduced or that they originate from the south-west. Furthermore, Figure 7.10b shows the development of the sheltering during the dam progresses.

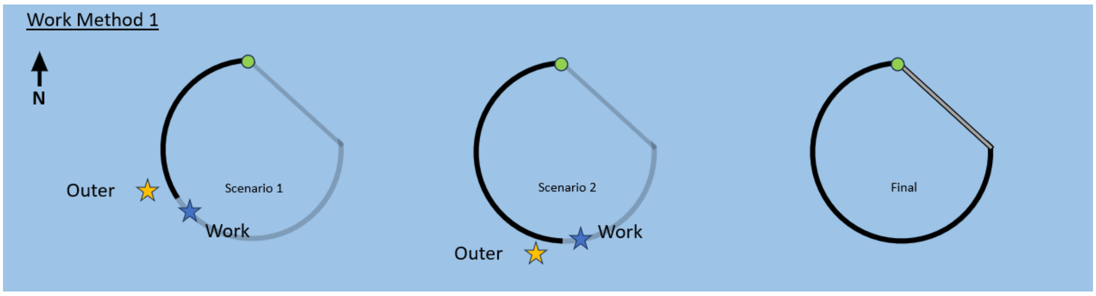
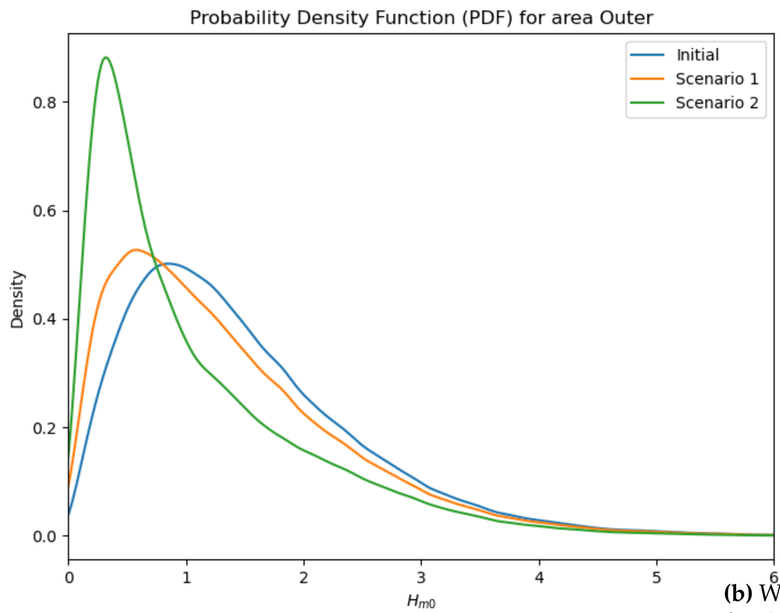


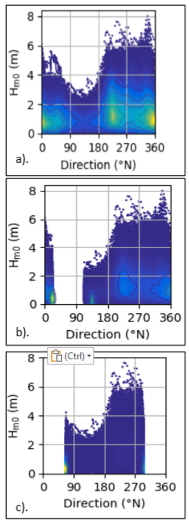
Figure 7.9: Indication of the Outer 2 location, shown by the orange star, for the different scenarios.

Table 7.1: Summary of the mean workability (exceedance probability of 50%) for a significant wave height of H_s = 2.5 m, for the Outer area.

Mean values	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Base	73%	84%	85%	94%	97%	97%	98%	95%	89%	81%	78%	72%	86%
Scenario 1	74%	84%	88%	95%	97%	97%	98%	95%	89%	82%	80%	73%	86%
Scenario 2	81%	90%	92%	98%	99%	99%	100%	98%	94%	86%	87%	81%	91%



(a) Probability density function (PDF).



(b) Wave height as a function of wave direction for a). scenario 1, b). scenario 2 and c). scenario 3.

Figure 7.10: Probability density function of the significant wave height (H_{m0}) for the construction area 'Outer 2' for three different moments in construction progression: initial wave conditions, scenario 1 (33%) and scenario 2 (50%).

Work Lastly, the caisson installation site at the southern end will be presented. Again, similar to the other installation site, it is assumed that the sheltering effects of the caisson dam will apply from Scenario 1 when the dam progression has reached 33%. From this moment, the area will be partly protected to frequently occurring northern waves, but fully exposed the other dominant south-western wave direction. After 50% the area will be more protected to southern waves but more exposed to northern waves. Depending on the wave climate and the diffraction the conditions might improve or worsen.

Figure 7.12a shows the probability density function of the installation site for the initial wave climate and the situations where sheltering can be expected. It turns out that sheltering can be substantial from a progression of 33% even further improving the wave conditions after 50%. For the first, the area is exposed to southern waves, whereas the last scenario is more exposed to northern waves, resulting in different angle of attack and distribution of waves.

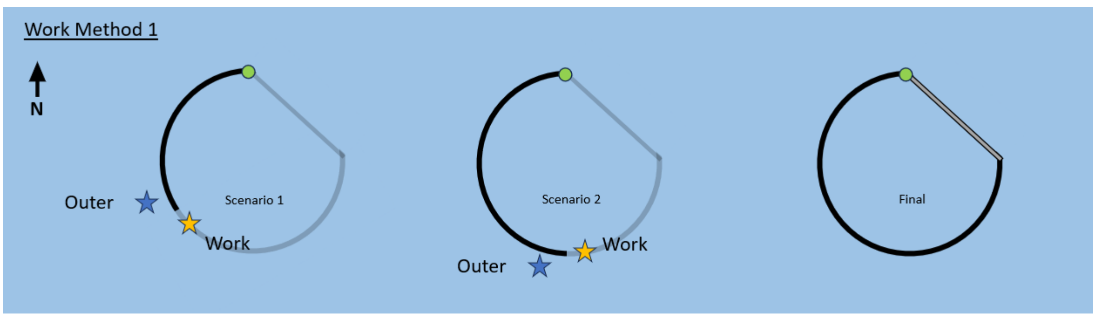
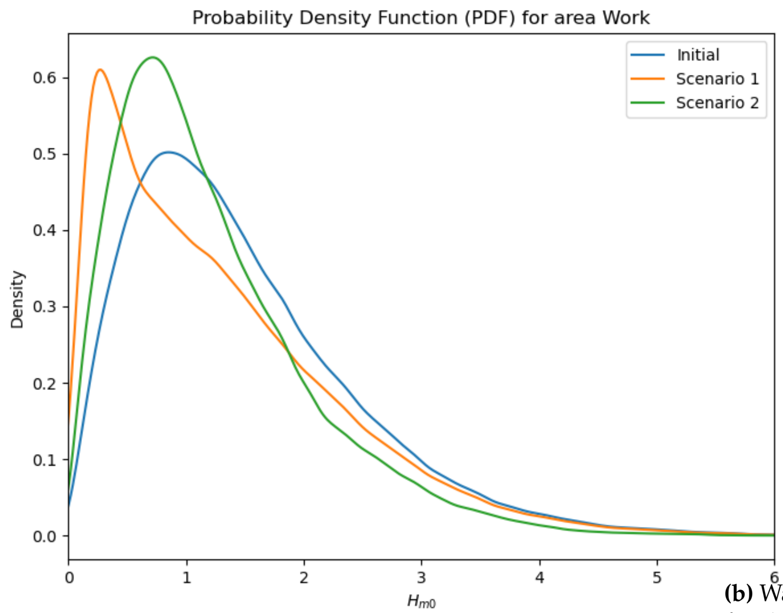
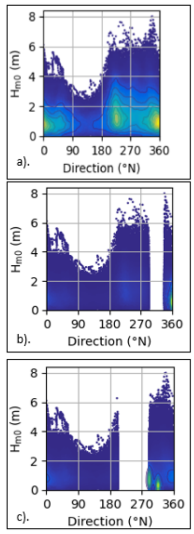


Figure 7.11: Indication of the Work 2 location, shown by the orange star, for the different scenarios.



(a) Probability density function (PDF).



(b) Wave height as a function of wave direction for a). initial offshore conditions, b). scenario 1 and c). scenario 2.

Figure 7.12: Probability density function of the significant wave height (H_{m0}) for the construction area Work for four different moments in construction progression: initial wave conditions, scenario 1 (33%) and scenario 2 (50%).

Table 7.2: Persistency for the caisson installation at the installation site 'Work' location in terms of workable windows per month, yearly average, for an exceedance probability of 50%. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and additional wind velocity of u_{10} = 7.9 m/s.

Mean values	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Base	14	17	23	31	35	37	39	36	27	18	15	12	25
Scenario 1	16	18	24	32	36	38	40	37	29	19	16	14	26
Scenario 2	15	19	24	33	35	37	40	37	28	19	16	14	27

Table 7.3: Persistency for the caisson installation at the 'Work' location in terms of workable windows per month, yearly average. Threshold limits used here are limiting wave conditions as mentioned in Section 5 without taking wind limiting conditions into account. Numbers correspond to the 50% exceedance probability.

Mean values	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Base	18	26	30	38	43	44	47	44	34	25	21	20	33
Scenario 1	23	28	33	44	49	47	49	47	37	31	26	22	36
Scenario 2	24	30	36	43	47	50	52	50	41	32	29	28	38

Conclusion

As has been expected, sheltering effects of the caisson dam have a substantial effect on the wave conditions behind the dam, which can be seen as an increase in probability of smaller waves in the PDF of the significant wave height for the various locations as well as the gaps in the wave height as a function of wave direction graphs. The implications of these sheltering effects also show an increase in workability and persistency for the outer activities and the caisson installation. First, the effects for the rock work installation will be discussed, afterwards the implications for the caisson installation will be elaborated.

Rock works installation Although the wave limiting conditions for the Side Stone Dumping Vessel (SSDV) are rather lenient (significant wave height of H_s = 2.5 m) the workability of this vessel in particular improves substantially. Starting at a yearly average of 86% for the initial offshore conditions, the workability increase marginally, with only a few percentage for during the off-season for Scenario 1. However, for Scenario 2 a substantial improvement can be expected: during the winter months an increase in workability up to 8% can be expected, while in the summer a few percentages are gained, resulting in a yearly averaged workability of 91%.

Despite the fact that the workability for the SSDV increases as the dam progresses, the installation rate and thus the construction time of the entire caisson dam mainly depends on the installation rate of the caissons. Only after a caisson has been placed, the scour protection can be installed and thus fully depends on the installation speed of the caissons. Therefore, the conclusion can be drawn that the only requirement for the installation of the scour protection is that it should not be the limiting factor: the caisson installation should never be hindered nor should it have to wait on the scour protection installation.

Caisson installation With regard to the caisson installation, the first thing that can be noticed is the fact that the wind limiting condition is governing for the installation process, as the number of workable windows in Table 7.2 do not increase over the dam's progression. This implies that, even though the wave conditions do significantly improve, the number of favorable windows are still limited by the wind conditions. This is confirmed when looking at the results when wind has been neglected (Table 7.3): for the base scenario the number of windows increase by up to 25%. Therefore, as has been elaborated earlier, wind limiting conditions will not be considered during the installation phase of the caisson unit.

When neglecting wind during installation, the improvement starts at Scenario 1 (33% finished) and becomes maximum for the last phase of the project after 50% has been constructed. Depending on the exact location of installation, the increase ranges from 10-15%. Interesting, when looking at the improvement per month, it turns out that the summer months show only little increase (maximum 10%) whereas the winter months (October - March) show a substantial increase of up to 50%!

One of the big implications of the non-uniform persistency increase would be an extension of the work season after the dam's progression has reached 50%. At the beginning of the project, it is assumed that all offshore construction works, i.e. the caisson installation, starts at April and lasts until September. The length and start of this work season mainly depends on the wave climate: generally, the milder the wave climate is, the longer and earlier the work season will be.

Looking at the number of workable windows for the start and end of the project (Base scenario) which are shown in Table 7.3 (in which the success rate of 10% has not been taken into account yet), the season starts in April in which on average 38 favorable windows occur and end in September in which 34 favorable windows occur. Considering a dam's progression of 33% (Scenario 1), the number of windows in April will increase with 13% to 44 windows and for September it improves with 15% to 37 windows, while on average the total number of favorable windows for the entire work season increase with 10%.

Regarding the development of the duration of the work season over time, for Scenario 1 the same work season applies as for the base case. However, for Scenario 2, the work season can be brought forward to March, which allows the installation of more caissons with a relatively high probability of occurrence. Seen from a project planning perspective, this will have a substantial impact on the construction planning and will reduce the construction time with months, which will be studied in great detail in the following section.

Effects on the Levelised Cost of Storage (LCOS)

To quantify the direct effects due to sheltering of the caisson dam, the project planning will be adjusted according to the results that followed from the previous section. For the caisson installation this implies that for the first half of the caisson dam the initial wave conditions and thus installation rate will be applied. After this point, diffraction will become substantial and subsequently will increase the installation rate. Finally, after 50% has been installed the last installation rate will be applied to determine the consequences for the entire project.

Per scenario an assessment will be made that is based on the number of caissons that will be installed, and verifies whether the initial equipment capacity is still sufficient for the new installation rate. For instance, the Trailing Suction Hopper Dredge (TSHD) that is deployed must fill all the caisson units with dredged material, and has been chosen based on the initial caisson installation rate: choosing a larger TSHD will reduce the utility rate of the vessel and thus is undesirable. However, when the number of caissons increases it might be necessary to also increase the capacity of the TSHD or choose to deploy two TSHDs.

To that end, to study the effects on the project planning, e.g. the reduction in project duration, the change in equipment deployment and the change in Levelised Cost of Storage (LCOS), an extensive overview of the distinct phases will be composed and analysed. Every phase will consists of a corresponding installation rate that follows directly from the persistency analysis as has been presented in Section 7.1.3. Prior to this analysis, the set-up of the assessment per scenario will be introduced and elaborated.

Boundary conditions The sheltering effects - such as the decrease in significant wave height and subsequently increase in workable windows for the caisson installation - have already been introduced in the previous section. The following section will introduce the starting points per scenario which will be used for the Levelised Cost of Storage (LCOS) assessment, which includes the start and end date of the considered work season and the number of installed caissons per month.

From the workability assessment the number of workable windows per month has been determined, for a various exceedance probabilities ranging from 10% (P10) to 90% (P90). The choice of which exceedance probability will be used in the project planning is an engineering and financial decision, which follows from the corresponding considerations.

Considering a too conservative exceedance probability, for instance P90, will account for all worst case scenarios but therefore providing a too conservative starting point for the project planning. This will also lead to an underestimate of the necessary equipment and construction materials, in case the conditions are more favorable than was expected. On the other hand, assuming a very progressive installation rate corresponding to a high exceedance probability (for instance P10), will likely overestimate the actual installation rate. Consequently, equipment and construction materials will be abundant since the actual weather conditions will be less favorable than expected. In addition, considering a high exceedance probability will also result in a shorter construction period, since the installation rate is higher than for another scenario. This optimistic project planning will be favorable for the client, however will not be realistic.

Choosing the correct exceedance probability for an offshore project with this size is a very difficult decision, since the consequences of an incorrect starting point might have devastating effects on the project. Therefore, a somewhat conservative approach will be followed, to incorporate extra certainty in the project planning. Hence, for this project an exceedance probability of 70% (P70) will be used, which is the result of the trade-off between certainty and reality.

Using the workability analysis including sheltering effects, the project can be broken down per scenario, for which the number of installed caissons can estimated. Table 7.4 shows the breakdown of the project, and justifies the installation rate per month. It is assumed that for Scenario 0 (base case) and Scenario 1 (33% constructed) the work season is the same, which has been described in Section 4, starting in April and ends in September. For Scenario 2 favorable sheltering effects cause the advance of half a month of the work season. The last column of the table lists the sum of caissons that is expected to be installed per year, and this specific number will be used to compose the project planning.

Table 7.4: Breakdown of the project planning and expected caisson installations per month, as a function of the dam's progression. The listed numbers are based on an exceedance probability of 70% (P70) and a installation success rate of 10%. The underlined and bold numbers indicate the duration of the work season.

Scenario	Progress	Start	End	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Sum
Initial	0 - 33 %	Apr	Sep	2.6	**3.5**	**4.0**	**4.2**	**4.3**	**3.9**	**2.9**	2.2	23
Scenario 1	33 - 50 %	Apr	Sep	2.9	**4.0**	**4.4**	**4.5**	**4.7**	**4.3**	**3.3**	2.4	25
Scenario 2	50 - 81 %	Mar	Sep	**3.2**	**4.0**	**4.5**	**4.7**	**5.0**	**4.7**	**3.7**	2.8	30

Due to the diffraction of the caisson dam, the conditions for the installation of the caisson units will improve. One of the implications is a change in project planning: as the installation rate changes over time, obviously this will also have an impact on the project duration. The following section will address the expected consequences with regard to the project duration, relative to the initial planning without sheltering effects.

Table 7.5: Adjusted project planning with regard to the caisson installation rate which develops over time (depending of the number of caissons that have been installed). Installation rate are the expected installed caissons for the given year, number of caissons left is determined at the start of the year, number of caisson installed and finished percentage is given at the end of the year.

Year	Installation rate	Caissons left	Caisson installed	Finshed
1	23	212	0	0%
2	23	189	23	9%
3	23	166	46	18%
4	23	143	69	26%
5	25	120	92	33%
6	25	95	117	45%
7	30	70	144	54%
8	30	40	174	65%
9	10	10	198	76%
10	-	-	212	81%

Results According to Table 7.5 the project duration decreases with 2 years due to the improvement of workability. This will can also been seen when determining the Levelised Cost of Storage (LCOS), which basically shows the total cost for various energy storing technologies. For the new situation, which incorporates the favorable wave conditions, an extensive expenditure overview is shown in Table N.2. A brief summary of the overview is shown in Table 7.6, which includes major expenditures categories, expressed in terms of the future and present value. Compared to the base scenario with initial wave conditions, a reduction of approximately 1.5 years in construction time is expected when including the favorable effects of diffraction. Consequently, this results in a decrease of Levelised Cost of Storage (LCOS) of 8 €/MWh, equivalent of 3.8% (initial LCOS: 212 €/MWh, new LCOS: 204 €/MWh).

Table 7.6: Concise overview for all categories considered in the LCOS, including the beneficial consequences of the sheltering effects of the caisson dam. Taking into account the year of expenditure and the discount rate, the LCOS can be determined.

Category	Total cost	Present Value (PV)
Caisson costs	€ 607 million	€ 3.8 billion
Rock works	€ 723 million	
Wave return wall	€ 180 million	
Turbine costs	€ 2 billion	
Powerhouse costs	€ 561 million	
Scour protection	€ 86 million	
Stand-by costs	€ 399 million	
Operational costs	€ 159 million	
Subtotal	€ 4.7 billion	
Risk costs	€ 127 million	€ 106 million
O&M costs	€ 10.5 billion	€ 1.4 billion
Charging costs	€ 4.5 billion	€ 0.6 billion
End-of-life costs	€ 120 million	€ 1 million
Subtotal	€ 15.2 billion	€ 2.0 billion
Total costs	€ 19.9 billion	€ 5.9 billion
Electrical discharge	2.21×10^8 MWh	2.87×10^7 MWh
LCOS	€ 204 MWh^{-1}	

7.2. Optimized work method

Additional to the previous study to assess the impact of the sheltering effects of the constructed caisson dam, an extra analysis dedicated to the work method will be conducted. In this study, the effects of adjusting the work method to a two-side operational project (in stead of a single installation area) will be analysed, eventually determining the implications on the project planning and subsequently on the Levelised Cost of Storage.

In Section E the initial work method which has been used so far has been introduced and discussed. From this initial perspective, caissons could only be installed on one end of the caisson dam. This restricted the construction works significantly, also hindering the progress when wave conditions exceed the wave limiting thresholds. In this additional study, both ends of the caisson dam are now considered as a project site, facilitating the installation of the caisson unit.

The increase in installation rate will have an effect on the required equipment, consequently increasing the expenditures related to equipment, both the operational and the stand-by costs. Contrary, working on both ends will likely increase the installation efficiency: at some point the caisson dam will shelter the northern end sufficiently to enable installation of the caisson at the north, whereas the wave conditions in the south are insufficient. This phenomena - adjusting the work method based on the wave conditions - has not been included during the previous optimization. Therefore, this section will study to what extent the improved work method will have an effect on the project duration, LCOS and also on the logistics related to the caisson production, transportation and storage.

With the knowledge obtained during the previous study on the sheltering effects of the caisson dam, it turned out that although the wave conditions improved significantly, the workability of the caisson installation did not change at all. The reason for this was that the installation limits, retrieved from the Costa Azul caisson installation project, were wind-governing: the weather conditions on the North-Sea exceed the wind limiting conditions that was imposed at the Costa Azul project ($U_{10} \leq 7.9$ m/s for a period of 48 hours) while the threshold period was decreased from 48 hours to 12 hours for this project. Therefore, it was decided that for the caisson installation the wind limiting condition will be neglected for the rest of the project. Subsequently, for the following study only the wave limiting conditions will apply, which can be found in Section 5.

Furthermore, one of the previous conclusions was the fact that the workability of the Side Stone Dumping Vessel (SSDV) for the scour protection at the outer area of the caisson dam was not the limiting factor during construction. It turned out that the caisson installation was normative in the project planning. Therefore, in the following assessment only the caisson installation areas will be considered, neglecting the outer areas where the SSDV's work. This implicitly means that it is assumed that the caisson installation is the restricting activity for the following assessment, whereas the rock works installation can easily follow unhindered.

First the approach of this study will be discussed, which work methods are considered to be interesting and why, then the composition and schematisation of the caisson dam by straight breakwaters will be presented that is necessary for the diffraction software. With the new time series, including diffraction due to the caisson dam, the results will be presented and extensively discussed. Afterwards, the implications of the diffraction on the Levelised Cost of Storage (LCOS) will be discussed, and subsequently the consequences on the project planning including things like logistics.

Set-up Work Methods

Two different work methods are studied that are considered to be very potential with regard to the improvement of the workability due to the adjusted approach. With respect to the original work method, the caisson installation only was possible on one end of the caisson dam. In the new work method, this has been changed so caisson installation can be conducted on both ends of the caisson dam, doubling the initial installation rate. Furthermore, the starting point of construction will be changed for one of the two work methods, so the sheltering effects of the caisson dam can be optimized to improve the workability even further.

Figure 7.13 shows the Work Methods that are considered in this study, indicating the starting point (green dot), the development of the two project sites (Work 1 and Work 2, indicated by the blue stars), the final orientation of the facility including the powerhouses and lastly the corresponding indexation of the scenarios. Work Method 2 shares the same starting point with the initial work method (both located in the north), but the caisson dam progresses at both ends of the dam. For Work Method 3 the starting point has been chosen based on the wave climate in the North-Sea (dominant wave directions are south-west and north-west).

The hypothesis with this starting composition is that after 1/4 has been constructed, the caisson dam creates a sheltered zone which allows the continuation of the installation for either north-western or south-western waves. For instance, for south-western waves during Scenario 5, project site Work 1 will be partly sheltered due to the dam and vice versa for north-western waves.

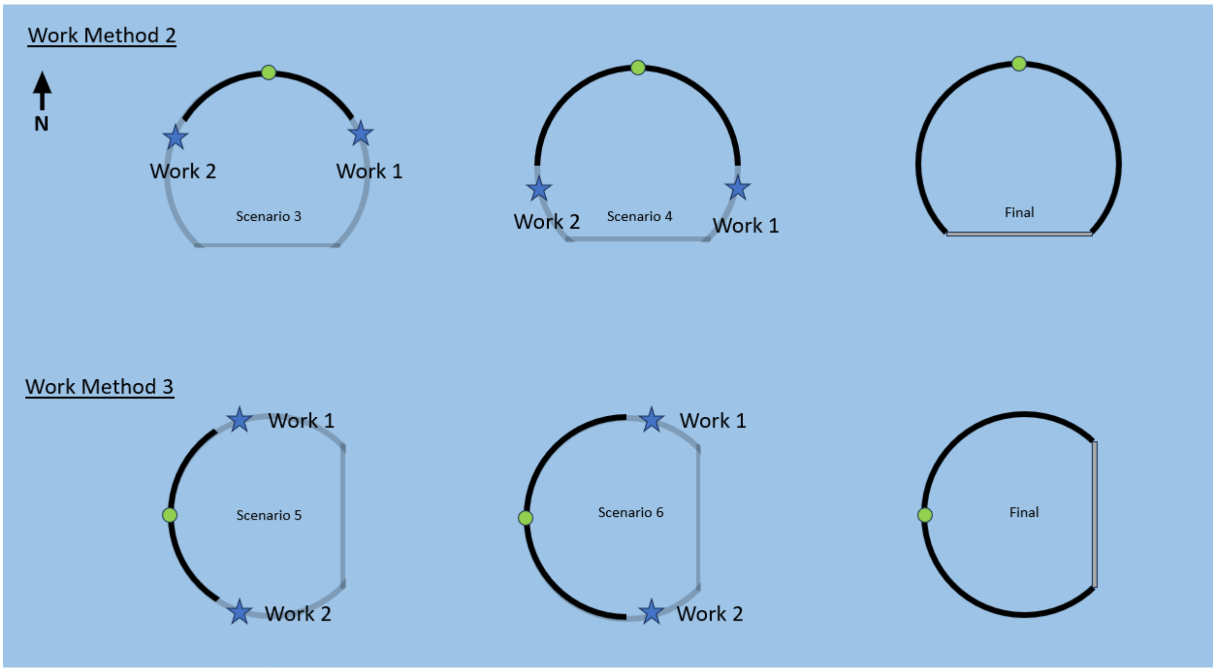


Figure 7.13: Set-up of the different work methods and the distinct scenarios for which the sheltering effects will be determined. Green dot indicates the starting point of the construction works for the Work Methods, the blue stars are the considered project sites and the last figure shows the final situation of the PSH facility.

Since the Goda diffraction diagrams only work for one-sided, finite straight breakwaters, the caisson dam shown in Figure 7.13 must be schematised and simplified in order to use Goda's theory. Similar to the previous study, for some wave directions it is assumed that the observed wave height at a project site consists of two wave components, that originate from both ends of the caisson dam respectively. To overcome this problem, the wave components will be included by taking the square root of the sum of squares of the individual components, also given by Equation K.1.

7.3. Results

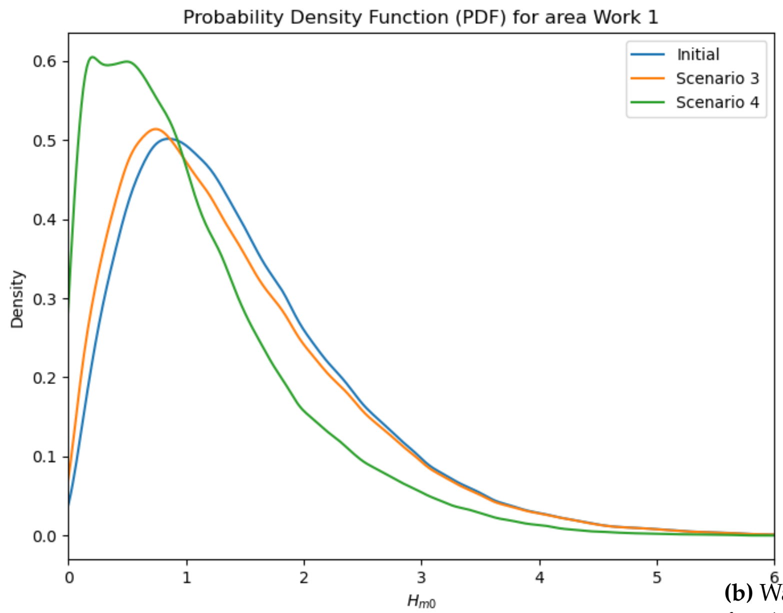
First the second work method will be discussed, which considers the situation where the installation starts in the north and progresses to both sides, which is schematised in Figure 7.13. Since this work method includes two distinct project sites with a different location, the influence of the caisson dam will be different per project site. Consequently the workability per site must be determined per site and scenario, which will be discussed individually.

Subsequently, the results of the third work method will be presented, which involves a western starting point and a double project site working on both sides of the caisson dam. Similar to previous studies, the wave conditions will be determined for both sites (Work 1 and Work 2) which will be the input for the new workability assessment.

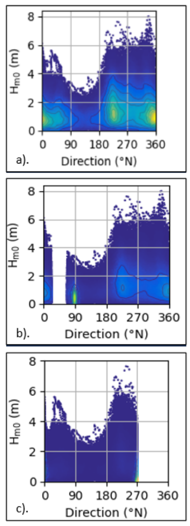
Work Method 2

Project Site: Work 1 The first project site is the eastern area of the caisson dam, which progresses clockwise. Looking at the configuration in Figure 7.13, it is expected that during Scenario 3 diffraction will only occur for north-western wave conditions; for the remaining the wave direction the site will be fully exposed. Later, during Scenario 4 the beneficial effects due to diffraction is expected to increase. This can also be seen in Figure 7.14a as the slight increase in probability for lower waves for Scenario 3 and a substantial improvement for Scenario 4.

Applying the workability limits, i.e. the wave limiting conditions, defined in Section 5, the correlated workability for the caisson installation can be determined. The mean number of workable windows regarding the caisson installation are summarized in Table 7.7. Including some uncertainty and being conservative, i.e. choosing an exceedance probability of 70% and a success rate of 10%, the expected number of installed caissons can be determined. Table 7.8 lists the number of installation caissons per month for the considered scenarios and the sum of installed caissons for the assumed work season, indicated by the underlined and bold numbers. It is assumed that due to the improvement in wave conditions during the spring, the work season can be advanced by one month, increasing the duration of the work season by 17%, which increases the total installed caisson substantially.



(a) Probability density function (PDF).



(b) Wave height as a function of wave direction for a). initial, b). Scenario 3 and c). Scenario 4 observed at site Work 1.

Figure 7.14: Probability density function of the significant wave height (H_{m0}) for the construction area 'Work 1' for four different moments in construction progression: initial wave conditions, Scenario 3 (33%) and Scenario 4 (50%).

Table 7.7: Work Method 2: Mean workable windows for the site Work 1, for the initial conditions, Scenario 3 and Scenario 4. Summation of workable windows is for the months April - September.

Mean values	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Initial	18	26	30	38	43	44	47	44	34	25	21	20	33
Scenario 3	21	28	35	41	45	47	49	47	36	28	25	25	35
Scenario 4	25	31	40	45	48	52	54	51	41	32	29	28	39

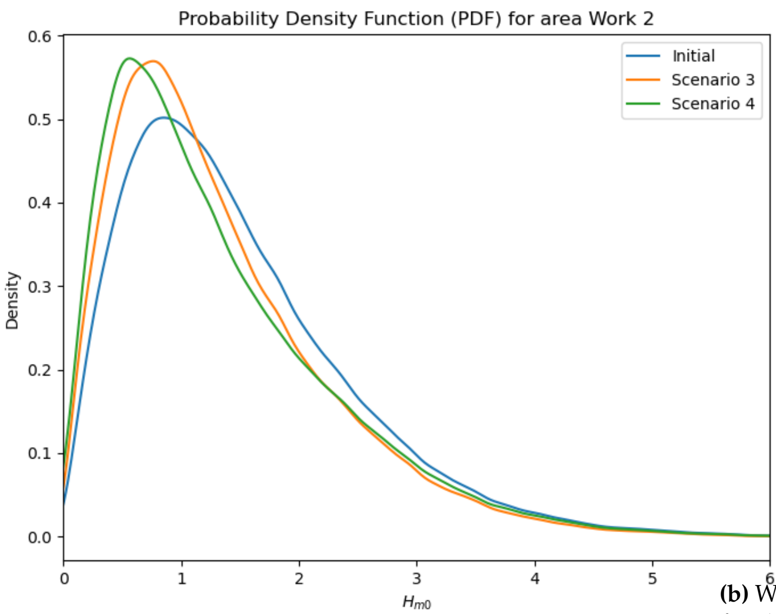
Table 7.8: Expected number of installed caisson per month and scenario for site Work 1, based on an exceedance probability of 70% (P70) and a success rate of 10%. Last column lists the expected total number of installed caissons per work season, which is indicated by the underlined numbers.

Installation	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Sum
Initial	1.6	1.9	2.6	**3.5**	**4.0**	**4.2**	**4.3**	**3.9**	**2.9**	2.2	1.5	1.5	3.3	23
Scenario 3	1.8	2.3	**2.9**	**3.8**	**4.2**	**4.6**	**4.7**	**4.5**	**3.3**	2.5	1.9	1.8	3.5	28
Scenario 4	2.0	2.7	**3.3**	**4.2**	**4.6**	**4.9**	**5.1**	**4.8**	**3.7**	2.8	2.7	2.2	3.9	31

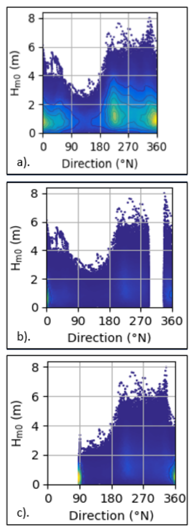
Project Site: Work 2 The second project site, Work 2, is located in the west and will therefore be sheltered from waves origination from the north-east direction. This can be seen in the wave height vs. wave direction graphs in Figure 7.15b: considering a dam progression of 1/3 (or 33%) the western site experiences shelter for the directions 20° - 70°, whereas it will be fully exposed to the north-west till south-west. This will improve when the caisson dam progresses: for Scenario 4 (50% finished) northern waves are diffracted, which slightly improves the workability.

The development of the wave climate over time can be seen in Figure 7.15a, which clearly indicates an improvement between Scenario 3 and Scenario 4. In the workability perspective, the number of workable windows also increases, which is shown in Table 7.10. This table lists the expected number of installation caissons per month for the three scenarios (initial conditions with no shelter, Scenario 4 and Scenario 5), based on an exceedance probability of 70% and a success rate of 10%.

For the initial situation, the work season is the same as has been described in Section 5 and starts in April and lasts till September. One of the advantages of the sheltering effects of the caisson dam is that the workability improves in the spring and autumn, which enables the extension of the work season. For instance, the project can start one month earlier when reaching a progression of 33%. In Table 7.10 the work season is indicated per scenario by the underlined and bold numbers. This increases the total number of installed caissons for one work season substantially.



(a) Probability density function (PDF).



(b) Wave height as a function of wave direction for a). initial, b). Scenario 3 and c). Scenario 4 observed at site Work 2.

Figure 7.15: Probability density function of the significant wave height (H_{m0}) for the construction area 'Work 2' for four different moments in construction progression: initial wave conditions, Scenario 3 (33%) and Scenario 4 (50%).

Table 7.9: Work Method 2: Mean workable windows for the site Work 2, for the initial conditions, Scenario 3 and Scenario 4. Summation of workable windows is for the months April - September.

Mean values	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Initial	18	26	30	38	43	44	47	44	34	25	21	20	33
Scenario 3	18	29	33	42	46	45	47	45	35	26	24	22	34
Scenario 4	21	30	35	45	48	48	49	46	37	29	25	22	35

Table 7.10: Expected number of installed caisson per month and scenario for site Work 2, based on an exceedance probability of 70% (P70) and a success rate of 10%. Last column lists the expected total number of installed caissons per work season, which is indicated by the underlined numbers.

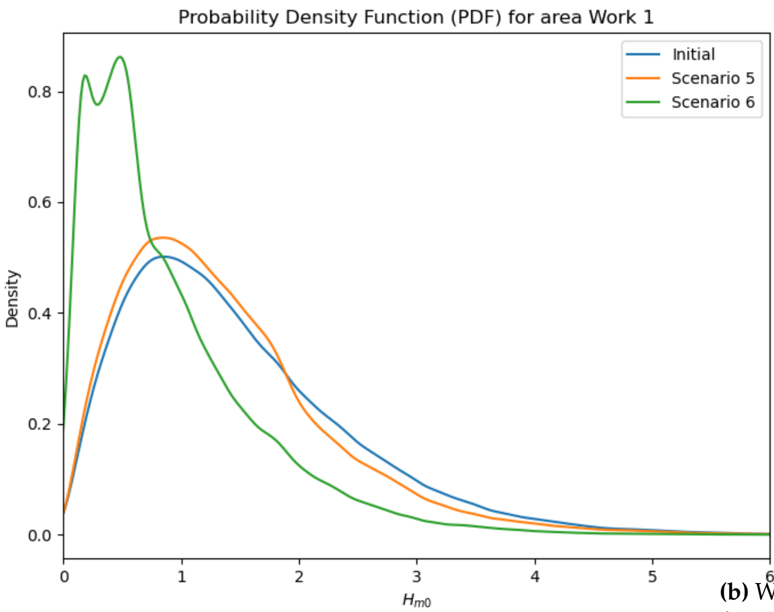
Installation	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Sum
Initial	1.6	1.9	2.6	**3.5**	**4.0**	**4.2**	**4.3**	**3.9**	**2.9**	2.2	1.5	1.5	3.3	23
Scenario 3	1.6	2.1	**3.0**	**3.5**	**4.2**	**4.2**	**4.4**	**3.9**	**3.1**	2.2	1.7	1.5	3.4	26
Scenario 4	1.6	2.4	**3.1**	**4.1**	**4.3**	**4.4**	**4.6**	**4.1**	**3.2**	2.3	1.9	1.6	3.5	28

Work Method 3

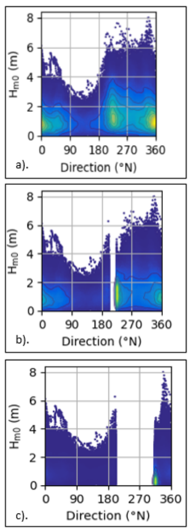
Site: Work 1 Work Method 3 involves Scenario 5 and Scenario 6, which have a different orientation compared to Scenario 3 and Scenario 4 of Work Method 2, therefore will be subjected and experience sheltering of different wave directions. Consequently, the wave conditions and subsequently the workability will also not be the same as the results for Work Method 2.

First, project site Work 1 will be analysed, which is the northern area of the project. As can be seen in Figure 7.13 this area is during the start of the project and during Scenario 5 fully exposed to north-western waves, but might already be sheltered for south-western waves. During the last scenario, the area will be fully sheltered for south-western waves and experience some diffraction for western waves. This can be seen in Figure 7.16b: for Scenario 5 only a little part of the directions around 200° are blocked completely, whereas the caisson dam in Scenario 6 provides shelter for waves between 200° - 300°.

This improvement in shelter can also be seen in Figure 7.16a, indicated by the substantial increase of the probability of smaller waves. In terms of workability for the caisson installation, the initial installation rate is estimated on 23 caissons per year. For Scenario 5 the work season can be brought forward one month, increasing the installed caisson to 27 units per year, an improvement of 17%. Considering Scenario 6 a huge increase can be expected, due to the fact that the construction works do not have to stop in the winter, but can continue the entire year. Obviously, this increases the number of installed caissons significantly and almost doubles to 52 caissons per year.



(a) Probability density function (PDF).



(b) Wave height as a function of wave direction for a). initial conditions, b). Scenario 5 and c). Scenario 6 for site Work 1.

Figure 7.16: Probability density function of the significant wave height (H_{m0}) for the construction area 'Work 1' for four different moments in construction progression: initial wave conditions, Scenario 5 (33%) and Scenario 6 (50%).

Table 7.11: Work Method 3: Mean workable windows for the site Work 1, for the initial conditions, Scenario 5 and Scenario 6. Summation of workable windows is for the months April - September.

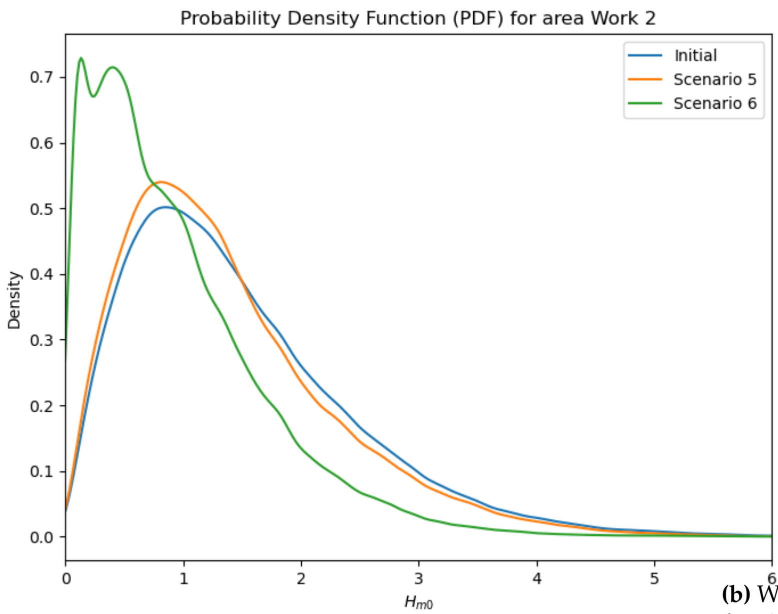
Mean values	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Initial	18	26	30	38	43	44	47	44	34	25	21	20	33
Scenario 5	21	27	32	41	46	47	49	46	37	28	24	22	35
Scenario 6	42	39	46	48	51	52	56	54	49	44	40	41	47

Table 7.12: Expected number of installed caisson per month and scenario for site Work 1, based on an exceedance probability of 70% (P70) and a success rate of 10%. Last column lists the expected total number of installed caissons per work season, which is indicated by the underlined numbers.

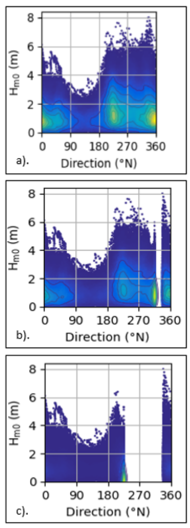
Installation	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Sum
Initial	1.6	1.9	2.6	**3.5**	**4.0**	**4.2**	**4.3**	**3.9**	**2.9**	2.2	1.5	1.5	3.1	23
Scenario 5	1.7	2.2	**2.9**	**3.6**	**4.4**	**4.3**	**4.5**	**4.3**	**3.1**	2.5	1.8	1.7	3.3	27
Scenario 6	**3.7**	**3.5**	**4.1**	**4.3**	**5.0**	**5.0**	**5.4**	**5.1**	**4.4**	**3.9**	**3.6**	**3.6**	4.6	52

Site: Work 2 Lastly project site Work 2, which is located in the south and hence vulnerable and subjected to south-western waves. On the other hand, north-western waves are diffracted or sheltered, depending on the dam's progression. This can be seen in Figure 7.17b: for Scenario 5 only little diffraction and sheltering will occur, blocking a small area around 300°, whereas Scenario 6 will experience a lot sheltering between 225° and 340°. Consequently, this also improves the wave conditions as is shown in Figure 7.17a.

Subsequently, the workability in terms of expected installed caissons, also improves substantially. Table 7.14 summarizes the number of caissons installed, based on an exceedance probability of 70% and a success rate of 10%. Similar to project site Work 1, the work season may be extended throughout the entire year, which increases the number to 50 caisson units per year. Relative to the initial conditions, this is an increase of 117%!



(a) Probability density function (PDF).



(b) Wave height as a function of wave direction for a). initial conditions, b). Scenario 5 and c). Scenario 6 for site Work 2.

Figure 7.17: Probability density function of the significant wave height (H_{m0}) for the construction area 'Work 2' for four different moments in construction progression: initial wave conditions, Scenario 5 (33%) and Scenario 7 (50%).

Table 7.13: Work Method 3: Mean workable windows for the site Work 2, for the initial conditions, Scenario 5 and Scenario 6.

Mean values	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Initial	18	26	30	38	43	44	47	44	34	25	21	20	33
Scenario 5	21	26	33	41	44	47	49	46	37	27	24	23	35
Scenario 6	36	37	44	48	51	52	56	55	47	41	37	36	45

Table 7.14: Expected number of installed caisson per month and scenario for site Work 2, based on an exceedance probability of 70% (P70) and a success rate of 10%. Last column lists the expected total number of installed caissons per work season, which is indicated by the underlined numbers.

Installation	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Sum
Initial	1.6	1.9	2.6	**3.5**	**4.0**	**4.2**	**4.3**	**3.9**	**2.9**	2.2	1.5	1.5	3.1	23
Scenario 5	1.6	2.3	**2.7**	**3.6**	**4.2**	**4.5**	**4.6**	**4.4**	**3.1**	2.4	1.9	1.7	3.3	27
Scenario 6	**3.0**	**3.3**	**4.1**	**4.4**	**4.9**	**5.1**	**5.4**	**5.3**	**4.5**	**3.8**	**3.2**	**3.1**	4.4	50

7.4. Effect on project planning and LCOS

The last step in the study into the optimized work method is the potential effect on the project planning, implications with regard to the deployment of equipment and finally the impact on the Levelised Cost of Storage (LCOS). The approach that will be followed during this last phase is similar to the previous study conducted in Section 7.1.5.

First, a timeline of the dam's progression will be made based on the expected yearly installed caissons, which follows directly from the previous section. The timeline will follow the same classification which has been used in the previous section (i.e. Scenario 3, 4, 5 and 6). For every scenario an equipment assessment will be made, to determine the necessary equipment to ensure the deployed capacity is sufficient to cope the increase in installed caisson units. After the determination of the required equipment, the operational, stand-by and risk costs can be calculated, subsequently the LCOS can be determined for the different work methods.

Work Method 2

Considering a double installation site, working clockwise and anti-clockwise simultaneously, the total project duration decreases substantially. Table 7.15 shows the expected project planning including the installation rate per year, number of installed caissons, number of caissons left and the caisson dam's progression (including the powerhouse section which is considered to be beyond the scope of this research). With the adjusted work method, it turns out that the installation rate is almost doubled compared when only including the sheltering effects, which results in a project duration of just over five years.

Table 7.15: Project planning considering Work Method 2, showing the construction year, the installation rate (site Work 1 and Work 2 combined), number of caissons left, number of caissons installed and the dam's progression in terms of the total length including the power house section.

Year	Installation rate	No. caissons left	No. caissons installed	Finished
1	46	212	0	0%
2	46	166	46	18%
3	54	120	92	35%
4	58	66	146	56%
5	8	8	204	78%
6	0	0	212	81%

According to the probabilistic approach a yearly installation of 23 caissons can be expected for the base scenario, when neglecting the sheltering effects. For the cost effectiveness the equipment has been adjusted and optimized based on the utility rate per vessel category, with input the installation rate. Compared to the base scenario, the installation rate of Work Method 2 assumes an initial installation rate of 46 caisson per year (twice the base scenario), subsequently more equipment and installation capacity is required to meet the new standard. Again, based on the utility rate per vessel, the necessary capacity can be determined.

For the Trailing Suction Hopper Dredge an utility rate around 85% is desired, for the Fall Pipe Vessel this can be up to 90% and the Side Stone Dumping Vessel an utility rate of 80% is appropriate. Applying these utility rates and the expected number of installed caissons the required number of vessels or vessel's capacity can be determined. Moreover, since in the new scenario there are two active project site equipment like the Heavy Lifting Barge (HLB), Anchor Handling Tug (AHT), towing tugs and auxiliary equipment are necessary at both sites. Table 7.16 lists the required number of vessels or vessel's capacity per construction year relative to the base scenario. This will be used to determine the stand-by and operational costs.

Table 7.16: Required number of vessels or vessel's capacity to meet the installation conditions for Work Method 2, listed per construction year, relative to the base scenario.

Year	**1**	**2**	**3**	**4**	**5**
Caissons installed	46 units	46 units	54 units	58 units	8 units
Works season	26 weeks	26 weeks	30 weeks	30 weeks	14 weeks
TSHD	2	2	2	2	1
FPV	1	1	1	1	1
SSDV	2	2	2	2.5	1
HLB	2	2	2	2	1
AHT	2	2	2	2	1
Towing tugs	2	2	2	2	1
Dock workers	1	1	1	1	1
Auxiliary equipment	2	2	2	2	1

In Section 6 all expenditures and weekly rates per vessel category has been determined. Using these rates, the correction for number of vessels deployed for the new work method with respect to the base scenario (Table 7.16) and including the development of the work season over time, the new stand-by rates can be calculated. Table 7.17 lists the expected stand-by costs per vessel category for each year.

Furthermore, the operational costs have been determined in Section 6 as incremental costs per installed caisson unit for all equipment types. This means that every installed caisson will cost a fixed amount for every equipment type, assuming that each vessel spends a constant period for the installation of one caisson and the related scour protections. Therefore, the operational costs can simply calculated by multiplying the caissons installed per year by the incremental costs.

The effect on the Levelised Cost of Storage (LCOS) will be determined in order to evaluate the change in feasibility of the project in a general sense. The LCOS, a financial metric to assess the cost-effectiveness of a certain project, has been introduced and discussed extensively in Section 6, together with all relevant expenditures. Table 7.18 shows all relevant costs, such as the new stand-by and operational costs. Considering the work method has a beneficial effect on the LCOS, and reduces to € 185 MWh^{-1}.

Table 7.17: Overview of the expenditures related to the stand-by activities for each equipment category for Work Season 2, listed per year. The third and fourth row show the total installed caissons and the duration of the work season per year.

Year	**1**	**2**	**3**	**4**	**5**
Caissons installed	46 units	46 units	54 units	58 units	8 units
Work season	26 weeks	26 weeks	30 weeks	30 weeks	14 weeks
TSHD	€ 25,654,886	€ 25,654,886	€ 29,601,792	€ 29,601,792	€ 6,907,085
FPV	€ 13,728,000	€ 13,728,000	€ 15,840,000	€ 15,840,000	€ 7,392,000
SSDV	€ 18,969,350	€ 18,969,350	€ 21,887,712	€ 24,623,676	€ 5,107,133
HLB	€ 13,728,000	€ 13,728,000	€ 15,840,000	€ 15,840,000	€ 3,696,000
AHT	€ 2,024,250	€ 2,024,250	€ 2,335,673	€ 2,335,673	€ 544,990
Towing tugs	€ 1,735,072	€ 1,735,072	€ 2,002,006	€ 2,002,006	€ 467,135
Dock workers	€ 1,820,000	€ 1,820,000	€ 2,100,000	€ 2,100,000	€ 980,000
Auxiliary equipment	€ 5,200,000	€ 5,200,000	€ 6,000,000	€ 6,000,000	€ 1,400,000
Total	**€ 82,859,559**	**€ 82,859,559**	**€ 95,607,183**	**€ 98,343,147**	**€ 26,494,343**

Table 7.18: Concise overview for all categories considered in the LCOS, including the beneficial consequences of the sheltering effects of the caisson dam. Taking into account the year of expenditure and the discount rate, the LCOS can be determined.

Category	Total cost	Present Value (PV)
Caisson costs	€ 607 million	€ 4.0 billion
Rock works	€ 723 million	
Wave return wall	€ 180 million	
Turbine costs	€ 2 billion	
Powerhouse costs	€ 561 million	
Scour protection	€ 86 million	
Stand-by costs	€ 389 million	
Operational costs	€ 292 million	
Subtotal	€ 4.8 billion	
Risk costs	€ 127 million	€ 117 million
O&M costs	€ 10.9 billion	€ 1.7 billion
Charging costs	€ 4.7 billion	€ 0.7 billion
End-of-life costs	€ 120 million	€ 1 million
Subtotal	€ 15.9 billion	€ 2.5 billion
Total costs	€ 20.7 billion	€ 6.5 billion
Electrical discharge	2.31×10^8 MWh	3.5×10^7 MWh
LCOS	€ 185 MWh^{-1}	

Work Method 3

Lastly, Work Method 3 considers a different starting point of the construction works and hence the orientation of the powerhouse section will face towards the east, providing good working conditions for the installation of the powerhouse units. This can also been seen in Table 7.19, which shows the expected project planning. With this new work method, the facility (excluding the powerhouse units) can already been finished within four years! The overview of the project planning, including the installation rate per year depending on the dam's progression. is shown in Table 7.19.

The first two years the installation rate will be the same as during the initial conditions, since the sheltering effects are negligible. During the second year, a dam progression of 33% will be obtained, implying that sheltering effects will favorably affect the conditions and hence the installation production will increase. From this moment, it is assumed that the installation rate is 54 caisson units per year, equivalent of 27 caissons per installation site. With this increased installation rate, a dam progression of 50% will be reached in the last period of year 3. As has been discussed in the previous section, this implies that construction works can be executed throughout the entire year.

Compared to the work season that is considered for Scenario 5 (dam's progression of 33%), the improved work season for Scenario 6 (dam's progression of 50%) is extended by five months in total: two months before and 3 months after the initial work season. The transition from Scenario 5 to Scenario 6 will be reached before the end of the work season, therefore at the end of the work season that applies to Scenario 5, the work season will also transits to the season that applies for Scenario 6. In other words, Year 3 will start with the Scenario 5 conditions, but will end with the Scenario 6 conditions. Consequently, Year 3 will end with a continuous work season. Installation-wise, this implies that in the last three months of Year 3 a total of 10 caisson units can be installed. This has been noted separately in Table 7.19.

The equipment that is necessary to meet the utility rate standards that have been assumed for the various types of vessels (SSDV: 80%, TSHD: 85% and FPV: 90%) and for practical reasons, are shown in Table 7.20. It can be noticed that the number of vessels or capacity, is similar to the required equipment that is necessary for Work Method 2. This has to do with the fact that although the absolute number of caissons that are installed per year are different, the relative installation rate (no. caissons per week) is approximately the same.

Subsequently, the stand-by costs related to all types of equipment can be determined, which depend on the multiplication factor and the duration of the work season. An overview of the expenditures are listed in Table 7.21, which also shows the yearly sum of the stand-by costs. Together with the project planning and all other expenditures that have been discussed earlier, the effects of sheltering and diffraction on the project planning and costs can be determined in terms of the change in Levelised Cost of Capital (LCOS). Table 7.22 lists all expenditures related to the project, including the adjusted stand-by and operational costs, in order to determine the LCOS: it turns out that the LCOS considering the new work method is € 180 MWh^{-1}.

Table 7.19: Project planning considering Work Method 3, showing the construction year, the installation rate (site Work 1 and Work 2 combined), number of caissons left, number of caissons installed and the dam's progression in terms of the total length including the power house section. *Due to the improved workability that will be achieved before the end of Year 3, the work season can be extended up to December. This enables the installation of an additional 10 caisson units during Year 3.

Year	Installation rate	No. caissons left	No. caissons installed	Finished
1	46	212	0	0%
2	46	166	46	18%
3	54 + 10*	120	92	35%
4	56	56	156	60%
5	0	0	212	81%

Table 7.20: Required number of vessels or vessel's capacity to meet the installation conditions for Work Method 3, listed per construction year, relative to the base scenario. *Year 3 will include the transition between the workability of Scenario 5 and Scenario 6. The former refers to the first phase (Scenario 5) of the year whereas the latter refers to the last phase (Scenario 6).

Year	1	2	3	4
Caissons installed	46 units	46 units	54 / 10* units	56 units
Work season	26 weeks	26 weeks	30 / 14* weeks	30 weeks
TSHD	2	2	2	2
FPV	1	1	1	1
SSDV	2	2	2.5	2
HLB	2	2	2	2
AHT	2	2	2	2
Towing tugs	2	2	2	2
Dock workers	1	1	1	1
Auxiliary equipment	2	2	2	2

Table 7.21: Overview of the expenditures related to the stand-by activities for each equipment category for Work Season 3, listed per year. The third and fourth row show the total installed caissons and the duration of the work season per year.

Year	**1**	**2**	**3**	**4**
Caissons installed	46 units	46 units	64 units	56 units
Work season	26 weeks	26 weeks	44 weeks	30 weeks
TSHD	€ 25,654,886	€ 25,654,886	€ 43,415,962	€ 29,601,792
FPV	€ 13,728,000	€ 13,728,000	€ 23,232,000	€ 15,840,000
SSDV	€ 18,969,350	€ 18,969,350	€ 40,127,472	€ 21,887,712
HLB	€ 13,728,000	€ 13,728,000	€ 23,232,000	€ 15,840,000
AHT	€ 2,024,250	€ 2,024,250	€ 3,425,654	€ 2,335,673
Towing tugs	€ 1,735,072	€ 1,735,072	€ 2,936,275	€ 2,002,006
Dock workers	€ 1,820,000	€ 1,820,000	€ 3,080,000	€ 2,100,000
Auxiliary equipment	€ 5,200,000	€ 5,200,000	€ 8,800,000	€ 6,000,000
Total	**€ 82,859,559**	**€ 82,859,559**	**€ 148,249,363**	**€ 95,607,183**

Table 7.22: Concise overview for all categories considered in the LCOS, including the beneficial consequences of the sheltering effects of the caisson dam. Taking into account the year of expenditure and the discount rate, the LCOS can be determined.

Category	Total cost	Present Value (PV)
Caisson costs	€ 607 million	€ 4.0 billion
Rock works	€ 723 million	
Wave return wall	€ 180 million	
Turbine costs	€ 2 billion	
Powerhouse costs	€ 561 million	
Scour protection	€ 86 million	
Stand-by costs	€ 410 million	
Operational costs	€ 298 million	
Subtotal	€ 4.9 billion	
Risk costs	€ 127 million	€ 118 million
O&M costs	€ 11.1 billion	€ 1.7 billion
Charging costs	€ 4.8 billion	€ 0.8 billion
End-of-life costs	€ 120 million	€ 1 million
Subtotal	€ 16.1 billion	€ 2.6 billion
Total costs	€ 20.9 billion	€ 6.6 billion
Electrical discharge	2.34×10^8 MWh	3.7×10^7 MWh
LCOS	€ 180 MWh^{-1}	

Sensitivity analysis The sensitivity of the LCOS with regard to various characteristics (utility rate, turbine efficiency, discount rate and future electricity price) are listed in Table 7.23. This assessment shows that the LCOS ranges between € 136 - 261/MWh, depending on the actual value.

Caisson storage One of the problems that might occur during high installation rates is a dis-function in the caisson storage. Either a shortage or abundance of caisson units might occur in case the installation rate is out of proportion relative to the production rate.

Therefore, a preliminary assessment has been made based on the installation rates introduced in Section 7. Furthermore, the production rate corresponds to a double caisson fabrication facility, as mentioned in Appendix E. This implies that every month, five caisson units will be delivered at the intermediate wet storage.

Table 7.23: Sensitivity analysis for the parameters of the offshore PSH plant that determine the levelised cost of storage (LCOS), for the utility, turbine efficiency, discount rate and future electricity prices. The initial parameters for this analysis: social discount rate r = 5%, turbine efficiency η = 70%, Operation and Maintenance (O&M) rate of 1% for the caisson dam and 4% for the powerhouse section and its turbines, Depth of Discharge DoD = 80%, Future electricity price P = € 10 / MWh, utility rate = 20%, for Work Method 3 excluding the powerhouse section.

Utility	30%	25%	20%	15%	10%
	€ 153	€ 164	€ 180	€ 207	€ 261
Turbine efficiency	80%	75%	70%	65%	60%
	€ 150	€ 164	€ 180	€ 199	€ 223
Discount rate	3%	4%	5%	6%	7%
	€ 136	€ 157	€ 180	€ 205	€ 231
Electric price	€ 0 / MWh	€ 5 / MWh	€ 10 / MWh	€ 15 / MWh	€ 20 / MWh
	€ 159	€ 170	€ 180	€ 190	€ 200

Figure 7.18 shows the number of caissons in storage (orange) and the number of caissons that will be installed per month (blue). It is assumed that the production will continue throughout the entire year, whereas the installation will be bounded by the determined work season.

It can be seen that the fabrication rate is sufficient for the installation rate: during the project, there will be no lack of caisson units. Before the last year, the storage will be maximum and peaks at approximately 50 caisson units. This large buffer will be necessary to cope with the high installation rates later in the project.

However, this also might be a problem with regard to spatial and logistic planning. The question rises where one can store 50 caisson units. Earlier in this research, a location in the Port of Rotterdam has been appointed as a potential temporarily caisson storage which can facilities 30 units. This means that, if the latter storage location is still a viable option, an extra storage location must be found.

One possible solution might be to anchor the units inside the reservoir, a sheltered area. However, this has not been part of this research, and hence has not been studied yet.

7.5. Conclusion

It turns out that the effects of sheltering due to the constructed caisson dam have a substantial impact on the overall workability, regarding the caisson installation and the vessel activities. A brief overview of the cases that have been studied, including the relevant parameters, is shown in Table 7.24.

Table 7.24: Overview of the study cases that are considered for the optimization process, among them are the inclusion of sheltering and diffraction phenomena, number of project sites, starting point of the construction works and the corresponding work direction.

Case	Sheltering	Project sites	Starting point	Work direction
Base	N/A	One	North	Anti-clockwise
Case 1	Sheltering and diffraction	One	North	Anti-clockwise
Case 2	Sheltering and diffraction	Two	North	Anti- and clockwise
Case 3	Sheltering and diffraction	Two	West	Anti- and clockwise

In order to present a comprehensive but brief summary, the parameters that have been discussed in the previous paragraphs will be shown, including: i). change in wave conditions between the initial conditions and final phase, ii). average installation rate, iii). construction time for only the caisson dam (excluding the powerhouse) and iv). the Levelised Cost of Storage (LCOS). An overview of these results is shown in Table 7.25.

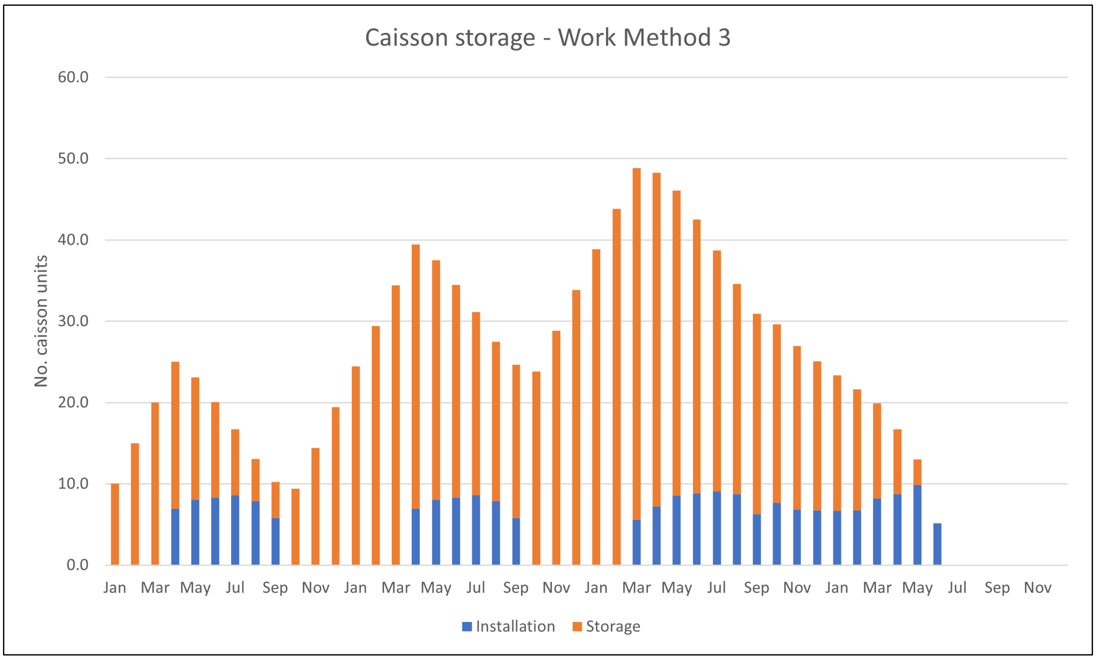


Figure 7.18: Caisson storage for Work Method 3 (optimized based on the sheltering effects) when considering a constant caisson production of 5 caisson units per month. The installation rate follows the approach discussed in Section 7. The installation has been visualised in blue and the storage in orange.

Table 7.25: Brief summary of the most relevant results for the case studies into the sheltering effects and work method optimization. Number of workable windows are given for the initial conditions, a dam progression of 33% and 50% respectively. Number of workable windows are given for an exceedance probability of 70%, for the three situations: no caisson dam (initial conditions) and a dam progress of 33% and 50% respectively.

Case	No. workable windows	Construction time	LCOS
Base	228 / 228 / 228	9 years and 2 months	€ 212
Case 1	228 / 252 / 298	8 years and 4 months	€ 204
Case 2	456 / 541 / 583	4 years and 2 months	€ 185
Case 3	456 / 542 / 1017	3 years and 6 months	€ 180

According at Table 7.25, there are some conclusions that can be drawn based on the chosen parameters.

It turns out that including sheltering effects of the caisson dam and its diffraction have a substantial impact on the project planning and subsequently on the cost-effectiveness (LCOS) of the project. Additionally, it is worth mentioning that the optimization with regard to work method (e.g. the starting point of installation in combination with the number of project sites) has a significant impact on the workability, construction time and in turn on the LCOS.

8 Discussion

This section reflects on the main results and its implications on the research questions. Furthermore, shortcomings or limitations that have been faced throughout the research and the subsequent impact on (the reliability of) the results will be addressed.

First, the impact that major limitations in the methodology and shortcomings in the assumptions may have had on the results will be examined. Second, the results of all four cases will be discussed and compared thoroughly. These consist of the base case that considers the probabilistic approach but neglects the limiting wave conditions, and three in-depth studies into the effects of sheltering, diffraction and also work method changes.

Levelised Cost of Storage (LCOS) For the determination of the cost-effectiveness, in this case the Levelised Cost of Storage (LCOS), various assumptions have been made. Often, these assumptions could not be underpinned with literature, but were made based on common sense or expertise of the engineers of TU Delft's and Boskalis. However, it is worth mentioning which assumptions have been made with the corresponding limitations and consequences.

The largest sensitivity of the LCOS is the discount rate. A social discount rate has been assumed, seen the social interests and importance. Typically, a social discount rate is more favorable than the commercial rate. However, literature offers a wide range for the discount rate, varying between 2-7%. As a first assumption, a discount rate of 5% has been used throughout this research. According to the sensitivity analysis, the difference between a 3% and 7% discount rate is a LCOS of € 147/MWh and € 300/MWh, respectively. Therefore, it can be concluded that the discount rate contains a substantial uncertainty.

Furthermore, the expected utility rate of the facility is hard to estimate. Conventional pumped storage hydropower utility rates range between 1% up to 40%, depending on the location and purpose of storage. The applied utility rate (20%) might be slightly high, which would over-estimate the production and hence the cost-effectiveness.

The PSH utilizes newly developed turbines, dedicated to low head differences, which aims to perform substantially better than existing turbines. The research and development is still underway. The turbine efficiency has a large contributing in the feasibility of the project in general. In case the full-scale turbine under-performs, the project might not be feasible anymore. Thus, the research and development of the turbine partly determines the fate of this project. However, it might be that similar turbines will be developed in the meantime, which are also suitable for this project.

Lastly, the electricity price during charging of the reservoir has been thoroughly assessed based on existing literature. In this research, it is assumed that the future electricity price will be € 10/MWh. However, it can be argued that, especially seen the enormous developments regarding the renewable resources, the charging price of the reservoir (which will be during the cheapest daily electricity hours) would be close to € 0/MWh. Especially when including the possibility of negotiations with energy suppliers. This would reduce the Operational Expenditures, even further enhancing the cost-effectiveness.

Goda software One of the limitations faced throughout this thesis is the diffraction software, which is based on the Goda diffraction tables. The representation of a round caisson dam by a combination of straight breakwaters should have given a first estimate of the sheltering effects. However, it must be noted that this simplification does not include important physical processes.

First of all, the hydraulic response to a round breakwater will be different than a straight breakwater. Physical processes near the caisson dam, such as currents and wave propagation, might be different than for the simple straight breakwater. This entails undesired risks with regard to the design of the caisson and rock works, but in particular would be interesting for the execution phase.

Additionally, the assessment only includes diffraction of the caisson dam, thereby neglecting reflection. Considering the vertical and impermeable characteristics of the caisson dam, the dam will be fully reflect incident waves. Hence, wave conditions on the windward side will worsen due to the reflected waves.

Summarizing, both limitations of the software affects the wave conditions for both considered locations (i.e. the area for rock works installation and caisson installation), either positive or negative. Consequently, this evokes a large uncertainty in the workability and therefore also for the project planning. In turn, the determined cost-effectiveness for each case is based on a simplified situation and could be substantial different when mitigating these limitations.

Sheltering effects and its cost-effectiveness This research has shown that including and optimization based on the sheltering effects, can substantially enhance the feasibility of an offshore project. It must be noted that the work method and construction phasing play a vital role in the effect on workability, construction time and LCOS. Choosing this appropriately, for instance to maximize the sheltering for the dominant wave direction (north-west and south-west) in an early construction stage, may even further increase the efficiency, in turn enhancing the cost-effectiveness of the offshore pumped storage hydropower (PSH) concept.

Logistic hurdles in the caisson stream The project's progress depends on the installation rate which follows from the workability assessment. Part of this approach is the choice to what extent uncertainty will be included in the form of the exceedance probability.

In general, a conservative value is more desired in projects since a setback is worse than a more favorable situation. Therefore, a workability that corresponds to an exceedance probability of 70% has been chosen.

By incorporating a conservative value, the project planning has already taken into account uncertainties and potential delays. The work method has been modified and the equipment has been optimized considering the conservative number of caisson units to be installed per year. However, it is plausible that a more favorable year may occur, leading to the installation of additional caissons. Consequently, this might require more equipment than what is currently accounted for in the work method.

The current work method, including the deployed equipment, may pose a restriction for caisson installation during 'good weather' years, as it has been designed based on conservative values. In case of consecutive favorable months, in which more caisson will be installed than expected, the demand might be higher than the supply.

This rises the following challenge: caisson storage. In Section 7 a quick calculation showed that the caisson storage must contain approximately 60 caisson units at its maximum. This would be the equivalent of a quay wall of 3.6 km! This poses a spatial challenge: where can these caissons be stored? In this research a temporarily wet storage in the Port of Rotterdam was appointed (with a capacity of 25 units) which was not sufficient for this project.

Structural and execution assessment During the construction of the structure, waves can attack both sides of the caisson unit. This research has only considered wave attack of the front side or ocean side, whereas waves on the reservoir side has been neglected. Corresponding structural and execution assessments has not been made, while these may be relevant. The outcome of this analysis might affect the work method or design of the structure, consequently increasing the construction costs.

Permits and environmental impact Although it was not part of this research, permits and the environmental impact of this project can not be neglected. Besides all the technical challenges that are faced during this process, many more will follow regarding the environmental impact and corresponding mitigation measures.

The construction of an offshore mega-structure will have consequences for the sea-life. To what extent these consequences can not mitigated or avoided, has not been the part of this research, and is yet unknown. Taking into account permits and mitigation measures for environmental impact will certainly have an effect on the project.

9 Conclusions

The objective of this study was to assess the cost-effectiveness of an offshore Pumped Storage Hydropower (PSH) plant and, ultimately, enhance its value by optimizing construction costs through adjustments in the construction phasing of the caisson dam. The main research question of this thesis was:

”To what extent can the construction costs be optimised by alternating the construction phasing of the caisson dam, not considering the powerhouse and related infrastructure?”.

The main findings can be summarized as follows:

- A new location, which has also been addressed by van der Wel (2021), has been proposed and is located 50 km off the coast of Texel. This location does not interfere with existing nor planned infrastructure. The maximum water level difference between the reservoir and sea level is estimated on 35 - 40 m.
- The facility consists of a caisson dam with a total length of 12.7 km and a powerhouse section of approximately 3 km. With a diameter of 5 km and a local water depth of 30 m, the total gross capacity is 23 GWh. The daily energy production has been determined on 6.7 GWh.
- On average, the Capital Expenditures are estimated on approximately € 5 billion. Additionally, over a life-time of 100 years, the Operational Expenditures (OPEX) are estimated on € 16 billion.
- By optimizing the work method and phasing based on the sheltering effects and governing wave directions, the construction time can be reduced from 9 years and 2 months to only 3 years and 6 months. Subsequently, this enhances the cost-effectiveness of the project from a LCOS of € 212/MWh to only € 180/MWh!
- Offshore Pumped Storage Hydro (PSH) emerges as a potential bulk energy storage technology, with a competitive Levelized Cost of Storage (LCOS) ranging from € 140-260/MWh, compared to alternative technologies such as lithium-ion (€ 200-400/MWh) and hydrogen storage (€ 200-1900/MWh).

It turns out that the caisson installation is the governing activity in the entire project. Maximizing the efficiency and installation rate of this process will enhance the construction time and thus the construction costs. When incorporating wave data statistics, a more realistic and sophisticated project planning has been composed, resulting in a construction time of 9 years.

Optimization of the work method applied for the caisson installation, based on the local wave climate, showed a substantial improvement with regard to the cost-effectiveness. For the initial scenario, which did not include sheltering effects nor work method optimization, the Levelised Cost of Storage (LCOS) was € 212/MWh. Taking into account sheltering effects, and a first optimization of the work method resulted in a reduction of the construction time by nearly half to 4 years and 2 months, subsequently improving the LCOS to € 185/MWh. Further adjustments to the work method led to a decrease in LCOS to € 180/MWh, along with a gain of 8 months in construction time.

In conclusion, the objective of this research was to enhance the existing knowledge and insight for the construction of an offshore pumped storage hydropower facility. The findings of this research demonstrate that by incorporating sheltering effects and optimizing the work method for offshore projects, there is a tremendous potential for significantly enhancing the cost-effectiveness. Moreover, this thesis highlights the competitiveness and feasibility of the offshore PSH once again!

10 Recommendations

Although this research was conducted with the intention to represent the actual conditions or scenarios as precise and accurate as possible in order to obtain the most realistic results, there are various aspects that should be mentioned and issued to put this research in the appropriate perspective. Furthermore, throughout this study several topics have passed which literature or knowledge currently lacks, and hence could use additional research to gain insight in these various topics.

Installation accuracy as a function of workability One of the key findings found during this research is the dependency between installation accuracy and workability. This finding has not been taken into account nor discussed, since the implications and processes are too complicated.

There is a relation between the desired installation accuracy and the workability: during though conditions the exact placement of a caisson might take substantially longer than during calm conditions. Thus, allowing an inaccuracy during installation might have a significant effect on the installation rate.

On the other hand, this implicitly means that the gaps between the caissons will also increase. In practice, it is found that filling large gaps (couple of meters) is challenging. Studying the effect of the installation accuracy on the workability, but also the adjustment in work method to cope with larger gaps, is recommended in further research.

Onshore / Offshore costs Many expenditure categories found in literature are often related to onshore activities, structures or maintenance and repair costs. For instance, the maintenance and operation costs for the powerhouse section used in this research were based on actual expenditures of existing pumped storage hydropower plants. However, it must be noted that offshore operations always will be more expensive due to the offshore type of equipment, reduced workability and limited accessibility of the project.

There was no literature found how to include these offshore characteristics, based on a similar onshore project. Consequently, it could be that the offshore character has not been represented appropriately and hence the levelised cost of storage may be different when taking this fact into consideration.

Hindcast weather data Using 42 years of hindcast data the probabilistic planning has been composed to evaluate and set a baseline for the optimization of the construction costs when including the sheltering effects of the caisson dam.

However, the problem with probabilistics is that it assumes independent, homogeneous data that does not change abruptly over time. Contrary to the current situation: extreme weather conditions occur at a more frequent base than it used to be. The effect on offshore wind and wave conditions could be substantial, as recent study shows that global wave power has increased due to global warming (Reguero, Losada, & Méndez, 2019).

As a consequence, excluding these developments and changes in global systems, the workability and subsequently the probabilistic planning, but also the design of the caisson unit is not represented appropriately. Therefore, for further research it is definitely interesting to study the effects of climate change on the construction planning of an offshore project and the developments with regard to offshore weather conditions.

Social discount rate The social discount rate is used for projects that function the society in one of many aspects. Often these projects are mainly funded by the governmental organizations and hence the desired return on the project is considerably less than a commercial project.

The actual discount rate depends on the terms and conditions in the joint venture agreement, but is mainly determined by the expected risk of the project. Since typical social discount rates vary between 2% and 7%, the subsequent outcome of the cost-benefit analysis or levelised cost of storage has a wide range.

The determination of the exact discount rate was considered to be beyond the scope of this research, and hence a value of 5% has been applied. Considering the significant impact of the discount rate on the levelised cost of storage and hence the feasibility of the entire project, additional research should including a determination of the social discount rate, based on existing similar (energy) projects.

Inner berm design The preliminary design of the structure contains an inner berm, constructed with sandy material dredged at the centre of the reservoir. Since this is an enclosed basin, only wind generated waves can occur. This is important with regard to the design of the inner berm revetment.

The first assumption is that the inner berm, requires a protection layer to prevent erosion of the berm due to the (wind) wave impact. For this project a stone revetment, consisting of a LM_{α} 10 - 60 kg grading and a geotextile over the entire inner berm, was considered to prevent extensive erosion of the inner berm.

However protecting the entire surface of the inner berm with a LM_{α} 10 - 60 kg grading will substantially increase the costs: from the cost assessment it follows that only the inner berm revetment was roughly one third of the total rock works expenditure. This has risen the question whether there are no suitable and viable alternatives to reduce this major expenditure. There are a few alternative solutions that are considered to be feasible and economically attractive relative to the stone revetment scenario.

It is recommended for further research to study alternative design options. Interesting could be to leave the inner berm unprotected, allowing erosion. In addition, a small dredging vessel will be deployed for repair works. An other option is only to protect the most attacked side of the reservoir with a stone revetment.

Optimization with Goda's diffraction figures It was expected that the implications of the diffraction and corresponding sheltering effects of the caisson dam would contribute to an enhancement of the cost-effectiveness, which is confirmed by the results. For this analysis, two potential methods were available: the Goda diffraction figures or the application of a hydrodynamic model.

Despite the fact that this method shows that including diffraction will enhance the wave conditions behind the caisson dam, the application of the Goda's theory does simplify the project substantially. Additionally, the reflection of the caisson dam has not been taken into account. Therefore, it is suggested that an extensive hydrodynamic study of a round caisson dam will be included in further research.

Powerhouse and related infrastructure The installation of the powerhouse and related infrastructure is beyond the scope of this research and hence has not been considered. The construction time of the powerhouse section has been assumed to be constant for all considered scenarios and will last 2 years.

However, especially when looking at the difference in workable windows during the last phase of the project for all cases (228, 298, 583 and 1017 workable windows for Base Case, Case 1 to 3, respectively) it can be argued that the installation time of the powerhouse units will not be constant. For Case 2 and Case 3 the workability almost improves with 100%.

For further research it is recommended to take this phenomena into account, as this will have a substantial impact on the project planning and subsequently on the cost-effectiveness.

Improvement in risk costs and installation success due to sheltering A part of the construction expenditures are related to risk costs, which has been determined as a fixed amount per caisson unit and constant over time. However, due to sheltering the wave climate improves and becomes milder. This raises the question whether this decreases the risk costs in general. Although the risk costs are not significant compared to the material costs, it might be an interesting topic to further investigate in additional research.

Furthermore, the success rate has been observed to be relatively low, estimated in the range of 10-20%. The success rate of offshore caisson projects, despite being briefly discussed, holds potential as an interesting topic for further investigation. Due to the sheltering effects, the success rate might increase, as the installation conditions will improve. Understanding the development of the success rate over time and its impact on project planning and construction duration could be valuable, ultimately enhancing the cost-effectiveness of the project.

Environmental and social impact The environmental and social impact of this enormous project has not been discussed at all in this study. Obviously, the construction of an offshore pumped storage hydropower plant has substantial impact on both aspects. However, since the goal of this research was to optimize the work method to increase the cost-effectiveness of this energy storage concept, these aspects has not been considered.

Construction an offshore reservoir which will induce high flow velocities near the turbines might have an effect on the flora and fauna. Moreover, it might be valuable to study the effect of the presence of an impermeable structure in the North Sea on (tidal) currents. Lastly, the impact on local fisheries should be taken into account as well. Including these aspects in further research through an extensive environmental and social impact assessment is recommended.

Conservative installation rate In general, a conservative value is desired in projects since a setback is worse than more favorable conditions. In turn, by incorporating a conservative value, the project planning has already taken into account uncertainties and potential delays. The work method has been modified and the equipment has been optimized considering the conservative number of caisson units to be installed per year.

The current work method, including the deployed equipment, may pose a restriction for caisson installation during 'good weather' years, as it has been designed based on conservative values. An extensive assessment which analyses the resilience of the equipment and caisson stream for scenarios with regard to favorable weather conditions is recommended.

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A
Site description

A.1. Geo-technical analysis

Prior to the design and planning phase of the project, a geo-technical assessment must be conducted to evaluate the soil characteristics and conditions to determine the successful and feasibility of a certain location in the North Sea. Since the PSH imposes a hydraulic head difference between two locations, i.e. sea level outside the dam and the local water level of the reservoir, in order to temporarily store energy, the soil characteristics must meet two requirements with regard to maximum imposed head difference.

Firstly, up-burst of the underlying impermeable subsoil layer may occur when insufficient mass or weight is resting on the particular layer. For instance, considering the maximum head difference - when the reservoir is empty - the upward pressure underneath the subsoil is still present, whereas the counteracting force induced by the water mass is absent, as the reservoir is empty. The only retaining force is the mass of the impermeable subsoil and the layer above the impermeable layer. Depending on this weight, up-burst of this layer may occur. This phenomena is shown in Figure A.1 (de Vilder, 2017). Here a permeable sand layer is located underneath an impermeable clay layer, of height h, with a head difference of δH resulting in an upward water pressure. The corresponding forces are given by Equation A.1 and A.2; the equilibrium - which is the threshold beyond which bursting of the clay layer is not expected - of the height of the clay layer is given by Equation A.3.

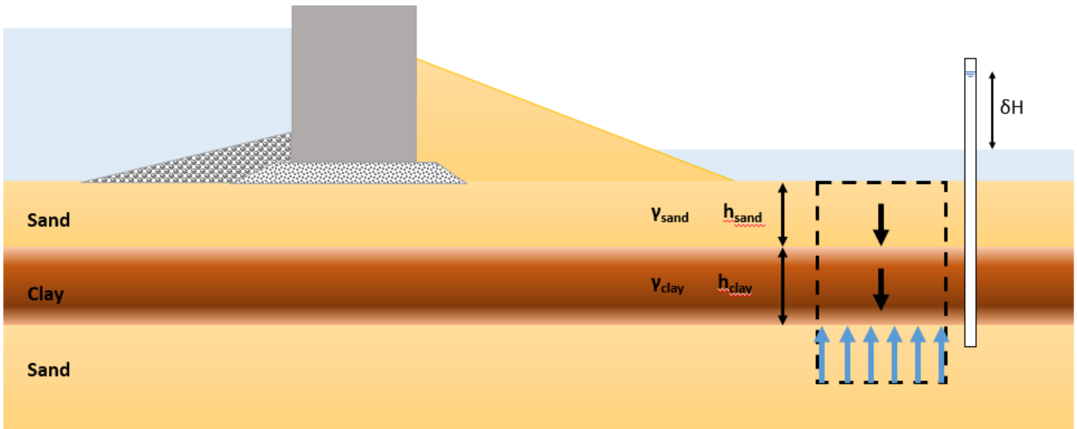


Figure A.1: Bursting - or lifting up - of an impermeable layer due to the difference in hydraulic head between the bottom and top of the impermeable layer, resulting in an upward water pressure, counteracted by the self-weight of the impermeable layer. After de Vilder (2017)

$$F_p = \gamma_w \cdot \delta H \tag{A.1}$$
$$F_s = (\gamma_{sat,s} - \gamma_w) \cdot h \tag{A.2}$$
$$h = \frac{\gamma_w}{\gamma_{sat,s} - \gamma_w} \cdot \delta H \tag{A.3}$$

Secondly, due to the water level difference, water seeps through the soil layers and will reach the reservoir, also known as seepage. Significant seepage will decrease the efficiency of the facility, since the water level in the reservoir will increase and hence the effective storage will decrease. The magnitude of seepage depends on the permeability and thickness of the soil layer, but also on the present hydraulic head difference. The specific discharge through the entire surface of the reservoir can be calculated by Darcy's law, which is shown in Equation A.4.

$$Q_u = \int_0^R 2\pi r \frac{\varphi - \varphi_0}{d} k dr = 2\pi R \frac{\varphi_1 - \varphi_0}{d} k \tag{A.4}$$

in which:

Q_u [m^3/d] = total seepages discharge through bottom of the reservoir
R [m] = radius of the reservoir
r [m] = distance from the centre of the reservoir
φ [m] = piezometric level for a specific r (φ_1 for r = R and φ_0 for r = 0)
k [m/d] = soil permeability
d [m] = thickness/height of the impermeable layer

This is important, especially for the design phase, in order to verify the current design on the possible failure mechanisms regarding piping and macro slope failure. Moreover, the up-burst criterion determines the maximum allowable head difference the PSH plant can generate, and hence will determine the maximum storage capacity.

DINOloket is a Dutch governmental authority that collects geo-technical data for the Netherlands, both on land and offshore, such as lithology and sieve curves for thousands of samples. Despite the fact that additional geo-technical surveys and investigations will be necessary during the final design phase, for a preliminary design the available data from DINOloket is sufficient to determine the most important parameters with some certainty. Prior to this research, the available geotechnical data from the DINOloket in the North Sea has been analysed by van der Wel (2021) and made a visualisation with regard to the most feasible locations considering the up-burst criterion and seepage, also shown in Figure A.2.

From this research several potential locations across the North Sea are suggested. From the results it can be concluded that the earlier proposed Site 01 and 02 show a lot of potential regarding the maximum head difference. Figure A.2 shows that in Site 02 a maximum water difference of approximately 30-35 m can be achieved. The only disadvantage is that the size of the corresponding area is not suitable for the desired size of the facility. To that end, Site 02 will not be considered during further analysis.

Furthermore, new interesting areas also arise, for example an area in front of the Waddeneilanden and some offshore areas more in the Northern part. Considering the location of the latter, the accessibility will not be great since it is more than 100 km offshore. Therefore, the site investigation will include Site 01 and the new proposed area nearby the Waddeneilanden.

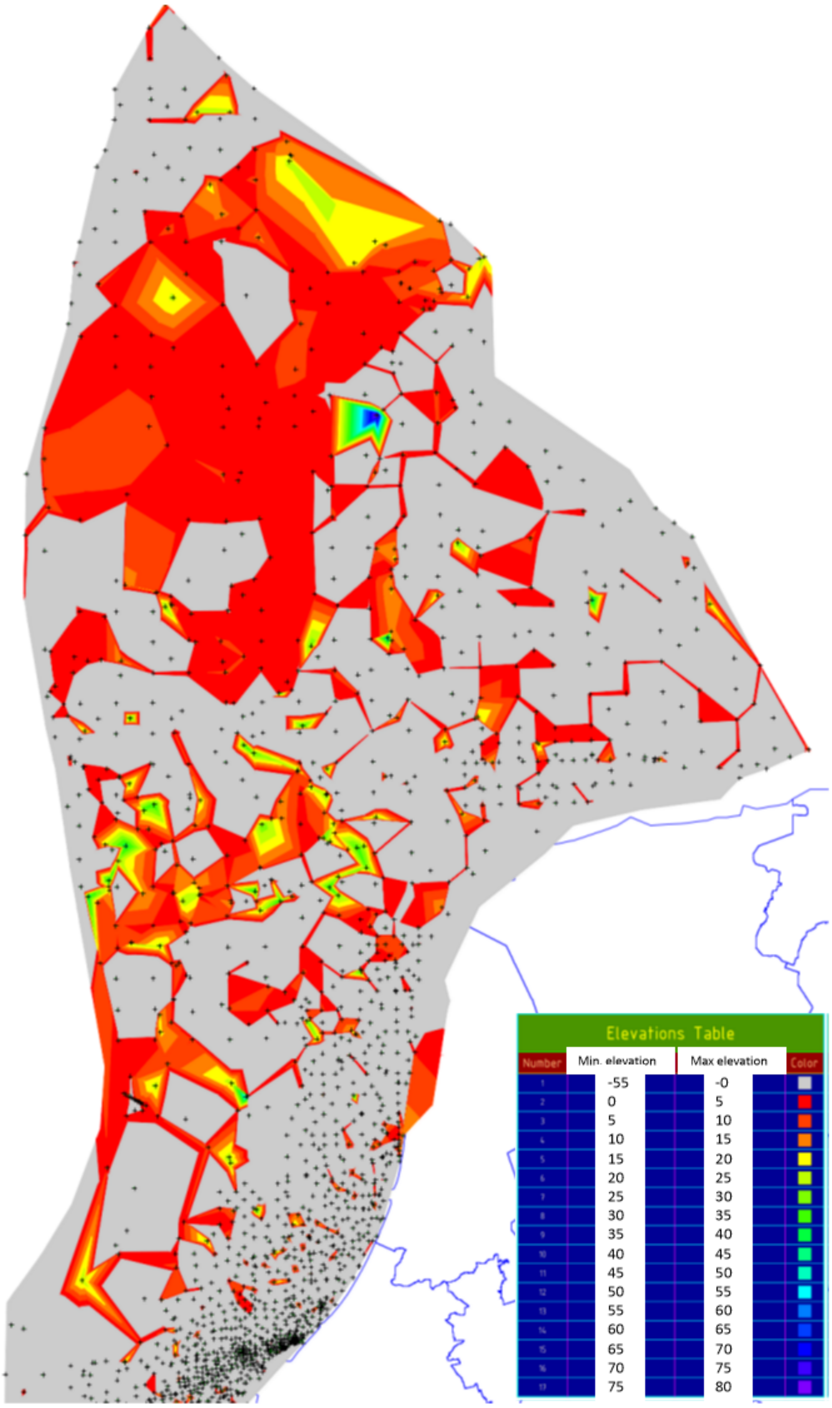


Figure A.2: Maximum possible head difference considering the soil characteristics. After van der Wel (2021).

A.2. Maritime shipping routes

Maritime routes are essential areas to ensure safe passage for vessels, avoiding physical constrains such as coasts, reefs and man-made structures like wind farms. Without taking the maritime routes into consideration, the construction of an offshore structure that will narrow the existing traffic routes, will have a large impact on the world wide transport industry. This has mainly to do with the fact that the North Sea is one of the most dense shipping areas in the world, connecting the largest ports of Europe: Antwerp, Rotterdam and Hamburg.

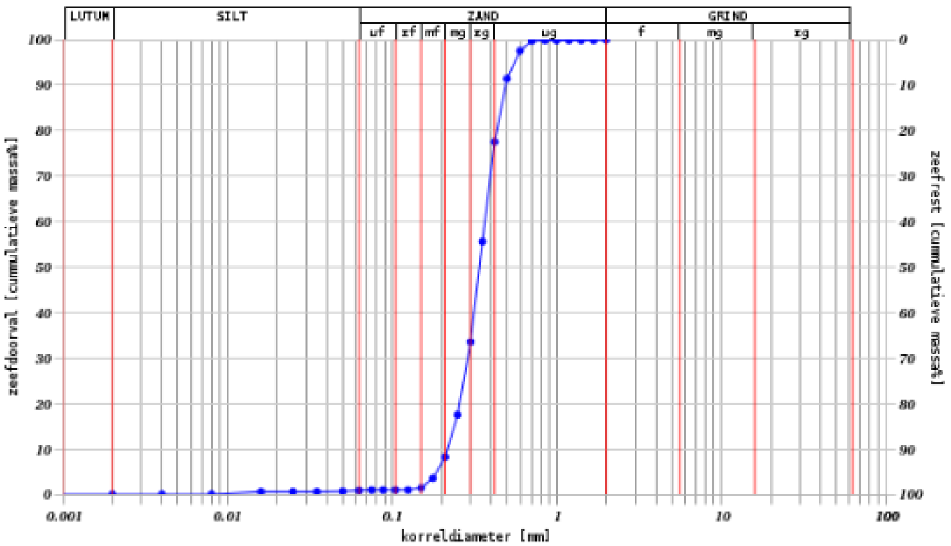


Figure A.3: Grain size distribution for a nearby observation point, BL100088

To that end, first the existing maritime shipping routes will be analysed to see whether they interfere with the proposed potential locations for the offshore PSH. The data that will be used is retrieved from Rijkswaterstaat, Viewer Waterinfo Extra, which is a GIS based application where multiple layers can be visualised such as wind farms, nature conservation areas and thus maritime shipping routes. This data will be imported and visualised with QGIS.

The maritime routes are visualised in Figure A.4. In addition to the previous available map, the shipping routes are shown. In this figure several shipping lanes can be distinguished: entrance to the port of Rotterdam, the entrance to the port of IJmuiden/Amsterdam and three routes that run parallel to the coast towards Germany and Norway.

More importantly, the proposed project Site 01 interfere with the existing shipping routes. Being entirely occupied by the maritime routes, it can be concluded that it is not feasible to construct a PSH near Site 01, and therefore drops out as candidate for construction of the PSH.

On the other hand, the potential site mentioned by van der Wel (2021) located near Texel shows no interference with the maritime shipping routes nor with (planned) wind farms, and therefore is still a good candidate for the PSH, also indicated in Figure A.4.

A.3. Power cables and pipelines

In addition to the Shipping Routes, that cover a lot of space in the North Sea, the European race to become climate neutral also entails the construction of multiple wind farms and related infrastructure. Besides power cables running from wind farms to coast, or directly from the Netherlands to United Kingdom or Norway, the sea bed also lies full with several hundred pipelines, telecom cables and operation cables.

Considering the safety, the construction of a structure on top of a pipeline or cable will increase the failure probability of the structure and subsequently would evoke costly mitigation measures to compensate this. Ideally, sea bed infrastructure - such as power cables and pipelines - should therefore be avoided when looking for the desired location.

Similar to the Maritime Shipping Routes, the sea bed infrastructure has also been retrieved from Viewer Waterinfo Extra and analysed and visualised with QGIS. The corresponding map is shown in Figure A.5, which shows the infrastructure in addition to the maritime shipping routes. It is remarkable to what extent the North Sea is already occupied, resulting in only a few potential locations that are viable for an offshore PSH.

The potential location that was recommended by van der Wel (2021) is presented in detailed map of

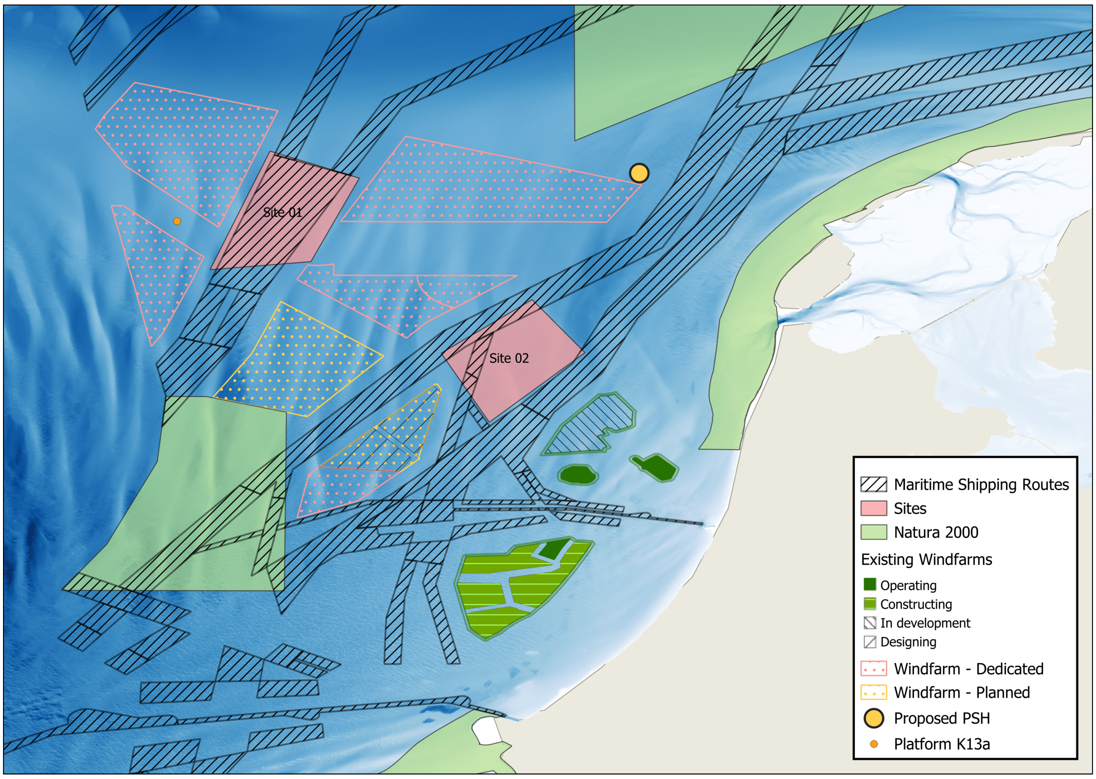


Figure A.4: Overview of the North Sea, the maritime shipping routes and existing, planned and permitted wind farms. Coordinates of the proposed location for the PSH are 53.3356°N, 4.3771°E.

the North Sea, just offshore from the Waddeneilanden. It turns out that the location coincide with two decommissioned pipelines and one small operating pipeline.

Despite the latter fact, the location is still highly favorable with respect to the alternatives due to the great potentials. In a later design phase, additional measure can be suggested to mitigate the effects of the existing pipelines, but this is considered to be outside the scope of my research. Moreover, if an offshore PSH will be built, the construction of such a structure will not commence within 10 years, enough time for the oil production to decrease and also to decommission more pipelines in the North Sea.

A.4. Bathymetry

Typical sand dunes in the North-Sea can reach an absolute height difference of 40 m, depending on the exact location. Using the EMODnet Map Viewer of the European Marine Observation and Data Network (2023), the exact bathymetry can be analysed, which will lead to potential areas for the construction of the PSH. For the desired location nearby Texel, the sea bed is, in absence of any major sand dunes, relatively flat. This can also been seen in Figure A.6, where the bathymetry of the structure's perimeter is plotted with respect to mean sea level, starting at the northern part of the structure. The lowest elevation in the perimeter is MSL -29 m, located halfway at the structure.

A.5. Lithography

The desired area consists of multiple boreholes that are all deeper than 10 m (four are deeper than 50 m) and their locations are shown in Figure A.7. The corresponding lithographies are presented in Figure A.8, and Figure A.3 shows the grain size distribution for a nearby sample.

A.6. Wave characteristics

Relative water depth

The relative depth of the component determines what working principles should be taken into account and which can be neglected. As such, the type of relative water depth must be derived for the structure, which can be shallow water, transitional water depth and deep water. This depends on the water depth

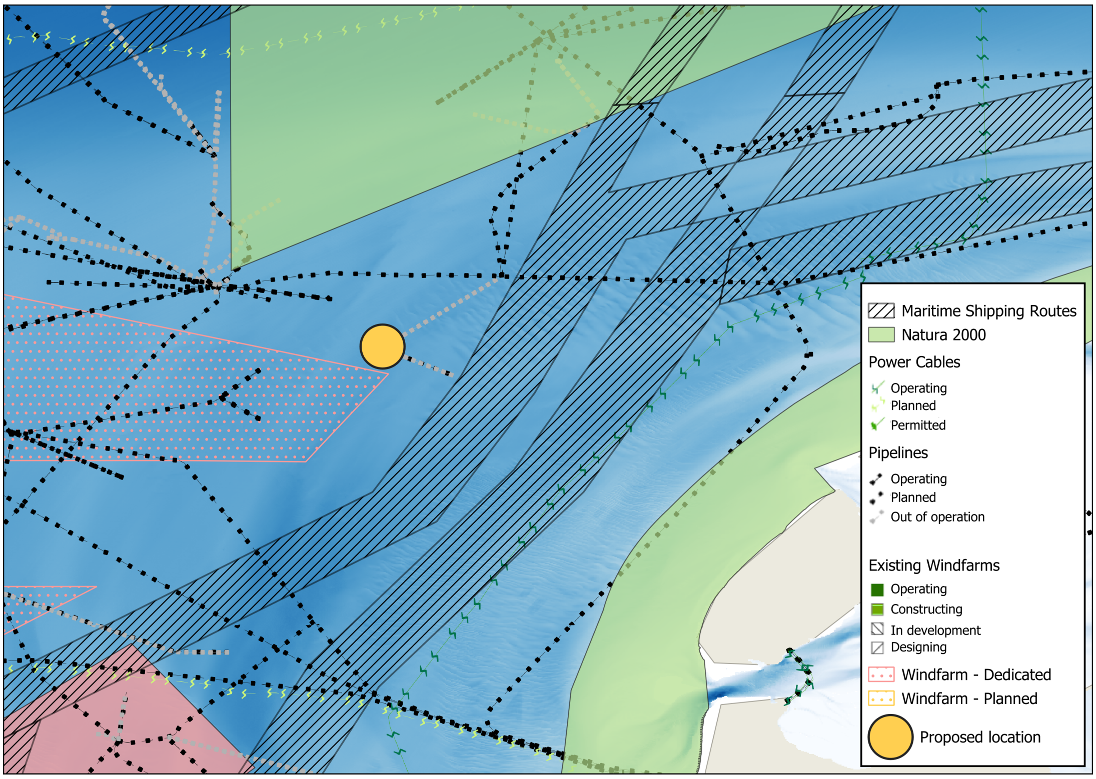


Figure A.5: Detailed map of the potential project site in the North Sea, located at approximately 40 km from the Dutch coast, consisting of the existing and planned wind farms, maritime shipping routes, existing, planned and permitted power cables and pipelines. Coordinates of the proposed location are 53.3356°N, 4.3771°E.

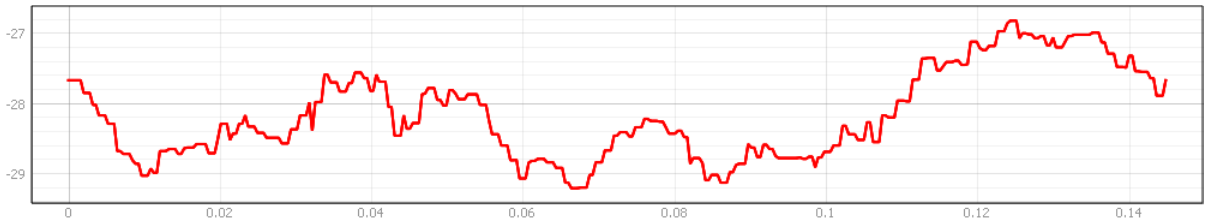


Figure A.6: Bathymetry near the proposed location (53.3356°N, 4.3771°E), with respect to mean sea level, starting at the Northern end of the structure. Horizontal unit is given in 100 km, so 0.1 equals 10 km.

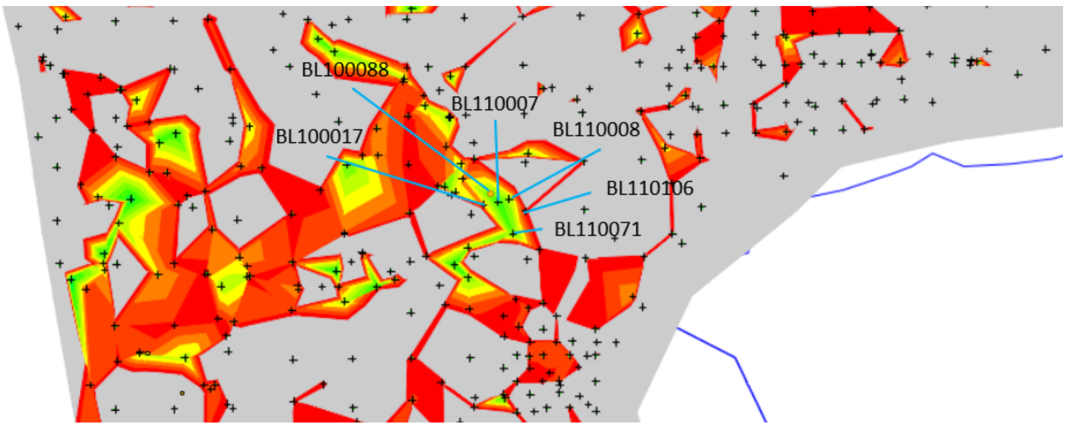


Figure A.7: Area of interest, after the the results of van der Wel (2021), indicated which boreholes make up the potential site.

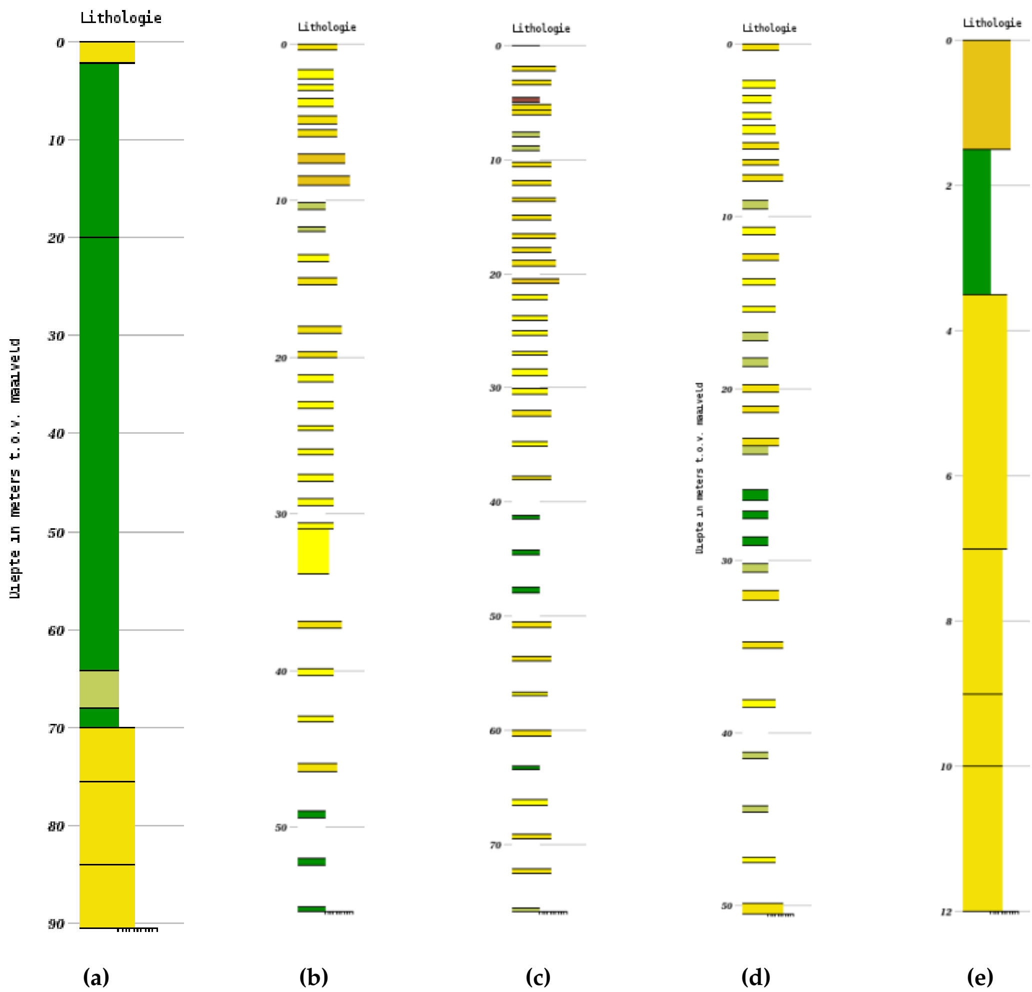


Figure A.8: Lithography for the following boreholes: (a) BL110071, (b) BL100017, (c) BL110007 and (d) BL110008. Colour legend: orange is coarse sand, yellow indicates sand, light green is peat and green is clay.

and wave length ratio. Since the offshore water depth near the desired location is approximately 30 m, the educated guess is that deep water characteristics will be present. According to Schiereck (2019), deep water characteristics will be governing when the water depth and wave length ratio is larger than 1/2, which is given by:

$$\frac{h}{L} > \frac{1}{2} \tag{A.5}$$

In which h is the water depth [m] and L the deep water wave length L_0 [m]. For the deep water regime, the corresponding wave length is equal to:

$$L = L_0 = \frac{gT^2}{2\pi} \tag{A.6}$$

in which L is the wave length [m], g the gravitational acceleration [m/s^2] and T the wave period [s]. Typical wave periods for offshore wave conditions are in the range of 5 - 20 s, resulting in a deep water wave length of 40 - 650 m according to Equation A.6. Hence, the ratio of water depth to wave length ranges between 0.20 - 0.05 and therefore it can be assumed that the wave characteristics can be considered as deep water waves.

Wave period

According to DHI (2017), the wave period for offshore waves is related to the significant wave height, and will be used for the preliminary stability assessment for the caisson unit. The corresponding expression is given by Equation A.7.

$$T_p \approx 5.3\sqrt{H_{m0}} \tag{A.7}$$

According to EurOtop (2018) the spectral wave period $T_{m-1,0}$ can be estimated with the peak period using Equation A.8

$$T_p = 1.1T_{m-1,0} \tag{A.8}$$

Wave data - Extreme Value Analysis (EVA)

In order to make a sophisticated design, one of the most important parameters, the wave height given for a certain return period, is required. The wave height determines nearly every failure mechanism, for instance overtopping, shear-criterion or rational stability. Therefore, an Extreme Value Analysis must be executed to find the corresponding wave heights per directional spread.

Important for an EVA is that the length of the times series of wave data must be sufficient to ensure the time series is stationary. Moreover, the training length of the data set must also be suitable in order to obtain reliable return values for longer return periods: for a return period of 1000 years the required training length must be in order of 25 years.

The Metocean Data Portal of (DHI, n.d.) contains over 40 years of hourly weather and hydraulic observations, such as wind speed and direction and wave height and period. Through hydrodynamic modelling, real observations from several locations around the North Sea are used for their models, in order to obtain reliable results reaching also in areas where no observations are known. The advantage of the application of the model, is the fact that spatial variations between the an observation point and the point of interest are all included and taken into account. For example, the model uses real data obtained from an observation point, for instance the wave height and wave direction, and uses this as input for the data combined with the real bathymetry. The model will then generate for every point the wave height and direction.

The desired location for the PSH plant is North West of Texel (shown in Figure A.4), with the following coordinates: 53.3356^{o}N, 4.3771^{o}E. The wave data will be extracted corresponding to these coordinates. Subsequently, the data is analysed with Python, for which a Peak-Over-Threshold (POT) has been applied to find the extreme values for the wave height. Afterwards, a Generalized Pareto Distribution (GPD) has been fitted over the extreme values to find the return period and their values. The results are shown in Table A.1. Since it is assumed that the dependency of wave height on the wave direction is significant - as Figure A.10 indicates - the EVA will be executed for every wave direction, divided per 45^o.

Table A.1: Results from Extreme Value Analysis (EVA) using Generalized Pareto Distribution (GPD). Mean values and 95% confidence interval (upper and lower bound are shown in the brackets) for the significant wave height [m] for multiple return periods are shown, distinguished on wave direction per 45^o.

Return period [years]	Wave direction								
	Omni	0^o - 45^o	45^o - 90^o	90^o - 135^o	135^o -180^o	180^o - 225^o	225^o- 270^o	270^o - 315^o	315^o - 360^o
1	6.01 (5.86,6.16)	3.73 (3.68,3.79)	2.86 (2.83,2.94)	2.0 (2.0,2.0)	2.2 (2.16,2.26)	4.9 (4.82,5.0)	5.11 (4.98,5.26)	5.44 (5.3,5.6)	5.57 (5.38,5.76)
2	6.47 (6.28,6.65)	4.18 (4.03,4.34)	3.27 (3.15,3.47)	2.15 (2.11,2.2)	2.43 (2.35,2.53)	5.18 (5.07,5.3)	5.55 (5.37,5.76)	5.87 (5.67,6.06)	6.15 (5.9,6.39)
5	6.99 (6.73,7.23)	4.76 (4.48,5.07)	3.71 (3.51,3.92)	2.36 (2.26,2.47)	2.66 (2.54,2.75)	5.49 (5.33,5.63)	6.14 (5.88,6.42)	6.36 (6.09,6.59)	6.79 (6.44,7.07)
10	7.33 (7.01,7.64)	5.2 (4.82,5.61)	3.98 (3.69,4.21)	2.51 (2.38,2.67)	2.78 (2.64,2.88)	5.69 (5.47,5.85)	6.58 (6.27,6.92)	6.69 (6.33,6.98)	7.19 (6.75,7.51)
25	7.72 (7.3,8.12)	5.79 (5.28,6.34)	4.26 (3.81,4.53)	2.72 (2.53,2.94)	2.91 (2.74,3.0)	5.91 (5.61,6.12)	7.16 (6.78,7.58)	7.05 (6.54,7.45)	7.63 (7.04,8.03)
50	7.97 (7.49,8.45)	6.23 (5.62,6.89)	4.44 (3.84,4.75)	2.87 (2.64,3.14)	2.98 (2.79,3.08)	6.04 (5.69,6.33)	7.61 (7.17,8.09)	7.29 (6.65,7.8)	7.91 (7.2,8.45)
100	8.19 (7.63,8.75)	6.67 (5.96,7.44)	4.59 (3.87,4.97)	3.02 (2.75,3.34)	3.03 (2.82,3.16)	6.16 (5.74,6.51)	8.05 (7.55,8.59)	7.51 (6.74,8.14)	8.15 (7.29,8.85)
250	8.44 (7.79,9.09)	7.26 (6.42,8.16)	4.74 (3.88,5.23)	3.23 (2.9,3.61)	3.09 (2.85,3.24)	6.29 (5.8,6.74)	8.63 (8.07,9.25)	7.75 (6.81,8.56)	8.41 (7.39,9.34)
500	8.61 (7.89,9.33)	7.7 (6.76,8.71)	4.84 (3.89,5.42)	3.38 (3.01,3.81)	3.12 (2.85,3.3)	6.37 (5.82,6.91)	9.07 (8.46,9.75)	7.91 (6.85,8.89)	8.58 (7.44,9.69)
1000	8.75 (7.98,9.53)	8.14 (7.1,9.26)	4.92 (3.89,5.58)	3.54 (3.13,4.01)	3.14 (2.86,3.34)	6.45 (5.84,7.09)	9.51 (8.84,10.25)	8.05 (6.87,9.25)	8.72 (7.47,9.97)

In addition to the EVA of the wave height, the wave direction has also been analysed and visualised. In Figure A.9 the windrose that corresponds to the desired location is shown. From these results, two conclusions can be drawn. First, the dominant wave directions are North North West to North and South West. To determine from which direction results in the highest waves and hence are governing, the distribution of wave direction as a function of the wave height must be evaluated. The result is presented in Figure A.10, and shows that the highest waves originates from the region between North and South West direction, whereas waves from the North East to South can be neglected due to the sheltered area created by the Dutch coast.

Figure A.9: Omnidirectional windrose for the desired location, consisting of 42 years of data.

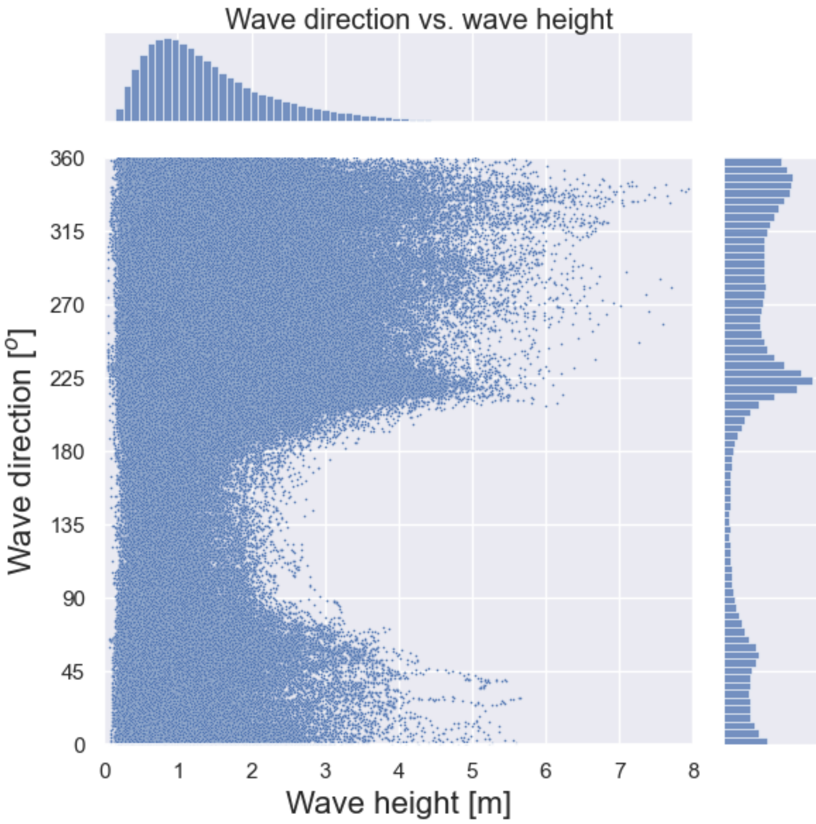


Figure A.10: Wave direction as a function of the wave height at the desired location.

A.7. Seasonal variability

Figure A.11: Seasonal variability in wave conditions for the desired location, shown in three types of monthly graphs: a windrose, a probability density function for the wave height and a wave direction versus wave height distribution.

A.8. Wind analysis

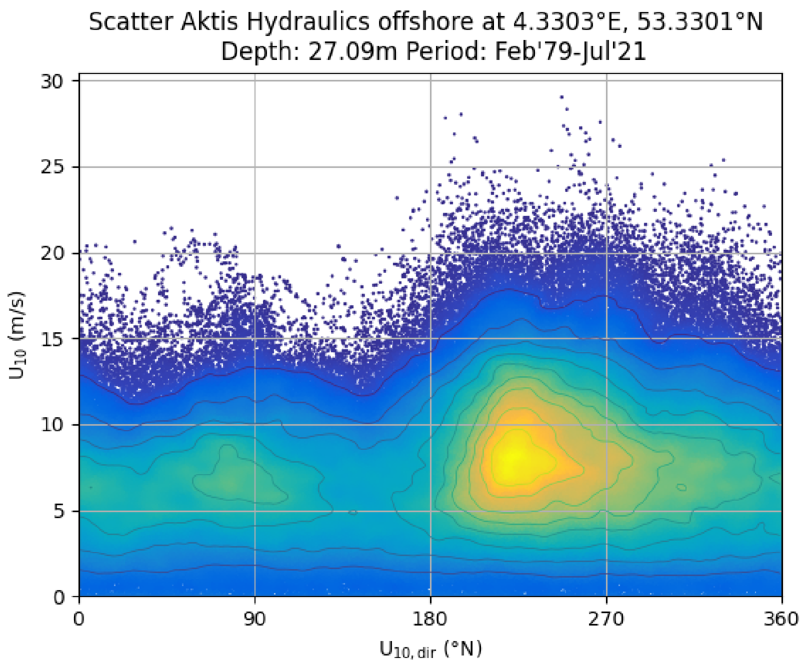


Figure A.12: Wind speed (u_{10}) distribution scatter for the proposed location (53.3301 °N, 4.3303 °E). Source: Aktis Hydraulics

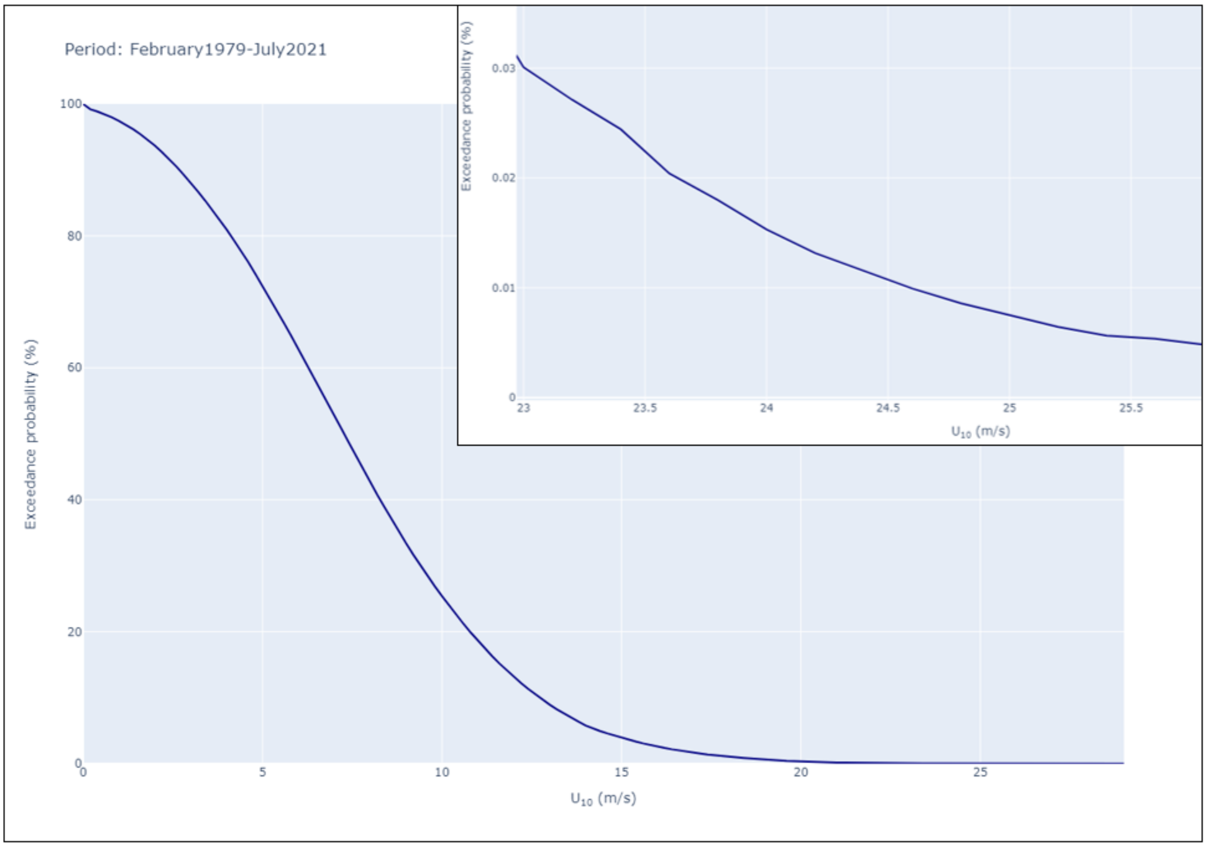


Figure A.13: Probability of exceedance curve for the wind speed u_{10} at the proposed location. At the top right a close up of the exceedance probability of 0.01 is shown, which will be used in the design.

A.9. Design water level

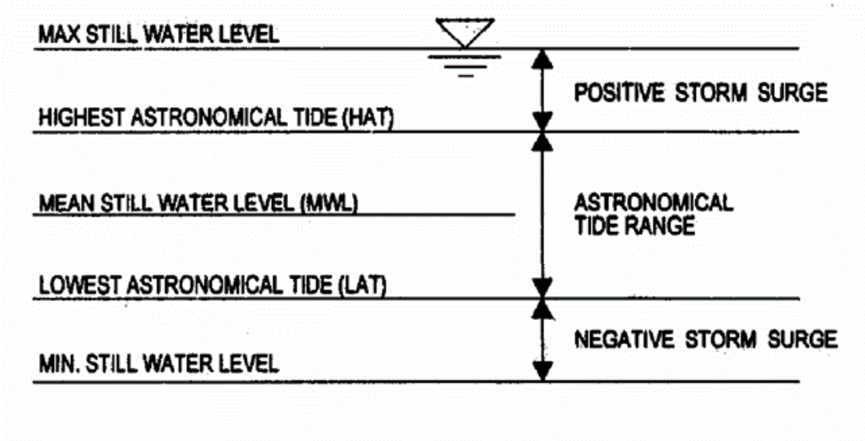


Figure A.14: Definition of the water levels: design water level, Highest Astronomical Tide (HAT), High Mean Water (HMW), Mean Sea Level (MSL), Low Mean Water (LMW) and Lowest Astronomical Tide (LAT).

A.10. Design return period

During the first phase of the project the structure's elements will be designed based on the governing design conditions. The robustness of those elements, in turn, depend on the consequences and its severity when the structure would fail, which is often expressed in a certain return period.

The corresponding return event can be derived by applying the Poisson equation in combination with the known parameters such as failure probability and the lifetime. The Poisson distribution is given by:

$$P = 1 - \exp(-fT) \tag{A.9}$$

where P is the probability of failure, f denotes the frequency of occurrence and T is the lifespan of the structure. Rewriting the expression to find the correct return frequency, for which the structure must be designed, results in:

$$f = \ln(1-P)/T \tag{A.10}$$

For this project two distinct scenarios are distinguished: i). the execution phase and ii). the operational phase. The former involves situations that includes temporary structures that are exposed or subjected to hydraulic loads for a limited amount of time, for example the situation where the caisson unit is ballasted entirely with sand. Here, the caisson unit is a temporary situation, since the unit will be protected and supported by the scour protection and inner berm. Structural elements that are finished, must be designed taking the operational conditions in to account.

Return period for the operational phase

The operational phase considers the situation when the facility is finished and operating. Since the structure will be a critical element in only the energy infrastructure, but no lives are at risk, the consequences will be relatively mild. Therefore the design requirements are similar to other offshore structures like breakwaters and closure dams. In line with the requirements for breakwaters mentioned by the ROM (2006), for the superstructure the following structural requirements are assumed: the lifetime of the structure is 100 years, with a failure probability of 20% in its entire lifetime.

Substituting the parameters into the latter equation, results in a return period of:

$$R = \frac{1}{f} = \frac{T}{\ln(1-P)} = \frac{100}{\ln(1-0.20)} = 450\,\text{years} \approx 500\,\text{years} \tag{A.11}$$

Execution phase - Temporarily lifetime of 1 year
The execution phase considers only temporary structures or execution sequences, which means that the lifetime of some structures can not be longer than the working season. Therefore, it is assumed that for these sequences the lifetime is only 1 year, with the same failure probability of 20%, which results in a return period of:

$$R = \frac{1}{f} = \frac{T}{\ln(1-P)} = \frac{1}{\ln(1-0.20)} = 4.5\,\text{years} \approx 5\,\text{years} \tag{A.12}$$

Execution phase - Temporarily lifetime of 5 years
On the other hand, there are also temporary structures with a lifetime more than one year, for instance the caisson unit with a partly heightened inner berm (but not entirely finished yet) that must be stable during extreme weather conditions. For these structures the lifetime is estimated on 5 years, which results in a design return period of:

$$R = \frac{1}{f} = \frac{T}{\ln(1-P)} = \frac{5}{\ln(1-0.20)} = 22\,\text{years} \approx 25\,\text{years} \tag{A.13}$$

B
Model parameters

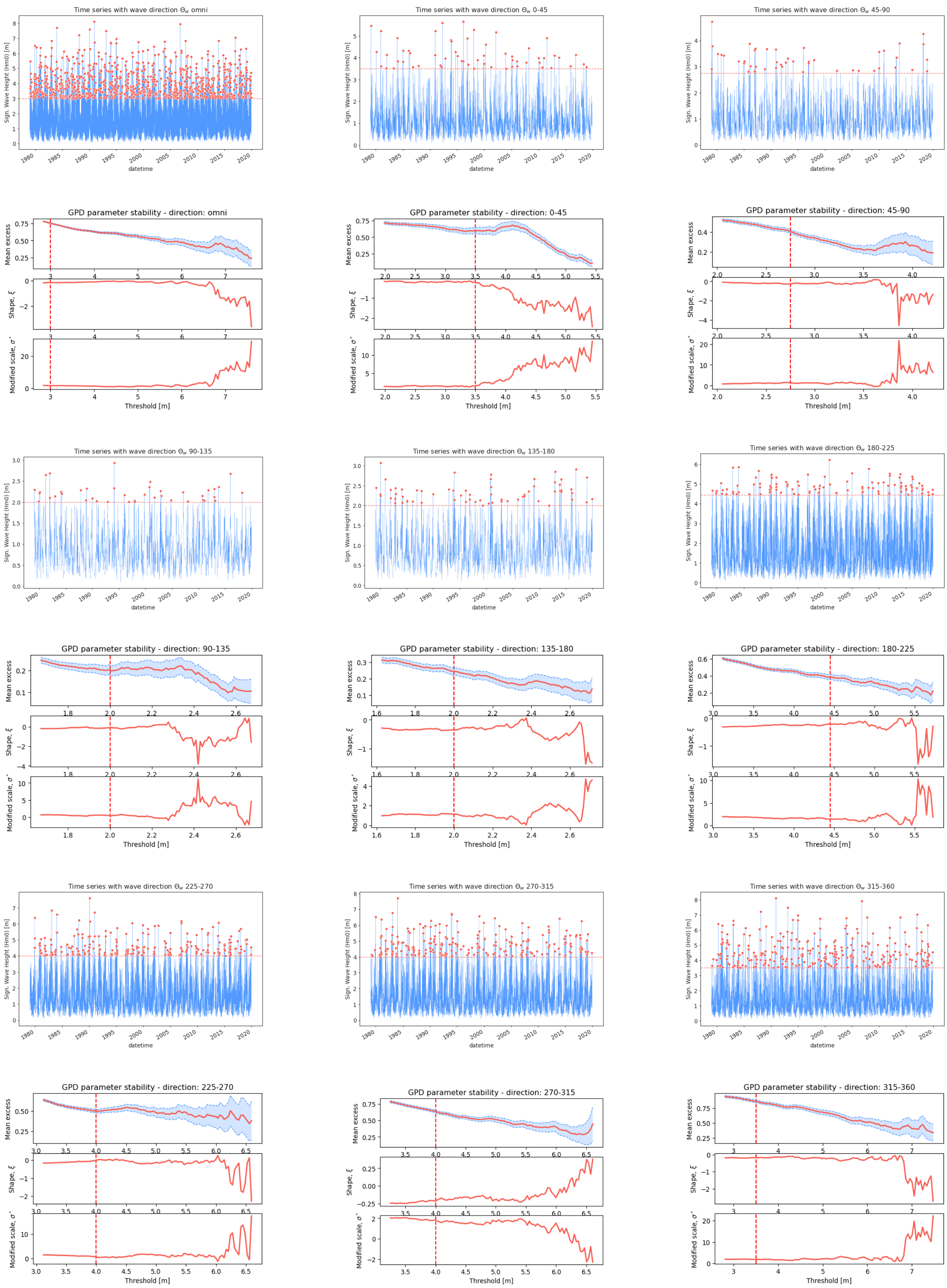


Figure B.1: GPD fit parameters (scale, shape and mean residual life) for all direction.

C Structural assessment - Calculations

According to the Spanish caisson design manual (ROM, 2006), the partial safety factors are less conservative than mentioned in the caisson design manual of Voorendt and Molenaar (2021). Figure C.1 lists the relevant safety factors.

Persistent or temporary situation	Comb	P_0,P_a	P_l,P_{lz}	E_a	P_r ,P'_r	E_r	P_t	E_T	E_{co},$P_{co,s}$	E_{so},$P_{so,s}$	P_{sc}
Floatation stage ▲+	ULS1	1.00/ 1.35(*)	1.35	1.35							
Installation stage ▲	ULS2	1.00/ 1.35(*)	1.35	1.35							
Pressure on a cell rest empty +	ULS3					1.50					
QUAYS (sheltered watrers)											
Service ▲◆	ULS4	1.00/ 1.35(*)	1.00/ 1.35(*)	1.35	1.00/ 1.35(*)	1.50	1.00/ 1.35(*)	1.00/ 1.50(*)			1.50
BREAKWATERS											
Ext. Pressure Wave Trough ▲◆	ULS5	1,00/ 1,35(*)	1,00/ 1,35(*)	1,35	1,00/ 1,35(*)	1,50	1,00/ 1,35(*)	1,00/ 1,50(*)		1,50	ψ_0 x 1,50(**)
Waves ▲◆	ULS6	1,00/ 1,35(*)	1,00/ 1,35(*)	1,00	1,00/ 1,35(*)	1,00	1,00/ 1,35(*)	1,00/ 1,50(*)	1,50		ψ_0 x 1,50(**)

P_o Self weight of the caisson (bottom slab and footing P_{oc} and walls P_{0p}). P_0=P_{0c}+P_{op}.
P_a Lift of the water (Archimedes' force).
P_l Weight of the water contained in the caisson cells.
P_{lz} Weight of the water that gravitates on the footing.
E_a Hydrostatic pressure.
Water pressure at the back of the quay or breakwater.
P'_r Weight of the fill that is transmitted through the caisson walls (Silo Effect).
P_r Weight of the earth fill that is transmitted through the surface of the cell (Silo Effect).
E_r Horizontal actions (Pressure) due to the Fill (Silo Effect).
E_T,P_t Earth pressure exerted at the back of quays or backfilled breakwaters. Weight of earth on footing.
E_{so} Wave trough (Design wave value).
$P_{so,s}$ Hydrodynamic uplift due to the passing of the wave trough.
E_{co} Wave crest (Design wave value).
$P_{co,s}$ Hydrodynamic uplift due to the passing of the wave crest.
P_{sc} Use overload.

Caption
▲ Potential critical state for outer walls.
+ Potential critical state for inner walls.
◆ Potential critical state for the foundations.
(*) For the purpose of making calculations for the bottom slab, both possibilities must be taken into account. In the case of actions due to the ground, the vertical and horizontal effects are increased or decreased simultaneously.
(**) It is assumed that the wave action predominates, this being the most habitual situation.

Figure C.1: Combinations of actions for persistent or temporary situations in ULS. (ROM, 2006)

Phase I - Installation of rock foundation

Prior to the installation of the caisson unit, a rock foundation layer must be installed on the sea bed to ensure proper stress transition from the caisson via the foundation layer to the subsoil, but also to increase the external friction between the caisson and the foundation. However, before the installation of the rock foundation the stability of the rock work must be determined. This could also have consequences regarding the planning, if it turns out that the applied grading will already be mobilized during tidal currents with a return period of one week, the installation must be well timed.

According to the preliminary design, the rock foundation layer consist of a light grading, 45 - 180 mm, with a mean diameter of $d_{n50} = 6.4\,\text{cm}$. According to the current data from DHI (n.d.), daily tidal currents range between 0.8-1.0 m s^{-1}. For the determination of the mobilization of the foundation, a maximum current that could occur during spring and summer, a maximum current of 1.3 m s^{-1} is considered, corresponding to a return period of 100 years. The governing water depth is the set-down scenario, where the water level will be lower than MSL. The corresponding set-down for a return period of 100 years is MSL -2.2 m.

$$\Delta d = 0.035 \frac{\Phi}{\Psi} \frac{K_t K_h}{K_s} \frac{u^2}{2g} \tag{C.1}$$

Furthermore, recalling the Pilarczyk formula (Equation C.1), the stability depends on the type of bed protection (Φ, Shield parameter (Ψ), turbulence factor (K_t), depth factor (K_h) and steepness factor (K_s). This stability check considers a two-layered rock foundation, and hence the stability parameter is Φ = 0.75, which will be dumped rock corresponding to a Shield parameter of Ψ = 0.035. Due to the slope of the rock foundation (1:2), the steepness factor is K_s = 0.4, and moreover it is expected that minor turbulence effects will occur, and is incorporated with a turbulence factor of K_t = 1.1. All parameters and results are shown in Table C.1. Considering the input parameters described, the required grain diameter to prevent mobilization is d_{n50} = 5.59 cm.

Parameter	Symbol	Value	
Stability parameter	Φ	0.75	
Shield parameter	Ψ	0.035	
Turbulence factor	k_t	1.0	
Depth factor	k_t	0.53	
Steepness factor	K_s	0.4	
Relative density	Δ	1.54	
Grain diameter	d	5.59	cm

Table C.1: Stability calculation for armour stone in running water, using Pilarczyk's approach.

The rock foundation consists of a 45 - 180 mm grading, with a mean diameter of 6.4 cm and thus will be stable under the design conditions. Although the layer will be stable, this does not implicitly mean that the entire foundation (total length of 15.7 km) can be installed immediately, prior to the remaining construction sequences. Considering migrating sand dunes and nearby dredging activities, foreign material like sand and silt can deposit on and within the rock layer, decreasing the functional/structural characteristics. To what extent the foundation layer can be installed prior to the installation of the caisson unit will be an educated guess. For now, it is assumed that the rock foundation layer will be installed in sections with a length of 10 caisson units, and will be installed when current foundation is smaller than 5 caisson lengths.

Phase II - Transportation of floating caisson

During the first phase of the project, the transportation of the caisson to the project site, the static and dynamic stability of the caisson must be sufficient to ensure a safe and damage-free journey. In Section 3.3.1 and 3.3.2 respectively, the methodology to determine the static and dynamic stability has been described. In this Section, first the static stability will be evaluated, after which the dynamic stability will be analysed.

Static stability According to the design dimensions of the caisson unit shown in Table 3.2, the static stability during flotation can be determined, following the steps described above.

$$V_{caisson} = l \cdot w \cdot h; \qquad e_{caisson} = \frac{1}{2} \cdot h$$

$$V_{gaps} = (l - 2 \cdot t_w) \cdot (w - 2 \cdot t_w) \cdot (h - t_b); \qquad e_{gaps} = t_b + \frac{1}{2} \cdot (h - t_b)$$

These expression can be substituted into Equation 3.4 to find the centre of gravity with respect to the bottom of the caisson:

$$\overline{KG} = \frac{V_{caisson} \cdot e_{caisson} + V_{gaps} \cdot e_{gaps}}{V_{caisson} + V_{gaps}} \tag{C.2}$$

Following the next steps as described in the previous section to determine the metacentric height of the caisson. The results are shown in Table C.2.

Table C.2: Determination of the metacentric height with respect to the centre of gravity, according to Figure 3.3.

	Parameter	Symbol	Formula	Value
Step 1	Centre of gravity	$\overline{KG}$	$\sum(V_i \cdot e_i)/\sum V_i$	12.29 m
Step 2	Caisson draft	d	$F_w/(w \cdot l \cdot \gamma_w)$	17.0 m
Step 3	Centre of buoyancy	$\overline{KB}$	$1/2 \cdot d$	8.5 m
Step 4	Moment of intertia	I_{yy}	$1/12 \cdot l \cdot w^3$	$1.9 \cdot 10^5$ m^4
Step 5	Displaced water volume	V_{dw}	$l \cdot w \cdot d$	$3.5 \cdot 10^4$ m^3
Step 6	Metacentre from B	$\overline{BM}$	I_{yy}/V_{dw}	5.67 m
Step 7	Metacentre from G	$h_m = \overline{GM}$	$\overline{KB} + \overline{BM} - \overline{KG}$	1.9 m

Since the metacentre with respect from centre of gravity is more than 0.5 m ($h_m > 0.5$ m), the centre of buoyancy is lower than the centre of gravity, and the centre of gravity is in turn lower than the metacentra, the caisson can be considered to be statically stable.

Dynamic stability The requirements for the caisson dimension to prevent undesirable movement of the unit, such as pitching or rolling, are described in Section 3.3.2. This requirement only applies on caissons that are being transported from the intermediate port (Port of Rotterdam) to the project site, at a certain moment in time. For now, it is assumed that the installation of the caisson will be the limited factor, due to the strict conditions. Hence, this creates some room in the window of opportunity for the transportation of the units. In other words, caisson will only be transported during calm to mild weather conditions; during storm conditions caissons will not be transported to site and shouldn't therefore meet the requirements that follow from the storm conditions.

This implies that the dimensions of the caisson should fulfill the requirements that follow from these calm to mild wave conditions. Since the dimensions of the caisson have already been chosen, the threshold until the point where transport is possible will be determined. From Equation A.6 it follows that with a caisson length of 60 m, the maximum wave length caisson can be safely transported is L_w = 42 m, which corresponds to a wave period of 5.2 s assuming deep-water regime.

Phase IVa

During Phase IVa of the construction sequence, the caisson will be positioned at the desired location, while ballasting the caisson with water in a controlled manner, which allows the caisson to submerge until it reaches the sea bed. Phase IVa considers the situation where the caisson is fully ballasted with only water, before the caisson will be filled with sand.

During this short period the caisson will be subjected to limited wave impact, as the weather conditions during installation are assumed to be calm and will not get worse shortly after. However, the caisson must be analysed regarding the shear-criterion and the rotational stability, according to the method described in Section 3.3.3 and 3.3.4 respectively, which will be elaborated in this Section.

Shear criterion - Caisson fully ballasted with water To ensure stability against sliding of the caisson shortly after installation, the horizontal forces acting on the caisson can not exceed the net friction force due to the vertical forces, i.e. the self weight of the structure including potential ballast weight. After installation, the caisson must be filled with a suitable ballast material, such as sand, however directly after the caisson will only be ballasted with water. To what extent provides the current dimensions and design characteristics of the caisson unit horizontal stability, while it's fully ballasted with water.

Since the installation of the caisson will be done during good weather conditions, it is assumed that the significant wave height shortly after placement will not be larger than 1.5 m, with a wave period of 6.5 s. To place this assumption into perspective, it is estimated that the installation of the caisson unit can only succeed when wave conditions do not exceed 1.0 m. The necessary hydraulic boundary conditions and soil parameters are shown in Table C.3. Following Sainflou's approach, the wave induced pressures and resulting forces are shown in Table C.4.

Table C.3: Hydraulic boundary conditions and soil properties for the caisson fully ballasted with water.

Wave and hydraulic characteristics			
Return period	R	<<1	years
Initial wave height	H_i	2.50	m
Increase in water level	h_0	0.19	m
Wave period	T_s	8.4	s
Wave length	L_0	109.64	m
Wave number	k	0.06	[-]
Water depth	d	29.0	m
Water depth @foundation	d'	26.5	m
Set up + Sea level rise	Δh	0.5	m
Sainflou - wave induced pressure	p_2	-	kPa
	p_1	27.07	kPa
	p_0	10.24	kPa
Soil characteristics			
Internal friction angle sand	φ	30	^{o}
Internal friction angle rock	φ	40	^{o}
Friction factor sand/concrete	$f = \tan(\varphi)$	0.502	-
Soil pressure factor	K	-	-
Berm height	h_b	-	m
Specific weight soil	γ_s	17.7	kN/m3

Table C.4: Wave induced pressures and forces, and sum of horizontal and vertical forces acting on the caisson. All forces are corrected with partial safety factors according to ROM (2006).

Parameter	Symbol	Formula	Value
Horizontal forces	H_1	Eq. 3.11	3278 kN
	H_2	Eq. 3.12	20 447 kN
	H_3	Eq. 3.13	24 875 kN
Total horizontal	$\sum H$		48 601 kN
Weight caisson	$G_{caisson}$		191 295 kN
Weight ballast	$W_{ballast}{}^b$		559 876 kN
Buoyant force	F_b		−553 843 kN
Wave induced force	V_1		−15 662 kN
Total vertical	$\sum V$		181 666 kN

Including the factors of safety discussed at the previous section, the total horizontal force is $\sum H =$ 23 583 kN and total vertical force is $\sum V = 181\,666$ kN. The required vertical force to retain the horizontal forces, and hence prevent the caisson from sliding horizontally, can be determined with Equation 3.6. It follows that:

$$F_{req} = \frac{\sum H}{f} = \frac{48\,601\,\text{kN}}{0.50} = 96\,772\,\text{kN}$$

Obviously, since the required vertical force ($F_{req} = 96\,772$ kN) is greater than the present vertical force ($\sum V$ = 181 666 kN), the unity check is:

$$\text{UC}_{shear} = \frac{\sum V}{F_{req}} = \frac{181\,666\,\text{kN}}{96\,772\,\text{kN}} = 1.88$$

Hence, the caisson unit, fully ballasted with water, will be stable until a wave height of 2.5 m.

Rotational stability The design conditions are similar to the previous stability check, and are summarized in Table C.3. The results for the turn-over criterion are shown in Table C.5. The sum of moments is $\sum M = -869\,189$ kNm (directed counter-clockwise) and the sum of vertical forces is $\sum V = 161\,596$ kN. Based on this, stability check according to Equation 3.22 can be executed:

$$e_R = \frac{\sum M}{\sum V} = \frac{-869\,189\,\text{kNm}}{161\,596\,\text{kN}} = 5.38 \text{ m} \leq \frac{1}{6}w = \frac{1}{6} \cdot 34 = 5.67 \text{ m}$$

Table C.5: Relevant forces and their distance that will generate a moment around point K. Factor of safety has been included in this table, according to Table C.1.

Parameter	Force [kN]	Distance Formula	Distance Value [m]	Moment [kNm]
H_1	3278	$d' + h_0 + H_{in}$	27.3	−89 608
H_2	20 447	$2/3 \cdot d'$	17.7	−361 234
H_3	24 875	$1/2 \cdot d'$	13.3	−329 594
V_1	−15 662	$1/6 \cdot w$	5.7	−88 752

Phase IVb - Caisson fully ballasted with sand

After ballasting the caisson with water, the caisson must be filled with sand as soon as possible, since the latter stability check for the scenario considering only water as ballast turns out to be stable only for waves less than 2.5 m. The boundary conditions are in general the same as for the previous situation, however since the caisson filled with sand will be unprotected for a longer period, the wave conditions will be slightly more severe than for the first situation. Hence, a wave height corresponding to a return period of 2 years will taken as the hydraulic boundary condition, which is the equivalent of a wave height of 6.4 m. The storm surge level with a return period of 2 years is +2.2 m in combination with an estimated sea level rise of 0.50 m in 2050, results in a design water level of MSL +2.7 m. All hydraulic boundary conditions and soil characteristics are shown in Table C.6.

Shear criterion Similar to the previous calculations, first the sum of forces is determined, both wave induced horizontal and vertical and vertical forces, after which the required downward force is required to ensure sliding of the caisson is prevent during storm conditions. The results are shown in Table C.7.

Table C.6: Hydraulic boundary conditions and soil properties for the caisson fully ballasted with sand.

Wave and hydraulic characteristics			
Return period	R	2	years
Initial wave height	H_i	6.4	m
Increase in water level	h_0	0.77	m
Wave period	T_s	13.5	s
Wave length	L_0	285.07	m
Wave number	k	0.02	[-]
Water depth	d	29.0	m
Water depth at foundation	d'	26.5	m
Set up + Sea level rise	Δh	2.7	m
Sainflou pressures	p_1	73.06	kPa
	p_0	53.62	kPa
Soil characteristics			
Internal friction angle rock	φ	40	degree
external friction angle	$\phi = 2/3\,\varphi$	27	degree
friction factor- sand/concrete	$f = \tan(\varphi)$	0.502	

Table C.7: Wave induced pressures and forces, and sum of horizontal and vertical forces acting on the caisson. All forces are corrected with partial safety factors according to ROM (2006).

Parameter	Symbol	Formula	Value
Horizontal forces	H_1	Eq. 3.11	20 145 kN
	H_2	Eq. 3.12	24 894 kN
	H_3	Eq. 3.13	126 531 kN
Total horizontal	$\sum H$		171 569 kN
Weight caisson	$G_{caisson}$		191 295 kN
Weight ballast	$W_{ballast}$ [b]		1 015 971 kN
Weight wave return wall	G_{wall}		40 000 kN
Buoyant force	F_b		−598 971 kN
Wave induced force	V_1		−73 665 kN
Total vertical	$\sum V$		574 629 kN

Applying Equation 3.6, the required vertical force can be determined:

$$F_{req} = \frac{\sum H}{f} = \frac{171\,569\,\text{kN}}{0.50} = 341\,622\,\text{kN}$$

Resulting in an unity check:

$$\text{UC} = \frac{\sum V}{F_{req}} = \frac{574\,629\,\text{kN}}{341\,622\,\text{kN}} = 1.68$$

Hence, the design is sufficient for wave conditions with a return period of 2 years.

Rotational stability The results for the sand ballasted phase are shown in Table C.8. The stability check is given by:

$$e_R = \frac{\sum M}{\sum V} = \frac{-2\,796\,027\,\text{kNm}}{563\,964\,\text{kN}} = 4.96\text{ m} \leq \frac{1}{6} \cdot w = \frac{1}{6} \cdot 34 = 5.67\text{ m}$$

Thereby, the caisson can be considered to be stable during the execution phase while storm conditions prevail with a return period of 2 years.

Table C.8: Relevant forces and their distance that will generate a moment around point K. Factor of safety has been included in this table, according to Table C.1.

Parameter	Force [kN]	Distance Formula	Distance Value [m]	Moment [kNm]
H_1	20 145	$d' + h_0 + H_{in}$	28.1	−574 124
H_2	24 894	$2/3 \cdot d'$	17.7	−439 786
H_3	126 531	$1/2 \cdot d'$	13.3	−1 676 530
V_1	−73 665	$1/6 \cdot w$	6.3	−417 435

Vertical bearing capacity Macro slope failure may occur when the resisting bearing capacity of the soil is exceeded by the acting soil stress of the caisson, its ballast and potential additional loads, either constant or variable, such as working equipment. To prevent geo-technical slope failures, the resisting soil stresses can not be exceeded. The determination of the soil capacity is shown in Table C.9.

Table C.9: Maximum bearing capacity of sand according to Prandtl & Brinch Hansen.

Parameter	Symbol	Value
Specific weight sand	$\gamma_{s,sat}$	17.7 kN/m^2
Specific weight water	γ_w	10.1 kN/m^2
Internal friction sand	ϕ	30^o
Surcharge factor	N_q	18.4
Subsoil factor	N_γ	30.1
Shape factor	s_γ	0.83
Horizontal load factor	i_γ	0.34
Bearing capacity	p'_{max}	1218.9 kPa

The acting soil stresses due to the weight of the structure and the horizontal forces, can be determined as follows:

$$\sigma_{k,max} = \frac{\sum V}{w \cdot l} + \frac{\sum M}{\frac{1}{6} l w^2} = \frac{564\,003\,\text{kN}}{34 \cdot 60} + \frac{2\,796\,027\,\text{kNm}}{\frac{1}{6} \cdot 60 \cdot 34^2} = 518\,\text{kPa}$$

$$\sigma'_{k,max} = \frac{w}{w'} \cdot \sigma_{h,max} + \gamma'_{sand} \cdot h_{sill} = \frac{34}{42} \cdot 518 + 16 \cdot 2.5 = 503\,\text{kPa}$$

As the acting soil stress does not exceed the maximum bearing capacity, there will be no macro stability issues for the caisson during storm conditions, and therefore the current design is sufficient.

Phase VI - Construction inner berm

Shear criterion It turns out that the caisson is insufficiently stable when considering wave conditions with a return period of 25 years. In order to prevent damage to the structure, an inner berm must be constructed before the end of the work season, which is roughly the beginning of the storm season. So, the last construction sequence is the nourishment of the inner berm of the caisson dam, that must provide enough horizontal resistance to prevent sliding of the caisson during the winter season.

However, despite the fact that the construction of the inner berm is inevitable, it is not said that the inner berm must be entirely heightened to the design level, but only must be sufficient to provide enough stability for wave conditions with a return period of 25 years. As such, in this Section the minimal required berm height is calculated to must be present at the end of the work season. When this minimum is not met, the probability of failure increases significantly, potentially resulting in failure of the caisson dam.

In Figure C.2 a schematic overview of the horizontal acting soil stresses, hydraulic pressures and the wave induced pressures are shown that are considered for the passive soil failure of the inner berm. With this, the required berm height can be determined considering a balance of forces.

Table C.10: Hydraulic boundary conditions and soil properties for the caisson fully ballasted with sand to determine the required berm height to resist a wave conditions with a return period of 25 years.

Wave and hydraulic characteristics			
Return period	R	25	years
Initial wave height	H_i	8.12	m
Increase in water level	h_0	1.13	m
Wave period	T_s	15.1	s
Wave length	L_0	356.12	m
Wave number	k	0.02	[-]
Water depth	d	29.0	m
Water depth @foundation	d'	26.5	m
Set up + Sea level rise	Δh	3.1	m
Sainflou - wave induced pressure	p_2	70.30	kPa
	p_1	93.00	kPa
	p_0	71.53	kPa
Soil characteristics			
Internal friction angle sand	φ	30	^{o}
Internal friction angle rock	φ	40	^{o}
Friction factor sand/concrete	$f = \tan(\varphi)$	0.502	-
Soil pressure factor	K_p	3	-
Berm height	h_b	4	m
Specific weight soil	γ_s	17.7	kN/m3

Table C.11: Wave induced pressures and forces, and sum of horizontal and vertical forces acting on the caisson. All forces are corrected with partial safety factors according to ROM (2006)..

Parameter	Symbol	Formula	Value
Horizontal forces	H_1	Eq. 3.11	38 754 kN
	H_2	Eq. 3.12	28 510 kN
	H_3	Eq. 3.13	190 884 kN
	P_1	Eq.	236 620 kN
Total horizontal	$\sum H$		494 768 kN
Bearing capacity	S_1	Eq. 3.19	−152 714 kN
	S_2	Eq. 3.20	−54 298 kN
	S_3	Eq. 3.21	−25 428 kN
Total bearing capacity	$\sum S$		−232 440 kN
Weight caisson	$G_{caisson}$		191 295 kN
Weight ballast	$W_{ballast}$		1 015 971 kN
Weight wave return wall	G_{wall}		40 000 kN
Buoyant force	F_b		−607 176 kN
Wave induced force	V_1		−109 629 kN
Total vertical	$\sum V$		530 460 kN

To determine whether the proposed berm height is sufficient to resist wave conditions with a return period of 25 years, the bearing capacity of the inner berm in combination with the horizontal friction resistance of the self-weight of the structure may not be exceeded by the wave induced forces. In terms of an expression, it follows that:

$$\sum H < \sum S + \sum V \cdot f \tag{C.3}$$

According to the results presented in Table C.11, the terms can be substituted:

$$\begin{aligned} 494\,768\,\mathrm{kN} &< 232\,440\,\mathrm{kN} + 530\,460\,\mathrm{kN} \cdot 0.50 \\ 494\,768\,\mathrm{kN} &< 498\,847\,\mathrm{kN} \end{aligned}$$

Since the corrected bearing capacity is greater than the horizontal acting force, an inner berm height of h_b = 4 m is sufficient to withstand incoming waves of 8.12 m, equivalent to a return period of 25 years. However, considering the uncertainties with regard to erosion of the inner berm during the months prior to the design storm, it probably will occur that the height of the inner berm is not sufficient at the end of the winter. To include this uncertainty, the berm will be additionally heightened to ensure sufficient stability at the end of the winter. This results in a required total berm height of 10 m.

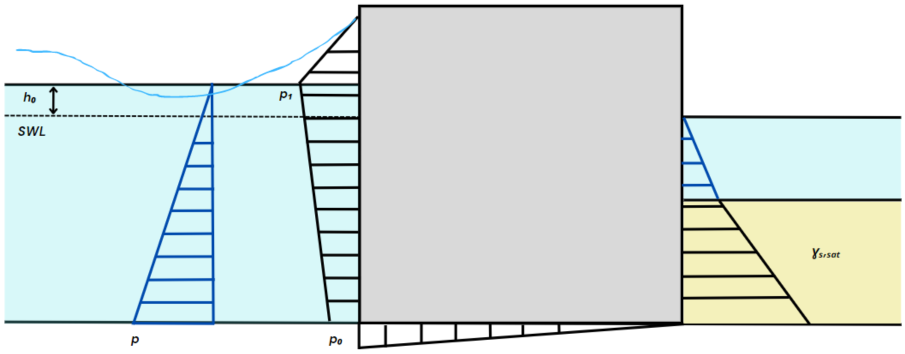


Figure C.2: Governing conditions and stresses for passive soil failure of the inner berm. Stresses and water level difference are indicated qualitatively.

Phase VIII - Facility in operation

Shear criterion When finished, an inner berm will provide stability for the caisson unit acting as passive soil pressure, for both the horizontal stability (shear criterion) and for the rotational stability. However, considering active soil pressure in combination with the negative wave force generated during the through of the wave, the horizontal stability must be verified.

First, the design conditions must be determined: which water levels are governing considering this scenario? It turns out that a decrease in water level (also known as set-down) in combination with the highest waves are the governing conditions, rather than a positive storm surge. Subsequently, the design water level consists of the estimated sea level rise (2.0 m) in 100 years and a set-down of -2.5 m (R = 500 years) results in a design water level of MSL -0.5 m. The event that is considered, regarding the wave height, has a return period of 500 years and has a wave height of 9.75 m.

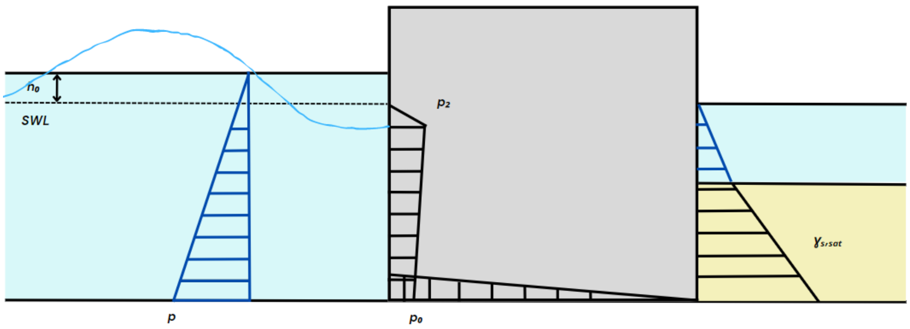


Figure C.3: Governing conditions and stresses for active soil failure of the inner berm. Stresses and water level difference are indicated qualitatively.

The requirement for the stability check is:

$$UC = \frac{\sum V}{F_{req}} = \frac{\sum V}{\sum H/f}$$

With the resulting forces given in Table C.13, the stability check is given by:

$$UC = \frac{817\,686\,\text{kN}}{313\,317\,\text{kN}/0.50} = \frac{817\,686\,\text{kN}}{623\,865\,\text{kN}} = 1.31$$

It turns out that the unity check for the shear criterion considering active soil stresses in combination with negative wave induced stresses is UC = 1.31, and therefore the design is sufficient.

Table C.12: Hydraulic boundary conditions and soil properties for the caisson dam while the facility is operating, considering the stability when active soil stresses act on the structure for the governing wave conditions.

Wave and hydraulic characteristics			
Return period	R	500	years
Initial wave height	H_i	9.75	m
Increase in water level	h_0	1.88	m
Wave period	T_s	16.5	s
Wave length	L_0	427.61	m
Wave number	k	0.01	[-]
Water depth	d	29.0	m
Water depth @foundation	d'	26.5	m
Set down + Sea level rise	Δh	-2.5	m
	p_2	79.10	kPa
Sainflou - wave induced pressure	p_1	116.98	kPa
	p_0	92.24	kPa
Soil characteristics			
Internal friction angle sand	φ	30	^{o}
Internal friction angle rock	φ	40	^{o}
Friction factor sand/concrete	$f = \tan(\varphi)$	0.502	-
Soil pressure factor	K_p	3	-
Berm height	h_b	3	m
Specific weight soil	γ_s	17.7	kN/m3

Table C.13: Wave induced pressures and forces, and sum of horizontal and vertical forces acting on the caisson. All forces are corrected with partial safety factors according to ROM (2006).

Parameter	Symbol	Formula	Value
Horizontal forces	H_1	Eq. 3.11	−28 002 kN
	H_2	Eq. 3.12	−8792 kN
	H_3	Eq. 3.13	−123 421 kN
Total horizontal	$\sum H$		−313 317 kN
Active soil force[b]	S_1	Eq. 3.19	−1803 kN
	S_2	Eq. 3.20	−27 296 kN
	S_3	Eq. 3.21	−124 003 kN
Total	$\sum S$		−153 102 kN
Vertical forces	$G_{caisson}$		191 295 kN
	$W_{ballast}$		1 015 971 kN
	F_b		−492 305 kN
	G_{wall}		40 000 kN
	V_1		62 725 kN
Total vertical	$\sum V$		817 686 kN

Vertical bearing capacity Similar to the previous situation, when in operation the inner berm will exert a horizontal stress on the caisson unit which might cause an exceedance of the vertical bearing capacity of the subsoil, especially considering extreme high water levels without taking into account the rising sea level. Therefore, the design water level consists of the maximum high storm surge for R = 500 years (mMSL +3.1 m) without the effect of sea level rise. The boundary conditions are listed in Table C.14, the loads are determined in Table C.15 and the maximum bearing capacity according to Prandtl & Brinch Hansen is given in Table C.16.

Table C.14: Hydraulic boundary conditions and soil properties for the caisson dam while the facility is operating, considering the vertical bearing capacity when active soil stresses act on the structure for the governing wave conditions.

Wave and hydraulic characteristics			
Return period	R	500	years
Initial wave height	H_i	9.75	m
Increase in water level	h_0	1.50	m
Wave period	T_s	16.5	s
Wave length	L_0	427.61	m
Wave number	k	0.01	[-]
Water depth	d	29.0	m
Water depth @foundation	d'	26.5	m
Set up	Δh	5.1	m
	p_2	82.91	kPa
Sainflou - wave induced pressure	p_1	113.17	kPa
	p_0	88.29	kPa
Soil characteristics			
Internal friction angle sand	ϕ	30	^{o}
Internal friction angle rock	ϕ	40	^{o}
Friction factor sand/concrete	$f = \tan(\phi)$	0.58	-
Soil pressure factor	K_a	0.33	-
Berm height	h_b	30	m
Specific weight soil	γ_s	17.7	kN/m3

Table C.15: Wave induced pressures and forces, and sum of horizontal and vertical forces acting on the caisson. All forces are corrected with partial safety factors according to ROM (2006).

Parameter	Symbol	Formula	Value
Horizontal forces	H_1	Eq. 3.11	−30 761 kN
	H_2	Eq. 3.12	−3691 kN
	H_3	Eq. 3.13	−121 135 kN
Total horizontal	$\sum H$		−308 689 kN
Active soil force	S_1	Eq. 3.19	−1803 kN
	S_2	Eq. 3.20	−27 296 kN
	S_3	Eq. 3.21	−124 003 kN
Total	$\sum S$		−153 102 kN
Vertical forces	$G_{caisson}$		191 295 kN
	$W_{ballast}$		1 015 971 kN
	F_b		−650 253 kN
	G_{wall}		40 000 kN
	V_1		60 035 kN
Total vertical	$\sum V$		657 048 kN

The acting soil stresses due to the weight of the structure and the horizontal forces, can be determined as follows:

Table C.16: Maximum bearing capacity of sand according to Prandtl & Brinch Hansen.

Parameter	Symbol	Value
Specific weight sand	$\gamma_{s,sat}$	17.7 kN/m^2
Specific weight water	γ_w	10.1 kN/m^2
Internal friction sand	ϕ	30^o
Surcharge factor	N_q	18.4
Subsoil factor	N_γ	30.1
Shape factor	s_γ	0.83
Horizontal load factor	i_γ	0.21
Bearing capacity	p'_{max}	751.7 kPa

$$\sigma_{k,max} = \frac{\sum V}{w \cdot l} + \frac{\sum M}{\frac{1}{6} l w^2} = \frac{657\,048\,\mathrm{kN}}{34 \cdot 60} + \frac{4\,518\,607\,\mathrm{kNm}}{\frac{1}{6} \cdot 60 \cdot 34^2} = 762\,\mathrm{kPa}$$

$$\sigma'_{k,max} = \frac{w}{w'} \cdot \sigma_{h,max} + \gamma'_{sand} \cdot h_{sill} = \frac{34}{38} \cdot 762 + 16 \cdot 2.5 = 721\,\mathrm{kPa}$$

Overtopping - ULS The Ultimate Limit State considers the situation or conditions that lead to excessive deformations, resulting or approaching the collapse of components or the structure as a whole. Severe overtopping can damage the structure due to erosion of the inner berm, which could in turn lead to stability issues of the caisson unit. For the overtopping criterion, the tolerable discharge is 125 - 150 l/m/s, assuming the inner berm is well protected and can withstand these discharges. With these overtopping discharges the reservoir will lose partly its efficiency as the reservoir will be filled, however this scenario will statistically take place once in the 500 years and therefore will not influence the efficiency of the PSH.

In this Section the required crest freeboard and the dimensions of the additional wave return wall will be determined, according to EurOtop (2018). The hydraulic boundary conditions that will prevail during the 500 year storm, are shown in Table C.17. The design water level consists of the estimated sea level rise of 2.0 m in 100 years (Table 2.2) and the storm surge level of 3.0 m (R = 500 years), resulting in a design water level of MSL + 5.0 m.

Non-impulsive wave impact can be expected, therefore Equation 3.29 will be applied to determine the overtopping discharge while neglecting the reducing characteristic of the bullnose. Afterwards, the influence of the wave return wall will be calculated according to the methodology shown in Figure 3.14, to eventually find the corrected overtopping discharge. The results are shown in Table C.18; it turns out that a crest freeboard of 10.0 m with a bullnose with dimensions 1.5 x 0.75 m (width x height) is necessary to ensure non-exceedance of the threshold of 125 l/s/m. Taking into account the over-height of 0.5 m of the caisson unit relative to the design water level, the wave return wall must be 9.5 m high.

Overtopping - SLS The Serviceability Limit State is defined as the situation at which the structure losses its serviceability for the actual service load that the structure is subjected to, i.e. the structure cannot remain functional during daily or weekly wave/weather conditions. Not only results extensive overtopping in a smaller efficiency of the PSH, the crest and the inner area must remain safely accessible for maintenance purposes during wave conditions with a return period of one week.

In this Section the overtopping discharge will be calculated, considering the crest freeboard and wave return wall dimensions that follow directly from the previous Section (ULS overtopping), to verify the expected overtopping discharges will not exceed the acceptable discharges. The tolerable overtopping discharge related to ensure safely access of driving vehicle to the inner area and the crest is estimated on 10 l/s/m, which is based on EurOtop (2018).

The design water level with a return period of less than 1 year - conditions that will occur more than once a year - consists of the estimated sea level rise in of 2.0 m 100 years (Table 2.2) and a storm surge of 2.0 m with a return period of 1 year, which results in a design water level of MSL +4.0 m. Due to non-impulsive wave conditions, Equation 3.29 will be used to determine the overtopping discharge with absence of the

Table C.17: Hydraulic boundary conditions for the ULS overtopping criterion.

Parameter	Symbol	Value	
Wave period	T_s	16.5	s
Wave height	H_{in}	9.75	m
Wave length	L_0	427	m
Wave number	k	0.015	-
Water depth	d	29	m
Sill height	h_{sill}	2.5	m
Sea level rise	Δd_{SLR}	2	m
Storm surge	Δd_{SS}	3.2	m
Corr. water depth	d'	31.5	m

Table C.18: Overtopping discharges considering the ULS situation, where a discharge of 125 l/s/m is tolerated.

Parameter	Symbol	Value	
Tolerated overtopping	q_{tol}	125	l/s/m
Crest freeboard	R_c	9.5	m
Overtopping discharge without bullnose	q	464.8	l/s/m
Bullnose width	h_r	0.75	m
Bullnose height	B_r	1.5	m
Reduction factor	k_{bn}	0.24	-
Overtopping discharge with bullnose	q	109.3	l/s/m

wave return wall. The effect of the bullnose is determined with the methodology described in Figure 3.14.

The wave height corresponding to the return period of 0.1 year is 4.10 m. The results are shown in Table C.20.

It turns out that the SLS criterion is not met, as the overtopping discharge exceeds the tolerable discharge of 10.0 l/s/m. However, this results from the wave height corresponding to 1 year. and hence a crest freeboard of 11 m is sufficient to ensure non-exceedance of the tolerable overtopping discharges.

Table C.19: Hydraulic boundary conditions for the SLS overtopping criterion.

Parameter	Symbol	Value	
Wave period	T_s	10.7	s
Wave height	H_{in}	4.1	m
Wave length	L_0	179.8	m
Wave number	k	0.035	-
Wave steepness	$s_{m-1,0}$	0.028	-
Water depth	d	29	m
Sill height	h_{sill}	2.5	m
Sea level rise	Δd_{SLR}	2	m
Storm surge	Δd_{SS}	2	m
Corr. water depth	d'	30.5	m

Table C.20: Overtopping discharges considering the SLS situation, where a discharge of 10 l/s/m is tolerated.

Parameter	Symbol	Formula	Value
Tolerated overtopping	q_{tol}	10	l/s/m
Crest freeboard	R_c	12.1	m
Overtopping discharge without bullnose	q	4.4	l/s/m
Bullnose width	h_r	0.75	m
Bullnose height	B_r	1.5	m
Reduction factor	k_{bn}	0.18	
Overtopping discharge with bullnose	q	0.8	l/s/m

Piping Piping refers to the process through which grains of the subsoil are removed due to a hydraulic force, imposed by the difference in hydraulic head. Initially, piping starts at the outflow side, the point where the water level is lower - where particles which cannot withstand the hydraulic gradient will be mobilised and hence transported. If this process is maintained for a sufficient period, a pipe can develop backwards where soil particles are removed, which has a positive feedback loop: by shortening the seepage length, the hydraulic gradient is increased and hence the outward flow becomes stronger, mobilising even more particles. Eventually, this backwards erosion process will cause severe stability issues, resulting in a overall failure of the caisson dam.

During maximum storage capacity, a maximum water level difference of 30 m will be imposed, resulting in substantial hydraulic gradients in the inner berm. The required seepage length to ensure sufficient safety with regard to piping can be determined with the method of Bligh and Lane: in case of Bligh the maximum tolerable hydraulic gradient is 8.3% for coarse sand. Assuming that the rock foundation layer is permeable, the flow path starts at the tip of the rock foundation layer and penetrates into the inner berm. Based on this, the stream can develop into multiple paths shown in Figure C.4 and will choose the path with the least resistance.

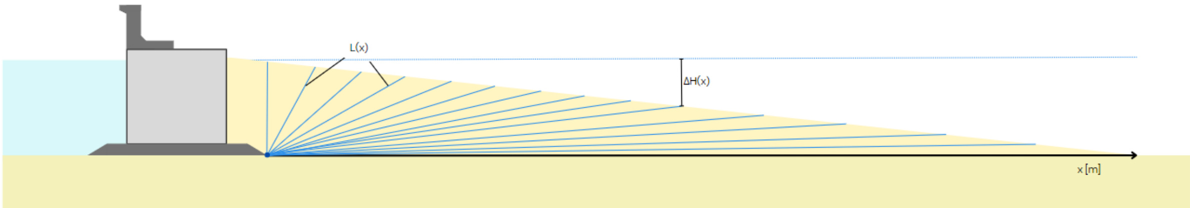


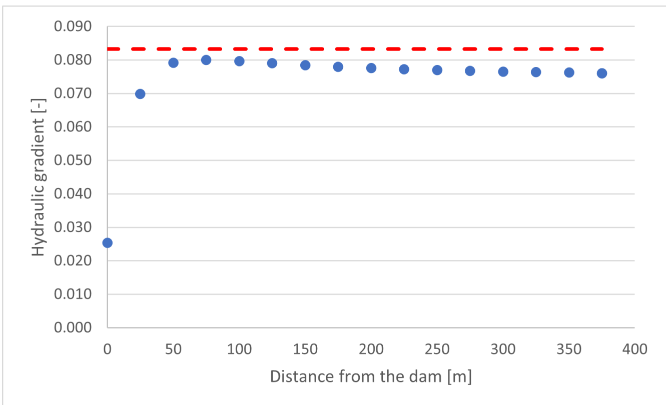
Figure C.4: Flow paths through the rock foundation layer into the inner berm.

Based on this approach, the initial design of the inner berm (with a slope of 1:10) was not sufficient, as the maximum hydraulic gradient exceeded the 8.3% and hence piping could be expected. In order to prevent this from happening, the slope of the inner berm should be decreased, subsequently decreasing the hydraulic gradient. it turns out that a slope of 1:13.5 results in hydraulic gradients all along the inner berm that do not exceed the maximum hydraulic gradients for coarse sand, according to Bligh. Table C.21 and Figure C.22 shows the results as a function of the distance from the caisson dam.

Table C.21: Determination of the hydraulic gradient as a function of the distance from the caisson dam. The values that are indicated in bold represent the maximum hydraulic gradient through the inner berm.

x [m]	$h_{berm}(x)$ [m]	$L(x)$ [m]	$\Delta H(x)$ [m]	$i(x)$ [-]
0	29.26	29.26	0.74	0.025
25	27.41	37.10	2.59	0.070
50	25.56	56.15	4.44	0.079
75	**23.70**	**78.66**	**6.30**	**0.080**
100	21.85	102.36	8.15	0.080
125	20.00	126.59	10.00	0.079
150	18.15	151.09	11.85	0.078
175	16.30	175.76	13.70	0.078
200	14.44	200.52	15.56	0.078
225	12.59	225.35	17.41	0.077
250	10.74	250.23	19.26	0.077
275	8.89	275.14	21.11	0.077
300	7.04	300.08	22.96	0.077
325	5.19	325.04	24.81	0.076
350	3.33	350.02	26.67	0.076
375	1.48	375.00	28.52	0.076
400	0.00	400	30.00	0.076

Table C.22: Hydraulic gradient as a function of the distance from the dam, where x = 0 represents the end of the rock foundation layer and x = 400 the end of the inner berm. The red dotted line represents the maximum hydraulic gradient that corresponds to coarse sand.



Seepage Seepage refers to the process during which water flows underneath the dam into the reservoir, imposed by a hydraulic head difference. Although this stream of water will not cause any damage to the structure nor subsoil, high discharge streams are undesired as this decreases the capacity and efficiency of the reservoir.

In order to determine the reduction of the capacity, the seepage discharge is calculated with Darcy's law, which is given by Equation C.4. For the maximum seepage discharge through the subsoil, the parameters corresponding to the highest hydraulic head will be used, since at that point the seepage will be maximum. The location of maximum hydraulic gradient and the corresponding value is indicated in Table C.21. The results and boundary conditions are shown in Table C.23.

$$Q = k \cdot A \cdot i = k \cdot A \cdot \frac{\Delta H}{L} \tag{C.4}$$

where:

Q [m^3/s] = seepage discharge
k [m/s] = permeability or hydraulic conductivity
A [m^2] = surface area of the stream
i [-] = hydraulic gradient
ΔH [m] = hydraulic head difference
L [m] = length of seepage path

Parameter	Symbol	Value	
Permeability	k	10^{-3}	m / s
Head difference	ΔH	6.3	m
Seepage length	L	78.7	m
Hydraulic gradient	i	0.080	-
Layer thickness	h	10	m
Discharge	q	8.0×10^{-4}	m^3/s/m
	Q	12.6	m^3/s

Table C.23: Seepage parameters and results.

Although an absolute discharge of 12.6 m^3/s is substantial, considering the fact that the entire reservoir capacity is approximately 6×10^8 m^3, the seepage discharge is not significant and hence can be neglected.

Scour protection - Powerhouse After the entire caisson dam has been installed, including the powerhouse elements that accommodate the turbines for the PSH plant, the scour protection for the inlet and outlet of the reservoir must be installed. High discharge streams near these in- and outlets are expected during charging and discharging phases, also associated with high turbulence. According to an extensive study into the powerhouse elements and civil structures for the turbines, the maximum discharge or design discharge is expected to be 130 m^3/s. The end of the inlet and outlet has a diameter of 10 m, with an area equivalent of approximately 80 m^2, resulting in an average flow velocity of 1.6 m/s.

Similarly to the stability of the rock foundation, an alternative form of the Izbash formula can be used to determine the minimum rock diameter to ensure sufficient stability under the most critical conditions. The boundary conditions imposed for this stability assessment are given in Table C.24. It must be noted that due to the high turbulence near the outlets, the turbulence factor r has been set to 0.3 (which is also used for situations that involve stream jets).

$$\Delta d = 0.47 \frac{(u_c(1+3r))^2}{2g} \tag{C.5}$$

in which:

- Δ [-] = Relative density of stone in sea water.
- d [m] = Stone diameter.
- u_c [m/s] Critical flow velocity.
- r [-] = Turbulence factor for the specific scenario.
- g [m/s^2] = Gravitational acceleration.

Parameter	Symbol	Value	
Flow velocity	u_c	1.6	m/s
Turbulence factor	r	0.3	-
Relative density	Δ	1.56	-

Table C.24: Stability calculation for armour stone in running water, using Izbash's approach.

Applying the Izbash's formula (Equation C.5), the minimum diameter to ensure stability for the high turbulence flow near the in- and outlet, is approximately 16 cm, which is the equivalent of the LM_A 5 - 40 kg grading (which has a mean diameter of d_{n50} = 17 cm). The length of the bed protection, which must be installed at both sides of the dam, has been estimated on 100 m, with a layer thickness of 2 d_{n50}, approximately 40 cm. The layout of the scour protection near the turbines is shown in Figure C.5. In addition to the scour protection that is required to prevent erosion due to the in- and outflow of the reservoir, close to the dam the regular scour protection is necessary to absorb the wave impact.

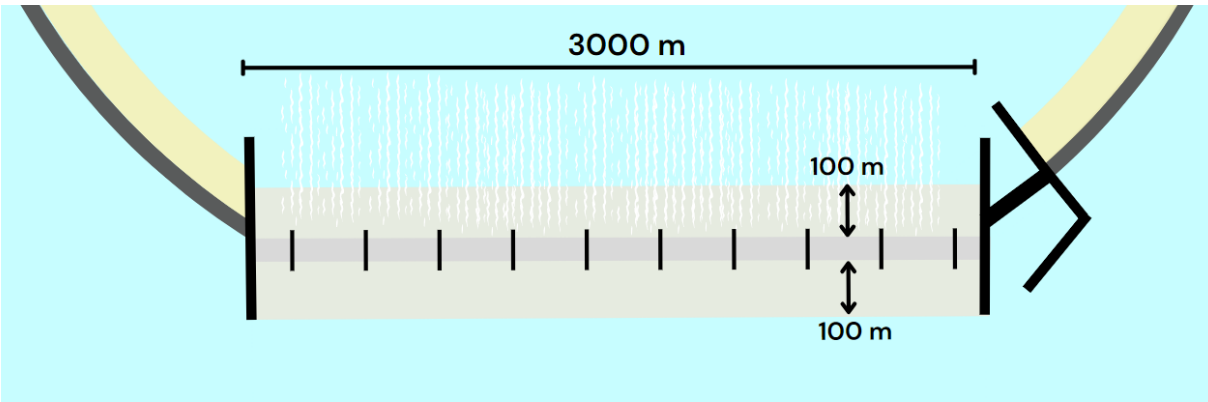


Figure C.5: Schematic layout of the bed protection to prevent scour due to the charging and discharging phases of the turbines. The scour protection consists of a LM_A 5 - 40 kg grading, applied over a stretch of 3 km by 200 meter, extending 100 meter on both sides of the dam.

Inner berm revetment According to the Bretschneider equations the significant wave height and period can be determined, knowing the fetch length (5000 m, equivalent to the reservoir diameter) and the water depth, which varies between 29 m and 6 m. The wind speed U_{10} has been extracted from the Aktis Hydraulics database, and the wind speed versus wind direction scatter is shown in Figure A.13.

Assuming the revetment will be designed for a storm with a return period of 100 years, the corresponding design wind speed u_{10} is 25 m/s. Considering the fact that the reservoir can vary in water level (since water will be pumped out, the water level will drop), several situations have been considered. The governing scenario that leads to the most extreme wind waves is when the reservoir completely full, with a water depth of 29 m. For this situation the expected wave height in the reservoir will be $H_s = 1.4$ m with a significant wave period of 3.5 s (peak period of $T_p = 3.7$ s).

The van der Meer equation has been composed to determine the rock size as a function of many parameters, among them are wave height and damage level. The equation is given by :

$$\frac{H_s}{\Delta \cdot d_{n50}} \begin{cases} c_p P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \zeta_m^{-0.5} & \text{if } \zeta_m < \zeta_{cr} \text{ (plunging)} \\ c_s P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \zeta_m^P \cot \sqrt{\alpha} & \text{if } \zeta_m > \zeta_{cr} \text{ (surging)} \end{cases} \tag{C.6}$$

where ζ_{cr} is known as the Iribarren number, and is given by:

$$\zeta_{cr} = \left[\frac{c_p}{c_s} P^{0.31} \sqrt{\tan \alpha}\right]^{\frac{1}{P+0.5}} \tag{C.7}$$

For wave conditions with a period of 3.5 s and height of 1.4 m on a slope of 1:14, plunging waves will be expected ($\zeta_m = 0.28, zeta_{cr} = 0.08$). Assuming a damage level of $S = 4$, number of waves in a storm of $N = 1000$ and a permeability of $P = 0.1$, the van der Meer equation estimates the $d_{n50} = 0.21$ m. According to the standard gradings (Schiereck, 2019; CIRIA et al., 2007) this would correspond to a LM_A 10 - 60 kg grading.

D
Choice of equipment

D.1. Sea bed preparation

For the choice of equipment for this activity, there are many options with different characteristics and features. An inventory of type of equipment that can be used for dredging works can mainly be distinguished into the following categories:

1. Back Hoe Dredge (BHD) The Backhoe dredger is a stationary dredger and consists of a hydraulic excavator installed on a pontoon. One of the characteristics is that BHDs can dredge in very shallow waters, with a high accuracy and a wide range of materials. The largest BHD of Boskalis has a bucket of 40 m^3 and can dredge to a depth of 32 m.

Considering the scale of the project, a total length of approximately 16 km, and the fact that the dredge activity will be executed offshore, the BHD dredge will not be a viable option.

2. Cutter Suction Dredge (CSD) The Cutter Suction Dredges, either self-propelled or assisted by tugs, are equipped with a rotating cutter head. This rotating cutter head is excellent for cutting all materials, including rock, silt, clay and sand. Due to the limited draft of the vessel, the CSD is deployed in the construction and maintenance projects of ports, land reclamation and coastal defences, and dredging trenches for pipelines. The dredged material can be pumped and transported via floating pipeline to the desired location.

Similar to the BHD, the deployment of the CSD for the levelling dredging is not desired, since the height of the dredging works is limited, making the CSD less efficient.

3. Trailing Suction Hopper Dredge (TSHD) The TSHD are oceangoing vessels that can dredge sand and silt with two large dragheads which are lowered along the vessel. These dragheads move slowly over the sea bed and collect sediment with their water jets and blades. This slurry of sediment and water is then pumped via a suction pipeline onboard and deposited in the hopper. Excess water can leave the hopper via an overflow, which in turn increases the capacity of the dredger, but also generates a plume of sediment which may have environmental impact.

An overview of the available vessels is shown in Figure D.1. Due to the oceangoing aspect of the TSHD, the workability limit is high compared to the other alternatives. Moreover, in view of the scale of the project, the TSHD would also be the best option since these are autonomous vessels with a high production and hopper capacity. Taking these aspects into account, the TSHD will be the desired equipment to deploy for the levelling dredging works.

(a) (b) (c)

Figure D.1: Type of equipment related to dredging activities: (a). Back Hoe Dredger (BHD), (b). Cutter Suction Dredge and (c). Trailing Suction Hopper Dredge (TSHD).

D.2. Rock foundation

For the installation of the rock layers (foundation and scour protection A) multiple potential vessels can be deployed, ranging from very basic barges to highly advanced, dynamically positioned vessels. In general, there are four types of equipment that can be distinguished into the following categories:

1. Side Stone Dumping Vessel (SSDV) The SSDV is a widely applicable vessel, that dumps rock along both sides of the vessel by pushing the stones off the ship. In general, SSDV vessels have a loading capacity of 5,000 tons. As the stones are dumped in to the sea from the deck, the placement of the rock is inaccurate and executed in an uncontrolled manner, especially during installing in larger water depths.

2. Split barge As the name suggests, the split barge is a vessel that can split the hull along the length of the ship, thereby dumping the (rock) content. The capacity of a split barge ranges between 600-1,000 m^3, or roughly 1,500-2,500 tons. Similar to the SSDV vessels, due to the uncontrolled dumping of the rocks the installation of the rock layer will be inaccurate, resulting in more levelling work afterwards.

3. Pontoon with crane An alternative of the self-propelled vessels, is a ocean-going pontoon with excavator. Barges or pontoons will supply the crane with filter rock, which is then installed by the crane. Since the stone is applied with a crane, the accuracy will be slightly higher compared to the latter two options, however is still insufficient and will need additional levelling before installation of the caisson.

4. Fall Pipe Vessel (FPV) The Fall Pipe Vessel is a high advanced vessel that offers a dynamically positioning function for precision installation of the rock layer. Equipped with a fall pipe, that runs from the deck to the sea bed, the rock can be supplied at the top with a crane and will fall through the pipe. At the end of the pipe, multiple propellers are installed to maneuver the pipe to the desired location. In general, FPV are large ocean-going vessels with a large capacity extending to 15,000+ tons. This method offers great installation accurate due to the controlled manner, and hence requires less levelling works afterwards.

An overview of the potential equipment dedicated to the placement of the rock works is shown in Figure D.2. Considering the high accuracy and controlled manner of placement, and therefore less (levelling) work is needed afterwards, in combination with the high capacity, the FPV will be the equipment of choice for this task. One of Boskalis' Fall Pipe Vessels, the Seahorse, is shown in Figure E.7. With a volume capacity of 17,500 m^3, a length overall of 162 m and an installation capacity of 1,500 t/h this could be a viable option for this task.

(a) (b) (c) (d)

Figure D.2: Type of equipment related to rock works installation: (a). Side Stone Dumping Vessel (SSDV), (b). Split barge, (c). Pontoon with crane and (d) the Fall Pipe Vessel (FPV)

D.3. Caisson

Construction method

Considering the dimensions of the caisson, two viable options can be distinguished for construction. The first option is to construct the caissons in an adjacent temporarily working dock, which must be constructed for this project, and can be flooded in a controlled manner to float caissons for transportation. The other option is the construction on a pontoon like the Kugira, which is in fact a floating, semi-submersible factory, which allows fast production of the caissons.

The working dock dock will be similar to a polder: dikes will be build from land extending in to the water, closing an area from which water can be pumped out. This will provide a dry working space, where the caisson can be constructed. After the construction has finished, all equipment is moved to a safe location, after which the temporarily dikes can be breached, flooding the area with finished caissons. Now the caissons are afloat, and can be transported with tugs to a designated storage. The cycle continues, by reconstruction of the dikes, pumping the working area and construction of the caissons. This method was used to construct parts of the Deltawerken in The Netherlands; the construction of the caisson for the Volkerakdam is shown in Figure D.3.

Figure D.3: Temporary construction dock for caissons consists of a polder surrounded by dikes, which can be flooded in a controlled manner to float and thereby enable transportion of the caissons to the desired location. [Image: Rijkswaterstaat]

Floating caisson-building docks become more popular, as they are more compatible and have less requirements and constrains than other methods like the construction harbour. Equipped with dedicated equipment like cranes and concrete mixers, and over 200 people work simultaneously enables the construction of over 200 meters of docks and quays within a month. The fabrication starts with the bottom slab of the caisson, followed by the walls and finally the top part is constructed. After construction has finished, the deck will be semi-submersed, enabling the transportation of the caisson while the caissons are afloat. The Kugira, Europe's biggest caisson-building deck, is shown in Figure D.4 while deploying a constructed caisson.

Figure D.4: Floating caisson-building dock Kugira deploying a caisson. [Image: Acciona]

The main advantage of a floating caisson-building dock is the high production efficiency compared to the working dock. Due to the efficient structured factory, the Kugira can produce a caisson with similar dimensions of the one that is proposed for this project, within 11 days. One of the disadvantages of the building dock is the fact that after the completion of the construction, the entire area must be cleared from equipment after which the area can be flooded. Subsequently, the caissons can be transported to a nearby storage area, the dikes must be reconstructed and then the water must be pumped out.

Considering the number of caissons that must be produced and thereby the foreseen potential time savings that can be achieved, the construction of the caisson will be done at floating caisson-building dock. Seen the favorable deep water ports that is required for the semi-submersible factory and HTV, construction will be executed at southern part of Spain. After completion of a caisson, it will be stored near the production site. When enough caissons (5-6 units) have been finished, they will be loaded onto a Heavy Transport Vessel (HTV) and sailed to an adjacent temporary storage in The Netherlands, where they will be offloaded.

Dedicated equipment
The caisson stream consists out of a total of 6 activities that require dedicated equipment, which can be categorized as follows:

1. Fabrication of caisson Since the fabrication of the caisson is fully automatized and executed on the floating caisson-building dock, no additional vessels are needed. Only during the launch of the caisson, during the semi-submersed phase of the dock, one or two coastal/harbour tugs are needed to guide the floating structure in a controlled manner to leave the dock, e.g. the IMO River shown in Figure D.5a.

2. Towage in local port After the caisson has been safely guided out of the floating dock, two tug boats are needed to tow the caisson to the intermediate wet storage, adjacent to the floating dock. Here the caissons are moored temporarily, until enough caissons are ready to transport with the Heavy Transport Vessel. Again, coastal/harbour tugs can be deployed like the one in Figure D.5a.

3. Transportation from manufacture to intermediate storage Transporting the caissons from Spain to The Netherlands will be done by one of the Heavy Transport Vessels (HTV), like the Boka Vanguard shown in Figure D.5b. With a length of 275 m and a width of 70 m, several caissons with the proposed dimensions can be transported at once.

(a) Coastal/Harbour tug.

(b) Heavy Transport Vessel (HTV) Boka Vanguard.

Figure D.5: Dedicated equipment for the transportation from manufacture to intermediate wet storage at Port of Rotterdam.

4. Towage in local port and to project site Offloading at the local port in the country of destination, will be guided with at least two coastal/harbour tugs like Figure D.5a, and towed to a dedicated mooring until further notice. When the caisson will be installed, oceangoing tugboats or multi-purpose vessels are needed to tow the caisson to the project site. First the caisson will be prepared with navigation lights and mooring winches in the port, and is then connected to an Anchor Handling Tug (AHT). Under guidance of port tugs, the AHT and caisson leave the port until a point where the assistance is no longer needed and the port tugs and pilot can stop their service. From here, the AHT will sail to the project site, approximately 100 km from the Port of Rotterdam.

5. Installation of caisson Prior to the transportation of the caisson from the port to the project location, a Heavy Lifting Barge (HLB) will sail to the designated location where it will anchor and wait for the first caisson. At arrival of the caisson with the AHT, the caisson will be positioned between the HLB and the AHT and is assisted by two push tugs, positioning to the desired location. Then the ballasting operations commence. After placement of the caisson, mooring lines will be disconnected and the AHT sails back to the port for the next caisson. The HLB and AHT are shown in Figure D.6a and D.6b respectively.

After installation of the caisson, the wave return walls will be lifted by the HLB and placed on top of the caisson unit. Figure D.7 shows the load diagram of the HLB, and has a safe load capacity of is maximum

600 mT with an associated lifting radius of 12.2 m, however to ensure safe lifting the goal is to fabricate the wave return wall below 400 mT to obtain a radius of 30 m.

(a) Heavy Lifting Barge (HLB) Giant 7. **(b)** Anchor Handling Tug (AHT) Fighter.

Figure D.6: Vessels dedicated for the positioning and installation of the caisson.

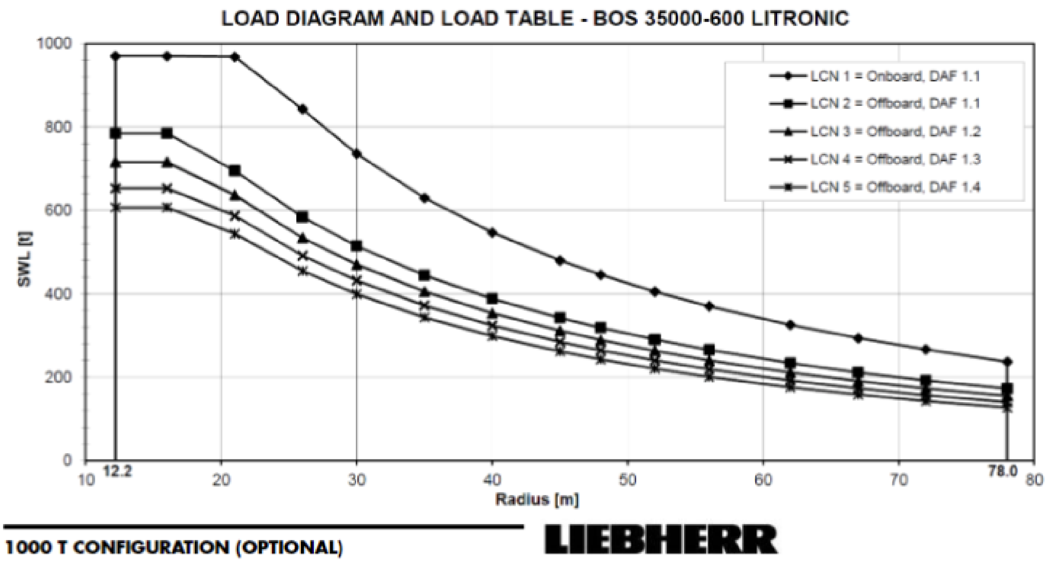


Figure D.7: Load diagram of the Giant 7.

E
Work method

The work method is a comprehensive executive summary of the entire project, which will give an overview of all construction works, the dedicated equipment that will be deployed and the estimated volumes to obtain the proposed design of the caisson dam presented in Figure E.1. Therefore, this Section will discuss the set-up for the execution, fabrication and installation of all offshore works, including:

- Dredging works
- Rock works
- Caisson stream
- Reclamation works

The works described above will be discussed in chronological sequence.

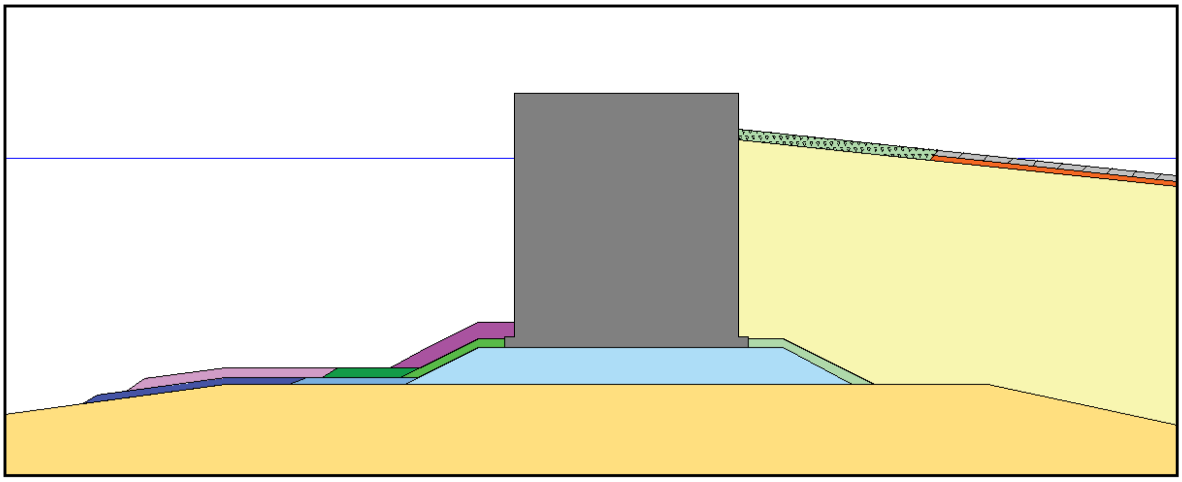


Figure E.1: Design of the caisson dam of the PSH, based on Project Alpheus.

First, the construction starting point will be determined based on the prevailing wave conditions. Afterwards, the work method per construction sequence will be described, with a general description of what the sequence includes, the dedicated equipment and logistics and finally the estimated volumes.

E.1. Construction phasing

Construction phasing is the process of breaking down a construction project into smaller steps in a logical sequence, with the goal to achieve a successful and safe project outcome. The starting point of the construction phasing for an offshore project is important when taking into account the workability which largely depends on the wave conditions.

Therefore, this Section will first shortly introduce the construction phasing and the importance related to an offshore structure. Afterwards, the starting point will be determined considering the governing wave direction in the North Sea, near the project location. Moreover, for the deterministic planning the construction sequence will be simplified and consider only one 'work train', i.e. from the starting point construction of the dam will be continued only in one and same direction.

Basically, the construction phasing determines where and in what order the construction sequences will commence. For instance, for an apartment building this would consists of the installation of the foundation first, then the construction of the ground floor walls and ceilings, continuing with the first floor etc..

The offshore PSH will be built in the middle of the North Sea, subjected to offshore wind and wave conditions, and hence must be constructed entirely with vessel-based equipment. All vessels have their maximum workable conditions, i.e. wind and wave conditions beyond work will become unsafe, also known as the workability limit. It is assumed that after a caisson unit have been installed, the wave conditions behind the unit will improve, simply because the structure will (partly) block direct incoming waves. With this assumption, the composition of the phasing (where should construction works commence) could have a great influence on the time construction works can still continue.

Two qualitative examples of the difference in sheltered zone are shown in Figure E.2a and E.2b, both for a Northern wave direction. In the first drawing the caisson dam faces North East, whereas the second dam faces directly North, resulting in a different sheltered zone indicated by the dashed lines. Depending on the directional wave distribution in the North Sea, playing around with the construction phasing could be beneficial regarding the project duration and costs. Working with different scenarios, and their consequences, will be done later on in this research, but for now the starting point will face towards the governing wave direction.

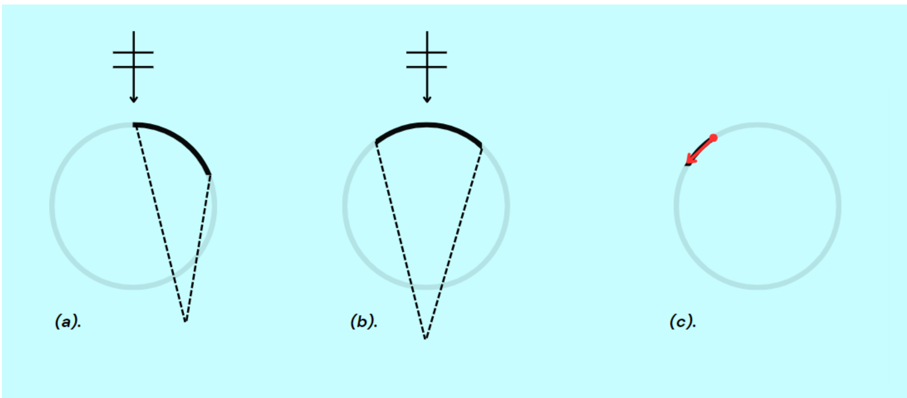


Figure E.2: Figures (a) and (b) show a qualitative sketch of how the choice of starting point may influence the working conditions and hence has an effect on the workability. (c) Indication of the preliminary starting point, based on the governing wave direction in combination with wave height.

Governing wave conditions / Determination of construction phasing

The phasing for an offshore project depends mainly on two parameters, the wave conditions (wave height and wave direction) in combination with the probability of occurrence. For the first part of the research the construction phasing is only important to have some reference and starting point, rather than provide a planning on a high level of detail, as this will be at a later stage. Therefore, this Section will only base the phasing on the most extreme wave conditions and consider these conditions as governing, which has been analysed thoroughly in Section 2.

From that Section, it turns out that two governing wave directions prevail: North-North-West and South-West. Taking into account the corresponding wave height for these wave directions, higher waves originates from the former direction, as also can be seen in Figure A.10. Therefore, the construction works will commence at the North-West part of the dam working Southwards, which has also been visualised in Figure E.2c.

E.2. Dredging works

The first construction activity is the seabed preparation for the placement of the rock foundation, which is the foundation of the caissons. This layer must be horizontal leveled and constant, to prevent stability concerns regarding the caisson and deviations in crest height. As is discussed earlier, the proposed location contains multiple sand dunes. To that end, levelling dredging works must be executed to obtain a smooth surface that allow the installation of the rock layer.

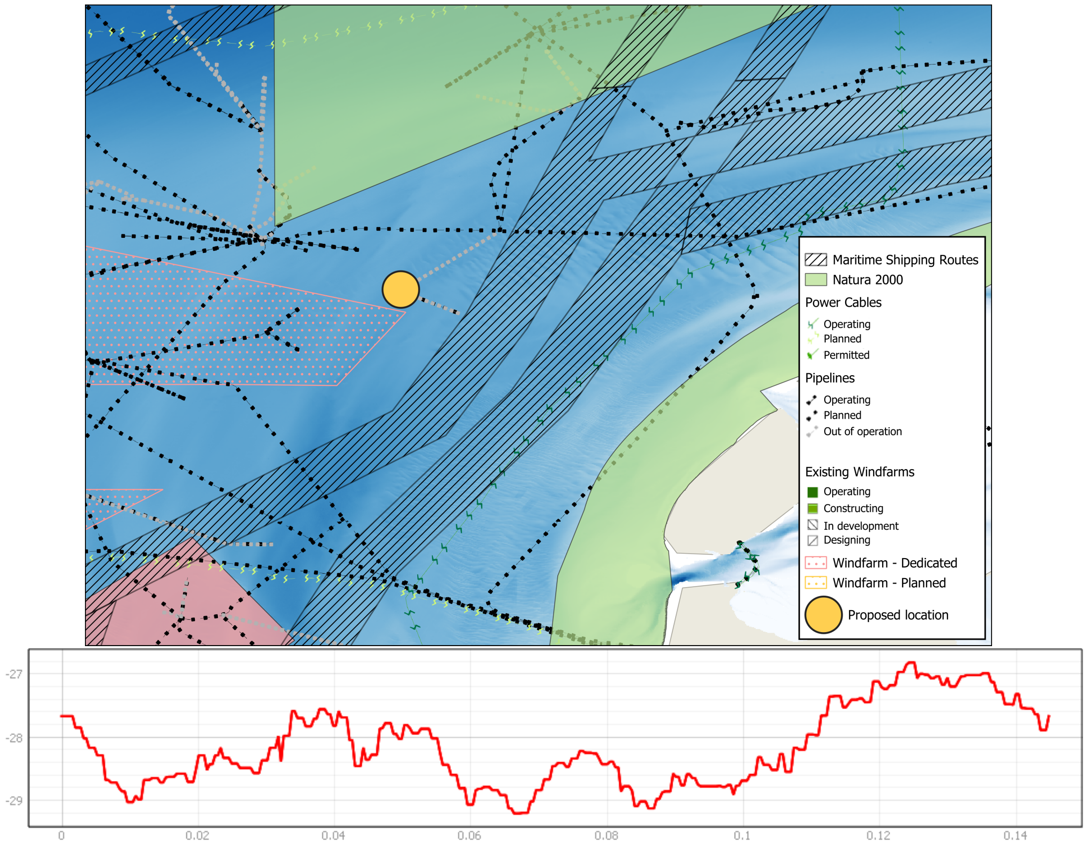


Figure E.3: Detailed overview of the project area consisting of the current and planned North Sea infrastructure, maritime shipping routes and desired construction site. Below the chart, the bathymetry of the perimeter is shown with respect to mean sea level.

Dedicated equipment

Appendix D discusses the available vessels with regard to the sea bed preparation, i.e. dredging works, while taking into account important aspects like the amount of dredging works and production rates of the vessels. Eventually, the advantages of the Trailing Suction Hopper Dredge, shown in Figure E.4, could not be matched by other alternatives and is therefore the desired type of equipment.

Figure E.4: Trailing Suction Hopper Dredge 'Fairway'.

Volume estimation

According to the site investigation - discussed in detail in Section 1.3 - the newly proposed and desired location for the PSH facility is North of the Waddeneiland Texel, approximately 30 km off the Dutch coast

and 150 km from the Port of Rotterdam. Important aspects for the dredge activities are the type of soil that must be dredged and the estimated dredge volume. Both aspects have been elaborated or presented in Section 1.3. According to the boreholes from DINOloket, the soil top layer is sand (see Figure A.8) which suits the TSHD.
An estimation for the dredge volume has been made with QGIS, in which the bathymetry has been extracted with a profile tool that provides the sea bed elevation for the entire perimeter with an interval of 40 m in length. Considering a reference level of MSL -29.0 m and a dredge area with a width of 60 m, it results in a total volume of 734 000 m^3.

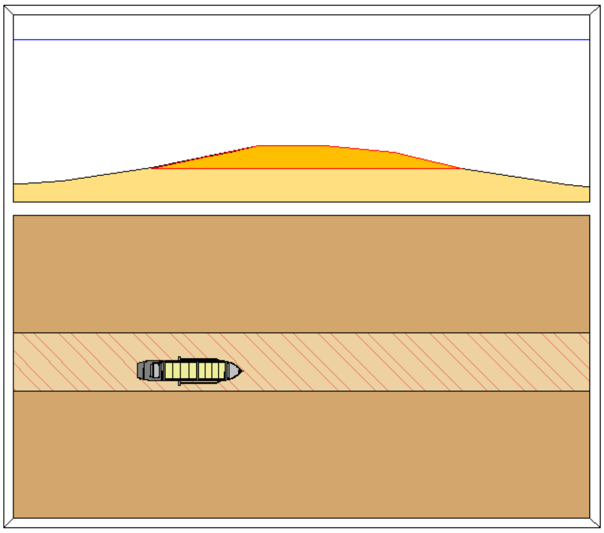


Figure E.5: Schematic overview of the sea bed preparation prior to the installation of the rock foundation layer. At the top the cross section is shown, where the orange highlighted area indicates the volume that must be dredged. At the bottom a top view of the construction sequence is shown, where the TSHD is operating.

E.3. Rock foundation and scour protection

After the sea bed preparation, a rock layer will be installed to ensure a stable and leveled foundation for the caissons, and will be installed prior to the caisson installation. Part of this sequence, a scour protection layer will be installed at the toe (sea-side) to prevent erosion during installation of the caisson and the time until full coverage of the final scour protection. This sequence has been visualised in Figure E.6. After applying the foundation layer, before installation of the caisson the layer must be levelled to ensure the stability of the structure.

As has been discussed earlier, in Section 3, the stability of the rock foundation layer and scour protection - consisting of the same 45 - 180 mm grading - is ensured until a flow velocity of 1.3 m/sec, while daily flow velocities fluctuate between 0.8 - 1.0 m/sec. This would mean that the entire rock layer can be installed prior to the installation of the caisson unit, regardless of the progress of the caisson installation.

The advantage of this method is that there will be no hiccups in the planning, since the caisson can always be placed. However, it is uncertain how the sea bed will react on a sudden large disturbance (installation of the rock layers) and therefore the response in the form of erosion and deposition of sediment is also unclear, potentially damaging the rock layers. Moreover, while dredging activities continue (sea bed preparation and filling of the caisson units) fine sediments are brought into suspension. This could potentially lead to the deposition of (fine) sediment on and into the rock layers, reducing the structural characteristics.

Therefore, an adaptive way of working will be applied for the installation of the foundation layer and scour protection. Basically, this method will adapt on the progress of its later stage (installation of the caisson unit). The minimum distance between the end of the caissons and the end of the rock works will be 5 caisson lengths after which a new layer will be installed with a length of 5 - 10 caisson units, depending on the prediction on the amount of caissons that can be installed in that year.

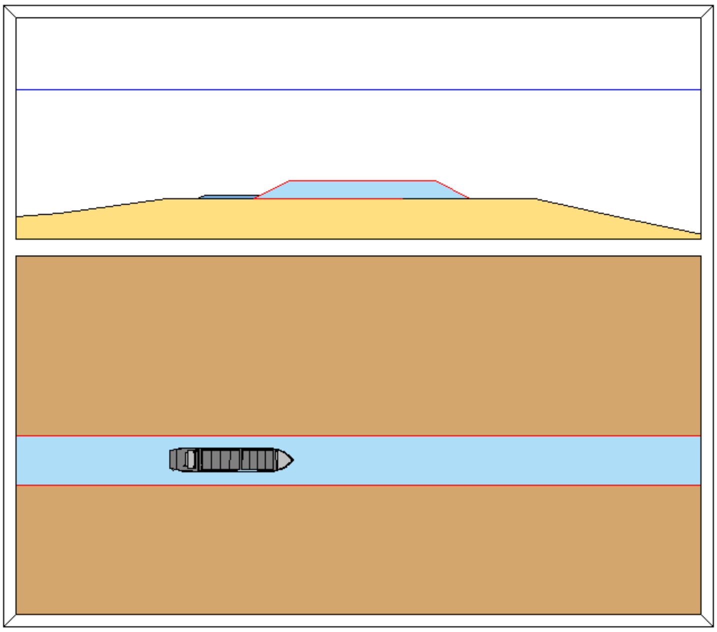


Figure E.6: Design of the rock works, prior to the caisson installation and after the dredge activities.

Dedicated equipment

In Appendix D an extensive overview of the potential types of equipment has been shown and is discussed thoroughly in which multiple parameters and aspects has been taken into consideration, for instance the amount of work, production rates and suitability with regards to the (offshore) conditions. From this consideration, it turns out that the great installation accuracy and high production rates are of the Fall Pipe Vessel is of great importance to this project and hence will be dedicated to this task.

Figure E.7: The Seahorse, one of the Fall Pipe Vessels (FPV) of Boskalis' fleet, during installation of a rock layer.

Volume estimation

The first sequence of rock works include the rock foundation and the first scour protection, located at the toe of the structure. The exact dimensions of the rock foundation are 60 m wide with an average height of 2.5 m and a slope of 1:2 which will be applied over the entire length of the caisson dam, hence a length of 15.7 km. These dimension result in a volume of 7.7×10^3 m^3 m^3 or 2.0×10^3 ton per caisson unit, assuming 262 caisson units for the entire project this equals 2.0×10^5 m^3 or 5.3×10^6 ton (assuming $\rho = 2.6$ ton/m^3).

The scour protection at the toe of the rock foundation has the following dimensions: 15 m wide with an average height of 0.5 m. Also this rock works stretch over the entire length of the project, and hence must cover the total length of 15.9 km (taking into account the increase in length due to larger radius). This results in a volume of 479 m^3 or 1246 ton per caisson unit, again assuming 262 caisson units this results in a total of 1.26×10^5 m^3 or 3.26×10^5 ton (assuming $\rho = 2.6$ ton/m^3) for the scour protection layer.

Layer	Grading	Vessel	Volume/unit [m^3]	Total Mass [mT x 10^3]	Legend
Rock Foundation	45 - 180 mm	FPV	7725	5262	
Scour Filter (inner)	45 - 180 mm	FPV	479	326	

Table E.1: Volume estimations for the scour protection and filter layers.

Rock supply and loading operations

The material for the rock foundation, levelling material and scour protection originates from quarries in Norway, which will be transported with bulk carries to the port of Rotterdam. Here, the material will be stored temporarily in distinct stockpiles, separated based on its purpose. After it has been stored temporarily, the FPV will moor at the quay wall and the material will be loaded onto the FPV, by a land-based conveyor belt system or by simple land-based equipment. After the FPV has been loaded, it will sail to the designated location, deploy the fall pipe and install the rock layer. The transportation of the rock material is shown in Figure E.8.

Although the required total volume of rock that is large (5.26×10^6 ton for the foundation and 3.26×10^3 ton for the scour protection), the installation of these layers will be spread out gradually over time, as construction progresses, and hence the material will be spread out over several years to distribute the load on the vessels, supplier and temporary storage.

Figure E.8: Chart of the transportation of rock material from quarries in Norway to the Port of Rotterdam in the Netherlands by bulk carrier (in black), afterwards it is loaded onto the FPV and transported to project site (in orange).

E.4. Caisson stream

The caisson is the major component of the entire design, and can be seen as the spine of the entire superstructure. After installation, the caisson will not only absorb the wave impacts, but also retains water when the reservoir is empty and the crest will also provide access to the entire dam during maintenance or emergencies.

The design of the main caisson unit has been described in Section 3.1 and verified in Section 3, resulting in the final dimensions: 60 x 32 x 34 m (length x height x width). Although it is part of the optimization section, it is worth mentioning that while construction works progresses a smaller size caisson might be used instead of the main caisson. For example, looking at the wave conditions with regard to the wave directions, waves originating from the South West will have less severe impact on the structure compared to the other wave directions. In line with this reasoning, a simple optimization with regard to the caisson use would be to install smaller caissons in the South East region, where the wave conditions will be far less. A qualitative sketch of the superstructure including the caisson array is shown in Figure E.9, where two types of caissons are indicated: green indicates caisson that are smaller than red. Nevertheless, for now it is assumed that all caissons will be the same dimensions.

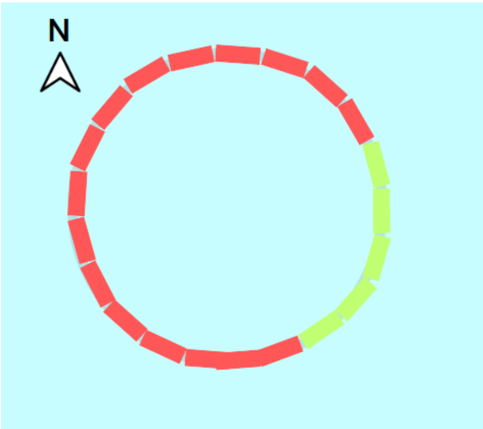


Figure E.9: Conceptual spatial layout of the caisson arrangement. Green area indicate smaller caisson compared to the red area due to the expected wave impact.

Construction methods

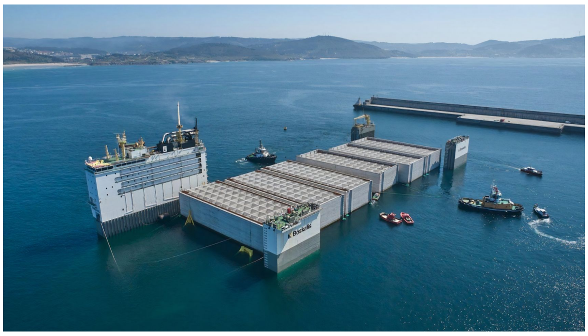
Generally, there are two methods for the fabrication of the caisson units that are widely applied: i). fabrication in a temporarily (dry) polder that can be flooded or ii). fabrication on a floating caisson construction facility, specially dedicated for this activity. In Appendix D all relevant aspects are taken into consideration. It turns out that the latter method, the fabrication on a floating facility, is more suitable on every aspects and hence will be used for this project.

Work method - caisson stream

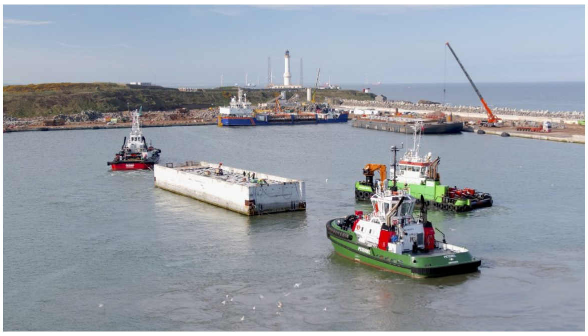
A visualisation of the work sequences of the entire caisson stream (an overview of all activities), starting from fabrication to installation at site, is shown in Figure E.10.

(a) Caisson fabrication at a floating caisson-building dock.

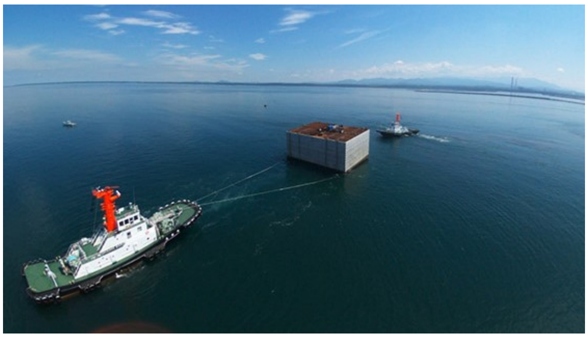
(b) Launch and towage of the caisson to intermediate storage.



(c) Loading and transportation to intermediate storage in The Netherlands.



(d) Offloading of the caisson from the Heavy Transport Vessel and towage to the intermediate wet storage.



(e) Transportation from intermediate storage to project site, and placement of the caisson to the desired location.

(f) Installation of the caisson, including the ballasting and positioning.

Figure E.10: Sequence of activities for the caisson stream, for the floating caisson-building dock.

Dedicated equipment

An extensive overview per construction sequence can be found in Appendix D. In this Section only a concise inventory will be given, excluding for example details of the vessels.

In general, for the caisson stream three types of vessels are necessary: i). at the floating construction facility, tugboats are needed to gently immerse the caisson and tow the unit to the desired temporarily wet storage until further transport, ii). for the transportation to the temporarily storage in The Netherlands, a HTV in combination with two or three tugs are necessary, both at the departing and arrival storage, iii). for the transport from the storage near site to the project area an AHT with smaller assistance tugboats are required.

Caisson fill

The caisson will be filled primarily with dredged material obtained during the levelling dredging activity or sea bed preparation. In a later stage of the project, when the sea bed preparation is finished, an adjacent borrow area will be appointed from which material can be used for the caisson filling. This borrow area will likely be the inner perimeter of the reservoir.

Filling of the caissons will be done by a modified Trailing Suction Hopper Dredger (TSHD), with additional pumps in the hopper well to enable direct filling via modified pipelines. The TSHD will connect via the bow to floating pipelines which runs to a special inlet of the caisson, which has been installed during fabrication. These integrated spread bars in the caisson allow a fast connection and transition between caissons, and are easy to install during fabrication. In addition, prefab holes in the caisson allow overflow during unloading of the TSHD in the caisson. This filling process is similar to the hydraulically discharge method of the TSHD, that is used for reclamation.

Sea- and wave return wall

After filling the caisson, the additional seawall (or wave return wall) - which is indicated in Figure E.11 - must be installed to finish the caisson entirely. These concrete prefab elements are fabricated on shore, for instance at the Port of Vlissingen or at the Harbour Port of Great Yarmouth in the United Kingdom, and loaded onto the HLB and will sail to the project site. After the caisson has been filled with sand, these concrete elements will be lifted with the crane of the HLB and installed on the caisson. Multiple trips of loading and installation are necessary, but meanwhile the installation and filling stream of the caissons can continue by means of tug boats.

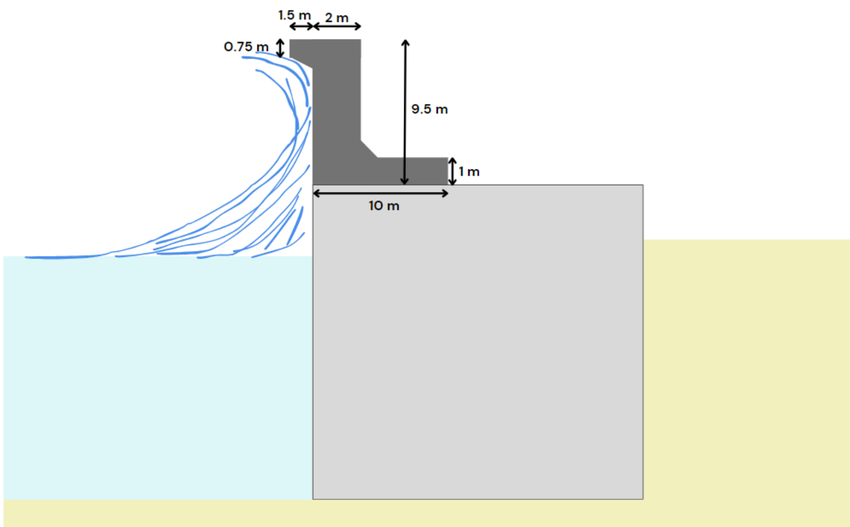


Figure E.11: Schematic view of the wave return wall on top of the caisson unit.

The dimensions of the wave return wall have been determined in Section 3.1, and are given below. The length of the wave return wall has been chosen based on the characteristics of the lifting capacity of the HLB to ensure safe lifting, and has resulted in a length of 6 m. The concrete element has each an approximate weight of 450 mT, which is below the safe lifting capacity of the HLB. The details of the load diagram of the crane barge can be found in Appendix D. Moreover, it is assumed that the HLB can accommodate 40 wave return wall elements on deck, after which it must sail to the Harbour Port of Great Yarmouth to load a new batch. This implies that when fully loaded, four caissons can be entirely constructed including the wave return walls.

- Length: 6 m
- Width: 10 m
- Height: 9.5 m
- Thickness slab: 1 m
- Thickness wall: 2 m
- Height bullnose: 0.75 m
- Width bullnose: 1.5 m

Intermediate wet storage
The intermediate wet storage involves the temporary mooring of the caisson units, until the dedicated resources are available and weather conditions are appropriate for the transport and installation. Such a storage area is necessary since the installation of the caisson units is a faster process than the fabrication and transport of the units itself. Therefore, a certain caisson buffer must be present to cope with fast installation periods.

After the caisson have been fabricated in Spain, they will be transported with a semi-submersible Heavy Transport Vessel to a port in the vicinity of the project site. Because the proposed location is North of the Netherlands, the Port of Rotterdam would be a obvious first choice. The HTV arrives at the dedicated port where it will be unloaded. The caissons will be towed to the intermediate wet storage area, where they will be moored to the mooring dolphins. Meanwhile, construction works will continue and demand caisson units for the installation of the caisson dam. Consequently the caissons will be moored in the port until they are needed for the construction works.

This intermediate wet storage plays a vital role in maintaining the construction rate, as its literally the stockpile for the project. Moreover, the unpredictable weather is also an important parameter in this process: during good weather conditions the stockpile must be sufficiently large in order to supply the project with caissons. While on the other hand, during a long period of bad weather, the caisson installation will progress at a lower rate and hence the stockpile outflow will be less while the inflow is constant. With insufficient logistic planning, the stockpile can be flooded with caissons during bad weather, potentially hindering the caisson factory in Spain, but with good weather the stockpile may be insufficient to supply the demand, thereby delaying the project.

A proper logistic plan is therefore key to a successful project. Especially when considering the optimisation of processes, design or other aspects in this project, the construction rate will most likely increase, thereby further increasing the strain on the logistics. Therefore it is necessary to inventory what the possibilities are, potential limitations and sensitivities are of the intermediate wet storage.

The size of the buffer is associated with multiple criteria, among them are the number of construction fronts (i.e. the number of sites where caissons are installed simultaneously), the expected length of good weather windows, the number of caisson fabrication platforms and the production rate. Although the buffer size is a very important parameter that is associated with the project risk, the exact determination of the caisson buffer is considered to be beyond the scope of this research.

To that end, a conservative buffer size of 30 caisson units has been designed which is located in the Port of Rotterdam. Figure E.12 shows the layout of the caisson buffer which is issued nearby the Amaliahaven in the Port of Rotterdam. The configuration includes possible berthing facilities and the local bathymetry. In case the proposed wet storage is insufficient with regard to the demands from the project, extra alternative locations such as Vlissingen, IJmuiden, Terneuzen and even harbours along the UK coast can be approached.

Besides the caissons available at the intermediate wet storage, during the construction season the arrival of caissons from the fabrication is constant (fabrication time of one caisson is 11 days): every 2 months 5 caissons will be delivered at the Port of Rotterdam, which can continue unobstructed the entire year. This implies that in one year a total of 30 caissons can be produced.

Assuming that the storage area is filled for 80% at the beginning of the work season (24 caissons), the maximum installation quantity of caissons will be the storage quantity plus the additional caissons that arrive from the factory. Considering a work season of 6 months, in total 15 caissons will be delivered at the intermediate storage. Consequently, the maximum installation capacity in one year will therefore be 39 caissons, but only when the storage has been filled for 80%.

In order to restore the storage during winter to 80% (24 caissons), the fabrication will take roughly 10 months. This means that the production period of the caissons to restore the intermediate storage takes longer than the construction off-season. So, at the beginning of the second year the storage will not be at the desired quantity, potentially resulting into project delay. Therefore, mitigation measures must be taken in order to prevent a lack of caissons during the work season.

One possible mitigation measure would be to increase the caisson production during the winter, by adding an other floating caisson fabrication pontoon. This will double the production rate of the caissons, resulting in the production of 5 caissons per month (in stead of 2 months). Consequently, this allows the desired storage capacity of 25 units to be reached by the end of the off-season.

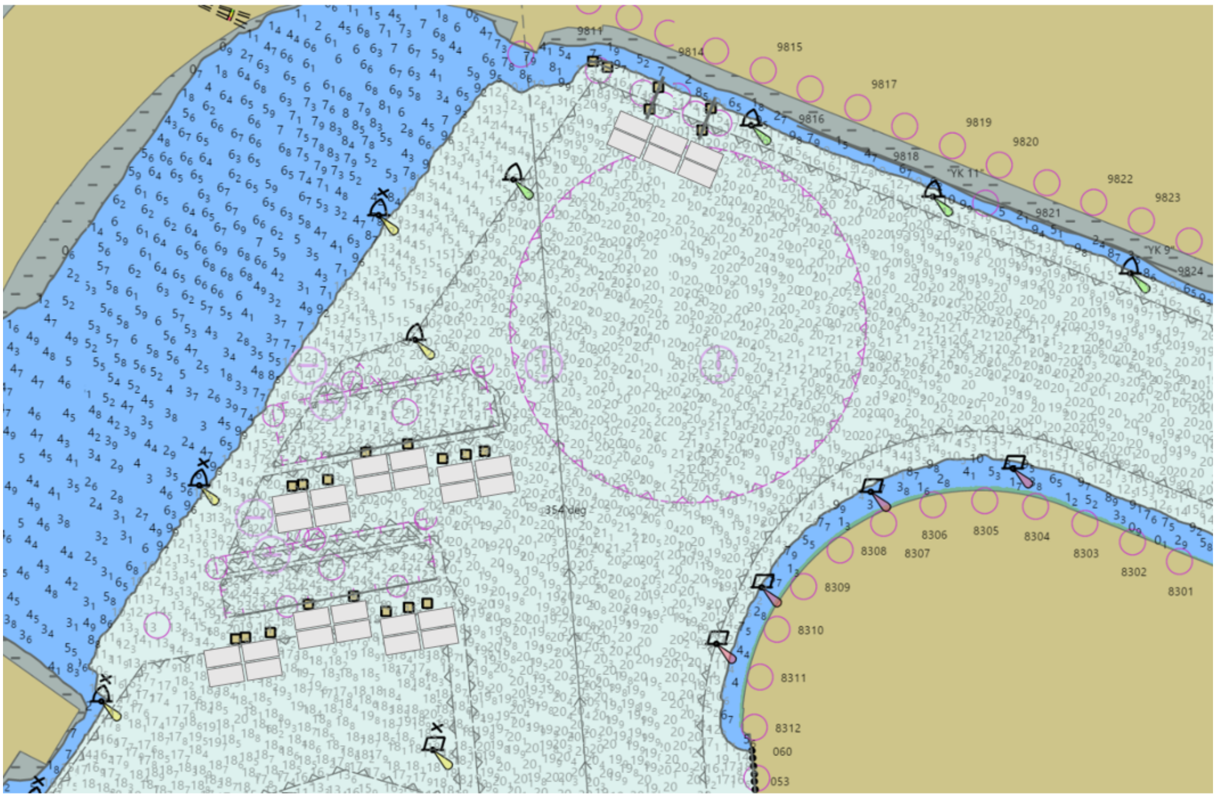


Figure E.12: Intermediate wet storage in the Port of Rotterdam dedicated to caisson units, which functions as a buffer when installation works exceeds the production rate of the caisson units.

Limiting wave conditions

Multiple papers were found that have investigated the workability limitations with regard to the installation process. The first study was conducted by Lin, El Chahal, and Shao (2020), in which caissons were used to construct a breakwater for a LNG and bulk terminal. Numerical computation was used to find the limiting wave conditions for caisson installing operations at larger water depths of 30 - 35 m for a confidential project along the African coast. In this study three types of caissons were considered (30 x 30 m (length x width) with a height ranging from 22 m to 32 m). Despite the quantitative results of the limiting wave conditions, the assessed caisson designs did not match with the caissons used in this project, and hence the limiting conditions could be not representative.

The second research was a PhD dedicated to the installation of caisson breakwaters under wave action, conducted by Alderson (2015). Despite the extensive and comprehensive analysis of both the physical and numerical modelling, also this study considered different dimensioned caissons. Although these limiting wave conditions could function as a indicative value, the uncertainty would be too large.

Lastly, a case study about the construction of a breakwater at Costa Azul, Mexico has many similarities with this project (Hibbs et al., 2010). An offshore breakwater at Sempra LNG's Energia Costa Azul terminal in Baja California Mexico consists of 12 reinforced concrete caissons, with a total length of approximately 700 m. The caissons must be installed in a water depth of 25 m, on a low rubble mound foundation pad. Although the caisson's shape is different than in the design for the PSH plant (for the breakwater a L-shaped caisson has been used), the dimensions are roughly the same: 68.7 m x 38.6 m x 27 m (length x width x height).

From this analysis, weather windows prevail that determine whether the caisson installation could continue or not. These weather windows are periods of 48 hours in which transportation to site, positioning and installation of the caisson unit must occur, but also allows for a safe rendezvous of the caisson unit to the nearby port in case when the weather becomes too extreme. For this research the same weather conditions are applied, however a shorter window of 12 hours is used, since it is assumed that the caissons will be anchored temporarily at a sea anchorage on the project site.

The caisson installation at Costa Azul was different than the proposed work method of this research. In the project, the caisson were L-shaped and were installed in two stages: i). with precised ballasting the caisson were installed inclined on some distance from the desired location, after touching the foundation layer with end of the caisson the opposite ballast chambers were filled to entirely sink the caisson and ii). the caisson will be re-floated, after which it will be positioned at the exact desired location and ballasted again with water. This installation method was beneficial under the local circumstances, and allowed the installation to occur even during unfavorable conditions.

Despite the differences in installation procedure, the same installation thresholds will be used for this project. The corresponding installation limits are shown in Figure E.13, and follow directly from the case study executed for Costa Azul (Hibbs et al., 2010). In order to account in some way for the difference in installation technique, the more strict limiting wave conditions will be used (left side of Figure E.13).

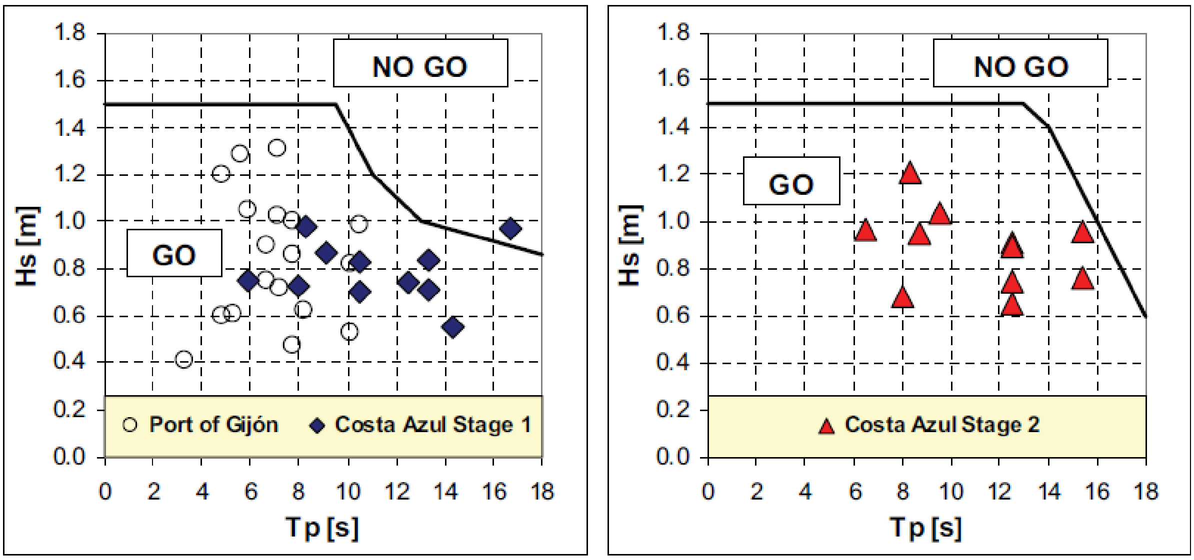


Figure E.13: Limiting wave conditions for the caisson installation. Source: Hibbs et al. (2010)

E.5. Scour protection

The scour protection layers will be placed after the caisson installation, and consists of many rock layers ranging from 45 - 180 mm (filter layer) to 3 - 6 ton (armour rock). The installation of the scour protection layers is an ongoing activity, and will proceed parallel to the caisson installation.

General description

The scour protection and toe consists of multiple layers with different rock gradings, for which different vessels are required. Besides the FPV as described in Section E.3, a Side Stone Dumping Vessel (SSDV) and multi-purpose barge will also be deployed for the more heavy gradings. The following layers and corresponding gradings will be installed:

- Scour filter: 60 - 300 kg
- Rock filter (A): 300 - 1000 kg
- Scour protection: 3 - 6 ton
- Toe filter 1: 45 - 180 mm
- Toe filter 2: 60 - 300 kg
- Toe protection: 300 - 1000 kg

Dedicated equipment

For the other layers that are installed after the rock foundation and scour A, SSDVs will be used with the option to deploy an additional crane when controlled placement is required. For this activity the Ndeavor of Boskalis, shown in Figure E.14, will be used. The dynamic positioned Ndeavor is equipped with eight stone dumping departments with a load capacity 625 ton per department, resulting in a total loading capacity of 5000 ton. The dump process of the loading departments is controlled by hydraulic sliding systems. Considering the safe working conditions, the SSDV will only install the scour protection until a safe working distance from the caissons, after which a crane must be used.

The SSDV Ndeavor with an onboard excavator will install the 60 - 300 kg and the 3 - 6 T armour rock units at the sea side of the caisson dam. Considering the offshore conditions, that applies for both sides of the caisson dam, the SSDV will also be deployed due to the manoeuvrability of the SSDV for the installation of the filter layer with 60 - 300 kg at the reservoir-side. According to CIRIA et al. (2007) a typical SSDV has a production rate of 4000 ton/hour. Including the maneuver, turning and positioning time, the net production rate is estimated at between 500 and 1000 ton/hour.

Figure E.14: Dynamic Positioned (DP) Side Stone Dumping Vessel the Ndeavor.

Volume estimation

The required volumes of the scour and toe protections and the additional filter layers are calculated with AutoCAD 2023, and are shown in Table E.2.

Rock supply and loading operations

Similar to the rock supply and loading operations of the first rock foundation for the caissons, the rock material originates from multiple quarries which are now foreseen to be located in Norway. From Norway

Layer	Grading	Vessel	Volume/unit [m^3]	Total Mass [mT x 10^3]	Legend
Scour Filter (inner)	60 - 300 kg	SSDV	540	368	
Scour Filter	60 - 300 kg	SSDV	484	330	
Scour Filter	300 - 1000 kg	SSDV	479	326	
Scour protection	3 - 6 ton	SSDV	3257	2218	
Toe filter	45 - 180 mm	FPV	795	542	
Toe protection	300 - 1000 kg	SSDV	1001	682	

Table E.2: Volume estimations for the scour protection and filter layers.

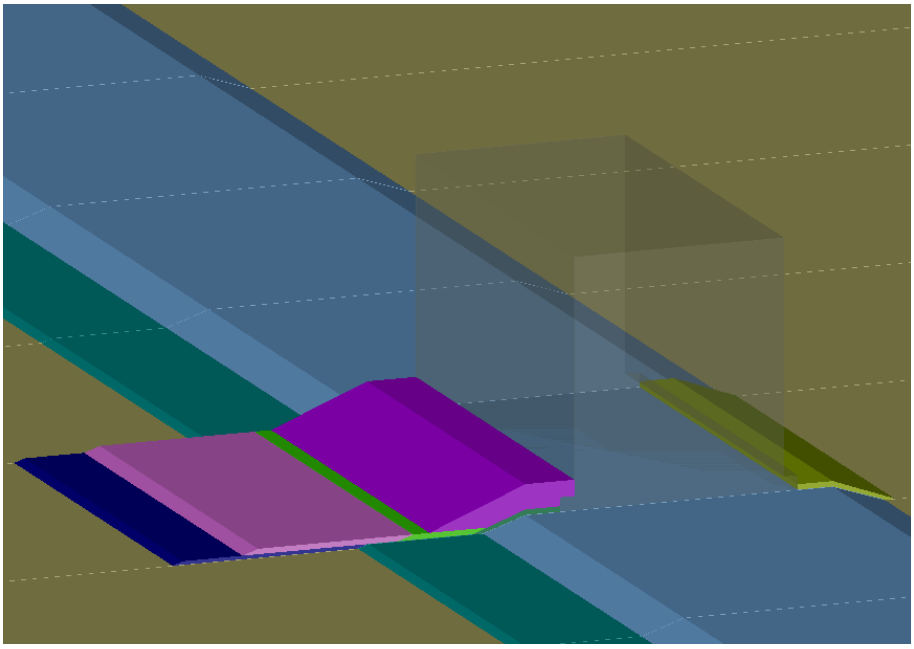


Figure E.15: Design of the caisson dam, highlighted one unit of the entire dam. Colours correspond to distinct layers, also mentioned in Table E.2.

a bulk carrier transports the rock material from the local quarry to the port of Rotterdam, or another port in the vicinity of the project site depending on the availability and capacity of the storage. The rock material will be loaded onto the Ndeavor moored at the quay wall by excavator and wheel loader.

E.6. Reclamation works

At the rear side of the caisson dam an inner berm is located, to provide sufficient horizontal stability for the caisson to absorb wave load impact. The inner berm of the reservoir consists of a gentle slope, constructed with material obtained during the sea bed preparation and a not yet appointed borrow area, but will be in the centre of the reservoir. The gentle slope will be chosen according to the natural slope of the dredged material to ensure minimization of scour during temporarily construction periods when the berm is left unprotected. A natural slope for sandy material will be approximately 1:14.

Considering possible soil failure mechanisms, such as liquefaction or macro stability problems, the inner berm must be slowly constructed to release excess pore pressures gently over time. To that end, the inner berm will be divided into several smaller layers of roughly 2-3 m which will be constructed evenly over time to ensure sufficient consolidation time. In addition, according to the stability checks in Section 3 it turns out a minimum berm height of 3 m is required to prevent damage during winter seasons, and therefore should be constructed before the end of the working season.

Another important factor to take into account is the fact that due to the gentle slope the size of the berm will be in the order of 200 m. Starting with the installation of the first berm the sand will spatially spread out naturally, creating a circle/pyramid shape around the first point of installation. The distribution of sand will be both in the longitudinal and cross-sectional direction. To prevent sand from reaching the to be constructed caisson unit, installation of the berm can only commence if the sufficient units have

Figure E.16: Sailing routes per activity and dedicated vessel.

been finished. In other words, the inner berm can not be constructed for a single caisson unit since the applied sand will literally block further construction works, i.e. installation of the next caisson unit. So, the construction of the inner berm will commence when several caisson units (5-6) have been installed including the additional rock works.

After 5 caisson units have been finished, the installation of the inner berm can slowly start. Considering the length of one caisson (60 m), the distance from the centre of 5 caissons to the end of the caissons is 150 m, meaning that in principle the inner berm can be constructed until a height of 15 m (slope 1:14). The exact design, soil properties and failure mechanisms, scour during the off-season and exact installation process is considered to be outside the scope of this research. Only the placement and duration has to be taken into account with regard to the planning.

The dimension of the inner berm in respect to one caisson until are roughly 30 m high, 60 m wide and 400 m long, assuming a slope of 1:14. According to the volume calculations of AutoCAD the volume of the inner berm per caisson unit is hence $2.72 \times 10^5\ \text{m}^3$, resulting in a total reclamation volume of $71.3 \times 10^6\ \text{m}^3$.

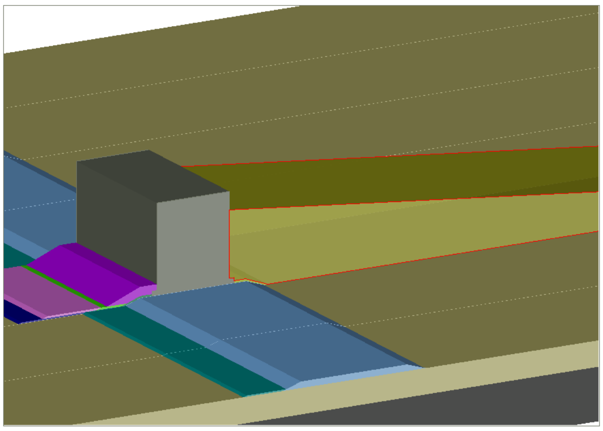


Figure E.17: Design of the caisson dam, highlighted in the figure is the inner berm.

E.7. Stone revetment

Finally, after the inner berm has been constructed and is stable, the stone revetment that will prevent erosion of the berm must be constructed. From Section 3 it follows that a stone grading of 40 - 200 kg is sufficient to withstand wind waves with a height of $H_s = 1.4$ m. For this construction phase offshore conditions do not previal anymore, as almost the entire reservoir has been closed by now. This allows also the deployment of non-offshore equipment at the reservoir, which is beneficial considering the construction costs.

Required filter function

Besides the sufficient stone grading, the entire system must also be 'sand tight', implying that the system should not allow sand to leave the system, for instance through large gaps between the stone revetment. In general, there are two options to disable the transport of sand through the revetment: i). geotextile or ii). geometrically closed system.

The first option is a relatively cheap and simple solution, by applying a geotextile underneath the revetment which pores are smaller than the grain sizes, water can pass the textile but it blocks the sand grains. Alternatively, a geometrically closed grading can be applied, consisting of multiple stone gradings. However, considering the additional costs of an extra filter layer underneath the revetment, the geotextile option will be economically feasbile, and thus preferable for this project.

Installation of the geotextile

So, prior to the installation of the stone revetment, the geotextile will be installed on the inner berm. Considering the fact that the geotextile must be installed both above and below the water line, two types of geotextiles will be used. For the area above the water a single layer non-woven geotextile will be applied, which can be easily rolled out over the surface, after which the 40 - 200 kg grading can be installed directly on the geotextile.

For the area that lies below the water surface, the regular installation of geotextile can not be applied as the geotextile can not be placed with accuracy. To overcome this problem there are several options, among them are the classic fascine mattress that can be used to install the geotextile, or alternatively a special designed and constructed vessel can roll out the geotextile over the sea bed.

The fascine mattress is a historical method of the installation of geotextiles in The Netherlands. With twig branches (or other types of wood such as willow) large squares are made, that are tied together to form a large mattress. Then, a geocomposite (consisting of a non-woven and a woven geotextile) will be attached underneath this mattress. The mattress, with geotextile, is then pulled onto the water, and is towed to the desired location. Then a sink layer is placed on top of the fascine mattress, which will sink the entire mattress. Once placed on the sea bed, the final protection layer will be installed with cranes or vessels.

Although this installation technique has been applied for many years, and is nowadays still a very popular method to install a geotextile below the water surface, it also is a very labor intensive work. Moreover, considering the enormous area that must be covered with geotextile (the entire inner berm, 16 800 m^2) a different technique might be beneficial.

The Cardium is a specially designed and constructed vessel, dedicated to the installation of the bed protection for the Easternschelde, consisting of a block mattress. This vessel is a large pontoon which contains a pre-fabricated roll of bed protection. After the vessel is in place, the roll with bed protection is simply unrolled, allowing precise installation. In Figure E.18, the Cardium that has been used for the Easternschelde, is shown.

Although the Cardium installed the entire bed protection (a block mattress that contains both the filter function while also retaining the high flow velocities), there was no additional rock works to install on top of this block mattress. Contrary to this project, the idea is to unroll the geotextile and place it on the sea bed, after which the final bank protection can be installed. Therefore, a less sophisticated vessel has to be designed and constructed, the vessel must be able to position at the desired location and unroll the geotextile. For instance, Figure E.19 shows a type of equipment that is less sophisticated but will be very useful.

Figure E.18: Specially designed vessel Cardium which has been deployed for the installation of the bed proection for the Easternschelde.

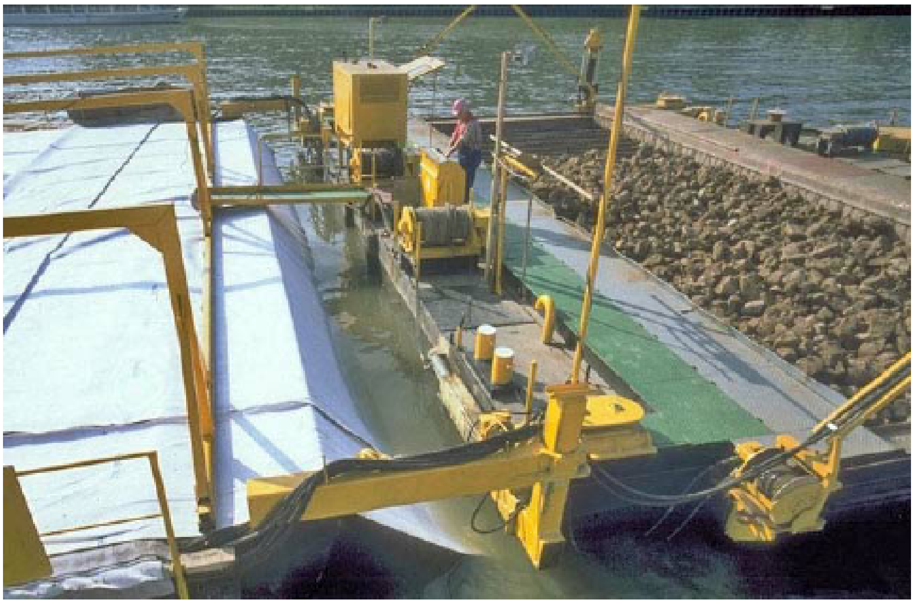


Figure E.19: Geotextile filter placement under water with dedicated equipment. Source: Heibaum (2008)

Comparing the fascine mattresses and the dedicated roll equipment, the latter is preferred for the geotextile installation, although it is not sure whether such equipment already exists that can be used for a project of this scale. Especially due to the extra logistics that are associated with the twigs, the many workers that are involved and the relatively slow installation speed, the application of fascine mattresses is not preferable.

Logistics
Considering the fact that the reservoir will be closed at some point, this hinders vessel activities executed inside the reservoir. This includes the installation of the stone revetment and scour protection but also potential future repair works on the inner berm, revetment, powerhouses or turbines. It is assumed that only the last part of the stone revetment on the inner berm will experience hindrance from the total closure, but the largest part of the revetment can be installed without this hindrance.

The ideal situation is when a ship lock will be constructed in the caisson dam, allowing vessels to enter the reservoir when necessary. However, this will substantially increase the construction costs, and therefore is not preferable.

An alternative solution is to construction on both sides of the dam a small work harbor, which is schematically shown in Figure E.20. On the reservoir side some auxiliary equipment can be deployed, such as a simple barge, crane and multi-purpose vessel, and will be mobilized for the rest of the structure's lifetime. This equipment will be responsible for the maintenance and repair works that will emerge in the future, for instance for some basic dredging activities.

On the ocean side of the dam, a small work harbor will be necessary for the transfer of crews during the construction of the dam, but also when the plant is in operation and the loading and unloading of necessities dedicated to the hydropower plant. Moreover, this harbor can be used for the installation of the last part of the stone revetment on the inner berm and the additional scour protection of the powerhouses.

A bulk carrier will unload the stone gradings on the quay wall, and transported to either a storage area or loaded directly onto the barge on the reservoir side. Then, the barge with crane will sail to the desired location and install the rock works.

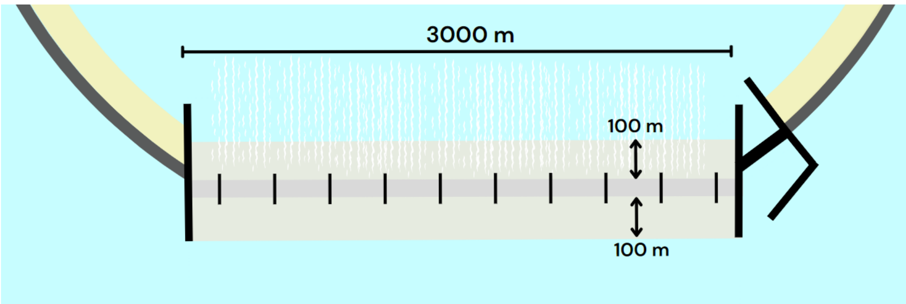


Figure E.20: Schematic overview of the powerhouse section including the work harbor on the right side.

E.8. Conclusion

The aim of this section was to determine the entire work method, starting from the preparation works like the caisson fabrication, until the installation of the inner berm. This enables the composition of a simple deterministic planning, but also already indicates what steps might be crucial during the project and what kind of logistic hurdles must be overcome in order to successfully execute this project.

Examples of logistics hurdles that follow directly from this assessment is the intermediate wet storage of the caisson units, but also the transportation and temporarily storage for the rock works. To overcome these problems, a preliminary solution has been provided, but is still in a low-level of detail.

F

Visualisation work method

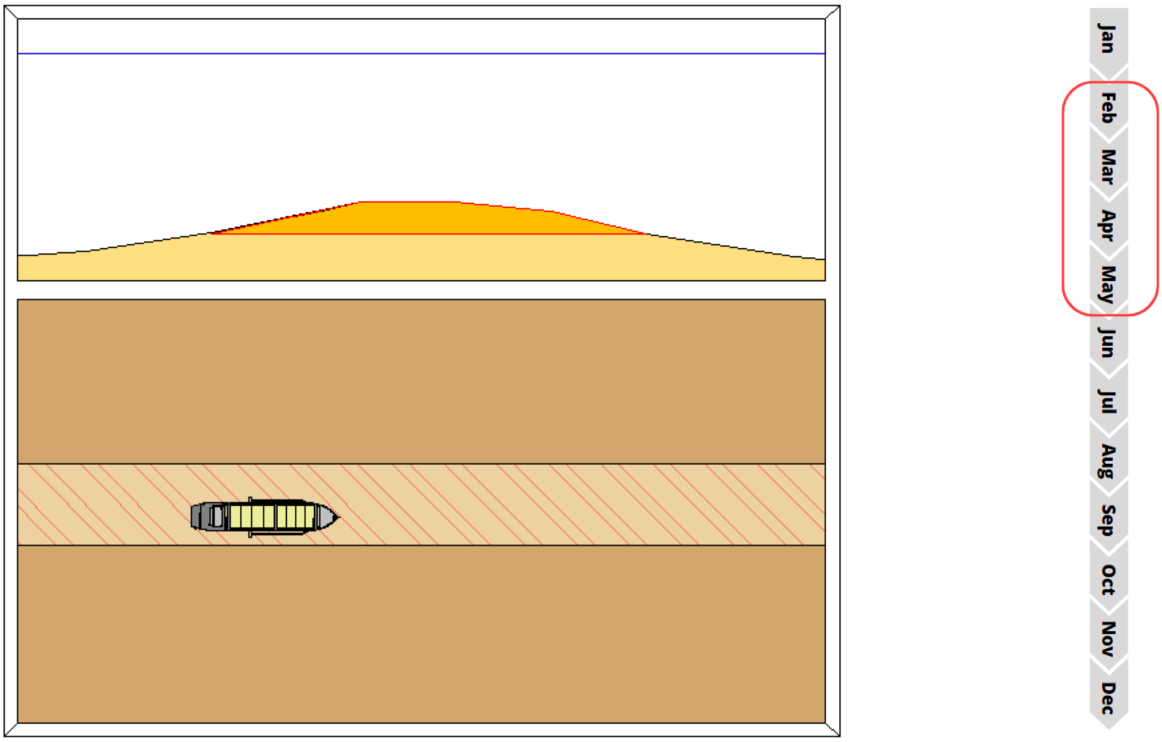
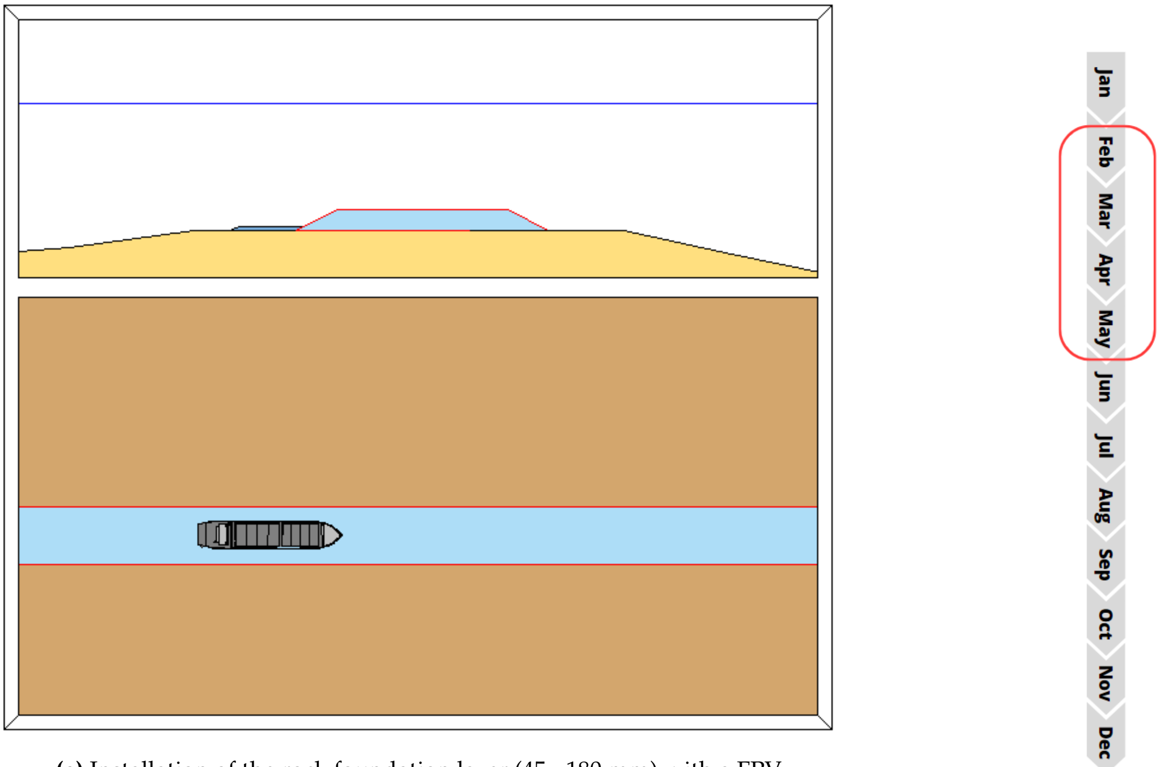
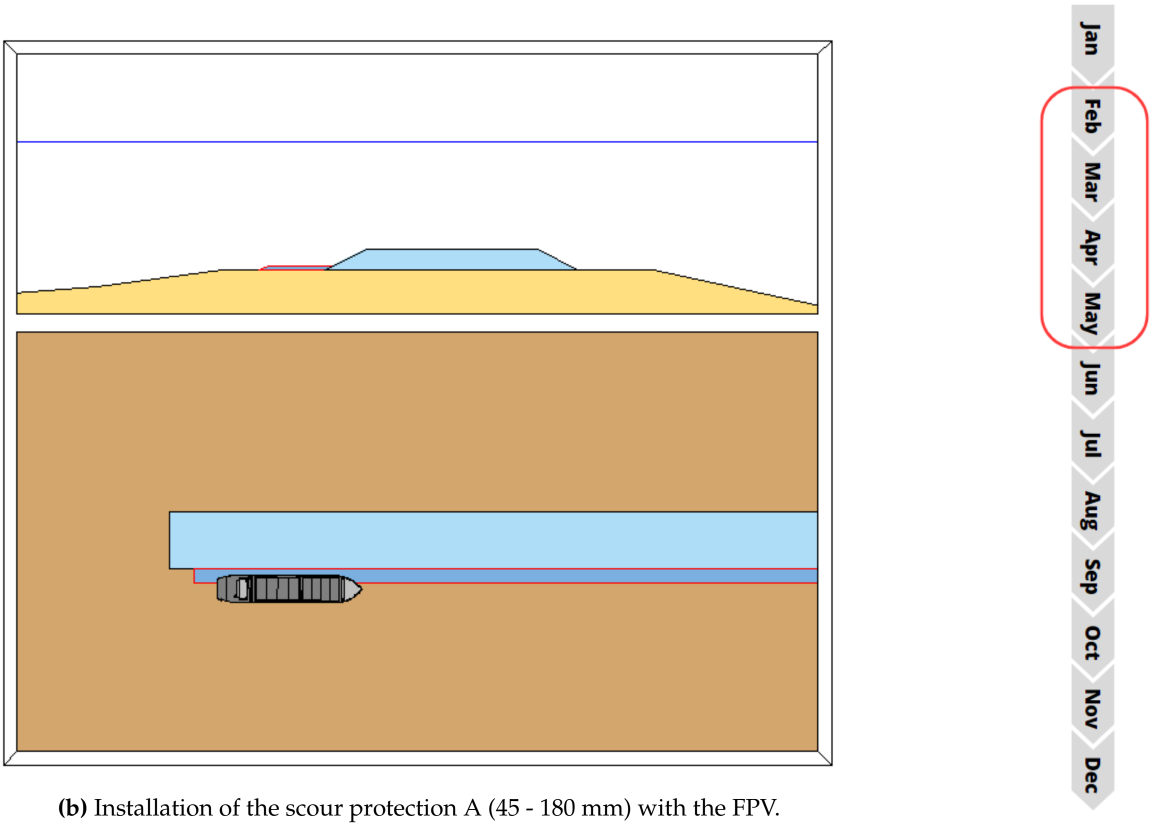


Figure F.1: Leveling dredging or sea bed preparation with a TSHD prior to the installation of the foundation layer. Orange highlighted area will be removed by the TSHD.

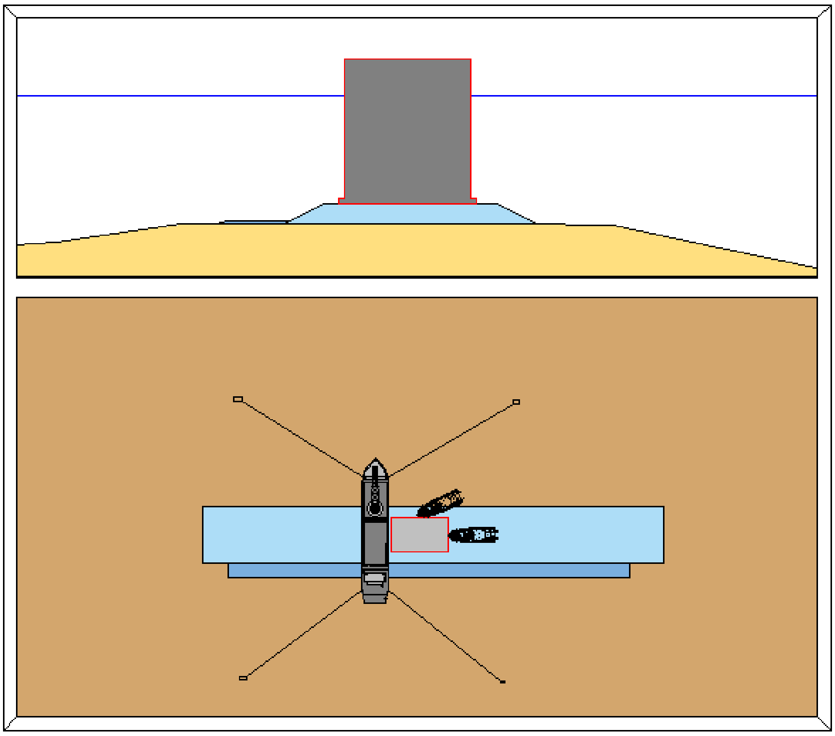


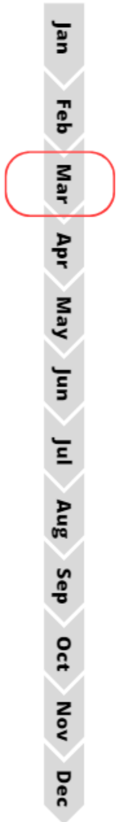
(a) Installation of the rock foundation layer (45 - 180 mm) with a FPV.



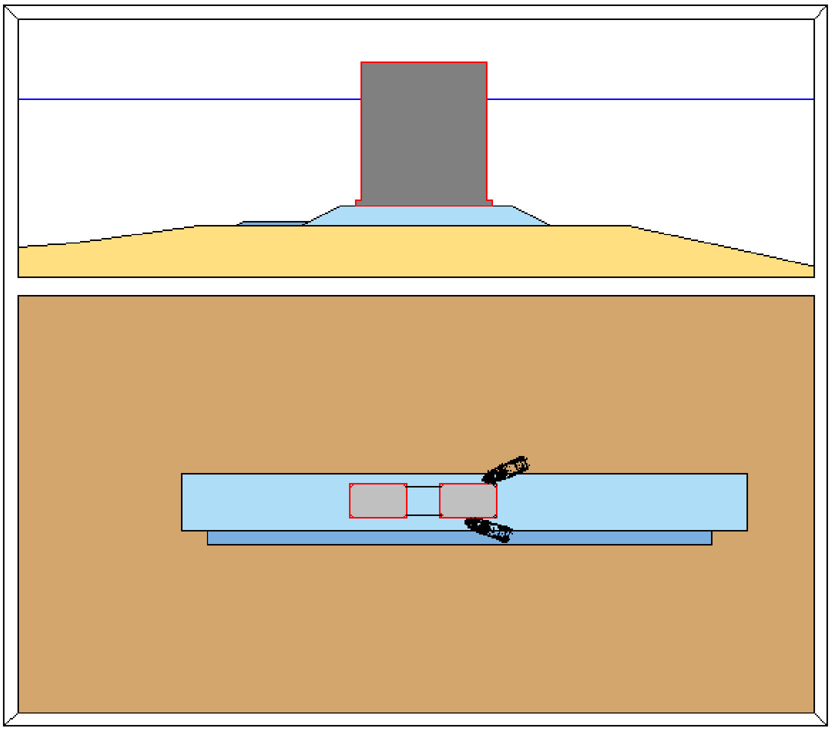
(b) Installation of the scour protection A (45 - 180 mm) with the FPV.

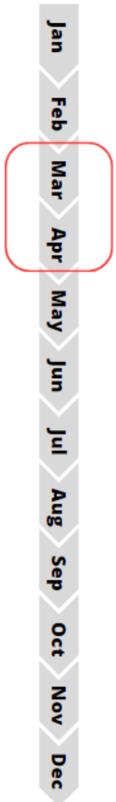
Figure F.2





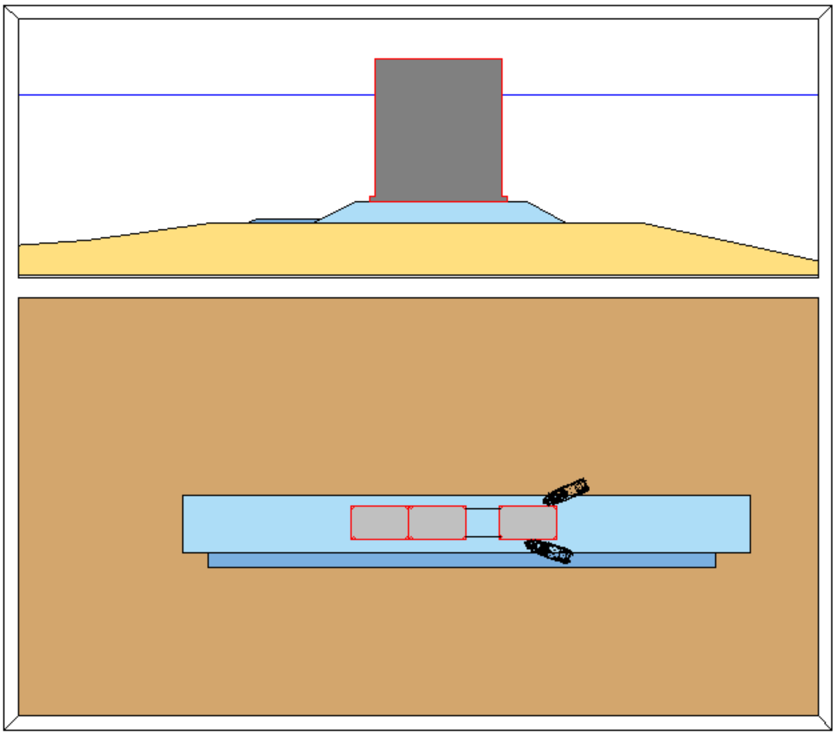
(a) Placement of the first caisson, positioned by two push tugs onto the HLB until the desired position has been reached, after which the ballasting process will commence.

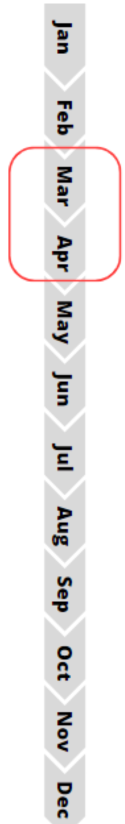




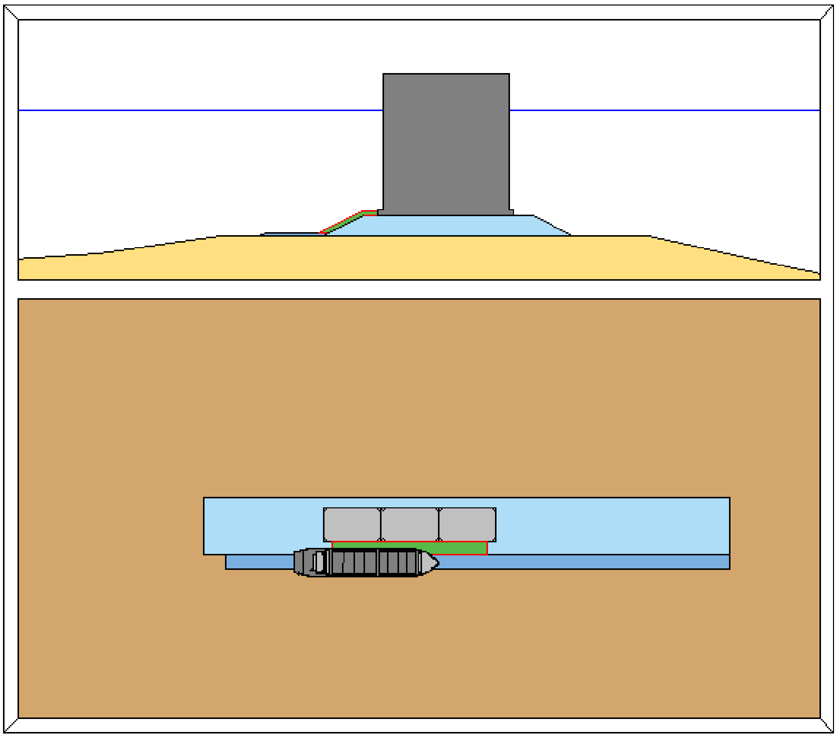
(b) Installation of the following caissons only requires two push tugs, and two guiding winches from the installed caisson which pulls the caisson towards it.

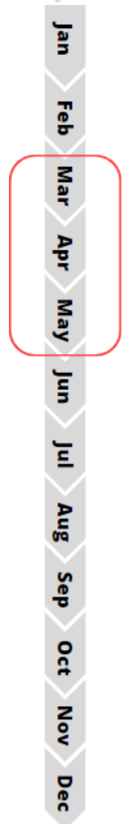
Figure F.3





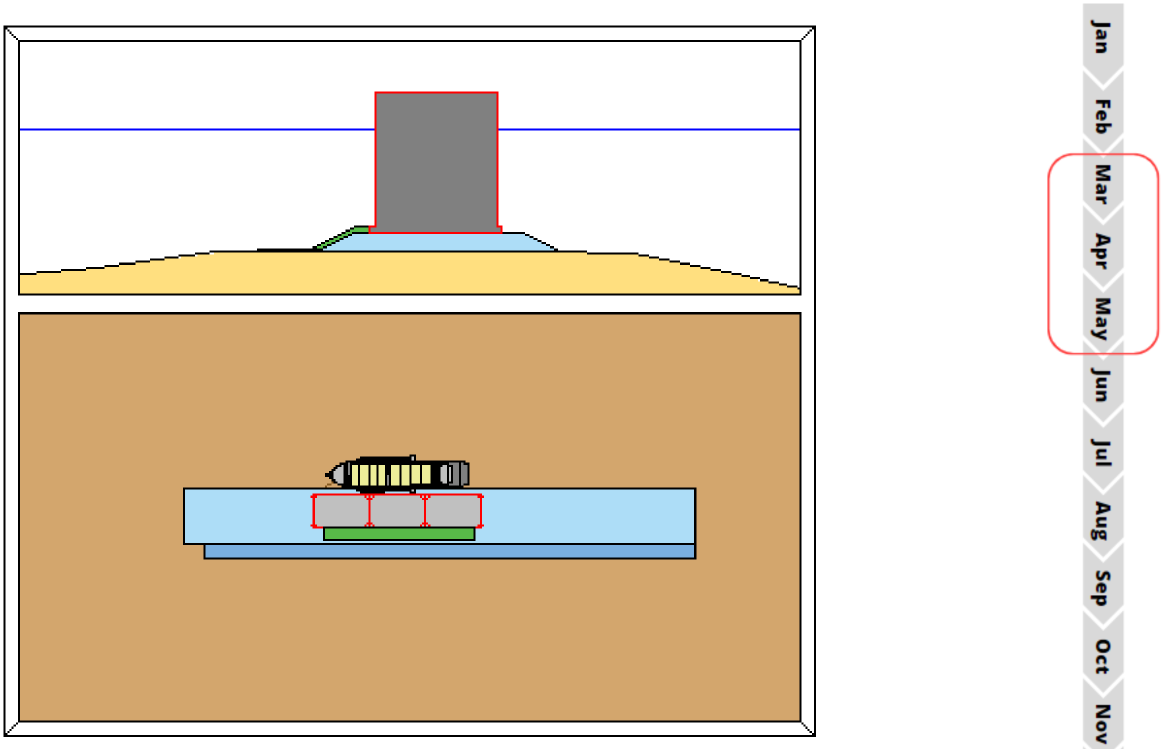
(a) Installation of the next caisson will happen in the same manner as previous described.



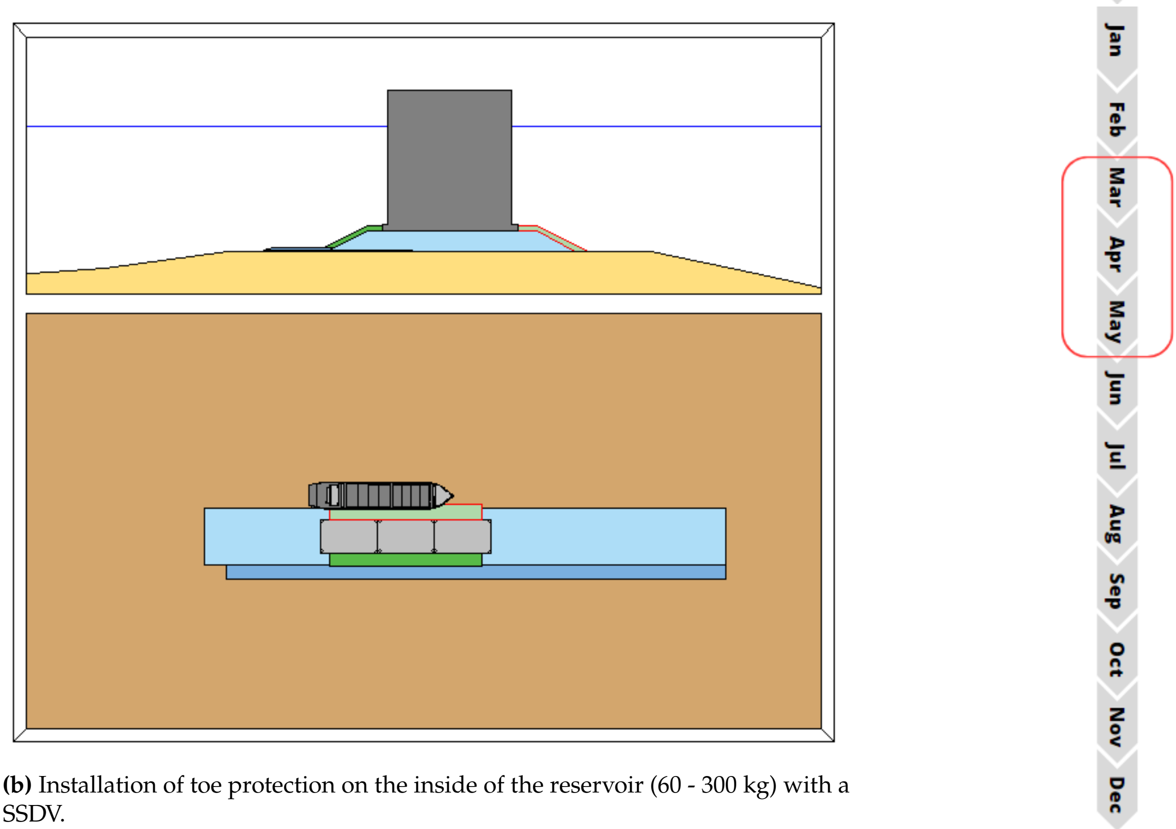


(b) Installation of the filter layer for the toe protection (60 - 300 kg) with a SSDV.

Figure F.4

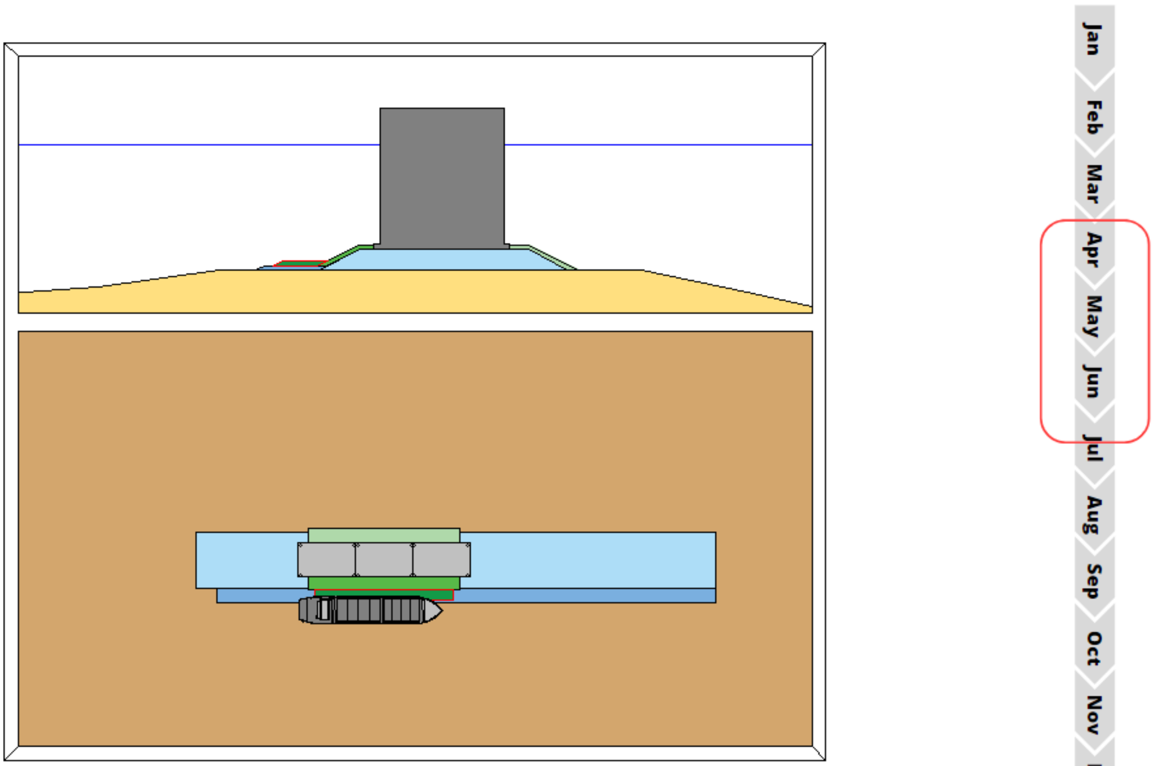


(a) Caisson unit filling with dredged material from adjacent sea bed preparation areas with a TSHD.

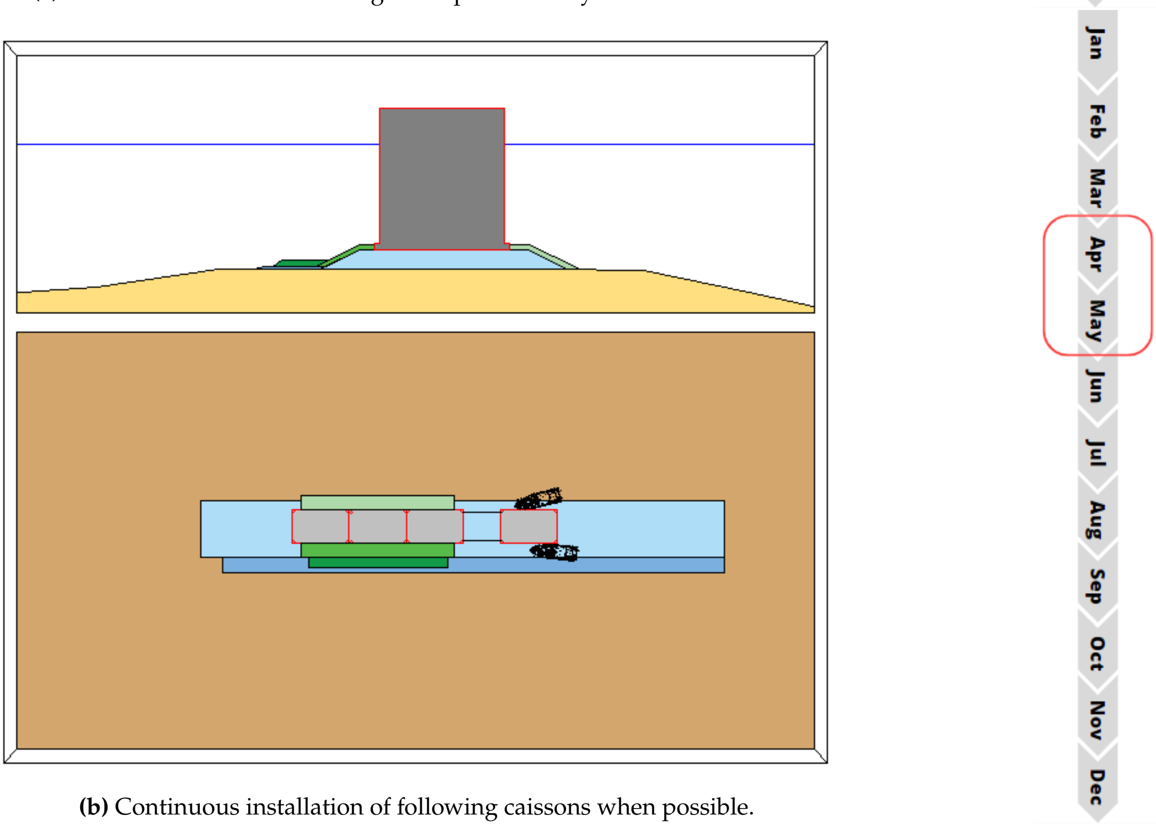


(b) Installation of toe protection on the inside of the reservoir (60 - 300 kg) with a SSDV.

Figure F.5

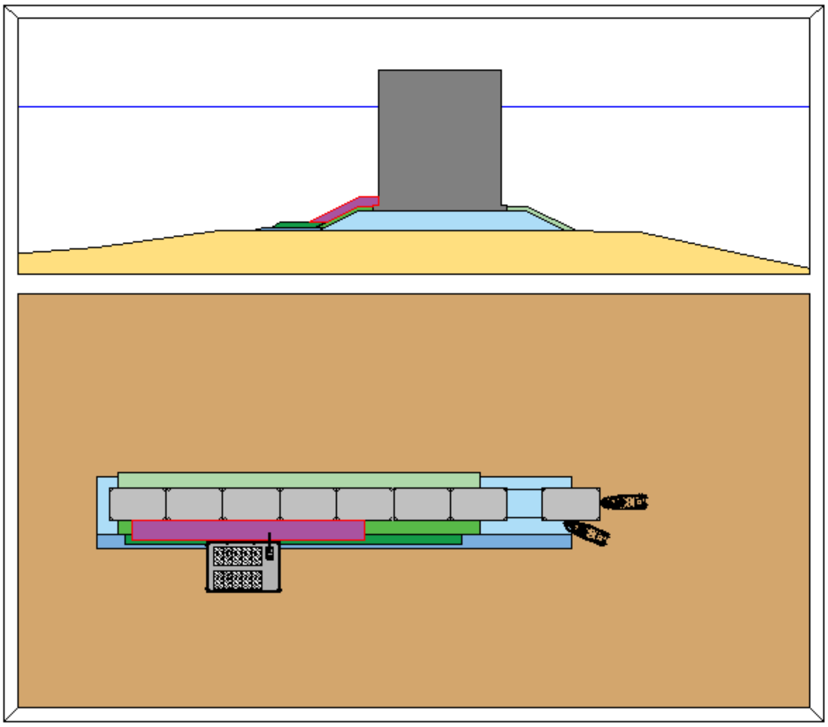


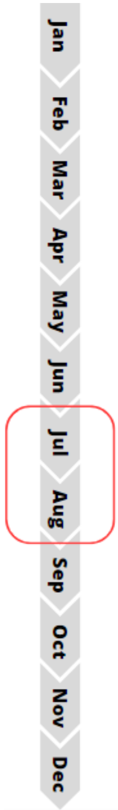
(a) Placement of the 300 - 1000 kg scour protection by a SSDV with crane.



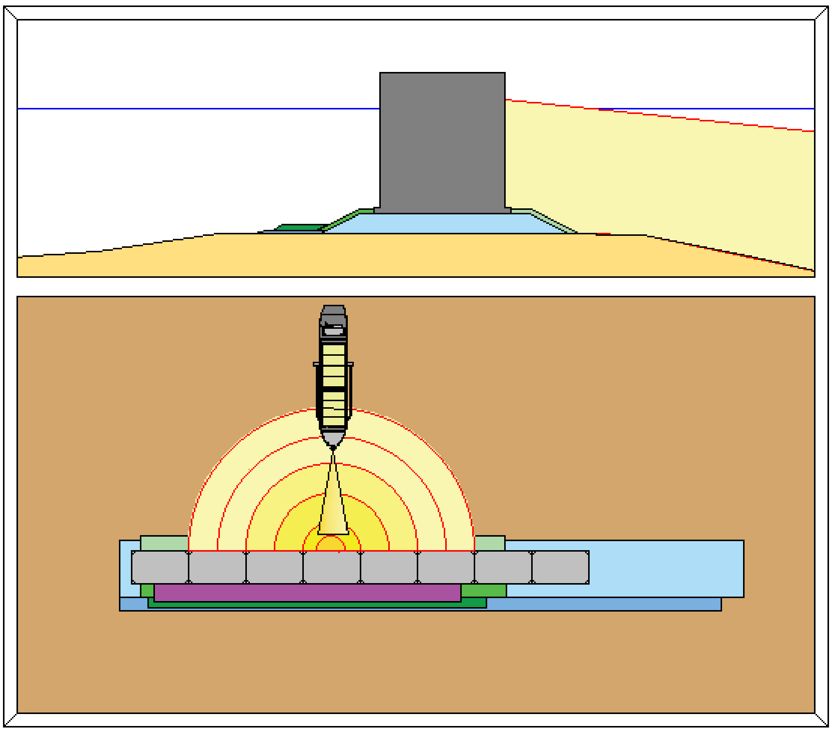
(b) Continuous installation of following caissons when possible.

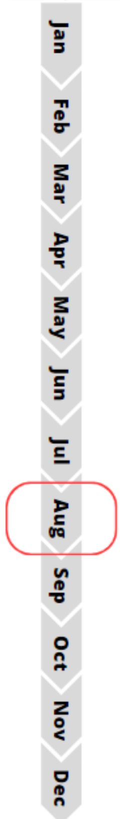
Figure F.6





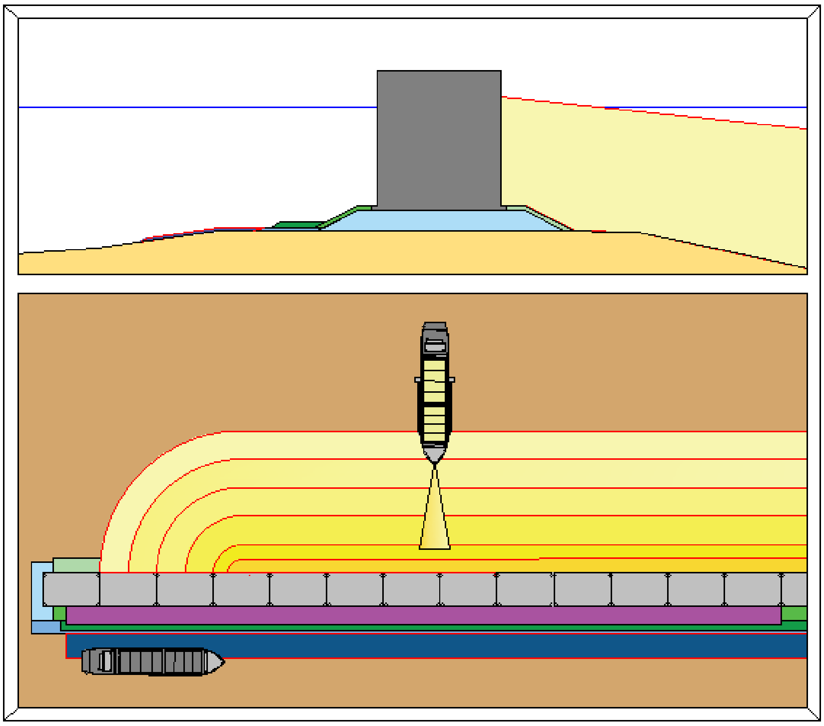
(a) Installation of the toe protection at the sea-side, armour rock of 3 - 6 T, executed by SSDV with crane.



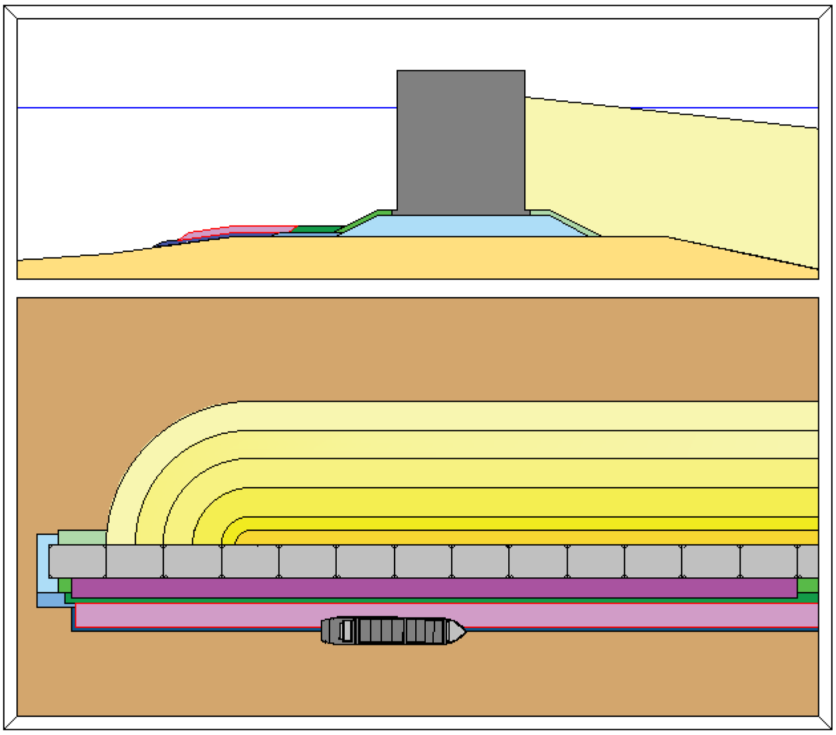


(b) Construction of inner berm by TSHD with dredged material obtained during the sea bed preparation or from the borrow area.

Figure F.7



(a) Continuous placement of dredged material at the inner-side of the caisson unit. At the seaside, SSDV will install a scour protection filter layer (45 - 180 mm).



(b) Installation of the toe protection layer 300 - 1000 kg with SSDV.

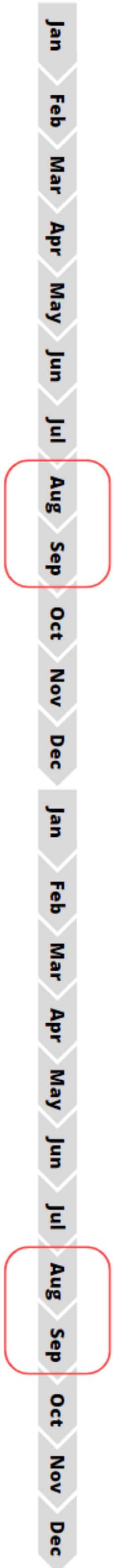


Figure F.8

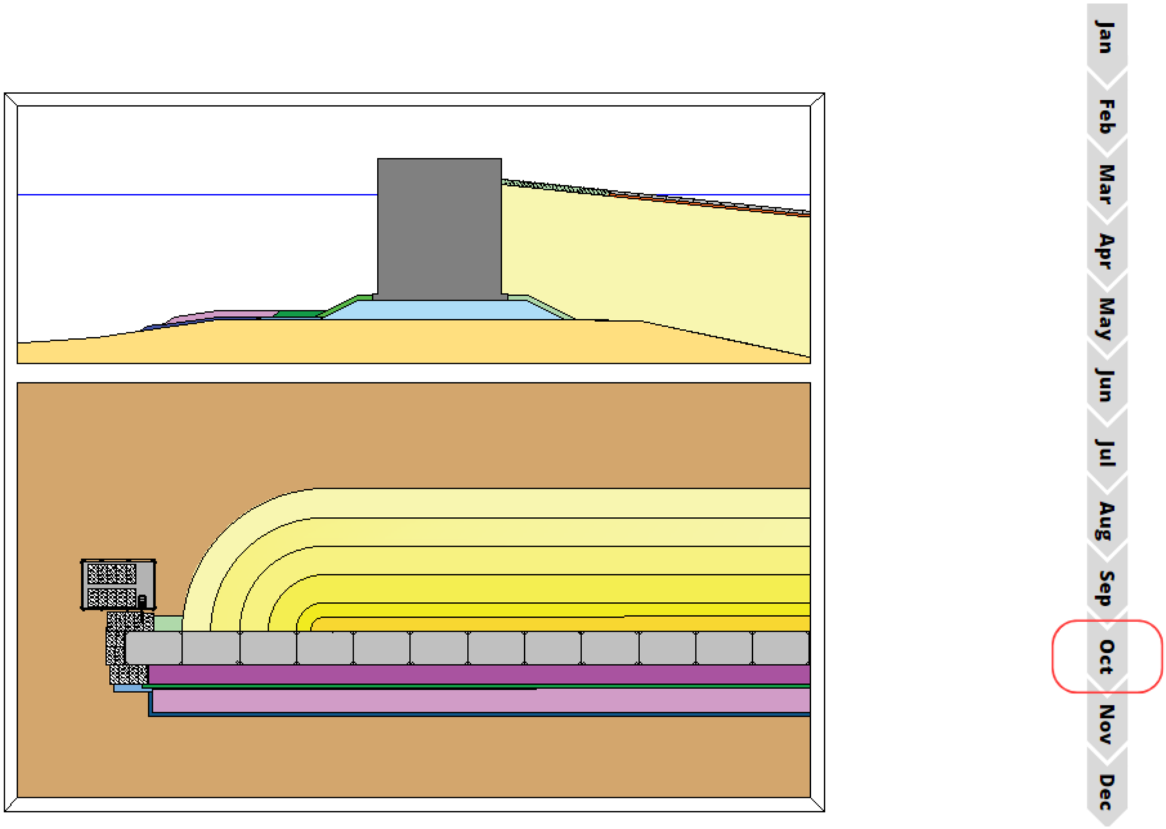


Figure F.9: Before the working season ends, placement of temporary scour protection units will prevent extensive scour around the unfinished parts of the caissons during the winter months.

G

Deterministic planning

G.1. Learning curve

The learning curve for a particular sequence includes the time reduction for the next phase of the same sequence, due to a gain in experience. It is a graphical representation of the relationship between the workers' efficiency and their amount of experience. Generally, an increase in experience will result in an increase in proficiency. For instance, after the first caisson has been placed the execution team has gained experience and knowledge that they can use for the placement of the second caisson, for example the deployment of an extra tugboat will help to install the caisson faster or with more accuracy.

$$Y = aX^b \tag{G.1}$$

where:

Y = activity duration
a = activity duration for the first execution
X = denotes the total amount of executions completed
b = represents the slope of the function

It is assumed that the potential of the learning curve has been fully utilized after 5 repetitions, after which the sequence duration will stay constant. The development of the additional time on top of the baseline duration will gradually decrease, as the experience increases and hence the learning curve reaches its potential. The learning curve parameters - activity duration for the first execution and the slope of the function - are listed per construction sequence in Table G.1. The effect and development of the learning curve has been visualised qualitatively in Figure G.1.

Activity	First activity duration	Slope	Duration after 5th execution
Preparation	18 hours	-0.25	8 hours
Towage	22	-0.3	4
Position caisson unit	12	-0.5	6
Ballasting	8	-0.15	6
Filling	5	-0.4	3
Joint construction	12	-0.15	10

Table G.1: Learning curve parameters for various construction activities. Activity duration is given in hours.

G.2. Simultaneous operations (SimOps)

Simultaneous Operations - also known as SimOps - during construction works involve multiple activities concurrently, executed at the same or near the same location. Often designers and engineers working on the activities during the design phase are not fully aware of the allocation, safety and coordination of the operations. During the preparation phase, the execution team of the deployed vessels - who know their capabilities, limitations and limits better than anyone else - may encounter situations which they consider to be unsafe, unrealistic or unfeasible. Subsequently, an alternative or solution must be found to cope with this simultaneous operation, often implying an allocation of resources, disruption of other activities or overall delay of the project.

Therefore, proper planning, communication and risk management are essential to ensure that simultaneous operations are executed safely and efficiently. To that end, during the planning phase the critical path

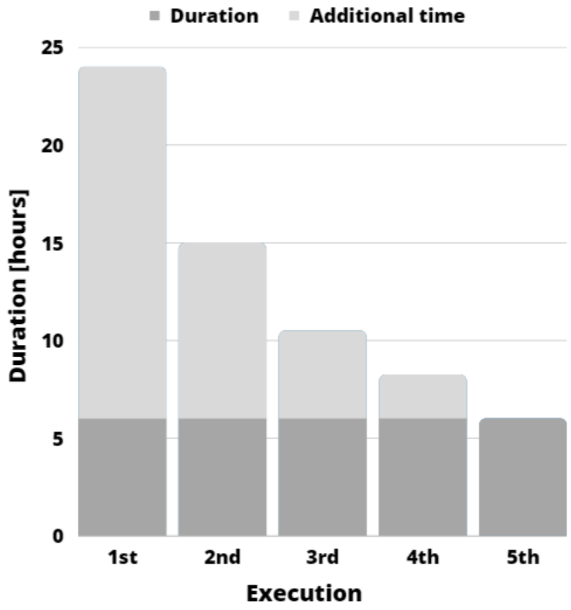


Figure G.1: Qualitative graph of the construction duration for five repetitions, indicating the effect and development of the learning curve.

of the project will be assessed, identifying potential conflicts, dependencies and allocate resources and activities accordingly.

In order to prevent the occurrence of SimOps, the locations of the construction works are also defined in the planning, thereby forcing an allocation of construction works in case their locations interfere. For example, the placement of the second caisson will block all further construction works at the first and second caisson, preventing also the (rock) works executed at the latter caisson.

G.3. Prioritization

The prioritization of construction works is critical and essential to successful planning as it allows for the efficient use of time, resources and subsequently minimizing the budget. Prioritizing construction works ensures that the most critical and essential construction steps will be executed first, increasing the amount of the remaining construction works that could be executed during less critical wave conditions in the case of an offshore structure.

In case of this project, the most critical sequence is the installation of the caisson unit, as this requires very mild wave conditions, i.e. wave heights in the order of 1.0 - 1.5 m, and is therefore considered to be the most sensitive task. To put this in perspective, the installation of the rock works is limited to a wave height in the order of 2.0 - 2.5 m. Therefore, the installation of the caisson unit will be the leading in the construction sequence, allocating other construction works when the installation can commence.

This entails that for the deterministic planning the rock works will lag several caisson behind, and only can commence when the construction location is not occupied. In other words, the rock works of the first caisson can start after the third caisson has been installed. Whereas in reality, the placement of the caisson will certainly not be continuously, resulting in a more even distribution of the installation of the caisson unit and the rock works, resulting in a planning shown in Figure G.2.

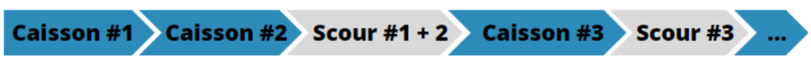


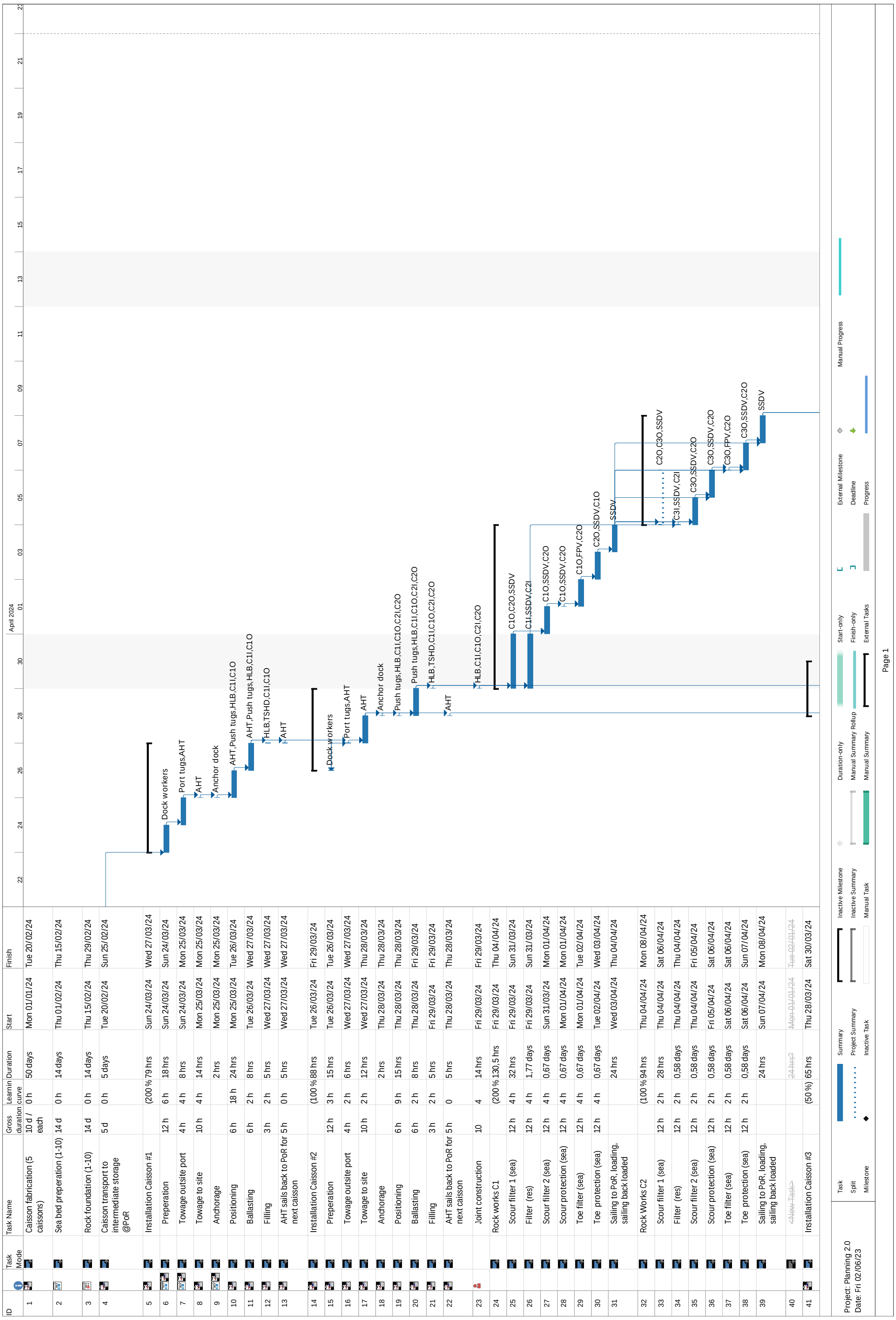
Figure G.2: Schematic planning of the construction sequences taking into account the difference in workability and wave conditions.

G.4. Activity duration

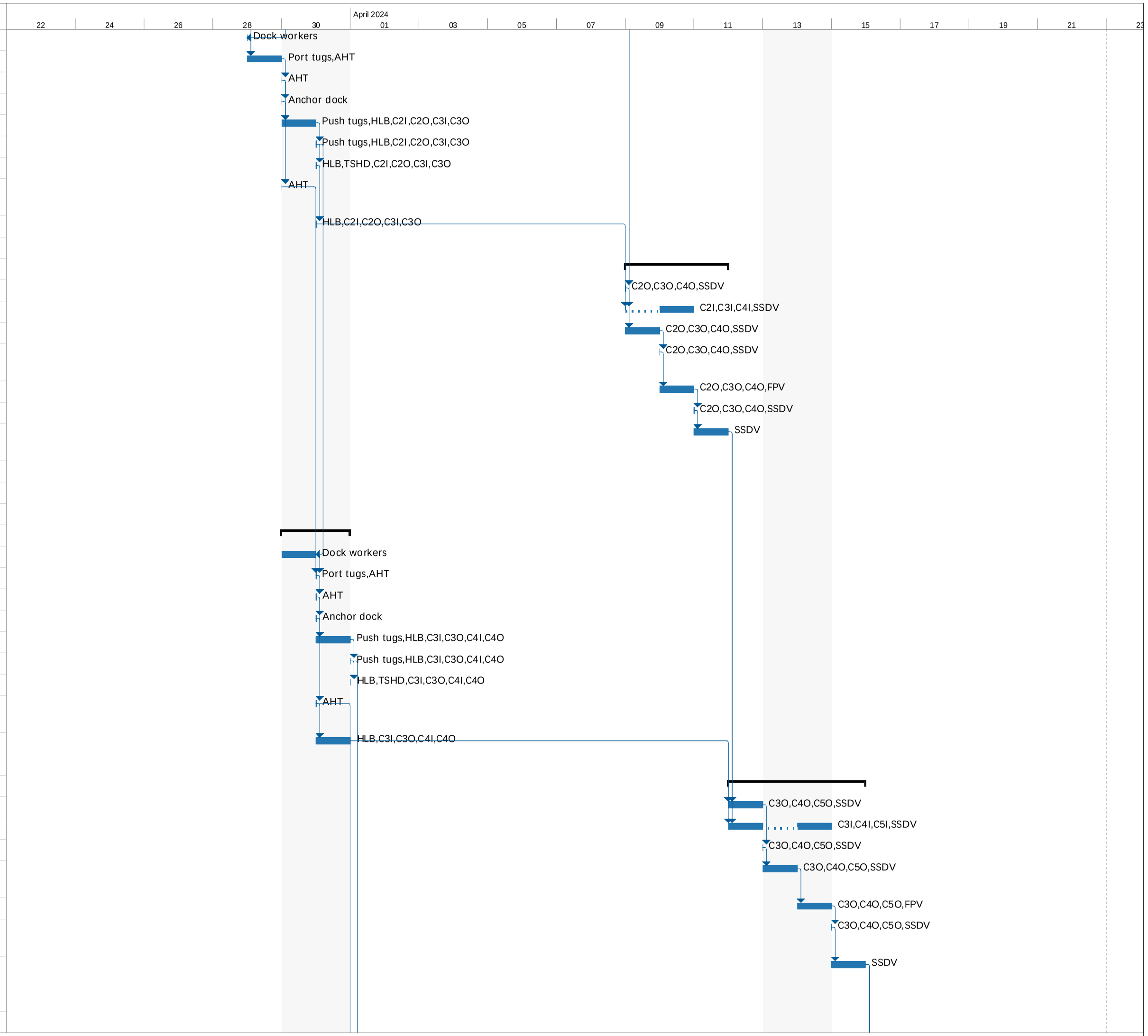
Phase	Activity	First activity duration	Description
Sea bed preparation	Leveling dredging	14 days	Leveling dredging for caisson unit 1 - 10
Rock works A	Installation rock layer	14 days	Installation of the rock foundation and scour protection A for caisson unit 1 - 10
Caisson transport	HTV Caisson transport	5 days	Caisson transport from fabrication facility to the temporarily storage at the Port of Rotterdam
Caisson stream	Preparation	8 hours	Preparation of the caisson unit, installation of navigation lights, winches, etc.
	Towage outside port	8 hours	AHT assisted with 2 port tugs tows the caisson outside harbour area
	Towage to site	14 hours	AHT tows the caisson to the project site
	Anchorage		Temporarily anchorage for the caisson in case installation is not possible immediately
	Positioning	12 hours	Positioning of the caisson with HLB and tugboats
	Ballasting	8 hours	Ballasting with water
	Filling	5 hours	Filling the caisson unit with dredged material to increase overall stability
	Joint construction	10 hours	Construction of the joint between two caisson
	AHT back to PoR	10 hours	AHT sails back to PoR, to prepare the transport of the next caisson
Rock works	Scour filter (seaside)	12 hours	Installation of the scour filter at the sea side
	Filter layer (reservoir)	10 hours	Installation of the filter layer at the reservoir side
	Scour filter 2	10 hours	Installation of the second scour filter, at the sea side
	Scour protection	10 hours	Installation of the scour protection at the sea side
	Toe filter	10 hours	Installation of the toe filter at the sea side
	Toe protection	10 hours	Installation of the toe protection at the sea side
	Loading SSDV at PoR	24 hours	SSDV sailing to Port of Rotterdam, loading and sailing back
Inner berm	Reclamation	-	Reclamation for the inner berm, discharging via the TSHD

Table G.2: Activity duration for the first construction sequences, including their learning curves.

G.5. Deterministic construction planning



ID	Task Mode	Task Name	Gross duration	Learning curve	Duration	Start	Finish
42		Preperation	12 h	1,5	13,5 hrs	Thu 28/03/24	Thu 28/03/24
43		Towage outside port	4 h	1	5 hrs	Thu 28/03/24	Fri 29/03/24
44		Towage to site	10 h	1	11 hrs	Fri 29/03/24	Fri 29/03/24
45		Anchorage			2 hrs	Fri 29/03/24	Fri 29/03/24
46		Positioning	6 h	4,5	10,5 hrs	Fri 29/03/24	Sat 30/03/24
47		Ballasting	6 h	1	7 hrs	Sat 30/03/24	Sat 30/03/24
48		Filling	3 h	1	4 hrs	Sat 30/03/24	Sat 30/03/24
49		AHT sails back to PoR for next caisson	5 h	0	5 hrs	Fri 29/03/24	Fri 29/03/24
50		Joint construction	10	2	12 hrs	Sat 30/03/24	Sat 30/03/24
51							
52		Rock Works C3		(50 %)	89 hrs	Mon 08/04/24	Thu 11/04/24
53		Scour filter 1 (sea)	12 h	1 h	13 hrs	Mon 08/04/24	Mon 08/04/24
54		Filter (res)	12 h	1 h	1,08 days	Mon 08/04/24	Wed 10/04/24
55		Scour filter 2 (sea)	12 h	1 h	0,54 days	Mon 08/04/24	Tue 09/04/24
56		Scour protection (sea)	12 h	1 h	0,54 days	Tue 09/04/24	Tue 09/04/24
57		Toe filter (sea)	12 h	1 h	0,54 days	Tue 09/04/24	Wed 10/04/24
58		Toe protection (sea)	12 h	1 h	0,54 days	Wed 10/04/24	Wed 10/04/24
59		Sailing to PoR, loading, sailing back loaded	24 h		24 hrs	Wed 10/04/24	Thu 11/04/24
60							
61							
62							
63		Installation Caisson #4		(25 %)	48 hrs	Fri 29/03/24	Sun 31/03/24
64		Preperation	12 h	0,75	12,75 hrs	Fri 29/03/24	Sat 30/03/24
65		Towage outside port	4 h	0,5	4,5 hrs	Sat 30/03/24	Sat 30/03/24
66		Towage to site	10 h	0,5	10,5 hrs	Sat 30/03/24	Sat 30/03/24
67		Anchorage			2 hrs	Sat 30/03/24	Sat 30/03/24
68		Positioning	6 h	2,25	8,25 hrs	Sat 30/03/24	Sun 31/03/24
69		Ballasting	6 h	0,5	6,5 hrs	Sun 31/03/24	Sun 31/03/24
70		Filling	3 h	0,5	3,5 hrs	Sun 31/03/24	Sun 31/03/24
71		AHT sails back to PoR for next caisson	5 h	0	5 hrs	Sat 30/03/24	Sat 30/03/24
72		Joint construction	10	1	11 hrs	Sat 30/03/24	Sun 31/03/24
73							
74		Rock Works C4		(25%)	86,5 hrs	Thu 11/04/24	Mon 15/04/24
75		Scour filter 1 (sea)	12 h	0,5 h	12,5 hrs	Thu 11/04/24	Fri 12/04/24
76		Filter (res)	12 h	0,5 h	1,04 days	Thu 11/04/24	Sun 14/04/24
77		Scour filter 2 (sea)	12 h	0,5 h	0,52 days	Fri 12/04/24	Fri 12/04/24
78		Scour protection (sea)	12 h	0,5 h	0,52 days	Fri 12/04/24	Sat 13/04/24
79		Toe filter (sea)	12 h	0,5 h	0,52 days	Sat 13/04/24	Sun 14/04/24
80		Toe protection (sea)	12 h	0,5 h	0,52 days	Sun 14/04/24	Sun 14/04/24
81		Sailing to PoR, loading, sailing back loaded	24 h		24 hrs	Sun 14/04/24	Mon 15/04/24
82							

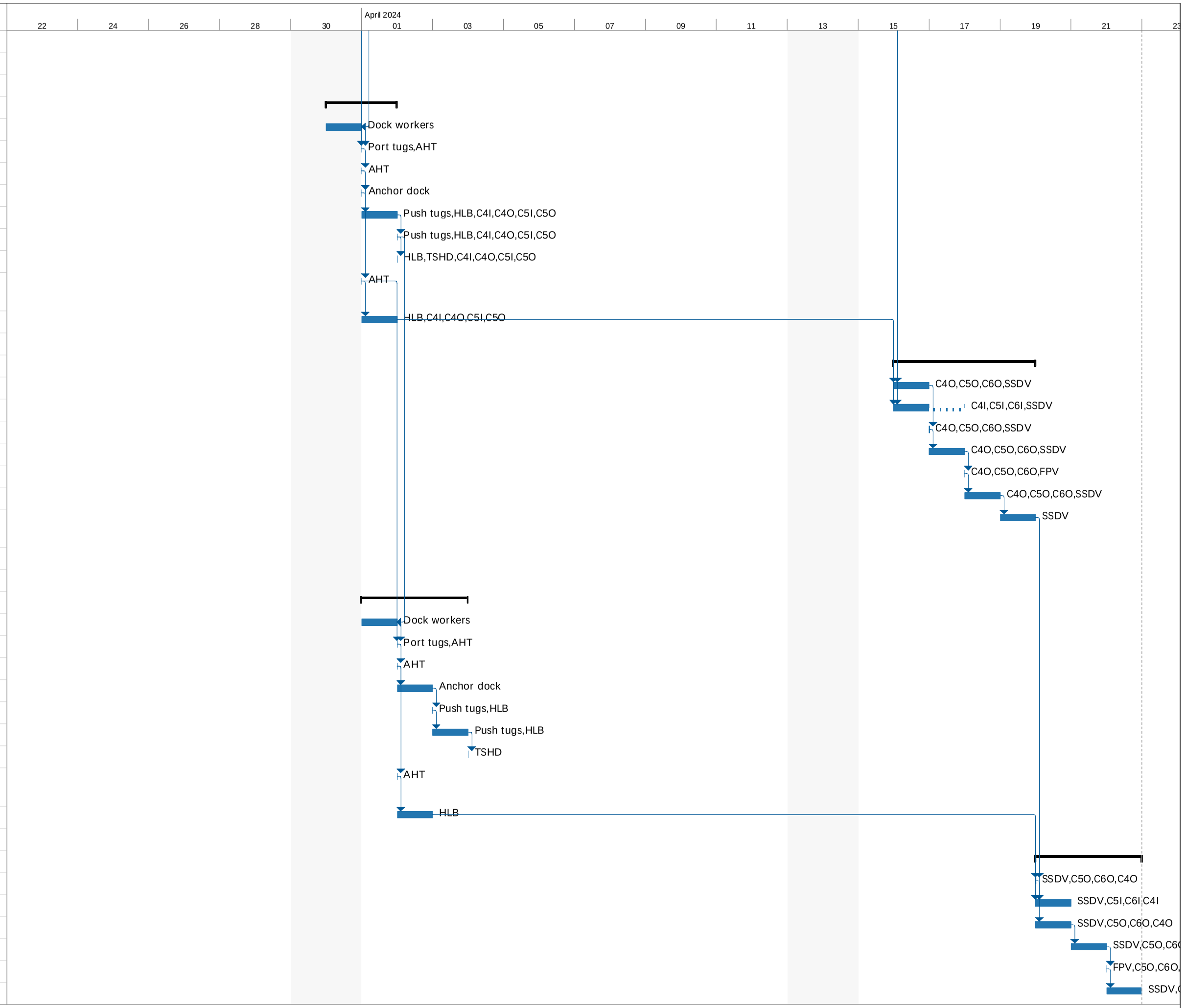


Project: Planning 2.0
Date: Fri 02/06/23

Task, Split, Milestone, Summary, Project Summary, Inactive Task, Inactive Milestone, Inactive Summary, Manual Task, Duration-only, Manual Summary Rollup, Manual Summary, Start-only, Finish-only, External Tasks, External Milestone, Deadline, Progress, Manual Progress

Page 2

ID	Task Mode	Task Name	Gross duration	Learning curve	Duration	Start	Finish
83							
84							
85							
86		Installation Caisson #5		(0 %)	43 hrs	Sat 30/03/24	Mon 01/04/24
87		Preperation	12 h	0	12 hrs	Sat 30/03/24	Sun 31/03/24
88		Towage outsite port	4 h	0	4 hrs	Sun 31/03/24	Sun 31/03/24
89		Towage to site	10 h	0	10 hrs	Sun 31/03/24	Sun 31/03/24
90		Anchorage			2 hrs	Sun 31/03/24	Sun 31/03/24
91		Positioning	6 h	0	6 hrs	Sun 31/03/24	Mon 01/04/24
92		Ballasting	6 h	0	6 hrs	Mon 01/04/24	Mon 01/04/24
93		Filling	3 h	0	3 hrs	Mon 01/04/24	Mon 01/04/24
94		AHT sails back to PoR for next caisson	5 h	0	5 hrs	Sun 31/03/24	Sun 31/03/24
95		Joint construction	10	0,5	10,5 hrs	Sun 31/03/24	Mon 01/04/24
96							
97		Rock Works C5		(0 %)	84 hrs	Mon 15/04/24	Fri 19/04/24
98		Scour filter 1 (sea)	12 h	0 h	12 hrs	Mon 15/04/24	Tue 16/04/24
99		Filter (res)	12 h	0 h	1 day	Mon 15/04/24	Wed 17/04/24
100		Scour filter 2 (sea)	12 h	0 h	0,5 days	Tue 16/04/24	Tue 16/04/24
101		Scour protection (sea)	12 h	0 h	0,5 days	Tue 16/04/24	Wed 17/04/24
102		Toe filter (sea)	12 h	0 h	0,5 days	Wed 17/04/24	Wed 17/04/24
103		Toe protection (sea)	12 h	0 h	0,5 days	Wed 17/04/24	Thu 18/04/24
104		Sailing to PoR, loading, sailing back loaded	24 h		24 hrs	Thu 18/04/24	Fri 19/04/24
105							
106							
107		Installation Caisson #6		(0 %)	65 hrs	Sun 31/03/24	Wed 03/04/24
108		Preperation	12 h	0	12 hrs	Sun 31/03/24	Mon 01/04/24
109		Towage outsite port	4 h	0	4 hrs	Mon 01/04/24	Mon 01/04/24
110		Towage to site	10 h	0	10 hrs	Mon 01/04/24	Mon 01/04/24
111		Anchorage			1 day	Mon 01/04/24	Tue 02/04/24
112		Positioning	6 h	0	6 hrs	Tue 02/04/24	Tue 02/04/24
113		Ballasting	6 h	0	6 hrs	Tue 02/04/24	Wed 03/04/24
114		Filling	3 h	0	3 hrs	Wed 03/04/24	Wed 03/04/24
115		AHT sails back to PoR for next caisson	5 h	0	5 hrs	Mon 01/04/24	Mon 01/04/24
116		Joint construction	10	0,5	10,5 hrs	Mon 01/04/24	Tue 02/04/24
117							
118		Rock Works C6		(0 %)	72 hrs	Fri 19/04/24	Mon 22/04/24
119		Scour filter 1 (sea)	12 h	0 h	12 hrs	Fri 19/04/24	Fri 19/04/24
120		Filter (res)	12 h	0 h	1,5 days	Fri 19/04/24	Sat 20/04/24
121		Scour filter 2 (sea)	12 h	0 h	0,5 days	Fri 19/04/24	Sat 20/04/24
122		Scour protection (sea)	12 h	0 h	1 day	Sat 20/04/24	Sun 21/04/24
123		Toe filter (sea)	12 h	0 h	0,5 days	Sun 21/04/24	Sun 21/04/24
124		Toe protection (sea)	12 h	0 h	0,5 days	Sun 21/04/24	Mon 22/04/24



Project: Planning 2.0
Date: Fri 02/06/23

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Page 3

H

Production duration

In this Section the cycle times, production duration and working time has been defined based on a yearly installation of 33 caisson units.

H.1. Trailing Suction Hopper Dredge (TSHD)

Category	Value	
Hopper volume	25 000	m^3
Installed power	32 000	hp
Dredging cycle		
Sailing to borrow area	1	hour
Dredging	2.5	hours
Sailing to project site	1	hour
Discharging	3.5	hours
Total	8	hours

Table H.1: Basis characteristics of the TSHD and its assumed dredge cycle.

Sea bed preparation

Category	Value	
Volume	7.3×10^5	m^3
Cycles	31	cycles
Duration	245	hours
Efficiency[2]	90	%
Net duration	220	hours

Caisson ballasting

Category	Value	
No. caissons	33	units
Total ballast volume	1.9×10^6	m^3
Dredge cycles	79	cycles
Duration	633	hours

Inner berm

Category	Value	
Inner berm sections	33	units
Total volume	9.0×10^6	m^3
Dredge cycles	374	cycles
Duration	2993	hours

Total production duration	3850	hours
	23	weeks

Average production duration	120	hours/unit

Table H.2: Amount of production and associated duration for all dredge activities for the first year. Also the averaged production duration per caisson is determined.

H.2. Fall Pipe Vessel (FPV)

Category	Value	
Capacity	15 000	ton
Installed power	18 000	hp

Installation cycle

Sailing to borrow area	7.5	hours
Installation	7.5	hours
Sailing to PoR	7.5	hours
Loading	12	hours
Total	33	hours

Table H.3: Basis characteristics of the FPV and its assumed installation cycle.

Foundation installation		
Category	Value	
No. foundations	33	-
Mass	20 085	ton/foundation
No. cycles	44	cycles
Duration	1458	hours

Scour protection installation		
Category	Value	
No. sections	33	-
Mass	2070	ton/section
No. cycles	5	cycles
Duration	150	hours

Total production duration	1610	hours
	10	weeks
Average production duration	49	hours/unit

Table H.4: Amount of production and associated duration for all dredge activities for the first year. Also the averaged production duration per caisson is determined.

H.3. Side Stone Dumping Vessel (SSDV)

Category	Value	
Capacity	5000	ton
Max installation rate	4000	ton/hour
Installed power	10 000	hp
No. of vessels	2	

Installation cycle		
Sailing to project site area	8	hours
Installation	2	hours
Sailing to PoR	6	hours
Loading	2	hours
Total	20	hours

Table H.5: Basis characteristics of the SSDV and its assumed installation cycle.

Rock works installation		
Category	Value	
No. foundations	33	-
Mass	37 180	ton/foundation
No. cycles	123	cycles

Total production duration	2450	hours
	14.6	weeks
Average production duration	75	hours/unit

Table H.6: Amount of production and associated duration for all dredge activities for the first year. Also the averaged production duration per caisson is determined.

H.4. Heavy Lifting Barge (HLB)

Category	Value	
Lifting capacity	600 / 1000	ton
Loading capacity	40	L walls
Installation cycle		
Sailing to project site	10	hours
Positioning of 4 caissons	4 x 12	hours
Installation of wave return walls	4 x 24	hours
Sailing to Harbour Port of Great Yarmouth	10	hours
Loading	10	hours
Total	174	hours

Table H.7: Basis characteristics of the HLB and its assumed installation cycle.

Category	Value	
No. caissons	33	-
No. L-walls	330	-
No. cycles	8.3	cycles
Total production duration	1450	hours
	8.5	weeks
Average production duration	45	hours/caisson

Table H.8: Deployment of the HLB.

H.5. Anchor Handling Tug (AHT)

Category	Value	
Installed power	10 000	hp
Installation cycle		
Towage outside port	4	hours
Towage to project site	15	hours
Sailing to PoR	6	hours
Total	25	hours

Table H.9: Basis characteristics of the AHT and its assumed installation cycle.

Caisson towage

Category	Value	
No. caissons	33	-
No. cycles	113	cycles
Total production duration	825	hours
	5	weeks
Average production duration	25	hours/unit

Table H.10: Deployment of the AHT.

H.6. Towing Tugs and Dock workers

Category	Value	
Deployment	12	hours/unit

Table H.11: Deployment per caisson unit of the towing tugs.

I Construction costs

I.1. Material costs for the caisson dam

The material costs capture all expenditures that are related to the construction materials, for instance the costs related to concrete and reinforcement bars for the caisson unit and wave return wall but also the labour required to fabricate the elements. Despite the fact that the real material and labour costs are unknown, a qualitative estimate can be derived based on the required quantity per category (concrete, reinforcement bars and labour) and the price per unit.

Table I.1 lists the costs associated with the caisson unit, based on the concrete volume of the element and the concrete price, the estimated reinforcement steel and the labour and Table I.2 summarizes the material and fabrication costs with regard to the wave return wall. The exact calculation of the fabrication costs involve the determination of the actual prices for concrete, reinforcement steel and labour. However, this is considered to be beyond the scope of this research, therefore the prices are estimated as follows:

- Concrete price: €200/m^3
- Reinforcement steel: €1.0/kg
- Labour (Spain): €25/hour
- Heavy transport vessel (capacity = 5 caissons): € 10 000 day^{-1}caisson^{-1}

Caisson unit	Remark	Quantity	Price per unit	Total cost
Concrete		8500 m^3	200 €/m^3	€ 1 700 000
Reinforcement steel	100 kg/m^3	850 ton	1.0 €/kg	€ 850 000
Labour	40 workers	11 days	25 €/hr	€ 264 000
Transport to PoR	€ 10 000 day^{-1}caisson^{-1}			€ 50 000
Total cost				€ 2 864 000

Table I.1: Cost estimation for the caisson fabrication and transport to the Port of Rotterdam.

Wave return wall	Remark	Quantity	Price per unit	Total cost
Concrete		2800 m^3	200 €/m^3	€ 560 000
Reinforcement steel	100 kg/m^3	280 ton	1.0 €/kg	€ 280 000
Labour	40 workers	2 days	25 €/hr	€ 12 000
Total cost				€ 852 000

Table I.2: Cost estimation for the wave return wall fabrication.

Similar to the costs with regard to the concrete elements, an accurate cost determination for the rock works depend on the actual quarry and transport prices, and is also considered to be beyond the scope of this research. Based on Wagner (2004), for each specific rock layer the price per ton for purchase, storage and transport of the rocks is estimated, adjusted for the purchase prices found today. The assumed price per ton for all the rock gradings, consisting of the purchase price at the quarry and transport costs to the Port of Rotterdam, are listed in Table I.3. Table I.4 summarizes the quantity, price per unit and total cost of the specific layer for one caisson unit length.

Grading	Purchase	Transport	Total
HM_A 3000 - 6000 kg	€ 50	€ 25	€ 75
HM_A 300 - 1000 kg	€ 45	€ 25	€ 70
LM_A 60 - 300 kg	€ 40	€ 20	€ 60
LM_A 40 - 200 kg	€ 35	€ 20	€ 55
LM_A 10 - 60 kg	€ 30	€ 20	€ 50
CP 45 - 180 mm	€ 25	€ 15	€ 40

Table I.3: Estimated total prices for stone gradings per ton, including purchase at the quarry and transport to the Port of Rotterdam.

Rock works	Grading	Quantity [m^3]	Quantity [ton]	Price per unit [ton^{-1}]	Total cost [unit^{-1}]
Rock foundation	45 - 180 mm	7500	20 085	€ 40	€ 803 400
Scour filter (inner)	60 - 300 kg	500	1300	€ 60	€ 78 000
Scour filter	60 - 300 kg	500	1300	€ 60	€ 78 000
Scour filter	300 - 1000 kg	500	1300	€ 70	€ 91 000
Scour protection	3 - 6 ton	3300	8580	€ 75	€ 643 500
Toe filter	45 - 180 mm	800	2080	€ 40	€ 83 200
Toe protection	300 - 1000 kg	1000	2600	€ 70	€182 000
Inner berm protection	40 - 200 kg	8500	22 100	€ 55	€ 1 215 500
Geotextile	N/A	24 000 m^2	N/A	€ 10 per m^2	€ 240 000
Total cost					€ 3 414 600

Table I.4: Cost estimation for the rock works per caisson unit based on the required volume/tonnage. Price per unit for the quarry material is an estimation for purchase, storage and transport of the rocks.

Rock works	Grading	Quantity [m^3]	Quantity [ton]	Total cost [unit^{-1}]	Total costs
Filter layer	45 - 180 mm	240 000	624 000	€ 40	€ 24 960 000
Filter layer	60 - 300 kg	25 000	65 000	€ 60	€ 3 900 000
Scour protection	300 - 1000 kg	25 000	65 000	€ 70	€ 4 550 000
Scour protection	3 - 6 ton	165 000	429 000	€ 75	€ 32 175 000
Scour protection	5 - 40 kg	180 000	468 000	€ 45	€ 21 060 000
Total cost					€ 86 million

Table I.5: Cost estimation for the rock works per caisson unit based on the required volume/tonnage. Price per unit for the quarry material is an estimation for purchase, storage and transport of the rocks.

I.2. Material costs - Scour protection

I.3. Turbine costs

Since the concept of a low-head pumped storage hydropower plant is relatively new, the turbines that will be used for this sort of hydropower plants has not been extensively designed and developed compared to the traditional hydropower turbines. Normally, in case of a high-head pumped storage hydropower plant, the high water pressure or high flow velocity in turbine causes the blades or buckets to spin the turbine's shaft, thereby converting the potential energy into kinetic energy.

For low-head turbines, the maximum water pressure and flow velocity is significantly smaller than the high-head turbine, and hence require a different working principle. In this case, the low-head turbines use the flow rate of the water to rotate the turbine's blade, requiring a higher discharge than the high-head turbines.

Although low-head hydropower nor low-head pumped storage hydropower plants are not common, extensive studies have been conducted to develop low-head turbines such as the one shown in Figure I.1. Especially interesting for low-lying countries close to the sea which can not utilize the mountainous terrain to generate high-head hydropower, tidal basins can be used to generate low-head hydropower in stead. For instance, Rolls-Royce plc and Atkins Ltd (2010) conducted a comprehensive feasibility study for two low-head hydropower plants, located in the Severn Estuary near Cardiff (United Kingdom).

Interesting for this research were the costs associated with the turbine costs. Since designs (either a tidal or hydropower plant) with low-head turbines are simply rare, estimating the costs of such turbines is also complicated. Therefore, this research can be used as a starting point, taking the additional off-

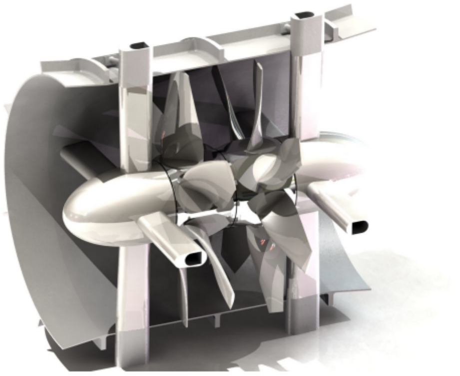


Figure I.1: (Very) Low head hydro turbine, designed by Rolls-Royce, power output up to 10 MW. Source: Rolls-Royce plc and Atkins Ltd (2010)

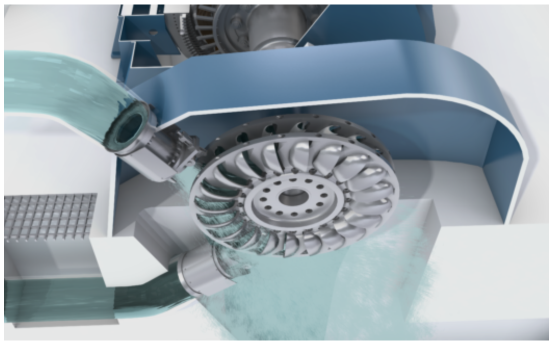


Figure I.2: Hydro impulse turbine (Pelton) designed for high head differences (between 200 m and 1800 m). Power output could be up to 350 MW. Source: GE Renewable Energy.

shore aspect of the PSH into consideration, which will complicate all processes and hence increase the costs.

In the design of the tidal power plant the capital costs (or CAPEX) for the turbines was estimated on € 0.85 million/MW. Including the offshore aspect of the conceptual pumped storage hydropower plant, an incremental rate of 15% will be included. Consequently, the turbine costs will be € 1 million/MW. This means that for the proposed turbines (10 MW) the cost per turbine will be € 10 million each, with a total sum of € 2 billion for all turbines.

I.4. Powerhouse costs

The powerhouse costs are after the material costs the largest expenditure regarding the capital investment and hence should be taken into account for the levelised cost of storage assessment. As has been discussed earlier, the powerhouse is the civil structure in the dam that accommodates the hydropower turbines, a possible control and operation room, additional repair storage and facilities but also all electrical infrastructure that is necessary for proper operation.

[redacted]. To estimate the costs per powerhouse unit, the same approach as for the regular caisson unit will be followed, determining the required amount of concrete and reinforcement steel will result in the cost price of the element.

[redacted]. The powerhouse facilitates not only the contra-rotating pump-turbines (CRPT) but also related utilities and auxiliary equipment.

[redacted]

Considering the installed power of 10 MW per turbine, in total 200 turbines are necessary to achieve a total power capacity of 2 GW for the PSH plant. Consequently, this results in a length of approximately 3 km for the turbine side [redacted], which is shown in the current design of the PSH in Figure I.6.

Assuming that 30% of the caisson's volume will be made out of concrete, a total volume of roughly 7560 m^3 will be used. All parameters are listed in Table I.6; the fabrication costs for the powerhouse caisson

[redacted]

Figure I.3: [redacted], [redacted]. *Due to confidentiality, this figure has been left out.*

[redacted]

Figure I.4: [redacted], [redacted]. *Due to confidentiality, this figure has been left out.*

[redacted]

Figure I.5: [redacted]. *Due to confidentiality, this figure has been left out.*

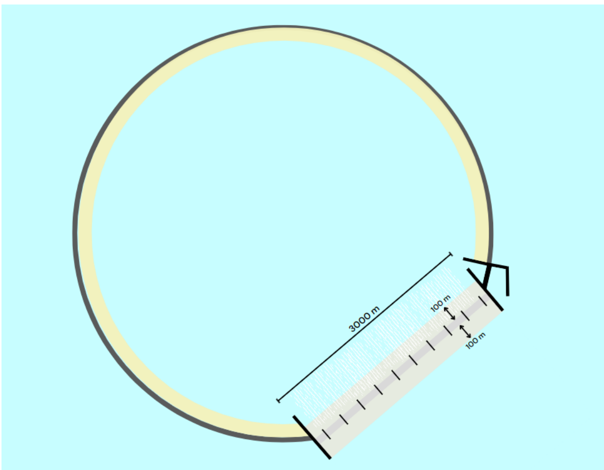


Figure I.6: Layout of the facility including the powerhouse / turbines infrastructure.

has been estimated on almost € 3 million per element. With a total of 200 installed elements, the total costs with regard to the powerhouse caissons is estimated on € 600 million.

Caisson unit	Remark	Quantity	Price per unit	Total costs
Concrete		7560 m^3	200 €/m^3	€ 1 512 000
Reinforcement steel	100 kg/m^3	756 ton	1.0 €/kg	€ 756 000
Labour	80 workers	22 days	25 €/hr	€ 528 000
Transport to PoR	€ 2500 $day^{-1}caisson^{-1}$			€ 12 500
Total cost				€ 2 808 500

Table I.6: Cost estimation for the powerhouse caisson fabrication and transport to the Port of Rotterdam.

Additionally, the powerhouse elements also require an extra stretch of bed protection in front of the powerhouses, to ensure the discharge of water during pumping and turbining periods will not erode the foundation. This bed protection will be installed in addition to the currently planned scour protection, as this protection is designed to absorb wave impact.

According to the stability calculations, a bed protection consisting of a LM_A 5 - 40 kg grading is sufficient to withstand the in- and out-stream of the reservoir. The costs associated with the bed protection layers, both dedicated to the wave impact and flow, are summarized in Table I.5. The costs for the entire scour protection (waves and flow) are estimated on € 101 million.

For construction projects in general, it is rare to account for all expenditures with regard to material cost in the first year, instead these costs are spread out over difference phases or period of the project, depending on the project size, construction timeline and payment terms. To simplify the exact payment timeline, it is assumed that the material costs are spread out over the entire project duration evenly, resulting in a constant cash flow (regarding the material costs) over time. Table I.7 summarizes all material costs for the project, including the caisson dam, powerhouse and turbine costs.

Element	Units	Cost per unit	Total cost	Cash flow
Caisson unit		€ 2 864 000	€ 607 million	€ 60 million
Wave return wall	212	€ 852 000	€ 180 million	€ 18 million
Rock works		€ 3 414 600	€ 723 million	€ 72 million
Powerhouse elements	200	€ 2 808 500	€ 561 million	€ 56 million
Scour/Bed protection	N/A	N/A	€ 86 million	€ 9 million
Turbine	200	€ 10 million	€ 2 billion	€ 40 million
Total			€ 4.2 billion	€ 420 million

Table I.7: Total material cost for the concrete and rock elements of the structure and the associated cash flows spread out over a payment period of 10 years.

I.5. Equipment costs

Equipment costs refer to all expenditures associated with the dedicated equipment required for the preparation, execution, operation and maintenance of the structure. Examples of these type of expenditures are the chartering of specialized machinery such as heavy lifting cranes or diving systems. Generally, in construction projects equipment costs are a substantial part of the overall cost, particularly for offshore projects for which specialized equipment is required.

Since the dredging and marine market is a very competitive market, actual charter costs for dredging equipment is classified and hence sensitive information for the public. Although the actual daily rates can not be used, the cost of dredging equipment has also been estimated by CIRIA (2009) and provides an indication of the expected costs.

The equipment cost per vessel type stated by CIRIA (2009) consist of two categories of costs, 1). Depreciation and interest (D + i) and 2). Maintenance and repair (M + R). The first category involves the depreciation of the vessel's value over time (taking into account the expected life time) but also the annual interest, which is based on the viable business return on capital (which has been taken as seven per cent annually). The second category involves the maintenance and repair works that ensure the wear and tear of the components will not interrupt, stop, delay or decrease the production of the vessel, but exclude costs associated with fuel, lubricants and crew.

It is assumed that the former category (depreciation and interest costs) can be seen as a constant cost, even when the vessel is moored in the harbour, depreciation and interest is applicable. Contrary to the latter category, which includes maintenance and repair works, which will only emerge while the vessel is deployed / in production (only then wear and tear of the components will take place). To incorporate the additional costs related to fuel consumption, a multiplication factor on the M + R costs will be applied.

Subsequently, two types of costs emerge from this assumption, namely the vessel's base cost and the vessel's operation cost. The first type involves the depreciation and interest cost of the vessel over the entire work season even when the vessel does not work for the entire year, whereas the operation cost only considers the additional or incremental cost due to the maintenance and repair works when deployed. In turn, the D + i costs can be seen as the base costs and the M + R costs as the incremental operational costs.

For the construction works in the harbour, e.g. preparation of the caisson unit, dock workers must be stand-by the entire working season and only should work when the caisson unit can leave the harbour. Even though the workers will also be paid for the stand-by time, the costs will be less due to a reduction in the usage of the harbour utilities. All rates are listed in Table I.8, including the marginal cost that is defined as the increment from the stand-by rate to the operation rate.

Vessel type	Installed power	Capacity	D + i	M + R
Trailing Suction Hopper Dredge (TSHD)	23 000 hp	17 000 m^3	€ 373 760	€ 129 730
Fall Pipe Vessel (FPV)	19 000 hp	15 000 ton	€ 400 000	€ 125 000
Side Stone Dumping Vessel (SSDV)	10 000 hp	5000 ton	€ 276 360	€ 90 240
Heavy Lifting Barge (HLB)	15 000 hp	600 / 1000 mT	€ 200 000	€ 75 000
Anchor Handling Tug (AHT)	7000 hp	116 t bollard pull	€ 29 491	€ 27 379
Towing tugs	6000 hp	91 t bollard pull	€ 25 278	€ 23 468
Dock workers	20 workers	N/A	€ 70 000	€ 87 500
Auxiliary equipment	N/A	N/A	€ 100 000	€ 50 000

Table I.8: Characteristics for different vessel types, with an indication of the weekly operational and stand-by rates based on the expenses of the TSHD. Rates are expressed in euros per week. Abbreviations: D = Depreciation of the vessel, i = Interest costs, M = Maintenance costs and R = Repair costs.

Since the cost guideline has been composed in 2009, inflation and other price changes (e.g. steel and oil prices) have increased the overall costs substantially, subsequently the rates stated in the manual are out-dated. To include this phenomena, CIRIA publishes the cost indexation relative to 2009 annually, from which the current cost estimate can be calculated. Table I.9 summarizes the cost indexation per vessel type, the rates in 2009 and the corrected rates that apply to 2023.

Intermezzo - Cost indexation

Cost indexation is a method used to adjust the values of prices based on inflation or deflation. This means that the cost of an item is modified based on the change in the general price level of goods and services. The purpose of cost indexation is to ensure that economic prices and rates reflect the actual impact of inflation or deflation. This is an important parameter with regard to financial planning, and hence project management.

For example, if the rate of product A in 2020 is € 100 per week and inflation causes the general price level to increase by 3%, the cost indexation factor would increase the weekly rate to € 103. The change in value is expressed in terms of indexation value: the change is expressed relative to a base year (which represents 100%). An increase in value of 3% is expressed as an indexation of 103 and subsequently a decrease in value of 3% is expressed as an indexation of 97.

Based on Table I.9 the associated cash flows per year can be estimated for the baseline scenario, considering a project duration of 9 years (installation of 33 caissons per year). For the equipment expenditures the work season duration is important as this determines the stand-by duration for all vessels and equipment. In turn, the actual deployment duration, or operational time, depends on the planning that has been

Vessel type	Cost index[1]	D + i (2009)	M + R (2009)	D + i (2023)	M + R (2023)
Trailing Suction Hopper Dredge (TSHD)	132	€ 373 760	€ 129 730	€ 503 490	€ 129 730
Fall Pipe Vessel (FPV)[2]	132	€ 400 000	€ 125 000	€ 528 000	€ 165 000
Side Stone Dumping Vessel (SSDV)	132	€ 276 360	€ 90 240	€ 364 795	€ 119 117
Heavy Lifting Barge (HLB)[1]	132	€ 200 000	€ 75 000	€ 264 000	€ 99 000
Anchor Handling Tug (AHT)	132	€ 29 491	€ 27 379	€ 38 928	€ 36 140
Towing tugs	132	€ 25 278	€ 23 468	€ 33 367	€ 30 977
Dock workers	100	€ 70 000	€ 87 500	€ 70 000	€ 87 500
Auxiliary equipment	100	€ 100 000	€ 50 000	€ 100 000	€ 50 000

Table I.9: Cost estimate for different vessel types, with an indication of the weekly operational and stand-by rates based on CIRIA (2009). Rates are expressed in euros per week and corrected with cost indexation from 2009 for 2023. Abbreviations: D = Depreciation of the vessel, i = Interest costs, M = Maintenance costs and R = Repair costs. [1] Cost indexation for the year 2023. [2] Estimated based on the costs provided in the guideline.

composed in Section 5.

For the TSHD, FPV and Dock workers the work season starts one month earlier than the other types of equipment. Subsequently, the work season of the former lasts 7 months or approximately 26 weeks (March - September), for the latter it lasts 6 months or approximately 22 weeks (April - September).

Stand-by costs Considering a constant annual installation of 33 caissons, the project duration would be 9 years. The stand-by costs for the entire project - assuming a work season of 22 and 26 weeks - are shown in Table I.10. During the work season the weekly rate is estimated on € 2.2 million, excluding the additional costs associated with operation. This implies that one week of delay during the construction costs € 2.2 million.

Element	Stand-by weeks	Stand-by rate	Total stand-by costs
Trailing Suction Hopper Dredge (TSHD)	26	€ 503 490	€ 18.4 million
Fall Pipe Vessel (FPV)	26	€ 528 000	€ 13.7 million
Side Stone Dumping Vessel (SSDV)	22	€ 364 795	€ 8.0 million
Heavy Lifting Barge (HLB)	22	€ 264 000	€ 5.8 million
Anchor Handling Tug (AHT)	22	€ 38 928	€ 0.9 million
Towing tugs	22	€ 33 367	€ 0.7 million
Dock workers	26	€ 70 000	€ 1.5 million
Auxiliary equipment	26	€ 100 000	€ 2.6 million
Total		€ 2.1 million	€ 51.7 million

Table I.10: Duration of the stand-by period, the stand-by weekly rate and the total stand-by costs for the different type of vessels.

Operational costs Besides the stand-by costs, the incremental cost that is associated with the actual deployment of the equipment must be determined in order to find the total equipment costs. The additional operation costs are given by the maintenance and repair costs (M + R) that are given in Table I.9 on a weekly period. As has been discussed earlier, the M + R costs do not capture the incremental costs such as fuel and lubricants consumption. To include the this category, as a first estimate the M + R costs will be multiplied by a factor 2.

The deployment period per vessel type can be estimated by determining the average production time per caisson unit, based on the cycle time. In Appendix H the production duration, cycle times and production rates are determined per vessel. Depending on the number of caissons placed in one year, the operational costs can be calculated. A summary for the incremental deployment costs is shown in Table I.11, which lists the cycle time, average deployment per caisson and the associated deployment costs per equipment type.

Category	Capacity	Cycle time	Deployment
Trailing Suction Hopper Dredge (TSHD)	17 000 m^3	8 hours	75 hours
Fall Pipe Vessel (FPV)	15 000 ton	33 hours	48 hours
Side Stone Dumping Vessel (SSDV)	5000 ton	20 hours	68 hours
Heavy Lifting Barge (HLB)	40 L-walls	174 hours	44 hours
Anchor Handling Tug (AHT)	116 t BP	25 hours	25 hours
Towing tugs	N/A	12 hours	12 hours
Dock workers	N/A	12 hours	12 hours

Table I.11: Averaged incremental deployment costs per installed caisson unit for all equipment types. Deployment is given in terms of average hours per installed caisson respectively.

A metric that is often used to express the efficiency of deployment for a vessel is the utility rate, which is defined as the working time expressed in percentages relative to the total duration. Based on this metric, a different work method, equipment choice or design can be chosen, as it basically shows how efficient the money is spent. To give an indication what the utility rates are for the current project, Figure I.7 summarizes the rates per vessel category.

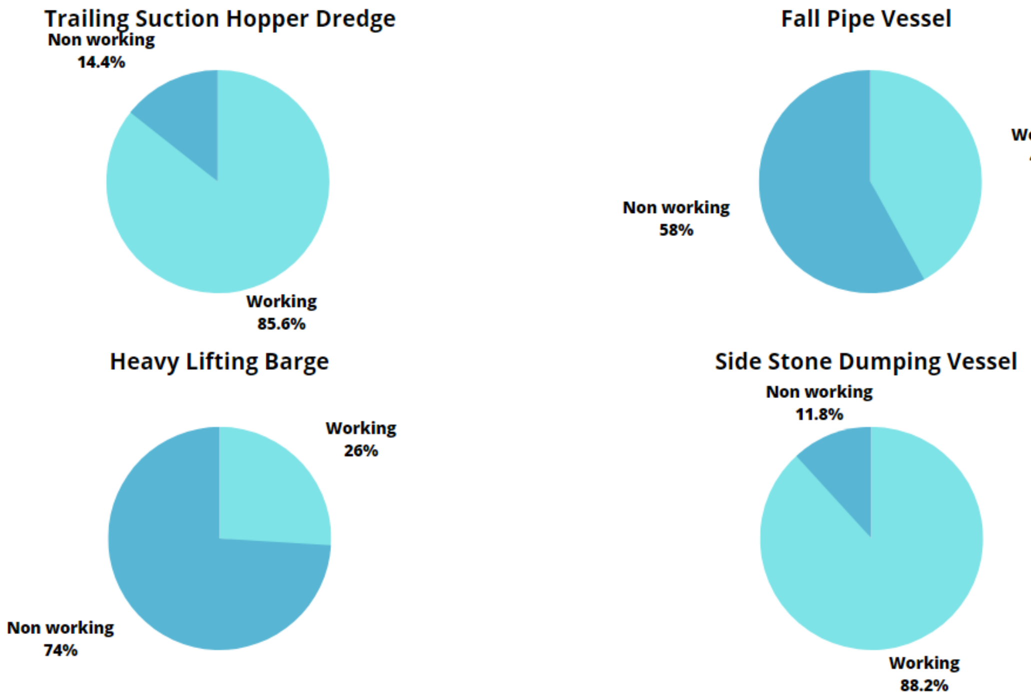


Figure I.7: Utilization rate, an efficiency term, defined as the percentage of operational hours expressed in terms of total available hours, evaluated for the trailing suction hopper dredge, fall pipe vessel, heavy lifting barge and side stone dumping vessel.

Figure I.8: Equipment cost evaluation (fixed cost vs. operational cost) for the three largest expenditures: trailing suction hopper dredge, fall pipe vessel and the side stone dumping vessel. Abbreviations: TSHD = Trailing Suction Hopper Dredge, FPV = Fall Pipe Vessel, SSDV = Side Stone Dumping Vessel, HLB = Heavy Lifting Barge.

I.6. Risk costs

Sea bed preparation The deployment of the trailing suction hopper dredge for the sea bed preparation, as the first step of the project, is considered to be a relatively risk-free operation, since the activities are not uncommon practice or under unknown or uncertain circumstances. Therefore, only minor events with little impact on the overall project or monetary value are assumed. In case of any delay, it is assumed that only the operational costs of the trailing suction hopper dredge will be relevant, as there are no successor tasks that depend directly on the sea bed preparation (only when the vessel has a downtime of more than one week, the delay could also have an effect on the installation of the rock foundation layer; however, this would be practically impossible). The events that have been considered and the consequences are shown in Table I.12.

Category	Consequence	Description
Unforeseen downtime due to mechanical issues	€ 100 000 and 2 days delay	Mechanical issues could refer to blockage of a pipe, breaking down of one of the pumps, issues with the draghead or problems with the operation system. Despite the wide range of events, the consequence is assumed to be minor. Repairs works can easily be done at the Port of Rotterdam and will cost 2 days.
UXO encounter	€ 0 and 1 days delay	During trailing, an unexploded ordnance (UXO) can be sucked into the draghead or even into the suction pipes of the vessel. All operations must hold, until a special explosion team has safely removed the UXO.
Severe delay of the trailing suction hopper dredge	More than 1 or 2 weeks delay	This event will have a major impact on the start of the project, as the preparation phase would be delayed so severely that this will effect the start of the structural season, i.e. the good weather season in which caisson installation and all successor tasks are possible. Although the preparation phase starts one month a head of the structural phase, a delay of one to two weeks will delay the start of the structural season directly. Therefore, the costs related to this event will be the fixed weekly rate of all vessels that are deployed during the structural season.

Table I.12: Events that could occur during the first phase of the project, the sea bed preparation executed by the trailing suction hopper dredge.

Rock foundation layer After the sea bed has been prepared, i.e. horizontally levelised and at the desired elevation, the foundation layer for the caisson unit must be installed by the fall pipe vessel. Similar to the activities for trailing suction hopper dredge, the installation of the foundation layer is considered to be a common task under normal circumstances. Therefore, for this construction phase there are only minor events considered, with little impact on the project planning and/or have a small impact on the monetary value. The only event with a large impact is the case where the fall pipe vessel is delayed so severely, the rest of the project must be postponed. The events that have been considered and the consequences are shown in Table I.13.

Category	Consequence	Description
Unforeseen downtime due to mechanical issues	€ 100 000 and 2 days delay	Mechanical issues could refer to blockage of a pipe, breaking down of conveyor belt, issues with the fallpipe or problems with the operation system. Despite the wide range of events, the consequence is assumed to be minor. Repairs works can easily be done at the Port of Rotterdam and will cost 2 days.
Rock foundation layer has been spilled with (dredged) sediments	€ 0 and 1 week delay	The rock foundation layer must provide stability and bearing capacity for the caisson unit, and should be a clean patch of rock. Contamination by sediment will reduce the stability and bearing capacity, and should be removed from the layer. Special equipment can be used to remove sediments.
Severe delay of the fall pipe vessel	More than 1 or 2 weeks delay	Similar to the trailing suction hopper dredge, substantial delay of the fall pipe vessel would hinder directly the start of the construction season. In case the fall pipe vessel has unforeseen downtime, the rock foundation layer will not be finished in time, thus the installation of the caissons must be postponed. The weekly rate related to this delay is the sum of all deployed equipment during the construction season.

Table I.13: Events that could occur during the second phase of the project, the installation of the rock foundation layer executed by the fall pipe vessel.

Caisson towage Prior to the caisson installation, the caisson will be towed to the project site where it will be moored to an offshore anchor. From here the caisson will be towed (during favorable weather conditions) to the desired location, positioned and ballasted. During the towage of the caisson unit, starting at the Port of Rotterdam, several events may occur or issues may be encountered. While most of the events have a relatively minor impact, sinking of the caisson unit during transport would be a major concern. Although this is absolutely not acceptable, the consequences would be enormous, in particular when the caisson unit sinks at the deeper parts of the North Sea (shipping routes): in that case the caisson unit might be fully submerged. Salvage of the caisson unit will not only contribute substantially to the consequence (monetary value), the execution of the salvage works is a hurdle on its own. Fortunately, the sinking of the caisson will not hinder the project directly: possibly only one or two days are missed due to this event. The events that have been considered and the consequences are shown in Table I.14.

Category	Consequence	Description
Breaking down of the winch(es)	€ 100 000 and 1 days delay	Breakage of the winches will cause uncontrolled movement of the caisson unit, whereas the caisson unit must be returned to the Port of Rotterdam.
Structural failure of the winch basements	€ 500 000 and 1 days delay	Failure of the winch basement, at the corners of the caisson. During towage the forces are redirected to the foundation of the winches; when the limit is exceeded, the foundation will fail. Repair is possible, but takes a while.
Caisson sinks	€ 5 million and 2 days delay	Transportation of the caisson during unfavorable conditions might lead to severe wave overtopping of the caisson, which might result in sinking of the entire caisson unit. Salvage of the caisson might be difficult in case the caisson is fully submerged.
Other minor damages	€ 100 000 and no delay	Category that refers to all events with little impact on the planning and on the caisson unit.

Table I.14: Events that could occur during the towage of caisson unit to the project site.

Installation of the caisson unit The most vital part of this project can be considered to be the installation of the caisson unit, as this phase involves many events or occurrences that could lead to a failure or delay of the installation. Since the element is the most expensive in the entire project and this tasks has the most successor tasks, the consequences are the largest, both the delay time and the monetary value. Fortunately, a total loss of the caisson unit is not considered since the local water depth does not allow the caisson unit to be fully submerged, in other words there will always be some of the caissons crest above the water surface. Consequently, salvage of the caisson unit is not necessary, since emptying the ballast tanks will create buoyancy subsequently refloating the caisson unit again. The events that have been considered and the consequences are shown in Table I.15.

Category	Consequence	Description
Damage to foundation layer and / or caisson unit during uncontrolled heave	€ 250 000 and 7 days delay	Installation process during unfavorable weather conditions, resulting in uncontrolled and undesired movement of the element. Near the bed, the caisson unit can hit the foundation layer, damaging both the caisson unit and the foundation layer. Repair works include inspection of the caisson unit and foundation layer, and will take at least a week.
Flow velocities underneath the caisson unit mobilises the rock foundation layer	€ 100 000 and 7 days delay	Due to exceedance of the wave conditions, high flow velocities may prevail when the caisson unit very close to the bed. High flow velocities, as a result of the wave response of the caisson unit, may mobilise the foundation layer. Consequence includes inspection to the foundation layer, with the possibility for repair works.
Uncontrolled / Uneven ballasting results in capsizing of the caisson unit	€ 1 million and 7 days delay	Due to the uneven ballast distribution of the caisson unit in combination with severe wave impact, the caisson unit might capsize or sink at an undesired during the installation process. Since the caisson unit will not be fully submerged (caisson height and width are larger than the local water depth), the consequence are relatively small. Refloatation of the caisson unit is possible by pumping water out of the ballast tanks.
One of the four winches breaks	€ 0.1 million and 1 day delay	An event with minor impact with regard to monetary value or delay, although due to the relatively high probability should be included
Sliding of the caisson during or after filling	€ 1 million and 7 days delay	When the wave conditions exceeds the threshold of the caisson's stability, the caisson might become unstable and consequently may slide over the foundation layer. Despite the fact that little displacement is not acceptable, additional (repair) work is only needed when the displacement is beyond a certain point. For instance, in case when the joint can not be constructed properly. Then, the unit must be refloated by removing the ballast material from the ballast tank to the point where the caisson unit will have some buoyancy.
Joint construction	€ 500 000 and no delay	In case the gap between two installed caissons is too big, it might be difficult to make the joint construction. In that case all alternative options should be considered, from refloatation of the caisson unit, to alternative joint construction methods. The consequences of this event are expressed only in the delay rather than in a currency.

Table I.15: Events that could occur during the installation of caisson unit.

Installation of the wave return wall The final step for the caisson installation is the construction of the wave return wall (or L-wall), which includes the hoisting operation from the crane vessel to the caisson unit. Hoisting or lifting operations are known to be very strict with regard to the weather conditions, as heave and high wind velocities will result in undesired uncontrolled movement of the load. The largest impact on the project is the total loss of the wave return wall due to breaking of the winches or incorrect placement on the caisson unit, possibly resulting in the total loss of the L-wall. Salvage experts must be involved to retrieve the element from the sea bed, which will be a complicated and lengthy operation. The events that have been considered and the consequences are shown in Table I.16.

Category	Consequence	Description
Winch breaks, no damage to the load	€ 500 000 and 7 days delay	During the lifting of the wave return wall one of the winches break down, resulting in a total stop of all activities associated with the crane vessel. Although the load of the vessel has not been damaged, the repair works will last one week.
Anchorage of the hoisting point on the wave return wall fails	€ 1 million and 1 days delay	Failure of the anchorage of the hoisting point on the wave return wall is not allowed, however should also be considered. During the hoisting operation, the anchorage may fail, resulting in uncontrolled movement of the load, potentially damaging the vessel, caisson unit or other nearby equipment. Depending on the type of damage, a new wave return wall will be used or an alternative installation method may be applied.
Total loss of the wave return wall	€ 2.5 million and 7 days delay	During installation, either hoisting or construction to the caisson unit, the wave return wall ends up on the sea bed. It is considered as a total loss, since the element can not be used anymore. Salvage experts must be involved to retrieve the element, and recycle / re-use the remaining parts.
Other minor damages	€ 500 000 and 0 days delay	This category refers to many other minor damages that could occur, but do not have a large impact on the overall planning.

Table I.16: Events that could occur during the installation of wave return wall (L-wall).

Installation of the scour protection After the caisson unit has been installed, including the joint construction and the wave return wall, the scour protection layers can be installed to prevent undesired erosion of the sea bed and the rock foundation layer. During this phase the largest risk is the occurrence of a (summer) storm while the scour protection has not been finished completely at some places. This will, obviously, increase the probability of erosion and damage to the unprotected layers substantially. The level of damage depends on several parameters, among them are the length and the severity of the storm, and are combined to only one general category. The events that have been considered and the consequences are shown in Table I.17.

Category	Consequence	Description
Unforeseen extra installation losses	€ 250 000 and no delay	Due to unforeseen conditions, the actual installation losses are high than the expected, therefore additional logistics are needed to cope with this event.
Erosion of unprotected rock works	€ 500 000 and 1 day delay	During the installation of the scour protection, severe weather conditions prevail, stopping all construction works including the installation of the scour protection. In case the scour protection has not been finished completely at the moment the storm passes by, damage to the unprotected layers may occur. After the storm, additional surveys and possibly repair works must be executed.
Other minor damages	€ 500 000 and no delay	This category includes all events that have little impact on the planning nor a large financial impact.

Table I.17: Events that could occur during the installation of scour protection layers.

Construction of the inner berm and the stone revetment Lastly, the inner berm and the stone revetment will be installed, which will provide stability for the caisson unit and to prevent fierce erosion of the inner berm, respectively. Since the inner berm will be constructed in several steps, this implies that the berm will be left unprotected for a substantial period, but more particular during the winter months. Consequently, it is almost inevitable that parts of the inner berm are unprotected the moment that a winter storm passes by. This means that erosion of the inner berm is also inevitable, resulting in repair work for the trailing suction hopper dredge. However, to what extent the inner berm will be eroded, is beyond the scope of this research. Therefore, only a fixed monetary value is assumed. The events that have been considered and the consequences are shown in Table I.18.

Category	Consequence	Description
Severe erosion to the yet unprotected inner berm	€ 500 000 and no delay	After a severe storm, the inner berm will be heavily damaged since at many places the berm is yet still unprotected. During a storm, either swell waves or wind waves generated at the reservoir will erode the dune, transport the sediment to the centre of the reservoir. Although the berm will provide stability to the caisson unit, additional material is needed to reconstruct the berm to the desired shape. Considering the sensitivity of the unprotected inner berm, it is almost inevitable that additional deployment of the hopper is required.
Damage to the revetment	€ 250 000 and no delay	During the installation of the geotextile and the revetment, it is possible that damage occurs to the revetment or the geotextile, resulting in the replacement of geotextile and revetment. Especially when operating with the new equipment to install geotextile under water, the probability is higher that any damage will occur.

Table I.18: Events that could occur during the installation of the inner berm and the stone revetment.

I.7. Electrical discharge

The electrical discharge of the PSH plant refers to the energy yield by discharging sea water into the reservoir. In the LCOS metric, the cost of storage is expressed per unit energy (MWh), thereby the method offers the opportunity to compare different energy storage methods.

The capacity of the reservoir depends on the amount of energy that can be stored in the gravitational potential energy by pumping water from a lower reservoir to a higher reservoir, the working principle of a pumped storage hydropower plant. The increase of potential energy, and hence the storage of energy, is determined by the mass of water pumped to the higher reservoir and the height difference between the two reservoirs. In general, the gravitational potential energy is given by:

$$U = M \cdot g \cdot h \tag{I.1}$$

where U denotes the potential energy in Joule, M is the mass in kg, g is the gravitational acceleration in $\mathrm{m/s^2}$ and h is the height difference in m. With regard to a pumped storage hydropower plant, the amount of energy stored can be expressed in terms of the imposed head difference and the surface area of the reservoir, and is given by Equation I.2. A schematic overview of the change in centre of gravity of the water body in the reservoir for the full and empty situation with the corresponding height differences, is shown in Figure I.9.

$$U = M \cdot g \cdot h = V \cdot \rho \cdot g \cdot \Delta h = A \cdot \Delta h \cdot \rho \cdot g \cdot \frac{1}{2}\Delta h \tag{I.2}$$

where:
- U [J] = Potential gravitational energy
- ρ [$\mathrm{kg/m^3}$] = Density of (sea)water
- g [$\mathrm{m/s^2}$] = Gravitational acceleration

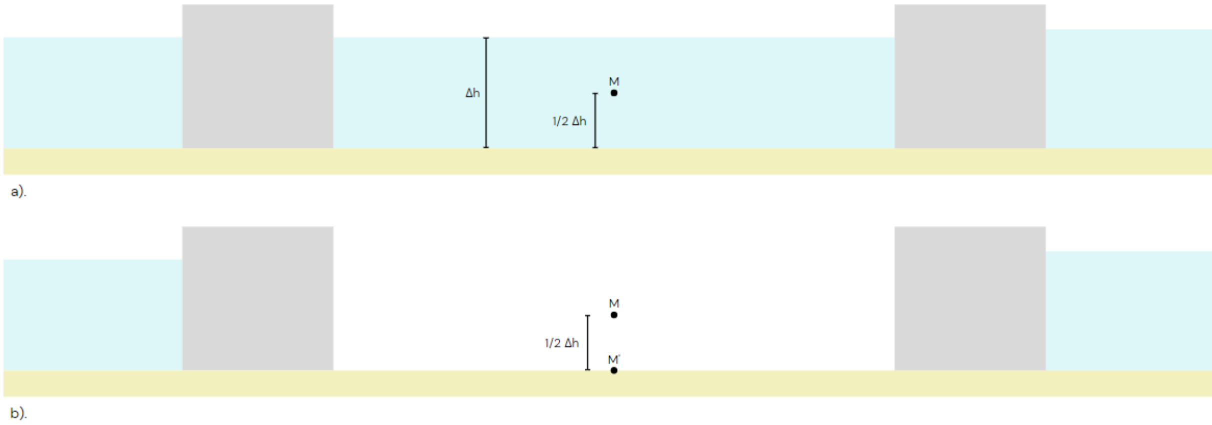


Figure I.9: Schematic overview of the change in centre of mass of the entire reservoir for the a.) full and b.) empty situation and the corresponding height difference.

- Δh [m] = Head difference
- A [m^2] = Area surface of the reservoir ($A = 1/4\pi D^2$)

The calculation for the energy storage capacity given a diameter of 5 km and a head difference of 27 m (water level inside the reservoir varies from MSL to -27 m MSL, 2 meters above the sea bed) is shown in Table I.19. Following this theoretical approach, the reservoir yields a total storage capacity of 23 GWh.

Category	Symbol	Value	
Diameter	D	5000	m
Surface area	A	3×10^7	m^2
Head difference	Δh	29	m
Volume	V	5.7×10^8	m^3
Mass	M	5.8×10^{11}	kg
Storage capacity	U	23	GWh

Table I.19: Energy storage capacity of the pumped storage hydropower plant, given the dimensions and a head difference of 29 m.

Depth of Discharge (DoD) The Depth of Discharge (DoD) refers to the amount of a battery's capacity that has been utilized when it is discharged. For the battery management this concept is crucial, as it influences the battery's health, lifespan and performance (considering 'regular' batteries, e.g. lithium-ion) (Waag & Sauer, 2009).

In case of the pumped storage hydropower, the depth of discharge also plays a vital role with regard to the operational and maintenance management. By imposing a minimum water level when fully charged (i.e. the reservoir is nearly empty), one must prevent that the turbines will fall dry and hence be exposed to the atmosphere. The reaction between silty water and the atmosphere can increase and accelerate corrosion of the turbine's elements.

Moreover, when the water level drops below a certain point during pumping, the flow velocities inside the reservoir will exceed the critical velocity of the sand grains, thereby mobilizing the soil inside the reservoir. Consequently, sand will be mobilised inside the reservoir and discharged through the turbines into the ocean. As the sediments will be transported through the powerhouse and turbine's elements at high velocities, wear and tear of the direct elements will substantially increase.

Therefore, a minimum water level in the reservoir must be imposed in order to prevent the situations mentioned above. However, by imposing a minimum water level the maximum head difference (of 29 m) can not be fully utilized. Quantifying the exact minimum water level involve many technical aspects and depends on other parameters such as the exact installed height of the turbines. As a first estimate, it is assumed that 80% of the total capacity of the reservoir can be utilized.

Taking into account the depth of discharge fraction for the pumped storage hydropower plant, the storage capacity decreases with 20% from 23 GWh to 18.5 GWh.

Conclusion Although the total energy storage capacity of the reservoir is 18.5 GWh (including the depth of discharge), the daily yield will be considerably less. This is due to working principle and the goal of this facility. The PSH plant aims to overcome the daily energy gap which is evoked by intermittency of renewable energies. This implies that the facility will generate energy during a small window: only when renewable energy resources could not answer the demand.

Therefore, it is assumed that the utility rate, i.e. the fraction during which the facility generates electricity, is only 20% or 5 hours per day (which may be slightly high compared to other PSH plants according to Kougias and Szabó (2017)). To estimate the daily electric discharge, Equation I.3 has been applied.

$$E_{dis,daily} = S_{net} \cdot \eta_t \cdot \eta_p \cdot DoD \cdot T \quad \text{(I.3)}$$

in which:
- $E_{dis,daily}$ [MWh/year] = Daily electrical discharge
- S_{tot} [MWh] = Total reservoir storage capacity
- η_t [%] = Turbine efficiency
- η_p [%] = Facility's utility rate
- DoD [%] = Depth of discharge
- T [hours] = Number of hours in one day

Based on the latter equation, the electrical discharge can be estimated. Table I.20 lists all relevant parameters for the daily electrical discharge. Following this approach, the daily electrical discharge has been determined at 6.8 GWh/day. To put this number into perspective, this would be the equivalent of the power consumption of 900 000 households with an average consumption of 7.6 kWh per day (City Centre Retreat, 2022).

Category	Symbol	Value	
Total storage	S_{tot}	23	GWh
Turbine efficiency	η_t	70	%
Facility's utility rate	η_p	20	%
Depth of Discharge	DoD	80	%
Daily electrical discharge	$E_{dis,daily}$	6.8	GWh/day

Table I.20: Reservoir's parameters to determine the daily and annual electrical discharge.

I.8. Electricity prices

Charging of the reservoir or imposing the head difference over the dam, thereby creating the energy potential in the reservoir, requires electricity to pump water from the lower to the higher reservoir. Consequently, electricity prices determine the charging cost of the PSH plant. Moreover, considering the functionalities of the plant (that is to overcome the intermittent energy gap / problem of renewable energy resources by charging during energy abundant periods and generating energy during energy absent periods) the moment of charging also plays a key role with regard to the charging costs.

Nowadays, due to the drastic increase in renewable energy resources (with high intermittency), electricity prices have a high volatility, implying that the difference between the lowest and highest daily price is substantially and can even be larger than € 100/MWh. Furthermore, during favorable weather conditions for either wind turbines or PV cells, the lowest daily price is often less than € 0/MWh, which would imply that charging the PSH plant would be free of cost.

However, the development of the electricity prices over time is still uncertain. Although the installation of renewable energy resources happen at an unprecedented rate, the energy consumption also increases rapidly, partly due to the electrification of the transport sector (International Energy Agency (IEA), 2023). An increase in both electricity generation and consumption would suggest that the lowest price will further fall whereas the highest price will increase.

This development has also be found in a study that looked into the energy and electricity market and prices for the multiple scenarios for 2020, 2023 and 2030. Considering the current policy of the Dutch government with regard to renewable energy resources, Afman, Hers, and Scholten (2020) have composed the expected energy prices for the years 2020, 2023 and 2030 taking into account the increase in energy production and consumption. The most progressive scenario (2030-RES) in the study considers a total installed solar PV power of 20 GW, 20 GW offshore wind and 8 GW onshore wind.

Currently, the total installed power of solar PV is already 19 GW (CSB, 2023) and offshore wind farms have an installed capacity of 5 GW (RVO, 2023) and is projected to further increase to an incredible 50 GW in the year 2040 (Rijksoverheid, 2022). This makes the most progressive scenario (2030-RES) that is considered in the study of Afman et al. (2020) a realistic assumption.

The development of the electricity prices (daily average prices during the cheapest hours and during the most expensive hours) are determined in the latter research, calculated for multiple scenarios. The expected average prices during the cheapest hours are presented in Figure I.10, where multiple percentiles are shown. These prices indicate the lowest electricity price for some X% hours per day considering two quantitative scenarios[3] (low and high price), and hence would be the price for which the PSH plant would be charged. In line with these results, considering the 50% cheapest hours scenario, it would imply that the reservoir could be charged with an average price of € 10/MWh.

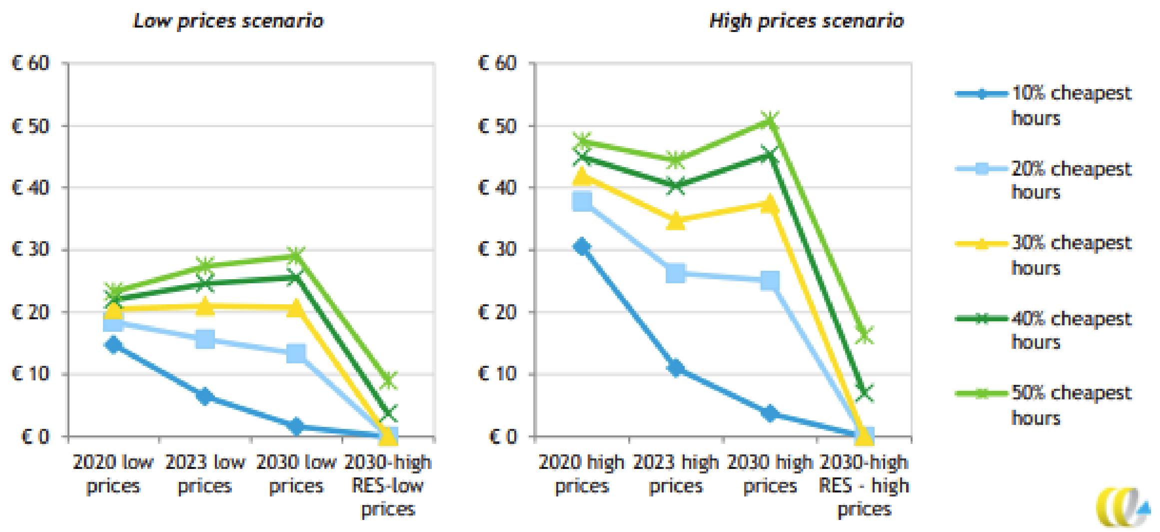


Figure I.10: Average price during the X% cheapest hours of the year. Source: Afman et al. (2020)

[3]The distinction between the future price scenario has been made based on an environmental fee: the expected CO_2 price per ton. The low price scenario considers a conservative CO_2 price of € 20.1 ton^{-1} in the year 2030, whereas the high price scenario considers a more progressive CO_2 price of € 40.6 ton^{-1}. Consequently, the future prices for coal and gas increases substantially when comparing the low and high price scenario for the year 2030. The coal price increases from € 55 to € 89 (low price and high price scenario respectively) and the gas price from € 21 to € 34 (low price and high price, respectively).

Parameter	Value		Description
PSH gross capacity	23	GWh	Total energy storage capacity of the PSH plant neglecting the turbine efficiency rate and depth of discharge (DoD). Including DoD the net capacity of the PSH plant is 18.4 GWh.
Installed power	2	GW	Total installed turbine power, both pumping and discharge.
Depth of Discharge (DoD)	80	%	Percentage of the reservoir that can be utilized for electrical discharge.
Turbine efficiency	70	%	Average pump/turbine efficiency.
Utility rate	20	%	Percentage in time for which the plant is generating energy.
Net pumping period[1]	7	hours	The time it takes to empty the reservoir completely.
Discharge period	4.8	hours	The time it takes to fill the reservoir again.
Idle time	12	hours	Total daily idle time of the reservoir.
Gross cycle	∼ 24	hours	Sum of pumping and discharge periods.
No. cycles	∼ 1	cycles/day	Taking into account the utility rate of the plant, the number of cycles per day can be determined
Charging energy	13.9	GWh	The charging energy includes the turbine's efficiency.
Energy yield	6.8	GWh	The discharge energy / energy yield includes the turbine's efficiency.
Electricity price	€ 10 000	per GWh	Expected averaged low electricity price per GWh.
Annual charging costs	€ 50.8 million		

Table I.21: Characteristics for the PSH and turbine specifications, and parameters that determine the total charging cost.[1] Assuming the installed power is the total power, and not the effective pumping power, 70% of the installed power will effectively be used for pumping. In line with this reasoning, the net pumping period will therefore be 43% longer (= 1 / 70%), subsequently will last 14 hours.

J Potential in-depth studies

Considering the fact that this project involves various aspects that increase the complexity and uncertainty of the planning, execution and associated logistics - e.g. the offshore characteristics of the project entails a substantial increase in complexity regarding execution and planning - several topics in these three categories are suitable and sufficient for further research in an in-depth study. The following topics will be discussed in detail, including what they involve and how this can contribute to the optimization of the project in a more wider view:

- Dynamic behavior/response of the caisson unit
- Joint construction
- Sheltering effects
- Damage to unprotected / unfinished parts of the scour protection and inner berm
- Design optimization for last caisson of the season

Dynamic behavior / response of the caisson unit The dynamic behavior or response of the caisson unit to wave conditions determines the limiting wave conditions during which the installation of the caisson unit can be installed. As has been discussed earlier, the installation of the caisson unit is the most determining construction phase of the entire project, hence also determining the pace for the project.

Improving the dynamic behavior of the caisson unit will immediately result into more favorable limiting wave conditions during installation as the caisson unit will be less sensitive to wave conditions. Therefore, this improvement will directly lead to an increase in the installation rate and hence a shorter project duration.

The current caisson design has been based on the structural stability checks that are described for preliminary design studies in the Manual Hydraulic Structures (Voorendt et al., 2020), neglecting the entire installation operation. The in-depth study for this aspect would involve the optimization of the design in a wide term, including the dimensions of the caisson to the design of a new or other shaped caisson that will improve the response. This could be done by for example marine modelling of different types of caisson units.

Joint construction The complexity of the joint construction between two caissons mainly depends on the size of the gap in between: the larger the gap, the more complex the operation will be. In turn, the gap size between caissons depends on the allowed tolerance during installation: an increase in tolerance will result in larger gaps. It must be noted that the installation tolerance has a major effect on the workability, as it improves substantially when the allowable tolerances are increased.

This effect can be explained when the forces in the winches are observed, through which the caisson unit is controlled and position by the tugs and heavy lifting barge. Basically, a high installation tolerance implies that the winch forces to keep the element in position are less than for the low tolerance case. When this concept is combined with the additional wave motion of the caisson unit, the maximum winch force will be sooner reached for the low installation tolerance. In other words, for the more strict tolerance lower limiting wave conditions prevail due to the higher winch forces as a result of the strict tolerance, contrary to the flexible installation tolerance which can proceed during higher wave conditions.

The consideration of on one hand increased workability and on the other hand a more complex joint construction is a relevant and utterly interesting topic that can be perfectly researched in a in-depth study. To what extent does the project duration change if the installation tolerance will be increased, what is the consequence for the joint construction and what is the perfect mix of installation tolerance and complexity of the joint construction.

Sheltering effects During the construction of the caisson dam the wave conditions often play a limiting role with regard to the execution of certain activities. However, as the caisson dam progresses at a certain point it will behave like an offshore breakwater, creating a sheltered area behind the dam where the wave conditions improve. In this sheltered area the waves are reduced due to the direct blockage of the caisson dam, resulting in an improvement in the workability.

One can imagine that due to this process of improvement in workability, the installation rate of the caisson units and other wave limiting construction works will increase over time, since the sheltered area created by the dam will increase over time. In turn, the actual installation rate will be higher than the theoretical 22 caissons per year, resulting in a shorter construction period and hence reduction in equipment costs.

The sheltering effects can be studied for several scenarios that represent certain stages of the dam progression, by either composing the change in boundary conditions and bathymetry in a wave model like SWAN or by simply applying diffraction tables dedicated to breakwaters and other marine structures. Regardless of the methodology, the result is a change in wave height given the wave direction for the considered scenarios. Subsequently, this new wave data can be used to determine the workability for the sheltered zone of the caisson dam.

Damage to unprotected / unfinished parts of the scour protection and inner berm One of the largest uncertainties in terms of construction risk is the damage to unprotected or unfinished parts of the scour protection and inner berm reclamation. During construction (in the favorable months) it is not excluded that storms can not occur, which implies that if construction works do not finish before a storm arrives parts of the structure may be damaged due to the exceedance of wave conditions.

The determination of the probability of occurrence and the consequence for various scenarios would be relevant for the in-depth study, for instance for damage to the scour protection or for erosion of the inner berm. This would not only create more insight in this process, but also might help in prioritizing the various construction steps, further reducing the associated risk costs.

Design optimization for last caisson of the season In September, the last month of the work season, the last caisson will be installed in this month, which will be vulnerable for wave impacts during the entire off-season. High flow velocities in combination with turbulence makes this particular element and adjacent area sensitive for damage and consequently failure.

In this research little attention has been spend on the prevention of (severe) scour around the tip of the caisson dam during the winter months, as this was beyond the scope of this research. An additional in-depth study could provide insight in the working principle of this phenomena and could also provide mitigation measures. A better designed caisson will eventually result in the reduction of risk costs, associated with damage to the scour protection, inner berm or the rock foundation layer.

Conclusion The choice for the in-depth study has been made based on several topics, among them are the beneficial value to the insight and knowledge, the expected effect on the project and the expected complexity and length of the in-depth study.

Firstly, the beneficial value of the results with regard to insight and gain in knowledge for these problems capture the first category. Regardless of the fact whether the gain in knowledge will be utilized in this specific project, it might be useful for other projects with similar characteristics. For instance, the dynamic behavior of a differently designed caisson might be also beneficial for other offshore projects.

Secondly, the expected effect on the construction costs, planning and execution as a result of the optimization in line with the in-depth study. The goal of this in-depth study is to minimize the construction costs, however in addition there are various side-outcomes that can be beneficial to the project in a general sense. For instance, the design optimization for the last caisson could result in very short installation time of the last caisson, subsequently resulting in a longer effective work season in which more caissons can be installed.

Thirdly, the required effort and complexity to understand the problem, formulate a research plan and methodology, setting up the model or optimization, running it and then formulate the conclusion and recommendations, must all perfectly fit in the available time. Despite the fact that every topic entails a different type of research, a quantitative assessment of the complexity and length of the research can be made. For instance, setting up an entire offshore vessel model to evaluate different types of caissons and the winch forces will be very complex.

A quantitative assessment has been made for all potential in-depth studies to find the most suitable and appropriate research, based on the described categories. Table J.1 lists per category the value ranging from favorable (++) to unfavorable (- -), and summarizes the total score for the in-depth studies. For this evaluation, setting up a model was considered to be unfavorable in terms of the corresponding complexity in combination with available time, subsequently scoring (- -) for the dynamic behavior of the caisson unit. Despite the huge beneficial value and the expected effect, this study did not is suitable for the in-depth study.

The sheltering effects of the caisson dam have potential effects on the execution phase of the project, provides more insight in the offshore behavior and response of caisson dams and is considered to be a relatively simple research with an appropriate length. Therefore, the overall score of this in-depth study outweighs the alternative studies, thereby winning the assessment.

Table J.1: Evaluation of the optimization or in-depth research for various topics, qualitatively assessment based on three parameters: i). beneficial value to knowledge, ii). expected effect on construction costs, planning and execution and iii). length and complexity of the in-depth study. Evaluation scores range from ++ to - -.

Study	Beneficial value	Expected effect	Complexity	Overall
Dynamic behavior / response of the caisson unit	++	++	- -	++
Joint construction	++	++	-	+++
Sheltering effects	+	+	++	++++
Damage to unprotected / unfinished parts of the scour protection and inner berm	++	+	-	++
Design optimization for last caisson of the season	+	0	-	0

K
Model setup

Example Now the approach has been introduced and elaborated, it will be clarified with an example, for which all steps will be reproduced.

Figure K.1 considers the situation where the caisson dam has been constructed for 50% and is subjected to waves originating from the south-west. From the four points of interest, the southern two points will be fully exposed and hence the sheltering effects are considered to be negligible, subsequently the initial wave conditions will prevail here. On the other hand, the northern locations do experience the effects of the constructed dam and hence must be taken into account. This is shown in Figure K.1a by the blue and orange stars respectively.

First, the caisson dam must be schematised by a (combination of) breakwater(s), depending on the possible wave propagation into the reservoir and around the caisson dam. For the right star, which represents the work front, the composition is shown in b). and c). in Figure K.1 and consists of a breakwater facing west with a northern open end and a breakwater facing south-west with a south-eastern open end. Here, waves will diffract from the tip of the breakwater and propagate into the sheltered area.

Depending on the incident wave angle relative to the breakwater configuration, the wave length and the location of the point of interest, the diffraction coefficient and direction will be determined with the digitized Goda diagrams shown in Figure K.3 and K.4, resulting in a diffracted wave height, period and direction. In the case where waves can diffracted from two ends of the breakwater, a bi-directional wave field will prevail: one from each end of the breakwater. For this situation the wave height will be determined by the square root of the sum of squares of the individual wave components:

$$H_{diff,total} = \sqrt{H_{diff,a}^2 + H_{diff,b}^2} \tag{K.1}$$

For the point of interest which lies outside the reservoir, the northern star, only waves that passes the caisson dam will contribute to this point. This situation can be schematised by one breakwater, shown in Figure K.1d. This procedure must be repeated for all other remaining wave directions, in order to obtain a new time series which consists of i). unaltered wave conditions for the cases where the locations are fully exposed and ii). diffracted wave conditions due to the sheltering function of the caisson dam.

The next step is to merge all data frames to one chronological time series, which can be analysed with Python and the workability tool Viktor. First, to give a visual impression of what the effects of the caisson dam are, Figure K.2 shows a scatter plot of the wave height as a function of wave direction for the northern outer point. Figure K.2a shows the initial undisturbed data and Figure K.2b shows the wave conditions when taking into account the diffraction of the caisson dam.

From this Figure various things can be noticed:
- i). a gap between approximately 110° and 240°,
- ii). a peak of waves around 220°,
- iii). a peak of waves around 245°.

First, looking at the orientation of the breakwater waves that originates from the south-east to south-west will be blocked entirely or at least strongly reduced. This also explains the gap seen in Figure K.2. Waves from the south-east have a direction of 112.5° - 157.5° and waves from the south-west have a direction of 202.5° - 247.5°. Therefore, the gap shows the effects of the presence of the caisson dam and its sheltering effects.

Next, two small peaks within the gap between 110° and 240° can be seen, which may be difficult to intuitively explain, as the last paragraph just explained why there are no waves originating from that particular area. Looking into diffraction mechanism for the waves from the south and south-west with the current representation of the caisson dam in more detail, one can find the reason.

For these two wave directions, the angle of incident waves relative to the breakwater (also denoted as Θ_w in Figure 7.6) is larger than 90°, which implies that waves do not directly penetrate into the sheltered area but only via transmission of the diffracted waves. According to Goda's diffraction theory, waves with an angle of incidence larger than 90° are transmitted as a new point source behind the breakwater, which results in roundly shaped wave pattern. This is illustrated in Figure 7.6: waves incident at an angle of > 90°, after encountering the breakwater they are transmitted in round shaped waves that penetrates into the sheltered area. When the observer is located behind the breakwater, the waves will originate from the tip of the breakwater and hence have a constant wave direction, denoted by ϕ_d in Figure 7.6.

Consequently, the peak of waves around 220° are the residue of all initial southern waves and are diffracted at the tip of the breakwater. Similarly, the peak of waves around 240° are the remaining of the south-western waves.

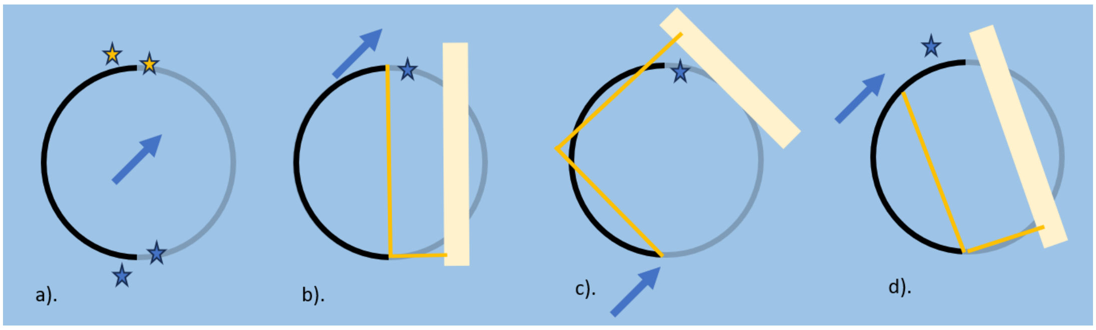
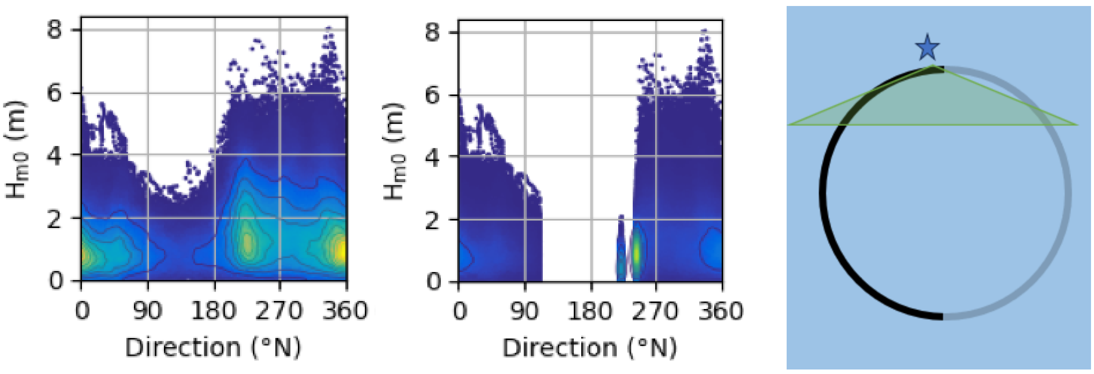


Figure K.1: Simplification of one particular scenario for a half finished caisson dam and south-western waves. Here the round dam can be schematised by multiple straight finite breakwaters, in order to apply Goda's diffraction diagrams. The orange stars indicate the work areas that experience improved wave conditions due to sheltering, whereas the blue stars are fully exposed for the given wave direction. Figure b). and c). represent the work area and c). represents the outer area.



(a) Undisturbed initial data. **(b)** Diffracted data. **(c)** Representation.

Figure K.2: Significant wave height H_{m0} as a function of direction for location **Outer** the a). undisturbed initial data and b). scenario when taking into account diffraction of the caisson dam. Direction is given relative to the true north, i.e. 0°/360° represents northern waves. c). Representation of the situation. Green area indicates the sheltering effects for which waves will be blocked or diffracted.

K.1. Goda's diffraction tables

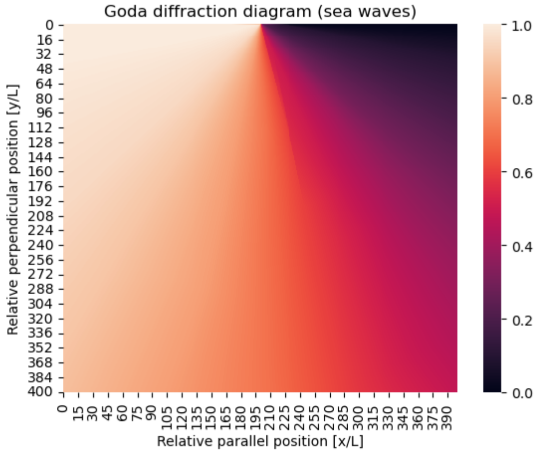


Figure K.3: Visualisation of the Goda diffraction diagram for sea waves. X- and y-positions are shown relative to the breakwater in terms of wave lengths.

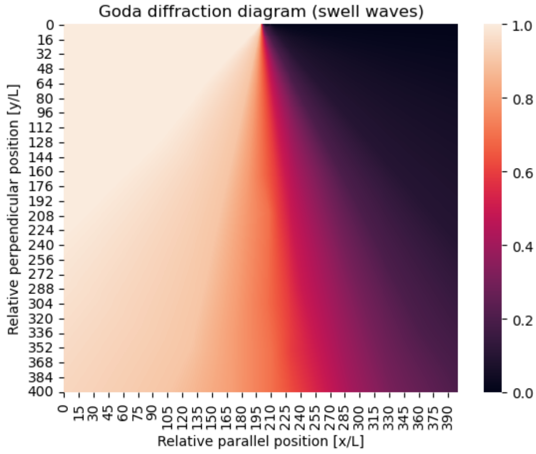


Figure K.4: Visualisation of the Goda diffraction diagram for swell waves. X- and y-positions are shown relative to the breakwater in terms of wave lengths.

L
Sheltering results

L.1. Workability Outer 2

Table L.1: Initial workability for undisturbed offshore conditions, considering a wave limiting threshold of $H_s = 2.5$ m.

WORK METHOD 2 - INITIAL - OUTER												
Base	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Year**
Average	71%	78%	84%	93%	96%	97%	97%	94%	88%	81%	77%	72%	86%
P10	90%	91%	95%	99%	100%	100%	100%	99%	96%	95%	91%	89%	89%
P20	85%	90%	94%	98%	99%	99%	100%	98%	95%	91%	86%	84%	88%
P30	80%	86%	90%	98%	98%	98%	100%	97%	93%	84%	84%	79%	87%
P40	78%	85%	88%	97%	97%	98%	99%	96%	91%	82%	81%	75%	87%
P50	73%	84%	85%	94%	97%	97%	98%	95%	89%	81%	78%	72%	86%
P60	68%	81%	83%	93%	96%	96%	97%	93%	88%	80%	75%	70%	85%
P70	63%	74%	80%	91%	96%	95%	95%	92%	86%	78%	70%	65%	84%
P80	61%	67%	75%	89%	95%	94%	94%	91%	84%	75%	63%	62%	83%
P90	51%	60%	71%	84%	93%	94%	92%	89%	78%	69%	61%	57%	83%

Table L.2: Workability for Scenario 1, considering a wave limiting threshold of $H_s = 2.5$ m.

WORK METHOD 2 - SCENARIO 1 - OUTER													
Scenario 1	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Year**
Average	73%	80%	85%	94%	96%	97%	97%	94%	89%	82%	77%	73%	86%
P10	92%	96%	97%	99%	100%	100%	100%	99%	96%	95%	91%	90%	90%
P20	89%	93%	95%	98%	99%	99%	100%	98%	95%	91%	87%	85%	90%
P30	82%	90%	93%	98%	98%	98%	100%	97%	93%	86%	86%	82%	88%
P40	79%	86%	90%	97%	97%	98%	99%	96%	92%	84%	82%	76%	87%
P50	74%	84%	88%	95%	97%	97%	98%	95%	89%	82%	80%	73%	86%
P60	68%	83%	83%	94%	96%	96%	97%	93%	88%	80%	76%	70%	86%
P70	64%	77%	81%	92%	96%	95%	95%	92%	86%	78%	71%	66%	85%
P80	61%	67%	76%	91%	95%	94%	94%	91%	84%	76%	65%	62%	84%
P90	53%	60%	71%	84%	93%	94%	92%	89%	82%	69%	62%	57%	83%

Table L.3: Workability for Scenario 2, considering a wave limiting threshold of $H_s = 2.5$ m.

WORK METHOD 2 - SCENARIO 2 - OUTER													
Scenario 2	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Year**
Average	79%	85%	89%	98%	99%	99%	99%	97%	94%	87%	85%	80%	91%
P10	94%	96%	98%	100%	100%	100%	100%	100%	99%	98%	96%	97%	94%
P20	92%	95%	96%	99%	100%	100%	100%	100%	97%	96%	93%	91%	93%
P30	89%	94%	95%	99%	100%	100%	100%	100%	96%	93%	91%	87%	92%
P40	85%	91%	93%	99%	100%	100%	100%	99%	95%	89%	90%	85%	91%
P50	81%	90%	92%	98%	99%	99%	100%	98%	94%	86%	87%	81%	91%
P60	78%	89%	88%	98%	99%	99%	99%	97%	94%	85%	85%	78%	90%
P70	70%	85%	86%	98%	98%	98%	98%	97%	93%	83%	82%	74%	90%
P80	68%	74%	84%	97%	98%	97%	98%	94%	91%	80%	76%	67%	89%
P90	62%	64%	76%	95%	96%	96%	95%	92%	89%	77%	72%	62%	88%

L.2. Workability Work, U = 7.9 m/s

Table L.4: Persistency for the caisson installation at the 'Work 2' location in terms of workable windows per month, yearly average and summation for work season lasting from April till September for base case. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 1 - INITIAL - WORK													
Base	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	15	17	24	31	35	37	39	36	26	18	14	14	26
P10	25	25	37	40	44	45	49	43	35	27	23	25	28
P20	21	24	30	37	40	44	44	41	34	23	21	21	27
P30	17	21	27	35	37	40	42	39	30	21	19	19	27
P40	15	19	25	33	36	38	41	39	29	19	17	17	26
P50	14	17	23	31	35	37	39	36	27	18	15	12	25
P60	12	15	22	30	33	35	37	33	25	15	12	10	25
P70	11	12	19	28	32	34	34	32	23	14	10	9	24
P80	7	11	17	26	29	32	33	30	20	13	9	7	24
P90	4	6	11	24	27	28	28	29	16	10	6	4	23

Table L.5: Persistency for the caisson installation at the 'Work 2' location in terms of workable windows per month, yearly average and summation for work season lasting from April till September for Scenario 2. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 1 - SCENARIO 1 - WORK													
Scenario 1	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	16	17	25	33	36	38	40	37	28	19	16	15	27
P10	27	25	37	40	44	47	49	45	36	28	24	26	29
P20	23	24	30	38	41	45	45	42	34	25	23	22	28
P30	19	22	28	35	37	42	43	40	31	23	20	20	28
P40	18	19	25	34	37	40	42	39	29	21	18	17	27
P50	16	18	24	32	36	38	40	37	29	19	16	14	26
P60	13	17	23	32	35	37	39	35	26	15	13	12	26
P70	11	13	21	30	32	35	37	33	25	14	12	10	25
P80	8	11	19	28	30	34	34	31	22	13	10	8	25
P90	6	7	13	25	29	29	29	29	19	11	6	6	25

Table L.6: Persistency for the caisson installation at the 'Work 2' location in terms of workable windows per month, yearly average and summation for work season lasting from April till September for Scenario 2. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 1 - SCENARIO 2 - WORK													
Scenario 2	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	16	18	25	33	35	38	39	37	28	19	16	16	27
P10	26	26	38	41	44	46	50	46	36	28	23	26	29
P20	23	24	32	37	40	44	45	42	34	25	22	23	28
P30	19	22	30	35	37	40	43	40	31	22	20	21	27
P40	17	20	26	33	36	39	41	39	29	20	18	19	27
P50	15	19	24	33	35	37	40	37	28	19	16	14	27
P60	14	16	23	31	34	37	37	34	27	18	14	12	26
P70	12	14	21	29	33	36	36	33	25	16	12	11	26
P80	8	12	19	28	30	35	34	31	22	13	11	9	25
P90	6	7	16	25	27	28	29	30	17	11	6	6	24

L.3. Workability Work, U = N/A

Table L.7: Persistency for the caisson installation at the 'Work' location in terms of workable windows per month and yearly average for the initial offshore conditions. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 1 - INITIAL - WORK													
Base	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	20	23	30	39	43	44	46	44	33	26	21	20	32
P10	33	35	45	50	49	50	55	51	43	38	31	31	35
P20	29	31	36	46	47	49	52	49	39	33	28	29	35
P30	24	29	33	43	46	47	49	47	39	30	27	27	34
P40	21	27	32	41	45	45	48	45	37	27	25	24	33
P50	18	26	30	38	43	44	47	44	34	25	21	20	33
P60	16	23	29	37	42	44	46	42	31	23	18	16	32
P70	16	19	26	35	40	42	43	39	29	22	15	15	31
P80	12	14	23	31	38	39	40	37	27	18	14	12	31
P90	7	8	18	29	35	35	36	36	20	13	10	7	30

Table L.8: Persistency for the caisson installation at the 'Work' location in terms of workable windows per month and yearly average for Scenario 1. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 1 - SCENARIO 1 - WORK													
Scenario 1	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	24	26	33	43	47	48	49	46	37	29	25	23	36
P10	37	39	48	52	55	55	59	55	47	41	37	34	39
P20	32	34	39	48	53	53	54	52	44	35	33	33	38
P30	29	31	37	46	51	51	52	50	41	33	30	29	37
P40	26	30	35	45	50	50	50	49	39	32	28	26	36
P50	23	28	33	44	49	47	49	47	37	31	26	22	36
P60	19	26	31	43	46	46	49	44	34	25	24	21	35
P70	17	23	29	40	44	45	47	43	33	24	19	18	35
P80	15	15	28	37	42	43	44	40	31	21	14	14	34
P90	11	9	22	35	39	39	39	37	28	18	14	11	33

Table L.9: Persistency for the caisson installation at the 'Work' location in terms of workable windows per month and yearly average for Scenario 2. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 1 - SCENARIO 2 - WORK													
Scenario 2	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	25	29	37	43	47	49	51	50	40	32	28	27	38
P10	38	40	49	51	51	53	57	58	49	41	39	37	40
P20	35	37	46	48	50	52	57	52	46	38	35	35	40
P30	29	35	40	46	49	51	55	52	43	36	32	32	39
P40	27	31	38	45	47	50	54	50	41	33	30	30	39
P50	24	30	36	43	47	50	52	50	41	32	29	28	38
P60	22	28	33	42	46	49	50	49	39	31	26	23	38
P70	21	25	32	40	45	47	50	47	37	28	22	21	37
P80	16	21	29	38	44	46	47	46	35	26	19	19	36
P90	12	16	26	36	41	43	42	44	31	22	17	14	36

M
Work Method Optimization

M.1. Work Method 2

Table M.1: Persistency for the caisson installation at the 'Work 1' location in terms of workable windows per month and yearly average for Scenario 3. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 2 - SCENARIO 3 - WORK 1													
Scenario 3	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	22	26	33	41	44	46	49	47	36	28	24	23	35
P10	35	37	46	50	51	51	57	55	47	39	34	35	38
P20	32	34	41	46	49	50	55	51	42	35	32	31	37
P30	26	32	37	44	47	49	53	49	40	32	29	27	36
P40	22	30	36	43	46	49	51	49	38	30	27	27	36
P50	21	28	35	41	45	47	49	47	36	28	25	25	35
P60	19	25	31	40	44	46	47	46	34	26	22	19	34
P70	18	23	29	38	42	46	47	45	33	25	19	18	34
P80	12	16	26	35	40	44	43	42	30	24	17	16	33
P90	11	12	22	32	38	39	39	38	25	17	12	11	33

Table M.2: Persistency for the caisson installation at the 'Work 1' location in terms of workable windows per month and for Scenario 4. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 2 - SCENARIO 4 - WORK 1													
Scenario 4	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	29	32	40	47	49	52	55	52	44	35	32	31	42
P10	41	43	50	55	56	56	60	59	50	43	42	42	44
P20	37	39	48	52	54	56	59	58	48	41	40	39	43
P30	35	37	45	50	52	55	58	56	47	39	37	36	42
P40	33	36	43	48	51	55	56	53	46	38	34	34	42
P50	29	33	42	47	49	54	55	53	44	36	32	30	42
P60	26	31	39	46	48	53	55	52	42	33	29	27	41
P70	23	29	35	45	47	51	53	50	41	32	28	25	41
P80	17	24	32	42	46	49	51	48	39	28	25	23	40
P90	16	16	30	41	43	46	50	44	38	27	18	21	40

Table M.3: Persistency for the caisson installation at the 'Work 2' location in terms of workable windows per month and yearly average for Scenario 3. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 2 - SCENARIO 3 - WORK 2													
Scenario 3	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	21	26	33	41	45	44	46	44	34	27	22	22	34
P10	34	38	45	52	53	51	56	51	45	39	32	33	37
P20	30	36	42	48	51	50	52	50	42	33	30	32	36
P30	28	32	36	47	47	48	50	47	39	30	28	28	35
P40	23	31	35	44	47	47	48	45	38	28	27	25	35
P50	18	29	33	42	46	45	47	45	35	26	24	22	34
P60	17	28	32	41	44	44	46	42	32	23	19	18	33
P70	16	21	30	35	42	42	44	39	31	22	17	15	32
P80	12	14	25	32	40	39	40	38	28	18	14	13	32
P90	8	8	18	29	36	35	36	36	22	16	10	7	31

Table M.4: Persistency for the caisson installation at the 'Work 2' location in terms of workable windows per month and yearly average for Scenario 4. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 2 - SCENARIO 4 - WORK 2													
Scenario 4	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	23	27	34	44	48	47	48	45	36	28	24	23	36
P10	37	40	47	53	57	54	58	54	46	41	35	35	39
P20	34	36	43	50	55	52	55	51	44	34	31	34	38
P30	29	34	38	48	52	51	51	49	41	32	29	28	37
P40	24	31	37	47	49	50	50	47	39	32	29	25	36
P50	21	30	35	45	48	48	49	46	37	29	25	22	35
P60	18	28	33	43	47	45	48	44	35	25	20	20	35
P70	16	24	31	41	43	44	46	41	32	23	19	16	34
P80	14	14	25	36	42	42	43	40	29	19	14	14	33
P90	10	9	21	33	39	38	38	37	25	16	12	9	33

M.2. Work Method 3

Table M.5: Persistency for the caisson installation at the 'Work 1' location in terms of workable windows per month and yearly average for Scenario 5. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 3 - SCENARIO 5 - WORK 1													
Scenario 6	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	22	25	33	41	45	46	49	46	35	29	24	22	35
P10	33	38	46	50	50	51	57	54	46	39	37	35	38
P20	33	35	41	48	49	50	54	51	43	36	31	31	37
P30	26	30	36	46	49	49	52	50	40	33	30	30	36
P40	23	29	34	44	47	49	51	48	38	32	28	28	35
P50	21	27	32	41	46	47	49	46	37	28	24	22	35
P60	20	24	31	38	45	46	48	44	35	27	20	19	34
P70	17	22	29	36	44	43	45	43	31	25	18	17	33
P80	16	16	24	33	42	42	43	40	29	20	16	13	33
P90	10	9	22	31	38	38	41	39	24	17	13	8	32

Table M.6: Persistency for the caisson installation at the 'Work 1' location in terms of workable windows per month and yearly average for Scenario 6. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 3 - SCENARIO 6 - WORK 1													
Scenario 7	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	40	39	45	47	51	51	55	55	47	43	40	41	46
P10	50	48	56	55	57	56	59	60	55	54	50	51	49
P20	48	45	53	53	54	55	58	60	54	50	47	48	48
P30	45	43	49	50	53	54	58	58	51	47	43	46	47
P40	43	41	48	49	52	54	57	57	50	45	42	44	47
P50	42	39	46	48	51	52	56	54	49	44	40	41	47
P60	40	36	44	47	51	51	55	52	47	40	37	38	46
P70	37	35	41	43	50	50	54	51	44	39	36	36	46
P80	32	33	39	42	47	49	52	50	42	37	35	34	44
P90	28	31	36	38	44	45	47	49	41	35	31	31	43

Table M.7: Persistency for the caisson installation at the 'Work 2' location in terms of workable windows per month and yearly average for Scenario 5. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 3 -SCENARIO 5 - WORK 2													
Scenario 6	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	22	25	33	41	44	46	48	46	35	28	23	22	34
P10	35	37	45	50	51	52	55	56	46	39	33	32	37
P20	32	34	40	47	49	50	54	51	42	34	30	30	36
P30	27	30	36	45	47	48	52	49	39	31	28	28	36
P40	22	30	34	42	46	47	51	48	38	30	26	26	35
P50	21	26	33	41	44	47	49	46	37	27	24	23	35
P60	19	24	31	38	43	46	47	44	33	25	20	18	34
P70	16	23	27	36	42	45	46	44	31	24	19	17	33
P80	12	16	25	35	40	42	43	39	29	23	16	15	33
P90	10	10	22	32	37	38	38	38	25	18	11	9	32

Table M.8: Persistency for the caisson installation at the 'Work 2' location in terms of workable windows per month and yearly average for Scenario 6. Threshold limits used here are limiting wave conditions as mentioned in Section 5 and neglecting wind limiting conditions. These results do not include the success rate of caisson installation.

WORK METHOD 3 -SCENARIO 6 - WORK 2													
Scenario 7	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average	35	36	44	47	50	52	55	55	47	41	36	37	45
P10	44	46	55	55	54	57	60	60	54	50	44	48	46
P20	42	43	51	52	53	54	59	58	52	45	43	43	46
P30	40	41	48	50	52	54	58	58	50	44	41	40	46
P40	39	39	45	49	51	53	58	56	49	42	39	39	45
P50	36	37	44	48	51	52	56	55	47	41	37	36	45
P60	33	35	43	46	49	51	56	54	46	39	33	33	45
P70	30	33	41	44	49	51	54	53	45	38	32	31	44
P80	27	28	39	43	46	50	52	51	44	37	31	30	43
P90	24	26	36	41	44	47	51	50	38	34	27	29	43

N

Overview: Levelised Cost of Storage (LCOS)

The following section will present the overview of the LCOS for the considered scenarios. First, Table N.1 considers the Base Case, Table N.2 represents Case 1 and includes sheltering and diffraction, Table N.3 shows the results for Case 2 considering Work Method 2 and lastly, Table N.4 lists the expenditures for Case 3 corresponding to Work Method 3.

Table N.1: Base Case: Extensive overview of all expenditures considering the baseline scenario related to the construction of the offshore energy island, including operational and stand-by costs for equipment, construction materials and risk costs.

Caissons installed	23	23	23	23	23	23	23	23	23	5	0	0			212	
Caisson remaining	189	166	143	120	97	74	51	28	5	0	0	0				
Material costs	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ -**	**€ -**			**€ 4,160,032,200**	
Stand-by costs	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 10,025,961**	**€ -**	**€ -**			**€ 425,100,740**	
Operational costs																
TSHD	€ 7,962,688	€ 7,962,688	€ 7,962,688	€ 7,962,688	€ 7,962,688	€ 7,962,688	€ 7,962,688	€ 7,962,688	€ 7,962,688.32	€ 1,731,019.20	€ -	€ -			€ 73,395,214	
FPV	€ 2,603,877	€ 2,603,877	€ 2,603,877	€ 2,603,877	€ 2,603,877	€ 2,603,877	€ 2,603,877	€ 2,603,877	€ 2,603,876.79	€ 566,060.17	€ -	€ -			€ 24,000,951	
SSDV	€ 4,850,550	€ 4,850,550	€ 4,850,550	€ 4,850,550	€ 4,850,550	€ 4,850,550	€ 4,850,550	€ 4,850,550	€ 4,850,549.54	€ 1,054,467.29	€ -	€ -			€ 44,709,413	
HLB	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 256,339	€ -	€ -			€ 10,868,786	
AHT	€ 247,387	€ 247,387	€ 247,387	€ 247,387	€ 247,387	€ 247,387	€ 247,387	€ 247,387	€ 247,386.56	€ 53,779.69	€ -	€ -			€ 2,280,259	
Towing tugs	€ 101,782	€ 101,782	€ 101,782	€ 101,782	€ 101,782	€ 101,782	€ 101,782	€ 101,782	€ 101,781.90	€ 22,126.50	€ -	€ -			€ 938,164	
Dock workers	€ 287,500	€ 287,500	€ 287,500	€ 287,500	€ 287,500	€ 287,500	€ 287,500	€ 287,500	€ 287,500.00	€ 62,500.00	€ -	€ -			€ 2,650,000	
Auxiliary equipment	€ 453,030	€ 453,030	€ 453,030	€ 453,030	€ 453,030	€ 453,030	€ 453,030	€ 453,030	€ 453,030	€ 98,485	€ -	€ -			€ 4,175,758	
Subtotal	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 3,746,292**	**€ -**	**€ -**			**€ 158,842,787**	
Risk costs	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 3,000,000	€ -	€ -			**€ 127,200,000**	
PV risk cost	€ 13,800,000	€ 13,142,857	€ 12,517,007	€ 11,920,959	€ 11,353,294	€ 10,812,661	€ 10,297,772	€ 9,807,402	€ 9,340,383	€ 1,933,827	€ -	€ -			**€ 104,926,163**	
O&M Costs	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -					€ 117,584,872	€ 117,584,872	**€ 10,347,468,736**	€ 1,291,629,671
Charging costs	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -					€ 72,546,584	€ 72,546,584	**€ 6,384,099,379**	€ 796,899,454
End of life cost														€ 120,000,000	**€ 120,000,000**	€ 958,166
															€ 16,978,768,115	
Electric discharge [MWh]													2.49E+06	2.49E+06	**2.19E+08**	2.73E+07
Year	0	1	2	3	4	5	6	7	8	9	10	11	. . .	99		
Investment cost	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 429,775,473**	**€ -**	**€ -**			**€ 4,743,975,727**	
															€ 16,978,768,115	
Present Value	€ 479,355,584	€ 456,529,127	€ 434,789,645	€ 414,085,376	€ 394,367,025	€ 375,587,643	€ 357,702,517	€ 340,669,064	€ 324,446,727	€ 277,037,102	€ -	€ -			**€ 3,854,569,811**	€ -

Table N.2: Case 1: Sheltering and diffraction included Extensive overview of all expenditures for Case 1 in which diffraction of the caisson dam improves the workability of the hinter-lying area. Relevant costs are related to the construction of the offshore energy island, including operational and stand-by costs for equipment, construction materials and risk costs.

Caissons installed	23	23	23	23	25	25	30	30	10	0	0	0			212	
Caisson remaining	189	166	143	120	95	70	40	10	0	0	0	0				
Material costs	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ -**	**€ -**			**€ 4,160,032,200**	
Stand-by costs	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 46,119,420**	**€ 53,214,715**	**€ 53,214,715**	**€ 15,373,140**	**€ -**	**€ -**	**€ -**			**€ 398,519,090**	
Operational costs																
TSHD	€ 7,962,688	€ 7,962,688	€ 7,962,688	€ 7,962,688	€ 8,655,096	€ 8,655,096	€ 10,386,115	€ 10,386,115	€ 3,462,038.40	€ -	€ -	€ -			€ 73,395,214	
FPV	€ 2,603,877	€ 2,603,877	€ 2,603,877	€ 2,603,877	€ 2,830,301	€ 2,830,301	€ 3,396,361	€ 3,396,361	€ 1,132,120.34	€ -	€ -	€ -			€ 24,000,951	
SSDV	€ 4,850,550	€ 4,850,550	€ 4,850,550	€ 4,850,550	€ 5,272,336	€ 5,272,336	€ 6,326,804	€ 6,326,804	€ 2,108,934.58	€ -	€ -	€ -			€ 44,709,413	
HLB	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 1,179,161	€ 1,281,696	€ 1,281,696	€ 1,538,036	€ 1,538,036	€ 512,679	€ -					€ 10,868,786	
AHT	€ 247,387	€ 247,387	€ 247,387	€ 247,387	€ 268,898	€ 268,898	€ 322,678	€ 322,678	€ 107,559.38	€ -	€ -	€ -			€ 2,280,259	
Towing tugs	€ 101,782	€ 101,782	€ 101,782	€ 101,782	€ 110,633	€ 110,633	€ 132,759	€ 132,759	€ 44,253.00	€ -	€ -	€ -			€ 938,164	
Dock workers	€ 287,500	€ 287,500	€ 287,500	€ 287,500	€ 312,500	€ 312,500	€ 375,000	€ 375,000	€ 125,000.00	€ -	€ -	€ -			€ 2,650,000	
Auxiliary equipment	€ 453,030	€ 453,030	€ 453,030	€ 453,030	€ 492,424	€ 492,424	€ 590,909	€ 590,909	€ 196,970	€ -	€ -	€ -			€ 4,175,758	
Subtotal	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 17,232,944**	**€ 18,731,461**	**€ 18,731,461**	**€ 22,477,753**	**€ 22,477,753**	**€ 7,492,584**	**€ -**	**€ -**	**€ -**			**€ 158,842,787**	
Risk costs	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 13,800,000	€ 15,000,000	€ 15,000,000	€ 18,000,000	€ 18,000,000	€ 6,000,000	€ -	€ -	€ -			**€ 127,200,000**	
PV risk cost	€ 13,800,000	€ 13,142,857	€ 12,517,007	€ 11,920,959	€ 12,340,537	€ 11,752,892	€ 13,431,877	€ 12,792,264	€ 4,061,036	€ -	€ -	€ -			**€ 105,759,430**	
O&M Costs	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -					€ 117,584,872	€ 117,584,872	**€ 10,465,053,608**	€ 1,357,105,328
Charging costs	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -					€ 72,546,584	€ 72,546,584	**€ 6,456,645,963**	€ 837,296,106
End of life cost														€ 120,000,000	**€ 120,000,000**	€ 958,166
Electric discharge [MWh]													2.49E+06	2.49E+06	**2.21E+08**	2.87E+07
Year	0	1	2	3	4	5	6	7	8	9	10	11	...	99		
Investment cost	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 479,355,584**	**€ 480,854,101**	**€ 480,854,101**	**€ 491,695,688**	**€ 491,695,688**	**€ 438,868,944**	**€ 416,003,220**	**€ -**	**€ -**			**€ 4,717,394,077**	
															€ 17,168,899,571	
Present Value	€ 479,355,584	€ 456,529,127	€ 434,789,645	€ 414,085,376	€ 395,599,859	€ 376,761,770	€ 366,910,893	€ 349,438,946	€ 297,043,776	€ 268,159,385	€ -	€ -			**€ 3,838,674,361**	

Table N.3: Case 2 - Work Method 2: Extensive overview of all expenditures considering the Work Method 2 related to the construction of the offshore energy island, including operational and stand-by costs for equipment, construction materials and risk costs.

Caissons installed	46	46	54	58	8						0	0			212	
Caisson remaining	166	120	66	8	0	0	0	0	0	0	0	0				
Material costs	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ -**	**€ -**			**€ 4,160,032,200**	
Stand-by costs	**€ 82,859,559**	**€ 82,859,559**	**€ 95,607,183**	**€ 101,079,111**	**€ 26,494,343**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**			**€ 388,899,754**	
Operational costs																
TSHD	€ 31,850,753	€ 31,850,753	€ 37,390,015	€ 40,159,645	€ 2,769,631	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 144,020,797	
FPV	€ 5,207,754	€ 5,207,754	€ 6,113,450	€ 6,566,298	€ 905,696	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 24,000,951	
SSDV	€ 19,402,198	€ 19,402,198	€ 22,776,493	€ 30,579,551	€ 1,687,148	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 93,847,589	
HLB	€ 4,716,643	€ 4,716,643	€ 5,536,929	€ 5,947,071	€ 410,143	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 21,327,429	
AHT	€ 989,546	€ 989,546	€ 1,161,641	€ 1,247,689	€ 86,048	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 4,474,470	
Towing tugs	€ 407,128	€ 407,128	€ 477,932	€ 513,335	€ 35,402	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 1,840,925	
Dock workers	€ 575,000	€ 575,000	€ 675,000	€ 725,000	€ 100,000	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 2,650,000	
Auxiliary equipment	€ 1,812,121	€ 1,812,121	€ 2,127,273	€ 2,284,848	€ 157,576	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 8,193,939	
Subtotal	**€ 63,149,022**	**€ 63,149,022**	**€ 74,131,460**	**€ 85,738,590**	**€ 5,994,067**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**			**€ 292,162,161**	
Risk costs	€ 27,600,000	€ 27,600,000	€ 32,400,000	€ 34,800,000	€ 4,800,000	€ -	€ -	€ -	€ -	€ -	€ -	€ -			**€ 127,200,000**	
PV risk cost	€ 27,600,000	€ 26,285,714	€ 29,387,755	€ 30,061,548	€ 3,948,972	€ -	€ -	€ -	€ -	€ -	€ -	€ -			**€ 117,283,990**	
O&M Costs	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -					€ 117,584,872	€ 117,584,872	**€ 10,935,393,096**	€ 1,653,424,005
Charging costs	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -					€ 72,546,584	€ 72,546,584	**€ 6,746,832,298**	€ 1,020,116,459
End of life cost														€ 120,000,000	**€ 120,000,000**	€ 958,166
															€ 17,929,425,394	
Electric discharge [MWh]													2.49E+06	2.49E+06	**2.31E+08**	3.50E+07
Year	0	1	2	3	4	5	6	7	8	9	10	11	...	99		
Investment cost	**€ 562,011,800**	**€ 562,011,800**	**€ 585,741,863**	**€ 602,820,921**	**€ 448,491,630**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ -**	**€ -**			**€ 4,841,094,115**	
															€ 22,770,519,509	
Present Value	€ 562,011,800	€ 535,249,334	€ 531,285,137	€ 520,739,377	€ 368,975,174	€ 325,949,408	€ 310,428,008	€ 295,645,722	€ 281,567,354	€ 268,159,385	€ -	€ -			**€ 4,000,010,698**	€ -

Table N.4: Case 3 - Work Method 3: Extensive overview of all expenditures considering the Work Method 3 related to the construction of the offshore energy island, including operational and stand-by costs for equipment, construction materials and risk costs.

Caissons installed	46	46	64	56							0	0			212	
Caisson remaining	166	120	56	0	0	0	0	0	0	0	0	0				
Material costs	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ -**	**€ -**			**€ 4,160,032,200**	
Stand-by costs	**€ 82,859,559**	**€ 82,859,559**	**€ 148,249,363**	**€ 95,607,183**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**			**€ 409,575,663**	
Operational costs																
TSHD	€ 31,850,753	€ 31,850,753	€ 44,314,092	€ 38,774,830	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 146,790,428	
FPV	€ 5,207,754	€ 5,207,754	€ 7,245,570	€ 6,339,874	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 24,000,951	
SSDV	€ 19,402,198	€ 19,402,198	€ 33,742,953	€ 23,620,067	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 96,167,417	
HLB	€ 4,716,643	€ 4,716,643	€ 6,562,286	€ 5,742,000	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 21,737,571	
AHT	€ 989,546	€ 989,546	€ 1,376,760	€ 1,204,665	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 4,560,518	
Towing tugs	€ 407,128	€ 407,128	€ 566,438	€ 495,634	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 1,876,327	
Dock workers	€ 575,000	€ 575,000	€ 800,000	€ 700,000	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 2,650,000	
Auxiliary equipment	€ 1,812,121	€ 1,812,121	€ 2,521,212	€ 2,206,061	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			€ 8,351,515	
Subtotal	**€ 63,149,022**	**€ 63,149,022**	**€ 94,608,099**	**€ 76,877,070**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**	**€ -**			**€ 297,783,212**	
Risk costs	€ 27,600,000	€ 27,600,000	€ 38,400,000	€ 33,600,000	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			**€ 127,200,000**	
PV risk cost	€ 27,600,000	€ 26,285,714	€ 34,829,932	€ 29,024,943	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -			**€ 117,740,590**	
O&M Costs	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -					€ 117,584,872	€ 117,584,872	**€ 11,052,977,968**	€ 1,736,989,378
Charging costs	€ -	€ -	€ -	€ -	€ -	€ -	€ -	€ -					€ 72,546,584	€ 72,546,584	**€ 6,819,378,882**	€ 1,071,673,961
End of life cost														€ 120,000,000	**€ 120,000,000**	€ 958,166
															€ 18,119,556,850	
Electric discharge [MWh]													2.49E+06	2.49E+06	**2.34E+08**	3.68E+07
Year	0	1	2	3	4	5	6	7	8	9	10	11	…	99		
Investment cost	**€ 562,011,800**	**€ 562,011,800**	**€ 658,860,682**	**€ 588,487,473**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ 416,003,220**	**€ -**	**€ -**			**€ 4,867,391,076**	
															€ 22,986,947,926	
Present Value	€ 562,011,800	€ 535,249,334	€ 597,606,061	€ 508,357,605	€ 342,246,879	€ 325,949,408	€ 310,428,008	€ 295,645,722	€ 281,567,354	€ 268,159,385	€ -	€ -			**€ 4,027,221,555**	€ -