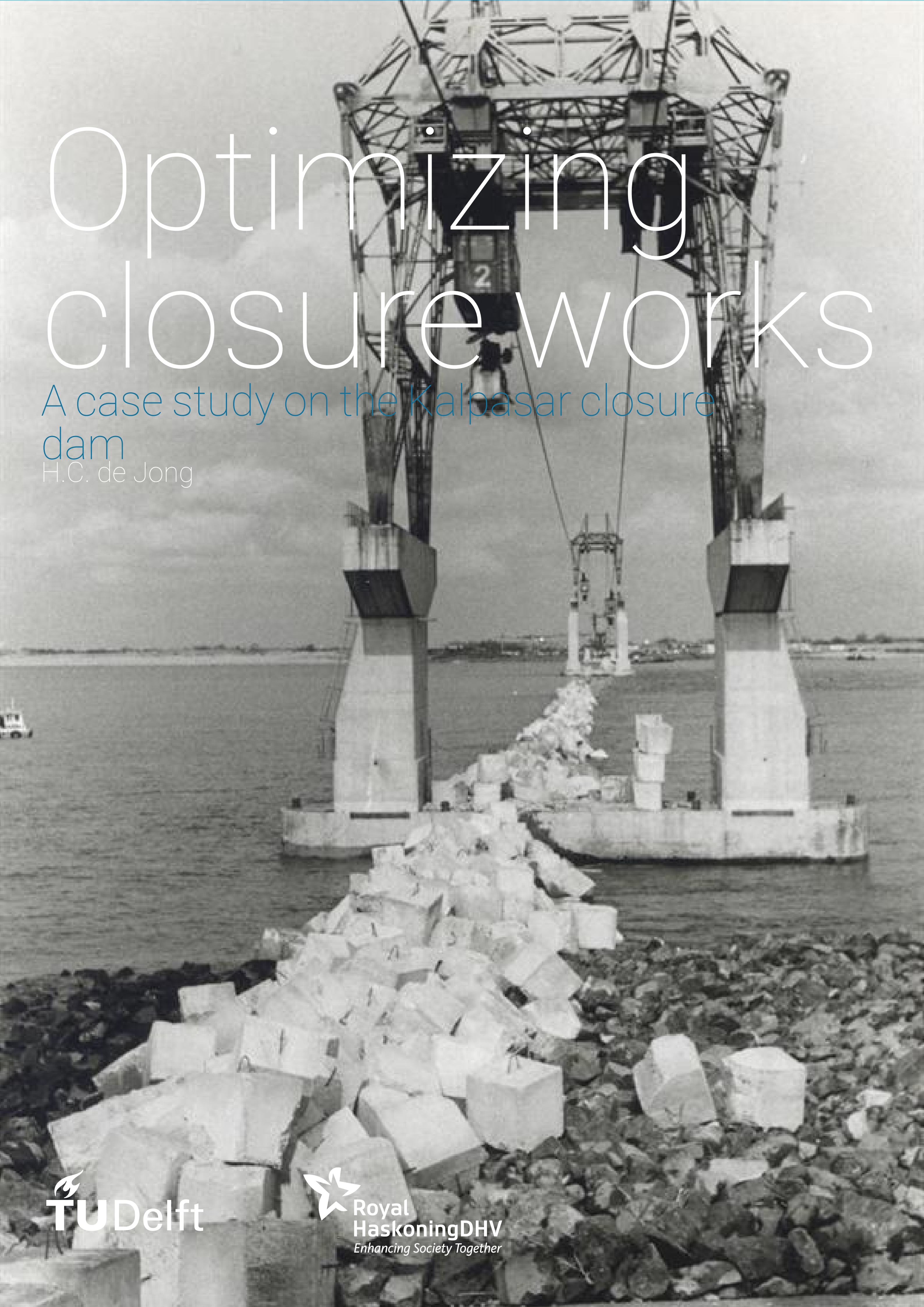
Optimizing closure works

A case study on the Kalpasar closure dam

H.C. de Jong



TU Delft

Royal HaskoningDHV
Enhancing Society Together

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by

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Project duration:	October, 2017 - October, 2018
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Preface

This thesis marks the finish of the Master of Science program in Hydraulic Engineering at the Delft University of Technology. The research was conducted in partnership with Royal HaskoningDHV, one of the leading engineering and consultancy firms in the Netherlands.

The research in this thesis revolved around the closure of the Gulf of Khambhat in India known as project Kalpasar, which aims at the creation of the world's largest fresh water reservoir at sea. In 1998, Royal Haskoning was involved in the pre-feasibility studies. However, the project was never realized. Twenty years later, they set up an interdisciplinary research team of master students involving three different universities to revive the project. I can proudly say that i was one of the team members involved. Although the course of this project took some large unexpected turns, the project team stood strong during the process and part of the result is presented by this thesis.

My personal fascination with Hydraulic Engineering was never so clear at the start of my studies. However during the master degree, the pieces of the puzzle fitted perfectly; My love for water, my creativity in designs and my long interest in practically applied solutions based on mathematics and physics all came together during this enriching program to become a Hydraulic Engineer. Working directly with problems such as climate change to help society to a better future creates motivation and generates priceless rewards.

I would like to thank the University of Delft for this amazing educational journey, both theoretical and practical. Secondly, I would like to thank Royal HaskoningDHV for educating me on the practical side of Hydraulic Engineering by providing a stimulating work environment during my internship and graduate internship. Special thanks go out to my supervisors Jeroen, Erik, Leslie and Bas for their critical views on my work and directing this research to a good end.

Personally, I would like to thank all my friends in- and outside of civil engineering for all the good times, conversations and support. They have helped me find out who I am and who I want to be. Furthermore, I would like to thank my father for supporting me both mentally and financially, especially during the last years. The unfortunately loss of my mother during this journey was unimaginable. However, her strength endures in me and has motivated me to enjoy life an make the best of it. I want to thank her for always believing in me and sending me care-free into the world to make my own mistakes.

Finally, I want to thank Anna, who I met during this amazing journey. She supported me through the complete process which softened the sometimes harsh environment of reality. I have learned so much about myself from her and I hope to learn much more in the future!

H.C. de Jong
Delft, October, 2018

Abstract

Constructing a dam across a tidal basin has alway been a long-term integral solution to many water related problems of the surrounding area such as flooding, river control and fresh water storage. However, immense challenges are accompanied with the closure works of large basins. This research treats the closure strategy to close the Gulf of Khambhat in India. The project is known as "Kalpasar", which aims to create of a fresh water reservoir in the Gulf of Khambhat by constructing a 35 km dam across the estuary. The Kalpasar project has been on the Indian Governments agenda since 1986. Royal Haskoning was involved in the pre-feasibility study, which was presented in 1998. However, due to an alignment change to a more northern position, earlier proposed closure work designs are now considered out of date.

To avoid irrelevance of this research through time and assist the Kalpasar development project with optimizing a new design for the closure works, this research treats the development of a fundamental parametric optimization tool to quickly perform a first-order evaluation of possible closure strategies on costs. The tool as a product along with case results are delivered to the Kalpasar development project for further design optimization.

Closing the tidal basin involves closing a certain wet cross section along the chosen dam alignment through which large tidal currents penetrate caused by tidal differences up to 11 m. Complexity is caused by increasing tidal flow velocities due to increasing constriction of the wet cross section during the closure.

The developed optimization tool can evaluate and compare six pre-programmed strategies to close a multi-sectional wet cross section in time on costs of three fundamental design requirement or "cost factors": Required *dam material*, *bed protection* and *equipment*. Using a multi-sectional storage model to compute the flow velocities in the gap, the channels and tidal flats can be individually modeled after which they are linked as a system. The model reacts as a system to changes in flow area by closing certain cross sections (a channel or a tidal flat). The individual cross sections can be closed strategically by defining their *closure method* (horizontal, vertical or sudden), *execution phase* and *construction capacity*. These are called "strategic input parameters". Defined for all sections, they determine the closure sequence of the system in time. Optimization is achieved when the strategic input parameters define a closing sequence which minimizes the combined cost of all cost factors.

Subsequent to the storage model, three computational models are introduced to quantify the required dam material, bed protection and equipment. Based on earlier research, the material model utilizes only quarried rock for gradual closures and sluice caissons for sudden closures. The equipment model utilizes large dump trucks for horizontal closures and ships or a temporary cable-way/bridge system for vertical closures. The construction capacity is linked to material and bed protection models, since both design requirements are time dependent. Increasing construction capacity can therefore decrease these requirements.

Since subsequent models largely depend on the flow velocity, an attempt to validate and calibrate the storage model was performed using results from previous research and a 2D-H Delft3D model. Deviations with respect to the Delft3D model were significantly large (factor 2-3), because storage models can only be utilized if the basin size and the remaining gap are small (usability limits). Therefore, calibration was performed by introducing an artificial contraction factor to compensate for the error in the flow velocity. An exponential relation was determined linking the error to the constriction percentage of the gap. With increasing constriction percentage, the error decreased due to increasing validity of the storage model usability limits. The artificial contraction factor can be used to optimize the closure of the Gulf of Khambhat. However, for general use, the model should be calibrated to each specific site.

Case study results show that using multiple cross sections to model the bathymetry with respect to a single cross section, the optimal strategy can change from fully vertical to a combination of horizontal and vertical with a specific capacity. Utilizing the developed model for the Kalpasar case is therefore recommended because the complex bathymetry creates many possible strategies and can't be reliably modeled with single cross-sectional models. The strategy that showed the most potential for further optimization is: First closing the tidal flats horizontally by forward dumping of rocks, while closing the channels up to 40% of their depth with dumping ships after which the remaining gap is closed vertically by a cable-way or bridge system. This strategy is commonly suggested by existing literature, thereby increasing reliability and validity of the optimization model.

A second case study showed negative effects of increasing construction capacity on the total cost. However, these case results are based on assumed costs and cost functions for equipment, which should be verified by contractors first. Bed protection requirements did decrease significantly by increasing construction capacity, showing potential for development of high capacity closure equipment to avoid these costs. Further future development should focus on vertical closure equipment to decrease both material and bed protection costs.

To conclude the recommendations, more case studies should be performed to quantify influences of parameters already included in the model, such as the permeability of the dam, the presence of a tidal power facility and the use of a sudden caisson closure to relieve the final closure. Secondly, further validation of the storage model is essential to generate more reliable results. Furthermore, research should be performed into cost functions of several existing or new high capacity equipment for vertical closures, relating costs to construction capacity to improve usability of the optimization model.

Contents

1

Introduction

1.1. Background

1.1.1. General introduction

The construction of a closure dam across a tidal basin can be done for a large range of purposes, such as protection from the sea or creating a fresh water basin for water conservation. In any tidal basin closure, the impact on the surrounding environment is extensive. A thorough study of these impacts is always necessary to avoid unknown risks. Although, the impact of a closure dam might be extensive, to construct the dam in the first place has been a challenge for centuries. The construction of a closure dam is called the closure works. Every closure operation in a tidal basin is a struggle with or against the forces of nature. Any action executed in the closure process is counteracted by nature itself. The flowing water from the tides constantly change in this intrinsic dynamic process. (K. d'Angremond, 2004)

From the early twentieth century, increasingly larger projects required new science and practical models for hydraulic engineers and contractors. In the Netherlands, the construction of a 30 km long closure dam called the 'Afsluitdijk' (figure 1.1) lead to new ways to perform tidal computations and compute the flow during a closure sequence. Research was performed with regard to flow and erosion patterns during closure events lead by professor Rehbock from Karlsruhe university in the 1930's (De Blocq van Kuffeler, 1950). From thereon knowledge of closure works increased through the many studies and laboratory experiments. This led to the successful closure operations required for the Deltaworks in 1986 (Huis in 't Veld, 1987). At this moment, the Afsluitdijk has been surpassed in length by the Seamunguem dam in Korea. This 31 km dam required complicated closure works where flow velocities in the final gap reached up to 7 m/s due to an tidal range up to 3 m. The knowledge gained from all experiences has led to many other closures projects currently planned around the world. Nevertheless, the changing conditions during execution of closure works are often difficult to predict. Flexibility in operations that is incorporated in the design provides an important tool (K. d'Angremond, 2004).

Figure 1.1: Final closure works: Afsluitdijk (Beeldbank Rijkswaterstaat, 2018)

Figure 1.2: Gulf of Khambhat, west coast of India

This research treats the closure of the Gulf of Khambhat in India (figure 1.2), which aims at the creation of a fresh water reservoir in the Gulf by partly closing the current estuary with a dam. Immense challenges are accompanied with the closure works of this basin. The tidal difference at some locations is measured at 11 meters (figure 1.3) which generates high flow velocities of 2-3 m/s in the basin in its natural unconstricted condition. The initial length would be around 53 km, enclosing the largest river estuary in the Gulf, the Narmada (figure 1.4), as well. Depths reach up to 40 meters below Lowest Astronomical Tide (LAT) which leads to dam heights of up to 53 meters (Royal Haskoning, 1998b). This incredible project called the Kalpasar project functions as case study throughout this research.

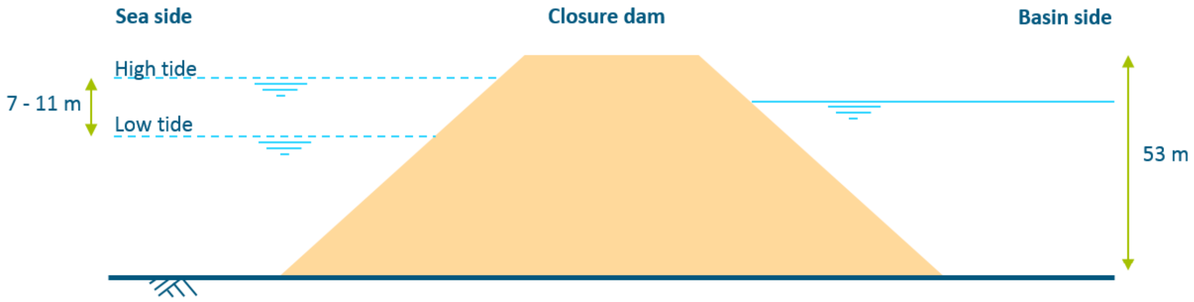


Figure 1.3: Dimensions cross section closure dam with tidal levels

1.1.2. The Kalpasar project

Gujarat and the Gulf of Khambhat Gujarat is an important state in the Western part of India at the Arabian coastline. Gujarat has the highest agricultural growth rate of 10.97% every decade which on average is about 2% in India(Bussiness Standard, 2013). The current Chief Minister is Vijay Ramniklal Rupani who succeeded Anandiben Patel in August of 2017. Up until 2007 Naramanda Modi (the current prime minister) was Chief Minister of Gujarat. (Times of India, 2013)

Figure 1.4: Gulf of Khambhat on the West coast of India. Current proposed position of the dam with optional small tidal energy basin (Kersten, 2018)

The state surrounds the Gulf of Khambhat, a large tidal basin about 80 km in length and 30 to 50 km in width (figure 1.4). During low tide a large part of the land runs dry and the gulf is very shallow with shoals and sandbanks. During flood period, the water rushed in with large velocities due to the high tidal differences of 8 to 11 meters. The inlet is of utmost importance to the export of central Gujarat, Ahmedabad and Broach (Britannica, 1994). A port is located at Bhavnaghar and large jetties for liquid bulk at Dahej. They are vital for in- and export.

One of the main problems of the state is severe water deficiency. The state occupies 6.4% of the total land area in India and it houses 5% of the total population (statoids, 2017).However, the state receives only 2% of India's surface water resources. The water availability per capita per year is around 990 cubic meter, while a minimum of 1700 cubic meter is required (Kalpasar Department , 2017).

Gujarat spends nearly 40% of its domestic energy generation to extract sufficient groundwater to meet its freshwater requirements On average a volume of 38,000 MCM (Million Cubic Meters) of surface water is yearly available and only 20,480 MCM (54%) can be stored (Kalpasar Department , 2017). This includes the storage capacity of the Sardar Sarovar Dam located 150 km upstream the Narmada river which was opened in September 2017 (Indian Express, 2017). 46% of the surface water availability drains into the Gulf at this moment and there is no suitable site to store volumes of water of this magnitude in the state of Gujarat other than to store it in a closed off part of the Gulf of Khambhat (Kalpasar Department , 2017). For this reason, the Gulf of Khambhat Development Project was initiated. One component of this project envisaged a large dam across the Gulf of Khambhat to close off the basin. The dam would provide a large fresh water basin and a road across the Gulf, significantly reducing distances between the region of Saurashtra and the southern part of Gujarat (figure 1.4). Furthermore, an optional small tidal basin was added in the design to generate energy (figure 1.4). The project is called Kalpasar which means: *"A lake that fulfills all wishes"* and has been on the agenda of the Gujarat government since 1986.

Project history and current status Royal Haskoning was involved in the pre-feasibility study, which was presented in 1998. Since then, the existence of a full feasibility report is still not confirmed and the status of the announced feasibility studies by the Indian government is unknown.

An Expert Advisory Group (EAG) consisting of international experts, has been installed to guide and coordinate the preparation of the feasibility report. They recommended carrying out more studies with respect a new dam alignment which involved the construction of a Narmada Diversion Canal and a Barrage near Bhadbhut both visible in figure 1.4. The last confirmed work on the Kalpasar project was documented in the 2012 meeting of EAG. The report highlights the confirmed new alignment and adjustments (Expert Advisory Group, 2012). However, Royal HaskoningDHV suspects no actual progress on the studies of these adjustments since the deliverance of the pre-feasibility study in 1998.

In 2017, Royal HaskoningDHV decided to initiate a cooperation with three Dutch universities (Delft University of Technology, University of Amsterdam and University of Maastricht) to set up an interdisciplinary research team to revive the project with new insights and techniques. The objective of the team is to develop a business case which can generate attention to the project by identifying opportunities and threats and attract investors.

This thesis is part of the interdisciplinary research and focuses on the closure works with the overall goal to decrease capital cost by optimizing the design. The scope is elaborated in section 1.3 of this chapter. The main driver behind this research is Royal HaskoningDHV. However, the World bank involvement in the Kalpasar project caused potential interest in the new research team and therefore they partly support the project as well.

1.1.3. Dam and closure works design

Design Royal Haskoning 1998 With the finalized prefeasibility report in 1998, Royal Haskoning had presented a complete dam design (Royal Haskoning, 1998a). The design proves the technical feasibility and was analyzed by a large project team at Royal Haskoning for several years.

In figure 1.5, the old proposed dam alignment with the tidal energy basin and Narmada spillway (in case of overflow) is depicted. As mentioned before, one of the most challenging aspects of the design is the closure.

Figure 1.5: Design overview of dam design RHDHV 1998 (Expert Advisory Group, 2009)

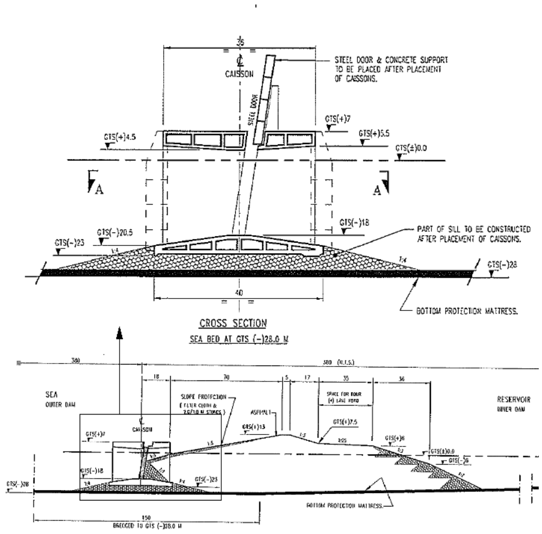


Figure 1.6: Design of sluice caissons sudden closure proposed by Royal Haskoning (Royal Haskoning, 1998a)

On this aspect, Royal Haskoning proposed expensive sluice caissons for the final closure gap (figure 4.15). Since the technical feasibility in this design was the most important thing to prove; only known and proven closure techniques were considered. The sluice caissons were technically feasible; however, large risks and high costs are directly related to this option.

The total cost of the tidal closure without the tidal energy component is estimated around 20.000 crore Indian Rupees (RS). A crore indicates the number 10 million, so total dam costs where about 200 billion RS, which was about 2.7 billion euro's in 1998 and around 11.5 billion euro's in 2018 with an average interest rate of 8% (F.R.E.D., 2018). Including the tidal energy component, the total costs are estimated around 53,000 crore RS, which amounts to about 30 billion euro in 2017 (Royal Haskoning, 1998a). The final closure was estimated at 7,500 crore RS by Royal HaskoningDHV in 1998 (38% of total dam costs). This includes work harbors, bottom protection and construction pits for caissons.

Furthermore, the deepest dam section in the middle of the closure gaps is the second most expensive element at 2,000 crore RS (10%). In third place, the spillway is also an expensive component with 1,070 crore RS (5%) (Royal Haskoning, 1998a). In appendix A, a complete overview of this design can be found.

Design Broos and Wiersema 1998 In 1998, the Royal Haskoning closure works design was re-evaluated in a master thesis by former master students E.J. Broos and K.J. Wiersema, because it was highly qualitative design, but too expensive. Together they analyzed cheaper possibilities to realize the closure. The result was to include the tidal power facility in the closure operation. This could significantly reduce current velocities during the final closure. Furthermore, they evaluated two promising closure techniques: (Wiersema and Broos, 1998).

- A rock dumping railway bridge which dumped conventionally sized rocks or uses an overkill closure with smaller rocks
- A sand-geotextile structure in the shape of a large caisson which they called a superbag.

Broos recommended investigating the final closure techniques in more depth. The design from Royal Haskoning is also known as a typical Dutch design. However, Broos and Wiersema also evaluated the technical feasibility of executing the closure works in India, taking into account the use of local materials, construction techniques and construction capacities (Wiersema and Broos, 1998).

1.2. Problem statement and relevance

The closure of the Gulf of Khambhat is an enormous design study comprising of a strategy for the design and execution of the closure dam. In 1998, a complete design was presented by Royal Haskoning and engineers Broos and Wiersema have tried to improve this design by introducing new strategies for the expensive caisson closure of the final closure gap. They focused on radical designs and execution possibilities to significantly reduce closure cost. Studies from Royal Haskoning resulted in an alignment choice presented in appendix A. However, in 2009, this alignment changed to a more northern alignment. The new alignment requires a new closure strategy and the reports made by Royal Haskoning and Broos and Wiersema are considered outdated and the design of closure works can't be used anymore as optimal closure strategy.

At the moment, research into the Kalpasar case is relevant because the location has changed, new techniques are developed, other materials are available and computing techniques have been improved. Providing a complete new detailed design is possibly irrelevant, since there is a high probability that this design becomes outdated as well. This research requires a structurally different approach to the problem by trying to eliminate the negative effects that large infrastructural projects often create; detailed designs are often outdated when execution starts due to outdated data and overall changes in project design requirements.

This problem especially circumcises the Kalpasar case, since numerous costly investigations have been executed and until now no designs have been realized. The design requirements change for example due to increasing water and power shortage. Other reasons are the increasing insight in environmental impact and social and economic feasibility. Even if a design is presented, the Gulf of Khambhat is a highly dynamic basin in which many design conditions change rapidly. At the moment, research into the Kalpasar case should be focused on this highly dynamic project environment and should be robust enough to governmental and environmental changes. To again quote d'Agremond from the general introduction (section 1.1.1): "Flexibility in operations that is incorporated in the design provides an important tool "(K. d'Angremond, 2004).

Currently, there is a need for digital tools to quickly evaluate the best strategy to base further designs on. Such tools have already been widely developed for breakwater designs (Vos, 2016). Even the use of artificial intelligence will become interesting in the future to evaluate optimal designs. These tools can be used for quick evaluation and generate insight into important design aspects. Therefore, developing such a tool seems to be very relevant for research into the Kalpasar case. As stated before, a general tool can also be used to give insight into important design aspects.

Conclusions by Broos and Wiersema showed relevant out of the box ideas that could increase project feasibility future. Such ideas can be generated by first finding the design requirements for the future. This mean that first relevant engineering aspect with respect to large tidal closures should be identified which can potentially serve as a basis for new ideas.

From an economic point of view this is called supply and demand. When the demand is identified, the supply will follow. Since the project is very large and expensive, almost anything can be done to realize it. This has for example been done for a part of the Brouwersdam closure (Deltawerkenonline, 2015). Special equipment was designed to execute the closure of this basin when it was identified that a certain closure method was the optimal strategy. In this case, it was respectively a cable-way to close the basin vertically.

To **summarize** the problem and the relevance of the research:

- The closure works are the most expensive part of the dam design.
- Earlier proposed closing strategies are out of date and became irrelevant due to design and environmetal changes.
- Further research faces the same threat of irrelevance through time.
- A general optimal strategy is unknown, so it is unknown which design aspects are the most relevant for future development.

If research should be performed on the Kalpasar case it should involve a more general approach to avoid irrelevance through the passing of time and it should identify important design aspects to help focus future developments, thereby improving the feasibility.

1.3. Research objective and methodology

1.3.1. Objective

In order to assist the Kalpasar development project with the task to design the optimal closure works with respect to cost, it is required to evaluate numerous closure strategies. Increasing the number of evaluated options creates a larger assurance the least costs closure strategy is actually the optimal strategy. In order to perform quick evaluations on closure strategies and to understand the relevant topics in the process and future demands in the design of the closure works it is therefore attempted to create a general and robust model to assist in choosing the optimal closure works. This with regard to the changing conditions with respect to the surroundings but also the political climate. The research objective is formulated as follows:

Assisting the Kalpasar Development Project in optimizing a closure strategy and identifying key design considerations, by creating a general and robust tool to perform a first-order evaluation on cost regarding possible closure strategies.

1.3.2. Research questions

Based on the research objective, the following research questions and subquestions are defined:

1. *"How to develop a **robust method** to **optimize** the closure strategy?"*

    (a) *What are important cost factors that should be included in a first order evaluation of a closure strategy? and which physical processes should be included to quantify these cost factors?*

    (b) *Can all cost factors be quantified using these processes and (how) can they be combined into a reliable overall first-order evaluation of a closure strategy?*

    (c) *For each cost factor: How do different strategic input parameters affect that cost factor or its related process?*

    (d) *How do the different cost factors affect the overall evaluation?*

2. *"Which **design considerations** within the Khambhat closure case are relevant for **future optimization of the closure strategy** to increase project feasibility?"*

Below, a clarification of highlighted words is given to specify the questions.

Robust method	A method that is robust to future technical developments.
Optimize	A method/tool to find an optimal closing strategy for a given (single) dam alignment on the criteria of cost
Design considerations	consderations of design aspects (such as parameters, materials and design conditions) and execution aspects (such as cost and availability of equipment, labor forces and materials) within the engineering field of closure works.
Future optimization	The project is not being designed in full detail at the moment, so there is still time to develop certain engineering aspects before realization.

1.3.3. Methodology

The project is divided into three parts for which the corresponding chapters are projected on the right:

1. Review of literature regarding closure strategies of large tidal basins.	chapter 2
2. The development and validation of a strategy optimization tool (question 1).	chapters 3, 4 and 5
3. Performing case studies to identify relevant design considerations (question 2).	chapter 6

All three parts are elaborated in the following paragraphs.

Part 1 During the first part a review of the existing literature on closing strategies is executed. In this part the minimally required criteria, design parameters and design relations for a well-founded optimization are assessed. Interesting design aspects are the determination of the flow velocity in time during the closure sequence, material stability of several conventional materials, scour hole development to asses bottom protection requirements and finally, the type of closure methods available. For practical information, several reference projects can be used. The literary study is presented in chapter 2.

Part 2 During the second part, a general tool is developed to assess the optimal closing strategy in the Gulf of Khambhat for a given single dam alignment. This tool is required to evaluate all different possibilities and evaluate the optimal closure strategy by comparison on design criteria; mainly cost, and risk. The case study uses the dam alignment chosen by the government in 2009. An answer to research question 1 is given when the tool operates and is validated on flow only, since this is the most important design parameter during closures. Furthermore, it requires to minimally taking the following design aspects into account for all situations in the Gulf of Khambhat:

- Dam dimensions
- All possible closure methods
- Execution phasing
- Materials
- Bottom protection
- Equipment and construction capacity
- Costs

In chapter 3, the resulting method for optimization is presented along with the scope of the method. The optimization model including all above-stated aspects is presented in detail in chapter 4. The optimization model is validated an calibrated on flow in chapter 5 using a Delft3D model.

Part 3 The third part is to acquire influential parameters in the design or execution. General design parameters, executional capacities and basic data are used as input for the tool several cases studies are performed to evaluate these influential parameters. Relevant strategic design considerations for this investigation are for example the equipment capacity, phasing options, type of closure method and the choice of material used. The answer to research question 2 is given when the case studies shows the impact of each of these parameter on the total cost. All case studies and their results are discussed in chapter 6.

2

Theory and background

This chapter contains a brief overview of the general literature concercing design of closure works in large tidal basins. Three books have been especially helpful in this overview: "The closure of tidal basins" bij J.C. Huis in 't Veld, "Breakwaters and Closure dams" by K. d'Angremond and "Afsluitdammen, regels voor ontwerp" By J.L.M. Konter. All books explain relevant *practical* considerations in the design of closure works on the basis of theory and case studies from the Deltaworks. In section 2.1.8, these considerations are summarized.

Sections 2.2, 2.3 and 2.4 contain the theory on the most relevant design aspects of closure works: Flow velocities, material stability and bed protection. In section 2.2, the theory of flow in tidal basins is elaborated and its practical application in the design process of closure works . Section 2.3 explains more about the stability of rocky materials and section 2.4 describes theory of local scour and a practical model to compute the quantity of bed protection to be used in the design of closure works.

2.1. Closure works

2.1.1. Introduction

The main purpose of closure works is to close a water course. The closure works are **temporary works** that are specially designed to close a tidal basin or river estuaries. The works may later be part of the dam, but the design and execution method is often not based on the final dam design.

Closure works need to close the cross sectional flow area available at the location of the dam (figure 2.1). The dimensions of closure works are based on the amount of flow area that needs to be closed and the tidal amplitude at the location. Larger tidal amplitudes lead to a larger cross sectional area to close, but also to higher flow velocities during flood and ebb in the gap.

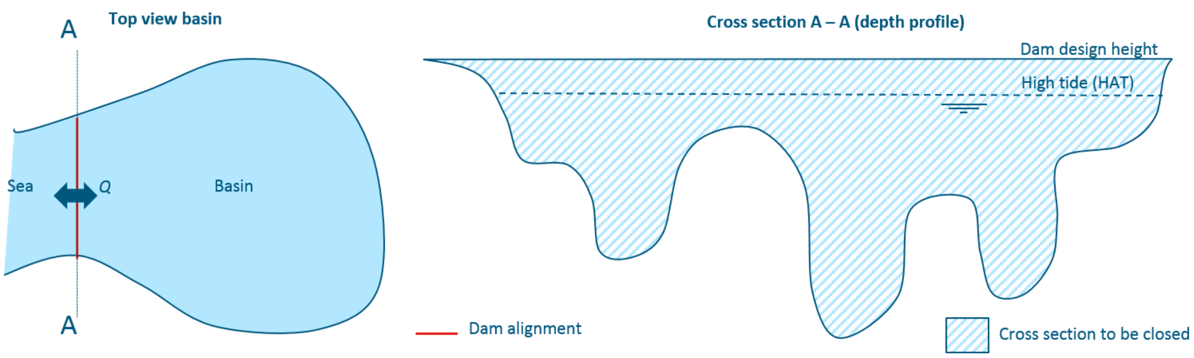


Figure 2.1: Wet cross section to be closed by temporary closure works

2.1.2. Location and Alignment

Closure work designs are generally site specific. For each location the circumstances are largely varying and that is why general detailed recommendations on closure works hardly exist. For example, to close a large tidal basin, a large amount of material is required. The price and availability of the material will define the final closure works design. Other parameters are considered irrelevant. The closure of a small basin required less material, but the equipment is still required. In this case, the equipment cost the relevant parameter.
after the choice of location, the specific dam alignment can be further optimized by including this choice in the design of the closure works and dam design. This specific dam alignment depend the following aspects:

- **The configuration of the bed:** The position of the channels and flats are important. In determining the final dam alignment, the bathymetry of these channels and flats should be utilized optimally (Huis in 't Veld, 1987).
- **The composition of the bed:** The main focus lies on the bearing capacity of the bed. The structures inside the dam (locks and spillways) should be stable along with the dam itself.
- **The connection with the shores:** The distance between the dam and existing infrastructure such as roads and train tracks can be very large. The the position of the connection with the shore should therefore be considered
- **The closure method :** A certain bathymetry is preferred for each type of closure; with a gradual closure, wide gaps with shallow parts are preferred to minimize filling material needed. When doing a sand closure, the final gap must be shallow, since this goes faster and less sand is lost. In case of a sudden closure, a deep channel is preferred, since this minimizes the amount of placements (Huis in 't Veld, 1987).

The final dam alignment will be inside the defined boundaries, therefore minimizing the influence on the first set of location determining aspects. However, in some cases the closure works might be the dominant factor for the choice of the general location when other criteria are less important. An integral assessment of site characteristics is therefore required before deciding on the location and the final alignment (Huis in 't Veld, 1987).

2.1.3. Types of closure methods

A main distinction in a type closure works can be made according to the construction method. Each method is directly related to the equipment and materials used. Therefore the preferred method is chosen for which the materials and equipment required have the least costs and risks.

Gradual In this case, flow velocities are acceptable to use relatively small sized materials which are deposited at the place of construction until the flow is completely blocked. Construction of the closure dam takes longer than one tidal cycle so the occurring flow in the gap will be at maximum tidal velocity during construction. In some cases, if it takes more than two weeks, even spring tidal velocities will occur. There are three main construction types to execute a gradual closure (K. d'Angremond, 2004)

1. **Horizontal closure:** Closing the dam from one or both sides narrowing the closure gap
2. **Vertical closure:** Stacking layers of material from the bed upwards to decrease the depth in the closure gap until the dam closes of completely
3. **Sand closure:** Depositing large quantities of sand in the gap until the gap is closed

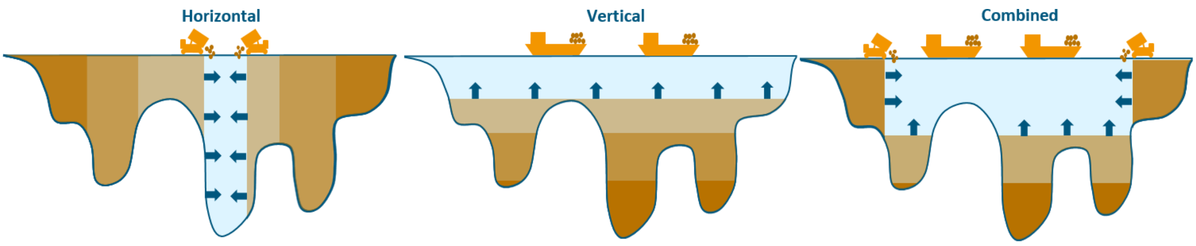


Figure 2.2: Gradual closing methods

The main distinction is made between a horizontal (using land based equipment) and a vertical closure (using waterborne equipment). A sand closure can be used if peak flow velocities are low. A part of this sand will be lost during the closure operation. However, the high costs of bed protection are saved (veld, 1987). A combination of these methods is often used and is called a combined closure. All options have be graphically explained in figure 2.2

Sudden A gradual closure is in some cases impossible to close the basin completely. Occurring flow velocities in the final gap are too large to deposit locally available materials. It can also occur that the required material size is too large to deposit due to insufficient material transport or dumping capacity. In this case, a sudden closure is required. Such a closure method involves that a large part of the final gap in the dam is closed within a certain period of time to avoid the occurrence of high flow velocities (figure 2.3. This can mean either within one tidal cycle or within two spring tides. Although, if it's done within two spring tides it is often still called a gradual closure. A sudden closure can be performed by using pre-installed flap gates, sliding gates using sluice caissons or placement of a blocking caisson or vessel (K. d'Angremond, 2004). An example of a sluice caisson can be found in figure 4.15. These were used in the design to perform a sudden closure over the last 10 km in the Gulf of Khambhat.

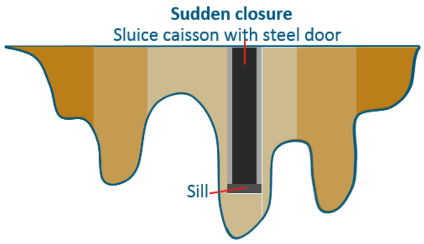


Figure 2.3: Example of sudden closure method utilizing a sluice caisson with a steel door

Importance of phasing Another type of distinction in closure methods is based on the bathymetry at the location of the dam. The tidal basins usually have channels and tidal flats and therefore the closure of a tidal basin is usually performed in several phases. The type of closure method used is different for both of these locations because the flow conditions are defined differently. These closure methods are defined as follows (figure 2.4):

- **Tidal gully closure:** The closure of a channel in which high flow velocities occur. This channel is therefore eroded to a large depth.
- **Tidal flat closure:** The closure of a tidal flat which is generally very shallow or dry at low water. The critical flow velocity will occur at certain characteristic tidal water levels.
- **Simultaneous closures:** horizontal or vertical over the full length of the cross section

One of the main challenges is to assess a closure phasing that is convenient and cheap to execute.

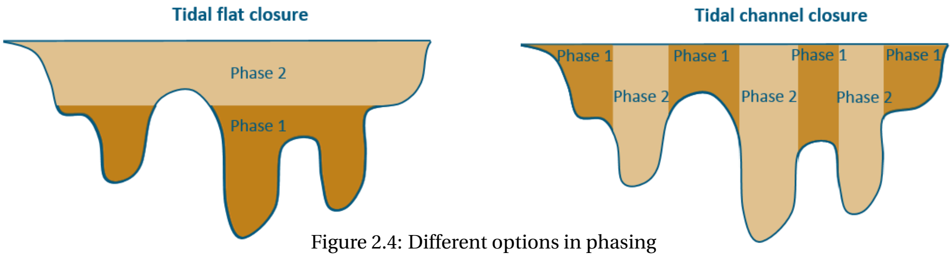


Figure 2.4: Different options in phasing

2.1.4. Construction equipment and materials

For the construction of closure works, several types of material and equipment can be used. The type of equipment depends on the closure method used. It can either be locally acquired or imported from somewhere else. For this reason it's practical to gain the some information about locally available and imported equipment. Sometimes it might be cheaper to use larger cranes or dumping trucks to be quicker in executing the final closure(K. d'Angremond, 2004). The type of material also depends mostly on the local availability, although quarry rocks are often used in high velocity environments. A complete overview of the possible construction materials and equipment with respect to the closure options is given in table 2.1. In the next subsection, the specifics about quarry rock is discussed, since this is the most common material to use in large closure dams and is readily available in the area around the Gulf of Khambhat.

Table 2.1: Overview materials and equipment possibilities for different closure options

	Materials	**Equipment**
Horizontal	• Clay • Sand • Gravel • Rubble (quarry rock) • Sand bags • Gabions • Concrete blocks	• Manual labor • Trucks • Cranes
Vertical	• Clay • Sand • Gravel • Rubble (quarry rock) • Sand bags • Gabions • Concrete blocks	• Manual labor • Trucks/trains that ride a temporary bridge • Floating equipment: Dumping vessels or cranes • Temporary cable-way • Helicopters
Sudden	• Caissons	• Preparation in a dry dock. Floating and sinking at slack water • Hoisting with high capacity floating cranes

2.1.5. Quarry rock

Quarry stone is natural rock, obtained from quarries and is one of the most important construction materials for closure dams. All over the world, commercial quarries which produce large size rocks that can be used for hydraulic engineering can be found. Rocks are more stable with increasing size and can therefore be used in high velocity environments such as a closure of a tidal basin. The most important aspect about rock is the dimensions required for a stable closure without any damage by instable rocks. If the required rock dimensions are too large, the cost will go up due to the need for heavier equipment. The quarry yield curve is also an important influence. This curve shows the gradation of stone masses that can be delivered by a quarry and can be seen as a supply curve. An example of a quarry yield curve is given in figure 2.5

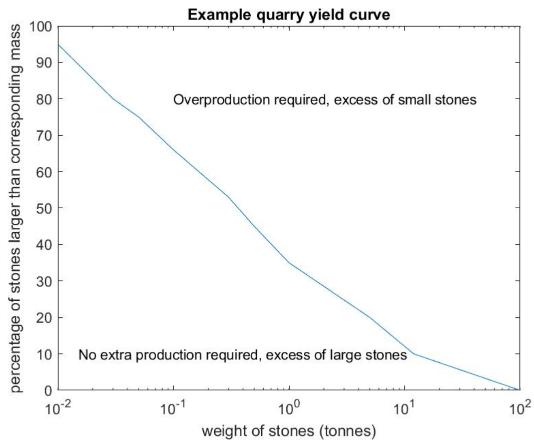


Figure 2.5: Example of quarry yield curve

The demand curve is determined by the design of the closure works. The main goal of designing the rocks sizes in a rocks closure operation is to match the demand curve to the supply curve. In this case, extra cost for overproduction or downsizing of rock is avoided. To determine the rock sizes required during the closure sequence, stability relations are used. The stability relation of rocky materials is discussed in the section 2.3.

2.1.6. Bed protection

Bed protection is vital to protect the structure from instabilities due to scouring. Flow velocities increase during the closure and therefore scour hole development exponentially increases. Before the closure a certain quantity of bed protection required should be placed in all places scour is expected. If the scour hole develops far enough from the dam due to placement of a sufficient amount of bed protection, the dam is considered safe from soil instabilities. The scour process and dam stability are further elaborated in section 2.4.

2.1.7. Design approach

For initial evaluation of closure strategies, computing the flow velocity is an important aspect. This determines material design sizes and bed protection. To compute the flow velocity through a constricted wet cross section a storage model can be utilized, this model is further elaborated in section 2.2. If the flow velocity is known, it can be translated to a design rock size with the Shields equations (section 2.3).

The storage model can be used to develop a a general design graph to define the velocity an rock sizes during specific closure strategy (figure 2.6, left). Using the Shields equation (relation E.1) the figure on the right can also be plotted. However, these graphs can only be produced if only one rectuangular cross sectional flow area is used in the model. The model is run for a single month with increasing sill levels (y-axis) and decreasing gap widths (x-axis), each producing the maximum velocity of that month which is plotted as a point on the graph. The velocities are linearly interpolated to produce a velocity field and lines of equal velocity can be plotted (left figure) with respect to the sill level (y-axis) and gap width (x-axis), which is normalized with the storage area of the basin in this figure.

For a completely horizontal closure, the horzontal (red) line can be followed to track flow velocities and corresponding rock sizes during the closure (decrasing gap widht). For a completely vertical closure, the vertical (green) line can be followed (increasing sill height). Any combinations of these form a combined closure (blue line).

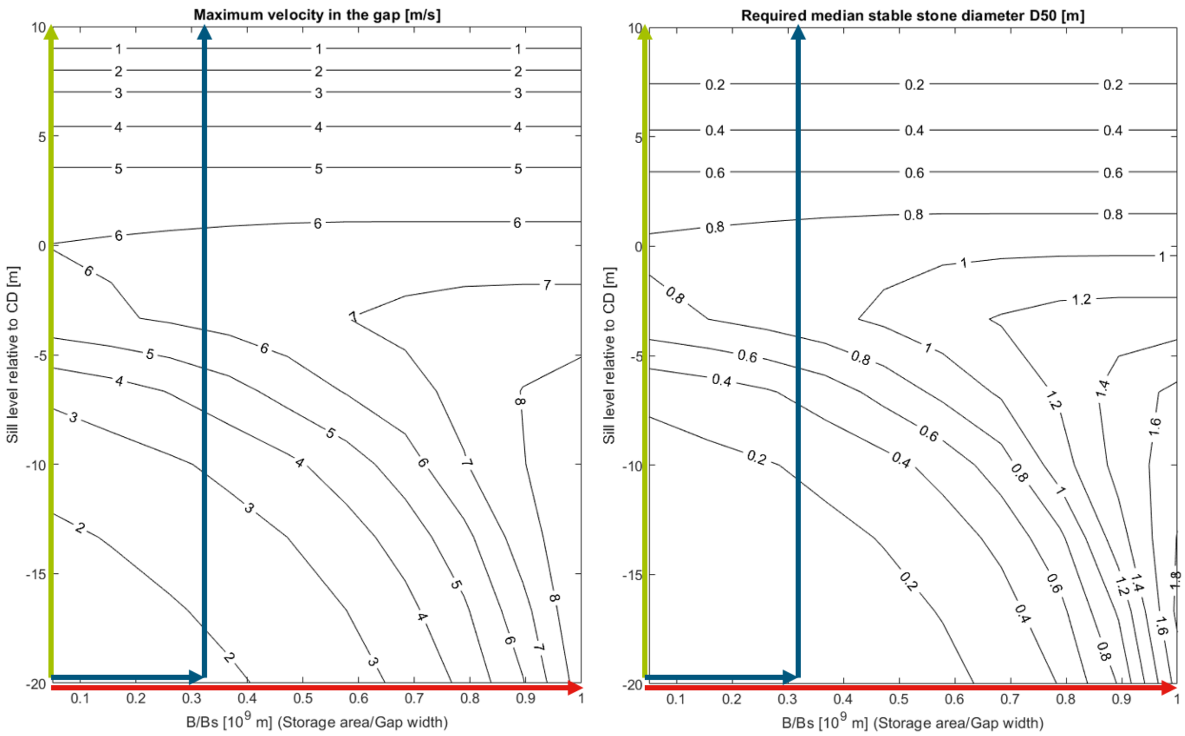


Figure 2.6: Design graph flow velocities and rock sizes for the Kalpasar closure dam

2.1.8. Optimization

The most important practical considerations in closure works have been discussed in the previous section. To summarize: The optimal strategy to close depends for a large part on the location; the bathymetry, tidal amplitude and local available materials. In the Kalpasar case, the location is already set and most of these parameters are already known. To optimize the strategy in the Kalpasar case, the most important consideration becomes the chosen closure method and phasing. For each strategy, the occurring flow velocities during the closure are different. For this reason, the required material sizes, the quantity of equipment and the quantity of bed protection will vary. In the following sections, theory about these main cost factors are discussed in more detail.

2.2. Flow in tidal basins

This section focuses on the flow characteristics inside tidal basins. Assessing the flow velocities during the closure is of utmost importance to the design. The flow velocity constantly changes during the process and both material dimensions and the design of bed protection are highly dependend on the occurring flow velocities. The theory of flow in tidal basins starts with the continuity and momentum equations which are reduces to more simplified and practical equations that can be used to model flow velocities in tidal basins to design closure works.

2.2.1. Free surface flow

From the balance between inertia, forcing and resistance the equations of motion for fluids are derived. These equations are known as De Saint-Venant equations or shallow water equations, because they assume depth is relatively small compared to typical length dimensions such as the wave length. The equations consist of the continuity (C.1) and momentum balance equation (C.2). (Battjes and Labeur, 2014)

$$B\frac{\partial h}{\partial t} + \frac{\partial Q}{\partial s} = 0 \tag{2.1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial s}(\frac{Q^2}{A_s}) + gA_c\frac{\partial h}{\partial s} + c_f\frac{|Q|Q}{A_c R} = 0 \tag{2.2}$$

Together, the equations form a set of hyperbolic partial differential equations which define the water level (h) and discharge (Q) as a functions of location (s) and time (t). The geometric parameters A_c, R, B and c_f are required to be known functions of the s, h and Q so that mathemamatically speaking they are known variable coefficients. With a set of initial and boundary conditions these functions can be integrated. It depends on circumstances whether some contributions to the momentum balance are important or negligible.

2.2.2. Small-basin approximation and storage model

A large simplification can be made when the flow is unsteady but the domain in which it occurs is relatively small, for example a short basin connected by a restricted opening to the sea with a time-varying surface elevation (tidal motion). Because the basin is closed (except for the opening to the sea) and short, the velocities in the basin are small thus inertia and resistance play no role. It follows from Equation C.2 that the slope of the surface can be neglected inside the basin. Thus the surface elevation in the basin is a function of the time only written as $h_b(t)$. The approximation disregard the spatial variations in the basin. However, the disregard is only allowed if the dimensions of the basin (length of the channel system) compared to the typical wave length of the tide in the domain are small. Thus phase differences become negligible (Battjes and Labeur, 2014).

Writing A_b for the free surface area inside the basin, the rate of change of the basin water level is defined with respect to the discharge through the opening (Q_{in}, positive for flow into the basin) as equation 2.3

$$Q_{in} = A_b\frac{dh_b}{dt} \tag{2.3}$$

The practical application of the equation in the design of closure works is that if the discharge is known, the change in basin level can be computed. The discharge depends on the head difference between the basin and the sea level. To compute the discharge, relations for flow over a weir can be used (relations4.1 and 4.2. (CETMEF et al., 2007).

$$Q = B\cdot h_b\cdot\sqrt{2g\cdot(H-h_b)} \qquad \textbf{For subcritical flow} \tag{2.4}$$

$$Q = B\cdot\frac{2}{3}H\cdot\sqrt{\frac{2}{3}gH} \qquad \textbf{For supercritical flow} \tag{2.5}$$

In these equations, H is the sea level and B is the width of the basin at a given cross section. The combination of equations 2.3, 4.1, 4.2 form the storage model. A storage model is a differential equation which can be solved in time if the sea level is known. The model computes the velocities in the cross section in time and these can be used in the design of closure works. An elaborate derivation of this model can be found in appendix C and is used as the basis for the optimization method.

2.3. Stability of rocky materials

During closure works that are executed with stone like material (quarry rocks, gabions or concrete blocks) hydraulic forces act on the structure. In case of a horizontal closure the, the hydraulic loading on the head of the dam is considered normative. In case of a vertical closure, the normative hydraulic load is present on the crest over the full length of the dam. (Akkerman and Konter, 1985)

While dimensioning the closure works with quarry stones, it is therefore important to check the stability of the quarried rock on the crest of the sill (vertical closure) or on the dam head (horizontal closure). The general stability formulation from Shields for uniform flow is commonly used to dimension the rocks sizes required. This formulation is defined by equation 2.6(Akkerman and Konter, 1985)

$$U^2 = C^2 \cdot \psi \cdot D_n \cdot \Delta \tag{2.6}$$

In which:

C	=	Roughness parameter	$[m^{0.5}/s]$
Δ_r	=	Relative density of the quarry stone in water	[-]
D_n	=	Nominal diameter of the quarry stone	[m]
ϕ	=	Damage parameter or shields parameter	[-]

This equation is widely used as stability relation for large rocks. However, the ranges of the results are also quite large and that proves it unreliable. This is why the formulation is calibrated to a more practical relation for that can be used in closure work designs. (Akkerman and Konter, 1985). The complete derivation of the calibrated shields formulation can be found in appendix E.

2.4. Bed protection

The function of using bed protection during a closure is to prevent structural instabilities in the direct surroundings of the closure structure. The bed protection keeps the bed level around the dam structure at a constant level thereby preventing large scour holes near the structure. The requirements for the design of bed protection can be deduced from its three functions:

1. **Sand tight** This prevents the sand from moving and therefore a stable bed is guaranteed.
2. **Stability** The bed protection must be able to withstand the incoming forces of the flow.
3. **Length** The bed protection requires enough length to keep away possible instabilities in the soil generated by scour holes from the structure. The occurrence of a scour hole can't be prevented, but the instability of the soil near the structure can with enough length of bed protection.

The required volume of bed protection is closely related to the total cost of bed protection, therefore this defines for a large part the total cost for this cost factor. With this in mind, the required *length* becomes the most costs influential aspect of bed protection. The *stability* and *sand tightness* is guaranteed by choosing a stable type of bed protection. The possible types each have a specific layer thickness which is stable for a range of design flow velocities.

In this section, a calculation method is presented for the required quantity of bed protection in square meters based on theory. To start, the required amount of bed protection is directly related to the shape of the scour hole that forms at the end of the bed protection. The depth and slope angle of this hole can be modeled with the use of the theory of local scour (Schierkeck, 2012) and is explained in section 2.4.1. The stability of the structure depends on the sliding plane of the soil and the distance from the structure. This is explained in section 2.4.3

2.4.1. Theory of local scour

Flow over a sill is described by separation and reattachment of flow. Directly behind the sill the flow separates and forms a new shear layer (figure 2.7). This layer bifurcates at the at the reattachment point. The reattachment point is located about 5-8 times the sill height in horizontal direction downstream of the sill. The bifurcation of the flow partially generates a recirculation near the sill and partially forms a new sub boundary layer after this point (Tani et al., 1961). Turbulence intensity and shear stresses decrease in this new sub boundary layer. Because this decrease of turbulence and shear stresses is a slow process, the formation of a fully developed boundary layer takes around 30 times the sill height in horizontal direction downstream of the sill. The flow velocity in this layer and occurring turbulences are determining factors of the scour hole depth at the end of the bed protection if applied.

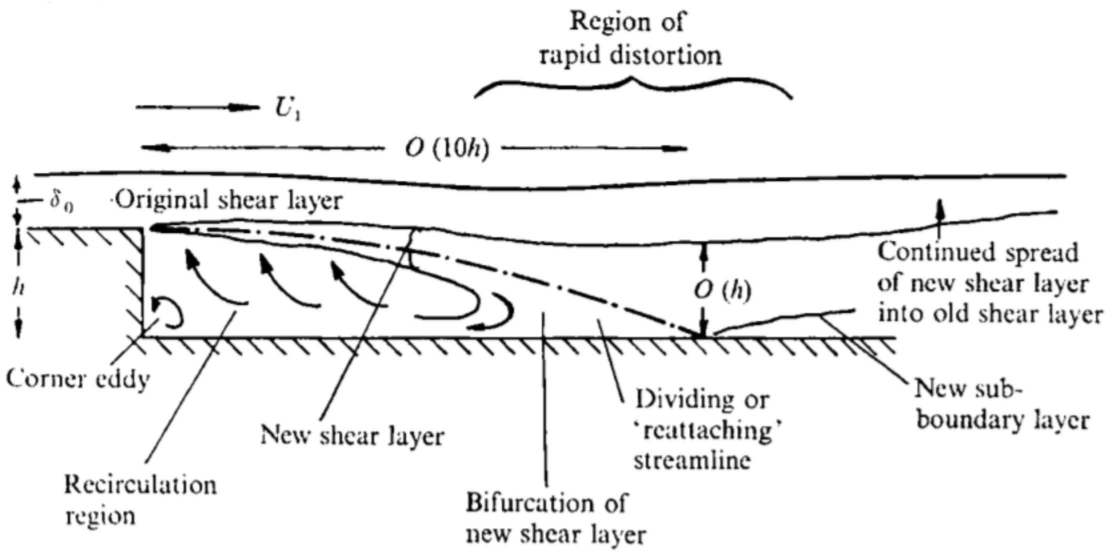


Figure 2.7: schematization of shear layers behind a sill (Tani et al., 1961)

The development of a scour hole at the end of the bed protection is described by four phases:

1. **Initial scouring stage**: In this stage the flow over the bed has an equilibrium velocity profile. (Chandavari and Palekar, 2014)
2. **Developed scouring stage**: In this stage, the scouring process continues and expands in downstream direction. (Wiersema and Broos, 1998)
3. **Stabilization stage**: In this stage the slope angle of the upstream slope stays constant. The scouring process continues slowly.
4. **Equilibrium stage**: The velocity is equal to the critical velocity. The maximum scour depth is defined as the equilibrium scour depth.

2.4.2. Practical use for closure works

For the practical use of this theory in bed protection design of closures stage two is relevant. Closures are highly dynamic and time dependent. The scour hole that forms is also highly dependent on time. Significant relations in closure strategy and required amount of bed protection can be found by analyzing this stage. If time does not play a role in the design of the bed protection, the equilibrium scouring depth can be used to define the scour hole depth.

Time dependent scour In the developed scour stage, the depth of the scour hole can be described as a function of time and depending on several other parameters. Since a sill is present and assuming that this structure blocks the sediment from going over the sill, it can be stated that the scour type that occurs is "clear water scour". Therefore, Breusers scour model for clear water scour can be used to compute the depth of the scour hole in time.

The model is based on equation 2.7 in which the scour hole is dependent on the flow velocity, the bed material, the water depth, the grain diameter and time. Turbulence is also included in the turbulence factor α_t. (Schierkeck, 2012)

$$h_s(t) = \frac{(\alpha_t u - u_c)^{1.7} * h_0^{0.2}}{10\Delta^{0.7}} t_{scour}^{0.4} \tag{2.7}$$

In which:

$h_s(t)$	=	Depth of the scour hole in time	[m]
t_{scour}	=	Runtime for the scour process in days	[days]
Δ_s	=	Relative density of the bed material in water.	[-]
h_0	=	Waterdepth in the basin	[m]
D_{n50}	=	Median nominal diameter of the bed material.	[mm]
α_t	=	Turbulence factor	[-]

2.4.3. Stability of the structure

The failure of stability of structure happens when the soil below the structure slides away as a consequence of the steep angle of the upstream slope of the scour hole becomes too steep. This sliding of the soil does not have to be prevented specifically; the structure just needs to be place far away enough from the scour hole that such a slide does not reach the structure. If the slope slides, the slope will slide until a certain angle . This resulting angle is important in defining the length of the bed protection. The angle after sliding of the inner slope is defined by the type of bed material and depends on the geotechnical conditions of the soil. The problem is now defined by geometry (figure 2.8) and the length of the bed protection is only dependent on the depth of the scour hole and the angle of failure before and after the slope fails. A detailed derivation of this computation can be found in appendix D

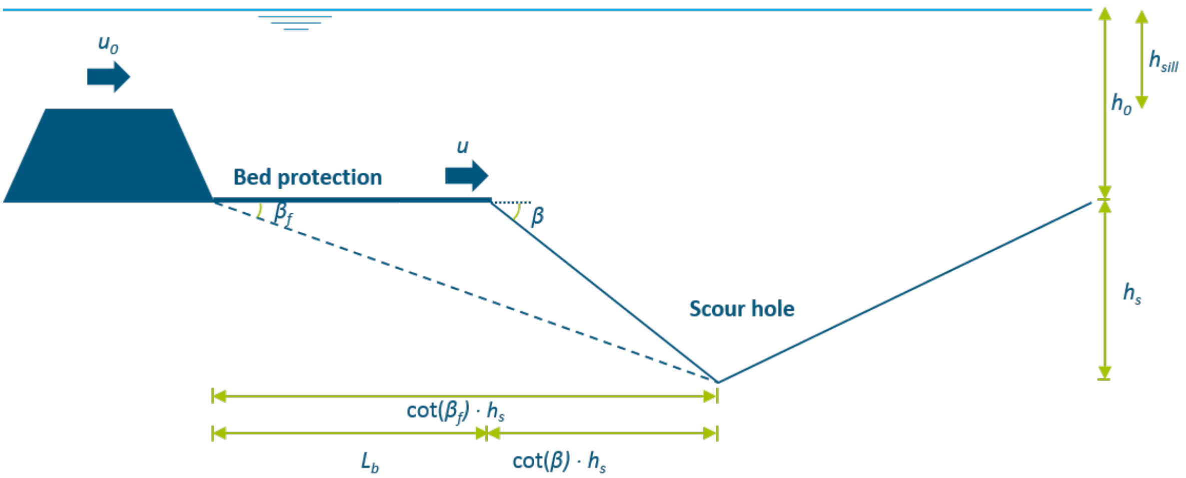


Figure 2.8: Geometric relation for the length of bed protection

3
Method for optimization

To answer the research questions explained in chapter 1, it has been decided to design a robust method to optimize the closure strategy of the Gulf of Khambhat. A robust method requires adaptability to changes and reform. Furthermore, the method has to optimize the closure strategy for the given circumstances in the Gulf of Khambhat. However, what is an optimal closure strategy? And what are the most relevant design aspects that require thorough evaluation in choosing the optimal design? And which can be left out to minimize the amount of variables in the method. Some relevant cost factors have already been discussed in the previous chapter, such as the required material dimensions, equipment and quantity of bed protection. In this chapter, these questions will be answered in more detail.

In section 3.1 the definition of optimal closure strategy is explained based on the literary review. In the next section the scope of the method is defined. In the scope the evaluation criteria are defined. In section 3.3, out of scope topics are addressed.

3.1. Optimization

To define an optimal closure strategy it is first required to define what is meant with "optimal". The simplest answer to this question is minimal cost plus risk related costs. Presently, both are important factors in the design and construction of large civil engineering infrastructure. Evaluating on both cost and risk would be the most favorable. However, quantifying all risks in terms of costs is very broad and complex. To include the actual cost and risk related costs into the evaluation, a quantitative risk and cost analyses has to be performed. This also requires data and a level of detail in the design which cant be acquired in the method, since it should be as simple method for a first-order evaluation. For this reason, the total cost plus risk related cost are only quantified for a few relevant design requirements (or cost factors) that can be easily translated into cost.

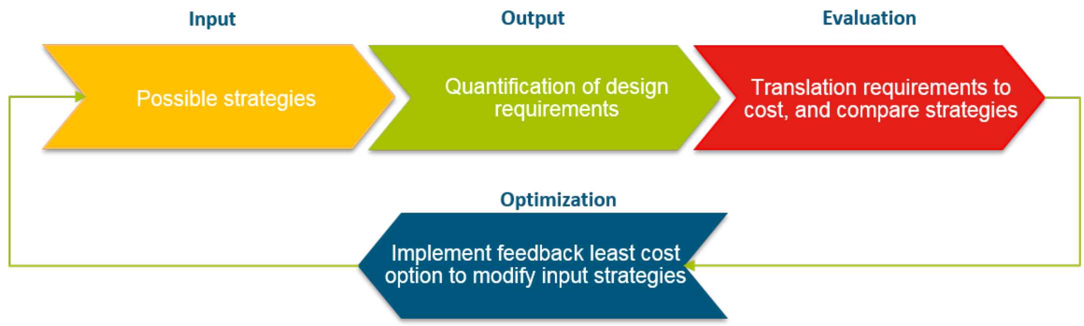


Figure 3.1: Basic setup optimization method

In short, the proposed optimization method is graphically described in figure 3.1. The first element of the method should include the precise definition of possible closure strategies as input. What is a closure strategy? what parameters can be defined to come to a complete strategy and can they be optimized? The output should be in the form of quantified design requirements for all earlier defined strategies. The third element is the evaluation and this should translate quantitative design requirements to cost and compare all strategies. The final element is implemented to optimize the strategy. This element should give feedback about the least cost option to design new strategies to test.

An important question to ask is why the technical feasibility is not in this evaluation? Technical feasibility is not a question here, since the third research question is purely focused on what is required to minimize the cost of the closure works. In other words, it is not relevant to define the feasibility because in the model everything is assumed to be feasible. Feasibility is the last level of evaluation and can be provided by a design presented as an answer to the third research question.

3.2. Scope

In this section, the scope of the method is elaborated. The goal of the model is to evaluate the optimal strategy and it is relevant to identify what is minimally required to design the method. From the theory described in chapter 2, the design of closures works deal with many design aspects, many very practical. However, which design aspects are the most relevant to *distinct* closure strategies and more important, which aspects can be ignored to minimize the total amount of influential parameters. The relevant parameters can be defined as cost factors for the quantitative model and they are elaborated in the next subsection. Less relevant parameters are discussed in section 3.3

3.2.1. Cost factors

The following cost factors have been identified as most relevant to distinct closing strategies with respect to cost and risk related costs

A Equipment required

B Dam materials required

C Quantity of bed protection required

In the following subsections, an argumentation is presented to substantiate each cost factor

A Equipment required

This cost factor refers to the equipment that is required for the execution of a certain closure strategy. Different construction capacities, material choices and closure strategy options have an influence on the type, quantity and time of operation of the equipment. Therefore the equipment required can vary significantly with every choice of closure strategy. This variation can has a significant influence on the total cost of the strategy. The final cost evaluation of this cost factor in the optimization method is therefore influenced by the quantity, type and operation time of equipment required to achieve the closure. Risk related costs due to equipment failure are not taken up in the cost factor.

Example For a horizontal closure, trucks can be used to perform forward dumping. The number of trucks required each phase follows from the required building capacity and the time they need to drive, load and unload each loop. The time the trucks need to be active depends on the construction time of that phase.

B Dam Materials required

Closure works are often not produced from one type of material. During the closure, the velocities in the closure gap increase and different types of material might be required to perform the final closure. Usually sand and small rocks can be used in the first phases of the closure. In later phases larger rocks, gabions or even sluice caissons need to be used. Larger rocks are more expensive, as are gabions. The use of sluice caissons adds a significant amount of cost to the project (Royal Haskoning, 1998a) and is therefore not preferred. To summarize, different closure strategies require different materials or larger, more expensive, dimensions of the material. For this reason, the influence on the total cost is significant to include this a cost factor. The final cost evaluation of this cost factor in the optimization method is influenced by the volume and type of material required to achieve a safe closure with minimal risk of failures due to instabilities of the material. Risks related costs are therefore partially included in this cost factor

Example In case of a horizontal closure, the dam needs to be wide enough to transport trucks which requires extra materials. Secondly, the quarry yield curve can be used to determine if overproduction of material is expected or not. Overproduction of rocks in the quarry leads to extra unnecessary cost.

C bed protection required

This cost factor refers to the quantity of bed protection required during the execution of a closure strategy. The cost of the bed protection forms a significant part of the total closure cost according to the calculations of Royal Haskoning in 1998. Although bed protection is often required over the full length of the dam during all possible closure operations, the size differs significantly with each closure strategy. For this reason, the influence on the total cost is significant to include this a cost factor. The final cost evaluation of this cost factor in the optimization method is influenced only by the quantity of bed protection required to achieve a safe closure with minimal risk of failure of the bed protection.

Example Often, less bed protection is required for vertical closures, since the flow over the dam is higher in the water column and directly decreases behind the structure due to the depth increase (continuity). In case of a sudden caisson closure, more bed protection will be required, since the flow through the caissons flows through the full vertical length of the water column. The flow velocity does not decrease due to minimal depth increase behind the sill. Speed of execution is also an important factor since the scour hole behind the structure grows in time.

3.2.2. Evaluation

The total costs and risk related costs included in the model with respect to the above-stated cost factors are evaluated with the a total cost function (relation 3.1)

$$\textbf{Total costs} = \text{Costs} + \text{Risk related costs} \tag{3.1}$$

In which:

Costs	=	Cost of equipment + Cost of bed protection + Cost of materials
Risk related costs	=	Cost increasing safety level bed protection + Cost increasing safety level material

3.3. Out of scope

From the previous section, cost factors B and C are quantitatively evaluated on cost plus risk. In this study the risk is translated into extra cost. More on this can be read in the next chapter. Risks in cost factor A are not quantified and should be analyzed in more depth in further studies. The risks analyses on the equipment required can be performed by cooperating with a large contractor with more inside information about this topic. In this study the detailed information about quantifying influential risks could not be retrieved .

The next sections contain other technical design aspects of closure works that are considered out of scope and therefore not relevant to include in the evaluation of the optimal design.

3.3.1. Failure of the dam

Costs related to failure of the dam that can occur due to the following instabilities are considered out of scope:

- squeeze
- liquefaction
- sliding
- piping

In all cases of the closure strategy, these aspects have to be dealt with. This means it does not distinct any closure strategy from another one. The differences in design are minimal and therefore not in the scope of the quantitative model. In case of piping, a caisson closure might require more attention, however, this only increases the cost of a caisson closure which is already the most expensive and least preferred closing option. The extra cost to stabilize the caisson against piping problems is taken up into the total cost function of a caissons

3.3.2. Compartmentalization of the storage area

In some closure strategies it's preferred to first close a part of the basin to reduce the total storage area. Subdividing the area diminishes the storage capacity of the individual areas. Each sequent closure can be executed with more ease, possibly permitting the use of locally available materials which reduces the cost. (K. d'Angremond, 2004)

During studies in 1998 from RHDHV and Broos and Wiersema, both concluded that this would only increase the cost, because large dam bodies had to be build that had to be able to resist the large tidal difference. To build stable dams, the sheer size of the dam components would increase material and equipment cost drastically which would not weigh against the decreased cost of the final closure of the larger closure dam. This is the reason why this possibility is out of scope for this research.

The method described will therefore not be applicable to all closures, only the Kalpasar case. In a more general case, it can be optimal to compartmentalize the storage area. If through research, this possibility is excluded, this method can be used to determine an optimal the single dam closure. However, the method is still able to evaluate the cost of a single dam closure and therefore if compartmentalization is applied, costs of both dams could be quickly analyzed with the method.

3.3.3. Operation, maintenance, repair costs

During construction stages, maintenance and repairs are less important. Since the closure of a dam is only a construction stage these aspects will not be discussed further. Repairs can be done on site because the contractor has the equipment available. Maintenance should be taken up in the final design and therefore not in the closure itself. Risks of structural failures or equipment failures should still be taken up (K. d'Angremond, 2004)

3.3.4. Duration execution

The execution duration of the closing strategy is relevant to include. However, it is very complex to quantify costs to a time component, since many people and equipment are involved in the complete process but it's complex to allocate the specific working or operation hours related to a closure strategy. For the use of the equipment, a time component can be added to the cost, but in this research, the working hours of people are out of scope.

3.3.5. Risks with respect to equipment

Operation of large equipment always has risks. In this thesis these risks are not quantified because the complexity is too great for a first-order evaluation of a closure strategy.

3.4. Discussion

For a complete overview in which all possible considerations of a closure are combined into model, the aforementioned cost factors are sufficient enough to determine a detailed optimal strategy. However, the level of detail for the optimization depends on the level of detail in each cost factor. For example, equipment cost can be analyzed to some level of detail (the required amount of dump trucks and ships), but from a certain point, the extra cost for certain decisions can't be taken up anymore. For example, extra cost due to the requirement of a work island in the middle of the basin. The same holds for bed protection and material. Required volumes can be computed, but increasing transportation cost of these materials because the quarry is empty and the next quarry is much further can't be taken up. These considerations are all relevant to include in a final design, but could also be included in the optimization model if the time it takes is worth its investment.

4

Optimization model

The optimization method described in chapter 3 will be further elaborated in this chapter. The goal of the optimization method is to assess the local conditions, define initial design parameters with respect to the cost factors and evaluate on these for each of the possible closing strategies. To achieve this, a model is developed to execute the optimization method. The set-up of the model will be discussed in section 4.1 and in subsections 4.1.1, 4.1.2, 4.1.3, 4.1.4, the different elements of the method are elaborated in more detail such as input, computations, output, evaluation and also the choice of software for the model. All successive sections, describe the optimization model in detail (see also figure 4.1):

In section 4.2 the basis of the model is described and how the model performs the strategy optimization. For this, four models are used to evaluate the strategies on all cost factors. In section 4.3, a flow model is introduced to compute the flow velocities during the closure. On the basis of these flow velocities, the material sizes and required volumes are computed using the material model described in section 4.4. In section, the bed protection model that computes the bed protection volumes required is described 4.5. The last model, presented in section 4.6, computes the required equipment to execute the strategy. In section 4.7, the raw output from the models is translated to cost in the evaluation. Conclusions regarding the complete optimization model are presented in section 4.8.

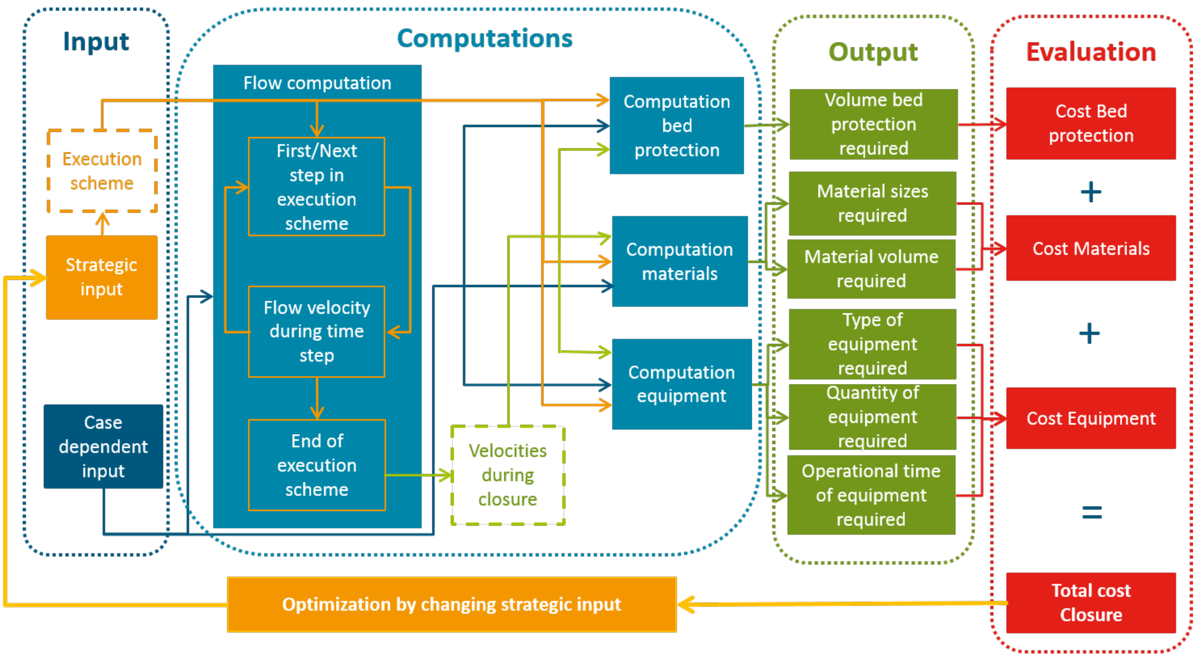


Figure 4.1: Flow diagram optimization model setup

4.1. Setup

The complete setup of the model is presented in figure 4.1 and every element is elaborated in the following subsections.

4.1.1. Input

The most important question is how to choose the minimal amount of input parameters with which the optimization process is to be executed. An optimization can be performed with more ease if the amount of parameters is kept small. Every extra parameter taken up in the optimization exponentially increases the difficulty to optimize and the decreases the reliability of the process. However, since a relative small amount of data is available and the model should be robust enough to handle changes to the surrounding area, most of the variables need to be kept as variables. This problem is therefore divided into two types of input:

- Parameters related to the type of closing strategy
- Parameters related to the specific location and case

The strategic input (figure 4.1) is related to the optimization process. The case dependent parameters define the boundaries of the computational models in which the chosen strategy is tested. These parameters are related to the location and should be able to define with some basic research on site. This divide of strategy and case parameters allows the separation of the case related model and the general applicable model. Furthermore, it allows an easier optimization process that is independent of case related parameters.

4.1.2. Computational elements

To evaluate an optimal closing strategy quantitatively, several design aspects have to be implemented in a computational model. The computations that have to be performed by the model are displayed in table 4.1

Table 4.1: Computational elements

Computation	**Type**	**Model used**	**Elaboration**
Flow	Deterministic numerical	Storage model	Discharge (and flow velocity) calculation based on the weir formula for several types of flow
bed protection	Probabilistic	Breusers scour equation	Calculation of (1) Length and (2) Area of bed protection required
Materials	Deterministic & Probabilistic	Calibrated Shields equations	Calculation of material stability, required material sizes and volumes of each size
Equipment	Deterministic	Self-developed model	Calculation of the equipment required to execute the programmed closure strategy

Numerical flow calculation One of the most relevant aspects of closures is the change in flow velocities during construction (in time). These velocity changes have to be known during every construction phase in order to dimension the materials and equipment required. Several systems can be used to perform a flow calculation in time. From literature, it's recommended to use a 1D flow model such as Duflow or Sobek. However, in smaller basins the simplified storage area approach (section 2.2.2) usually suffices. The advantage of using the storage area approach is that the equations are relatively simple to model and therefore different closure methods can be evaluated in a short amount of time.

Computation of required materials To compute the required material types, sizes and volumes, simplified stability formula's will be used for gradual rock closures. However, The stability formula's involve several parameters with a wide range because conditions vary significantly along the dam. Therefore, the design D_{50} can best be computed with a probabilistic computational model. In this case an accepted level of safety should be introduced to relate to design D50. Risk related costs are then equal to extra design costs due to implementation of a higher or lower safety level. The model used for this computation is Shields (section 2.3), although the equations derived are calibrated for closure works with rocky materials (appendix E). For sudden closures, the volume of caissons required is deterministically computed.

Probabilistic computation of required volume of bed protection To compute the required quantity of bed protection, a more complicated computation has to be performed (section 2.4). The process is influenced by many factors and is often computed probabilistically in terms of a probability of failure of the dam body. It is therefore chosen to approach this computational model probabilistically and include the accepted level of failure into the input. Just as with the material design costs, the bed protection cost increase if a higher safety level is adjusted. This is defined as risk related costs. The model used for this is based on the scour equations from Breusers.

Deterministic computation of required equipment To determine the type and amount of equipment required, a deterministic calculation can offer a good approximation. Since this is already a detailed step in the computation, the significance is rather low and therefore the order of magnitude will suffice as output. In following research, this process can also be handled probabilistically including risks.

4.1.3. Output

The output of the model consists of several quantitative graphs where data is presented. The closure options are assessed quantitatively on cost factors. The velocities and discharges create a basis for all other computations and is therefore the most important. The raw output of the model and it's relation to the cost factors is displayed in table 4.2.

Table 4.2: Raw output and relation with evaluation criteria

Raw Output	Relevant cost factor
Flow velocities and discharges during closure	Equipment, Bed protection, Materials
Length and area of bed protection required	Bed protection
Total volume material required	Materials
Material sizes	Materials
Type of equipment required	Equipment
Quantity of equipment required	Equipment
Operational time of equipment	Equipment

4.1.4. Evaluation and optimization

The final evaluation is based on a summation of the quantified cost output of every cost factor. Optimization can be achieved by changing the strategic input.

4.1.5. Choice of software

Based on the required computational elements of the model and the need to combine them, it has been decided to use MATLAB to create the optimization model. The main advantages of this software is that it allows both numerical and probabilistic computation to be performed in one model. Furthermore, it allows unlimited variables and has no limit in expansion. MATLAB is fast and the output can be analyzed quickly.

4.2. Model basis and strategy input optimization

The strategic input parameters form the basis of the model and the optimization method. The parameters have an influence on all computational models (figure 4.1). In this section, the variables used are explained in more detail in order to understand the essence of the model.

The optimization model consists of several cross sections, predefined by the bathymetry at the position of the dam (lower boundary). An example is displayed in figure 4.2. The bathymetry in the Gulf of Khambhat is plotted and is approximated in the model with eight differently sized cross sections (three channels and five tidal flats) .The upper boundary of the cross section is defined by the dam design height.

The goal of the strategic closure works design is to close these cross sections in a cost effective way. For each cross section, three important strategic decisions have to be made to accomplish this:

- What type of closure method should be used to close the section?
- In what phase should the section be closed?
- With what construction capacity should the section be closed?

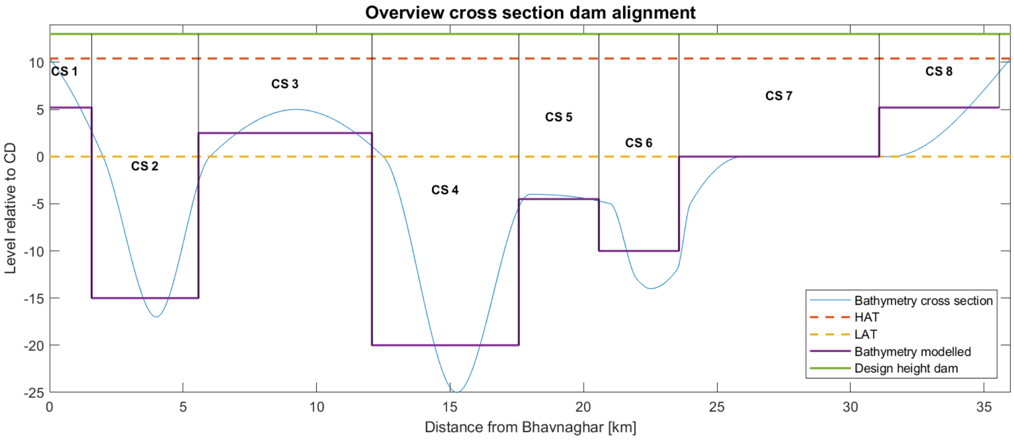


Figure 4.2: Cross sections modelled

Another option is to partially close a cross section in one phase and fully close it in another phase. In this way, a combined closure is also made possible. This creates an infinite amount of possibilities which works well for the optimization process (no option can be left out). An example of the resulting execution scheme is displayed in figure 4.3; On the left, the input matrix resulting from the strategic decisions. On the right the execution scheme.

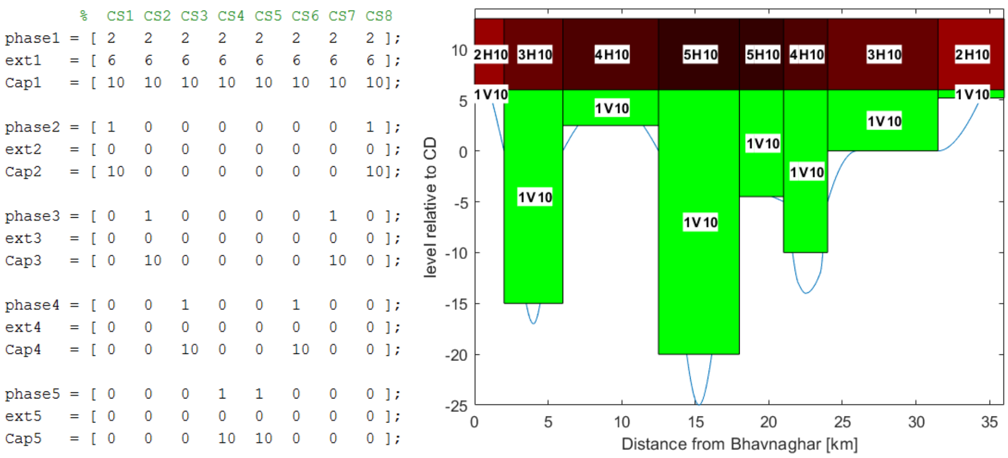


Figure 4.3: Strategy related input, left: Input matrix from MATLAB, right: Resulting execution scheme

For the input matrix (left), three lines are used for every phase, the first line is the type of closure method: 1 for horizontal, 2 for vertical and 3 for sudden. The second line can be used for a partial closure of that cross section and is called the execution parameter. Here, the remaining sill level (in case of a partial vertical closure) or the remaining gap width (in case of a partial horizontal closure) can be defined. In the third row, the capacity can be defined (x 1000 m^3/day).

To visualize the result, the execution scheme is plotted with colors and inscriptions for every cross section. In this example, a combined closure is tested where the sill level is vertically raised until CD +6 m and is then closed horizontally from both sides in phases 2,3,4 and 5. Note that the execution parameter is used in the first phase to divide every cross section in two parts (see input matrix). The inscriptions in every cross section complete the scheme with all information; for example **5H10** indicates: In the **5**th phase, **H**orizontally with a capacity of **10**.000 m^3/day. Furthermore, it can be noticed that green represents a vertical closure and red a horizontal closure (gray is used for sudden caisson closures). The darkness of the color defines the phase. In this way, it can be clearly retrieved what execution scheme is implemented in the model.

In the model, six schemes can be tested and compared in one run with respect to the cost factors. To start six completely different closing options should be tested. From these, the least cost option can be chosen for further improvement. In this way, the optimization cycle is complete (figure 4.1) and the strategic closure scheme can be optimized.

4.3. Numerical Flow model

4.3.1. Introduction

To compute the hydraulic conditions during the closure of the Gulf of Khambhat, a storage model with a tidal inlet has been utilized. The storage model is the same non-dimensional (0D) model used by RHDHV and by Broos and Wiersema in 1998 . The storage model computes the flow velocities in the remaining gap during changing conditions caused by the continuous execution of the previously defined execution scheme. The resulting flow velocities can be used to compute required material sizes, the quantity of bed protection and the type of equipment to be used (see: computational part in figure 4.1)

4.3.2. Considerations

During the development of the storage model, several choices were made. The most important consideration made in the setup of the flow model is to incorporate a changing gap size in the model by introducing a construction capacity. Often this type of storage model is only used to analyze a constant situation. Several model runs have to be executed to analyze the flow situation in all different stages of the closure process. In this study, the cross section decreases in size during the flow computation with the speed of the defined construction capacity of that cross section. In this way, the model runs from the start to the end of the execution and computes the velocities in every time step. The advantage is that the duration of each phase and the total closing time is also known.

4.3.3. Overview model

The complete set of equations for the storage model are derived in appendix C. However, the basic equations have already been discussed in chapter 2. The model is based on storage of water in a basin (figure 4.4.1).

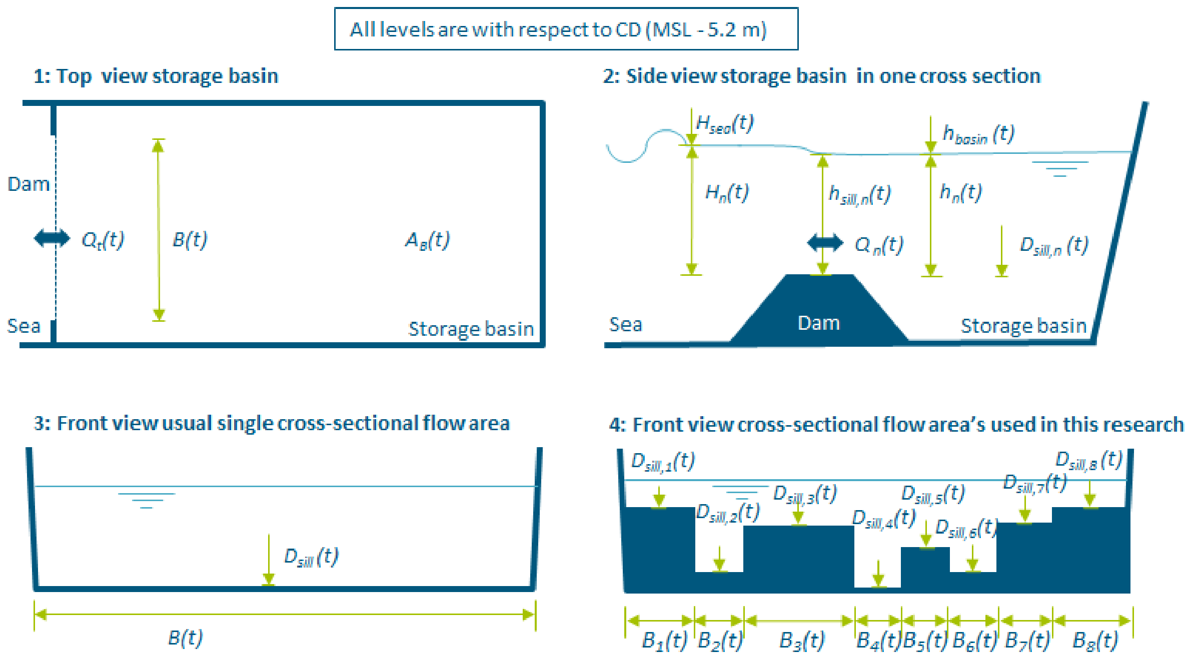


Figure 4.4: Overview storage model

The boundary between the basin and the sea is set at the position of the dam. Due to the changing tide, water flows in and out the basin of which the quantity is defined by the total discharge caused by the head difference between the sea and the basin (figure 4.4.2). The storage model usually uses only once cross section to model the complete closure gap (figure 4.4.3). However in this study, the gap is separated in several cross sections with number 1..n (figure 4.4.4). The discharge through the cross sections is computed for every time step with use of weir formula's displayed in equations 4.1, 4.2, 4.4 and 4.4 previously defined in chapter 2. Distinction is made in flow situation (subcritical or critical) and direction (from sea to basin and from basin to sea).

For flow from sea to the basin:

$$Q_n(t) = B_n(t+1)\cdot h_{n,sill}(t)\cdot\sqrt{2g\cdot(H_n(t+1)-h_n(t))} \qquad \textbf{For sub-critical flow: } H_n(t) > \frac{2}{3}h_n(t) \tag{4.1}$$

$$Q_n(t) = B_n(t+1)\cdot\frac{2}{3}H_n(t+1)\cdot\sqrt{\frac{2}{3}gH_n(t+1)} \qquad \textbf{For supercritical flow: } H_n(t) < \frac{2}{3}h_n(t) \tag{4.2}$$

For flow from basin to the sea:

$$Q_n(t) = -B_n(t+1)\cdot h_{n,sill}(t)\cdot\sqrt{2g\cdot(h_n(t)-H_n(t+1))} \qquad \textbf{For sub-critical flow: } h_n(t) > \frac{2}{3}H_n(t) \tag{4.3}$$

$$Q_n(t) = -B_n(t+1)\cdot\frac{2}{3}h_n(t)\cdot\sqrt{\frac{2}{3}gh_n(t)} \qquad \textbf{For supercritical flow: } h_n(t) < \frac{2}{3}H_n(t) \tag{4.4}$$

In which:

$h_n(t)$	=	Basin water level above sill level	=	$h_{basin}(t) - D_{sill,n}(t+1)$	$[m]$
$H_n(t)$	=	Sea water level above sill level	=	$H_{sea}(t+1) - D_{sill,n}(t+1)$	$[m]$
$h_{sill,n}(t)$	=	Minimal depth above sill level	=	$min(H_n(t), h_n(t))$	$[m]$
$H_{sea}(t+1)$	=	Updated sea level			$[CD+m]$
$h_{basin}(t)$	=	Basin level as result of previous time step			$[CD+m]$
$D_{sill,n}(t+1)$	=	Updated sill level in the cross section			$[CD+m]$
$B_n(t+1)$	=	Updated width of cross section			$[m]$
$Q_n(t)$	=	Discharge through the cross section			$[m^3/s]$
n	=	Number of cross section			$[-]$
t	=	Number of time step			$[-]$

Note that the sill level $D_{sill,n}(t+1)$ and the cross-sectional width $B_n(t+1)$ are updated every time step resulting from the predefined execution scheme. This calculation is performed using the predefined construction capacity and the dam dimensions. The sea level (outer boundary condition) is updated from the predefined tidal conditions in the Gulf of Khambhat (see appendix G). Furthermore, in rock dams water also flows through the dam (permeable). This concept is called throughflow and is also taken up in the model. An elaboration on this concept can be found in appendix C. A toggle is implemented in the model to include the permeability of the dam based on the material parameters and the dam size.

For the total discharge, a summation of the discharges through all cross sections is taken. This is defined by relation 4.5. Due to the incoming (or outgoing) total discharge, the water level in the basin changes every time step. Relation 4.7 describes this change and is derived using the small basin approximation from chapter 2. The updated basin level to use in the next time step is defined by relation 4.7.

$$Q_t(t) = \sum_{1\le n\le 8} Q_n(t) \tag{4.5}$$

$$dh_{basin}(t) = \frac{Q_t(t)\cdot dt}{A_b(t)} \tag{4.6}$$

$$h_{basin}(t+1) = h_{basin}(t) + dh_{basin}(t) \tag{4.7}$$

In which:

$Q_t(t)$	=	Total discharge through gap	$[m^3/s]$
$A_b(t)$	=	Wet basin area	$[m^2]$
$dh(t)$	=	Difference in basin water level	$[m]$
$h_{basin}(t+1)$	=	Updated basin level	$[m]$
dt	=	Time step	$[s]$

Note that the wet basin area $A_b(t)$ changes with every time step due to the irregular shape of the basin. In appendix G, the equation that describes the wet basin area is derived.

4.3.4. Computational diagram

The basic computational diagram of the model adopted is presented in figure 4.5. Since the model computes the discharge and updated basin level for each time step a numerical scheme has to be implemented to avoid numerical errors.

The numerical scheme chosen is the method of Heun (equations 4.8 and 4.9). The scheme was also used by Broos and Wiersema in 1998. The scheme is based on predictor and corrector computations. After both computation have been done, the average is taken of the sum of the computations.

$$h^*_{t+1} = h_t + f(t_t, h_t) \qquad \textbf{Predictor with } t > 0 \tag{4.8}$$

$$h_{t+1} = h_t + \frac{1}{2}(f(t_t, h_t) + f(t_{t+1}, h^*_{t+1})) \qquad \textbf{Corrector with } t > 0 \tag{4.9}$$

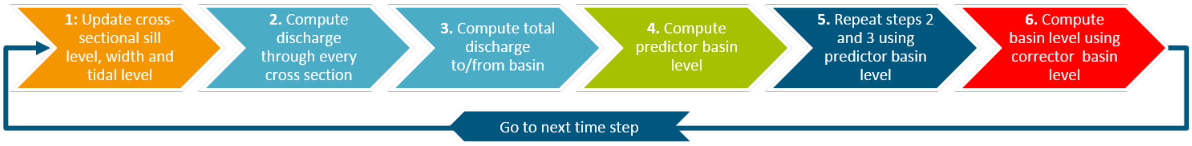


Figure 4.5: Computational diagram

4.3.5. Flow velocity

The flow velocity in the cross sections is computed using the discharge relations. The discharge is divided by the cross sectional flow area on top of the sill (relation D.7).

$$U_n(t) = \frac{Q_n(t)}{B_n(t+1) \cdot h_{n,sill}(t)} \tag{4.10}$$

The flow velocity and execution times are the most important output of the model and they are used to dimension the material sizes, bed protection and equipment requirements.

4.3.6. Results

The model result is the basin level and its adaption with respect to the sea level during closure. This is displayed in figure 4.6. In figure 4.7, the maximum flow velocities are plotted for 6 different closing strategies ranging from fully horizontal (option 1) to fully vertical (option 6) and combinations (options 2-5).

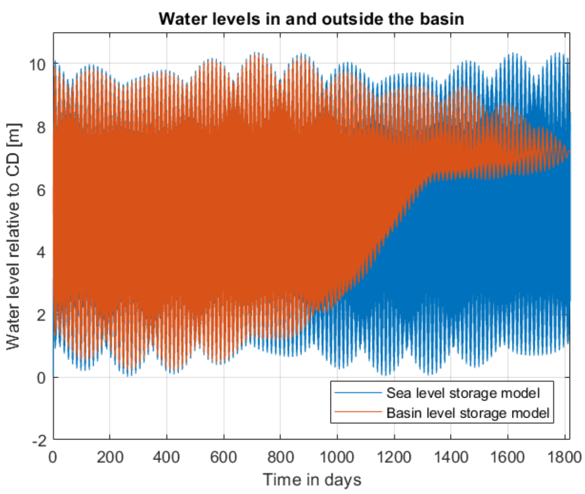


Figure 4.6: Water levels in the basin and sea resulting from partial vertical closure (CD +6 m) followed by a horizontal closure

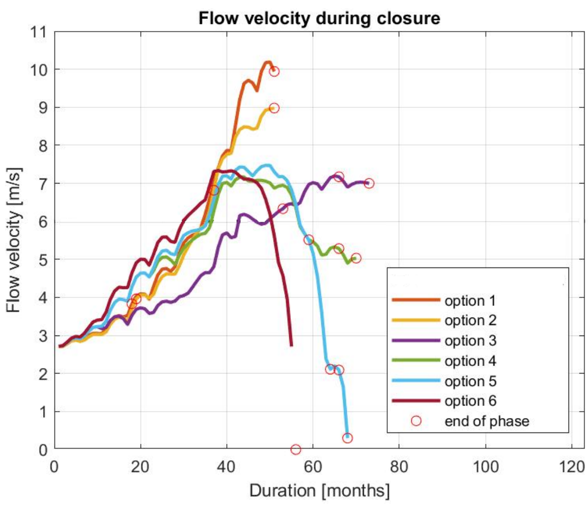


Figure 4.7: Maximum flow velocities of six strategies during closure, from fully horizontal (option 1) to fully vertical (option 6)

4.3.7. Discussion

The used flow model is non spatial dimensional based on the continuity equation alone, which means only the dimension in time is used and all spatial derivatives are neglected. The result is the storage equation (equation 4.7), which can only be used if the storage basin is small enough with respect to the tidal wave length. Furthermore, negligence of the momentum equation (especially the inertia and advective inertia terms) can only be disregarded if the flow gap is small. otherwise significant momentum can be transferred to the basin. Both assumptions don't hold in this case. The storage basin is relatively large and the gap size always starts at the full width of the Bay. It can be assumed that the influence of neglecting the momentum equation and spatial variations in the continuity equation are significant and should be further researched. A validation and calibration of this model was proposed and is elaborated in chapter 5. Since only the flow velocity is relevant, it can be speculated qualitatively what the influence could be on this. The flow velocity would be overestimated by this model due to the slow adaptation of the basin water level (equal water level everywhere) to the incoming sea level, resulting in a larger head difference at all times which causes a larger flow velocity. The phenomenon behind this theory is explained in more depth in chapter refchap:flowvalidation.

Secondly, the storage model uses a linearized storage area. In practice, this is no linear. For a better explanation about this phenomenon read section G.1.4 of appendix, G about the hypsometric curve for the Gulf of Khambhat. Here the relation between storage surface and water level is explained. The result of this simplification is not significant, but will cause a slight underestimation of the flow velocity.

Thirdly, the same outside boundary condition is used for all cross sections. In the Gulf, water level fluctuations over a length of more than 36 km become significant enough to include. The representative boundary condition used in the model has the largest tidal variations measured at Bhavnaghar. Other places along the dam position show less variation. Using the representative boundary condition for all cross section will overestimate the flow velocities in certain cross sections.

fourthly, the bathymetry is approximated with rectangular cross sections, while the actual bathymetry follows a more natural line. This approximation will cause deviations in discharge with respect to a more accurate bathymetry due to the tidal levels which reduce the cross section width at low water. However this deviation can be minimized by increasing the amount of cross sections, up to a certain limit. By increasing the quantity of cross sections, the complexity of programming the closure strategies is also increased. It is therefore not recommended to use more cross sections than the total number of channels and tidal flats.

Even if the wet cross-sectional area is the same as the bathymetry and many cross sections are used to account for the width decrease of the channels. The rectangular shape still influences the flow rate. it can't ever approximate the flow correctly, because spatial variations (momentum equation) over the depth profile are not included. For example: steep slopes of channels are always modelled without a slope. Every section has a flat bottom and every next sections bottom is a little bit deeper. In reality, flow would be drawn to the deeper channels because of the slope reducing the water levels on the tidal flats.

The influence of the rectangular bathymetric approximation is not quantified. Qualitatively speaking, the model will slightly under or overestimate the discharges depending on the location of the cross sections with respect to the actual bathymetry and the tidal water levels. Furthermore, the flow rate distribution across all cross sections will vary with reality due to the negligence of the spatial variations in the flow of water along the dam alignment.

4.4. Material model

4.4.1. Introduction

In this section, a computation of required materials and their dimensions is discussed. The main material used for gradual closures in the model is quarry rock. For sudden closures, sluice caissons are chosen. The cost of quarry rock depends on the required volumes per rock mass class. The required volumes per class should match the quarry yield curve to avoid extra costs, earlier discussed in chapter 2. To arrive at the required volumes per rock mass and to compare them to the quarry yield curve, a probabilistic computation is used. For sluice caissons, the required volume of caissons is the main cost factor and is deterministically determined in the model .

In subsection 4.4.2, some general considerations (considerations and summary) about this model are discussed to explain why certain decisions have been made. In subsection 4.4.3, the probabilistic computation to determine the cost of rocks in the closure design is explained. In subsection 4.4.4 the computation to determine the cost of caissons in the closure design is explained.

4.4.2. Considerations and summary

The most important considerations during the development of the material model are summarized here:

Model for gradual closure material The largest cost difference during closure works regarding materials is the chosen type and size. With respect to type, sand is often the cheapest, followed by small rocks, large rocks, gabions (nets with small rocks), concrete blocks and finally sluice caissons. In earlier research done by Broos and Wiersema, an overkill sand closure was evaluated and considered impossible due to high flow velocities. Since an abundance of quarry rock is available in the area around the Gulf of Khambhat, the main material preference goes to rocks. In this model, the rock sizes are determined probablistically by relating them to an accepted failure probability. In case the rock sizes become too large, an overproduction is required. This can be solved by producing the extra rocks required or choosing gabions to take up the overproduction. Gabions are more expensive per volume than an overproduction, but if large overproduction volumes are needed, this might offer a cheaper solution. In the model, this decision is automatically taken after the computation and is presented in the results.

Concrete blocks are not implemented in this tool, because quarry rock is widely available in Gujarat and will in almost all cases be cheaper than fabricating concrete blocks. However, implementation of this material in the model is simple since the Shields equation also holds for concrete blocks.

Model for sudden closures A threshold value for the use of a sudden (sluice caisson) closure is implemented in the model. This decision is based on the design of Royal Haskoning in 1998. They estimated that the rock size would become too large to handle with regular equipment when the flow velocity crossed the 6 m/s value. During the flow calculation of the predefined closure strategies, the moment and position where the velocity crosses the threshold value is recorded. These are then used to compute the required amount of caisson volume. The amount of caisson volume can be added to the total material costs. The rock computation is also affected by this since the amount of volume taken up by the caisson is not required in rocks anymore.

4.4.3. Probabilistic model design rock sizes

4.4.3.1. Relevance and setup

Dimensioning of rocks is performed using the *calibrated Shields equations* for rock closures (see appendix E) on the basis of the flow velocity computed by the storage model from section 4.3. Contrary to the deterministic approach, a probabilistic (Level III) approach allows to gain insight into the failure probability of the rock for the tidal closure. The probabilistic approach applies statistical distributions for all parameters, excluding the Shields parameter.

The probabilistic model applied in this study utilizes and accepted failure probability per layer in vertical closures and per extent in horizontal closures as input. Layers are assumed to be 0,5 m high and extents 200 m but can be adjusted (figure 4.8). If a layer/extent is covered by a next layer/extent, it's assumed to be stable during the rest of the execution. For every layer and extent one design stone size is selected in the model. Changing the accepted failure probability from 1/100 to 1/1000 per layer/extent increases the rock size (D50) and vice versa. The failure criterion is defined as start of damage which relates to a Shields parameter of 0.03 (CETMEF et al., 2007). The size of the extent is practically determined at 200 m otherwise too many computations are required to perform a quick initial evaluation.

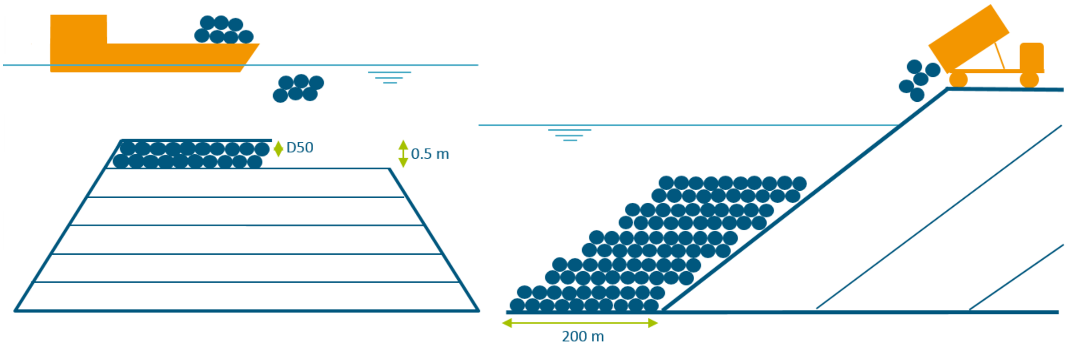


Figure 4.8: Definition layer (vertical closure) left and extent (horizontal closure) right

The probabilistic model can be applied as a basis for a risk analysis. For a risk analysis, damages associated to the risk of failure have to be investigated. Furthermore, the failure probability per layer/extent needs to be translated to a failure probability associated to the damage. Here, special attention is required for the correlation of failure between the layers/extents. Finally, the Level III analysis can be upgraded to a Level IV analysis, which includes damages. However, this is not performed in this research and therefore the accepted failure probability can be maximized without consequence. For this reason, the accepted failure probability is assumed to be constant (1/100 per layer) and can be compared to a minimum safety level.

4.4.3.2. Influence of execution time and layer size

The accepted failure probability of a rock layer/extent is also influenced by its execution time and its size since the failure probability is expressed per layer/extent during its execution. With the failure probability assumed to be constant these factors can still influence the safe design rock sizes.

This design consideration resulting from the probabilistic approach can quantitatively be included in the model. With longer execution times or larger layer/extent sizes, the rocks are exposed longer to the occurring flow velocities. The occurrence probability of an event with extreme flow velocities increases and therefore the failure probability of the rocks increases. To maintain the predefined failure probability, the design rock size (D50) must increase, thereby increasing cost. The faster the execution or the smaller the layer/extent sizes, the shorter the exposure time of the rock, thus the smaller the rock sizes can be.

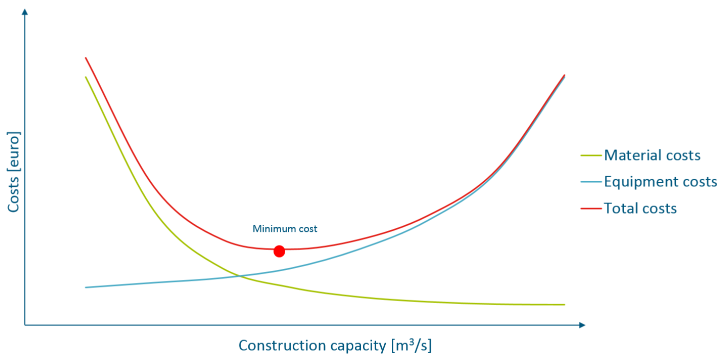


Figure 4.9: Cost optimization for equipment and material costs by influencing the construction capacity

In the model, the layer size can be by dividing the gap into more cross sections increasing the number of phases required to close. This is not recommended, since minimization of the number of phases is always recommended for an efficient closure. The size of an extent (cross-sectional dam area) is already minimized by default, concluding that for the most part only the execution capacity (strategy input) influences the material cost. The resulting consideration that is created is extra equipment cost versus extra material cost. The optimum between these cost factors can be determined with the model and is influenced by the accepted failure probability. Together they can be classified as an important risk consideration.

4.4.3.3. Design D50 computation

To design the D50 for each layer, the challenge is to match the actual failure probability with the accepted failure probability. The actual failure probability can be computed with use of a Monte Carlo Simulation. In this simulation the limit state function (relation 4.11) for failure of the D50 of the layer is solved n times, counting the number of failures n_f $(Z < 0)$.

$$Z = U_c - U_s \tag{4.11}$$

In which:

U_c	=	Critical flow velocity of rock (Shields)	$[m/s]$
U_s	=	Soliciting flow velocity	$[m/s]$

The probability of failure is therefore defined by relation 4.12

$$P_f = \frac{n_f}{n} \tag{4.12}$$

Critical flow velocity To determine the critical flow velocity U_c, a statistical distribution is determined for every parameter in the calibrated shields equations (relations 4.13 and 4.14) derived in appendix E

$$\Delta D_{n50} = A \cdot U_c^2 \quad with \quad A = \frac{0.64}{K_\alpha \cdot \psi \cdot C^2} \quad \text{For horizontal closures} \tag{4.13}$$

$$\Delta D_{n50} = A \cdot U_c^2 \quad with \quad A = \frac{0.36}{K_\alpha \cdot \psi \cdot C^2} \quad \text{For vertical closures} \tag{4.14}$$

Where 0.64 and 0.36 are correction factors due to turbulence (F_t, v and F_t, h. Where the factor K_α is a correction factor for the of the slope dam head in case of a horizontal closure or for the downstream slope of dam in case of a vertical closure. Both correction factors are defined by relations 4.15 and 4.16

$$K_\alpha = \sqrt{cos(\alpha) \cdot \sqrt{1 - \frac{tan(\alpha)^2}{tan\phi}}} \quad \text{For horizontal closures} \tag{4.15}$$

$$K_\alpha = \sqrt{\frac{sin(\phi - \alpha)}{sin(\phi)}} \quad \text{For vertical closures} \tag{4.16}$$

The quantity of rock parameters is reduced to the set shown in the table 4.3. For simplicity normal distributions have been chosen for all parameters and the standard deviation is a percentage of the mean.

Table 4.3: Reduced rock parameters and their representative distributions

Parameter		**Definition**	**Unit**	**Distribution**	**Parameters**	
					μ	σ
C	=	Chezy coefficient for large rocks	$[m^{0.5}/s]$	Normal	35	$0.05 \cdot \mu$
ψ	=	Shields parameter for rock	[-]	Normal	0.035	$0.1 \cdot \mu$
ρ_r	=	Density rock	[-]	Normal	2600	$0.01 \cdot \mu$
α	=	Slope of the dam head	[°]	Normal	27	$0.1 \cdot \mu$
ϕ	=	Natural slope of the material	[°]	Normal	40	$0.1 \cdot \mu$
F_t, v	=	Vertical correction factor turbulence	[-]	Normal	0.36	$0.02 \cdot \mu$
F_t, h	=	Horizontal correction factor turbulence	[-]	Normal	0.64	$0.02 \cdot \mu$

More information about the complete derivation of these equations can be found in appendix E. In appendix G, the chosen distributions for all parameters are explained in more detail.

Soliciting flow velocity For the distribution of the soliciting flow velocity U_s, peak velocities in the velocity matrix (from the storage model) are extracted as data points during the execution time of the layer/extent. The data points are then fitted with an extreme value distribution (figure 4.10). In this case a Gumbell distribution.

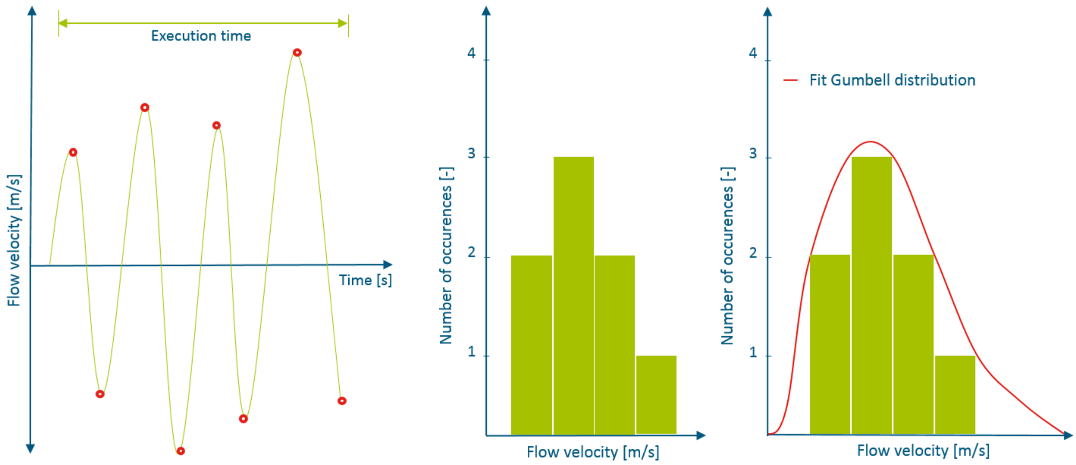


Figure 4.10: Conversion flow velocity data points to extreme value distribution

Monte Carlo Simulation With all distributions defined, the Monte Carlo simulation can be performed using randomly generated numbers for all parameters. To save computational time, number generation is performed using Latin Hypercube Sampling (LHS) .This concept is elaborated in appendix F.

The design D_{50} (normally distributed) per layer/extend is determined by a stepwise increase of the mean D_{50} in the distribution. Every loop the Monte Carlo simulation is performed and determines the failure probability. If the failure probability is larger than the accepted failure probability, the mean D_{50} is increased and the simulation is run again until the failure probability matches the accepted failure probability. Resulting in a D_{50} which fails with the accepted failure probability. The design loop is presented in figure 4.11

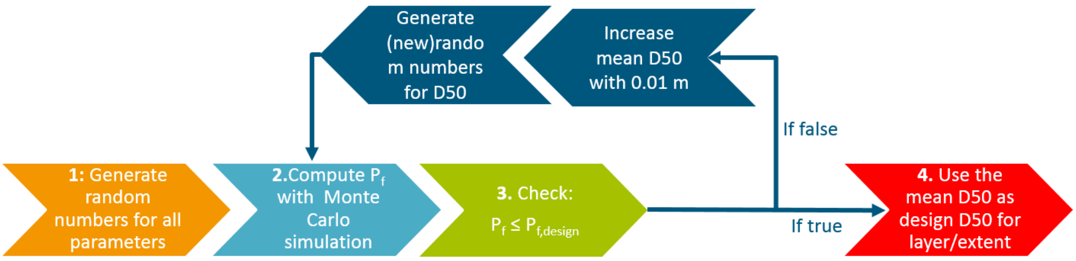


Figure 4.11: D50 design loop schematically presented

Results The last step is translating the design D_{50} to a mass (weight) and relate it to the volume of the layer/extent. 50.000 weight classes (with steps of 1 kg) have been defined in which all volumes and their corresponding rock design weights are sorted. regular design sortings can be used as well. In this way, a distribution of the volume percentage per stone mass class is conceived (see table 4.4). Which can be cumulatively plotted to compare with the yield curve (figure 4.12).

Table 4.4: Volumes required per weight class of stones

Volumes per	**Cross section 1**		**Cross section ...n**			
weight class [m^3]	**Layer 1**	**Layer...n**	**Extent 1**	**Extent...n**	**Total volume**	**Percentage**
0 - 1 kg	120	...	154	...	12.347	0.010 %
1 - 2 kg	...	...	...	...	13.403	0.012 %
2 - 3 kg	...	...	...	...	15.948	0.014 %
... - ... kg	...	...	...	...	...	...
49.999 - 50.000 kg	...	...	...	...	...	...
Total dam volume					**9.645.949**	**100 %**

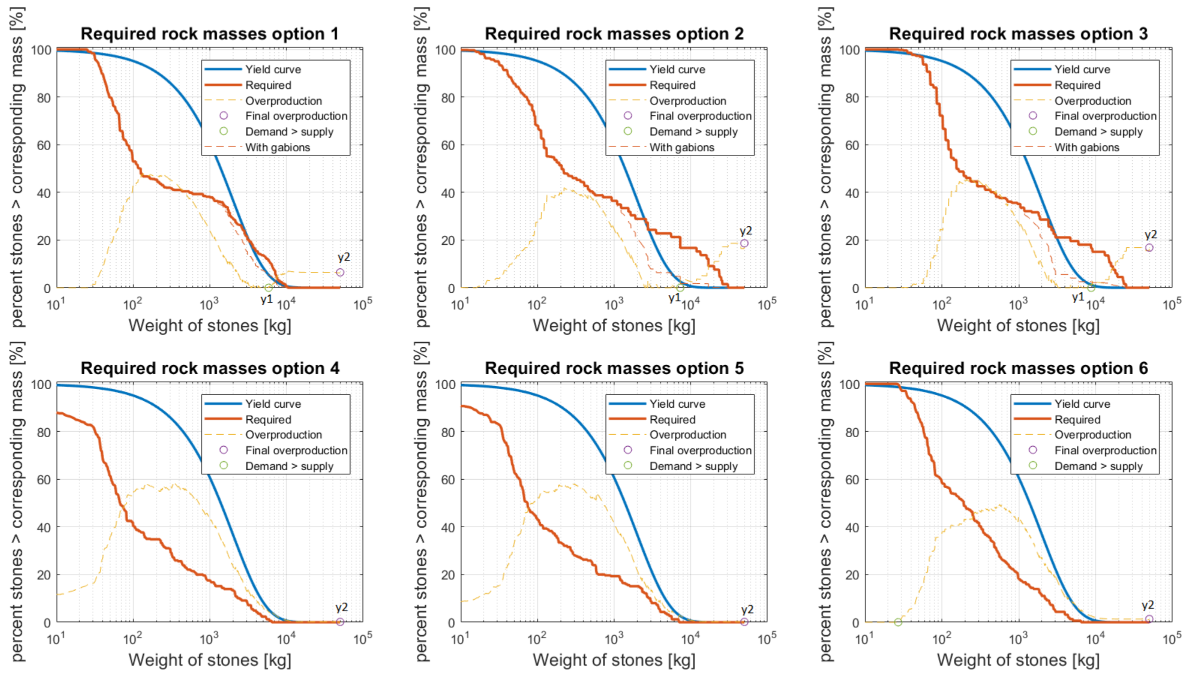


Figure 4.12: Example of output: quarry yield curve with expected volumes per stone class. Again, strategies range from fully horizontal (option 1) to fully vertical (option 6)

Interpretation of the results The results from table 4.4 are displayed in figure 4.12 for 6 different closing strategies and compared to the assumed quarry yield curve. A quarry yield curve expresses the available percentage of stones lighter (smaller) than the corresponding mass (cumulative percentage). For that reason, the results from table 4.4 are also cumulatively displayed. Please keep figure 4.12 within sight while reading this section.

The basic idea is that no overproduction is required if the demand line (orange) is completely below the supply line (blue). However, if it does cross, this doesn't directly mean an overproduction is required. The gap in the supply could be filled with larger stones (moving to the right of the graph) of which there is an excess (supply>demand). A third relation (relation 4.17) is introduced which is based on this principle to define the overproduction required (yellow line). This relation is defined as the cumulative difference of the derivatives and represents how much cumulative overproduction is required for every increasing stone class which can't be supplied by the total supply of smaller or equally sized stones.

$$P_{over}(w) = \sum_{1:w}^{w=1:50000} \big(V_{p,demand}(w) - V_{p,demand}(w+1)\big) - \big(V_{p,supply}(w) - V_{p,supply}(w+1)\big) \quad \text{with} \quad P_{over}(w) > 0 \tag{4.17}$$

In which:

$P_{over}(w)$	=	Cumulative overproduction at every stone weigth class	[% for stone classes $1:w$]
V_p	=	Volume percentage of rocks (from demand or supply curve)	[% of dam volume]
w	=	weigth class stones (step size 1kg)	$[kg]$

To understand this yellow line, read it from left to right (small rocks to large). If the line increases, more volume is needed of that specific stone size than can be delivered by the supply and so the overproduction required increases with every step (the line is still cumulative). However, when it decreases, the supply of that stone size is larger than the demand and the overproduction required decreases (demand of small stones can be filled by larger stones). The line can't go below zero, This would mean a storage is build up of smaller stones that would fill gaps in the larger stone requirements which is not possible. At the end, the line might not reach zero anymore which represents the volume of overproduction required (y_2). However, the stone sizes linked to this volume are still unknown. The smallest stone sizes required for overproduction is defined by the moment the line leaves zero (point from whereon the supply will always be smaller than the demand (y_1)). Now the volume of overproduction is known and the smallest stone size which needs to be overproduced.

The actual extra production depends on the quarry supply. For example, if only 20 % of the dam volume needs to be overproduced for a specific stone size, but only 1 % can be delivered, the total extra production required is 20 times the total dam volume! This factor of overproduction (F_{over}) is defined as the maximum ratio of the cumulative supply and demand curve between points y_1 and y_2 (relation 4.18) and figure 4.13.

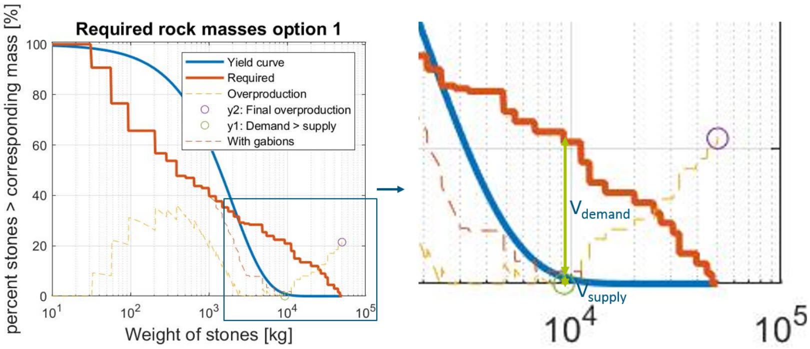


Figure 4.13: Example factor of overproduction: Largest ratio of demand and supply

The total volume that the quarry is required to produce extra ($V_{o,t}$) is defined by relation 4.19. The unused extra rock produced by the quarry is defined by the total extra production minus the extra production required for the actual dam construction.

$$F_{over} = max\Big(\frac{V_{p,demand}(w)}{V_{p,supply}(w}\Big) \quad \text{for} \quad y_1 < w < y2 \tag{4.18}$$

$$V_{o,t} = F_{over} \cdot V_{dam} \tag{4.19}$$

In which:

F_{over}	=	Factor of extra production of dam volume	$[-]$
y_1	=	Point of smallest stone size that requires overproduction	$[kg]$
y_2	=	Point of extra volume required for actual dam construction	[% of dam volume]
$V_{o,t}$	=	Total extra production required by quarry	$[m^3]$
V_{dam}	=	Total dam volume	$[m^3]$

Use of Gabions In practice, this extra production of stones can be extremely high due to the need for large stones. A solution to this problem is to use gabions (nets with small rocks), which are more expensive to produce than rock, but might offer a cheaper solution than overproduction. By using gabions, volume is transferred from the right side of the quarry curve (where supply is low) to the left side (where supply is high). The volume of gabions required is defined by the earlier defined required overproduction percentage (y_2). This is the volume of large stones that can't be supplied. Gabions usually consist of rocks between 1000 and 3000 kg. In this way, the volume percentage of gabions is equally divided between these weight classes resulting in a change of the demand curve 4.14.

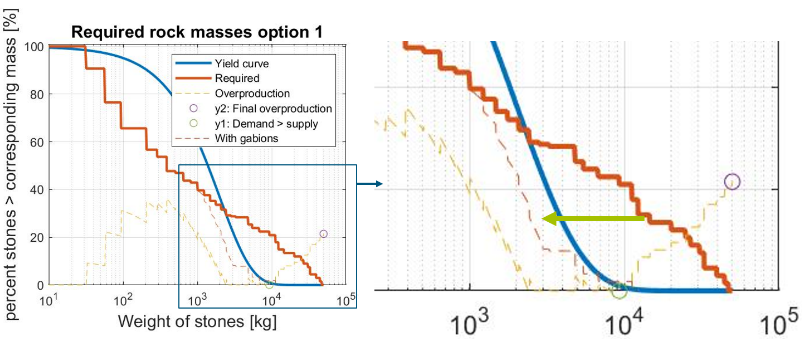


Figure 4.14: Example change in quarry yield curve due to using gabions (1000-3000 kg) instead of large rocks

For each strategy, the price of overproduction and gabions is computed of which the lowest is chosen. In figure 4.12, options 1, 2 and 3 have large overproduction requirements, so using gabions is the cheaper option. The adjusted demand curve (with gabions) is therefore also displayed in figure 4.12.

4.4.4. Computation of sluice caissons

4.4.4.1. Relevance and setup

The function of sluice caissons is to facilitate a sudden closure, if required. However, this is always an expensive solution and therefore not preferred by designers. Royal Haskoning estimated in 1998 that the rocks sizes would become too large to manage if the flow velocity crossed the 6 m/s value and sluice caissons should be implemented to finish the remaining gap. The idea to incorporate a threshold value from which point a sudden closure is required is good and it's one of the only strict boundary conditions which Haskoning revealed in the pre-feasibility report from 1998. In this study, a threshold value can be used by the model to compute the remaining gap size and therefore the volume of caissons required. This will vary with every strategy and is therefore important to incorporate in the evaluation of the optimization model. The amount of caisson volume can be added to the total material costs. The rock computation is also affected by this since the amount of volume taken up by the caisson is not required in rocks anymore.

4.4.4.2. Practical implementation in optimization model

In this section, the practical implementation of the threshold value for the flow velocity in the optimization model is further explained.

First, the complete model is run after which all possible strategies are evaluated. The designer can now evaluate if the strategy is feasible by assessing the rock sizes and the equipment required to execute the strategy. Also, the occurring flow velocities per strategy are presented to help with this assessment. The model can't perform the complete feasibility assessment since it depends mostly on practical limitations of the available materials and equipment capacities (which are considered unknown in this study). However, the model can act on a threshold value (flow velocity) for rocks, for when crossed, the strategy is continued with caissons instead of rock. The threshold value can be altered to optimize between *extra material cost* and *extra equipment cost*. Although in practice, a higher threshold value will always be preferred since the amount of (expensive) sluice caissons required is minimized. The reason for this is that the equipment model is still too simplistic, because it doesn't include the use of larger equipment for larger stones sizes. For this reason, the threshold value of 6 m/s is adopted (Haskoning, 1998), since it's the only known practical boundary condition. The feasibility still requires assessment by the designer or contractor which can determine if this boundary condition is correct or can be increased. The result of this model presents a second evaluation *with caissons* which can be compared to the first evaluation *without caissons*.

4.4.4.3. Computation caisson volume

In this section, the computation behind the required volume of caissons is elaborated in detail. First for a single cross-sectional model (basis) and secondly for a more complex multi cross-sectional model.

Single cross section To acquire the volume of caissons required, the location and time where the threshold value for the flow velocity is exceeded should be measured in the model. For a fully horizontal or vertical closure with a single cross section (see figure 4.15), this is relatively straightforward.

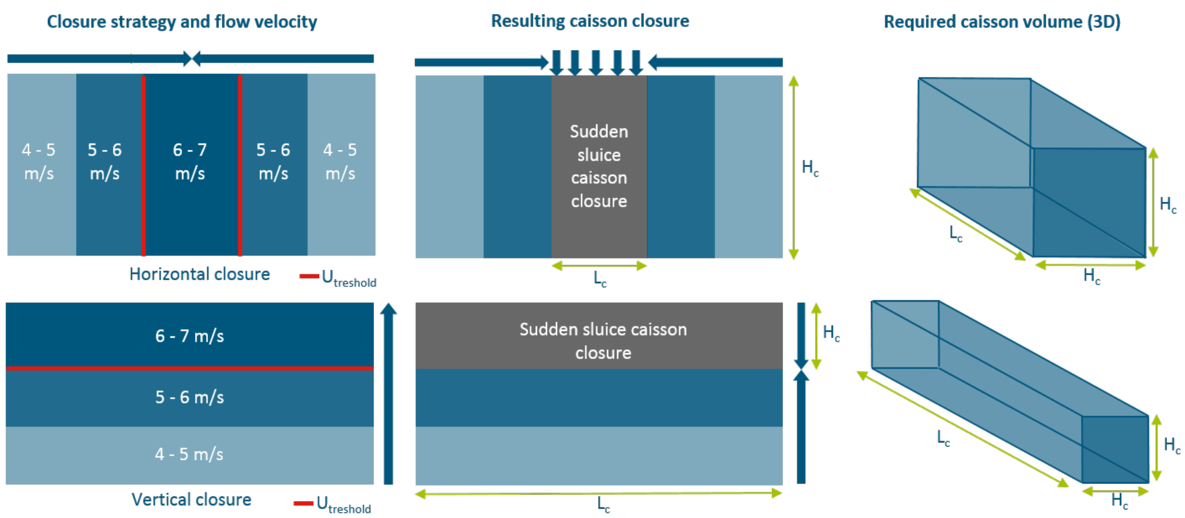


Figure 4.15: Example determination caisson volume

The cross section which is leftover after the threshold is exceeded is the cross sectional area of the caisson. The width of the caisson is set to be equal to the height, because in practice, the width will change in ratio to the height of the caisson. Although sluice caissons are not a solid volume of concrete (figure 4.15) , but actually slender structures, the cost will also increase in ratio with the length and height. To compare closure strategies, the solid outer volume therefore suffices to represent the costs and is defined by relation 4.20

$$V_c = L_c \cdot H_c^2 \tag{4.20}$$

In which:

V_c	=	Volume of caissons required	$[m^3]$
L_c	=	Length of caissons required	$[m]$
H_c	=	Height and width of caissons required	$[m]$

Multiple cross sections When dealing with several cross sections, the computational complexity of the caisson requirements increases.

In the first loop, the cross sections, time step and corresponding phases where the threshold value is exceeded is stored. The time step relates to the status of the closure in that cross section and the remaining width and length of the cross section is stored just like in the one cross-sectional model. Now the execution scheme is adjusted by first stopping the preprogrammed gradual closure in that cross section (horizontal or vertical) and implement a partial closure until the earlier stored remaining depth (in case of a vertical closure) or length (in case of a horizontal closure). In case no closure is executed in that phase, all preprogrammed closures in later phases are deleted. Secondly, a sudden closure is implemented in the scheme in the next phase, closing the remaining gap within one tidal cycle. The rest of the scheme is performed as programmed. This is exactly the same as in the one cross-sectional model.

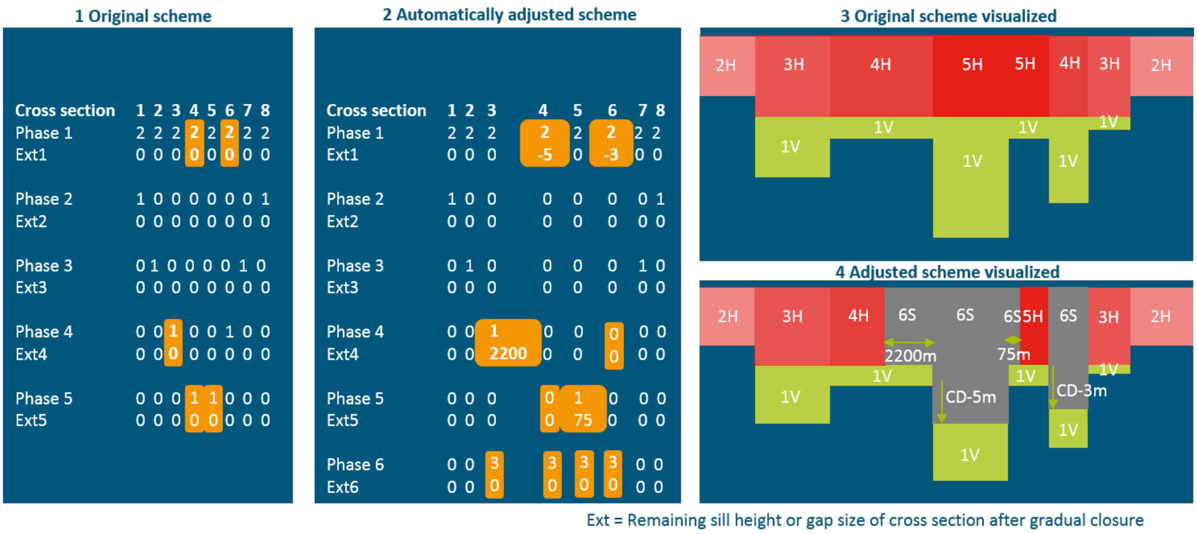


Figure 4.16: Example automatic scheme adjustments

However, this creates a problem. If a cross section is closed suddenly in the phase where the threshold is exceeded, other cross sections, where this threshold was not exceeded will experience increased flow velocities and possibly also exceed the threshold value in later phases. To solve this, the sudden closure needs to be executed at the end of all phases, which is also logical in practice. In the model, this is programmed in. An example of the outcome of the model is depicted in figure 4.16 where a combined closure is automatically adjusted by the model.

In figure 4.16.1, the original scheme of the combined closure is presented and the locations and times where during this phase the flow velocity exceeds the threshold value (orange). The scheme is then automatically adjusted by the model to the figure 4.16.2, in which all cross sections which exceed the threshold value are partially closed to the value Ext (dark orange) and then closed suddenly in the extra added phase 6 at the end (light orange). The preprogrammed horizontal gradual closure in phase 4 and 5 are deleted because they are not necessary anymore (red). In figures 4.16.3 and 4.16.4, the original and adjusted schemes are visualized for a better understanding. As can be seen, the sudden caisson closures are depicted in gray. From the measurements of these caissons, the total caisson volume is computed.

After this first loop, the model is tested again to see if the flow velocities indeed stay under the threshold value. However, this is not the case. the problem can best be described with the example from figure 4.16. In cross section 4, the sill level is adjusted to stay at CD -5 m in the first phase, because increasing it further increases the flow velocity exceeding the threshold value. However, even with this adjusted sill level, in phase 2 other cross sections are closed which also increases the flow velocity in cross section 4. To solve this, a design loop is created which measures the flow velocity and adjust the execution parameter ext and until the flow velocities are all under the threshold value. As example cross section 4 from figure 4.16 is used again. The adjusted sill level (CD - 5 m) is decreased with 0.5 meters to CD -5.5 m after which the flow model is executed. Flow velocities in all cross sections are measured and in cross section 4, the flow velocity still exceeds the threshold value. The sill level is adjusted again with 0.5 meters until the flow velocity in that cross section is below the threshold value.

Summary and results A model is created to introduce the first adjustments to the scheme automatically (figure 4.16) and implement sudden closures where needed. Secondly a model is created to adjust the execution parameters until all flow velocities are under the threshold value. The result is a scheme where the required caisson volume is computed keeping the flow velocities in all cross sections below the threshold value. A result of the actual model with 8 cross sections is presented in figures H.5 - 4.20. In this case, a threshold value of 7 m/s was used.

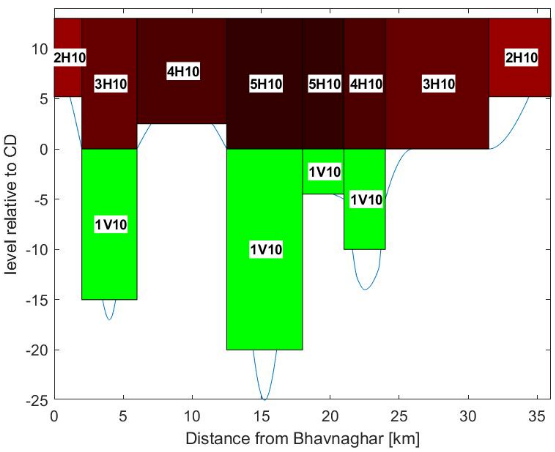


Figure 4.17: Scheme of unadjusted closure strategy option 3

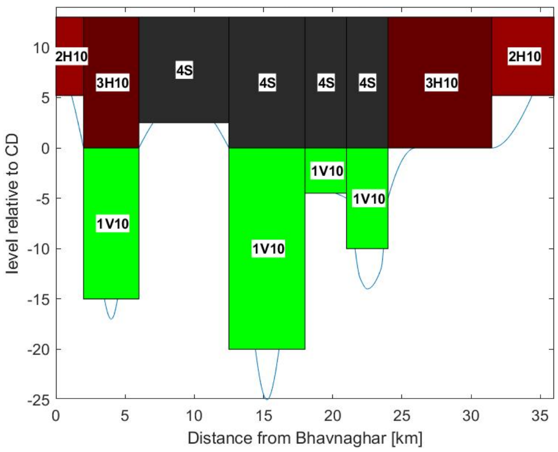


Figure 4.18: Scheme of adusted closure strategy option 3

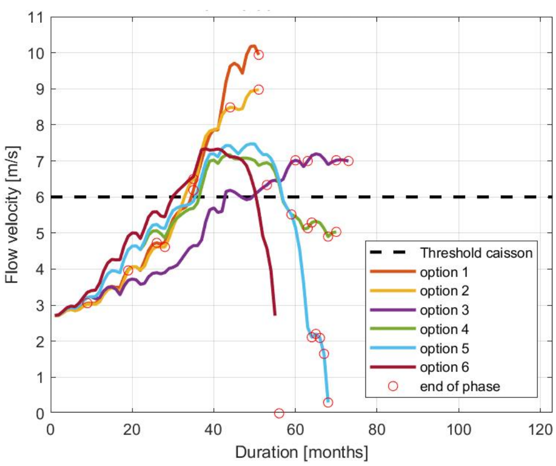


Figure 4.19: Flow velocities: for 6 strategies from fully horizontal (option 1) to fully vertical (option 6) before use of caissons

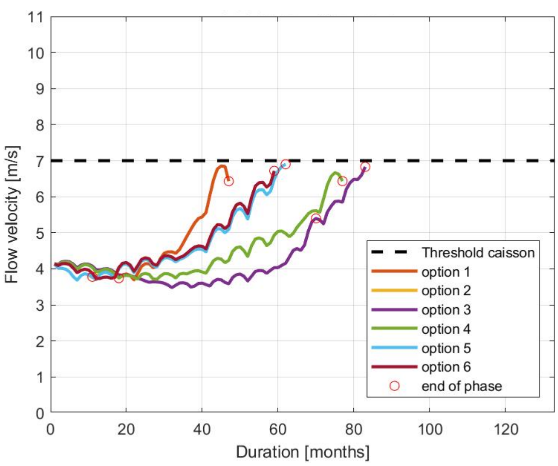


Figure 4.20: Flow velocities for 6 strategies: From fully horizontal (option 1) to fully vertical (option 6) after use of caissons

4.4.5. Results both models

The output of the models can be shown in one bar graph depicted in figures 4.21 and 4.22 without and with the use of caissons respectively.

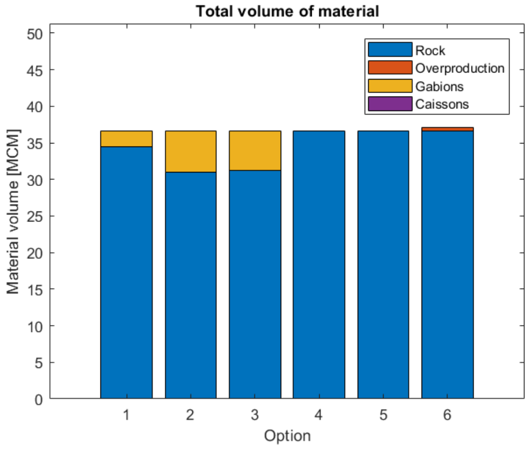


Figure 4.21: Material volumes for 6 strategies without caissons: From fully horizontal (option 1) to fully vertical (option 6)

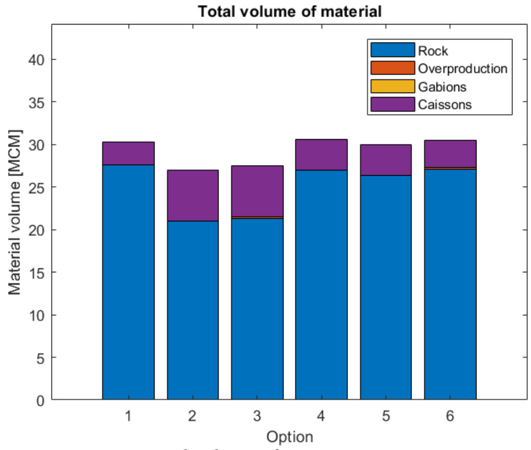


Figure 4.22: Material volumes for 6 strategies using caissons: From fully horizontal (option 1) to fully vertical (option 6)

4.4.6. Discussion

Rock model The probabilistic approach of the rock model increases the accuracy and decreases the bandwidth of the computation result. However, the computation requires long computational times if a small failure probability is chosen. A certain amount of minimum failures is required to perform a trustworthy Monte Carlo simulation. In this case, it is up to the user to define the minimum amount of runs with respect to the chosen minimum amount of failures required. In the case of a failure probability of 1/100, a reliable minimum amount of runs would be 1000 so at least 10 failures are needed to get to this failure probability.

By utilizing the calibrated Shields equation, scale problems can occur. The equations can't be used linearly with increasing scale. For this reason, the designer should be alert on the values used in the equation with respect to the size of the dam and resulting rock sizes. These stability deviations due to scale problems could be tested with a full sized physical model of the cross section or with a validated FPS model. The Shields factor could be used to calibrate the stability of stones in this model to match the FPS model.

As was mentioned earlier in this section, the damage cost due to volume loss is not included. Therefore, no direct consequence is quantified of lowering the accepted failure probability. One of the main recommendations to improve the rock model is to include the damage (or loss) of material. This could offer another important risk consideration that can be optimized with the construction capacity.

After performing the case studies in chapter 6, it was concluded that the stone sizes resulting from the model are relatively small with respect a deterministic computation. A more detailed elaboration on this discussion point can therefore be found in section 6.4.4

Caisson model With regards to the caisson model, the complexity of the scheme adjustment is great. The automated model is robust up to some level of strategy complexity. An example of a non-practical outcome can be seen in figure 4.16, where cross section 5 is partially closed horizontally, while a sudden closure is implemented on both sides. The horizontal closure could be performed there, but it would require a temporary work island which would increase the cost dramatically. However, if any of these impracticalities occur in the model, they can easily be adjusted by the model user.

Furthermore, caissons require placement before the closure starts. If a caisson is computed on top of a partial vertical closed layer, this layer must be executed in phase one without performing any other closures at the same time. In many resulting strategies, this will be the case. But if this is not the case, an extra run is required at the end to implement this extra phase in the model.

Lastly, other materials or caisson shaped units can be implemented in the model to function as sudden closure. However, the characteristic time scales to place these units must be adjusted. For a sudden sluice caisson closure, it is assumed that the steel doors are closed within one tidal cycle.

4.5. Bed protection model

4.5.1. Introduction

In this section, the probabilistic model to compute the required quantity of bed protection is elaborated in detail. The cost of bed protection is mostly related to the material volume that needs to be placed at the location. The goal of the model is to compute the volume of bed protection required for a safe closure. That is, without the presence of dam instabilities caused by failure of the slope of the scour hole behind the dam. More information on the theory behind the development of scour holes and the dam stability can be found in appendix D.

In subsection 4.6.2, some general considerations (choices and summary) about this model are discussed to explain why certain decisions have been made and to give a short summary of what the model does. In subsection 4.5.3, the computation to determine the volume of bed protection is explained . In subsection 4.5.4, the results can be found and are discussed in subsection 4.5.5.

4.5.2. Considerations and summary

The most important considerations during the development of the bed protection model are summarized here:

- Secondary influences on the total cost of bed protection are the type and placement costs. However, both influences indirectly relate to the required volume and therefore only the volume will be used as cost influence in this study. The type is directly related to the thickness required and the placement costs relates to the placement capacity which directly relates to required volume.
- The model computes the required length probabilistically with Breusers equations. Secondly, it computes the required area based on the resulting length and the strategic execution scheme. The required thickness depends on the type of bed protection which depends on the design flow velocity at the dam. At the end of the bed protection, less protection is required and therefore the minimum type class is used here. The average thickness of the complete area is the average of the assumed linear gradient between the design thickness (at the dam) and the minimum thickness (at the end). This thickness normally decreases exponentially and is therefore overestimated in the model.
- The placement of all required bed protection always takes place before the closure starts to avoid placement with high flow velocities. However, it is assumed that development of the scour hole behind the dam starts at the start of the closure. In principle, the scour hole will not develop too much in this time since the flow velocities are relatively low at the start. Also, it can be argued that during the initial phases of the closure, the flow velocity doesn't change drastically. It is possible to place the bed protection in this initial phase.
- The model is semi-probabilistic because the required area is probabilistically computed but the thickness is deterministic and follows from one of the pre-defined classes of bed protection. Therefore, the computation of the volume becomes semi-probabilistic

4.5.3. Computation

4.5.3.1. Relevance probabilistic approach

A probabilistic model is used to compute the required length of bed protection. Just as the computation of the required materials, a probabilistic (Level III) approach allows to gain insight into the failure probability of the bed protection during a tidal closure. The probabilistic approach applies statistical distributions for all parameters in the computation. The probabilistic model applied in this study uses the allowed failure probability per layer of bed protection behind a cross section.The failure criterion is defined as a collapse of a part of the dam due to instabilities of the soil during the execution of the closure. The length of the failed dam section is equal to the length of a cross section. In practice, a smaller dam part will collapse, since conditions will vary along the dam. However, in this study, it is assumed, the conditions are the same everywhere across the length of the cross section (no length effect). The computed situation for one cross section is depicted in figure 4.23.

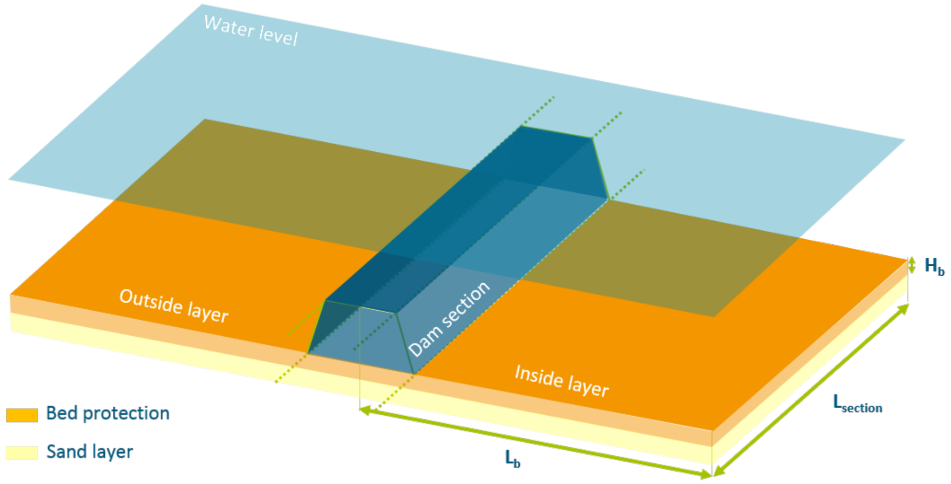


Figure 4.23: Example bed protection requirements for one cross section (dam element) in the dam

The probabilistic model can be applied as a basis for a risk analysis. For a risk analysis, actual damages associated to the risk of failure have to be investigated in more depth. However, this is not performed in this research and therefore the accepted failure probability can be maximized without consequence just as in the probabilistic model to determine the rock sizes. For this same reason, the accepted failure probability is assumed to be constant (In this case 1/10000 per layer) and can be compared to a minimum safety level per layer.

Time is an important aspect in the computation, since the scour hole behind the bed protection develops in time (figure 4.25). Just as the rock computation, the consideration to be made is execution speed (equipment cost) versus required bed protection. increasing the execution capacity will decrease the quantity of bed protection required and vice versa.

4.5.3.2. Computation probabilistic model

Length

The model used in this study computes the required bed protection length for *each dam section* in *each phase* probabilistically. To perform this computation, the complete method by Breusers shown in section 2.4 is too elaborate, the amount of parameters used is large and can be reduced significantly. Therefore some simplifications are made to still perform an accurate calculation but parameterize some definitions. The complete set of reduced parameters and their distributions are discussed in appendix G.1.6.

As was mentioned earlier, the stability of the dam is determined using the depth of the scour hole (h_s), the angle of the inner slope (β) and the angle of the slope after a possible soil slide (β_f). The problem is geometrically depicted in figure 4.24. An important note is to determine the depth of the scour hole, the flow velocity at the end of the bed protection is used. This is derived from the flow velocity on the sill u_0 with use of continuity $u = u_0 \cdot \frac{hsill}{h0}$. The required bed protection length is therefore less in vertical closures.

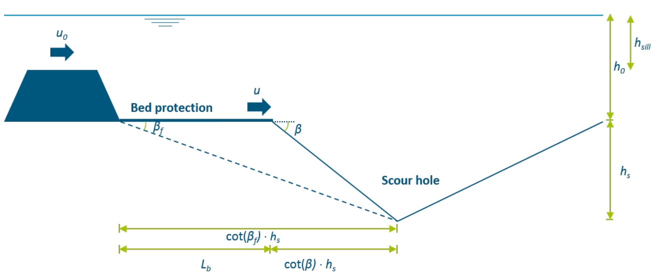


Figure 4.24: Geometric relation for the length of bed protection

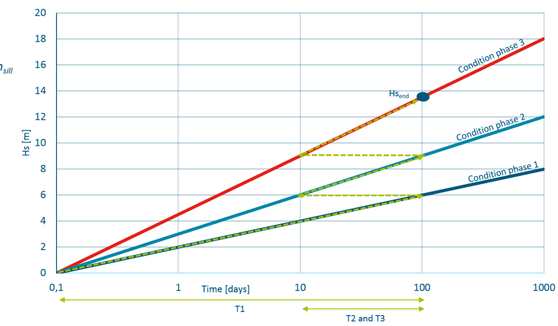


Figure 4.25: Scour hole development in phases

During a construction period, the conditions per construction phase vary from each other. Using the average velocity over the full length of the construction period is not the correct in defining the scour hole depth. Instead, the scour hole development under average conditions of the phase is computed over time, for each phase. e.g. a scour curve is computed for the average conditions in the phase starting at t = 0 (figure 4.25). At every end of the phase, the line is horizontally continued to the next scour development line with different conditions until the last phase is reached. The theory that founds this procedure is that the shape of the scour is similar under all conditions. This is one of the most important findings in scour research (Schierkeck, 2012). Failure of the dam is realized if the dam fall into the scour hole. The limit state function for this phenomenon is defined by relation 4.21.

$$z = L_b - h_s \cdot \left(cot(\beta_f) - cot(\beta)\right) \tag{4.21}$$

With use of a Monte Carlo Simulation (MCS), the limit state function can be evaluated for each dam section and for each phase. For each dam section, the maximum length required of all phases is normative. This is further elaborated in the next paragraph . Instead of using a distribution for the length L_b, and iteratively solve the mean of the distribution (as was done with the D50), the distribution is computed using the MCS. In this distribution, the value with the respective failure probability of 1/1000 is chosen as design length for that section, because its a temporary measure.

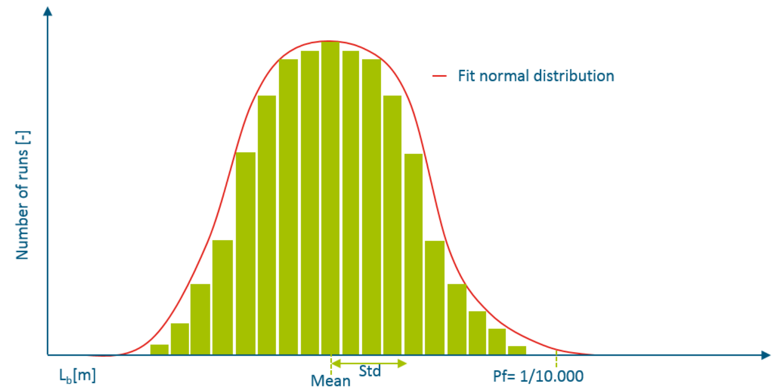


Figure 4.26: Resulting distribution of the length of bed protection required

Area

With the required length, the area can be computed. However, this differs for each type of closure (horizontal or vertical). The difference can be logically explained by the execution procedure of both closures (figure 4.28). In case of a vertical closure, the maximum velocity during a phase occurs across the complete section length. In case of a horizontal closure, this is only at the end (middle of the section). At the start (edges of the section), no bed protection is required because they are covered directly with stones. The actual gradient (from start to end) would be exponential, since the flow velocity increases exponentially (see figure 4.28), but in the model it's linearized. This is a safe assumption, since the exponential line is always under the linearized line.

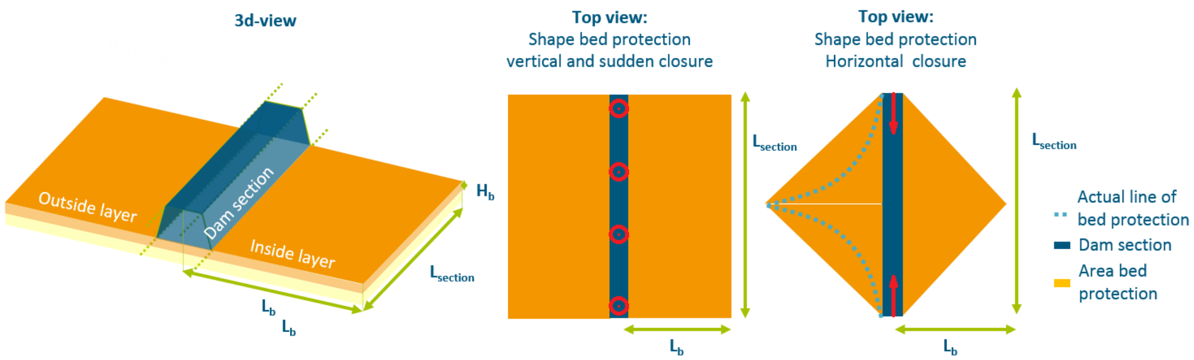


Figure 4.27: Definition of bed protection area for different closing methods

For more phases, the computation becomes more complex. Each sequential phase, requires information about the previous phases. even if nothing happens to a cross section, the bed protection should be computed because it might be the normative phase. To clarify, two examples of closure strategies for one cross section are discussed in figure 4.28. on the left side, a partial vertical closure is done in the first phase, which requires a square shaped bed protection with length L_{b1}. Following this phase, nothing happens for two phases, but the flow velocity (and time) increase and therefore the bed protection length increases during these phases to L_{b2} and L_{b3}. In the last phase, the section is closed horizontally with a normative length of L_{b4}. it is important to store what happened in the previous phase and this should be used to compute the next required area. The total area required is defined by relation 4.22.The example on the right has a partial horizontal closure, after which only a small section is left. The total area required is defined by relation 4.23

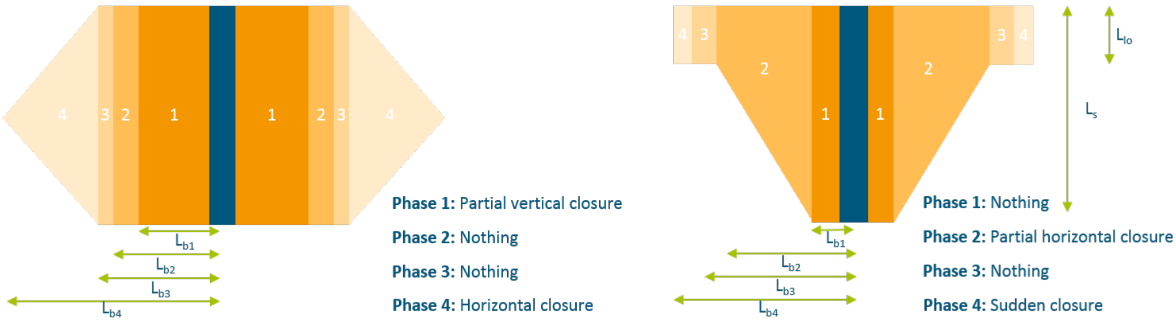


Figure 4.28: Definition of bed protection area for different closing methods with phases

$$A_{bp} = L_{b1} \cdot L_s + (L_{b2} - L_{b1}) \cdot L_s + (L_{b3} - L_{b2}) \cdot L_s + \frac{1}{2}(L_{b4} - L_{b3}) \cdot L_s \tag{4.22}$$

$$A_{bp} = L_{b1} \cdot L_s + \frac{1}{2}(L_{b2} - L_{b1}) \cdot L_s + (L_{b2} - L_{b1}) \cdot L_{lo} + (L_{b3} - L_{b2}) \cdot L_{lo} + (L_{b4} - L_{b3}) \cdot L_{lo} \tag{4.23}$$

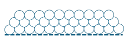
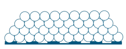
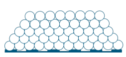
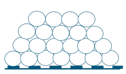
In which:

A_{bp}	=	bed protection area required per section	$[m^2]$
L_{bn}	=	Length of bed protection required for phase n	$[m]$
L_s	=	Length of the dam section	$[m]$
L_{lo}	=	Leftover dam section after partial horizontal closure	$[m]$

Type and resulting volume

Several types of bed protection exist. However, which one is best for the given situation depends on the maximum flow velocity occuring during the closure (design velocity). This determines the stability requirements for the top layer of the bed protection under which a filter is needed to prevent erosion of sand below the top layer. In 1998, the Dutch ministry of transport and public works designed four protection types for Haskoning (Wiersema and Broos, 1998). They distinguished the classes into *loose*, *light* ,*heavy* and *very heavy* each with corresponding range of design velocities (velocity behind the sill). All types are displayed in table 4.5.

Table 4.5: Types of bed protection used for initial design (Wiersema and Broos, 1998)

Type	Description	Design velocity [m/s]
	**Loose**:Geotextile filter layer +0.5 m stones	$u < 3$
	**Light**: Concrete block mat +0.8 m stones	$3 \leq u < 4$
	**Heavy**: Concrete block mat +1.0 m stones	$4 \leq u < 5$
	**Very heavy**: Concrete block mat +1.5 m stones	$u > 5$

The design velocity on each side of the dam section is measured using the flow model and the resulting type of bed protection required is noted. The volume of bed protection required is the required area multiplied with the layer thickness of the design type of bed protection.

4.5.4. Results

The result of the bed protection model can be displayed in a single bar graph comparing the required volume of bed protection of all six tested strategies (figure 4.29). The shape of the bed protection is not displayed, since this is involves complex unnecessary coding. The type of bed protection (although irrelevant for the evaluation) is displayed to create more insight into the meaning of the result.

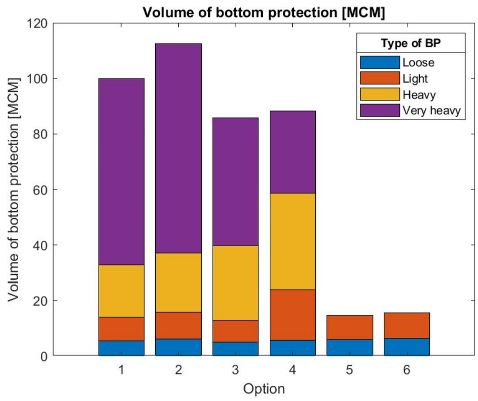


Figure 4.29: Volume of bed protection divided in required types for 6 strategies without caissons: From fully horizontal (option 1) to fully vertical (option 6)

4.5.5. Discussion

The validity of the Breusers equation for the depth of the scour hole is within a certain range and has been known to overestimate results with increasing scale. From the measurement during the Deltaworks program, the scour holes reached maximum depths of 4 times the water depth. Because the Kalpasar closure dam is relatively large in comparison with the scale from Breusers experiments, the maximum scour hole depth of 4 times the water depth is adopted. In some cases, this can still result in scour depths of 80-90 meters.

Two: The approximated cross sections with a flat bottom do not represent the actual shape of the bathymetry of the channels and flats. The bathymetry could also change significantly after 500 meters (order of length of bed protection). However, for a first-order comparison, this will not vary within strategies and can therefore be used.

Three: The average velocity per phase phase is utilized to determine the design flow conditions with which the scour hole depth is computed. If a small amount phases are used during the execution, using average conditions per phase can result in an underestimation of the actual scour hole depth, since the scour depth can increase exponentially with increasing flow velocity in certain stages of the scour development process. To account for this, average conditions per day or per month can be used depending on the duration of the closure.

Four: The model assumes only that one soil layer is present with homogeneous sediment characteristics. Scour hole development can significantly be influenced by secondary soil layers with varying sediment characteristics. Extra layers can be implemented in the model with relative ease by adjusting the sediment characteristics after a certain scour depth is reached.

Five: As was mentioned earlier in the discussion of the rock computation, the damage cost due with failure is not included. Therefore, no direct consequence is quantified of lowering the accepted failure probability. One of the main recommendations to improve the bed protection model is to include the damage after failure.

Lastly, from literature (CETMEF et al., 2007), it is known that accelerations of the flow can have a significant effect on the scour hole depth. Since large accelerations are present in the Gulf of Khambhat due to the large tidal amplitude, the model from Breusers will not accurately describe the scour hole development. Physical model test can indicate the deviations caused by the negligence of the acceleration in the flow. From this, it could be concluded if the Breusers equation can be utilized in this optimization model. Otherwise a calibration can be performed using the Shields parameter.

4.6. Equipment model

4.6.1. Introduction

In this section, the equipment model is elaborated in more detail. The model is based on basic equipment which is often used in closure operations. The related costs can be used in the final evaluation. Three elements influence the total cost of equipment: Type, quantity and operation time. These three elements are included in the model and the total cost is then computed using a cost function.

4.6.2. Considerations and summary

The most important considerations during the development of the equipment model are summarized here:

- A horizontal closure is executed with small dump trucks with a load volume of 10 m^3. More dump trucks can be used to increase capacity. However, a maximum capacity of 12.500 m^3 of material per day is set based on a 60 second interval between two dumps at one bund. The operation time is defined by the loading, unloading and driving times. Secondly, space on the crest is required and is therefore set at a minimum of 7 meters to provide room for the dump trucks. Furthermore, every 100 meters a larger bund is created for trucks to turn around. This is included as 1 meter of extra crest width (in material volume).
- A vertical closure is either executed with dumping ships (sill heigth below CD+ 2 m) or with use of a cable way/bridge (sill height above CD +2 m). Dumping ships are considered the cheapest option but are restricted in their operation by the water depth. They can dump one load per high water which results in a capacity of two loads per day. The amount of ships can unlimitedly be increased by increasing the capacity. A cable way/bridge system is defined as one type of equipment with a large constant capacity (based on previous cable way systems) which can't be increased.
- A sudden closure requires special equipment which is not determined by the model. The cost of this equipment is included in the volumetric cost of the sluice caissons.
- Base prices for the use of one unit of equipment are used (1 truck, ship or cable way), upon which an daily operational rates are added for trucks and ships. For a cable way, the required length is added to the price.

4.6.3. Computation

Horizontal closure

If a section is closed horizontally small dump trucks are deployed to deliver the dam material to the end of the bund and dump it. Some assumptions are discussed in the following points to define the boundary conditions for the computation. All assumpotions are based on the Saemungeum closure works (Wallingford, 2005)

- Per bund, small maneuverable dump trucks of capacity 15 tons (assumed able to deliver 10 m^3 of volume) will be deployed. These trucks will be deployed equally both sides of the gap. These trucks have been used at the closure of the Saemunguem dam and can be assumed to be the optimum trucks for use with respect to the required width of the dam crest for driving to and from the drop point.
- Construction will take place for 22 hours out of 24, the remaining 2 hours per day being allocated for work force shift changes.
- On average one dump truck is expected to be discharging its load every 60 seconds on both sides of the gap. So the minimum time between consecutive dumps is 60 seconds which results in a maximum capacity of material dumping at one bund of 12.500 m^3 per day.
- The trucks have a top speed of 40 km/h.
- The loading points where the material is stored is at the land connection of the dam
- The rock material is already quarried and in stockpiles ready for use.
- Gabions can be placed by end tipping from trucks in a similar way to that envisaged for the rock fill.

To compute the quantity of trucks required for the closure of the section, the capacity of the trucks needs to be known. The capacity is defined as the amount of volume one truck can deliver per day. Since the volume per load is known, the time it requires to dump the material, reload and dump again needs to be computed (round trip). Relation 4.24 defines the total time for one round trip.

$$T_{round} = T_{load} + T_{drive} + T_{unload} \tag{4.24}$$

In which:

T_{round}	=	Total time for 1 round trip (1 dump)			$[s]$
T_{drive}	=	Total drive time from and to the bund	=	Depends on distance (variable)	$[s]$
T_{load}	=	Loading time of the truck	=	300 (constant)	$[s]$
T_{unload}	=	Unloading and turn around time	=	60 (constant)	$[s]$

The distance traveled depends on where the bund is at that moment. This is on average the length of all previous sections closed horizontally plus half the section in question (figure 4.30).

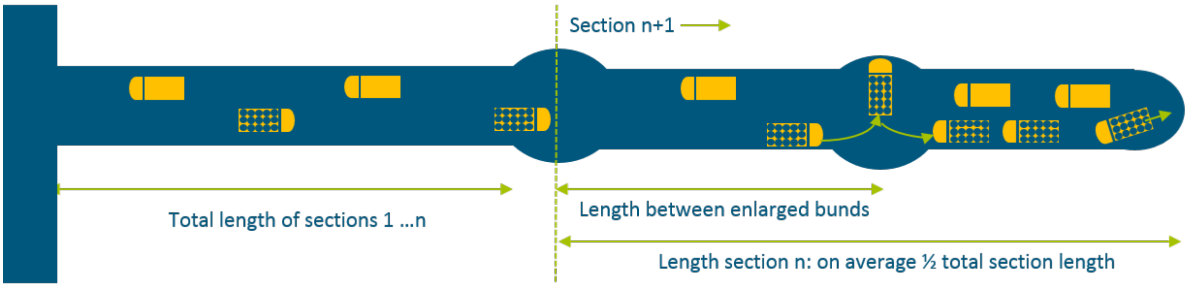


Figure 4.30: Execution scheme of horizontal closure width given length sections and wider bunds for turning

If the round trip time is known, the capacity per day per truck can be computed (relation 4.25). This is used to compute the total amount of trucks required with relation 4.26.

$$C_{truck} = 10 \cdot \frac{T_{day}}{T_{round}} \tag{4.25}$$

$$N_{trucks} = \frac{C_{tot}}{C_{truck}} \tag{4.26}$$

In which:

T_{day}	=	Operational time in a day	$[s]$
N_{trucks}	=	Number of trucks required to close a section	$[-]$
C_{truck}	=	Capacity of one dumping truck	$[m^3/day]$
C_{tot}	=	Total capacity required (strategy input)	$[m^3/day]$

The total amount of trucks during the complete execution is the maximum amount of trucks required of all cross sections. This is what is needed at the peak moment. The operational time follows from the number of trucks needed per section times the closure time. This is summed up for all sections closed horizontally.

Vertical closure

The computation for the vertical closure is less complex. This can either be done with dumping ships or a cable-way/bridge system. some assumptions are discussed in the following points to define the boundary conditions for the computation.

- A dumping ship has a capacity of 3000 tons which is around 1800 m^3 and can dump twice a day during High Water (Van Oord, 2015). Loading and shipping times are not considered normative since only one dump can be achieved during high water. In this way, the high water period defines the round trip time.
- Ship dumping can only be done until minimum high water level – 6.5 m (draught ship) lowest high water = CD + 8.5 m.
- A dumping ship is considered for a vertical closure up to a sill level of CD +2 m. A cable-way/bridge is considered for vertical closure up until dam design height (Van Oord, 2015)
- A cable-way or bridge has a maximum capacity of 2250 tons per hour. Which more than twice the capacity of cable-way used at Brouwershavense Gat in 1972. If needed, this capacity can be increased, but the cost per length measure associated with this increase should also be increased. (Wiersema and Broos, 1998)

The required amount of ships can now be defined (relation 4.27). The operational time follows from the required amount of ships and the closure time. The length of cable-way/bridge system required follows from the dam section length.

$$N_{ships} = \frac{C_{tot}}{C_{ship}} \tag{4.27}$$

4.6.4. Results

The results can be summarized in one bar graph (figure 4.31).

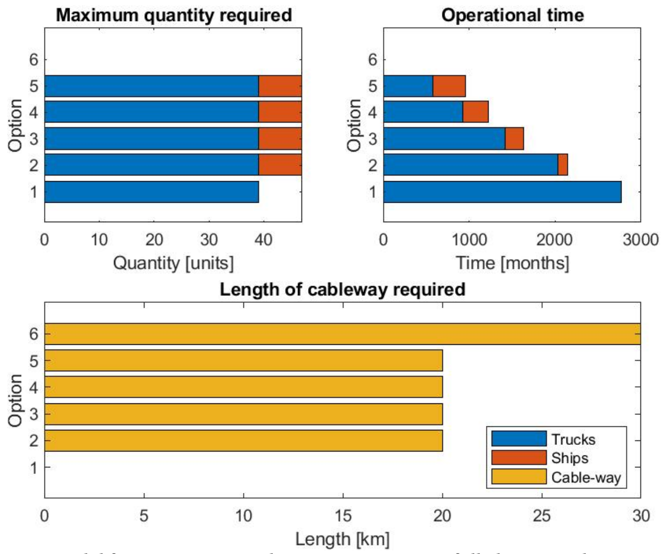


Figure 4.31: Results equipment model for 6 strategies without caissons: From fully horizontal (option 1) to fully vertical (option 6)

4.6.5. Discussion

The computation is based on many assumptions which should be verified or altered by the user if not correct. The used equipment (trucks, ships and cable-way) form the basis of the closure methods and therefore it gives a reasonable first impression of the differences between the execution of vertical and horizontal closures. More detailed information is not required for a first-order evaluation.

An improvement to the model would be including the capacity of a cable-way/ bridge system. Only using the required length to evaluate the costs is not sufficient. the capacity is included but must be raised in balance with the increased cost. Furthermore, the boundary condition for when it is used (depth) should be know prior to programming the strategic options, otherwise impractical and unrealistic cable-way alignments might be a result.

Furthermore, the model assumes all material is already stockpiled at the start of the dam. See appendix section G.2 for an overview of required work area's for these stockpiles. The model assumes a constant import of material so the stockpile is never empty. However, this is not realistic. Transportation delays occur from and the quarry might have a maximum daily capacity which it can produce. These factors can be implemented in the model with ease; A transportation delay could be implemented in the model by increasing driving times. Furthermore, a maximum quarry capacity can be implemented by adjusting the execution scheme to this maximum value.

Lastly, the use of alternative equipment measures and their implementation into the model is not considered. This is not an important factor, because horizontal closures will almost always be executed by dump trucks and vertical closures by ships (up to a certain level). In the final gap, the only interesting aspect is the capacity with which the equipment can deliver the material. If this a sudden closure with caissons, a cable-way or a bridge system is up to the designer. If the costs functions for each of these systems (or other systems) is known with the capacity as a variable, the cheapest equipment which fits the optimal closure strategy can be chosen. Designing new high capacity equipment might offer a solution. For example: Large high capacity cable-way systems could be installed with relative ease considering the heavy lifting equipment which is developed to install offshore wind turbines.

4.7. Evaluation

The evaluation of the strategic closure options is done using the results of all four models previously described. initially, the evaluation was done qualitatively. However, it was decided that this would present scale problems and individual cost factors could not be compared. The evaluation process is now based on costs and includes:

1. A list with elementary costs, (section 4.7.1)
2. Cost functions to translate raw output of the models to cost, (section4.7.2)
3. A summation of all costs, (section4.7.3)

For all cost factors: *material*, *equipment* and *bed protection*. In the following sections, these three elements are explained in more detail. In section 4.7.4, an evaluation poster, combining all relevant output is displayed.

4.7.1. List of elementary costs

To evaluate the total cost, a list of elementary cost is presented in table 4.6. The list contains a minimum amount of cost elements to minimize the amount of extra input parameters.

Table 4.6: Elementary costs

Symbol	**Cost parameter**	**Unit**
CT	Initial cost of truck	euro
CTR	Cost of truck operation	euro/day
CS	Initial cost of ship	euro
CSR	Cost of ship operation	euro/day
CC	Initial cost of cableway/bridge	euro
CCL	Cost of cableway/bridge	euro/m
CMR	Cost of rocks	euro/m^3
CMG	Cost of gabions	euro/m^3
CMCI	Initial cost of caissons	euro
CMC	Cost of caissons	euro/m^3
CMO	Cost of overproduction	euro/m^3
CBB	Cost of bed protection	euro/m^3

4.7.2. Cost functions

The cost functions compute the actual total cost of each cost factor based on the output of the models. This is often based on simplistic multiplications of a volume quantity (output) with an elementary cost per volume. Also for some elements, initial cost have been added (caissons, cable-ways). The cost functions are summarized into three equations (equations 4.28, 4.29 and 4.30) for an easier overview.

Cost functions Equipment

$$C_{equipment} = \sum_{Alltypes} \left(C_{unit} \cdot Q_{type} + C_{operation} \cdot T_{operation} \right) + C_{cable,in} + C_{cable,unit} \cdot L_{cb} \quad \text{For types: } \textit{trucks} \text{ and } \textit{ships} \tag{4.28}$$

Cost functions material

$$C_{material} = \sum_{Alltypes} \left((C_{in}) + C_{unit} \cdot Q_{type} \right) \quad \text{For types: } \textit{rock, gabions, overproduction} \text{ and } \textit{caissons} \tag{4.29}$$

Cost function bed protection

$$C_{bedprotection} = C_{unit} \cdot Q_{type} \quad \text{For type: } \textit{bed protection} \tag{4.30}$$

In which:

C_{unit}	=	Elementary cost of a unit	$[euro]$
$C_{operation}$	=	Elementary cost of operating a unit	$[euro/day]$
$T_{operation}$	=	Operating time of equipment units	$[days]$
L_{cb}	=	Length of cable-way/bridge system	$[m]$
Q_{type}	=	Quantity or volume of type	$[-/m^3]$
C_{in}	=	Elementary initial cost of type	$[euro]$

4.7.3. Results

The results can be summarized in one bar graph (figure 4.31) where all costs are summed up. It can be clearly observed which cost factor contributes the most to the total cost and which option has the least total cost.

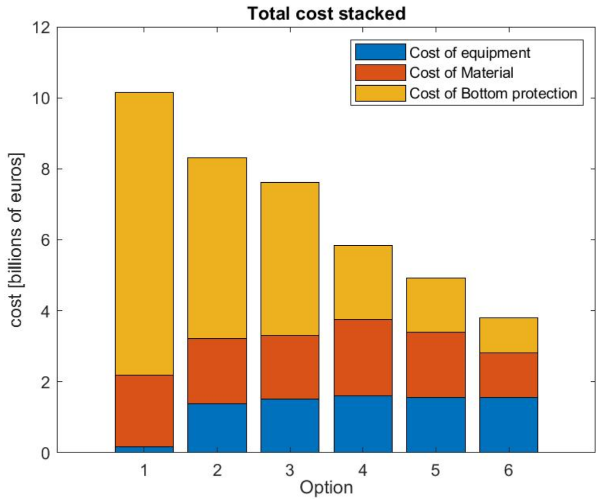


Figure 4.32: Results of evaluation, total cost for 6 strategies without caissons: From fully horizontal (option 1) to fully vertical (option 6)

In this case, performing a vertical closure is significantly cheaper than a horizontal due to minimal bed protection cost. The cable-way/bridge system that is required to perform the vertical closure is relatively inexpensive in compared to the savings from bed protection.

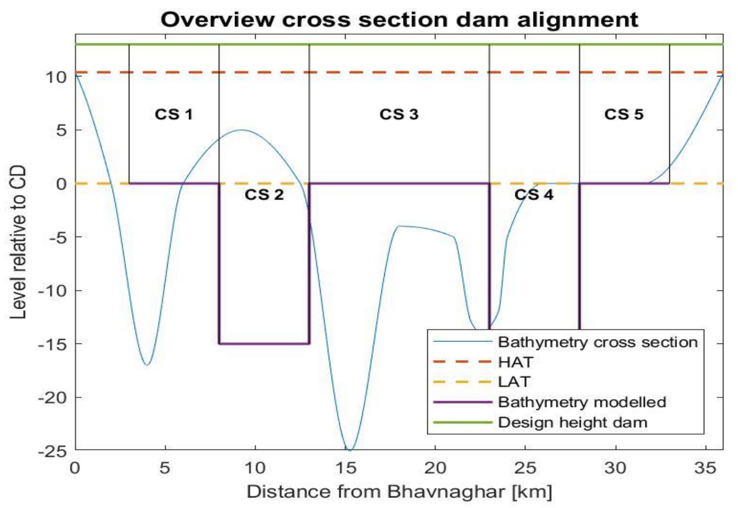
4.7.4. Evaluation poster

To be able to asses all strategic options tested and evaluate not only the final evaluation, but on all important aspects combined, an evaluation poster is printed after each run of the model (figure 4.7.4. The poster takes the designer in 5 steps through all relevant aspects of 6 closure strategies, including all aforementioned output generated:

1. Model input parameters and cross section: In the top left corner of the poster, all model parameters and toggles are displayed, including a figure of the cross section modeled and how it approximates the actual bathymetry.
2. Strategic closure options tested: In the top right corner, the six preprogrammed closure strategies are plotted with inscriptions and colors to visualize the sequences chosen.
3. Quantitative analyses on cost factors: In this step, the three quantitative data of each cost factors is displayed. This can be either a volume or a quantity.
4. Final evaluation: This step provides a figure of the summed cost of each cost factor. The quantitative data from step 3 is translated to costs.
5. Extra information on secondary factors: In this step, secondary information is plotted such as the maximum flow velocity during the closure with corresponding stone size, the quarry demand and yield curves and finally the closing time.

For a designer, the final evaluation is most important. However, if any results seem odd, the initial settings, the quantitative data and the secondary factors is there to support the evaluation. An important aspect of the poster is that the designer can calibrate the cost functions or elementary costs to its own preference by comparing the quantitative data with the final evaluation.

1 Model input parameters and cross section



Toggles	Value	Dim
Include Throughflow	Off	-
Use of gabions	On	-
Use of caissons	Off	-
TVC | Threshold value caissons	6	m/s

Basin dimensions	Value	Dim
A_{HAT} | Wet Area at HAT	1700	km^2
A_{LAT} | Wet Area at LAT	400	km^2
T_R | Tidal Range HAT	10.4	m

Dam dimensions	Value	Dim
H_d | Design Dam height	13	CD + m
$B_{c\text{-}H}$ | Crest width horizontal closure	15.5	m
$B_{c\text{-}h}$ | Crest width vertical closure	5	m
α_d | Slope angle dam	27	°

Rock parameters	Mean	STD	Dim
ψ_c | Shields number rock	0.035	10%	-
C_r | Chezy coefficient rock	35	5%	$m^{0.5}/s$
ρ_r | Density rock	2600	1%	kg/m^3

Bottom protection parameters	Mean	STD	Dim
d_{n50} | Grain size sand	0.0015	3	m
ψ_c | Shields number sand	0.035	2%	-
ρ_s | Density sand	2600	1%	kg/m^3
β_f | Failure slope after failure	4	10%	°
ϕ_f | Natural failure slope	45	10%	°
α | Turbulence factor	1.1	2%	-

Equipment parameters	Value	Dim
CT | Capacity truck	15	tons
VT | Velocity truck	40	km/h
LTT | Loading time truck	300	s
UTT | Unloading time truck	180	s
WT | Working time per day	22	hours
CDS | Capacity dumping ship	3000	tons

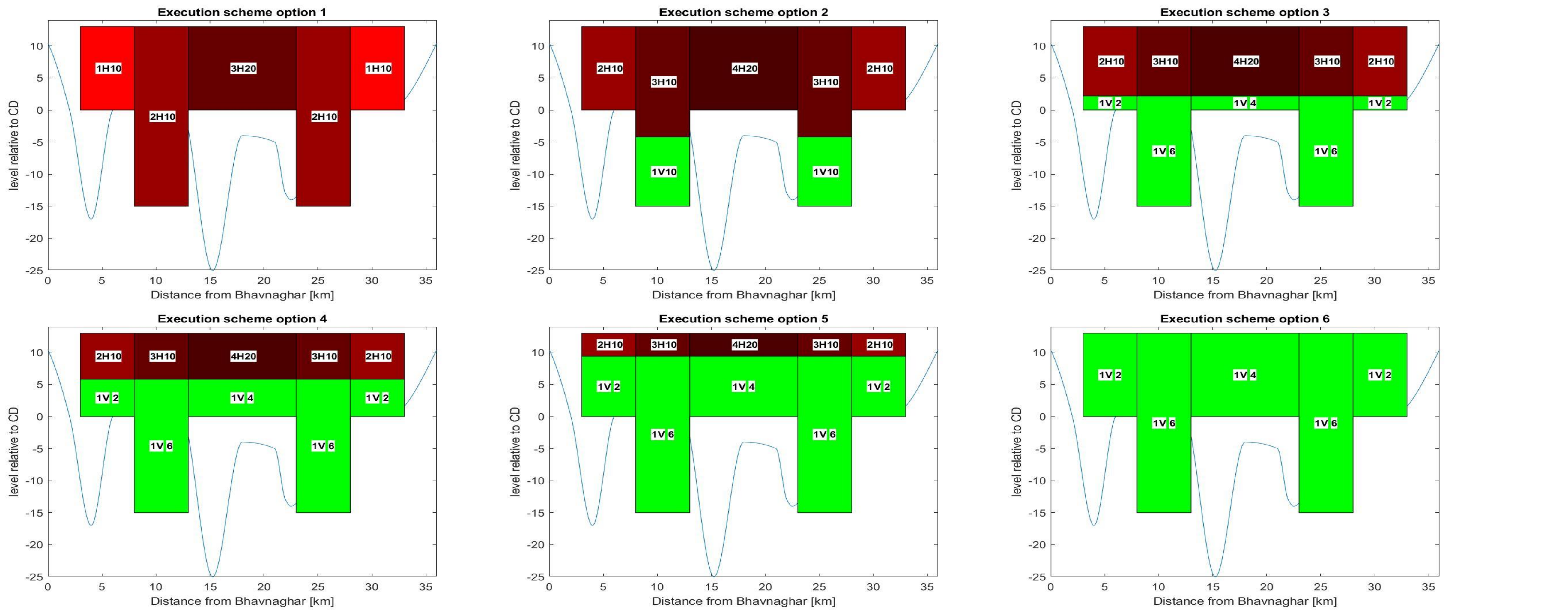
Probabilistic parameters	Value	Dim
nbs_R | Monte-Carlo Sim rocks	1000	-
nbs_B | Monte-Carlo Sim bottom protection	10000	-
$P_{f\text{-}R}$ | Failure probability rocks	0.01	per ex
$P_{f\text{-}B}$ | Failure probability bottom protection	0.001	per ex

Throughflow parameters	Value	Dim
p_r | Porosity rock	0.4	-
D_{n50} | Grain size Rock	1	m
C_t | Throughflow coefficient	0.5	-
μ | Contraction throughflow	0.5	-

Model parameters	Value	Dim
dt | Time step	300	s

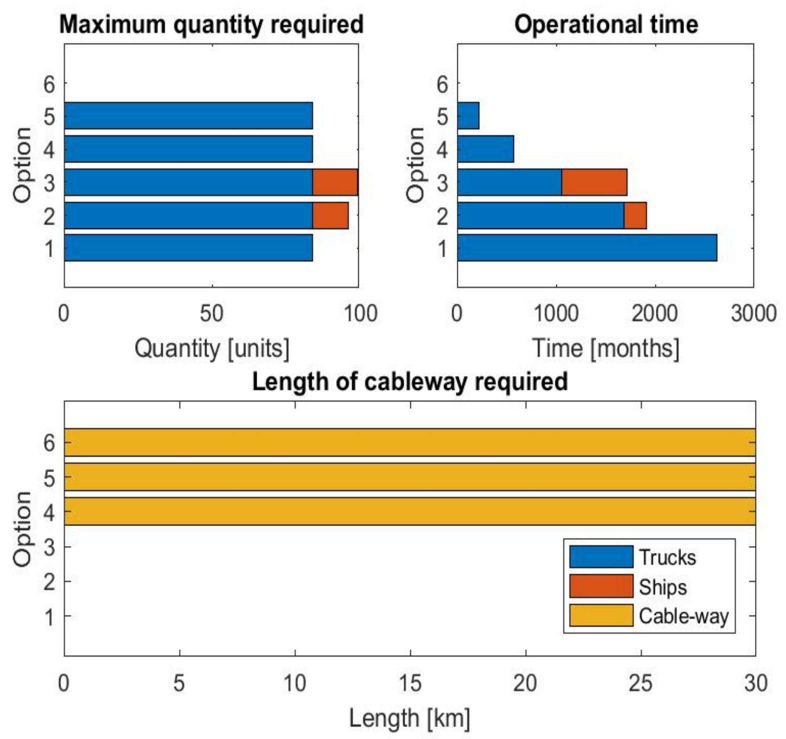
Evaluation cost parameters	Value	Dim
Cm1 | Cost Rock	50	€/m^3
Cm2 | Cost Overproduction	20	€/m^3
Cm3 | Cost Gabions	150	€/m^3
Cm4 | Cost Caissons	400	€/m^3
CoT | Initial Cost Truck	20.000	€/unit
CoS | Initial Cost Ship	1000.000	€/unit
CoTR | Cost of Truck Rent	2.000	€/day
CoSR | Cost of Ship Rent	50.000	€/day
CCW | Initial cost Cable-way	50000000	€
CCW | Cost Cable-way	50000	€/m
Cb | Cost Bottom Protection	70	€/m^3

2 Strategic closure options tested

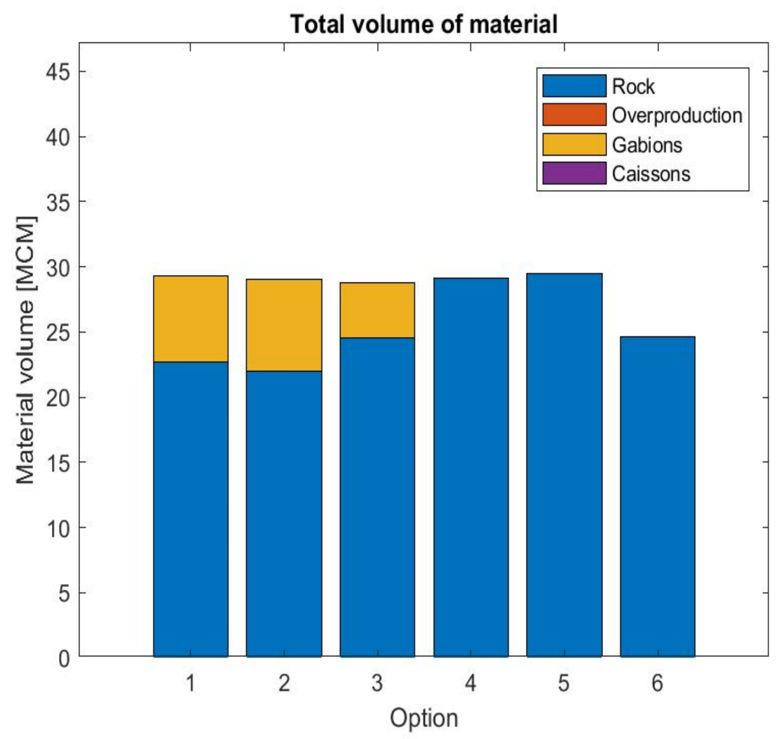


3 Quantitative analyses on primary cost factors

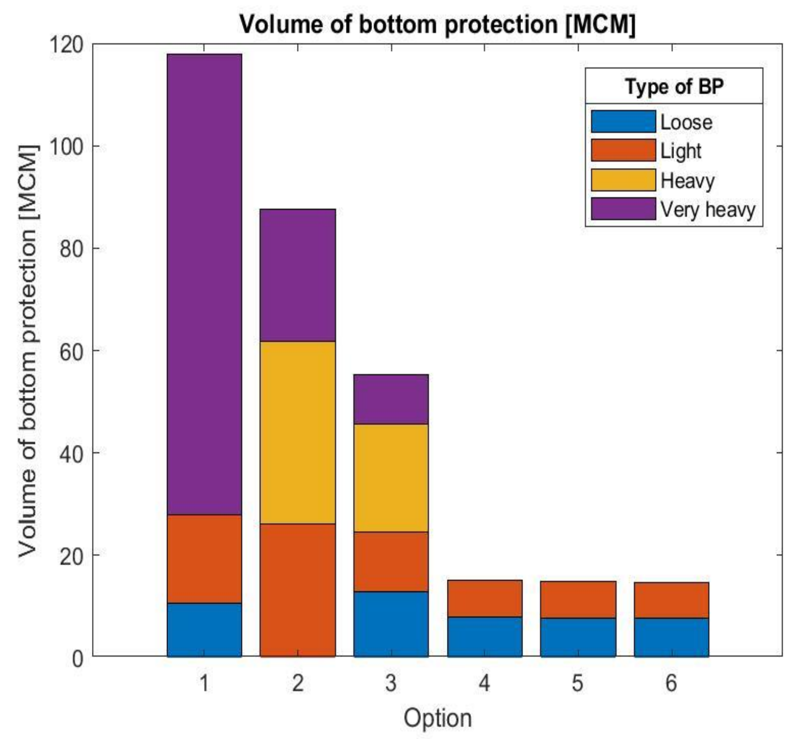
A Equipment required



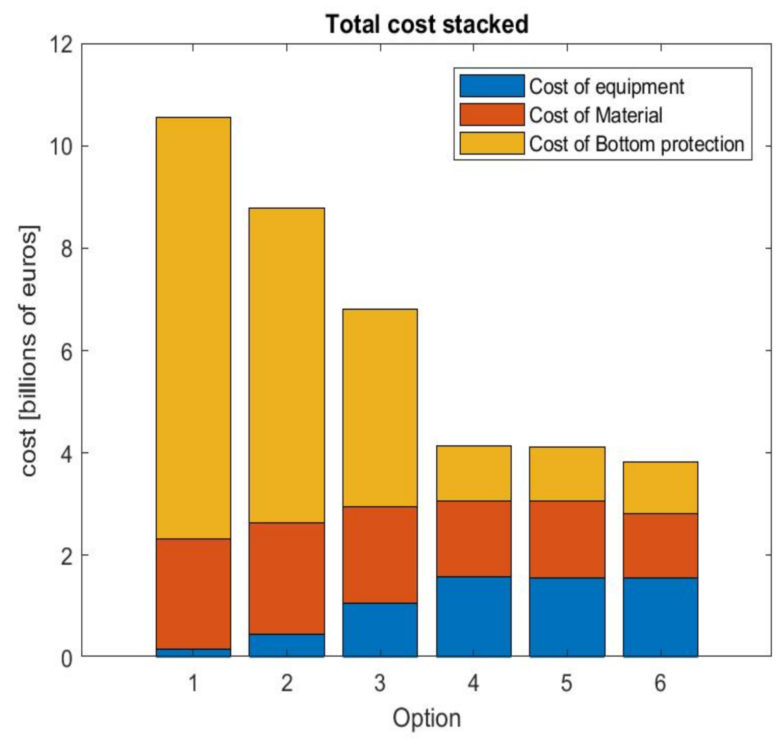
B Materials required



C Bottom protection required

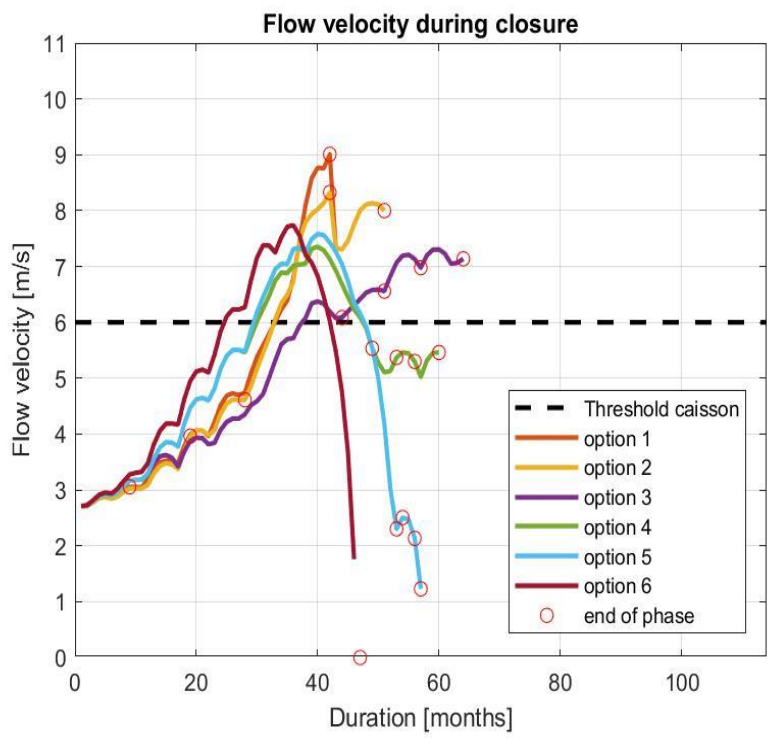


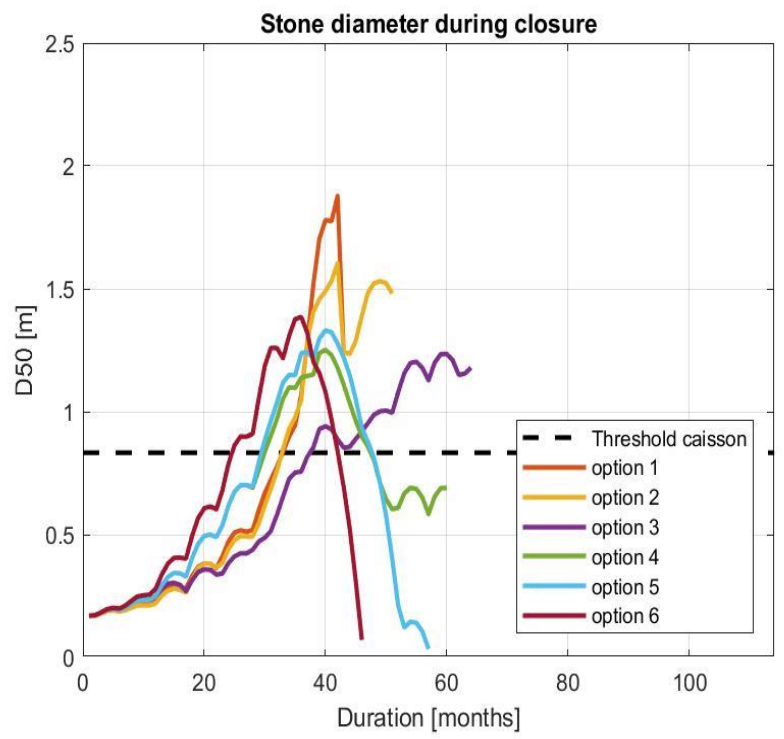
4 Final Evaluation



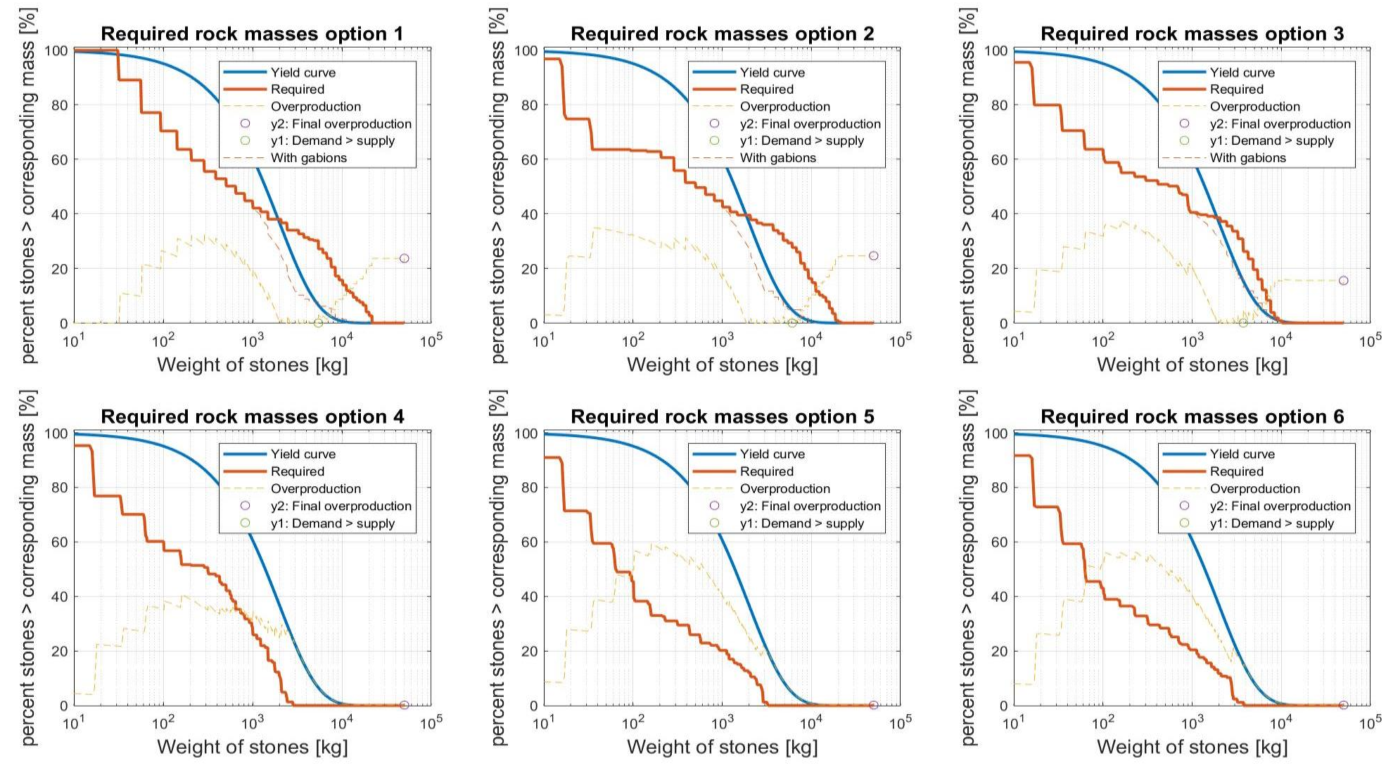
5 Extra information on secondary factors

Flow velocity and rock sizes during execution

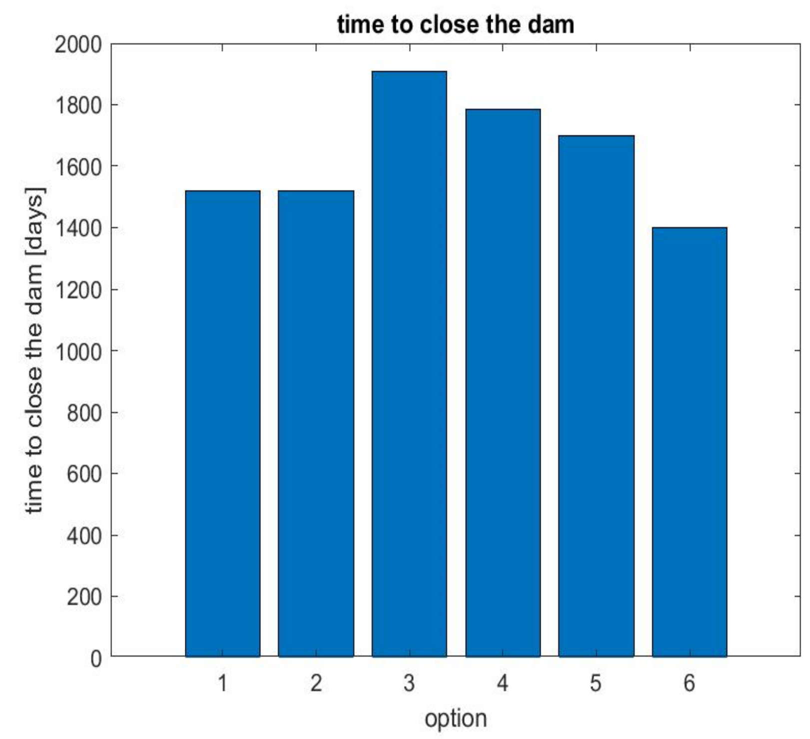




Required volume of rocks sizes & quarry yield curve



Closing time



4.7.5. Discussion

The reliability of the evaluation is the the most significant point of discussion. What determines the differences? and are all computations correct? At this stage in the complete process, it is complex to track through the model which processes influence the evaluation the most. If the result of all computations are correct, variations in the list of elementary cost still have an enormous impact on the end result (since they are at the end of the calculation). They determine the lowest cost option almost independently of the computational results. These should therefore be researched well before using the model. Other computations are less relevant, since difference in output are relatively small. To quantify the influence of all parameters and elementary costs, case studies are performed. The results from this study could indicate the relevance of each parameter. With these results, values can be determined with more care which increase the reliability of the evaluation.

4.8. Conclusion

In this section, concluding remarks about the optimization model are stated:

The optimization model is able to present a first-order evaluation on costs and partially on risk related costs of six different closure strategies. Risk related costs are the increased requirements of material and bed protection if the accepted probability of failure is increased. These are incorporated into the total design costs. The model performs this evaluation with use of four models, each developed to determine the requirements with respect to the three cost factors to achieve the chosen strategies. The first model is the flow model and computes the flow velocities at the location of the dam during the closure. These are then used in the material model to probabilistically design the required rock sizes which determine if an overproduction of material is required from the quarry, or if gabions offer cheaper solution. If flow velocities become significantly high, a sudden caisson closure is automatically inserted where needed. The goal in the optimization of this model is to minimize the use of caisson as much as possible. The strategies are evaluated with and without the optional caisson to create more insight in the impact of a caisson closure and help minimize the total required volume.

The third model computes the bed protection required an uses a level III probabilistic method to compute the required length. In this way, the reliability of the results increases by linking it to an accepted failure probability. The required area is computed using the execution scheme from the closure strategy and is therefore large influenced by this. Time also plays a significant role in both the material and bed protection model. Both model included this parameter and rock and bed protection sizes largely depend on the duration of the execution since the design condition is more extreme if execution takes longer. This aspect of the model is interesting, because this creates a possibility for optimization with use of the construction capacity and defines a interesting consideration for contractors.

The last model computes the basic required equipment to execute the strategy based on known equipment and their capacities. In this evaluation, the raw output of required material volumes, bed protection and equipment is translated to cost and summarized. With this information, The least cost option can be optimized by changing the strategy related input. That is, the closure method, phase and capacity of each section.

5

Flow model validation

The MATLAB optimization model is based on several hydraulic computations. At the basis of these computations is the numerical flow model described in section 4.3. All subsequent computations largely depend on the resulting flow velocities from this model. This is why the validation of the flow model is a priority.

In this chapter, the flow velocity and discharge from the flow model is compared to other models to attempt validation. In section 5.1, an introduction to the problem is explained. In section 5.2, the assumptions and general rules for application of the storage model are shortly summarized. In section 5.3, the flow velocities from the model are compared to an existing storage model created for the Gulf of Khambhat. In section 5.4, a comparison is made with a 2D hydrodynamic model in Delft3D. The difference in velocity in both models was significantly large that calibration was required. This calibration process is explained in section 5.5 . General conclusions about the use of the storage model are specified in section 5.6

5.1. Introduction flow model

In the numerical flow model, the flow velocities occurring on the dam alignment are computed in time during the closure sequence with use of a storage model. This model is based on some assumptions and is generally used for an initial calculation of the flow velocities during a closure. However, the use of the storage model is extended in this optimization method by using several cross sections to generate a more detailed overview of flow velocities in the complicated structure of channels and tidal flats in the Gulf of Khambhat. However, the implementation of the storage model using this approach might not be valid because the boundaries for the use are exceeded. The main question in this chapter is therefore:

"Can a simplified storage model based on eight cross sections correctly describe the flow velocities during the closure of the Gulf of Khambhat?"

5.2. General rules of application storage model

The storage model is based on two general theories:

- The small basin approximation
- Description of flow over weirs (appendix C)

Both theories combined form the storage model that can be solved in time with a numerical model. Detailed information about both theories can be found in appendix C.General rules of application of both theories are described in the following subsections.

5.2.1. The small basin approximation

As described in section 2.2.2, the small basin approximation is used to model the flow in small basins where the discharge in and out the basin can be described by relation 5.1

$$Q_{in} = A_b \frac{dh_b}{dt} \tag{5.1}$$

The relation completely removes all spatial variations (momentum equation) and is therefore considered a 0D model. The relation can be used under assumption that resistance and inertia can be disregarded due to

- The minimal spatial variation of the water level across the basin. This can only be true if the relative size of the basin is small compared to the tidal wave length. The boundary condition for a small basin is that the length of the channel is only 5% of the tidal wave length.
- A small opening connecting the basin and the sea, thereby minimizing velocities inside the basin.

In case of the Gulf of Khambhat, initially both assumptions are incorrect.

Both resistance and inertia can't be disregarded because the basin is very large. In the case of the Gulf of Khambhat, the tidal wave length for the largest component (M2 tide) can be computed using definition 5.2 for shallow water.

$$L+w = T_{M2} \cdot C_w = T_{M2} \cdot \sqrt{g \cdot D_{channel}} = 540km \tag{5.2}$$

In which:

C_w	=	Wave Celerity			$[m/s]$
$D_{channel}$	=	Depth of channel	=	15	$[m]$
T_{M2}	=	Tidal wave period M2 component	=	44700	$[s]$
L_w	=	Wave length			$[km]$

The length of the longest channel behind the dam is around 70-80 km, concluding that the relative size of the basin is large with a channel/wave length ratio of 12-14 %. This exceeds the maximum 5 % which is allowed for the use of this theory. Furthermore the opening in the basin is initially not small (approximately 36 km). This mean that the water level in the basin is not equal everywhere and therefore the basin level is computed incorrectly.

Both assumptions are initially wrong, however, during the closure the size of the opening reduces which rectifies both assumptions. The basin water level will vary less across the area of the basin (in both horizontal and vertical closures) which also reduces the flow velocities inside.

5.2.2. Flow over weirs

In section 2.2.2 the general formula's for flow over weirs have been discussed. These are implemented in the model as stated in this section. The most important assumption these formula's have in common are:

- The flow velocity is computed using the head difference therefore the assumed basin water level computed using the small basin approximation.
- The flow velocity is computed using an assumed wet cross section above the weir. This assumed wet cross section is based on the smallest depth on the weir in case of sub-critical flow and on 2/3 of the upstream water level during supercritical flow.

The first assumption is directly related to the assumption of the small basin approximation and is therefore the same.

The second assumption about the flow depth on the weir is based upon small weir experiments. However, the flow depth on the weir can vary due to the larger size of this dam (scale problem). Secondly, it can also be true that the shape of the weir during closure varies from the theoretical weir shape and thus the flow depth on the weir is different.

5.2.3. Conclusion and follow up

To summarize, the most significant error in the model for the Gulf of Khambhat is the small basin approximation. In section 5.4 a 2D Hydrodynamic Delft3D model is used to evaluate the order of the error and thereby the result of neglecting resistance and inertia. In the next section, the model is first compared to other storage models to validate the correctness of the initial model setup.

5.3. Comparison with other storage model

During the preliminary design phase of the Kalpasar closure dam in 1998, the storage model was used to evaluate the flow velocities occurring in de closure gap. Both Wiersema and Broos and Royal Haskoning used these models as boundary conditions for their closure strategy designs. In this subsection, the storage model used in this study is compared to the storage model used by Wiersema and Broos. Comparison to results from Royal Haskoning is not necessary, since the used the same model and had the same results as the study by Broos and Wiersema.

5.3.1. Setup

To compare the results from both models, first the data from Broos and Wiersema is analyzed. They used only one cross section representing the last 10.000 m of the closure. They tested only fully vertical and fully horizontal closure strategies. For this study, these two cases should be enough to compare both models. Model parameters used by Wiersema and Broos are displayed in table 5.1

Table 5.1: Input parameters storage model Broos and Wiersema

Model parameters	**Value**	**Unit**
Time step	300	[s]
Gap width	10.000	[m]
Sill level	-10	[m + CD]
Storage area at HAT	2187	[km^2]
Storage area at LAT	630	[km^2]
Maximum tidal range	10.4	[m]
μ - contraction coefficient subcritical flow	1	[-]
m - contraction coefficient supercritical flow	1	[-]

The outside boundary condition is defined by the dominant four tidal constituents displayed in table 5.2

Table 5.2: Tidal constituents used by Wiersema and Broos

Tidal constituent	**Amplitude [m]**	**Phase [°]**
M2	3.14	143
S2	0.96	190
K1	0.76	92
O1	0.34	95

5.3.2. Results

In figure 5.1, the results of the two cases are displayed. On the x-axis the constriction is set out in meters. In the horizontal closure case, the closure gap is reduces from 10.000 to 0 m width. In the vertical case, the sill level is lifted from CD - 10 m to CD + 10 m. On the y-axis, the velocity is displayed.

Figure 5.1 shows that the two models have the same order of results. The difference in the horizontal case is mainly the steepness in the curve, but the begin and end velocities are the same. The model used in this study starts around 5 m/s and only increases around 7000 m to 6 m/s all the way up to 9.3 at 0 m width.

The vertical closure case resembles the line from Broos and Wiersema perfectly at the start of the closure. However at the end it overestimates the velocity with 1 up to 2 m/s. This can be explained due to the difference in computation. Although the flow velocity is computed with the use of the storage model, both models work differently. This difference can explain the differences in both the horizontal and vertical case.

The model used in this study simulates the full closure sequence on the basis of a given capacity. This capacity defines the speed with which the closure is executed. Therefore, the situation changes over time until closed fully. The capacity does not change in this period, but the tide does change. This is because the constituents cause yearly fluctuations, which differs about 1 m. The yearly increase in tidal difference could be at the

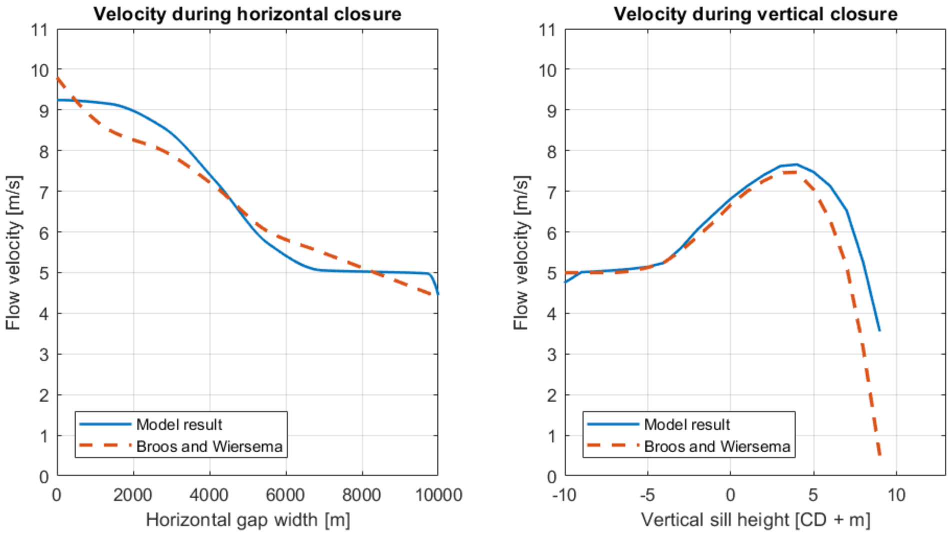


Figure 5.1: Comparison flow velocity from storage model with results Broos and Wiersema during horizontal and vertical closure under the same conditions

same time as the last stage in the vertical closure. Therefore this could be a reason for the higher velocities generated by the model during this stage.

The model used by Wiersema and Broos computed the maximum velocity for several closure gap sizes during one run of one month (so without changing the closure gap size during the run). It could be that they this difference shows a more linear increase in flow velocity with Broos and Wiersema and a more gradual exponential increase with the model from this study.

Based on the model setup of this study, the graph of the model from this study is created by extracting the velocities during the occurrence of a certain closure gap size during the closure sequence, where the closure gap size is defined at High Water (HW). From this array of velocities occurring at probably different time steps, the maximum velocity is extracted. The data points from Broos and Wiersema are generated the other way around (simpler method); The velocities are computed at a certain closure gap size and then plotted.

5.3.3. Conclusions

From the previous section, it became clear that the velocities computed by both models match quite well for an initial evaluation of possible closure strategies. The differences are minimal and can be explained by differences in the model setup of both models. The model used in this study is probably more accurate, since the full closure is simulated in time instead of several wet cross sections in time.

5.4. Comparison with 2D-H Delft3D Model

In section 5.2, it was mentioned that the storage model used in this study is based on several assumptions that might not be valid for the Gulf of Khambhat. With use of a 2D Hydrodynamic Delft3d model, these assumptions and resulting errors can be evaluated.

Deltares has provided the Kalpasar team with an old Delft3D model from 2012 which is validated for tidal levels in the basin according to the paper: “Tidal modeling in the Gulf of Khambhat based on a numerical and analytical approach” written by Giardino et al. A more extensive elaboration of the model can be found in the thesis report ”Hydraulic and morphological impact of a closure dam in the Gulf of Khambhat” by René Kersten, 2018. In his thesis, he describes the usability of this model to predict water levels and flow velocities in the Gulf of Khambhat with and without a closure dam.

5.4.1. Approach

The 2D hydrodynamic model can be used to evaluate the assumptions made by the storage model, so at first the water level in the basin is checked. This should be more or less equal from the location of the dam to the end of the basin. Secondly discharges and flow velocities through individual cross sections are compared for the initial situation without a dam and for situations with a partial dam in place.

To compare both models, boundary conditions and bathymetry should be the same. The same holds for the wet surface area at LAT and HAT in the basin. Since the storage model is parametrically designed, the adjustment will be done is this model to match the validated Delft3D model.

5.4.2. Adjustments storage model

The following adjustments are required to compare both models:

- Bathymetry along the dam alignment
- Outside boundary condition (sea level)
- Wet area of the basin at LAT and HAT

Bathymetry along dam alignment The bathymetry along the dam alignment shows large variations with the earlier defined bathymetry from the admiralty charts from appendix sectionG.1.2. However, for this validation, it is assumed the Delft3D model is correct, and the storage model will be adjusted accordingly. In figure 5.2, the initial bathymetry on the north side of the basin is displayed. The outer boundary conditions are placed far more south, but for this study the interesting part is at the location of the dam. In figure 5.3 , the bathymetry on the dam location is extracted from Delft3D and referenced to Chart Datum. In figure 5.4, the approximated bathymetry is shown. The sill level of each cross section is based on averaged depth on this line in Delft3D.

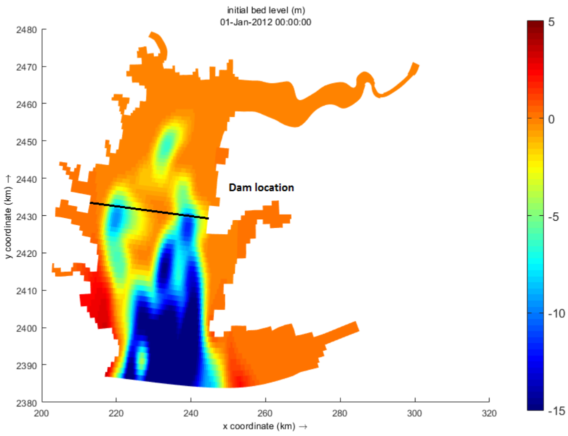


Figure 5.2: Inital bathymetry Delft3D model with dam location

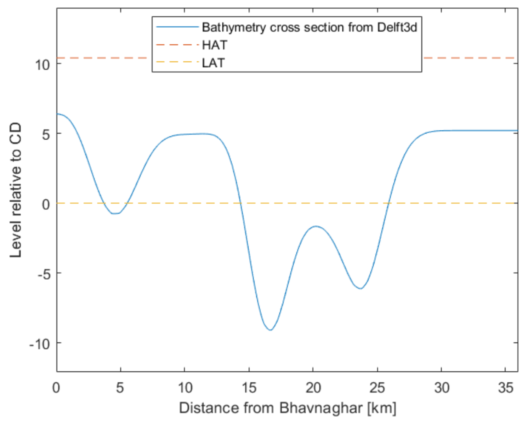


Figure 5.3: Bathymetry at dam location

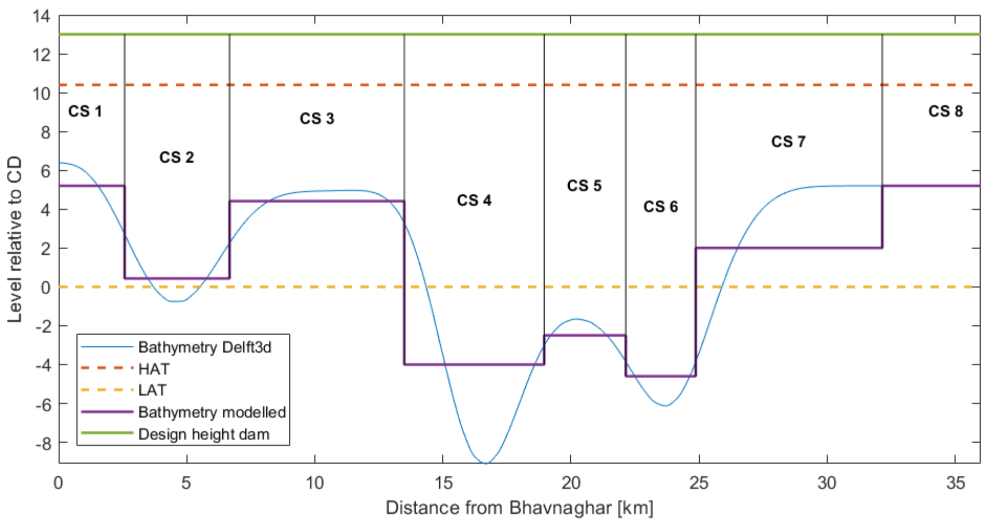


Figure 5.4: Bathymetry modelled with storage model

Outside boundary condition The outside boundary condition or better known as the fluctuating water level at the sea side of the basin, is different in both models. In Delft3D, the boundary conditions are positioned at the edge of the shelf, which is around 350 km south of the dam. The storage model uses the the water level just south of the dam as boundary condition.

In order to compare the models, the outside boundary condition in the storage model can be adjusted by using the water level at several positions (location of cross sections) along the dam alignment . For this, the average water level on the location of a cross section at every time step is extracted from Delft3D. The resulting boundary conditions for a runtime of one month are shown in figure 5.6. The old boundary condition used by Broos and Wiersema is shown in figure 5.5. The water level extracted from Delft3D resembles the shape of the original water level. However, it can be seen that the water levels on the tidal flats (cross sections 1,2,3 and 8) reach levels below the bed level. Furthermore, the maximum amplitude of 10.4 meters is smaller and is now 9 meters. This difference can be explained by the yearly differences in tidal water levels. In this validation, only one month of tidal levels is extracted from Delft3D. With more model time, the maximum amplitude in Delft3D will eventually equal the theoretical amplitude.

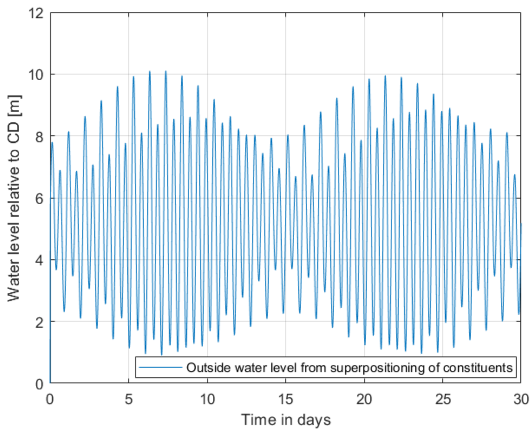


Figure 5.5: Water level resulting from superposition of theoretical constituents found at Bhavnaghar

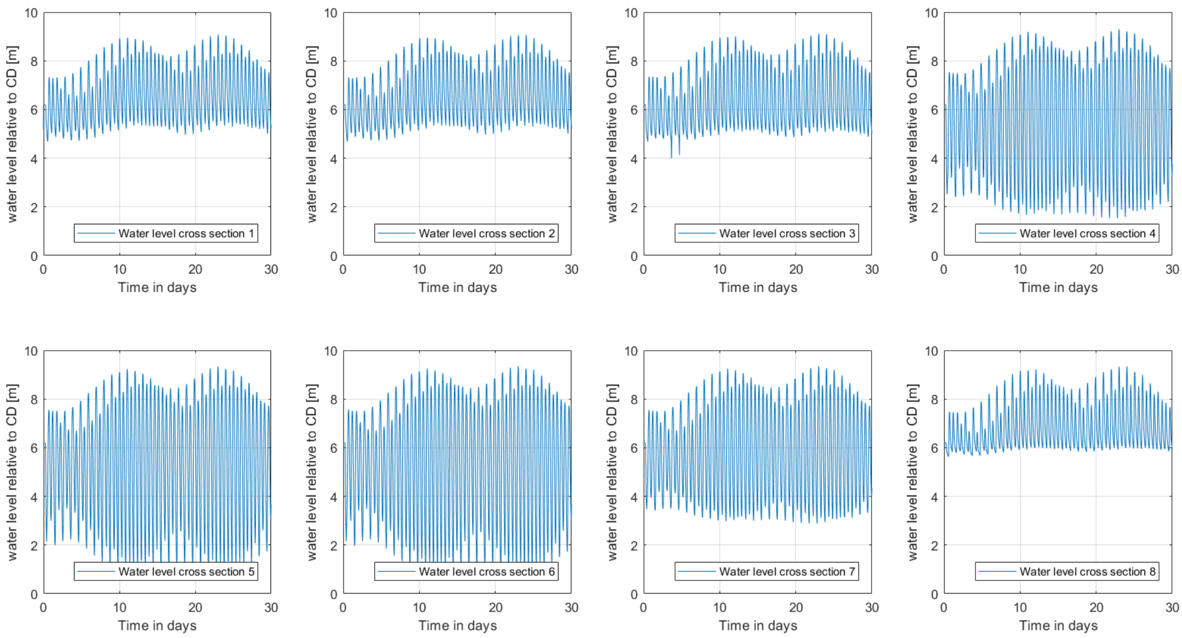


Figure 5.6: Water levels at every cross section extracted from Delft3D

Wet surface area of the basin The wet surface area at HAT and at LAT are extracted from Delft3D and are defined at 2141 km^2 at HAT and 181 km^2 at LAT. This show variation with the earlier defined 1700 and 400 $km^2$2 that were defined with use of Google earth engine (section G.1.4). However, since linear interpolation is used, to defined the storage area at every water level, the shortcoming of wet area at HAT and the overestimated wet area at LAT, compensate for the total storage capacity. In this validation process, the linear relation in the storage model is updated with the results from Delft3D.

5.4.3. Comparison basin water level

The storage model is adjusted to the conditions in the Delft3D model and can be used to compare the results of both models. First, the water level in the basin behind the dam is checked to confirm the assumption of an equal water level everywhere in the basin is correct. To confirm this, the water level in the basin plotted alongside a longitudinal line from the end of the basin to the outside boundary of the model (figures 5.7 and 5.8).

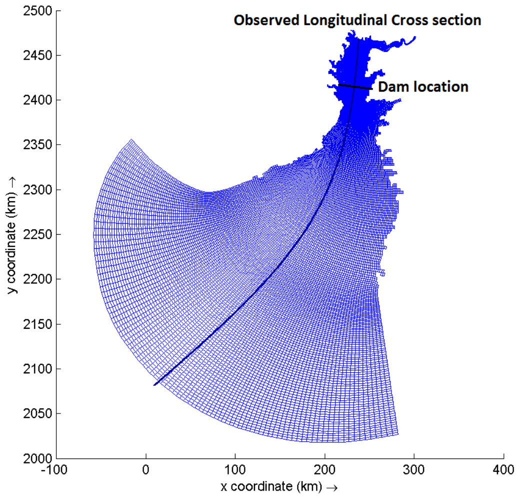


Figure 5.7: Location of longitudinal cross section

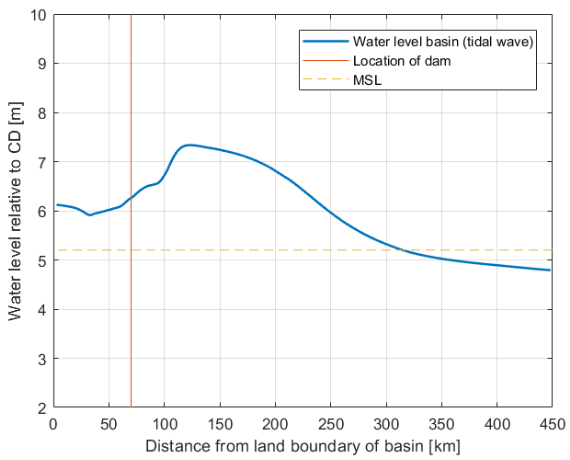


Figure 5.8: Water level (tidal wave) at longitudinal cross section

For a better view of what happens inside the basin, the last 90 km are observed more closely. The results are plotted for a high water and low water at the dam location with respective figures 5.9 and 5.10.

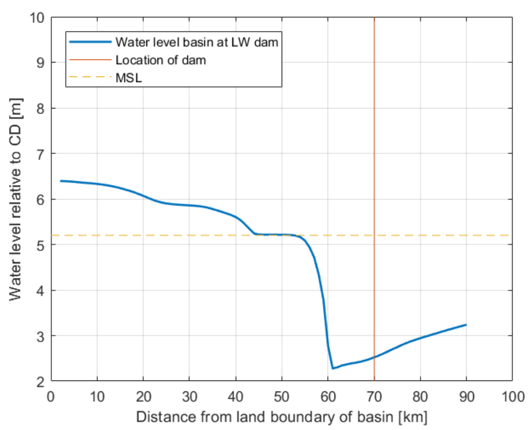


Figure 5.9: Water level in the basin (high water at dam location)

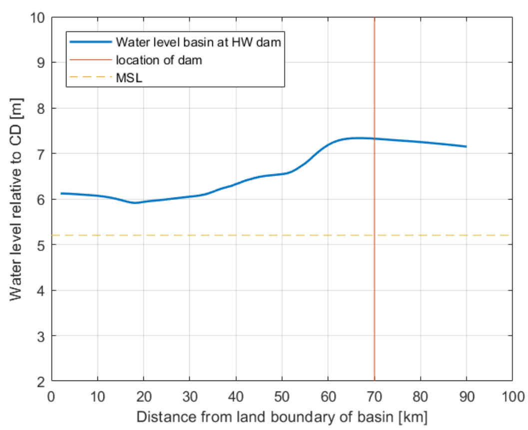


Figure 5.10: Water level in the basin (low water at dam location)

From figures 5.9 and 5.10, it can be observed that the water level between the end of the basin and the dam location is not equal. At high water, the difference is 1-1.5 meters . At low water, the influence of the bed level is clearly visible, explaining the out of the ordinary shape of the water level. This can be explained by friction generated in the shallow inner basin combined with the relative small length of the wave compared to the distance remaining to the end of the basin. More time steps have been analyzed to conclude that the tidal wave shoals due to decreased depth and loses its celerity. This process causes even larger differences in water level across the inner basin than depicted in figures 5.9 and 5.10 .

It can be concluded that the assumption of an equal water level across the inner basin is false. The error of this assumption can be quantified by analyzing the discharge and flow velocities through the cross sections at the dam location.

5.4.4. Comparison discharge

To compare the two models, the total discharge through the total cross section is compared. The total discharge each tidal cycle is equal to the tidal prism of the basin behind the, which is equal to the storage capacity of the basin. By comparing the total discharge in both models, it can be analyzed if the assumed storage capacity of the basin is correct. Furthermore, any difference in the amplitude of the discharge results in the incorrect determination of the flow velocities. In figure 5.11 the discharge for both models is plotted for one month.

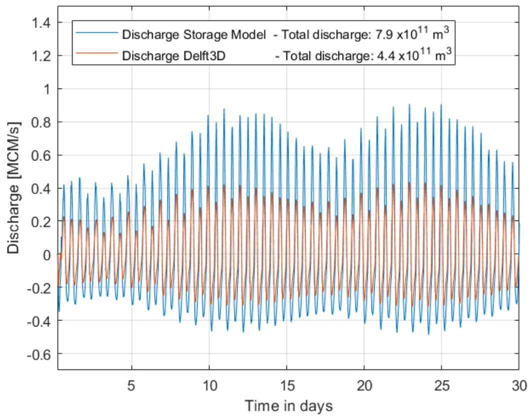


Figure 5.11: One month of discharge through cross section at dam location

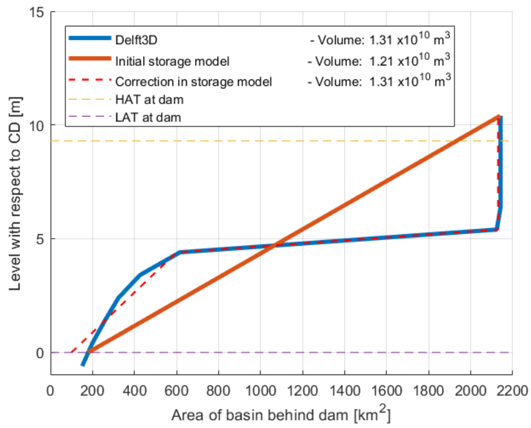


Figure 5.12: Relation basin area with respect to water level

From figure 5.13 it can be concluded that the discharge going through the cross section is larger in the storage model than in the Delft3D model. The total discharge in one month in Delft3D is $4.4 \cdot 10^{11} m^3$ and in the storage model this is almost 2 times more ($7.9 \cdot 10^{11} m^3$). The difference can be explained by the size of the assumed storage area. As was explained in section 5.4.2, the assumption of a linear relation of storage area with respect to water level might be incorrect. To investigate this, the wet area at several bed levels has been plotted in figure 5.12.

It can be concluded that the linear assumption is incorrect. Above MSL, the wet area stays the same (around 2142 km^2). Just below MSL, the wet area decreases quickly in size to around 600 km^2 after which is steadily decrease exponentially to around 180 km^2 at LAT. The effect on the total discharge is however still unexplained, since the total storage volume in the Delft3D model is larger (instead of smaller) than the assumed volume in the storage model. To quantify the influence of the non-linearity in basin size, the storage model is adjusted. To correct the linear approximation, a 3-step linear relation is established. The relation follows the line defined by equation 5.3. The resulting basin volume is the same as in the Delft3D model.

$$A_{wet}(h) = \begin{cases} \left(180 + \frac{600-180}{4.4} \cdot h\right) \cdot 10^6 & \text{for } 0 < h \leq 4.4m \\ \left(600 + \frac{2140-600}{1} \cdot (h-4.4)\right) \cdot 10^6 & \text{for } 4.4 < h \leq 5.4m \\ 2140 \cdot 10^6 & \text{for } h > 5.4m \end{cases} \tag{5.3}$$

In which:

h	=	Water level with respect to CD	[m]
A_{wet}	=	Wet basin area	[m^2]

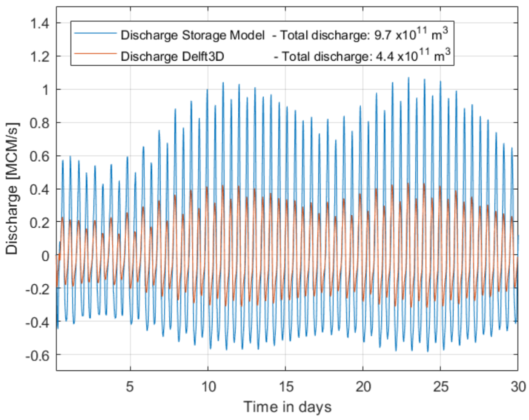


Figure 5.13: Discharge after correction storage relation

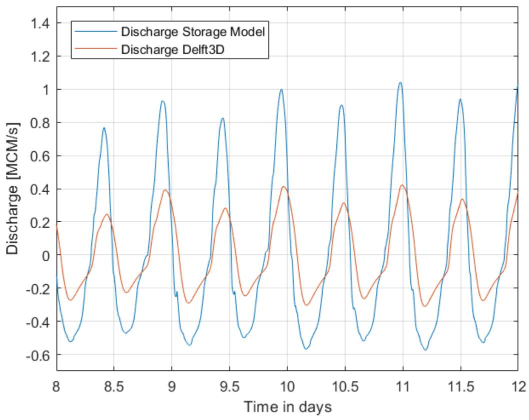


Figure 5.14: Discharge four days

The resulting discharge, after implementation of the new basin area relation is shown in figures 5.13(for one month) and 5.14 (for four days). The ingoing discharge (into the basin) has significantly larger peaks than the outgoing discharge in both models. Since the average discharge is zero in both models these short steep peaks are compensated by a long but slow outflow through. This can be explained by the asymmetric shape of the incoming tide. it rises fast and decreases slowly, This is caused by depth decrease in the basin and therefore the larger wave speed at peak of the tidal wave (due to larger depth) moves faster than the slower trough of the wave (shallower).

Furthermore, it can be concluded that the total discharge has increased even more, as was expected due to the increase of total storage area. However, the non-linearity of the wet basin area should have an influence on the shape of the discharge, but in this case this difference is not significant and therefore not displayed in this report.

The difference in total discharge should therefore be explained by the difference in actual storage volume. It is already established that the water level is not equal in all locations in the basin. Furthermore, a large part of basin lies above or just below MSL, therefore the wave length through the basin decreases significantly. To investigate the actual storage in the basin (and therefore the explanation for the difference in discharge), the water level is plotted in the longest channel of the basin (figure 5.15). Figure 5.16 shows the water level at HAT and LAT at the dam and the initial bed level.

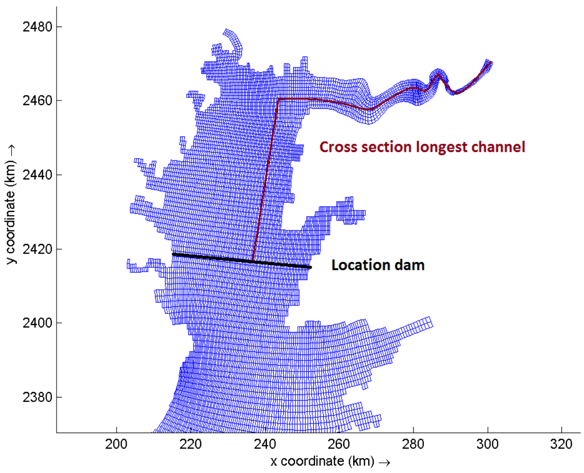


Figure 5.15: Cross section longest channel in basin

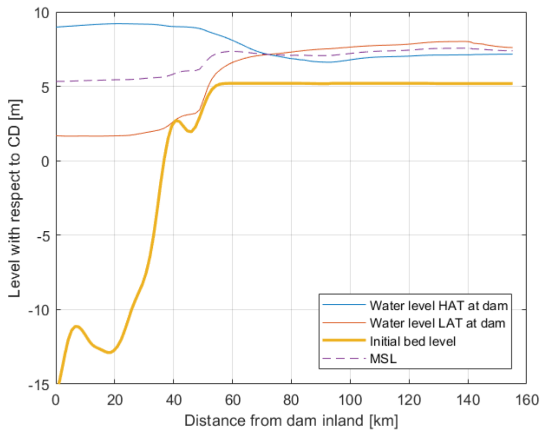


Figure 5.16: Water and bed level at HAT and LAT in the cross section

From figure 5.16, it can be concluded that the actual storage area is significantly smaller than assumed. At 65 km, the water levels are the same and cross over. The storage volume after this point is negative, since the water level at high water is lower than at low water. Furthermore, the basin doesn't have any dry points at he lowest water level. This caused by the friction of the bed which reduces discharges out of the shallow part into the deeper part. To conclude, from this image, it can be roughly estimated that the storage volume would be in the order of 40-50 % of the actual basin volume. This is correct since the total discharge in the Delft3D model is about 45 % of the storage model, which uses the whole basin area as storage.

By adjusting the wet area relation 5.3, it can be checked if this is indeed true. The relation is therefore changed to equation 5.4

$$A_{wet}(h) = \begin{cases} \left(180 + \frac{670-180}{4.4} \cdot h\right) \cdot 10^6 & \text{for } 0 < h \leq 4.4m \\ 670 \cdot 10^6 & \text{for } h > 4.4m \end{cases} \tag{5.4}$$

This relation reduces the size of the basin by 55%. Relation 5.4 is plotted in figure 5.17. The resulting discharge is displayed in figure 5.18.

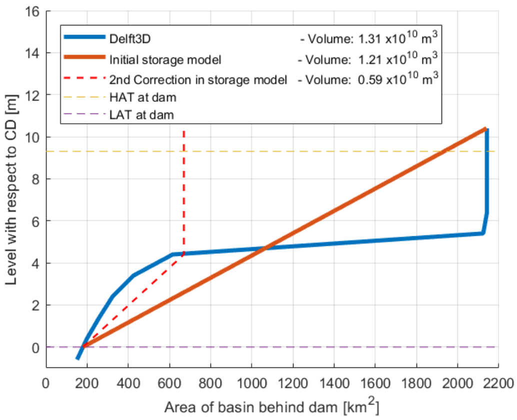


Figure 5.17: Relation basin area with respect to water level with 2nd correction

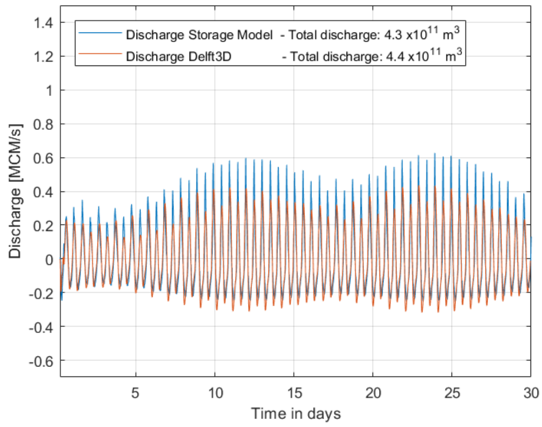


Figure 5.18: Discharge after 2nd correction storage relation

From figure 5.18 it can be concluded that although the total volume of water exchanged by the basin and the sea is the same, the storage model stills shows larger peaks during inflow and underestimates the discharge flowing out.

Furthermore, within one day, two high tides occur which differ in highest water level. it can be seen that the difference in discharge in Delft3D for these two peaks is significant. However, with the adjusted basin area, this difference has completely disappeared. It can be concluded that although a reduced basin size can correct the order of the error by a large factor, the shape-wise resemblance with the original discharge curve is disappeared

The larger peak discharge in the storage model can be also be explained by the assumption of equal water level in the basin; At high water at the location of the dam, the basin water level increases slowly since the whole area of the basin has to fill up at the same time. This causes the head difference with respect to outside the basin to stay large for a long period which increases flow velocities and thus the discharge. In case of the Delft3D model, the water level in the basin right next to the dam increases with a larger velocity, since the size of the basin and the generated friction keep the wave from spreading to the end of the basin. The head stays small, thus the velocity and discharge are smaller.

To summarize, the phase lag between the basin level and the sea level is larger in the storage model due to the slow adaptation of the basin level, therefore the head difference is larger and the in- and outflow of water is larger. The slow adaption of the basin level is caused by the large overestimation of the actual storage area. The difference in phase lag from the storage model and the Delft3D model is displayed in figures 5.19 and 5.20. Furthermore, this is still the complete cross section and the model requires an accurate prediction of the flow velocities in each individual cross section. Therefore the flow velocities in each individual cross section are analyzed in the next section.

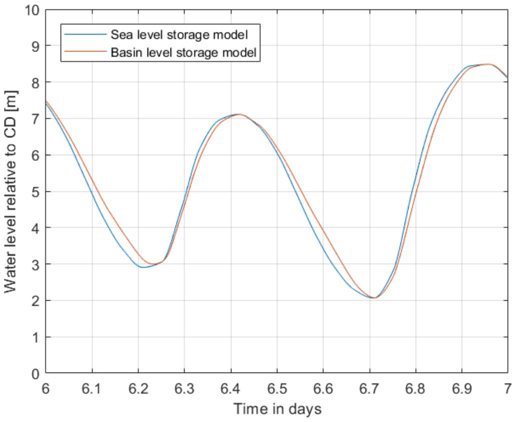


Figure 5.19: Sea and basin levels from storage model (large phase lag)

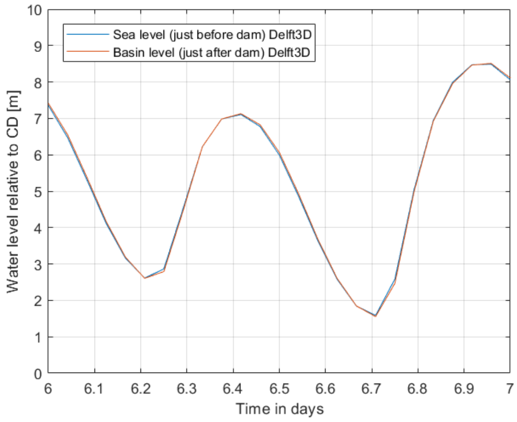


Figure 5.20: Sea and basin levels from Delft3D (small phase lag)

5.4.5. Comparison of flow velocities

The storage model is based on eight different cross section and for each cross section, the flow velocity determines the rocks sizes and quantity of bed protection required. For this reason, it is necessary to analyze the flow velocity in each individual cross section and compare it to the Delft3D model. To perform this comparison, the velocities are extracted from Delft3D at the earlier defined cross sections (see figure 5.4 for the model approximation). The results are plotted in figure 5.21 and divided in eight different graphs, each representing a cross section.

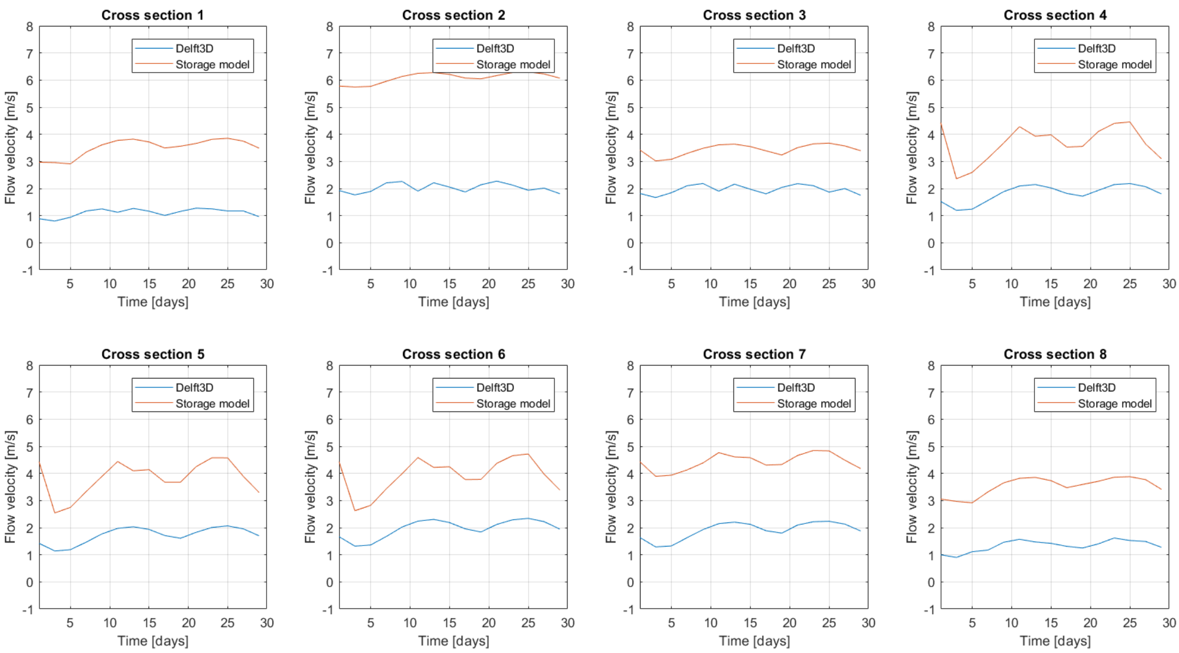


Figure 5.21: Comparison of flow velocities Delft3D and storage model for initial situation without dam or constriction

The large differences in flow velocities between both models was predicted by the large difference in discharge. However, the basin size in this case was already adjusted with relation 5.4. In most deeper cross sections (cross sections 2, 4,5,6 and 7) the difference is a factor 2-3. This is significantly large and the question is now if this model is still usable for the optimization of closure strategies. The difference in initial flow velocities can be explained by the larger steeper peaks in the total discharge, as was seen in section 5.4.4.

5.4.6. Discussion and conclusion

The storage model from this study relies on several assumptions. The most important one is the assumed equal water level in the basin. From previous sections it became clear that the water level in the basin differs and the tidal wave penetrates with decreasing amplitude through the basin.

The result of comparing the discharges is not in favor of the usability of the model, but it is in favor of the design flow velocities. The discharges are overestimated by a factor 2-2.5 and thus the flow velocities do not match as well. The reasons for this error is that this is an analysis of the situation without a dam. No constriction is yet implemented. Literature about the usage of storage models suggest that the usage of the weir formula's to compute the velocity are not accurate if the constriction not large enough, since the velocity is not computed on the basis of the head difference, but with the Chézy formulations (Konter et al., 1997).

Both resistance and inertia can't be neglected in the first stages of the closure, so a storage model overestimates the discharge. The overestimation is caused by the assumption of equal water level in the basin so the complete area of the basin can be used as storage. The opposite is true in the Delft3D model; the water levels fluctuate significantly due to the shallow bathymetry resulting in a reduction of storage area to around 45 % of the physical storage volume available (figure 5.16. This explains the error factor of 2-2.5, since the tidal prism (storage area) is equal to the cumulative discharge through the gap.

Reducing the basin size reduces the order of the error. However, reducing the basin size may be incorrect in further modelling of the closure. The storage volume changes with the increased constriction due to the decrease in amplitude of the tidal wave penetrating the basin and the possible extra storage generated by the closure. However, no direct relations can be defined for this phenomenon, since the storage volume is different is a horizontal and a vertical closure case. Using an adjusted basin size is therefore not an accepted possibility to calibrate the model.

Secondly, the most important parameter in this study is the flow velocity, therefore in section 5.4.5 the error of this assumption was quantified. The error is significantly large with deviations up to 3 times the actual value even with adjusted basin size. Again this deviation is caused by incorrect assumptions (no inertia and resistance) on which the storage model is based.

Since the storage model is easy to use, the goal would be to try and match the flow velocity to the Delft3D model. For this it has to be assumed that the Delft3D model is very accurate and correct in modelling the Gulf. However, this might not be the case. The storage model is widely used to model closures. However, in this situation, it shows large deviations in flow velocities due to the incorrect assumptions it is based on. On the other hand, the flow velocities in the Delft3D model resemble known measurements in unconstricted situation, which are in the order of 2-3 m/s (Royal Haskoning, 1998a). The validated Delft3D model should therefore be more accurate. In more constricted situations, the storage model might predict the flow velocities more accurately. However, the weir equations that are used in the storage model are also used by Delft3D. In that case it can be assumed the Delft3D model models the flow velocities correctly in all situations.

From here it is decided that the validated Delft3D model resembles the actual situation in the basin better than the storage model and will be used to calibrate the storage model

In this way, the model can be used to asses the situation for the Kalpasar case. To solve the above stated problem it has been decided to include a calibration factor for the evaluation of the flow velocity. In the following section this calibration process is discussed

5.5. Calibration

From the previous section, it became clear that there is only one option to solve the error in the flow velocity. In this section the calibration of the flow velocities is performed on the assumption the Delft3D model performs a more accurate analyses. To perform an accurate calibration the approach is discussed in the next section

5.5.1. Approach calibration

The most important aspect of calibration is that the factor of calibration should perform well under all conditions. During a closure the closure gap reduces in size and this is the most important parameter defining the status of the flow conditions. It is proposed to define a calibration factor and link it to the constriction percentage. The main assumption for this relation is that the storage model increases in performance when the constriction becomes smaller. From comparing several trial Delft3D cases with the storage model from this study, it can be stated that this is indeed true. The error reduces when the constriction becomes larger.

5.5.2. Calibration by basin size

From the previous sections, it can be concluded that the basin size is too large and therefore it is a possibility to decrease the size in the storage model to match the discharges from the delft3D model. Although this is the most convenient solution, the changing relation with the constriction percentage is not the same. during a horizontal closure, the reduced basin size correctly models the velocities. However, during a vertical closure, the basin level fills up during by increasing the sill height. The full storage of the basin area is then used. To summarize, the storage area increases during a vertical closure and decreases during a horizontal closure. This is why no direct relation can be found for both cases or any cases in between.

5.5.3. Calibration by contraction factor

Another possibility is to introduce a calibration factor for the depth averaged velocity. From several trail runs in Delft3D and the storage model, it was concluded that the error factor in the flow velocity is decreasing with increasing constriction factor. In this way an artificial calibration contraction factor μ_c can be included as a function of the constriction factor. Artificial, because it should not be confused with the physical contraction coefficient μ which is used to define the relative flow area in constricted flows.

On the basis of the trail runs, it is concluded that this function is a positive exponential function defined by definition 5.5

$$\mu_c(r) = X1 + X2 * e^{X3\cdot r - X4} \quad \text{with} \quad 0 \le r \le 1 \quad \text{and} \quad \mu_c \le 1 \tag{5.5}$$

With:

r	=	Constriction percentage	$= \frac{A_{w,t}}{A_{w,t=0}}$	[-]
A_w	=	Wet cross section at high water		[m^2]
$X1$	=	Constant 1 (initial error with no constriction)		[-]
$X2$	=	Constant 2 (exponential factor of upward slope 1)		[-]
$X3$	=	Constant 3 (exponential factor of upward slope 2)		[-]
$X4$	=	Constant 4 (exponential factor of upward slope 3)		[-]
μ_c	=	Artificial calibration contraction factor		[-]

Physically, it's logical to assume that the shape of this function is of this type, since the deviation decreases exponentially with increasing constriction percentage.

To determine this, several cases have to be evaluated to conclude a well founded correlation between the these two parameters. Furthermore for each cross section these cases define the values of the three constants $X1$, $X2$, $X3$ and $X4$. The following two cases are tested for several constriction percentages

- Fully vertical closure
- Fully horizontal closure

Since both cases show have the largest contrast in with respect to the many closure options possible. In other words, the assumptions is made that these cases form the boundaries for all situations possible in between by combining these cases and therefore, if the calibration factor is the same in both cases it can generally be used in all possible strategies that combine these cases.

The assumed exponential increase of μ_c with respect to constriction percentage r can best be tested on high constriction factors, because initial deviation are prospected to be more linear. Therefore, six different constriction percentages are analyzed for both cases. All tested cases can be found in table 5.3

Table 5.3: Calibration cases

Cases	**Constriction percentage r [%]**					
Vertical	10	50	80	90	95	99
Horizontal	10	50	80	90	95	99

To evaluate the cases. The maximum flow velocity occurring in one month of simulation is saved and compared in both models. The deviation factor can be computed by dividing the velocity from Delft3d with the flow velocity from the storage model (definition 5.6).

$$\mu_c(r) = \frac{U_{max,Delft3D}}{U_{max,storagemodel}} \tag{5.6}$$

To impose a constriction in the Delft3D model, a weir is used for vertical constriction and a thin dam is used for horizontal constriction. In case of the weir, Delft3D uses the same formulas to compute the flow velocity over the weir as the storage model (appendix C).

With use of cross sectional bathymetry, both the weir height and the length of the thin dams for every constriction percentage is determined beforehand. In this way, the results can directly be related to the constriction percentage.

5.5.4. Results

The results of the calibration cases are discussed in this section. In tables 5.4 and 5.5, the horizontal closure case is displayed. Maximum flow velocities for both Delft3D (D3D) and the Storage Model (SM) can be found in the first and second column. In the third column, the calibration contraction factor is computed.

As can be seen, the flow velocity is computed for all cross sections in order to be able to calibrate each cross section. However, for the horizontal case, only cross section 4 has enough data points to perform a reliable fit. Therefore, for a more reliable calibration, 7 other horizontal closure cases have to be performed (one for each cross section) in which the horizontal closure ends in that cross section. This is however outside of this study. The vertical closure case can help define relations for the other cross sections. The few data points that do exist for the outer cross sections will are still taken up into the calibration.

Table 5.4: Flow velocities both model during horizontal constriction cross sections 1-4

	$U_{CS1}[m/s]$			$U_{CS2}[m/s]$			$U_{CS3}[m/s]$			$U_{CS4}[m/s]$		
Constriction	D3D	SM	μ_c	D3D	SM	μ_c	D3D	SM	μ_c	D3D	SM	μ_c
0 %	1.25	3.8	0.33	2.18	6.32	0.34	2.1	3.61	0.58	2.21	4.56	0.48
10%	1.36	3.89	0.35	2.31	6.31	0.37	2.22	3.69	0.6	2.2	4.56	0.48
50%	0	0	0	0	0	0	2.98	4.06	0.73	2.92	5.92	0.49
80%	0	0	0	0	0	0	0	0	0	4.57	7.92	0.58
90%	0	0	0	0	0	0	0	0	0	6.02	7.97	0.76
95%	0	0	0	0	0	0	0	0	0	6.46	8.14	0.79
99%	0	0	0	0	0	0	0	0	0	6.78	8.2	0.83

Table 5.5: Flow velocities both model during horizontal constriction cross sections 5-8

	$U_{CS5}[m/s]$			$U_{CS6}[m/s]$			$U_{CS7}[m/s]$			$U_{CS8}[m/s]$		
Constriction	D3D	SM	μ_c	D3D	SM	μ_c	D3D	SM	μ_c	D3D	SM	μ_c
0 %	2.04	4.72	0.43	2.22	4.81	0.46	2.11	4.93	0.43	1.58	3.86	0.41
10%	2.08	4.71	0.44	2.37	4.83	0.49	2.28	4.92	0.46	0	0	0
50%	2.77	6.07	0.46	3.36	6.14	0.55	0	0	0	0	0	0
80%	4.34	8.25	0.53	0	0	0	0	0	0	0	0	0
90%	0	0	0	0	0	0	0	0	0	0	0	0
95%	0	0	0	0	0	0	0	0	0	0	0	0
99%	0	0	0	0	0	0	0	0	0	0	0	0

Table 5.6: Flow velocities both model during vertical constriction cross sections 1-4

	$U_{CS1}[m/s]$			$U_{CS2}[m/s]$			$U_{CS3}[m/s]$			$U_{CS4}[m/s]$		
Constriction	D3D	SM	μ_c	D3D	SM	μ_c	D3D	SM	μ_c	D3D	SM	μ_c
0 %	1.25	3.8	0.33	2.18	6.32	0.34	2.1	3.61	0.58	2.21	4.56	0.48
10%	1.97	3.93	0.50	2.25	6.07	0.37	2.69	4.32	0.62	2.82	5.62	0.50
50%	2.51	4.63	0.54	2.07	4.76	0.44	2.97	5.00	0.59	3.14	5.30	0.59
80%	2.57	3.32	0.77	2.34	3.31	0.71	2.50	3.36	0.74	2.44	3.55	0.69
90%	2.12	2.41	0.88	1.84	2.40	0.77	2.20	2.47	0.89	1.87	2.71	0.69
95%	1.60	1.76	0.91	1.60	1.73	0.92	1.62	1.83	0.89	1.55	2.15	0.72
99%	1.58	1.63	0.97	1.44	1.55	0.93	1.03	1.11	0.92	1.49	1.59	0.94

Table 5.7: Flow velocities both model during vertical constriction cross sections 5-8

	$U_{CS5}[m/s]$			$U_{CS6}[m/s]$			$U_{CS7}[m/s]$			$U_{CS8}[m/s]$		
Constriction	D3D	SM	μ_c	D3D	SM	μ_c	D3D	SM	μ_c	D3D	SM	μ_c
0 %	2.04	4.72	0.43	2.22	4.81	0.46	2.11	4.93	0.43	1.58	3.86	0.41
10%	2.76	5.79	0.48	2.91	5.85	0.50	2.78	5.39	0.52	2.26	4.01	0.56
50%	3.18	5.33	0.60	3.08	5.34	0.58	3.20	5.32	0.60	3.24	5.19	0.62
80%	2.54	3.58	0.71	2.49	3.59	0.69	2.32	3.60	0.65	2.68	3.57	0.75
90%	2.15	2.75	0.78	2.08	2.77	0.75	2.16	2.77	0.78	2.05	2.75	0.75
95%	1.62	2.20	0.74	1.72	2.22	0.77	1.85	2.22	0.83	1.69	2.19	0.77
99%	1.54	1.65	0.93	1.58	1.68	0.94	1.64	1.69	0.97	1.62	1.65	0.98

From tables 5.4, 5.5, 5.6 and 5.7, two clear patterns are visible:

- Increasing flow velocities in both models with increasing constriction percentage
- An exponentially increasing calibration contraction factor

From these cases, it can already be concluded that a consistent relation exists between constriction percentage and the calibration contraction factor. Furthermore, the assumption of increasing accuracy of the storage model with increasing constriction percentage is indeed hereby proved.

The results from the model runs are plotted and fitted with use of relation 5.5. The data points of both cases are plotted in one figure for every individual cross section. Data point with value zero are not taken up in the fit. Since the fit shows some deviations, a 15% upper and lower deviation limit are plotted as well. Since this is a calibration process and the flow velocities should not be underestimated, a safe line is chosen. In this case, the 15 % deviation up presents a representative relation for which almost all points are below the line. However in some cases this means that near a 100 % constriction a calibration contraction factor higher than 1 is found. This is easily solved by capping the value to a maximum of 1, which was already defined by relation 5.5.

In table 5.8, the resulting values of the exponential calibration coefficients mentioned in relation 5.5 are defined.

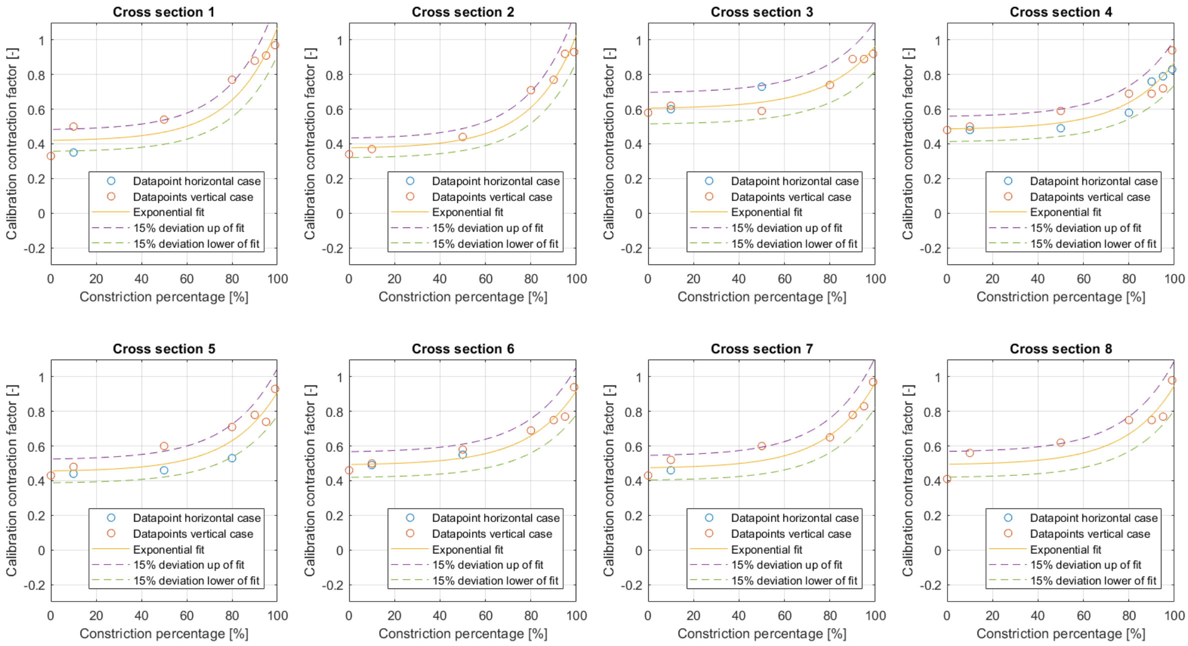


Figure 5.22: Results of fitting exponential function through velocity data points

Table 5.8: Results of calibration; values of exponential calibration coefficients

Calibration coefficients	X1	X2	X3	X4
CS 1	0.48	1.15	0.051	5.5
CS 2	0.43	1.15	0.051	5.5
CS 3	0.69	1.15	0.045	5.5
CS 4	0.55	1.15	0.045	5.5
CS 5	0.52	1.15	0.047	5.5
CS 6	0.56	1.15	0.047	5.5
CS 7	0.54	1.15	0.048	5.5
CS 8	0.56	1.15	0.047	5.5

5.5.5. Discussion and conclusions

In this section, the validity of the calibration is discussed. The first problem with the calibration is the assumption that the Delft3D model correctly models the flow velocities in the Gulf of Khambhat. As discussed in 5.4.6, the flow velocities in the Delft3D model resemble known measurements in unconstricted situation, which are in the order of 2-3 m/s (Royal Haskoning, 1998a) and not in the order of 5- 6 m/s. It might be that the Delft3D model is more accurate at the start of the closure, but deviates at the end. To account for this, the extra 15 % safety margin is taken up to assure that the storage model is correct at large constriction percentages.

Secondly, the assumption of a relation between constriction percentage and calibration contraction factor has been proven. With this knowledge it is possible to use both horizontal and vertical closure options with the same calibration coefficient. Quantitatively, the exponential relation has a high correlation with the data points. However, more cases should be analyzed to increase accuracy of the fit. Also for this reason, the extra 15 % safety margin is implemented.

Thirdly, in other closure case studies, the storage model is only used at larger constriction percentages at the last stages of the closure. However, in this study, the key issue is that the complete closure is modelled to evaluation an integrated solution. Therefore it is required to have accurate flow velocities during the complete closure sequence. With the calibration results it can be argued that the performance and accuracy of the model has increased significantly.

5.6. General conclusion usability storage model

The main question in this chapter was stated as follows:

"Can a simplified storage model based on eight cross sections correctly describe the flow velocities during the closure of the Gulf of Khambhat?"

The answer to the main questions is no. The variations in discharge and flow velocities between the storage model and the Delft3D are significantly large up to a factor 2-3. This is due to an inaccurate approximation of the storage area under the assumption of an equal water level in the basin (ignoring inertia and resistance). However, the error of the storage model with respect to the flow velocities in the Delft3D model reduces with increasing constriction of the closure gap, because influences of inertia and resistance are decreasing during the closure. This relation is further researched and quantified in a calibration process. The results show a high correlation between constriction factor and the error in the flow velocity. The relation is an exponential function and is calibrated for every cross section with use of four exponential coefficients.

With this calibration, the simplified storage model can describe the flow velocities in every cross section with more accuracy during the complete closure sequence. It especially lowers initial flow velocities with as result that smaller rocks or even sand can be used to perform the initial closure. Further validation for more cases should be performed before fully utilizing the artifical constraction factor for more (combined) strategies . Furthermore, the calibration is case dependent and can only be used for the closure of the Gulf of Khambhat with the given bathymetry and water levels. For other cases, the calibration procedure should be repeated.

6

Case studies

6.1. Introduction

In this study, the relation between the most important design requirements (material, bed protection and equipment) and the closing strategy is implemented in an optimization model which consists of several cross sections to incorporate all strategy related influences on the total costs (most important cost factors). In this chapter, the model is used to answer several questions regarding the optimal closure strategy and obtain advice for the Kalpasar Development Project regarding the key design aspects of closure strategies. The following questions are researched in the presented sections:

1. **Case study 1:** *"Does the optimal closing strategy significantly change if more cross sections are used in the first design phase compared to a single cross sectional model? "* – section 6.3
2. **Case study 2:** *"What is the effect of a change in construction capacity on cost for dam material and bed protection with respect to the equipment cost?"* - section 6.4

For the overall validation of the model and to answer the second research question, a secondary question is utilized for both cases:

"What strategic design choices significantly influence the costs factors and the total costs of a strategy?"

First, a general approach is presented in section 6.2. After all cases are discussed in there representative sections, a summary of the conclusions with respect to these three questions is presented in section 6.5.

6.2. Approach

The optimization model consists of more than 45 different influential parameters. The influence of these parameters can be tested. However, they can't be individually analyzed with respect to all other parameters. This would require a large quantity of computations. To evaluate the influence of the most relevant parameters a reference case is used. Using the reference case, several case studies are set up in which one or more parameters are centralized. The influence of these parameters can then be analyzed, from which a conclusion of the case study question is drawn. The reference case in this study is the closure dam in the Gulf of Khambhat and most of the parameters used in the model are known or can be guesstimated based on knowledge of old case data. The standard values for all parameters of the reference case are discussed in appendix G.

The importance of each parameters is tested in a number of case studies to see if the model performance is well enough to use it on the actual Kalpasar closure dam.

6.3. Case study 1: One vs multiple cross sections

During design processes of tidal basin closures, the wet cross section to be closed is often hydraulically modelled by one cross section with use of the storage model. This cross section represents the last channel which requires closing and is often the most important to analyze. The shallow parts are closed first, and then final closure takes place in the channel. This technique was also used by Royal Haskoning in 1998 and Broos and Wiersema followed them with their re-analysis using a model with only a single cross section. Both analyzed the last phase of the closure of the Gulf of Khambhat closely, which, according to Royal Haskoning, circumcised the last 10 km of the closure gap (single cross section).

According to Huis in ‘t Veld, the shape of the actual bathymetry has a large influence on the flow velocity and therefore the strategy chosen. The shape of the bathymetry could include several channels and tidal flats and can’t be modelled with the use of only one cross section but must be divided into more. The sequence and closure method chosen for each section influences the design requirements and therefore the simplicity of a single cross-sectional model is not enough to design the optimal closure strategy. To prove Huis in ‘t veld is right and to prove the relevance of the multi-sectional optimization model the most important question in this case study is defined by question one.

“Does the optimal closing strategy significantly change if more cross sections are used in the first design phase compared to a single cross-sectional model?”

6.3.1. Approach

First, the question is rewritten into a hypothesis which can be proved or disproved for the ease of performing research:

“The optimal closing strategy can significantly change if more model cross sections are used in the first design phase.”

To prove or disprove the hypothesis, several varying case studies are developed for the same closure case. This closure case is modelled for 1, 3, 5 and 8 cross sections, all having the same wet cross-sectional area to close. The bathymetry of the Gulf of Khambhat is not used here, because equal (symmetrical) conditions should be used for a better comparison. In multi cross-sectional models, the tidal flats (2/3 of length) are positioned at CD + 0 m and the channel (1/3) is positioned at CD -15 m. By keeping the wet cross section constant, the single cross-sectional model averages to a single "channel" at CD -5 m (figure H.11). In the case of 3 cross sections, one channel is present within two tidal flats (figure H.13). For 5 cross sections, two channels exist within 3 tidal flats (figure H.15). For 8 cross sections, 3 channels exist within 4 tidal flats (figure H.17).

Within these 4 differently cross sectioned models, 6 different closing options are tested ranging from completely horizontal to completely vertical with in the middle percentile combinations of a horizontal and vertical closure with respect to the wet cross section:

- **Option 1:** Completely horizontal
- **Option 2:** 20% vertical, 80% horizontal
- **Option 3:** 40% vertical, 60% horizontal
- **Option 4:** 60% vertical, 40% horizontal
- **Option 5:** 80% vertical, 20% horizontal
- **Option 6:** Completely vertical

For all 6 options stated above, several strategies within these options are still possible. For example: closing the tidal flats first and then the channels or vice versa. Again 6 possible cases are identified in which each of the above stated combined closures can be tested:

- **Case 1:** Horizontal from shoreline to middle, remaining part vertical
- **Case 2:** Horizontal flats first, remaining part vertical
- **Case 3:** Vertical from bottom up, remaining part horizontal

- **Case 4:** Vertical channels first, remaining part horizontal
- **Case 5:** Horizontal channels first, remaining part vertical (not recommended)
- **Case 6:** Vertical flats first, remaining part horizontal (not recommended)

Realistically, cases 3 and 6 are not logical. In cases where the flats get filled first, a horizontal closure is recommended otherwise a cable-way should be installed. In case the channels would be filled first, a vertical closure is recommended, otherwise a work island would be required from where a horizontal closure could start. Only cases 1,2,3 and 4 are therefore relevant to test.

The hypothesis posed in this case is very broad and it can be speculated that in some cases the resulting cheapest solution will not change if more cross sections are chosen. However, what can be proven is that if in any reasonable variation of this case study the cheapest option changes. With reasonable variation, the elementary cost is meant since these are the most unreliable parameters in the study. Small adaptations to their cost values might induce the change required to prove the hypothesis. Furthermore, a significant change must occur to prove the hypothesis, since a lot of parameters can influence the final evaluation. A small difference can be within the interval of possible evaluations. With many parameters and computations in the model, the 5 and 95% interval of the final evaluation is very complex to quantify. For this, an assumption is made: A relative difference in cost of more than 10% between strategies is established as significant enough.

The reference case and three variations are tested for all options:

1. Relatively higher material prices
2. Relatively higher equipment prices
3. Relatively higher bed protection prices

6.3.2. Results

The results of the runs are displayed in appendix H, here detailed discussions and conclusions can be found regarding the results of all cases.

In table H.1, the final results of the least cost options for al cases are presented. The second case showed the most promise to confirm the hypothesis and, the detailed results of this case can therefore be found in figure H.11 to H.18

Table 6.1: Summary results case study 1: least cost options for all tested cases

Case 1: 1 Horizontal - 2 Vertical, regular	**1 CS**	**3 CS's**	**5 CS's**	**8 CS's**
Reference prices	6	5	5	5
Increased bed protection prices	6	5	5	5
Increased equipment prices	5	4/5	5	5
Increased material prices	6	5	5	5/6
Case 2: 1 Horizontal - 2 Vertical, flats first				
Reference prices	6	5	6	4
Increased bed protection prices	6	5	6	5
Increased equipment prices	5	4/5	4/5/6	4
Increased material prices	6	5	6	4/5/6
Case 3: 1 Vertical - 2 Horizontal, regular				
Reference prices	3	6	6	6
Increased bed protection prices	6	6	6	6
Increased equipment prices	3	6	6	6
Increased material prices	6	6	6	6
Case 4: 1 Vertical - 2 Horizontal, channels first				
Reference prices	3	6	6	6
Increased bed protection prices	3/6	3/6	6	6
Increased equipment prices	3	3	6	6
Increased material prices	6	6	6	6

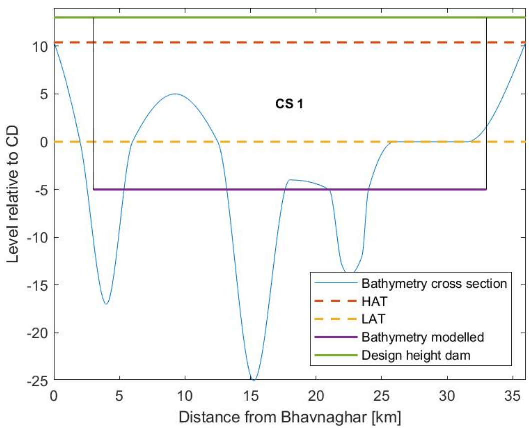


Figure 6.1: Single cross section

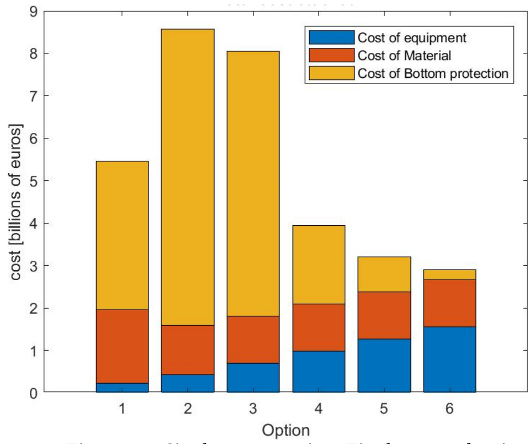


Figure 6.2: Single cross section: Final cost evaluation

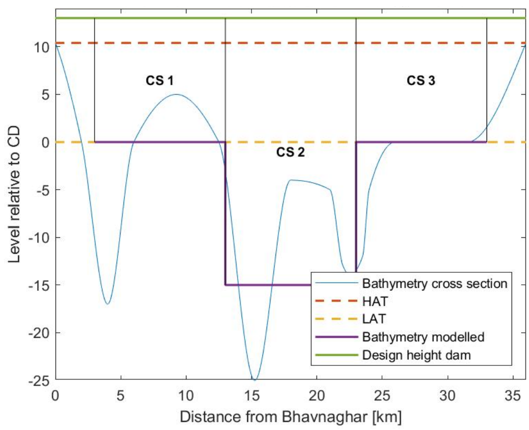


Figure 6.3: Three cross sections

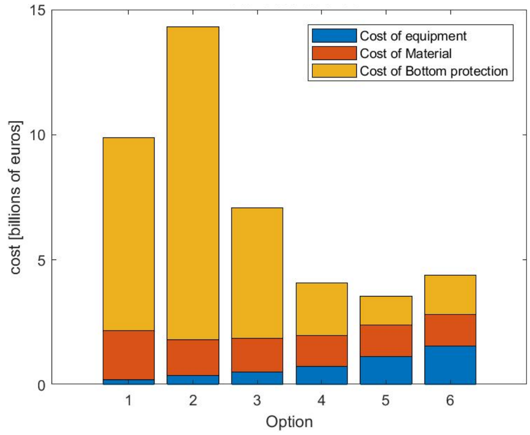


Figure 6.4: Three cross sections: Final cost evaluation

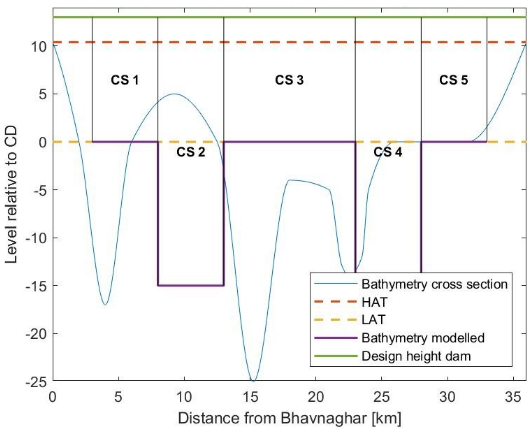


Figure 6.5: Five cross sections

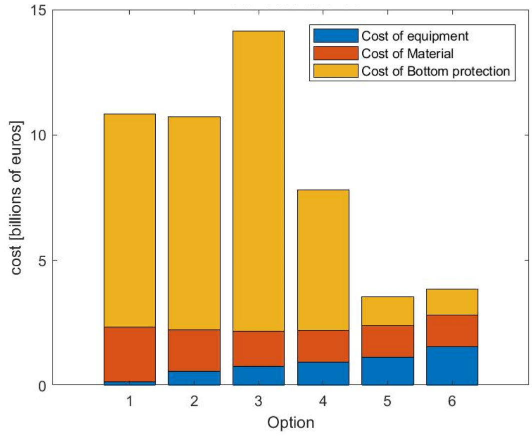


Figure 6.6: Five cross sections: Final cost evaluation

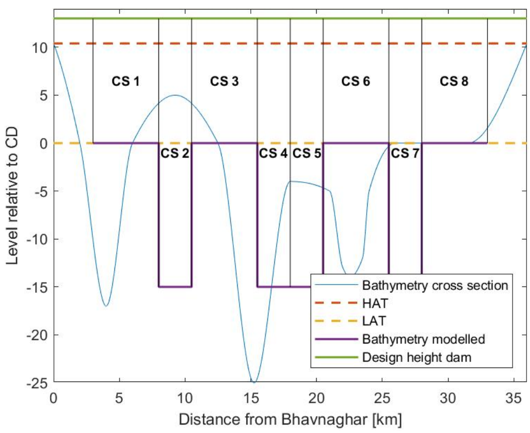


Figure 6.7: Eight cross sections

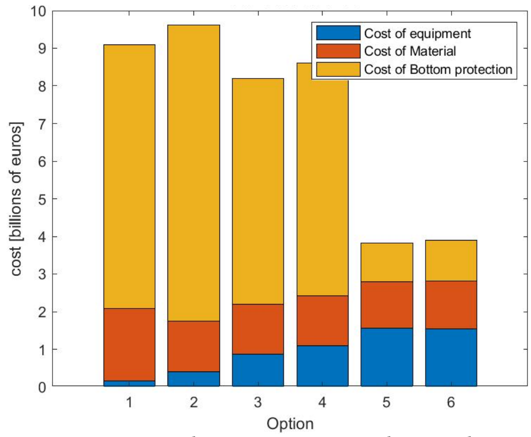


Figure 6.8: Eight cross sections: Final cost evaluation

6.3.3. Discussion

In this subsection, the results of the case studies are discussed (figures H.11 to H.18). The results of case 1 (1 horizontal, 2 vertical, regular) is discussed in more detail and is representable for most remarks and conclusions that could be drawn from the other cases. To compare the results, it is important to state that whatever strategy is tested, the capacity during the closure is always kept constant in every phase. In case of partial closures, the capacity is divided as such that all phases start and end at the same time so no significant delays occurred due to the difference in phasing. The only physical difference between the single and multi cross-sectional models is the size of the closure dam required, which is larger in case a channel is present. Furthermore, it must be stated that the choice of the elementary cost ratio's between equipment and bed protection has a large influence on the final evaluation and the least cost option. Some conclusions are based on these *case related parameters* and they could significantly change if adjusted. Lastly, the calibration factors determined in chapter 5 are not used, since they can only be used with the exact bathymetry of the Gulf of Khambhat.

First, it can be noticed that the required quantity of bed protection varies significantly by changing the strategy in all models. The influence of this cost factor can directly be concluded as the most important. The equipment cost also varies significantly with respect to the material cost, which is the least varying cost factor. The cost of material remains relatively constant in all options, thereby also proofing that these models can be compared to each other while having different bathymetries (and therefore different dam design cross sections).

In all models, the cost of bed protection increases exponentially with increasing percentage of horizontal closure. However in almost all models, option 1 (100% horizontal) defies this relation because it requires less bed protection than option 2 (80% horizontal). This exception can be explained by the increased cost of bed protection at the end of the closure. The last 20% is closed vertically under high velocity circumstances. Since the length of protection required in this last section is used across the complete width of this section, relatively more area of bed protection is required. A horizontal closure requires only half that area (although the length required could me more, the resulting area is less). In the 5 cross-sectional model, The largest peak in bed protection switched from option 2 to option 3 (60% horizontal, 40% vertical). This can again be explained by vertical closure at the end, which happens exactly in the shallow part in the middle of the bay. High flow velocities in shallow water cause large scour holes, since the discharge can't spread towards deeper parts (advantage of velocity dispersion with increasing depth through continuity of flow). The peak costs are therefore almost 15 billion euro, which compared to the single cross sectional model is a factor 1.5 larger. In the 8 cross-sectional model, options 1 to 4 experience the same problem of creating a high flow velocity on the tidal flats. Because there are more channels and flats included each option averages the final vertical closure accros a channel and a flat. The result is that they all are expensive (about 9 billion euro), but are relatively cheaper than option 2 in the 3 cross sectional model, or option 3 in the 5 cross-sectional model (15 billion euro).

The least bed protection cost is in almost all models found in option 6 (completely vertical). However, in all models with more than a single cross section, option 5 (80 % vertical and 20% horizontal) requires the same or even less bed protection. This can also be explained by the same phenomenon that was stated earlier; horizontal closures require only half the total area of bed protection with respect to the cross section width times the length required (see calculation method of the bed protection area in appendix D) . In this option, the area of bed protection is reduced at the start of the closure, where in all multi-cross sectional models a horizontal closure is executed on the tidal flats. This reduces the area required without increasing the flow velocity significantly.

The result of the development of closing the flats first horizontally influences the least cost option in the 3 and 5 cross-sectional models. In some cases option 5 and even option 4 is the cheapest strategy. bed protection needs are still low because the flow velocity has not increased significantly yet and the length of the cable-way required is reduced. This type of strategy was therefore also proposed by Royal Haskoning in 1998.

In case 2, first the flats are closed horizontally. This case confirmed the previous statement and therefore the least cost option in most multi cross-sectional models differs significantly with respect to the single cross-sectional model. The hypothesis can be confirmed based on this results.

6.3.4. Conclusions

To conclude the case, the hypothesis is stated again:

“The optimal closing strategy can significantly change if more model cross sections are used in the first design phase.”

The hypothesis can be confirmed based on the combined results of all cases. Case 1 and 3 can’t directly confirm the hypothesis, because the relative differences between the least cost options are too small. However, they showed potential differences. Case 2 and 4 confirm this potential. They showed a significant difference in the least cost option for multi cross sectional models. The most logical explanation is that more options are available to close the total cross section and in cases 2 and 4, these unique options are used to benefit the least cost option.

The secondary question was constructed to assist in answering the main question, to increase knowledge about the strategic optimization process and to validate the model performance. The question is stated here again:

"What strategic design choices significantly influence the costs factors and the total costs of a strategy?"

Relevant conclusions that can be drawn from the results, with respect to this question are:

- During a horizontal closure, more bed protection is required. By increasing the percentage of vertical closure, the quantity of bed protection required decreases significantly.
- The equipment cost increases significantly with increasing percentage of vertical closure and increases significantly when a cable-way/bridge system is required to execute the closure.
- The material cost are *relatively* constant during all closure strategies and is therefore less relevant in the choice for a strategic closure. It does proof (to some extent) that these models can be compared to each other while having different bathymetries (and therefore different dam design cross sections).
- Material costs increase with increasing percentage of horizontal closure, due to the increased need larger stones which can’t be delivered by the quarry. This has to be compensated by more expensive gabions. When closing fully vertically, the cost for material significantly decrease due to the decreased design width of the dam crest.
- The maximum cost increase if a channel is present in the wet cross section by a factor 1.25-1.5 due to increasing cost of bed protection. Using a more specific strategic closure method related to the uniqueness of the bathymetry in these cases is therefore recommended to minimize extra costs.
- Small percentages of initial partial vertical closures (20-40%) over the full width are not recommended since they close off the flats increasing velocities in the channel. The rest of the channel has to be closed horizontally, which requires much time, since the cross section of the dam is large in a channel. The reduction of the flow velocity through dispersion after the initially placed dam segment is still too small to significantly reduce the flow velocity, resulting in large amount of bed protection required.
- An inital partial horizontal closure of the tidal flats can decrease bed protection cost significantly

The validity and performance of the model can be assessed qualitatively by these conclusions. All conclusions can be assessed as logical and most of them are known factors in the design process of a closure strategy (Huis in ’t Veld, 1987). The best example is the initial horizontal closure (20-40%) of the tidal flats which exceptionally cheap with respect to all other cases with horizontal closures. This design method was therefore also used by Royal Haskoning in the Kalpasar case.

6.4. Case study 2: Construction capacity vs. material and bed protection costs

The first case proved to some extent that the required quantity of bed protection varies the most with respect to the choice of closure strategy. This cost factor decreases significantly with increasing percentage of vertical closure. The same pattern holds for the material costs but with less variation. The second largest cost factor is the equipment, which (in contrast) increases in cost with increasing percentage of vertical closure.

Therefore, it can be concluded that a trade-off exists between both material and bed protection costs and the equipment cost. The most interesting parameter in the choice of a closure strategy that influences all these aspects is the *construction capacity*, which was kept constant in the previous case study.

The material model includes this trade-off by including time in the dimensioning of rocks. The bed protection model also includes time in the dimensioning of the required area. For a more elaborate explanation how time influences the dimensions of material and bed protection required, read sections 4.4.3 and 4.5.3. The result of the setup of these models is that a faster construction time could decrease the costs of both cost factors. However, this will increase equipment costs. This trade-off is therefore analyzed in more depth in this second case study. The main question for this case study is therefore as follows:

"What is the effect of a change in construction capacity on the cost for dam material and bed protection with respect to the equipment cost?"

6.4.1. Approach

The answer to this question can be quantified with help of several cases where the construction capacity is varied. In this case study, the bathymetry of the Kalpasar case can be used and therefore also the artificial contraction factor from the calibration. The approximated cross sections used in the model can be found in figure 6.9.

In the model, equipment capacity in a horizontal closure is capped at 25.000 m^3 per day with the chosen type of trucks. However, to increase this capacity, larger dump trucks can be used. As a cost, the dam crest must be wider and the truck cost (initial and time dependent) increase. This is another trade-off with respect to material cost. A variety of 3 different dump trucks is used to cover the spectrum of tested construction capacities. All three options are displayed in table 6.2 and are based on existing dump trucks (Division of General motors, 1974):

Table 6.2: Truck capacities of three different truck types with specifications (Division of General motors, 1974)

	Initial cost [e]	**Operating cost** [e]	**Capacity** [m^3/u]	**Width** [m]	**Unload time** [s]	**Crest width** [m]	**Daily capacity** [$m^3/day/bund$]
Truck 1	100.000	1.000	15.000	2.5	60	8	12.500
Truck 2	500,000	1.500	180.000	3.6	180	10.2	50.000
Truck 3	2.000.000	2.000	300.000	9.2	200	21.2	75.000

For a partial vertical closure, it is assumed that the number of ships used can increase indefinitely, thereby increasing the capacity to any capacity within the spectrum of tested capacities.

For the cable-way system, also three systems are chosen to cover the complete spectrum of construction capacities. Each with specific initial and length dependent costs. In table 6.3, the cost for three cable-way/bridge systems with equivalent daily capacities as the trucks are defined. For this is is assumed the horizontal closure is performed from both sides and thus the total daily capacity doubles. The cost are based on a almost linear increase in cost with respect to the capacity. In reality these functions should be determined with much more precision.

Table 6.3: Capacities and costs for three different cable-way/bridge systems

	Initial cost [$euro$]	**Length cost** [$euro/m$]	**Daily capacity** [m^3/day]
Cable-way/bridge 1	50.000.000	30.000	25.000
Cable-way/bridge 2	300.000.000	100.000	100.000
Cable-way/bridge 3	600.000.000	200.000	150.000

To investigate the influence of the variation in construction capacity, several strategies are tested for the 3 variations in construction capacity associated with the three maximum capacities of the trucks and cable-way/bridge systems.

- 25.000 m^3/day
- 100.000 m^3/day
- 150.000 m^3/day

6.4.2. Strategies

The strategies tested can be subdivided into two main strategies, which were developed based on the results of case study one:

- 1: Horizontal, 2: vertical, closing flats first
- Fully vertical, closing channels first.

For each strategy, 3 alternative closure strategies are designed to cover most of the spectrum of possibilities. The resulting 6 options are described in more detail below, but are also visually explained in figures 6.10 to 6.15.

- **Option 1: Horizontal - vertical 1 , figure 6.10**
  1. Closing outer flats first horizontally, while closing middle cross sections vertically to 40% of the depth (to minimize the quantity of bed protection required)
  2. Vertically fill up the channels, in steps, starting with the deepest.
  3. Vertically raise the sill level over the remaining total width to design height .
- **Option 2: Horizontal - vertical 2 , figure 6.11**
  1. Closing outer flats first horizontally, while closing middle cross sections vertically to 40% of the depth.
  2. Vertically fill up the channels, in steps (starting with the deepest), while closing the tidal flat on the right (CS 7) horizontally.
  3. Vertically raise the sill level over the remaining total width to design height .
- **Option 3: Horizontal - vertical 3 , figure 6.12**
  1. Closing outer flats (CS 1 and 8) and inner flats (CS 3) first horizontally, while closing middle cross sections vertically to 40% of the depth.
  2. Vertically fill up the channels, in steps (starting with the deepest), while closing the tidal flat on the right (cross section 7) horizontally.
  3. Vertically raise the sill level over the remaining total width to design height .
- **Option 4: Vertical over full width 1, figure 6.13**
  1. Close vertically over the full width up to 70% of the depth of each cross section
  2. Close vertically in steps starting from the deepest channel up to the most shallow tidal flat.
  3. Vertically raise the sill level over the remaining total width to design height .
- **Option 5: Vertical over full width 2, figure 6.14**
  1. Close vertically over the full width up to 40% of the depth of each cross section
  2. Close vertically in steps starting from the deepest channel up to the most shallow tidal flat.
  3. Vertically raise the sill level over the remaining total width to design height .
- **Option 6: Vertical over full width 3, figure 6.15**
  1. Close vertically in steps starting from the deepest channel up to the most shallow tidal flat.
  2. Vertically raise the sill level over the remaining total width to design height .

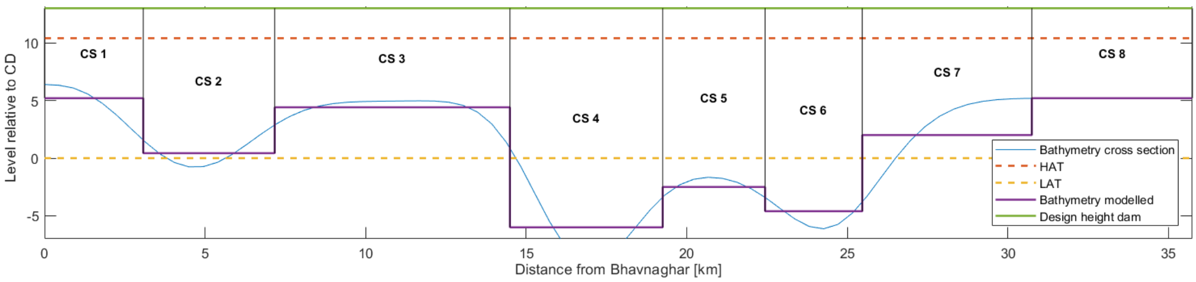


Figure 6.9: Cross section Kalpasar case

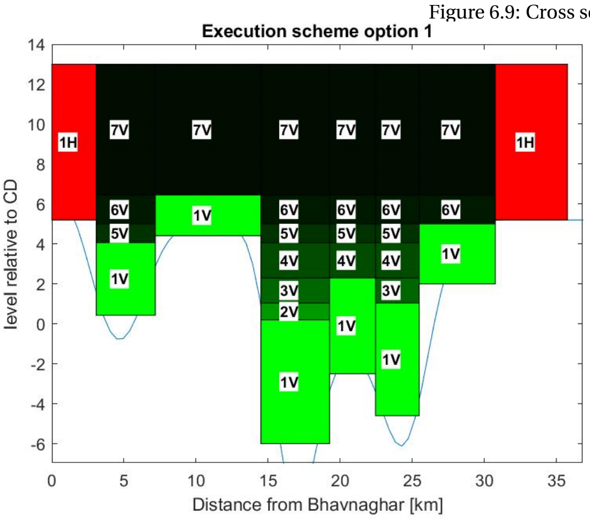


Figure 6.10: Execution scheme option 1

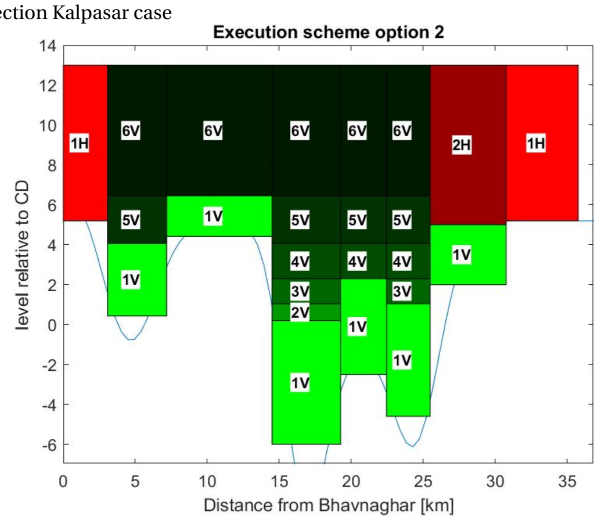


Figure 6.11: Single cross section: Final cost evaluation

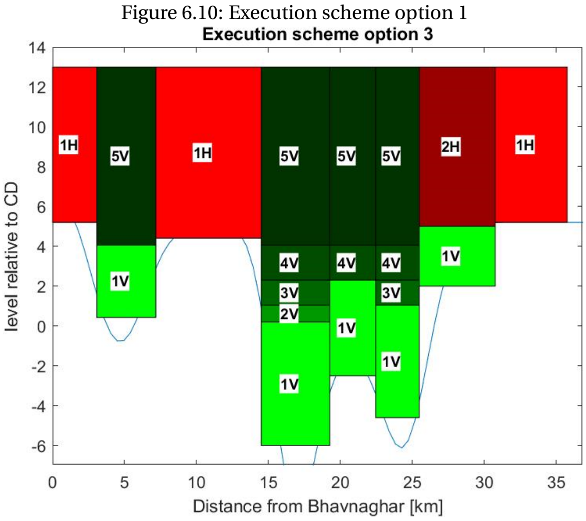


Figure 6.12: Execution scheme option 1

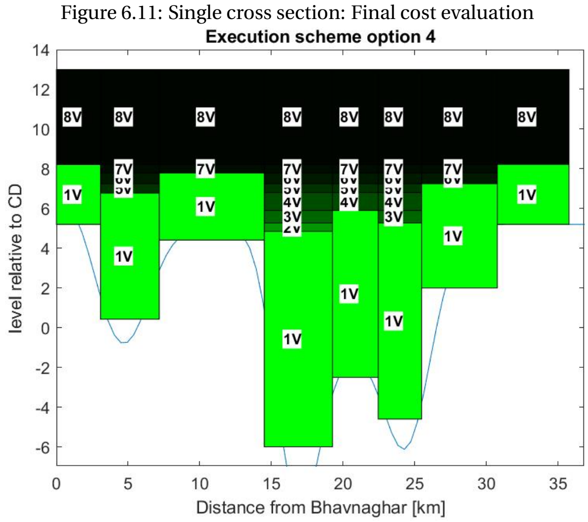


Figure 6.13: Single cross section: Final cost evaluation

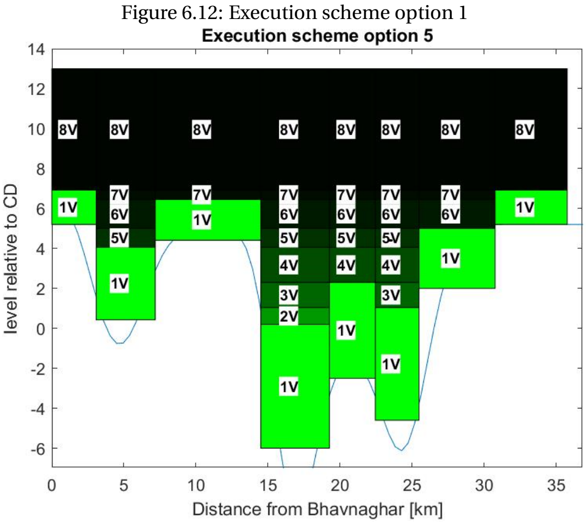


Figure 6.14: Execution scheme option 1

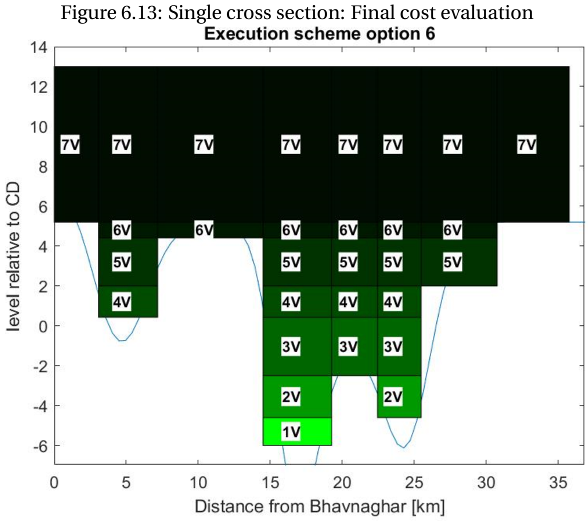


Figure 6.15: Single cross section: Final cost evaluation

6.4.3. Discussion result and conclusions

In the following paragraphs, the impact of increasing the construction capacity is assessed and discussed for each cost factor and the final evaluation.

Comparison material quantity In figures 6.16 to 6.18, the material required is plotted for three construction capacities. Opposite of what was expected, the material requirements increase with increasing capacity.

Both capacities of 25.000 and 100.000 m^3/day show no significant differences. Small differences can be noticed in options 1, 2 and 3 with increasing quantity respectively. This can be explained by the wider crest width that is required for the partial horizontal closures. With a capacity of 150.000 m^3/day, the influence of the larger geometric dam profile becomes significant in option 3. In this option, a large part of the length of the dam (cross sections 1,3,7 and 8) are closed using dump trucks that require a larger dam width.

The significance of the difference is however debatable. From case study 1, it was concluded that a significant difference in material requirements resulted from more expensive gabions that replace the unwanted overproduction. Increasing the capacity results in smaller stone and therefore less overproduction. However, in this case, an overproduction is not required in all strategic options. The calibration coefficient provided a significant decrease in flow velocity and because of the largely vertical closure, the maximum flow velocities generated in case one are less and therefore the required stone sizes decreased. It can be debated if the calibration coefficient is correct to use in all cases. This is further discussed in section 6.4.4 of this chapter.

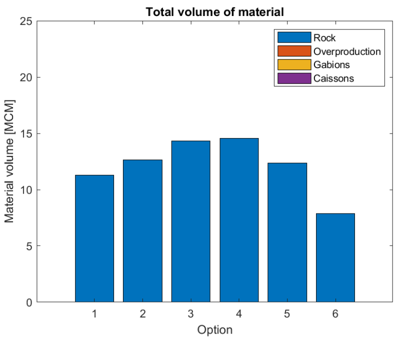


Figure 6.16: Material Required - 25.000 m^3/day

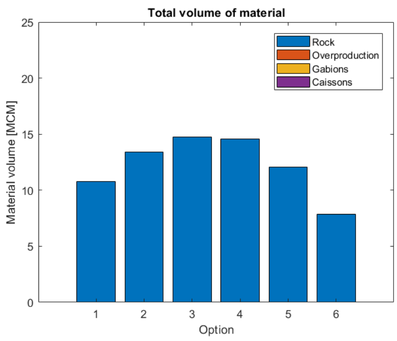


Figure 6.17: Material Required - 100.000 m^3/day

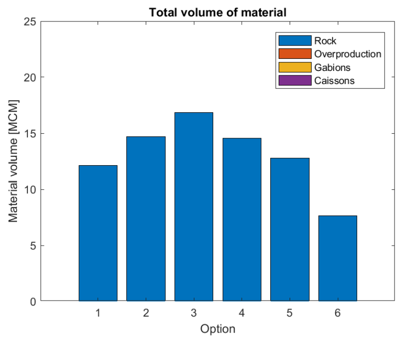


Figure 6.18: Material Required - 150.000 m^3/day

Comparison rock sizes In figures 6.19 to 6.21, the quarry yield curve and the resulting cumulative stone mass requirements (for option 3) are presented. All requirement lines are well below the quarry yield curve and thus no overproduction is required. However, the rock sizes are expected to decrease with increasing capacity, which follows from these results. In case of a capacity of 25.000 m^3/day, the slope of the *required* line is steep compared to the 100.000 and 150.000 m^3/day lines. The line starts at 80%, meaning 20% is less than 10 kg. With an increased capacity, the steepness of the slope of the line decreases, meaning less percentages are required of the larger rocks. However, the largest stone size required is in all cases the same. This can be explained by the relatively large step size of the iterative rock model which is elaborated in more detail in section 6.4.4. Furthermore, the largest stone sizes in all cases are exceptionally small after checking with a simple hand calculation. This deviation is also elaborated in more detail in section 6.4.4.

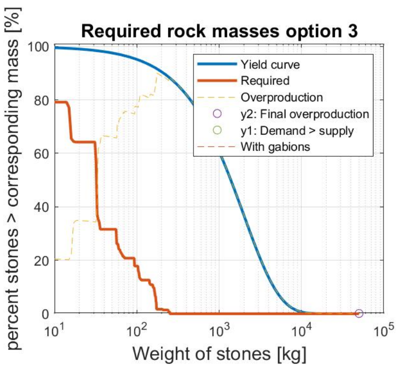


Figure 6.19: Rock size during the closure - 25.000 m^3/day

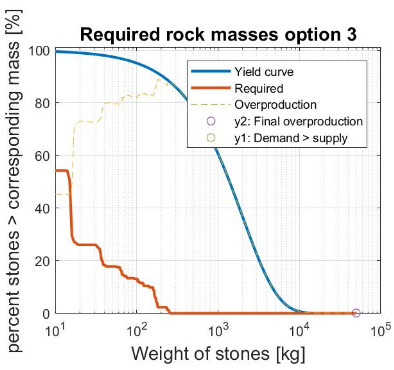


Figure 6.20: Rock size during the closure - 100.000 m^3/day

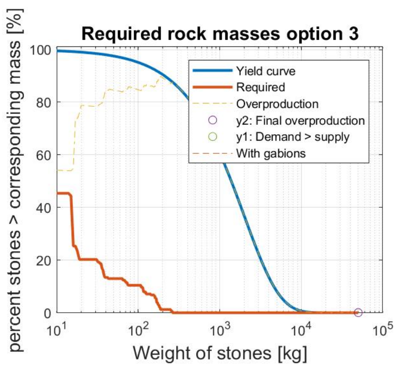


Figure 6.21: Rock size during the closure - 150.000 m^3/day

Conclusions material quantity and rock sizes Based on these results for the required material quantity and the rock sizes it can be concluded for this case that :

- Increasing the construction capacity increases the material quantity required for horizontal closures up to significant amounts due the increased crest width.
- Implementing time as a component in the rock model decreases the rock sizes required by increasing the construction capacity. This could have a significant effect on the required material quantity if overproductions are needed. However, in this case study, the influence is insignificant with respect to the extra material required for the crest width. In cases where large overproductions or gabions are required this feature can have more impact on the final evaluation and might offer a better trade-off.
- By creating a rock size design model which designs each layer of rock individually, and by executing it with a probabilistic model, the rock requirements are worked out in great detail. Rock sizes are exactly designed for a specific time period during the closure, resulting is smaller rock sizes than if computed with a deterministic computation. The difference of the probabilistic model and the deterministic computation is elaborated further in section 6.4.4

Comparison bed protection In figures 6.22 to 6.24, the quantity of bed protection is plotted for three construction capacities. The result follows the line of expectations; with an increased construction capacity, deep scour holes are prevented (figure 6.25) and the required quantity of bed protection decreases significantly . This is an interesting result, showing that the times scale of the development of the scour hole is in the same range as the time scale of the construction time.

However, the largest difference appears between the 25.000 and 100.000 m^3/day cases. A significantly smaller difference can be noticed between the 100.000 and 150.000 m^3/day cases. The large decrease of bed protection requirements is created by retaining the development of the scour holes with time. The 150.000 m^3/day case only subtracts one month of the total construction time with the 100.000 m^3/day case, while that case subtracts three complete years of the construction time of the 25.000 m^3/day case. An example of this explanation is displayed in figure 6.25, where the development of a scour hole is plotted with the occurring flow velocity in time.

Another observation is made regarding the type of bed protection. All types have decreased evenly in volume, since the maximum flow velocity is not influenced by the increased capacity, only the duration of exposure and therefore the length.

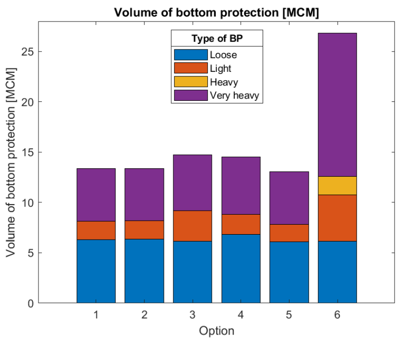


Figure 6.22: bed protection required - 25.000 m^3/day

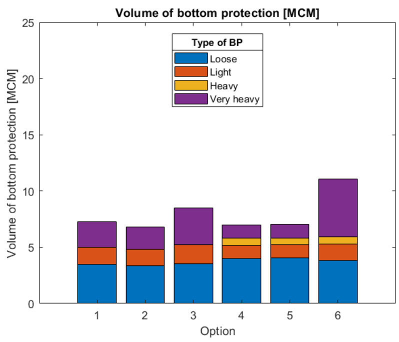


Figure 6.23: bed protection required - 100.000 m^3/day

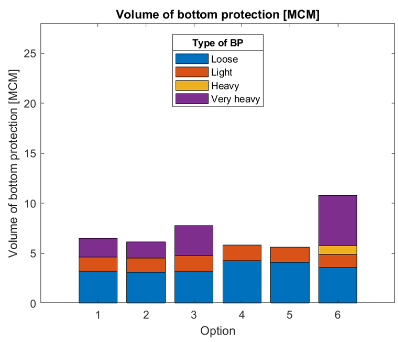


Figure 6.24: bed protection required - 150.000 m^3/day

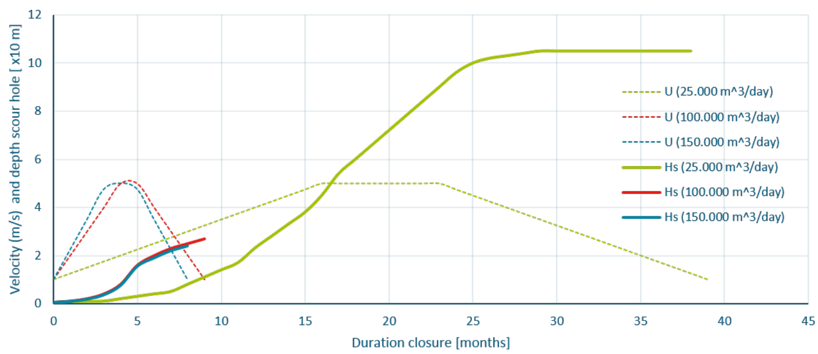


Figure 6.25: Development of a scour hole for three different construction capacities.

Comparison equipment In figures 6.26 to 6.28, the results of the equipment requirements are displayed for three different construction capacities. The results show only differences in the quantity of dump trucks used in options 1, 2 and 3, which decrease with increasing capacity. The explanation for this is that the number of dump trucks that can maximally operate at the same time decreases with increasing size of the trucks. However, they have significantly larger capacities thus require significantly less total operating time to close the dam.

On a separate note, options 1 and 2 require the same length of cable-way. However, in option 2, cross section 7 was also closed horizontally instead of vertically in option 1. If the closure sequence is analyzed correctly, cross section 7 was closed partially vertical in option 2. However, this was already above CD + 2 m, thus it was executed with a cable-way as well. The idea of saving cost on the cable-way in that section with a horizontal closure has failed. For the next run, it is recommended to fully close this section horizontally. This might have an large impact on the final cost.

Figure 6.26: Equipment required - 25.000 m^3/day

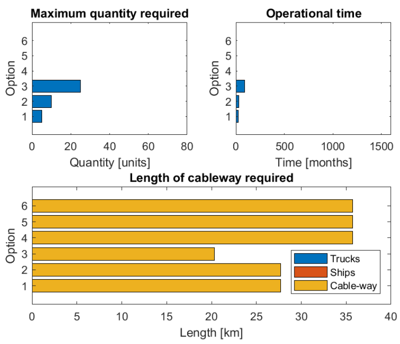


Figure 6.27: Equipment required - 100.000 m^3/day

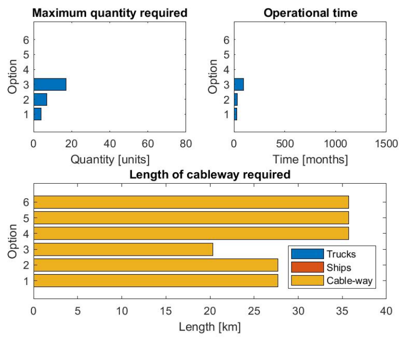


Figure 6.28: Equipment required - 150.000 m^3/day

Comparison final evaluation In figures 6.29 to 6.30, the results of the final evaluation are displayed for three different construction capacities. The results are counter intuitive, since costs increase with increase in equipment capacity. The assumption of a linear increase in cost with respect to capacity should therefore be re-evaluated. In reality equipment costs decrease respectively with increasing capacity.

Three conclusions can be drawn from these results:

- The cost of bed protection decreases with increasing capacity, but initial requirements were already small.
- The cost of equipment has three-folded due to the extra cost of the high capacity cable-way/bridge system.
- The total cost of the case with a construction capacity of 100.000 m^3/day exceeds the total cost generated by the 25.000 m^3/day. Rendering the case with 150.000 m^3/day out of range of the optimal strategy. The optimum balance between equipment and bed protection cost is somewhere between the 25.000 and the 100.000 m^3/day capacities or even below that.

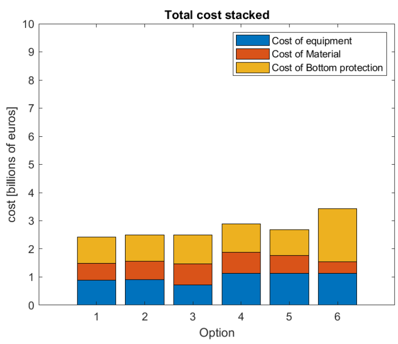


Figure 6.29: Final evaluation - 25.000 m^3/day

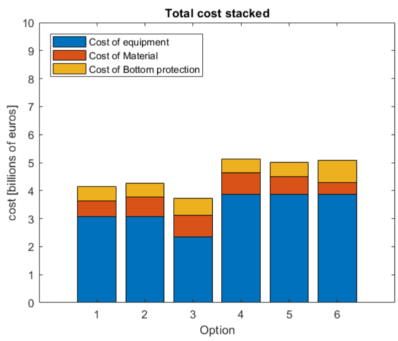


Figure 6.30: Final evaluation - 100.000 m^3/day

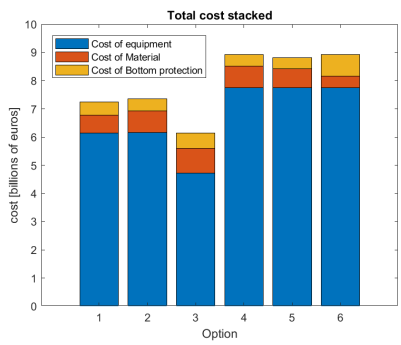


Figure 6.31: Final evaluation - 150.000 m^3/day

6.4.4. Discussion

Deviation in stone sizes The stone sizes in this case study are significantly smaller than expected. The reasons for this can be explained in twofold:

- The failure probability of a rock layer is set at 1/100 per layer, which is considered large.
- The assumed standard deviation of all parametric distributions are large. In combination with the Latin Hypercube Sampling technique, this resuls in wide distributions with numbers from all places in the distribution (even very improbable numbers). These wide distributions have a negative effect on the iteration process to a failure probability of 1/100. In this case, to get to this failure probability only 10 out of 1000 have to fail after which the iteration to the design D50 stops. These 10 failures are reached very early if the distributions are wide and contain numbers of the complete spectrum. Technically, the result is still correct and a design D50 is reached has a failure rate of 1/100 with the given distributions. However, decreasing the standard deviations and increasing the number of runs is recommended for a better result. The Latin Hypercube sampling is still recommended to decrease computational time. However, further research should be done on how this technique should be applied correctly in the model.

Flow velocity and artificial calibration contraction factor The maximum flow velocities occurring are in all cases is 5 m/s, while the peak velocities in case study 1 with the vertical closure where about 6 m/s. This deviation should not be created by the artificial contraction factor. The reason that the factor might not be valid in this case is that a variety of combined closure methods have been tested and the factor was calibrated using only a fully horizontal and a fully vertical case. A second reason, low flow velocities are measured is that the tidal boundary condition from the Delft3D model is used, instead of the four tidal constituents. The tidal range in each section varies to some extent, which could cause lower flow velocities. However, this would be more realistic, because the initial tidal boundary condition based on the constituents used overestimated the water levels significantly in some places along the dam.

6.4.5. Conclusions and recommendations

To conclude the case study, the main research question is stated again:

“What is the effect of a change in construction capacity on the cost for dam material and bed protection with respect to the equipment cost?”

The following conclusion are drawn for the influence of the construction capacity on the individual cost factor for this case:

- An increase in construction capacity decreases the bed protection cost, as expected, showing that the times scale of the development of the scour hole is in the same range as the time scale of the execution. However, by increasing the construction capacity to exceptionally large values, the quantity of bed protection required only decreases slowly, since no significant construction time can relatively be won by increasing the capacity of the equipment.
- An increase in construction capacity increased the quantity of material required, which is opposite of what was expected. Cost savings on this cost factor are significant if an overproduction or expensive gabions are required of which the use can be minimized by smaller stone requirements. In all strategic closure options, this was not the case. However, the stone sized required did decrease with increasing capacity, proving that the model performed as was intended. The extra cost was generated by the extra material required for the crest width within horizontal closure strategies (due to large trucks which need to drive over the crest).
- An increase in construction capacity increased the cost of equipment significantly. An increase in cost can definitely be concluded. However, nothing can be concluded about the quantitatively, since most of the cost were assumed and could vary significantly.

Based on these conclusions, the trade-off part of the question can be answered: The most important trade-off is bed protection vs. equipment costs. A reduction in material cost does not take place in this case, but could still have an important role in cases where large overproductions are required. The optimal construction capacity in this case can be found by evaluating more construction capacities between or below 25.000 and 100.000 m^3/day. If the cost functions and the elementary costs are defined correctly, it can be stated that an optimal capacity for any case can be found using the optimization model in a first-order evaluation of the design.

The secondary question was constructed to assist in answering the main question, to increase knowledge about the strategic optimization process and to validate the model performance. The question is stated here again:

"What strategic design choices significantly influence the costs factors and the total costs of a strategy?"

After this case study, it can be concluded that the construction capacity has a large influence on the total cost introducing a trade-of between equipment and bed protection cost. Furthermore, the strategic design choices that were made in this case study were defined on the basis of the first case, which implicates that the second loop in the optimization process is utilized. Variations in total costs between strategies are smaller and therefore the final evaluation is less reliable. However, in this case, all three strategies with a partial horizontal closure over the tidal flats show the least costs, proving that in this case with any given construction capacity, this strategy is always the cheapest. Within the case with 25.000 m^3 capacity, it can be concluded that option 1 is the least costly and balances the cost of bed protection, materials and equipment. Option 3, has potential to be the cheapest if the capacity is increased. However, to execute this option a temporary work island needs to be installed on the middle tidal flat to perform the horizontal closure. The extra cost for this should be researched in more depth to conclude if this option is potentially the cheapest. With this statement the boundary of the optimization model is defined. Further optimization will bring up extra costs for specific elements that are not included in this model. Since the variations in the least cost strategies are small, these can significantly influence the optimal strategy resulting in an unreliable evaluation.

6.5. Overall conclusion cases studies

In this section, a summary is presented of the most important conclusions that were drawn by the evaluation of the aforementioned case studies. Within the two cases studies, two unique questions were presented. Furthermore, for both cases a more general question was defined to increase knowledge about the strategic optimization process, validate the model performance and evaluate the second research questions of this thesis. On these three questions, the most important conclusions are presented:

“1: Can the optimal closing strategy (least cost option) significantly change if more model cross sections are used in the first design phase.”

Yes, the optimal strategy can significantly change if more cross sections are used to evaluate a closure case. The confirmation of this answer is based on the combined results of all cases within case study 1. While cases 1 and 3 (fully horizontal respectively vertical) couldn’t directly confirm the the question to be true, because the relative variations between the least cost options were too small, cases 2 and 4 (partially horizontal, tidal flats first, respectively partially vertical, channels first) could confirm it. They showed a significant difference (>10%) in the least cost option for multi cross sectional models with respect to a single cross sectional model. Especially in case study 2, the 20% horizontal and 80 % vertical was in all multi cross-sectional models the least cost option, while a fully vertical closure was cheapest in the single cross-sectional model. The most logical explanation for this is that more strategic options are available to close the total cross section and in cases 2 and 4, these unique options are used to benefit the least cost option with respect to a single cross sectional model.

“2: What is the effect of a change in construction capacity on the cost for dam material and bed protection with respect to the equipment cost?”

the trade-off part of the question can be answered: An increase in construction capacity decreases the bed protection cost, as expected, showing that the times scale of the development of the scour hole is in the same range as the time scale of the execution. Therefore, the most important trade-off is bed protection vs. equipment costs. A reduction in material cost does not take place in these cases, it even increases due to the increased width of the crest required for horizontal closures. However, this cost factor could still have an important role in the trade-off in cases where large overproductions are required, because the stone sizes did decrease by increasing the capacity. This proved despite the opposite results that the model performed as was intended. If the cost functions and the elementary costs are defined correctly, it can be stated that an optimal capacity for any case can be found using the optimization model in a first-order evaluation of the design.

"3: What strategic design choices significantly influence the costs factors and the total costs of a strategy?"

This conclusion is presented in two parts for both case studies: The first case study presents the first iteration optimization process and the conclusions drawn are very general and not directly related to the Kalpasar closure case.

- During a horizontal closure, more bed protection is required. By increasing the percentage of vertical closure, the quantity of bed protection required decreases significantly.
- The equipment cost increases significantly with increasing percentage of vertical closure and increases significantly when a cable-way/bridge system is required to execute the closure.
- The material cost are *relatively* constant during all closure strategies and is therefore less relevant in the choice for a strategic closure.
- Material costs increase with increasing percentage of horizontal closure, due to the increased need larger stones which can't be delivered by the quarry. This has to be compensated by more expensive gabions. When closing fully vertically, the cost for material significantly decrease due to the decreased design width of the dam crest.
- The maximum cost increase if a channel is present in the wet cross section due to increasing cost of bed protection and material needs.
- Small percentages of initial partial vertical closures (20-40%) over the full width are not recommended since they close off the flats increasing velocities in the channel. The rest of the channel has to be closed horizontally, which requires much time, since the cross section of the dam is large in a channel. The reduction of the flow velocity through dispersion after the initially placed dam segment is still too small to significantly reduce the flow velocity, resulting in large amount of bed protection required.
- An initial partial horizontal closure of the tidal flats can decrease bed protection cost significantly and is therefore recommended to used

The second case study presents the second iteration of the process and does use the exact conditions of the Kalpasar case. Strategic closure options were designed based on the conclusions from the first case study. The conclusions drawn from this case study are therefore more related to specifically the construction capacity and the level of optimization that can be reached with the strategic choices included in the model. The following conclusions were drawn with respect to the third question:

- The construction capacity has a large influence on the total cost introducing a trade-of between equipment and bed protection cost.
- The least cost option did not change by increasing the construction capacity. e.g. closing the the flats first horizontally with dump trucks, followed by a vertical closure executed by a cable-way bridge system.
- Within this strategy, two alternatives are possible with respect to the least cost option: Closing more tidal flats horizontally and closing less tidal flats horizontally in the first phases (options 1-3). These variations had almost equal total costs. To further define the cost variation between these three alternatives, more detailed design aspects have to be included into the model concluding that a third optimization loop would not present reliable results anymore. With this statement, the maximum level of the optimization that can be achieved with this model is defined.

7

Discussion and Reflection

In this chapter, the research is discussed and reflected by first analyzing the reliability, validity and usability of the optimization model for the closure of the Gulf of Khambhat (section 7.1). Secondly, the complete research is reflected on the usability within the engineering research field of closure works in section7.2. Finally, the optimization model as a product and the results of this research are put into perspective by discussing the contribution to the end goal of Royal HaskoningDHV to revive the Kalpasar project (section 7.3).

7.1. Overall reliability, validity and usability model for the Kalpasar case

This section reflects on the reliability, validity and usability of the developed optimization model for the closure of the Gulf of Khambhat. Due to many assumptions and subsequent computations, certain uncertainties are introduced. The most relevant uncertainties are discussed in this section and secondly, relevant design choices, tools and functions implemented in the model are elaborated to establish the boundaries and design approach for the overall model usability. The reliability and validity of all model elements are individually discussed focusing qualitatively on the effect of case related parameters on the optimal strategy. In subsection 7.1.1, the flow model and its validation is discussed. Subsections 7.1.2, 7.1.3 and 7.1.4 reflect on the underlying dam material, bed protection and equipment models. The evaluation is discussed in subsection 7.1.5. Secondly, overall usability of the model, user boundary conditions and design approach for the use of the model for the Kalpasar case are discussed in subsection 7.1.6.

7.1.1. Flow model

The flow model is based on the well-known storage approach (0D) which was compared to a 2DH Delft3D model in chapter 5 to attempt validation. However, a significant overestimation of the flow velocity at the start of the closure was determined due to incorrect assumptions of the model. Therefore, special focus was put on calibrating the maximum monthly flow velocities with Delft3D, since these velocities largely influence subsequent computations. Calibration was performed by introducing an error correction factor μ_c (defined by its physical representation as an artificial contraction factor) to account for large initial deviations of the storage model. However, the calibrated 0D storage model presented unrealistically small peak flow velocities for a combined closure case, questioning the reliability of the artificial contraction factor. Although overestimated initial flow velocities are correctly reduced with the artificial contraction factor, the relevant peak flow velocities during a combined closure are incorrectly reduced because they occur in a less constricted situation where the artificial contraction factor is still small («1). On a more general note, the artificial contraction factor is based on the assumption that Delft3D models the flow correctly in all situations (also highly constricted), which should be questioned since it was only validated for water levels and flow velocities in unconstricted situations.

Independent of the artificial contraction factor, other model characteristics and parameter definitions introduce uncertainties in defining the flow velocities. As was elaborately discussed in section 4.3, the choice of tidal boundary condition can slightly overestimate or underestimate actual flow velocities depending on the quantity of constituents used (causing monthly and yearly fluctuations). Furthermore, the bathymetry used only roughly approximates the shape of the actual bathymetry causing overestimation of the velocity on the tidal flats. Thirdly, the linearized storage relation overestimates flow velocities slightly, but is not significant. Lastly, a large time step increases the head difference between time steps (increasing flow velocities) and therefore must be decreased with respect to the rate of change of the tidal water level.

At the end of this research, the optimal strategy for the Kalpasar case study resulted in a combined closure, for which the artificial contraction factor is not reliable enough to use. However with a combined closure, deviations are expected to be smaller, so the model can be used without the artificial contraction coefficient for a re-analysis of the second case study. The model will be overestimating velocities at the start (less relevant), but correctly describing them at the end of the closure sequence (relevant). Further model reliability depends on the reliability of the bathymetric, storage and tidal data and size of the time step. If the aforementioned deviations are known to the designer and short sensitivity analyses is performed on the these relevant parameters, the flow model can be reliably used for a first-order evaluation of closure strategies for the Kalpasar case.

7.1.2. Material model

The probabilistic rock model determines the rock sizes with high accuracy by designing for the specific load situation within the specific load duration, resulting in smaller design rock sizes than a regular conservative deterministic approach. However, this design process is so specific that deviations in the construction sequence directly result in incorrect design rock sizes. For a first-order evaluation and comparison of closure strategies these differences are irrelevant. The largest uncertainty in the model is the direct link between the design stone size and the accepted failure probability where failure is defined as some initial movement of stones. The actual damage on system level is still uncertain. A complete risk analysis should be performed for a more reliable relation between small scale and large scale failure to determine a reliable accepted failure probability. Furthermore, periodic storm surges due to typhoons or other types of storms are not included into the probabilistic model, since these are irrelevant for first order evaluations. Storms last relatively short compared to the construction time, therefore this will hardly influence the overall rock sizes or bottom protection requirements, because only a small portion of the total dam material is exposed to the surge and the time scale for scour hole development is also larger. For a more detailed planning of the closure, risks due to storms should be included in the design. The second largest uncertainty is the amount of runs in the Monte Carlo simulation. This number should be sufficiently large to generate reliable results. Lastly, usability of the model is increased because the type of material can be altered to other types of rock or even concrete blocks. However, a sand closure is lacking in the model. This should be included if updates for the model are executed, since this might offer cheaper solutions to close the tidal flats.

The caisson model can compute the locations where a sluice caisson should be implemented with a large accuracy. That is, where the flow velocity crosses the predefined threshold value. The caisson size required is proportional to the depth and length required, therefore accurately representing the actual size and resulting costs. The execution capacity of sudden closures can be altered to fit any type of caisson increasing usability.

To conclude, with a correct choice of statistical distributions of all parameters in the Shields equations, a large number of runs for the Monte Carlo simulation and a conservative accepted failure probability, the rock model generates accurate and reliable design material sizes for a large range of rock types or even concrete blocks. However, because a sand closure is not yet included, the model can't evaluate accurately for low velocity closures. All material considered in the model is based on the earlier Kalpasar case research which increases model usability within the case, but decreases usability as a general purpose model. A validation process for this model seems unnecessary for a first-order evaluation. The results of the caisson model are accurate and can be used for first-order evaluations of sudden closures with any type of sluice caissons.

7.1.3. bed protection model

The probabilistic bed protection model suffers from several inaccuracies which decrease reliability and usability. These inaccuracies and uncertainties and their influence are all qualitatively discussed in section 4.5.5. Relevant inaccuracies are the influence of scale problems on the Breusers equation which overestimates requirements, the rectangle approximation of the bathymetry which underestimates requirements, the assumption of a homogeneous soil layer which overestimates requirements and the averaging process per execution stage which underestimates requirements. The effect on the order of the requirements is however small. This was concluded during the case studies in chapter 6. The time scale of the development of the scour hole and the maximum flow velocity are the largest influences on the order of the requirements and these parameters are (in this stage of the model) assumed to be correctly defined by the flow model.

The largest uncertainty in the model is again the direct link between the bed protection requirements and the accepted failure probability where failure is defined as a collapse of a dam section. The actual damage on system level is still uncertain. Again, a complete risk analysis should be performed for a more reliable relation between small scale and large scale failure to determine a reliable accepted failure probability.

To conclude, a first-order evaluation of the required quantity of bed protection can be performed to some level of detail, keeping possible deviations due to the above-mentioned processes of the model in mind. The model performance increases with more execution stages, less difference in bathymetry and the existence of a homogeneous soil layer.

7.1.4. Equipment model

The equipment model is still simplistic. The used equipment (trucks, ships and cable-way) form the basis of the closure methods and therefore it gives a reasonable first impression of the differences between the execution of vertical and horizontal closures. More detailed information is not required for a first-order evaluation. Furthermore, different types of trucks, ships and cable-way/bridge systems can be implemented in the model, increasing usability. All equipment considered in the model is based on the earlier Kalpasar case research which increases model usability within the case, but decreases usability as a general purpose model.

Although the model can identify basic equipment requirements, costs functions for each of these systems with the capacity as a variable are required for accurate evaluation. The largest uncertainty in this model is therefore the capacity with respect to the cost of the equipment. The second largest practical uncertainty is the required area for stockpiling material and the quarry capacity. Both are assumed to be unlimited in the model. However, these factors can both be implemented with relative ease if more data about the quarry and transportation options is known.

With equipment, the risks are often large or even unknown. In the model, risk of equipment failure has not been implemented. For a first-order evaluation, a qualitative risk assessment of equipment types can be included for a more accurate evaluation of the equipment required.

7.1.5. Evaluation

The reliability of the evaluation is the the most significant point of discussion. At this stage in the complete process, it is complex to track through the optimization model which of the above-mentioned processes in all models influence the evaluation the most . The cost functions used in the evaluation are simplistic and logical (quantity to cost) and can therefore be used accurately to translate raw output to cost. However, variations of the elementary cost still have an enormous impact on the end result, since they are at the end of the calculation. They determine the lowest cost option almost independently of the computational results. These should therefore be researched well before using the model. Other computations are less relevant, since difference in output are relatively small. As mentioned in subsection 7.1.4, the most important relation to define is equipment capacity with respect to cost.

If elementary costs and equipment cost-capacity functions are accurately known, the evaluation of the model is very reliable and can therefore be used for a first-order evaluation of the Kalpasar case.

The evaluation is performed on basic costs. However, it can be argued that value is also a relevant evaluation factor. In this research, value hasn't be taken up in the evaluation because of the complexity of quantification. An example of added value would be that in the case of a vertical closure with a temporary bridge, the bridge could be constructed permenantly to avoid the need for an extra train or highway lane on the dam itself. Other values could be minimization of CO_2 output by the bridge or cable-way systems, since these systems run on electricity. Together with aspects such as required temporay works should be implemented in the evaluation if a more detailed optimization is required.

7.1.6. Overall reliability and usability

The choices for types of material, bed protection and equipment are based on earlier studies for this cases study, increasing relevance and usability for this case study. However, this also indicates that not all types are considered and therefore reliability of the resulting optimal strategy decreases. More research should be performed on alternative materials, bed protection or equipment after which these can be implemented in the model.

The largest uncertainty in the complete optimization model is the definition of the elementary costs and assumed relation between equipment capacity and cost for several types of equipment. These uncertainties are at the end of the computation and are therefore most influential. Secondly, further validation and calibration of the flow model is required before accurate evaluations can be performed. All subsequent computations have uncertainties which depend on the flow model or are relatively small compared to the uncertainties generated by both errors in the elementary costs and the flow model.

Evaluating on relative differences between strategies is the main element of the model which increases reliability. However, some aspects of the computation method generate extra differences between strategies due to inaccuracy of the computation, most of these have been discussed in the previous subsections. To account for all these deviations, the model user should be well-informed of these inaccuracies in the model. Therefore, for better reliability and usability, the model generates an evaluation poster which can be used by the user. It shows all input parameters, costs, quantitative output and final evaluation. Usability is also increased by the parametric approach of the model which combined with the evaluation poster, generates a reliable overview of the strategic closure options and on which case related parameters the results are based. The user can therefore make a well-informed decision by identifying up to which level of optimization the model is still accurate.

Although, the model is largely coded in MATLAB, the readability and adaptability of the code has been taken into account from the start. Furthermore, a short manual is introduced with descriptions of all different code-files. Lastly, the formulas used in the model are basic and are often known to any hydraulic engineer. All these aspects of the model decrease the "black box" user experience and increase usability and therefore the reliability of the final decision.

7.2. Reflection usability research within the general field of closure works

Although all above-mentioned arguments suggest the model is designed for the Kalpasar case study, it can be argued that the parametric optimization model can be used for general-purpose research as well. In this section, first a short summary is presented to what extent current literature can draw general conclusions about closure strategies after which the position of this research is described with respect to this literature. Secondly, an idea is presented to improve and specify further general applicable research by utilizing the optimization model introduced in this research. This idea is further substantiated by some suggestions regarding the value and usability of the proposed research.

To start, the current approach for first-order evaluation of a closure strategy is best described in a report by Konter: "Afsluitdammen, regels voor ontwerp" published in 1992 (Konter et al., 1997). This study combines most laboratory research and practical lessons learned from several closure projects in the the Netherlands (closure of the southern sea and the Deltaworks) into a design guide for closure works. The computational models (flow, rock and bed protection) in the optimization model are designed based on this guide. However, the guide only describes design details for a single cross section closure and does not compare strategies. The rock manual (CETMEF et al., 2007) describes a comparison between three types of closure strategies (horizontal, vertical and combined) for one cross section and draws simplistic relations with design rocks sizes, qualitative bed protection requirements and some practical equipment options. Furthermore, it describes detailed design information concerning specific closure types such as rock, sand and caisson closures, but does not directly relate these to basin characteristics. The book "Breakwaters and closure dams" by K. d'Agremond (K. d'Angremond, 2004) describes systematic considerations to strategically close a multi-cross sectional gap (consisting of tidal flats and channels). However, only general considerations could be concluded such as: "If the channels are closed first, more bed protection is required across the whole gap" or " If the tidal flats are close first, a complex caisson closure might be required for the channels". However, the boundary condition for when the extra bed protection weighted up against the caisson closure could not generally be given due to the fact that this is always case specific.

This research pioneers in the field of first-order evaluations of closure strategies by combining quantitative costs of material, bed protection and equipment and relate them to a specific closing strategy for a large scale multi cross-sectional gap. A mentioned before, in research these relations have always been qualitatively described for only a single cross section. Furthermore, this research can also be used for smaller scale closures for which the model would increase in reliability due to increasing reliability of the flow model. However, smaller scale closures often have only one channel for which general considerations for an optimal strategy already exist as was mentioned above. The added value of this research is that a more specific strategy can be designed for closure gaps with more than one channel and therefore, the probability that the optimal closure strategy is within the evaluated options is larger.

To conclude, general purpose conclusions with respect to an optimal closure strategy are minimalistic due to many influential case related parameters of the basin. Furthermore, general boundary conditions for which a certain strategy is optimal to close multi cross-sectional closure are not described within literature. For the purpose of improving scientific knowledge about closure strategies, it would be interesting to know a certain closure strategy is always preferred within certain boundaries. For example, an interesting conclusion could be: "Horizontally closing the tidal flats first and vertically closing the channels second is always optimal if the dam length is more than 5 km, the tidal range is more than 5 m and the basin size is more than 200 km^2". In reality, a definitive scientific conclusion is very complex to draw due to the vast amount of influential parameters. However, this research presents the first indications that such conclusions might be possible to draw if the optimization model is utilized with more case studies.

After utilizing the optimization model for several case studies (chapter 6), this research could not generate definitive conclusions regarding a certain strategy to be optimal within certain boundary conditions. A suggestion would be to start a large scale research project and involve case studies for many basins in the world which could use a closure dam. This minimizes the required boundary conditions by linking them to a specific location, thereby removing the need for general applicability worldwide. This would be inconvenient, since there is only a relatively small amount of large tidal basins in the world that could benefit from a closure dam.

In this age, closing off tidal basins is not a preferred method anymore to deal with problems such as flooding, river control or fresh water storage due to the environmental impact. However, due to many researches executed on the negative effects of closure dams, these effects can possibly be prevailed in the future. With this in mind, closing off tidal basins can offer a large scale solution to many aforementioned deltaic problems in the world. In that case, large scale research on closing strategies should be performed to minimize governmental capital investment costs to make these integral solutions more attractive with respect to the often preferred cheaper local solutions.

7.3. Contribution to goal of interdisciplinary team RHDHV

In this section, the contribution of this research to the interdiscplinary team of RHDHV is described. First, the overall goal of the team is introduced. Secondly the results of this research are compared with this goal to define the precise contribution. Thirdly, the possible integration of other team results in this research and into a more complete business case is discussed. To conclude, several steps to come to a complete business case are proposed.

In 2017, Royal HaskoningDHV decided to initiate a cooperation with three Dutch universities (Delft University of Technology, University of Amsterdam and University of Maastricht) to set up an interdisciplinary research team to revive the Kalpasar project with new insights and techniques. The objective of the team was to develop a business case which can generate attention to the project by identifying opportunities and threats and attract investors. This thesis is part of the interdisciplinary research with the overall goal to decrease capital cost by optimizing the closure works design.

The results of this thesis fits partially within the goal of RHDHV, but lacks to present a detailed closure works design or new idea/technique to perform the closure with decreased cost. What it does present is a generally applicable tool to perform a first-order optimization of a closure strategy to use in further designs of the dam, should the project be tendered. In this way, flexibility is incorporated should the design or design conditions change when the project is tendered.

Furthermore, because of the parametric design, the results of the other team members can be implemented in the model. René Kersten studied the hydraulic and morphological impact of the dam on the outside of the basin (Kersten, 2018). Especially the increase in tidal range due to tidal amplification was an interesting conclusion. Both during and after the dam is constructed amplification will occur. These findings can be implemented in the model by increasing the tidal range during the closure and by increasing the design dam height to account for the increased High Water level. Also a lower Low water level occurs. This should be included in the operability of the dumping ships.

Dhruv Rajeev studied the optional tidal power facility to generate revenues for the project (Rajeev, 2018). He determined the quantity of turbines and the placement location. A tidal power facility can bring a relieve to the final closure by using the closing the area of the turbines with flaps. These flaps are present in the design and can therefore pose as a sudden closure in the optimization model.

To complete the business case, the results of these two researches can be incorporated after which a more detailed optimization of the closure strategy can be determined. The results of the optimization and the general applicable tool as a product can be integrated with the team results to form business case. As was mentioned in the previous section, attracting investors for this project can be done by minimizing cost, generate more revenue and minimize unknown risks. Combining the suggested measures by René Kersten to avoid large scale morphologic and hydraulic risks, implementing an optimized quantity of turbines into the dam to generate revenues and presenting a least cost closure strategy which decreases capital cost, should present a well-founded and interesting basis for the business case. However, to complete the business case much more research should be performed. until now, much focus has been put on minimizing costs and revenues, but many risks are not yet identified or quantified. A complete risk analyses would be the most interesting aspect to research. On the basis of these results, several studies should be executed to minimize these risks or prevent them with alternative designs. A large part of the team's research is robust enough to be valid for an alternative design (as was the intention), so insignificance of the research should not be a concern.

8

Conclusions and recommendations

In this chapter the conclusions and recommendations of this research are described. Section 8.1 includes the conclusions, followed by the recommendations in section 8.2.

8.1. Conclusions

The conclusions will be based on the evaluation of the research objective and answering the research questions defined in section 1.3.

8.1.1. Research objective

The research objective was formulated in chapter 1 as follows:

"Assisting the Kalpasar Development Project in optimizing a closure strategy and identifying key design considerations, by creating a general and robust tool to perform a first-order evaluation on cost regarding possible closure strategies."

The objective can be split up into two objectives: The "what" objective; assisting in the project and understanding the key design aspects. And the "how" objective; creating a general tool and perform several analyses . By evaluating the "how" objective first, it can be evaluated to what extent the "what" objective is met. Starting with the general tool:

A general robust optimization tool is presented in this thesis which can perform a first-order evaluation for a large variation of closure strategies. All possible strategies to close a multi-sectional wet cross section in time can be evaluated on basic costs and partially on risk related costs. The evaluation is performed based on the three essential cost factors: Bed protection, dam material and equipment costs. These cost factors show the largest variations with varying closure strategies and, if summed up, form the largest part of the total cost of a closure strategy. Risk related costs are partially included in the material and bed protection costs, by relating an accepted probability of failure to the design dimensions of these elements. Decreasing the accepted probability of failure therefore results in an increase of material or bed protection requirements. The tool can optimize by performing several design runs after which adjustment to the strategy can be implemented based on the key strategy choices of the least cost option.

The tool is developed for the Kalpasar case and can operate under varying conditions in the Gulf of Khambhat. A change in dam location would even be possible to implement. However, the tool is limited by the types of material and equipment that can be used; namely rocks, gabions and sluice caissons which are placed by dump trucks, ships or a cable-way/bridge system. The choices with respect to material and equipment types are based on the Kalpasar case, enabling general applicability in the Gulf of Khambhat. However, it can also perform as a general optimization model for larger rock or concrete block closures in other basins in the world.

The tool was successfully applied for two cases studies, in which the influence of several key strategic design aspects such as type of closure method, construction capacity and phasing were related to the cost factors. The tool can be used to quickly eliminate costly closure strategies to assure no strategy is overlook and show an optimal (least cost) option up to a general reliable level of detail. The reliable level of detail which the optimization can reach is case dependent. For the Kalpasar case, the reliable level of optimization resulted from the second case study, resulting in an advice to first close the tidal flats horizontally, while closing the channels vertically after which the remaining cross section is closed vertically. From this point, the model can assist in defining the most important considerations for further optimization. In the Kalpasar case, this would be either to use a cable-way/bridge system to close the final gap, or use sluice caissons. a second consideration is if it's worth it to construct a temporary work island on the middle tidal flat to perform a horizontal closure on the tidal flat or if the cable-way/bridge system can perform this cheaper vertically. The general tool and the resulting optimal closing strategy including the most important design considerations can be delivered to the Kalpasar development project.

8.1.2. Research questions

In order to meet the objective, several research questions were defined. The answers to these questions are summarized in the evaluation of the research objective, but are elaborated in more detail in this section:

1. *"How to develop a **robust method** to **optimize** the closure strategy?"* With the answers to the following subquestion, this research question is considered sufficiently answererd.

   (a) *What are important cost factors that should be included in a first order evaluation of a closure strategy? and which physical processes should be included to quantify these cost factors?*

   To evaluate a closure strategy on costs, a first order insight in the basic costs and risk related costs of a closure strategy is required. The cost of bed protection, dam material and equipment are established as the most influential cost factors with respect to varying closure strategies. Not only do they vary significantly with varying closure strategies, if summed up, they form the largest part of the total cost of a specific strategy. To quantitatively evaluate the strategies on these cost factors, the flow velocity during the strategy is the most important physical process to include. To compute the bed protection required, the time dependent development of a scour hole is the second most important. Thirdly, the stability formula's derived by Shield are important to analyze the stone size required during the closure. The last aspect is knowing equipment costs and operational capabilities. Combined, the cost factors and these three processes from the basis of a model to evaluate a strategy.

   (b) *Can all cost factors be quantified using these processes and (how) can they be combined into a reliable overall first-order evaluation of a closure strategy?*

   With the use of Matlab, several models were built to combine these processes to quantify the cost factors and form a final evaluation on costs of several closure strategies. The flow velocity is computed using a storage model with multiple cross sections to approximate the bathymetry of the closure gap. This is a 0d model and therefore performed unreliably in comparison with a 2D-Hydrodynamic Delft3D model. A calibration coefficient was added to the storage model by calibrating with the Delft3D model, which can be used only for the Kalpasar case. This restored the order of the flow velocities to a more realistic value. On the basis of the flow model, two probabilistic models were created utilizing the scour processes and the shields equations to evaluate the required quantity of bed protection and dam material respectively. The accuracy of these model was improved by linking the design dimensions to a predefined accepted failure probability of the related part of the dam. The equipment required is modeled using basic known equipment with corresponding characteristics such as capacity, operation speed and size. All models are influenced by the predefined execution scheme (strategy) which is determined by the designer. The quantitative output is given in volumes of bed protection and material or quantities of equipment required. These are then converted to a total costs of each cost factor utilizing cost functions based on elementary costs per volume, quantity or length unit. The final evaluation largely depends on the elementary cost input but the quantitative data can be analyzed reliably. Therefore an evaluation poster where all information is displayed regarding the closure options is utilized to assist the designer in making a reliable choice.

(c) *For each process: How do different strategic input parameters affect that process and its related cost factor?*

The strategic input parameters defined are the type of closure per cross section: horizontal, vertical, sudden or any combination of these three. The capacity with which each section is closed and the phase in which each section is closed. The cost are affected by these choices. From the case studies, it can be concluded that most bed protection is used when closed horizontally and the most equipment is utilized with a vertical closure. The material costs is affected in lesser quantities by the strategy choice. During a horizontal closure, the stone sizes become larger which in some cases can't be delivered by the quarry. In that case, more expensive gabions are used increasing the total cost. During vertical closures, stone sizes are considerably smaller. A caisson closure increases material cost drastically, since caisson are expensive. In case of a multi-sectional model, a combined closure (using multiple phases and varying closing methods) can utilize the unique closing options in different phases by reducing the quantity of bed protection and equipment required significantly.

The construction capacity influences the quantity of bed protection by utilizing the process for the development of the scour hole. An increase in construction capacity decreases the execution time which limits scour hole development, decreasing the bed protection required. Secondly, Smaller stone sizes per layer can be used by limiting exposure time to extreme conditions. The trade-off that is created is the cost of equipment which increases with increasing capacity.

(d) *How do the different cost factors affect the overall evaluation?*

The influence of the strategic input parameters on the cost factors is described in the previous question. In this question, the ratio between cost factors in the overall evaluation is discussed. The final evaluation varies significantly by changing the elementary cost parameters which define for a large the total cost of the cost factor. In this research, these costs are assumed to be able to perform case studies. For the remainder of this questions, the elementary cost parameters are not discussed anymore since they do not contribute to overall evaluation with respect to the strategy. They are only relevant in defining the least cost option (optimization). The variations of the cost factors is interesting for different closure strategies. The bed protection quantities vary significantly and can have a dominant role in defining the least cost option. However, if larger capacities are utilized the variation in equipment cost can increase to a level they have the most dominant role in defining the least cost option. The material cost factor has the least influence on the overall evaluation. Larger influences of this cost factor are noticed if sudden caisson closures are utilized or quarries can't deliver the rock sizes required for the closure. All these trade-offs are case dependent and can be optimized for a single case by changing the strategic closure parameters stated in question 1c. By basing the changes on key strategy choices from the least cost option of the first evaluation, the ratio's between the three cost factors can be optimized.

2. *"Which **design considerations** within the Khambhat closure case are relevant for **future optimization of the closure strategy** to increase project feasibility?"*

To answer the question, the resulting optimal closure methods needs to be explained first; On the basis of the case studies performed and the assumed parameters, the resulting optimal strategy is to close the tidal flats horizontally with dump trucks, while at the same time, closing the channels vertically with a cable-way/bridge system to 40% of their initial depth. from this point, the remaining channels are filled up in steps starting from the deepest point to the shallowest removing all channels to create a flat sill levels across all channels. The last step is to evenly close the remaining gap vertically with the cable-way/bridge system. This is performed with a horizontal closure capacity of 12.500 m^3/day per bund and a vertical closing capacity of 25.000 m^3/day.

An important design consideration for the Khambhat closure in the future is to evaluate if a cable-way system or temporary bridge is less costly to install than performing a caisson closure in the final gap. Secondly, constructing a temporary work island on the large tidal flat in the middle of the basin to perform a horizontal closure of this tidal flat coud be les costly than installing a cable-way/bridge system to close vertically. Furthermore, an optimization can be performed with respect to construction capacity used. A significant quantity of bed protection can be saved by increasing the construction capacity. Large high capacity cable-way systems could be installed with relative ease considering the heavy lifting equipment which is developed to install offshore wind turbines.

8.2. Recommendations

In this section, the five most important general recommendations regarding improvement of the optimization model and further research with respect to the closure of the Gulf of Khambhat are presented with decreasing priority. Specific topic related recommendations can be found the corresponding chapters.

With regard to the closure of the Gulf of Khambhat and the optimization model

1. Perform more research using the optimization model on Kalpasar closure case. Several important aspects have been included into the model, but were not tested. The following three questions are considered the most important with respect to utilizing the model and its validation:

    - What is the optimal strategy if a sudden caisson closure is required above a threshold value of the flow velocity.
    - What is in the influence of the permeability of the dam on the flow velocities? Does this affect the optimal strategy?
    - What is the optimal strategy if the last wet cross section can be closed by a tidal energy power plant and a spillway?

With regard to the closure of the Gulf of Khambhat

2. Perform research into cost functions of several existing or new equipment types with respect to their construction capacity. A fast closure is beneficial for the total quantity of bed protection required and might also decrease material costs. If general cost functions for several existing or new construction techniques are known, the optimal strategy can be determined by utilizing the optimization model.

3. Perform research into new equipment for the Gulf of Khambhat closure. A focus should be laid on a technique which can close vertical with a high capacity over a long length. Risks regarding the use of this newly designed equipment must be analyzed in detail, because flow velocities in the Gulf of Khambhat can be very high while closing. For this reason, designing high capacity equipment for a vertical closure is recommended, as well, This reduces operation time and flow velocities.

With regard to the optimization model

4. Perform detailed research to improve the calibration factor for the flow velocity in the flow model. The deviations in the flow model are large due to negligence of spatial variations (0D model). With use of Delft3D, a successful relation was found between the constriction percentage and the deviation in maximum flow velocity between the two models. However, during several case studies with combined closures ending with a vertical closure, the flow model results showed unreliably low velocities. The reason for this is that with vertical closures, the deviation of the flow velocity due to the negligence of spatial variation is less. This specifically should be researched in more depth by comparing both the storage model and the Delft3D model.

5. Perform research on the consequences of dam instabilities due to failure of the bed protection and rock layers and relate this to the accepted failure probabilities in the optimization model. For the rock model special attention is required for the correlation of failure between the individual layers or extents. Implementing the failure probability-damage relations, complete both probabilistic models to a full risk analysis (level VI reliability analysis). With this information, the influence of the construction capacity on the total cost can be analyzed in more depth. For future strategy optimization, working from a risk based planning is recommended. With this planning, the model can be flexibly utilized to evaluate the optimal strategy with increasing level of detail.

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Nomenclature

Abbreviations

Abbreviation	**Description**
RHDHV	Royal HaskoningDHV
EAG	Expert Advisory Group
TR	Tidal Range

Greek symbols

Symbol		**Description**	**Unit**
α_t	=	Turbulence factor (Breusers)	[-]
α	=	Angle of the dam slopes	[°]
Δ_r	=	Relative density of the quarry stone in water	[-]
Δ_s	=	Relative density of the bed material in water	[-]
μ_c	=	Artificial calibration contraction factor	[-]
ϕ_i	=	Phase lag of the tidal constituent i	[°]
ψ	=	Damage parameter or Shields parameter	[-]
ρ_s	=	Density sand	$[-]$
ρ_r	=	Density rock	$[-]$
τ	=	Resistance parameter	[-]

Roman symbols

Symbol		**Description**	**Unit**
A_b	=	Wet basin area	$[m^2]$
A_{bp}	=	Bed protection area required per section	$[m^2]$
A_c	=	Flow cross section of the channel	$[m^2]$
A_{HAT}	=	Area basin at HAT	$[m^2]$
A_i	=	Amplitude of the tidal constituent i	$[m]$
A_{LAT}	=	Area basin at LAT	$[m^2]$
A_w	=	Wet cross section at high water	$[m^2]$
B	=	Crest width dam	$[m]$
B_n	=	Width of cross section n	$[m]$
C	=	Chézy roughness parameter	$[m^{0.5}/s]$
C'	=	Resistance factor for throughflow	[-]
CT	=	Initial cost of truck	$[euro]$
CTR	=	Cost of truck operation	$[euro/day]$
CS	=	Initial cost of ship	$[euro]$
CSR	=	Cost of ship operation	$[euro/day]$
CC	=	Initial cost of cableway/bridge	$[euro]$
CCL	=	Cost of cableway/bridge	$[euro/m]$
CMR	=	Cost of rocks	$[euro/m^3]$
CMG	=	Cost of gabions	$[euro/m^3]$
$CMCI$	=	Initial cost of caissons	$[euro]$

Symbol		Description	Unit
CMC	=	Cost of caissons	$[euro/m^3]$
CMO	=	Cost of overproduction	$[euro/m^3]$
CBB	=	Cost of bed protection	$[euro/m^3]$
C_{tot}	=	Total capacity required (strategy input)	$[m^3/day]$
C_t	=	Throughflow coefficient	[-]
C_{truck}	=	Capacity of one dumping truck	$[m^3/day]$
C_w	=	Wave Celerity	$[m/s]$
dh_{basin}	=	Difference in basin water level	$[m]$
dt	=	Timestep	$[s]$
$D_{channel}$	=	Depth of channel	$[m]$
$D_{sill,n}$	=	Updated sill level in the cross section	$[CD+m]$
D_n	=	Nominal diameter of the quarry stone	$[m]$
D_{n50}	=	Median nominal diameter of the bed material	$[mm]$
F_{over}	=	Factor of extra production of dam volume	$[-]$
Fr	=	Froude number	$[-]$
F_t, v	=	Correction factor turbulence vertical closure	[-]
F_t, h	=	Correction factor turbulence horizontal closure	[-]
h_0	=	Waterdepth in the basin	$[m]$
h_{basin}	=	Basin level	$[CD+m]$
h_{dam}	=	Dam height	$[m]$
$h_{dam,d}$	=	Dam design height	$[CD+m]$
h_n	=	Basin water level above sill level in cross section n	$[m]$
h_s	=	Depth of the scour hole in time	$[m]$
H_c	=	Height and width of caissons required	$[m]$
H_n	=	Sea water level above sill level in cross section n	$[m]$
H_{sea}	=	Sea level	$[CD+m]$
$h_{sill,n}$	=	Minimal depth above sill level	$[m]$
k_r	=	Roughness height ($2D_n$)	$[m]$
K	=	Correction factor rock design (for Shields)	[-]
K_α	=	Correction factor voor slope dam head	[-]
K_*	=	Correction factor for the influence of turbulence	[-]
l	=	Length of the channel connecting to the basin	$[m]$
L_c	=	Length of caissons required	$[m]$
L_{cb}	=	Length of cable-way/bridge system	$[m]$
L_{bn}	=	Length of bed protection required for phase n	$[m]$
L_{lo}	=	Leftover dam section after partial horizontal closure	$[m]$
L_s	=	Length of the dam section	$[m]$
L_{st}	=	Effective length	$[m]$
L_w	=	Wave length	$[km]$
n	=	Number of cross section	$[-]$
n_v	=	Porosity dump material	[-]
N_{ships}	=	Number of ships required to close a section	$[-]$
N_{trucks}	=	Number of trucks required to close a section	$[-]$
P_f	=	Failure probability	[-]
P_{over}	=	Cumulative overproduction at every stone weigth class	[% for stone classes $1:w$]
P_{yield}	=	Percentage yield larger than corresponding stone weight	[%]
q	=	specific discharge	$[m^2/s]$
$Q_{flowthrough}$	=	Discharge due to throughflow	$[m^3/s]$
Q_n	=	Discharge through cross section n	$[m^3/s]$
Q_t	=	Total discharge through gap	$[m^3/s]$
r	=	Constriction percentage	[-]
t	=	Number of time step	$[-]$
t_{scour}	=	Runtime for the scour process in days	$[days]$
T	=	Duration of construction phase	$[s]$

Symbol		**Description**	**Unit**
T_{drive}	=	Total drive time from and to the bund	$[s]$
T_{day}	=	Operational time in a day	$[s]$
T_i	=	Period of the tidal constituent i	$[s]$
T_{load}	=	Loading time of the truck	$[s]$
T_{M2}	=	Tidal wave period M2 component (= 44700 s)	$[s]$
T_{round}	=	Total time for 1 round trip (1 dump)	$[s]$
T_{unload}	=	Unloading and turn around time	$[s]$
u	=	Flow velocity at the end of the bed protection	$[m/s]$
u_0	=	Soliciting flow velocity on the sill	$[m/s]$
$u_{a,s}$	=	Average relative soliciting flow velocity in a phase	$[m/s]$
U_c	=	Critical flow velocity of rock (Shields)	$[m/s]$
U_n	=	Flow velocity in cross section n	$[m/s]$
U_s	=	Soliciting flow velocity on the sill	$[m/s]$
V_c	=	Volume of caissons required	$[m^3]$
V_{dam}	=	Total dam volume	$[m^3]$
$V_{o,t}$	=	Total extra production required by quarry	$[m^3]$
V_p	=	Volume percentage of rocks (from demand or supply curve)	[% of dam volume]
w	=	weigth class stones (step size 1kg)	$[kg]$
$X1$	=	Calibration constant 1 (initial error with no constriction)	[-]
$X2$	=	Calibration constant 2 (exponential factor of upward slope 1)	[-]
$X3$	=	Calibration constant 3 (exponential factor of upward slope 2)	[-]
$X4$	=	Calibration constant 4 (exponential factor of upward slope 3)	[-]
y_1	=	Point of smallest stone size that requires overproduction	$[kg]$
y_2	=	Point of extra volume required for actual dam construction	[% of dam volume]

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A

Dam design Royal Haskoning 1998

As mentioned before, a preliminary design was delivered to the Kalpasar department by Royal HaskoningDHV in 1998. In this appendix a short description is given of the design elements based on the Prefeasibility Report (Royal Haskoning, 1998a).

First, the project area is discussed in section A.1. Secondly, the final dam aligment is shown and explained in section A.2. In section A.3, the considered closure techniques are elaborated and in section A.4, the dam design is explained. Finally, the costs of the proposed design are presented in section A.5.

A.1. Project area

In this section, a distinction is made between the area directly related to the closure of the Gulf of Khambhat and the area which benefits from the creation of the fresh water reservoir and/or is affected by the works. In this chapter only the closure works are examined and therefore only the area directly related to the closure will be described:

The area consists of the Gulf of Khambhat including the tidal flats. The southern boundary condition was taken at 21° 25' (this is including the mouth of the Narmada River); the northern boundary condition is considered directly south of the Dhadhar River due to the boundary condition that the outflow of the Narmada River has to be stored in the Kalpasar reservoir. Three large rivers flow into the Gulf:

- Narmada River
- Mahi River
- Sabarmati River

The Narmada is by far the largest river of the tributaries to the gulf. Piram Island is located inside this project area on the Western side of the Gulf.

A.1.1. Bathymetry and geotechnical conditions

The bathymetry of the Gulf was established by carrying out a bathymetric survey. The extent of high tidal and low tidal flats was determined by examining satellite images. The compiled bathymetric map is available at the RHDHV office of Rotterdam.

A.1.2. Hydraulic conditions

Tide Data on the astronomical tide for various locations in the area was taken for Admiralty Tide Tables. The project area is renowned for its large tidal range in the order of 8-11 m. Tidal currents in the area range from 2-4 m/s. These currents were verified by one and two dimensional mathematical model computations.

Wind Wind conditions were determined based on ship observation data. The Khambhat area is prone to South-Westerly winds during the monsoon period from May to September. Northerly winds dominate the weather conditions during the rest of the year. The wind speed is on average 0.5 times higher during the monsoon period that outside the monsoon period. In the project area cyclones have been registered. Wind speeds exceeding 33m/s have a recurrence interval of 50 years.

Waves Also the wave conditions were established based on ship observation data. Waves originate from the South-West during the monsoon period and from the North throughout the rest of the year. Waves are approximately two times higher during the monsoon period than outside the monsoon period. The wave period ranges from 6-12 seconds for waves ranging from 1-3 meters.

A.2. Dam alignment

Based on the results of geotechnical and geophysical surveys an analysis has been made on the foundation conditions present in the Gulf of Khambhat. The foundation conditions north of the Narmada river mouth are relatively good for dam construction. The soil conditions south of the Narmada River are influenced by deposits of the river and are less suitable for dam construction.

Very deep channels and trenches exist around Piram Island up to 100 meters deep; probably due to the high flow velocities is the area. This makes this area less suitable for dam construction.

One criterion was to include the discharge of the Narmada River into the basin, since it accounts for about 75% of the fresh water flowing into the Gulf. Therefore a southern alignment was chosen.

It follows from figure A.1 that the dam alignment is to be situation north of Piram Island close to the town of Gogha. The dam should cross the relatively shallow area directly offshore of Gogha in east-west direction (figure A.1, a). The deeper channels east of the shallow area should be crossed perpendicularly until approximately half way the gulf (b). The dam alignments heads in south-easterly direction in order to cross the deep channels west of Dahej perpendicularly while heading for the mouth of the Narmada river (c). The dam alignment heads straight east when the depth contour lines allow it to minimize the total dam length (d). Just south of Luhara point, the dam alignment heads south east to facilitate the inflow of discharges from the Narmada River into the basin (e).

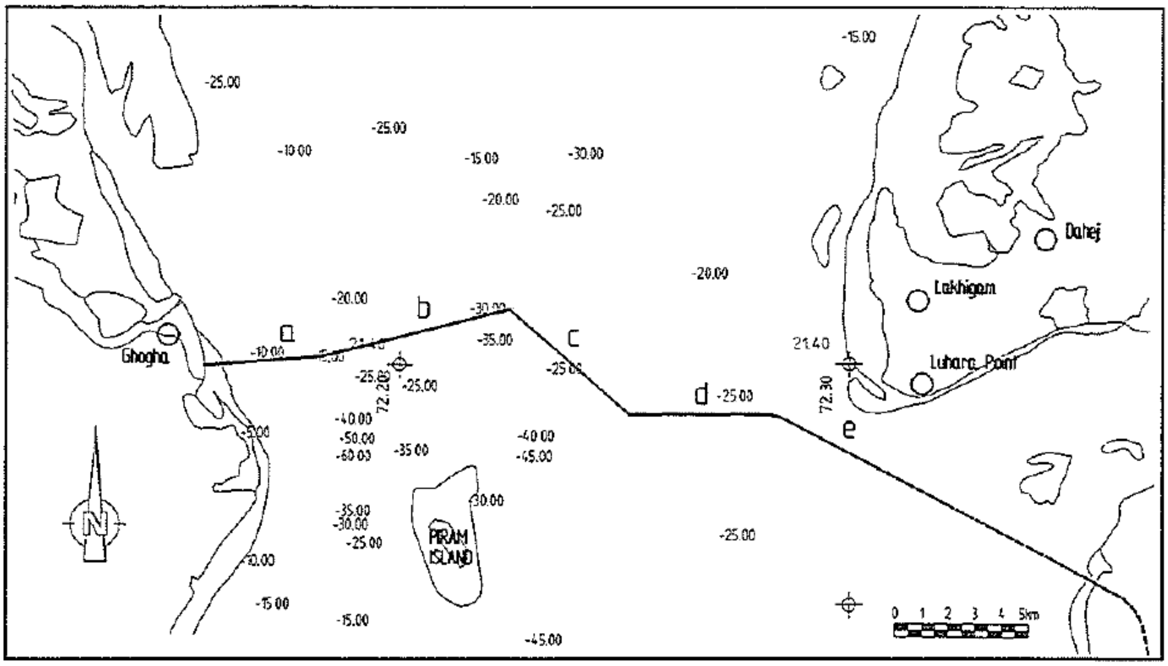


Figure A.1: Dam alignment Royal Haskoning 1998 (Royal Haskoning, 1998a)

A.3. Closure techniques

Three closure methods have been considered in case of the closure of the basin.

- Vertical
- Horizontal
- Sudden

A maximum acceptable flow velocity during closure of 6 m/s was adopted. The rock seizes required for the closure will become unrealistically high if this value is exceeded. Two conclusions were drawn from the result from their hydraulic computations:

- A gradual closure method can be applied to the closure gap until a remaining closure width of 10 km is reached. This will be done by forward dumping of rocks.
- The remaining 10 km of closure gap will have to be closed by means of a sudden closure. This can only be achieved by using open sluice caissons.

The closure of the mouth of the Narmada River is hydraulically speaking independent of the closure of the Gulf of Khambhat due to the construction of a temporary dam from the central Gujarat mainland to the headlands of the Khambhat dam. A spillway will be constructed in this alignment to evacuate flood waters. The first parts of this closure will be done by forward dumping of rocks. A gap will be left until the construction of the spillway has finished. Then the spillway can be used to divert the waters and finalization by forward dumping can be achieved now.

		Main dam body			Caisson	Dam Narmada river mouth	Secondary dam tidal power basin	Spill way	Miscellaneous structures
Current velocity	[m/s]	6.0			6.0	6.0	1.0	6.0	-
Wave attack	[m]	8.6			8.6	8.6	2.5	8.6	-
Water level	[m GTS]	+13.0			+7.0	+13.0	+8.0	+6.0	+13.0
Geotechnical conditions		Bore hole (3)	Bore hole (4)	Bore hole (5)	*see properties main dam body: bore hole (3) and (4)*	Bore hole (7)	Bore hole (9)	*see properties Narmada river mouth: bore hole (7)*	*for tidal power plant, see properties main dam body, bore hole (3)* *for ship lock Bhavnagar side, see properties main dam body, bore hole (3)* *for ship lock Dahej side, see properties main dam body, bore hole (4)*
layer (1)									
- depth	[m GTS]	-30.0 / -33.0	-21.0 / -23.0	-26.0 / -37.0		-27.0 / -39.0	-32.0 / -62.0		
- γ_{wet} / γ_{dry} (specific density)	[kN/m³]	17.0 / 13.7	17.0 / 15.0	18.1 / 14.6		16.7 / 13.1	17.3 / 13.9		
- ϕ (internal friction)	°	32.0	33.0	31.0		31.0	34.0		
- c (cohesion)	kPa	0	0	0		0	0		
layer (2)									
- depth	[m GTS]	-33.0 / -40.0	-23.0 / -34.0	-37.0 / -56.0		-39.0 / -57.0	-		
- γ_{wet} / γ_{dry} (specific density)	[kN/m³]	15.2 / 10.7	16.4 / 11.7	18.1 / 13.5		17.2 / 14.0	-		
- ϕ (internal friction)	°	29.0	30.0	26.0		36.0	-		
- c (cohesion)	kPa	0	0	60		0	-		
layer (3)									
- depth	[m GTS]	-40.0 / -65.0	-34.0 / -50.0	-		-	-		
- γ_{wet} / γ_{dry} (specific density)	[kN/m³]	16.5 / 13.3	17.7 / 14.7	-		-	-		
- ϕ (internal friction)	°	30.0	32.0	-		-	-		
- c (cohesion)	kPa	50	0	-		-	-		
Earthquake - horizontal acceleration	m/s²	0.125			0.125	0.125	0.125	0.125	0.125

Figure A.2: Design parameters for each individual dam section (Royal Haskoning, 1998a)

A.4. Dam design

The dam is designed using the following criteria for the construction phase and the operational phase:

- Stability under current attack (construction phase)
- Stability under wave attack
- Sufficient height against overtopping (operational phase only)
- Geotechnical stability including the effects of scour
- Structural stability

- Stability during earthquakes

Several dam sections can be identified along the dam alignment:

- Headlands on either side of the Gulf
- Dam body closure gap
- Dam body including caisson
- Dam Narmada river mouth.
- Secondary dam for tidal power basin.

The design parameters have been established for each element and can be found in figure A.2. The preliminary design for each element is presented in the following subsections.

A.4.1. Dam body headlands

The dam body of the headlands on either side of the Gulf comprises of two rock fill toes with both inner and outer slopes of 1:3. The outer dam slope has a slope of 1:5 above Mean Sea Level to reduce the wave run up. On the basin side of the dam a berm of 35 m is conceived to accommodate a road.

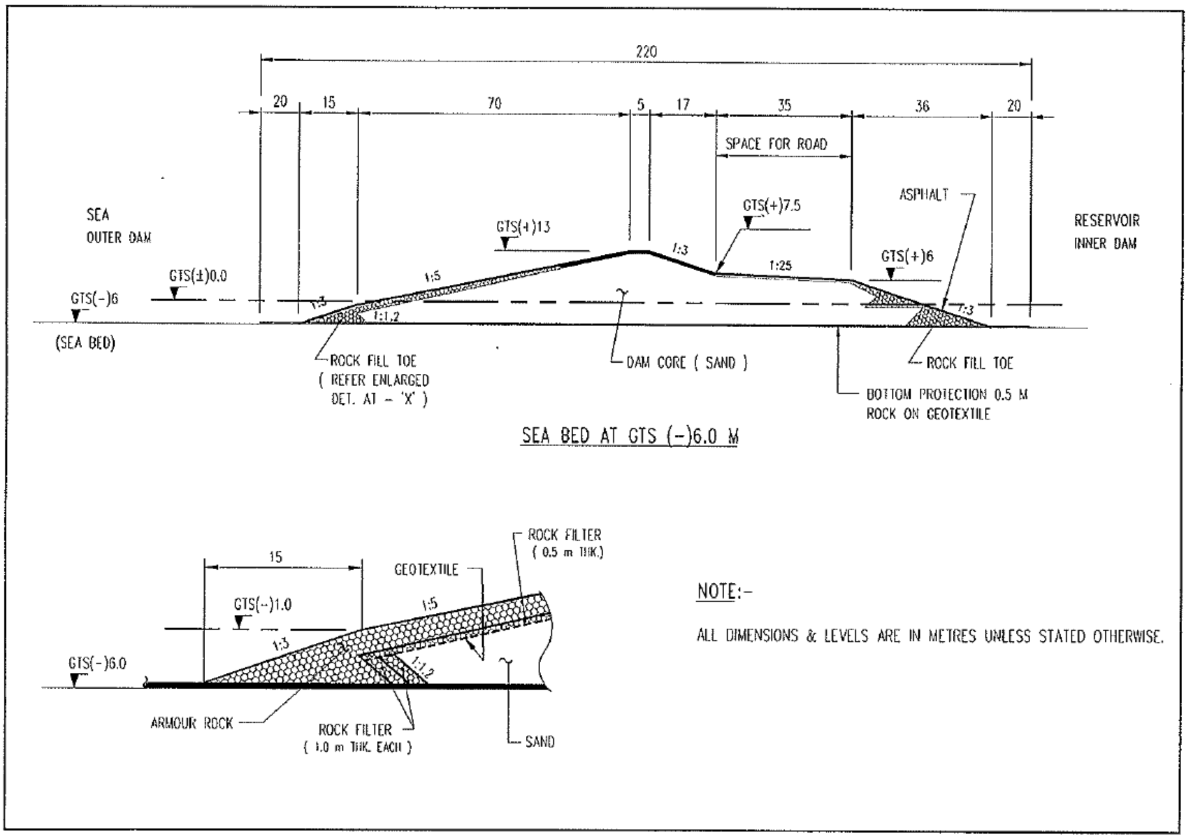


Figure A.3: Headlands (Royal Haskoning, 1998a)

The outer dam slope is protected against wave attack by a geotextile and a double layer of rock up to a level of GTS + 9m (GTS is chart datum). The outer dam above a level of GTS + 9m as well as the inner dam slope from the berm upwards are protected against wave attack and erosion by a layer of asphalt. The crest of the dam is situated at GTS + 13m. See figure A.3.

The dam core comprises of sand. A geotextile in between the dam core and the rock fill toes keeps the sand from washing out. The bottom protection consists of a geotextile covered with 0.5 m of rock in the shallow areas or block matts covered by a layer of 1.0 m rock fill in the deep areas (depths larger than 10 m)

A.4.2. Dam body closure gap

The dam body in the closure gap is the same only higher. The bottom protection consists of a mattress with a thickness of 1.0 m. See figure A.4.

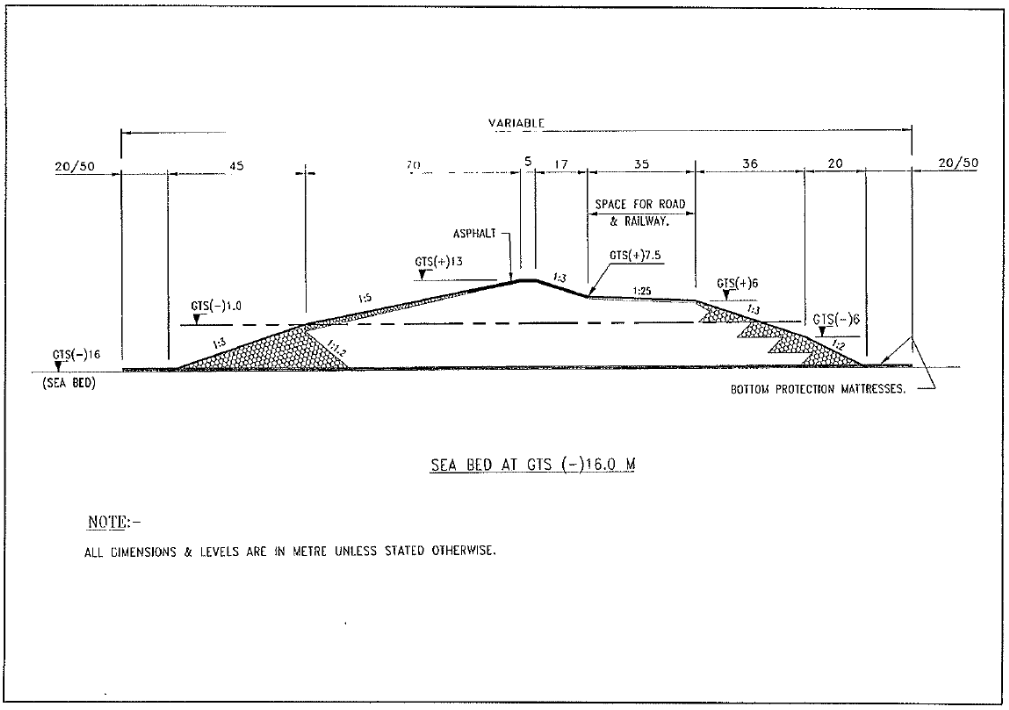


Figure A.4: Cross section dam design closure gap (Royal Haskoning, 1998a)

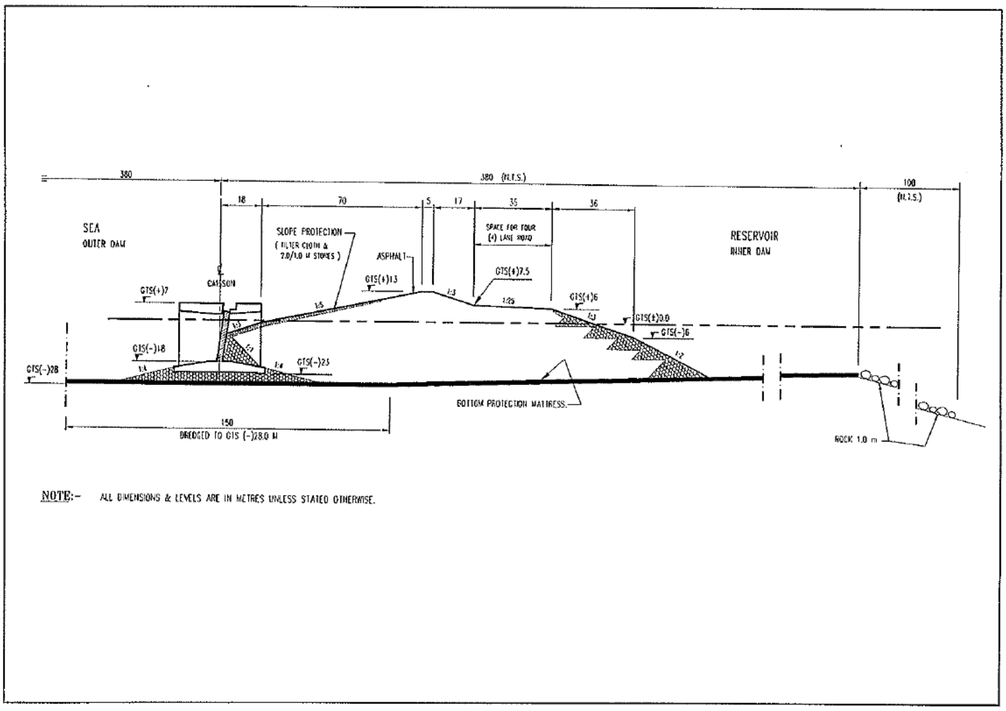


Figure A.5: Cross section dam design closure gap with caisson (Royal Haskoning, 1998a)

A.4.3. Dam body with caissons

The dam body including the caisson is depicted in figure A.5. The caissons will be 108 m long, 36 m wide and 30 m high. See figure A.6. The caissons will be in place during the closure of the rock fill closure gap. The tidal currents will be able to flow freely through the open caissons throughout the execution of the rock fill closure. All caissons are closed simultaneously by means of lowering large steel gates in the openings after finalization of the construction of the rock fill closure. The caissons are incorporated in the dam body itself after the closure has been completed.

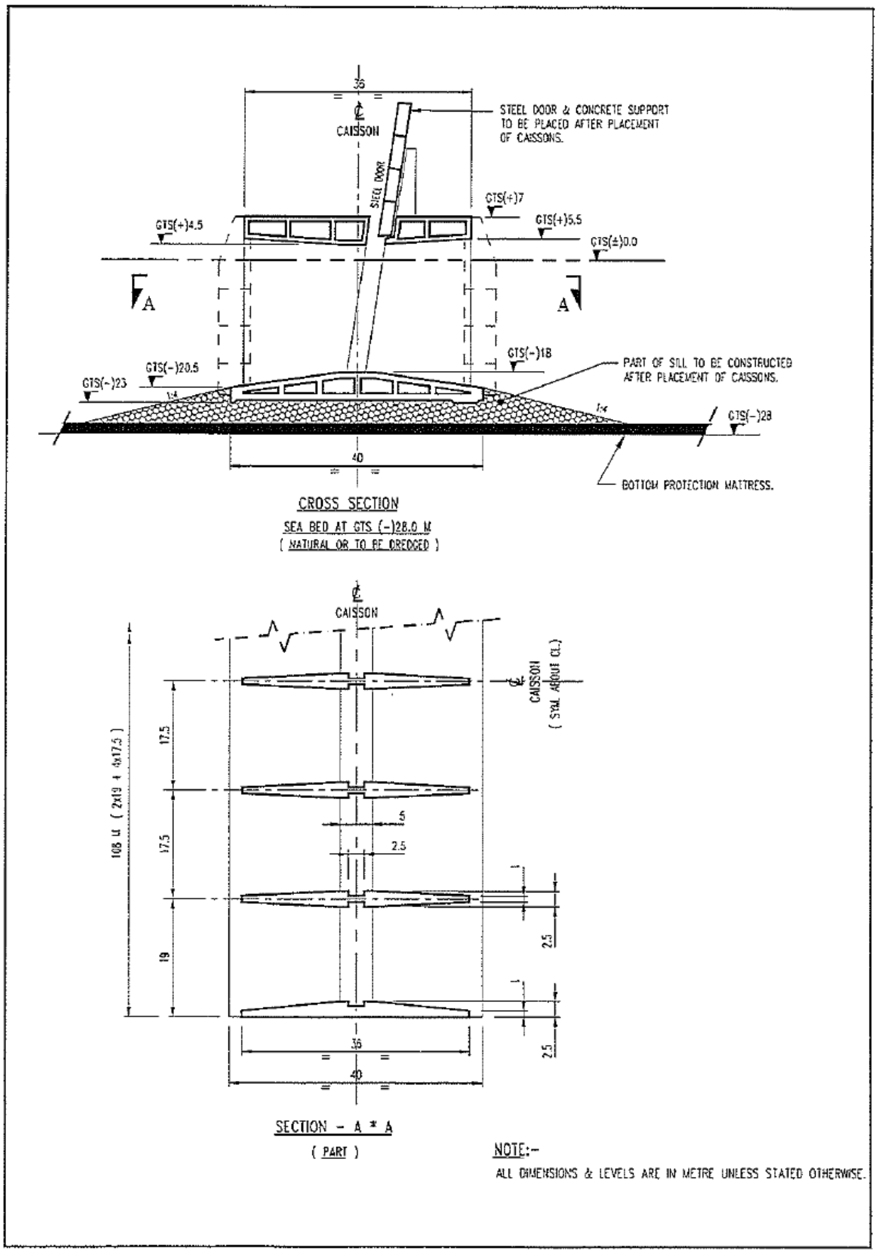


Figure A.6: Cross section dam design closure gap with caisson (Royal Haskoning, 1998a)

The caissons will be in place during the closure of the rock fill closure gap. The tidal currents will be able to flow freely through the open caissons throughout the execution of the rock fill closure. All caissons are closed simultaneously by means of lowering large steel gates in the openings after finalization of the construction of the rock fill closure. The caissons are incorporated in the dam body itself after the closure has been completed.

A.4.4. Dam Narmada river mouth

The dam in the mouth of the Narmada River comprises of a sandy core. The slopes on the seaside are protected from the effects of wave action by a geotextile and a double layer of rock. The slopes on the basin side of the dam are protected from the effects of wave attack by a double layer of rock up to a level of GTS + 5 m; above this level an asphalt layer is applied. See figure A.7.

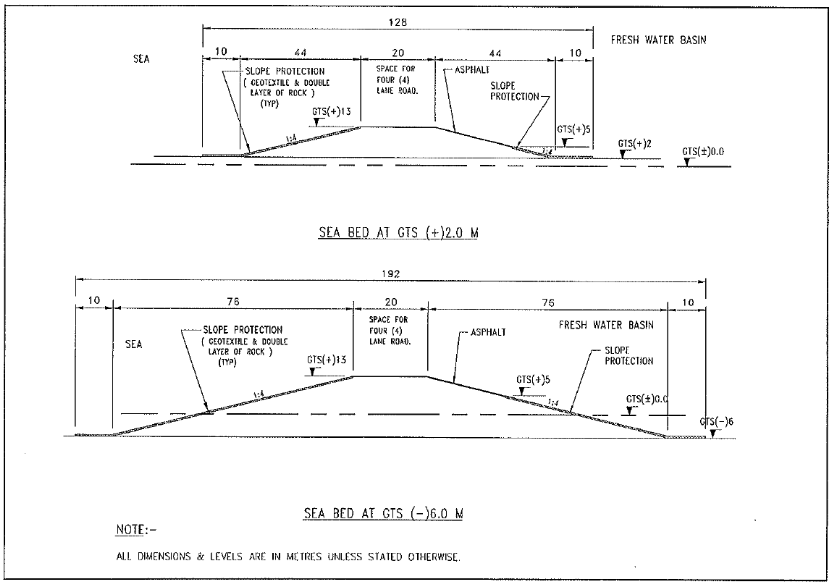


Figure A.7: Cross section of Narmada river dam (Royal Haskoning, 1998a)

A.4.5. Spillway

The discharge capacity in the mouth of the Narmada River has been designed as to allow the safe passage of the Narmada's Probable Maximum Flood (PMF) in combination with 50% of the PMF's of the other rivers entering the reservoir. The spillway comprises of 65 discharge openings with a width of 17 m. See figure A.8

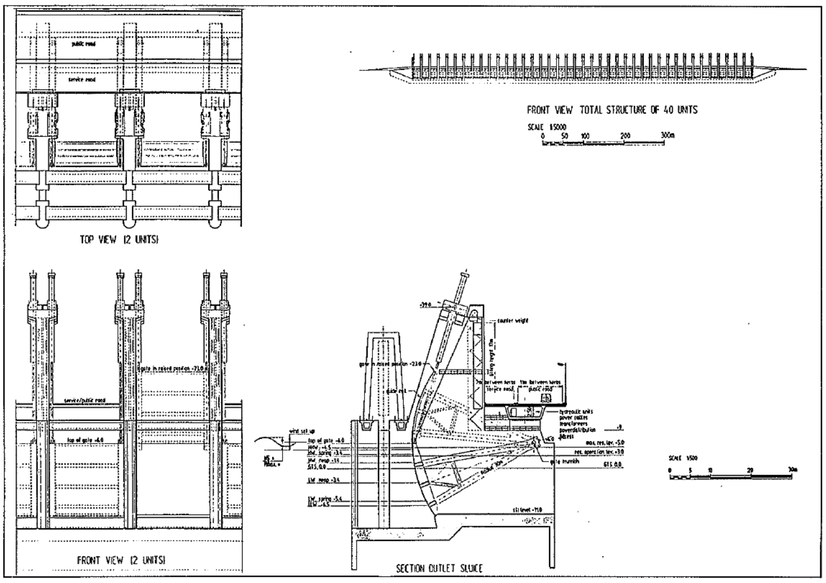


Figure A.8: Preliminary spillway design Royal Haskoning (Royal Haskoning, 1998a)

A.4.6. Shiplock

One ship lock measuring 150 x 25 x 6 m (at low water) has to be incorporated in the dam design to allow for shipping to and from the Arabian Seas. Ships of approximately 20.000 DWT could pass through the ship lock.

A.5. Costs

The total cost of the closure without the tidal energy component (figure A.9) is estimated around 20,000 crore RS (a crore is 10 million). That is 200 billion RS, which is about 2.7 billion euros in 1998 and around 11.5 billion euros in 2017 with an average interest rate of 8% (FRED, 2018) .Including the tidal energy component, the total costs are estimated around 53,000 crore RS, which amounts to about 30 billion euro in 2017. (Royal Haskoning, 1998a)

The final closure was estimated at 7,500 crore RS by Royal HaskoningDHV in 1998 (38% of total closure costs). This includes work harbors, bottom protection and construction pits for caissons. The deepest dam section in the middle of the closure gaps is the second most expensive element at 2,000 crore RS (10%). In third place, the spillway is also an expensive component with 1,070 crore RS (5%) (Royal Haskoning, 1998a)

FRESH WATER BASIN ONLY - INCL. NARMADA RIVER MOUTH					
Dam	Direct [cr RS]	Indirect [cr RS]	Additional [cr RS]	Misc. & unfores-een [cr RS]	Total dam costs [cr RS]
KHAMBHAT DAM					
Dam Gogha	186				
Ship lock 50,000 DWT constr. pit	283				
Caisson closure gap West	3,405				
Island West	144				
Central closure dam	2,006				
Island East	163				
Caisson closure gap East	3,312				
Normal dam	226				
Spillway	1,070				
Canal Narmada - basin	221				
Work harbours Gogha and construction pit caissons West	227				
Work harbours Dahej and construction pit caissons East	603				
Temporary dam Luhara point	44				
Subtotal	11,890	2,948	475	3,078	18,391
NARMADA DAM					
Dam1	8				
Closure gap	218				
Dam 2	241				
Dam Alia Bet	42				
Dam Hansot	46				
Subtotal	555	138	25	144	862
TOTAL					19,253

Figure A.9: Cost overview complete dam design (Royal Haskoning, 1998a)

B

Theoretical review of considerations in closure works

B.1. Introduction closure works

The main purposes of closure works are to close a water course. The closure works are usually temporary works that are specially designed to close of tidal basin or river estuaries. The works may later be part of the closure dam, but the design and execution method is often not based on the final dam design. Closure works need to close the cross sectional flow area available at the location of the dam. The dimensions of closure works are based the amount of flow area that needs to be closed and the tidal amplitude at the location. Larger tidal amplitudes lead to a larger cross sectional area to close, but also to higher flow velocities during flood and ebb in the gap. During a closure of a tidal basin, the changing hydrodynamics may cause the following:

- Change of tide (amplitude, flows) at the seaward side of the dam due to partial reflection
- Increased in flow velocity in the gap.
- Scour of channels and tidal flats in and around the closure gap due to increased flow velocities
- Reduced length of tidal window for shipping through the gap.
- Possible reduction in navigable depth in channels
- Large sediment depositions at either side of the dam due to sedimentation of eroded sediment

B.2. Location and alignment

Closure work designs are very site specific. For each location the circumstances are largely varying and that is why detailed recommendations on closure works hardly exist. For example, to close a long closure dam, a large amount of material is required, the price and availability of the material will define the final closure works design. Other parameters might not be so important any more.

The location of a closure dam in a tidal basin is based on several aspects:

- Purpose of the dam to be constructed
- Hydraulic conditions during and after the construction of the dam.
- Water management conditions after the dam has been completed
- Environmental aspects

These four aspects will generally decide the boundaries for the location of the dam. The specific dam alignment can be further optimized by including this choice in the design of the closure works and dam design. This specific dam alignment depend the following aspects:

- The configuration of the bed in situ
- The composition of the bed
- The connection with the shores
- The closure method

The final dam alignment will be inside the defined boundaries, therefore minimizing the influence on the first set of location determining aspects. However, in some cases the closure works might be the dominant factor for the choice of the general location when other criteria are less important. An integral assessment of site characteristics is therefore required before deciding on the location and the final alignment (Huis in 't Veld, 1987).

B.2.1. The configuration of the bed

The position of the channels and flats are important. In determining the final dam alignment, the bathymetry of these channels and flats should be utilized optimally. Some criteria are: (Huis in 't Veld, 1987).

- Deepest parts of channels should be avoided for both economic and technical reasons.
- The alignment should cross the channels perpendicularly to have short crossing dams but also to have the dam perpendicularly to the stream lines of the channel of both incoming and outgoing tide.
- Alignments near confluences or divisions of tidal channels should be avoided to be able to predict the flow more accurate during construction.

B.2.2. The composition of the bed

The composition of the bed is an important aspect. The main focus lies on the bearing capacity of the bed. The structures inside the dam (locks and spillways) should be stable along with the dam itself. If the soil is not sufficient, soil improvements may offer a solution, which implies dredging a trench, supplying new material and/or compact it to increase the bearing capacity (Huis in 't Veld, 1987).

B.2.3. The connection with the shore

The connection of the dam with existing infrastructure is also a problem. Current infrastructure such as roadway and train tracks might be positioned far away from the shore connections of the optimal dam alignment. The extra costs to connect the dam to existing infrastructure should then be taken into account (Huis in 't Veld, 1987).

B.2.4. The closure mehtod

A certain bathymetry is preferred for each type of closure; with a gradual closure, wide gaps with shallow parts are preferred to minimize filling material needed. When doing a sand closure, the final gap must be shallow, since this goes faster and less sand is lost. In case of a sudden closure, a deep channel is preferred, since this minimizes the amount of placements (Huis in 't Veld, 1987).

B.3. Types of closure methods

A main distinction in a type closure dam can be made according to the construction method. Each method is directly related to the equipment and materials used. Therefore the preferred method is chosen for which the material or equipment required have the least costs and risks.

B.3.1. Gradual closure

In this case, flow velocities are acceptable to use relatively small sized materials which are deposited at the place of construction until the flow is completely blocked. Construction of the closure dam takes longer than one tidal cycle so the occurring flow in the gap will be at maximum tidal velocity during construction. In some cases, if it takes more than two weeks, even spring tidal velocities will occur. There are three main construction types to execute a gradual closure (K. d'Angremond, 2004)

1. Horizontal closure; closing the dam from one or both sides narrowing the closure gap
2. Vertical closure; stacking layers of material from the bottom upwards to decrease the depth in the closure gap until the dam closes of completely

3. Sand closure; depositing large quantities of sand in the gap until the gap is closed

The main distinction is made between a horizontal (using land based equipment) and a vertical closure (using waterborne equipment). A sand closure can be used if peak flow velocities are low. A part of this sand will be lost during the closure operation. However, the high costs of bed protection are saved (veld, 1987). A combination of these methods is often used and is called a combined closure.

B.3.2. Sudden closure

A gradual closure is in some impossible to close the dam completely. Occurring flow velocities in the final gap are too large to deposit locally available materials. It can also occur that the required material size is too large to deposit due to insufficient material transport or dumping capacity. In this case, a sudden closure is required. Such a closure method involves that a large part of the final gap in the dam is closed within a certain period of time to avoid the occurrence of high flow velocities. This can mean either within one tidal cycle or within two spring tides. Although, if it's done within two spring tides it is often still called a gradual closure. A sudden closure can be performed by using pre-installed flap gates, sliding gates using sluice caissons or placement of a blocking caisson or vessel (K. d'Angremond, 2004).

B.3.3. Phasing of closures

Another type of distinction in closure methods is based on the bathymetry at the location of the dam. The tidal basins usually have channels and tidal flats and therefore the closure of a tidal basin is usually performed in several phases. The type of closure method used is different for both of these locations because the flow conditions are defined differently. These closure methods are defined as follows (figure B.1):

- Tidal gully closure: The closure of a channel in which high flow velocities occur. This channel is therefore eroded to a large depth.
- Tidal flat closure: The closure of a tidal flat which is generally very shallow or dry at low water. The critical flow velocity will occur at certain characteristic tidal water levels.
- Simultaneous closures: horizontal or vertical over the full length of the cross section

One of the main challenges is to assess a closure phasing that is convenient and cheap to execute. The options have different execution alternatives during the closure.

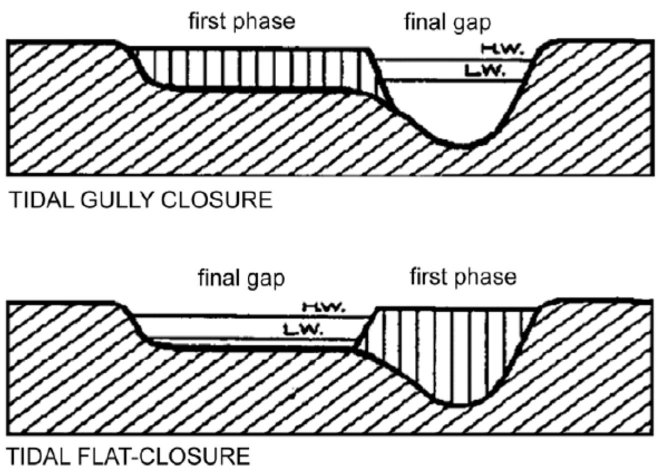


Figure B.1: Difference between tidal gully and tidal flat closure (K. d'Angremond, 2004)

Closing the flats first In this case the flats are closed of first and the final closure will take place in the channels. The flats can be closed with relative ease, since flow velocities will not increase drastically. The advantage in this is that the amount of bed protection required is minimized over the full length of the flats. In the channels, bed protection should be implemented to prevent local scour, because flow velocities have increased. There are three options to close the channels:

- Horizontal closure by tipping over of stones
- Vertical closure by sinking stones
- Sudden closure with help of caissons or sluice caissons

A vertical closure is unpractical and expensive, since the width of the remaining channels is often small. Vertical closures require a certain execution methods that are very costly and are only cheaper if the length of the gap is large. For example, installing a temporary bridge or cableway from which stones can be dropped or hiring large ships that dump stones is very expensive if the closure gap is very small.
For each channel it has to be calculated if regular forward dumping of stones is a viable option or if a caisson closure is necessary. Another variable is phasing, in other words, if the channels are closed simultaneously or subsequently. With subsequent closures, the velocity in the last gap will be very large. The channel does not have the capacity to handle the extra discharge, so the channel will erode severely due to the imbalance. Closing simultaneously can be an option if the required capacity is available or sluice caissons can be used.
A sudden closure of the last channel can be an option. However, the flow velocities during slack water should be small enough to have a significant amount of time to place them. If this is not the case sluice caissons can be used to avoid flow velocity increase with each caisson placement (K. d'Angremond, 2004).

Closing the channels first Closing the channels first can be advantageous. However, the first concern is that after closing the channels, the tidal flats will scour and protection against this has to be placed over the full length of the flats. Also if the channels are closed first the remaining gaps are shallow, wide and have presumed horizontal bottom profile. Such a profile is unnatural and a channel will soon be formed if no protection is applied. These extra expenses should be compensated by the reduced cost of the channel closures. An example of such a saving is the use of regular caisson instead of sluice caissons.
The rest of the flow cross section is shallow and can be closed with a gradual closure either horizontal, vertical or a combined closure. However, a vertical closure using dumping vessels or a cableway often can't compete with a dump trucks used for a horizontal closure (K. d'Angremond, 2004).

Closure over the full dam length Another option is to close over the full dam length with a horizontal, vertical or combined closure. Usually, this type of closure is done by first raising the sill level in the flats and channels up to one level and finishing either horizontally or vertically. For this a bed protection is required along the full length of the dam (K. d'Angremond, 2004).

B.4. Construction methods

In this section the conventional closure methods are discussed:

Horizontal closure A horizontal closure can be accomplished by the forward dumping of rubble, gravel, sand, clay, sand bags, concrete blocks or gabions (nets with rubble) over the head sections of the dam. This can be done with

- Manual labor,
- Trucks
- Cranes

Vertical closure A vertical closure can be accomplished by dumping rocky material (gravel, stones, concrete blocks or gabions), clay or sandbags on the sill of the dam. This can be done with

- Manual labor
- Trucks/trains that ride a temporary bridge
- Floating equipment such as stone dumping vessels or floating cranes
- Temporary cableway
- Helicopters

Sudden closures Placing caissons can be done in the following ways:

- Preparing the caissons in a dry construction dock, floating them to the required location and sinking them at slack water on a pre-installed sill.
- Hoist them in the water with use of high capacity floating cranes.

B.5. Groundwater flow

Damming watercourses will influence the groundwater pressures in the vicinity of the dam and in the dam itself. These changes may be short-term variations in relation to the daily tide levels, or long term adaptations, such as lining up with the rise in mean level of a basin. Differential pressures lead to flow of the groundwater. As long as the Reynolds-number is smaller than about 5, the flow velocity according to Darcy's law is defined by equation D.6

$$u = k \cdot i \tag{B.1}$$

In which k denotes the permeability of the soil and i the pressure head loss per unit length. The flow velocity$u$ has a complex relation to the actual velocity of the water in the pores of the soil. The quantity of water passing through the soil is given by the u multiplied by the full cross sectional area A of the soil, as if there were no soil particles (K. d'Angremond, 2004).

B.6. Geotechnical stability

The most important property of the soil is its bearing capacity, which frequently is a determining condition for the design and a limiting factor for the operations. The soil mechanics problems, related to bearing capacity, are: (K. d'Angremond, 2004).

- Sliding
- Squeeze
- Liquefaction

B.6.1. Sliding

In dam constructions, the instability of the slope at the embankments or at the front of the dam is called sliding. This sliding is caused by the extra weight of the embankment that is put on the soil layer. Important parameters during construction of a dam are:

- The water level change
- Erosion in front of the dam (in the gap)

The water level is important because the weight of the dam material is less when submerged. It might fail at low water. Erosion in front of the dam can be dangerous, since the resistance against sliding is dependent on the amount of soil in front of the embankment. To calculate the risk of sliding, the method of bishop is used (figure B.2). The model schematizes the subsoil and the surcharge layers in vertical slices. The individual slices generate forces which can be described by values of several parameters such as cohesion and angle of internal friction of the different types of subsoil. (Schierkeck, 2012)

The soil parameters are corrected by NEN 6740 andϕ has a multiplier of 1.15 for permanent work and 1.0 for temporary work. Some risk remains during the construction phase. The calculations for the initial and final profiles require different cohesion values because of the soil will gain strength after some initial loading (K. d'Angremond, 2004).
If the subsoil has weak layers, the soil might fail without failure of the dam elements. In this case the total weight of the stable dam material will press the weak subsoil to the sides of the dam, thus lowering itself into the soil. Deformations can occur in all locations (figure B.4). These deformations can be modelled using high end finite element software like Plaxis. A practical choice has to be made if the subsoil is too weak between soil improvement and deliberately induced subsidence(K. d'Angremond, 2004).

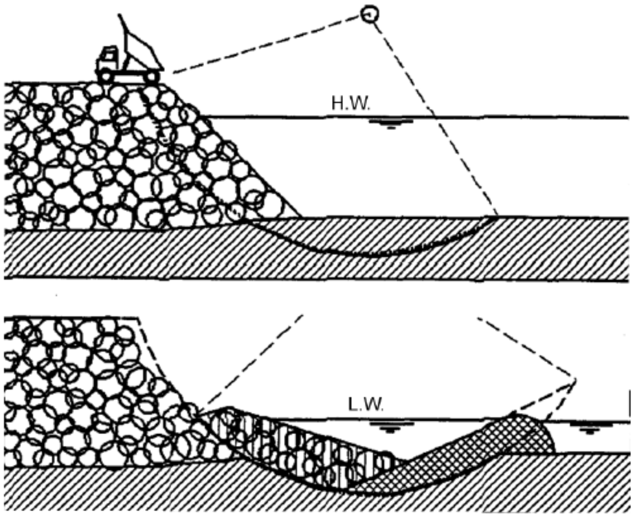


Figure B.2: Sliding circle failure mechanism (K. d'Angremond, 2004)

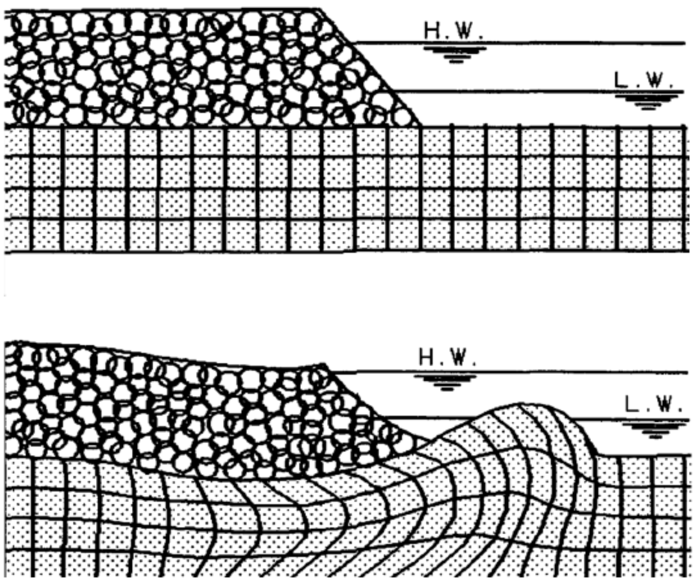


Figure B.3: Squeeze failure mechanism (K. d'Angremond, 2004)

B.6.2. Liquefaction

Liquefaction occurs when the material of the soil is loosely packed. Strength of a soil depends on the forces between the grains. If the soil is saturated, the voids in between the grains are filled with water. The water pressure does not bear the grains. If a force is exerted on the soil, the grains will move into a denser formation. This provides a pressure on the fluid that wants to escape from the voids. If this can't happen because the soil is very fine for example, the water will take over a part of the load that the grains have between each other. The grains lose their stability due to a decrease in contact forces. The soil behaves as a heavy fluid (K. d'Angremond, 2004)

In practice, loosely packed soils only occur in places where settling conditions are ideal. This means low turbulence, waves or current flows. Many failures during closures have been caused by instabilities of the soil. The variations in soil characteristics, limited soil information and unforeseen conditions are main causes for these failures (K. d'Angremond, 2004).

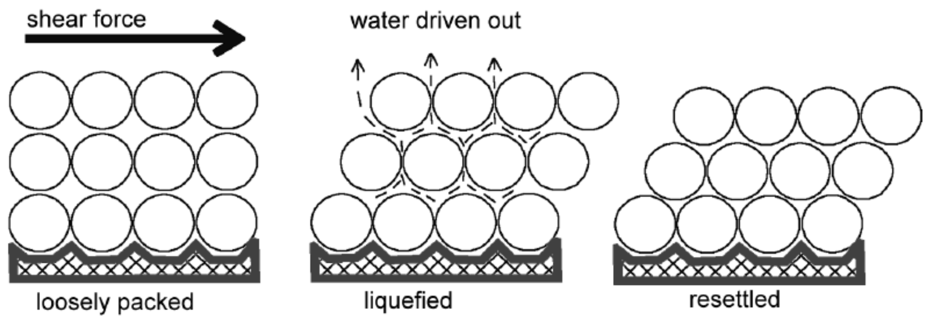


Figure B.4: Liquefaction failure mechanism (K. d'Angremond, 2004)

B.7. Maintenance and repairs

During construction stages, maintenance and repairs are less important. Since the closure of a dam is only a construction stage these aspects will not be discussed further. Repairs can be done on site because the contractor has the equipment available. Maintenance should be taken up in the final design and therefore not in the closure itself. Risks of structural failures or equipment failures should still be taken up (K. d'Angremond, 2004).

B.8. Labour

Taking up labour is essential in choosing the construction method in terms of cost. It is good to know if a skilled local labour force is available or if employment of expatriate labour is allowed. Also accommodation facilities should be available to house workforces (K. d'Angremond, 2004).

B.9. Design approach

Since the design and execution of a closure work is a practical problem, a deterministic approach will not suffice. Information should be available on the frequency of the specific loads occurring. "Transfer" relationships can be used to define the response function of the structure given the loading situation. In this way, damages can be assessed with respect to a given structure dimension (Huis in 't Veld, 1987). The risks of damage, structural failure or equipment failure, delays during execution can also be assessed when a probabilistic approach is used, giving more insight in risks of possible extra costs. This risk assessment can be used to evaluate the minimal total cost.

B.9.1. Summary

The most important practical considerations in closure works have been discussed in the previous section. To summarize: The optimal strategy to close depends for a large part on the location; the bathymetry, tidal amplitude and local available materials. In the Kalpasar case, the location is already set and most of these parameters are already known. To optimize the strategy in the Kalpasar case, the most important consideration becomes the chosen closure method and phasing. For each strategy, the occuring flow velocities during the closure are different . For this reason, the required material sizes, the quantity of equipment and the quantity of bed protection will vary.

C

Flow in tidal basins

This appendix focusses on the flow charachteristics inside tidal basins. A complete derivation of the equations required for the storage model is discussed here.

C.1. Free surface flow

From the balance between iniertia, forcing and resistance the equations of motion for fluids are derived. These equations are known as De Saint-Venant equations. These equations consist of the continuity equation C.1 and the momentum balance equation C.2. These equations are also called the shallow water equations, because it is assumed that the depth is very small with respect to typical length dimensions such as the wave length. (Battjes and Labeur, 2014)

$$B\frac{\partial h}{\partial t} + \frac{\partial Q}{\partial s} = 0 \tag{C.1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial s}(\frac{Q^2}{A_s}) + gA_c\frac{\partial h}{\partial s} + c_f\frac{|Q|Q}{A_cR} = 0 \tag{C.2}$$

Together, the equations form a set of hyperbolic partial differential equations which define the water level (h)

and discharge (Q) as a functions of location (s) and time (t). The geometric parameters A_c, R, B and c_f are required to be known functions of the s, h and Q so that mathemamatically speaking they are known variable coefficients. With a set op initial and boundary conditions these functions can be integrated.

It depends on the circumstances whether all contributions to the momentum balance are important or whether one or two are negligible.

C.2. Small-basin approximation

A large simplification can be made when the flow is unsteady but the domain in which it occurs is relatively small, for example a short basin connected by a restricted opening to the sea with a time-varying surface elevation (tidal motion). Because the basin is closed (except for the opening to the sea) and short, the velocities in the basin are small thus inertia and resistance play no role. It follows from Equation C.2 that the slope of the surface can be neglected inside the basin. The water level in the basin goes up and down, but is almost horizontal at any moment. Thus the surface elevation in the basin is a function of the time only written as $h_b(t)$. The approximation disregard the spatial variations in the basin. However, the disregard is only allowed if the dimensions of the basin (length of the channel system) compared to the typical wave length of the tide in the domain are small. Thus phase differences become negligeable. (Battjes and Labeur, 2014)

Writing A_b for the free surface area inside the basin, the rate of change of the basin water level is defined with respect to the discharge through the opening (Q_{in}, positive for flow into the basin) as equation C.3

The practical application of the equation in the design of closure works is that if the discharge is known, the change in basin level can be computed.

$$Q_{in} = A_b \frac{dh_b}{dt} \tag{C.3}$$

C.3. Discrete models

Certain flow systems such as tidal basins can be schematized with this approximation in terms of seperate but connected basins of finite dimension where either storage or transport occurs. this implies that the motion within the basins does not have a wave-like shape. Such systems are called discrete models.

a distinction can be made in two systems:

- basin with long channel between sea and basin (figure C.1)
- basin with short channel between sea and basin (figure C.2)

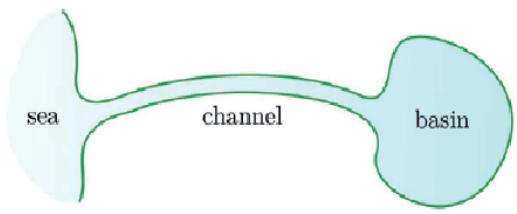


Figure C.1: Basin with long channel

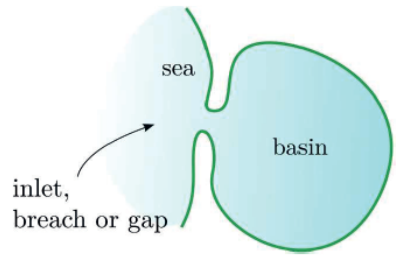


Figure C.2: Basin with short channel

In both cases the channel is small so that it nearly closes the basin. The channels connect the basin to the sea which has a time varying water level. The distinction is important to make because in the basin with a short channel inertia and wall resistance in the channel can be neglected where as they might be relevant in a long channel.

In this type of modelling the only task the basin has is storage and the only task the channel has is transport. In the basin, flow resistance and inertia are neglected, whereas in the case where the basin has a long channel connecting it to the sea, these factors maybe relevant in the channel. The channel generates head loss due to boundary resistance, but storage is not relevant here. Thus, in such discrete model these two functions of transport and storage are separated. (Battjes and Labeur, 2014)
For both systems equations of motions can be derived based on these assumptions. In case of a basin with a long channel the relation which describes the basin level based on the sea level is given by equation C.4

$$\frac{l}{g}\frac{A_b}{A_c}\frac{d^2h_b}{dt^2} + \tau\frac{dh_b}{dt} + h_b = h_s \tag{C.4}$$

In which:

l	=	length of the channel connecting to the basin	$[m]$
A_b	=	wet basin area	$[m^2]$
A_c	=	Flow cross section of the channel	$[m^2]$
τ	=	Resistance parameter	[-]

In which τ is the resistance factor described by equation C.5.

$$\tau = \frac{8}{3\pi}\chi\frac{A_b}{gA_c^2}\hat{Q} \tag{C.5}$$

With

$$\chi = \frac{1}{2} + c_f\frac{l}{R} \tag{C.6}$$

Which reduces to equation C.7

$$\tau \frac{dh_b}{dt} + h_b = h_s \tag{C.7}$$

In this equation, χ reduces to $\frac{1}{2}$

C.4. Practical application as storage basin

The change in water level in the basin defined by equation C.3 assumes a constant wet basin area at all tidal levels. However is is often not the case, since the wet area at Highest Astronomical Tide (HAT) is larger than at the Lowest Astronomical Tide (LAT). With help of the hypsometric curve of a tidal basin, the definition of the wet area at every occuring water level can be seen. However, it is common to use a linear interpolation between the water levels HAT and LAT for first estimations. The wet area of the basin is therefore defined by equation G.2

$$A_b = A_{LAT} + \frac{A_{HAT} - A_{LAT}}{TR} h_b \tag{C.8}$$

In which TR is defined as the maximum astronomical tidal range (equation C.9)

$$\sum_{i=1}^{n} A_i \tag{C.9}$$

The tidal water level component h_s in equation C.3 can also be defined as a superposition of the most common tidal constituents based on their amplitude and their phase which fluctuates around mean seas level (MSL) as a function of time. Often Chart Datum (CD) is taken at LAT to avoid negative sea levels. The outer boundary condition is therefore described by equation C.10

$$H_{sea} = \sum_{i=1}^{n} A_i + \sum_{i=1}^{n} A_i cos\left(\frac{2\pi t}{T_i} + \phi_i\right) \tag{C.10}$$

In which:

$H_s ea$	=	Water level at sea boundary	[CD + m]
A_i	=	Amplitude of the tidal constituent i	[m]
T_i	=	Period of the tidal constituent i	[s]
ϕ_i	=	Phase lag of the tidal constituent i	[°]

To completely define the water level fluctuations, the discharges through the small gap must be computed. Computing the discharges and velocities with high accuracy in the closure gaps is the most important for initial desing of closure works. The flow velocity computed can be rewritten as a discharge which can serve as input for equationC.3. In this way, the flow can can be modelled correctly by using a combination of equations C.3, C.10, G.2. In section C.5, the definition of the flow velocity is discussed.

C.5. Flow over weirs

In this section, the flow over and through weirs and dams is discussed. The relevance with closures is during a closure, several types of flow occur over a weir or dam. In this way, the flow velocity can be defined at any moment during the closure depending on the flow regime occuring.

C.5.1. Flow regimes

The discharge through the gap in case a rock dam is used can be computed using the flow regimes displayed in figure C.3.
For each regime a different stability relation exists. The fourth regime is called through flow and exists only in case a rock dam is used as a closure method. The following parameters are relevant to describe each regime and they are defined by figure C.4

In which:

A flow regime is defined by the Froude number (equation C.11)

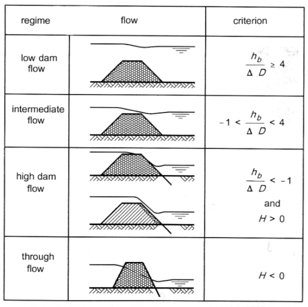


Figure C.3: Flow regimes (CETMEF et al., 2007)

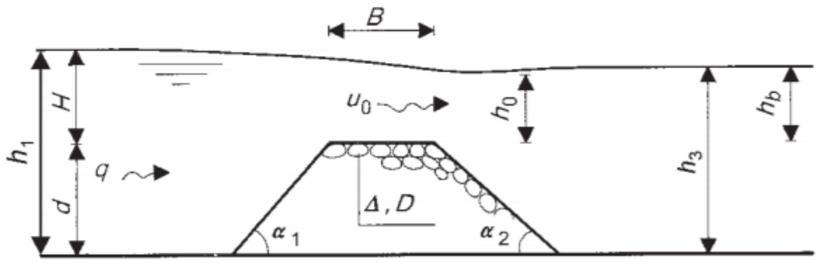


Figure C.4: Definitions for cross sectional dam flow (CETMEF et al., 2007)

h_1	=	Depth outside basin	[m]
h_3	=	Depth inside basin	[m]
H	=	Relative sea level with respect to the sill level	[m]
h_b	=	Tail water depth: basin level with respect to sill height	[m]
h_0	=	Minimum water depth on the sill	[m]
B	=	Crest width dam	[m]
δ	=	Relative buoyant density of the stones	[-]
D	=	Dn50 of the stones used	[mm]
α	=	Angle of the dam slopes	[°]
u_0	=	Velocity on the sill	[m/s]
d_{sill}	=	Sill height with respect to CD	[CD+m]
h_{dam}	=	Dam height	[m]

$$Fr = \frac{U}{\sqrt{gh}} \tag{C.11}$$

The flow over the crest is subcritical when the Froude number is smaller than 1 (Fr < 1) and supercritical when it is larger than 1 (Fr>1). However, this criterion requires a known flow velocity, which is unknown in the calculation. For a more practical approach, the tail water depth hb can be used. The critical tail water depth at which supercritical flow occurs is defined by equation C.12 (CETMEF et al., 2007)

$$h_{cr} = \frac{2}{3}H \tag{C.12}$$

This gives relations for subcritical, supercritical and through flow (only for rock dams)

Subcritical $h_b > \frac{2}{3}H$

Supercritical $h_b < \frac{2}{3}H$

Through flow $H < 0 \quad || \quad (H > 0 \quad \& \quad h_b < 0)$

C.5.2. Practical application of discharge relations

The specific discharge relations can now be derived for each flow regime. These are defined by relations C.13, C.14, C.15,C.16: (CETMEF et al., 2007)

$$q = \mu \cdot h_b \cdot \sqrt{2g \cdot (H - h_b)} \qquad \textbf{For subcritical flow } (H > \frac{2}{3}h) \textbf{ or } (h > \frac{2}{3}H) \tag{C.13}$$

$$q = \mu \cdot \frac{2}{3}H \cdot \sqrt{\frac{2}{3}gH} \qquad \textbf{For supercritical flow } (H < \frac{2}{3}h) \textbf{ or } (h < \frac{2}{3}H) \tag{C.14}$$

$$q = \sqrt{C'}\sqrt{h_3^3 \cdot 2g\left(\left(\frac{h_{dam}}{h3}\right)^3 - 1\right)} \quad \textbf{For throughflow} \quad \text{if} \quad H > 0 \quad \text{and} \quad hb < 0 \quad \text{(high dam flow)} \tag{C.15}$$

$$q = \sqrt{C'}\sqrt{h_3^3 \cdot 2g\left(\left(\frac{h_1}{h_3}\right)^3 - 1\right)} \quad \textbf{For throughflow} \quad \text{if} \quad H < 0 \quad \text{and} \quad hb < 0 \quad \text{(only throughflow)} \tag{C.16}$$

with

C'	=	Resistance factor	=	$\frac{\frac{1}{3}n_v^5 D_{n50}}{C_t \cdot L_{st}}$	[-]
L_{st}	=	Effective length	=	$B + (2d + 0.67(h_1 + h3)) \cdot cot(\alpha)$	[m]
n_v	=	Porosity			[-]
C_t	=	Throughflow coefficient			[-]

Flow regimes supercritical and through flow can occur at the same time. This representation is not accurate, because it simulates a permeable rock dam where water can flow through with an impermeable crest where water will flow over without friction. However, this regime is the high dam flow regime with a highly permeable rock dam, for which normally the contraction coefficient is greater (figure C.5). Because the range of this coefficient is not known for each case, this approximation will suffice for now. As can be seen in figure 9 contraction coefficients go up to 5, which is very high. With this approximation, the flow will probably be underestimated. This works in favor of dam closure velocities, since the greater the discharge through the dam, the less difference there is between the sea and basin level. For stability reasons, this will be different, since the discharge is underestimated and the stability of the rocks must be sufficient. In following stability calculations, this flow regime will be further analyzed.

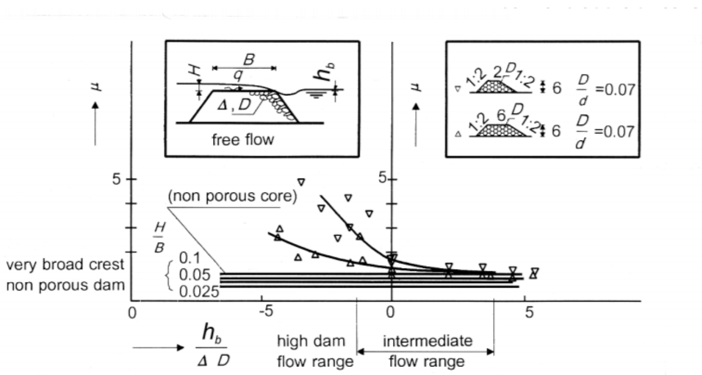


Figure C.5: Physical test results of contraction coefficient for high dam flow and intermediate flow (CETMEF et al., 2007)

The discharge can be computed by multiplying the specific discharge with the width of the cross section (equation C.17)

$$Q = q \cdot b \tag{C.17}$$

In case of a horizontal closure and it is a rock dam, b will be reduced. However the parts that are closed will still be open for through flow. This should be taken up when the width of the cross section decreases. For this equation C.18 can be used.

$$Q_{flowthrough} = q_{flowthrough} \cdot (b_0 - b) \tag{C.18}$$

River flow is not taken into account since the discharge in large tidal basins is relatively low compared to the discharge through the gap. (Order 1% at peak flow)

C.6. Storage model

In section C.2, the small basin approximation was used to derive a relation to describe the water level of the basin given the discharge into the basin. Then in section C.5, the discharge relations of flow over a weir were derived for several flow regimes. Furthermore, a linear relation was derived for the wet basin area at HAT and LAT in section C.4. In the same section a relation was derived to describe the sea level based on known tidal constituents.

Together, these relations form the storage model which can be solved in time given a known sea level. The storage model can compute the flow velocities in in time in the cross section that needs to be closed at all times. A changing width or depth of this cross section results in a change in flow velocity in the model and that is why the storage model is widely used by engineers to compute the changing flow velocities at all times during a closure operation.

D

Bed protection

D.1. Introduction

In this appendix, a calculation method is presented for the required volume of bed protection based on theory and practical knowledge. The function of using bed protection during a closure is to prevent structural instabilities in the direct surroundings of the closure structure. The bed protection keeps the bed level around the dam structure at a constant level thereby preventing large scour holes near the structure. The requirements for the design of bed protection can be deduced from its three functions:

Sand tight This prevents the sand from moving and therefore a stable bed is guaranteed. To meet this requirement the protection can be executed in the form of a filter layer which is permeable for water but not for sediments. Another possibility is to form an impermeable layer. However, in this situation, occurring water pressure differences will have to be dealt with.

Stability The bed protection must be able to withstand the incoming forces of the flow, waves and other occurring water forces. In the case of a closure dam, the flow velocity forms the normative force and is therefore the only one considered in the design of bed protection.

Length The bed protection requires enough length to keep away possible instabilities in the soil generated by scour holes from the structure. The occurrence of a scour hole can't be prevented, but the instability of the soil near the structure can with enough length of bed protection.

The required volume of bed protection is closely related to the total cost of bed protection, therefore this defines for a large part the total cost for this cost factor. With this in mind, the required *length* becomes the most costs influential aspect of bed protection. The area is computed by multiplying the length with the required dam length. The stability is guaranteed by choosing a stable type of bed protection. The possible types each have a specific layer thickness which is stable for a range of design flow velocities. The type therefore defines layer thickness and can be multiplied with the area to compute the required volume. The aspect of sand tightness is included in the design of each possible type of bed protection.

To start, the required length of bed protection is directly related to the shape of the scour hole that forms at the end of the bed protection and the possible resulting structural instability of the dam. The depth and slope angle of this hole can be modeled with the use of the theory of local scour (Schierkeck, 2012) (section D.1.1) and is explained in section D.1.2. The stability of the structure depends on the sliding plane of the soil and the distance from the structure. This is explained in section D.1.5. In section D.2 the practical application of the computational method using a probabilistic model is explained. To conclude, the possible types of bed protection are discussed with which the total volume can be computed.

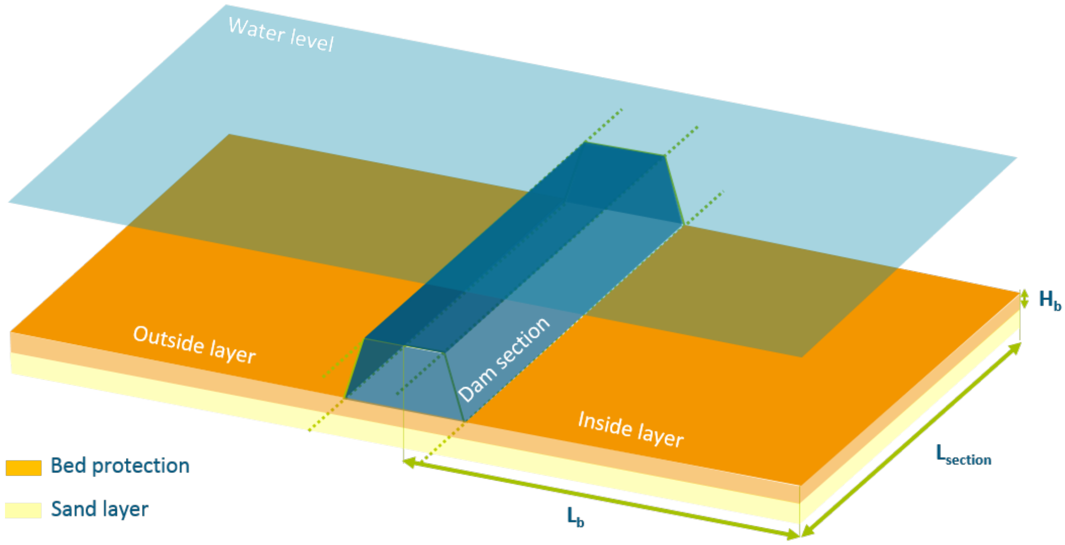


Figure D.1: Example bed protection requirements for one cross section (dam element) in the dam

D.1.1. Theory of local scour

Flow over a sill is described by separation and reattachment of flow. Directly behind the sill the flow separates and forms a new shear layer (figure D.2). This layer bifurcates at the at the reattachment point. The reattachment point is located about 5-8 times the sill height in horizontal direction downstream of the sill. The bifurcation of the flow partially generates a recirculation near the sill and partially forms a new sub boundary layer after this point (Tani et al., 1961). Turbulence intensity and shear stresses decrease in this new sub boundary layer. Because this decrease of turbulence and shear stresses is a slow process, the formation of a fully developed boundary layer takes around 30 times the sill height in horizontal direction downstream of the sill. The flow velocity in this layer and occurring turbulences are determining factors of the scour hole depth at the end of the bed protection if applied.

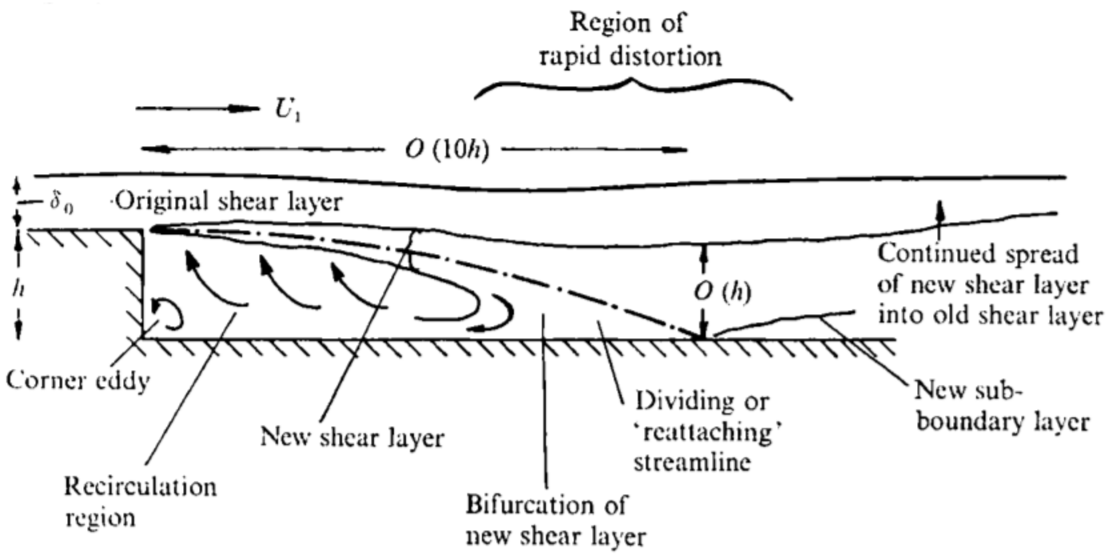


Figure D.2: schematization of shear layers behind a sill (Tani et al., 1961)

The development of a scour hole at the end of the bed protection is described by four phases:

Initial scouring stage In this stage the flow over the bed has an equilibrium velocity profile. If this velocity is larger than the critical velocity of the sediment grains, the scouring process starts. The end of this phase is defined by flow separation and the formation of a new shear layer and reattachment point. The slope angle at which this process start is estimated at around 7° (Chandavari and Palekar, 2014)

Developed scouring stage In this stage, the scouring process continues and expands in downstream direction. The upstream slope of the scour hole still starts at the end of the bed protection. The angle of this slope increases, but becomes constant while the hole gets deeper. Eddies start forming and take sediments to the lower parts of the scour hole. This stage takes longer and is interesting to analyze for tidal closures (Wiersema and Broos, 1998)

Stabilization stage In this stage the slope angle of the upstream slope stays constant. The scour hole has large dimensions and the flow velocity at the bottom of the hole slows down to near critical velocity. The scouring process continues slowly.

Equilibrium stage The velocity is equal to the critical velocity. Transport of sediments can still occur inside the hole, but do not leave the hole. The maximum scour depth is defined as the equilibrium scour depth.

D.1.2. Practical use for closure works

For the practical use of this theory in bed protection design of closures stage two is relevant. Closures are highly dynamic and time dependent. The scour hole that forms is also highly dependent on time. Significant relations in closure strategy and required amount of bed protection can be found by analyzing this stage. If time does not play a role in the design of the bed protection, the equilibrium scouring depth can be used to define the scour hole depth.

Time dependent scour In the developed scour stage, the depth of the scour hole can be described as a function of time and depending on several other parameters. Since a sill is present and assuming that this structure blocks the sediment from going over the sill, it can be stated that the scour type that occurs is “clear water scour”. Therefore equation D.1 can be used for clear water scour (Schierkeck, 2012)

$$h_s(t) = \frac{(\alpha_t u - u_c)^{1.7} \cdot h_0^{0.2}}{10\Delta^{0.7}} t_{scour}^{0.4} \tag{D.1}$$

In which:

$h_s(t)$	=	Depth of the scour hole in time		[m]
t_{scour}	=	This is the runtime for the scourprocess in days		[days]
Δ	=	Relative density of the quarry stone		[-]
h_0	=	Water depth in front or behind the dam		[m]
D_{n50}	=	Median nominal diameter bed materialr		[m]
α_t	=	Turbulence factor	$= \quad 1.5 + 5r_0 \cdot F_c$	[-]

α_t is a factor that accounts for the turbulence due to separation of the flow at the end of both dams. α_t decreases when long bed protections are used. For equation D.1 , r_0 and F_c can be obtained from the equations D.2 and D.3

$$r_0 = \sqrt{0.0225\left(\frac{1-D}{h_0}\right)^{-2}\left(\frac{L_b - 6D}{6.67h_0}+1\right)^{1.08} + \frac{1.45g}{C^2}} \tag{D.2}$$

$$F_c = max\left(\frac{C}{40}, 1\right) \tag{D.3}$$

In which:

D	=	$0.3 \cdot h_0$	[m]
L_b	=	Length bed protection	[m]
C	=	Chezy roughness coefficient	$[m^{0.5}/s]$

u_c is the critical velocity of the bed material and is given by the general stability equation from Shields (equation D.4)

$$u_c = C\sqrt{\Psi_c \cdot D_{n50} \cdot \Delta} \tag{D.4}$$

In which:

$$C = 18 \cdot log_{10}\left(\frac{12 \cdot h_0}{k_r}\right) \tag{D.5}$$

$$\Delta = \frac{\rho_s - \rho_{water}}{\rho_{water}} \tag{D.6}$$

The shields stability parameter Ψ_c is related to the water depth and the grain size. Since $\frac{h_0}{D_{n50}}$ is in almost all cases larger than 100, this parameter becomes constant and can be seen as a safe upper limit. (Schierkeck, 2012). The corresponding shields stability parameter is 0.03. The choice of this parameter remains subjective when there is no quantitative probabilistic information on construction cost and maintenance cost. Given the uncertainties, a practical choice can be made for the stability shields parameter and the roughness parameter k_r. A reasonable combination for these two parameters is therefore taken. The practical choice for the combination is: $\Psi_c = 0.03$ and $kr = 2 \cdot D_{n50}$.

D.1.3. Flow velocity

Determining the flow velocity u at the end of the scour protection is complicated, since 3D effects might be significant and this velocity depends on most of the other parameters. However, it's a conservative approach to use u_0 (the velocity on top of the sill) for this approximation. The velocity right after the sill can be approximated with the use of continuity and can therefore be scaled with the increased depth. The function for the flow velocity becomes:

$$u = u_0 \frac{h_{sill}}{h_0} \tag{D.7}$$

In which:

u	=	Flow velocity at the end of the bed protection	$[m/s]$
u_0	=	Flow velocity through the gap	[m/s]
h_{sill}	=	Depth above sill	[m]

The depth of the scour hole is now defined as a function of time and the constant flow velocity. It can't be modeled numerically in time steps, if the depth of the scour hole needs to be analyzed for a tidal system the average condition needs to be assessed. This can be done by integrating the turbulence factor and flow velocities over the complete time period T of a phase (relation D.8) (Schierkeck, 2012).

$$u_{a,s} = \frac{1}{T}\int_0^T (\alpha_t u - u_c)^{1.7} dt \tag{D.8}$$

In which:

$u_{a,s}$	=	Average relative soliciting flow velocity in a phase	$[m/s]$
T	=	Duration of phase	[s]

The new equation for the depth of the scour hole is now defined by D.9

$$h_s(t) = \frac{\frac{1}{T}\int_0^T (\alpha_t u - u_c)^{1.7} dt \cdot h_0^{0.2}}{10 \cdot \Delta^{0.7}} t_{scour}^{0.4} \tag{D.9}$$

D.1.4. Phasing

During a construction period, the conditions per construction phase vary from each other. Using the average velocity $u_{a,s}$ from equation D.8 over the full length of the construction period is not the correct in defining the scour hole depth. Instead, the scour hole development under average conditions of the phase is computed over time, for each phase. e.g. a scour curve is computed for the average conditions in the phase starting at t = 0 (figure D.3)

In this figure, the depth of the scour hole is approximated for three phases. In phase 1, the scour cuve follows the line (dark blue) belonging to the scour hole development for average conditions during phase 1. After time T1 (100 days), phase 1 ended and phase 2 starts. The average conditions in phase 2 are used to plot the second line (light blue) of scour development. However, phase 1 has ended at a scour hole depth of 6 meters ,thus this is from where the line continues with the conditions from phase 2. The time it takes for the conditions in phase 2 to develop the same scour hole is only 10 days. From here the second phase starts. T2 (90 days) ends at $t = 100$ days at $Hs = 9m$, from where a horizontal line is drawn again to the curve following the development of the scour hole during average conditions of phase 3. Under these conditions, the scour hole of 12 meters has already developed after 10 days. With a third phase lasting also 90 days, the resulting scour hole depth $H_{s,end}$ is 13.5 m.

The theory that founds this procedure is that the shape of the scour is similar under all conditions. This is one of the most important findings in scour research (Schierkeck, 2012).

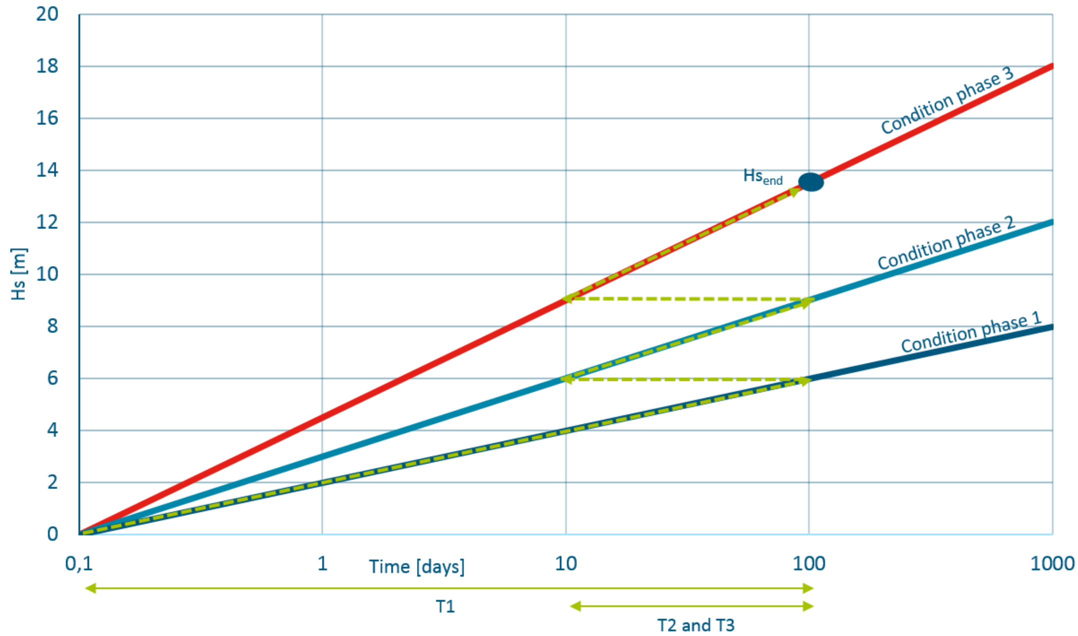


Figure D.3: Strategy to obtain the scour hole depth with varying conditions in each construction phase

D.1.5. Stability of the structure

The failure of stability of structure happens when the soil below the structure slides away as a consequence of the steep angle of the upstream slope of the scour hole becomes too steep. This sliding of the soil does not have to be prevented specifically; the structure just needs to be place far away enough from the scour hole that such a slide does not reach the structure. The sliding occurs if the inner slope of the scour hole becomes too steep. The definition of this angle is given by equation D.10

$$\beta = sin^{-1}\left(3 \cdot \frac{10^{-4} u_0^2}{\Delta g D_{50}} + (0.11 + 0.75 r_0) \cdot f_c\right) \tag{D.10}$$

In which:

D_{50}	=	Median diameter of bed material	[m]

The slope will slide until a certain angle depending on geotechnical conditions. This resulting angle is important in defining the length of the bed protection. It is assumed that sliding will occur. The failure slope angle after sliding of the inner slope is defined by the bed material. For loose sand and thus where liquefaction is possible, the slope defined by a slope of 1:15 or $cot(\beta_f)$ is 15. For each soil type and packing, a failure slope is defined. These can be found in table D.1 (Schierkeck, 2012).

Table D.1: Angle of the failed slope after sliding for several soil types, (Schierkeck, 2012)

Material	$cot(\beta f)$
Loosely packed sand (sensitive to liquefaction)	15
Densely packed sand	6
Clay	2

The problem is now defined by geometry and the length of the bed protection is only dependent on the depth of the scour hole and the angles of failure before (β) and after the slope fails (β_f). The geometry problem is sketched in figure D.4. The minimum required bed protection is therefore defined by equation D.11.

$$L_b = h_s \cdot \Big(cot(\beta_f) - cot(\beta)\Big) \tag{D.11}$$

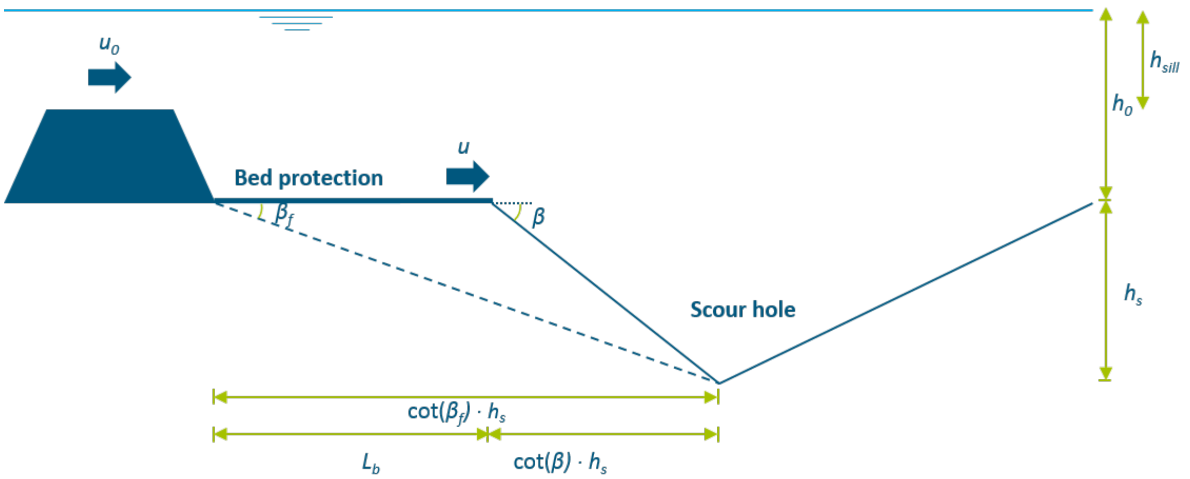


Figure D.4: Geometric relation for the length of bed protection

Stability in a deterministic calculation can now be calculated. The limit state function is defined by equation D.12

$$z = L_b - h_s \cdot \Big(cot(\beta_f) - cot(\beta)\Big) \tag{D.12}$$

D.2. Practical application in probabilistic model

D.2.1. Relevance probabilistic approach

A probabilistic model is used to compute the required area of bed protection. Just as the computation of the required materials, a probabilistic (Level III) approach allows to gain insight into the failure probability of the bed protection during a tidal closure. The probabilistic approach applies statistical distributions for all parameters in the computation. The probabilistic model applied in this study uses the allowed failure probability per layer of bed protection behind a cross section.The failure criterion is defined as a collapse of a part of the dam due to instabilities of the soil during the execution of the closure. The length of the failed dam section is equal to the length of a cross section. In practice, a smaller dam part will collapse, since conditions will vary along the dam. However, in this study, it is assumed, the conditions are the same everywhere across the length of the cross section (no length effect). The computed situation for one cross section is depicted in figure D.1.

The probabilistic model can be applied as a basis for a risk analysis. For a risk analysis, actual damages associated to the risk of failure have to be investigated in more depth. However, this is not performed in this research and therefore the accepted failure probability can be maximized without consequence just as in the probabilistic model to determine the rock sizes. For this same reason, the accepted failure probability is assumed to be constant (In this case 1/10000 per layer) and can be compared to a minimum safety level per layer.

Time is an important aspect in the computation, since the scour hole develops in time. Just as the rock computation, the consideration to be made is execution speed (equipment cost) versus required bed protection. increasing the execution capacity will decrease the quantity of bed protection required and vice versa.

D.2.2. Computation probabilistic model

The model used in this study computes the required bed protection length for *each dam section* in *each phase* probabilistically. To perform this computation, the computational method shown in section D.1.5 is too elaborate, the amount of parameters used is large and can be reduced significantly. Therefore some simplifications are made to still perform an accurate calculation but parameterize some definitions. The following assumptions have been taken to simplify the calculation:

- Assume a distribution for the failure slope cot(β_f)
- Assume a distribution for the slope before failure β (angle of repose)
- Assume a distribution for α_t (turbulence factor).

The required input is reduced all parameters in table D.2. The distribution and distribution parameters for each parameter are also defined. For simplicity, a normal distribution is used to model all parameters. The standard deviation is a percentage of the mean to even further reduce the amount of parameters. The choice for these specific values of the mean for each parameter are discussed below

Table D.2: Reduced parameters and their representative distributions

Parameter		**Definition**	**Unit**	**Distribution**	**Parameters**	
					μ	σ
D_{n50}	=	D_{50} = Median (nominal) grainsize diameter	$[m]$	Normal	0.0015	$0.03 \cdot \mu$
Ψ_c	=	Shields parameter sand	$[-]$	Normal	0.035	$0.02 \cdot \mu$
ρ_s	=	density sand	$[-]$	Normal	2600	$0.01 \cdot \mu$
$cot(\beta_f)$	=	Failure slope after failure	$[-]$	Normal	15	$0.1 \cdot \mu$
β	=	Slope of scour hole before failure	$[1/x]$	Normal	1.5	$0.1 \cdot \mu$
α_t	=	Factor for turbulence	$[m]$	Normal	1.1	$0.02 \cdot \mu$
$u_{a,s}$	=	Average soliciting flow velocity	$[m/s]$	Normal	eq. D.8	$0.03 \cdot \mu$

Length

With use of a Monte Carlo Simulation (MCS), the limit state function (equation D.12) can be evaluated for each dam section and for each phase. For each dam section, the maximum length required of all phases is normative. This is further elaborated in the next paragraph . Instead of using a distribution for the length L_b, and iteratively solve the mean of the distribution (as was done with the D50), the distribution is computed using the MCS. In this distribution, the value with the respective failure probability of 1/10.000 is chosen as design length for that section.

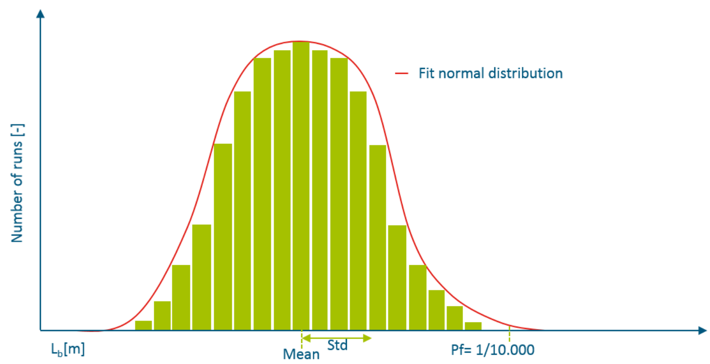


Figure D.5: Resulting distribution of the length of bed protection required

Area

With the required length, the area can be computed. However, this differs for each type of closure (horizontal or vertical). The difference can be logically explained by the execution procedure of both closure (figure D.6). In case of a vertical closure, the maximum velocity during a phase occurs across the complete section length. In case of a horizontal closure, this is only at the end (middle of the section). At the start (edges of the section), no bed protection is required because they are covered directly with stones. The actual gradient (from start to end) would be exponential, since the flow velocity increases exponentially (see figure D.6), but in the model it's linearized. This is a safe assumption, since the exponential line is always under the linearized line.

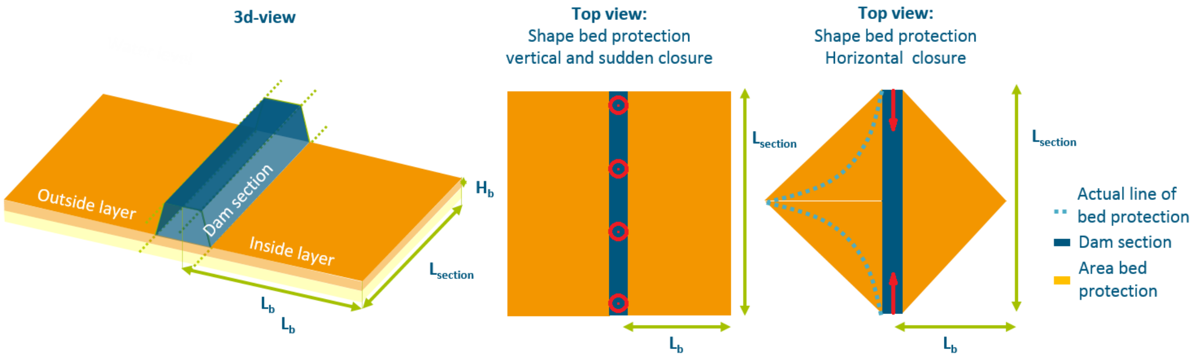


Figure D.6: Definition of bed protection area for different closing methods

For more phases, the computation becomes more complex. Each sequential phase, requires information about the previous phases. even if nothing happens to a cross section, the bed protection should be computed because it might be the normative phase. To clarify, two examples of closure strategies for one cross section are discussed in figure D.7. On the left side, a partial vertical closure is done in the first phase, which requires a square shaped bed protection with length L_{b1}. Following this phase, nothing happens for two phases, but the flow velocity (and time) increase and therefore the bed protection length increases during these phases to L_{b2} and L_{b3}. In the last phase, the section is closed horizontally with a normative length of L_{b4}. it is important to store what happened in the previous phase and this should be used to compute the next required area. The total area required is defined by relation D.13.The example on the right has a partial horizontal closure, after which only a small section is left. The total area required is defined by relation D.14

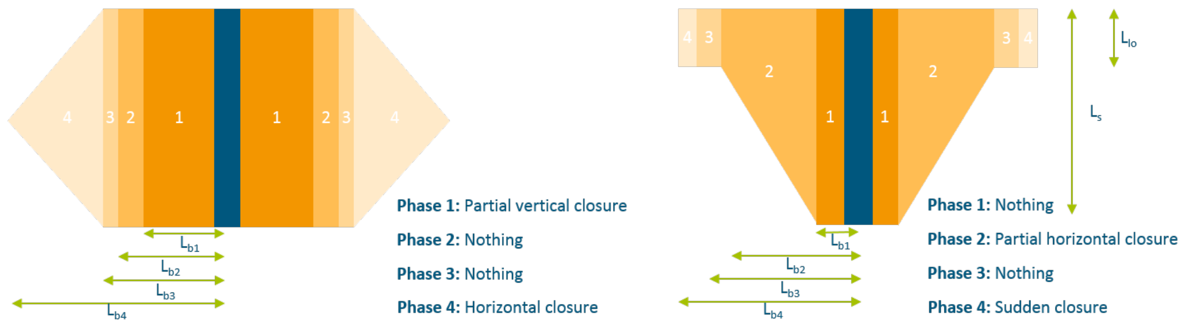


Figure D.7: Definition of bed protection area for different closing methods with phases

$$A_{bp} = L_{b1} \cdot L_s + (L_{b2} - L_{b1}) \cdot L_s + (L_{b3} - L_{b2}) \cdot L_s + \frac{1}{2}(L_{b4} - L_{b3}) \cdot L_s \tag{D.13}$$

$$A_{bp} = L_{b1} \cdot L_s + \frac{1}{2}(L_{b2} - L_{b1}) \cdot L_s + (L_{b2} - L_{b1}) \cdot L_{lo} + (L_{b3} - L_{b2}) \cdot L_{lo} + (L_{b4} - L_{b3}) \cdot L_{lo} \tag{D.14}$$

In which:

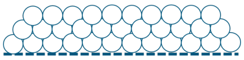
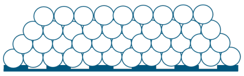
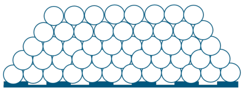
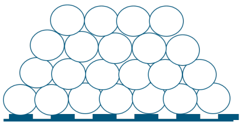
Type and resulting volume

Several types of bed protection exist. However, which one is best for the given situation depends on the maximum flow velocity occuring during the closure (design velocity). This determines the stability requirements

A_{bp}	=	Bed protection area required per section	$[m^2]$
L_{bn}	=	Length of bed protection required for phase n	$[m]$
L_s	=	Length of the dam section	$[m]$
L_{lo}	=	Leftover dam section after partial horizontal closure	$[m]$

for the top layer of the bed protection under which a filter is needed to prevent erosion of sand below the top layer. In 1998, the Dutch ministry of transport and public works designed four types of protection for the Haskoning study (Wiersema and Broos, 1998). They distinguished the classes into *loose*, *light* ,*heavy* and *very heavy* each with corresponding range of design velocities. All types are displayed in table D.3.

Table D.3: Types of bed protection used for initial design (Wiersema and Broos, 1998)

Type	Description	Design velocity [m/s]
	**Loose**:Geotextile filter layer +0.5 m stones	$u_{a,s} < 3$
	**Light**: Concrete block mat +0.8 m stones	$3 \leq u < 4$
	**Heavy**: Concrete block mat +1.0 m stones	$4 \leq u < 5$
	**Very heavy**: Concrete block mat +1.5 m stones	$5u > 5$

The design velocity on each side of the dam section is measured using the flow model and the resulting type of bed protection required is noted. The volume of bed protection required is the required area multiplied with the layer thickness of the design type of bed protection.

D.2.3. Discussion and limitation

The validity of the Breusers equation for the depth of the scour hole is within a certain range and has been known to overestimate results with increasing scale. From the measurement during the Deltaworks program, the scour holes reached maximum depths of 4 times the water depth. Because the Kalpasar closure dam is relatively large in comparison with the scale from Breusers experiments, the maximum scour hole depth of 4 times the water depth is adopted. In some cases, this can still result in scour depths of 80-90 meters, which is still large.

E

Complete derivation of material stability formula's

E.1. Material stability

During closure works that are executed with stone like material (quarry rocks, gabions or concrete blocks) hydraulic forces act on the structure. In case of a horizontal closure the, the hydraulic loading on the head of the dam is considered normative. In case of a vertical closure, the normative hydraulic load is present on the crest over the full length of the dam. (Akkerman and Konter, 1985)

While dimensioning the closure works with quarry stones, it is therefore important to check the stability of the quarried rock on the crest of the sill (vertical closure) or on the dam head (horizontal closure). The general stability formulation from Shields for uniform flow is commonly used to dimension the rocks sizes required. This formulation is defined by equation E.1(Akkerman and Konter, 1985)

$$U_c^2 = C^2 \cdot \psi \cdot D_n \cdot \Delta \tag{E.1}$$

In which:

U_c	=	Critical flow velocity rock	[-]
C	=	Roughness parameter	$[m^{0.5}/s]$
Δ	=	Relative density of the quarry stone	[-]
D_n	=	Nominal diameter of the quarry stone	[m]
ψ	=	Damage parameter or Shields parameter	[-]

For the roughness parameter, the formulation of White-Colebrook is used. This relation is defined in equation E.2.(Akkerman and Konter, 1985)

$$C = 18log_{10}(\frac{12h_0}{k_r}) \tag{E.2}$$

In which:

h_0	=	Waterdepth	[m]
k_r	=	Roughness height ($2D_n$)	[m]

For damage parameter ψ, a value of 0.04 is recommended, which corresponds to "some" transport. If the construction is exposed to the hydraulic load for an extended period of time, the value of ψ should be reduced to 0.03. This value corresponds with the criterion "beginning" of movement (Konter et al., 1997).

Equation E.1 serves as a basis for the derivation of stability relations for sill and dam. For this derivation, the equation is provided with a factor K, which is called the K-factor. (Akkerman and Konter, 1985)

Also the definition of the flow rate u will be changed slightly for the purpose of using equation E.1 as stability relationship for non-uniform conditions. As a flow rate, the relation presented in section C.5 is used. The use of this flow rate makes the mutual comparison of stability relations and closure methods possible. However, at small constriction percentages the use of this flow rate is not recommended. In such situations there will be virtually no local drop in head. This means that the flow rate can be estimated using the Chézy equation. In that case, the stability of quarry stone can be investigated with equation E.1 without any modifications. If the head drop is significant, a modification is neccessary. The modified relation is defined by equation: (Akkerman and Konter, 1985)

$$(K \cdot U_c)^2 = C^2 \cdot \psi \cdot \Delta D_n \tag{E.3}$$

or

$$\Delta D_n = A \cdot U_c^2 \quad \text{with} \quad A = \frac{K^2}{\psi \cdot C^2} \tag{E.4}$$

In which:

K	=	Correction factor	[-]

E.1.1. Horizontal closure

With a horizontal closure, the stability of the quarry stone on the bank of the dam head plays a central role. Model research shows that much damage occurs on this slope at the location of half water depth between the dam heads. In figure E.1 this area is indicated shaded.(Konter et al., 1997)

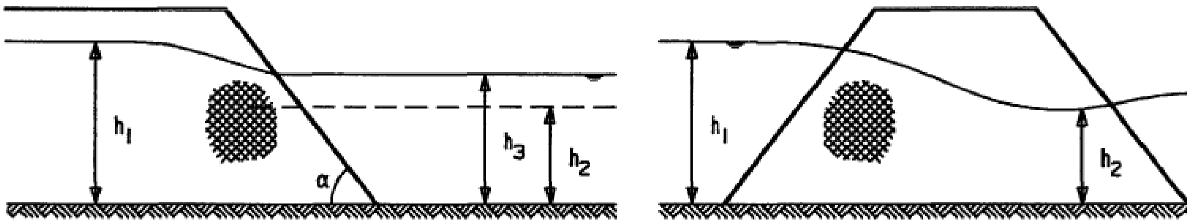


Figure E.1: Cross sections of the head of the dam with point of largerst hydraulic loading (normative) (Konter et al., 1997)

Systematic model research into such closures has been done by Naylor. with carefull re-analyses of this research, an alternative stability relation is defined.This relation is presented in equation E.5.(Konter et al., 1997)

$$\frac{U}{\sqrt{g \cdot \Delta D_n}} = \frac{K_\alpha}{K_*} Log(\frac{3h_2}{D_n}) \cdot \sqrt{\frac{\psi}{g}} \tag{E.5}$$

Where the correction factor K_α is defined by relation E.6

$$K_\alpha = \sqrt{cos(\alpha) \cdot \sqrt{1 - \frac{tan(\alpha)^2}{tan\phi}}} \tag{E.6}$$

In which:

K_α	=	Correction factor voor slope dam head	[-]
K_*	=	Correction factor for the influence of turbulence	[-]
α	=	Slope of the dam head	[°]
ϕ	=	Natural slope of the material	[°]

Using measurement data from Naylor, the K-factor has been established to be 0.8 for horizontal closures (figure E.2).The relation for the stability of stones at the dam head is therefore defined by equation E.7 and corresponds to the line in figure E.2 (Waterloopkundig Laboratorium, 1985).

$$\Delta D_n = A \cdot U^2 \quad \text{with} \quad A = \frac{0.64}{K_\alpha \cdot \psi \cdot C^2} \tag{E.7}$$

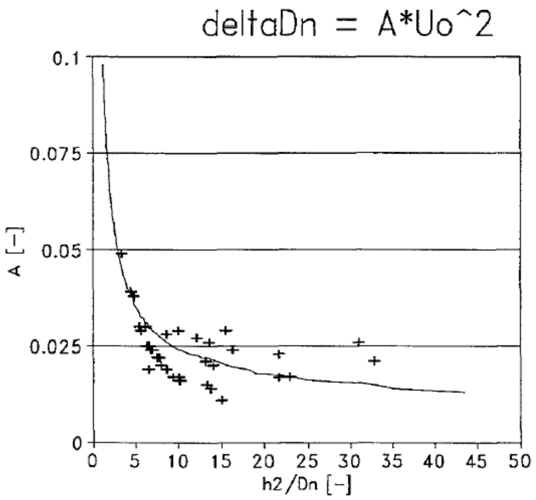


Figure E.2: Measurements from data and approximation line by equationE.7 (Waterloopkundig Laboratorium, 1985)

E.1.2. Vertical closure

As with the horizontal closure, the equations are re-analyzed for vertical closures . The K-factor is again composed of a K_* factor for other influences and a K_α for the influence of the slope of the downstream dam slope. The latter correction factor is defined by equation E.8. (Waterloopkundig Laboratorium, 1985)

$$K_\alpha = \sqrt{\frac{sin(\phi - \alpha)}{sin(\phi)}} \tag{E.8}$$

Again, using measurement data from Naylor, the K-factor has been established to be correspondin very well with 0.6 for vertical closures (figure E.3).The relation for the stability of stones is therefore defined by equation E.9 and corresponds to the line in figure E.3 (Waterloopkundig Laboratorium, 1985)

$$\Delta D_n = A \cdot U^2 \quad \text{with} \quad A = \frac{0.36}{K_\alpha \cdot \psi \cdot C^2} \tag{E.9}$$

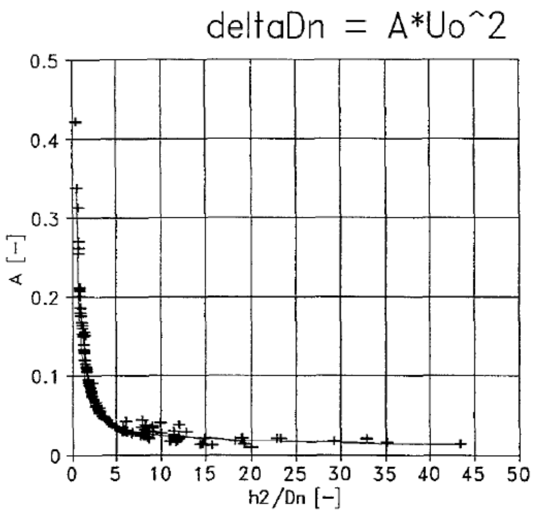


Figure E.3: Measurements from data and approximation line by equationE.9 (Waterloopkundig Laboratorium, 1985)

F

Probabilistic analysis

In this appendix, the theory of probabilistic analyses is described based on the lecture notes of the course Probabilistic design and risk management from TU Delft. In the first section, an introduction is presented which methods can be used to calculate the reliability of a structure or a related process or element (for example: bottom protection). For the probabilistic analyses, it has been decided to use a level III method to perform the reliability analyses. This will be performed using a Monte-Carlo simulation. The approach of this method is elaborated in F.2 and the Monte-Carlo simulation is further explained in section F.3.

F.1. Reliability methods

In general, five methods can be distinguished to perform a reliability analyses

- **Level IV methods (risk based)**: In these methods, the complete risk is analysed and used to compare different designs on an economic bases taking into account uncertainty, costs and benefits. The probability of failure is multiplied with the consequence to include this risk, therefore the methods distincts itself in including the consequence as well.
- **Level III methods (numerical)**: Each quantity (parameter) is modelled with an uncertainty with use of their joint distribution functions. The probability of failure can then be computed exactly using numerical integration. Another method to extract the distributions of the reliability and failure functions is a Monte-Carlo simulation.
- **Level II methods (approximation):** The uncertain parameters are modelled by the mean values and the standard deviations, and by the correlation coefficients between the stochastic variables. The stochastic variables are implicitly assumed to be normally distributed.
- **Level I methods (semi-probabilistic design):** The uncertain parameters are modelled by one characteristic value for load and resistance, for example partial coefficients (safety factors) in different codes.
- **Level 0 methods:** Deterministic calculations.

F.2. level III method elaborated

The reliability distribution that is defned is called the limit state function and is defined by equation F.1

$$Z = R - S \tag{F.1}$$

In which:
Z = limit state function
R = Resistance function
S = Solicitation function

Failure occurs when $R < S$, so when $Z < 0$, the probability of failure equals $P_f = P(Z < 0)$

For practical reasons, only the case with independent normally distributed variables is considered here. In reality, distributions might not be distributed normally, but with lognormal or gumbell distributions. The result is a distrubtion with normal parameters described by equation F.2

$$P_f = P(Z < 0) = \Phi(\frac{0 - \mu_z}{\sigma_z}) = \Phi(-\beta) \tag{F.2}$$

In which:
μ_z = Mean (parameter) of resulting limit state distribution Z
σ_z = Standard deviation (parameter) of resulting limist state distribution Z
β = Reliability index, distance from the mean value of Z and Z = 0 in units of the standard deviation.

Table F.1: Relation P_f and β

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.28	2.32	3.09	3.72	4.27	4.75	5.2

F.3. Monte-Carlo simulation

Monte-Carlo simulations are generally used in cases where $n > 2$, since numerical integration becomes quite difficult. In order to compute the range and distrubution of a design parameter, the deterministic computation can be tranformed into a probabilistic computation with help of the Monte-Carlo simulation, generating random samples.

Random samples can be generated with Matlab (or other software) by first drawing a random sample from a uniform distribution between 0 and 1. The distribution is defined by equation F.3

$$F_u(x) = \begin{cases} 0 & x < 0 \\ x & 0 < x < 1 \\ 1 & x > 1 \end{cases} \tag{F.3}$$

Random samples of an arbitrary cumulative distribution function $F(x)$ with its inverse $F^{-1}(x)$ can be drawn by using xu from the uniform distribution as probability.The definition for a random sample x_i is presented in equation F.4.

$$x_i = F^{-1}(x_{u_i}) \tag{F.4}$$

Each variable in the calculation is assigned a distribution and random samples are generated with equation F.4 and are stored in a variable-array. These arrays are then used to perform the computation. Hence, the outcome of the computation is also a distribution.

From this distribution, the type of distrubtion and distribution parameters can be extracted. To design structural elements such as length of bottom protection and material sizes, an accepted probability of failure can be used to determine the required design paramters.

F.3.1. Latin Hypercube Sampling

Typical Monte Carlo simulations use random sampling of input variables as described in section F.3 which may cause spurious correlations of input parameter values. In other words, the amount of samples required to produce an accurate final distribution is very large due to the many variables used. With help of Latin Hypercube Sampling (LHS) (Helton and Davis, 2003), the distribution range is divided into N equally probable intervals from which a sample is generated. This sampling scheme does not require more samples for more dimensions (variables); this independence is one of the main advantages of this sampling scheme.

Theory: A square grid containing sample positions is a Latin square if (and only if) there is only one sample in each row and each column. A Latin hypercube is the generalisation of this concept to an arbitrary number of dimensions (variables), whereby each sample is the only one in each axis-aligned hyperplane containing it (Helton and Davis, 2003).

An example is given in table F.2 where a sample is taken for each dimension on each row rembering that it can't be taken from the same row.

Table F.2: LHS sampling technique for each N (rows) and each equally probable interval *A* of the distribution (columns)

			A		
	X				
		X			
N					X
			X		
				X	

The Latin Hypercube Sampling technique is chosen to use in this research for performance improvement. The function is available in Matlab.

G

Kalpasar case specifics

G.1. Case related parameters

G.1.1. Tide

The tidal amplitude defines the outer boundary condition of the model and generates the flow velocities in the closure gap. The tidal amplitude in the gulf of Khambhat has been analyzed by many researchers and tidal constituents are known for many positions. The main constituents in the Gulf of Khambhat are the M2, S2, K1 and O1. In table G.1, the observed amplitudes and phases are presented (Nayak et al., 2015). In this paper, the tidal behavior was modelled using mathematical models to validate the water levels resulting from these constituents. The resulting amplitudes and phase differences have been presented as well.

Name of stations	M2					S2					K1					O1				
	Ao	Go	Am	Gm	R	Ao	Go	Am	Gm	R	Ao	Go	Am	Gm	R	Ao	Go	Am	Gm	R
Porbandar	65	314	77	309	8	24	349	13	336	7	35	56	34	60	1	17	54	17	59	0
Vereva	54	327	43	309	8	21	5	15	336	4	36	63	37	60	1	18	59	19	59	1
Kotra	45	0	54	12	6	18	44	20	37	2	40	70	39	68	1	21	70	20	63	1
Nawabandar	50	19	59	24	7	24	61	22	49	1	44	67	40	71	3	21	67	20	65	0
Jafarabad	62	57	73	47	8	26	88	27	73	1	47	72	42	77	3	20	71	21	70	1
Albert	87	75	82	60	4	34	105	30	88	3	50	77	43	81	5	22	72	22	74	0
Piva	88	75	87	68	1	35	104	32	97	2	50	76	43	84	5	23	72	23	76	0
Sul_cambay	212	110	127	68	51	82	147	52	97	17	999	80	60	105		30	52	23	76	5
Bhavnagar1	314	143	172	152	88	96	190	64	195	18	76	92	57	123	14	34	75	25	114	6
Bhavnagar2	143	144	111	184	15	98	190	39	228	38	77	90	46	147	22	34	70	19	132	11
Dehej	312	125	173	153	86	61	172	44	197	9	24	76	41	123	12	20	30	25	113	3
Ambheta	165	134	146	164	5	76	175	53	208	13	63	76	52	131	8	33	69	22	119	7

Figure G.1: Tidal constituents with observed amplitude and phase lag in the Gulf of Khambhat (Nayak et al., 2015)

To define the boundary condition for the storage model a representative location must be chosen. The constituents at Bhavnagar1 have been used in previous engineering reports by Broos and Wiersema and also by Royal HaskoningDHV in 1998. These constituents generate the maximum occuring tidal amplitude in the Gulf. According to reports from Royal Haskoning, the tidal range in the Gulf is between 7 and 11 meters. The constituents at Bhavnagar1 therefore describes the tidal wave very well, since it yearly ranges monthly between 6 and 9 meters with yearly fluctuations which add one meter (figure G.2) . The maximum yearly astronomical amplitude is 5.2 meters which results in a tidal range of 10.4 meters. This tidal behavior suffices as boundary condition for the storage model. Though it is simplistic and overestimates water levels alongside the dam alignment.

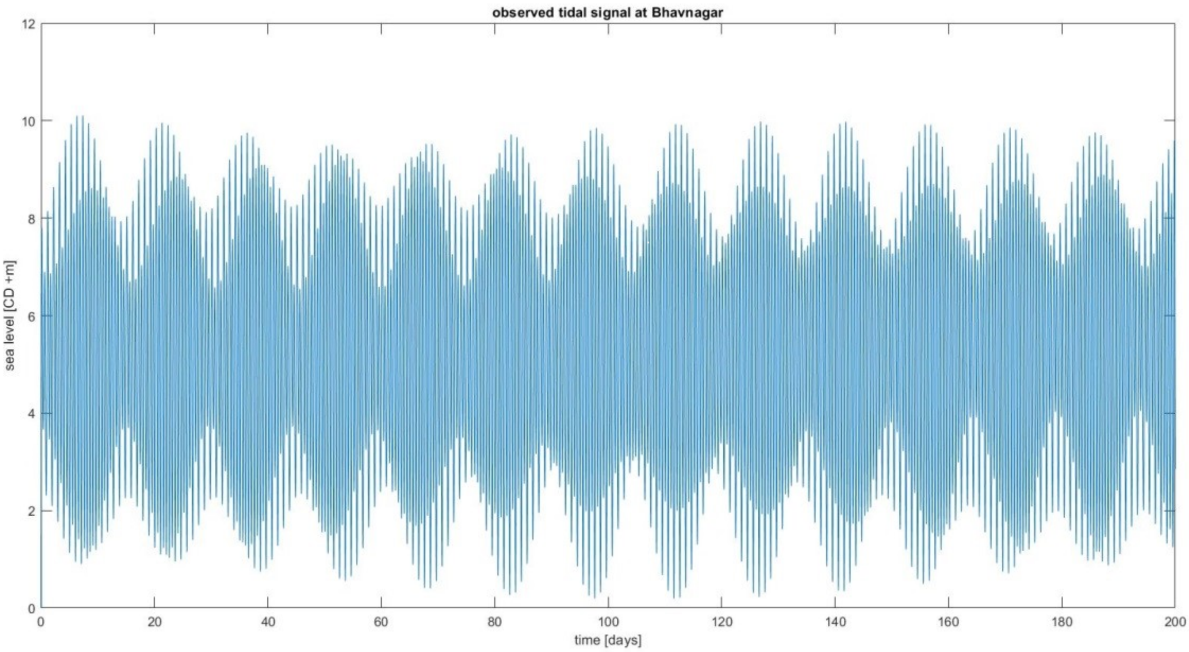


Figure G.2: Tidal constituents Bhavnaghar1 plotted with corresponding phase differences

G.1.2. Bathymetry

The bathymetry in the Gulf of Khambhat is constantly changing and therefore very dynamic due to the high flow velocities. This complicates research into the case. Furhtermore, the latest surveys for admirality charts (figure G.5) for some place alongside the dam are from 1855 (East india Company survey). However, the relevant location of the dam is positioned in section b (figure G.5) and is surveyed in 1986. From this bathymetry, the general location of the channels and tidal flats can be extracted and used for the optimization model. The model in this study uses a simplication of the bathymetry with straight channels and tidal flats to simplify the situation and to allow the flow model to compute the flow velocity. A detailed bathymetry is therefore not required.

The admiralty chart (figure G.5) shows the depth values relative to LAT and therefore Chart Datum (CD). Chart Datum is used throughout the model and is positioned at MSL - 5.2 m. In figure G.3 a rough sketch is depicted of the bathymetry at the dam location. This data has been translated into a more specific cross sectional area with use of Matlab and is depicted in figure G.3

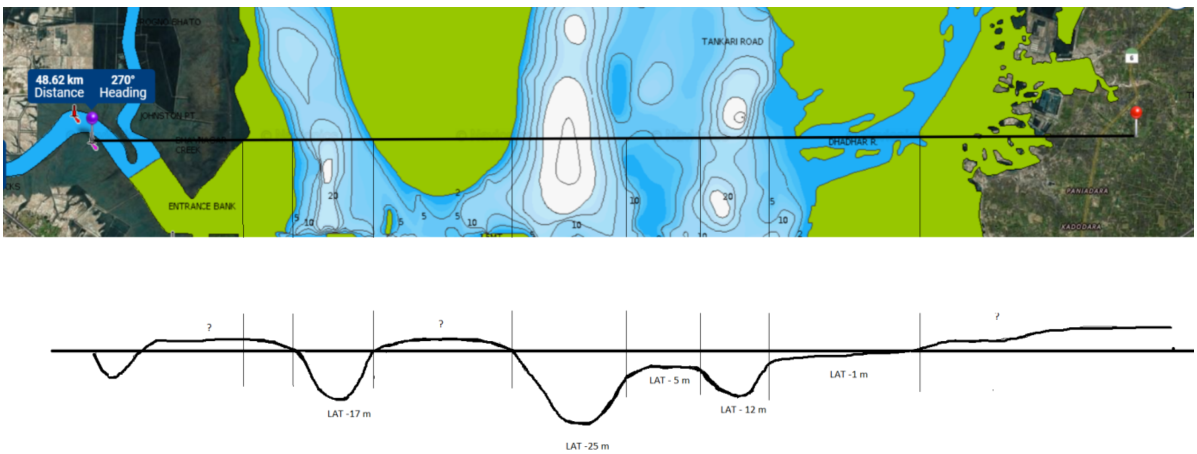


Figure G.3: Bathymetry at dam location roughly sketched with use of navionics naval charts (Navionics, 2018)

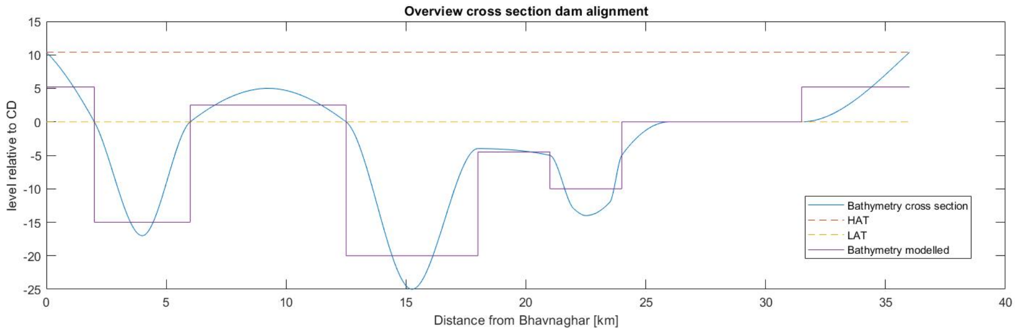


Figure G.4: Bathymetry at dam location roughly sketched with use of navionics naval charts (Navionics, 2018)

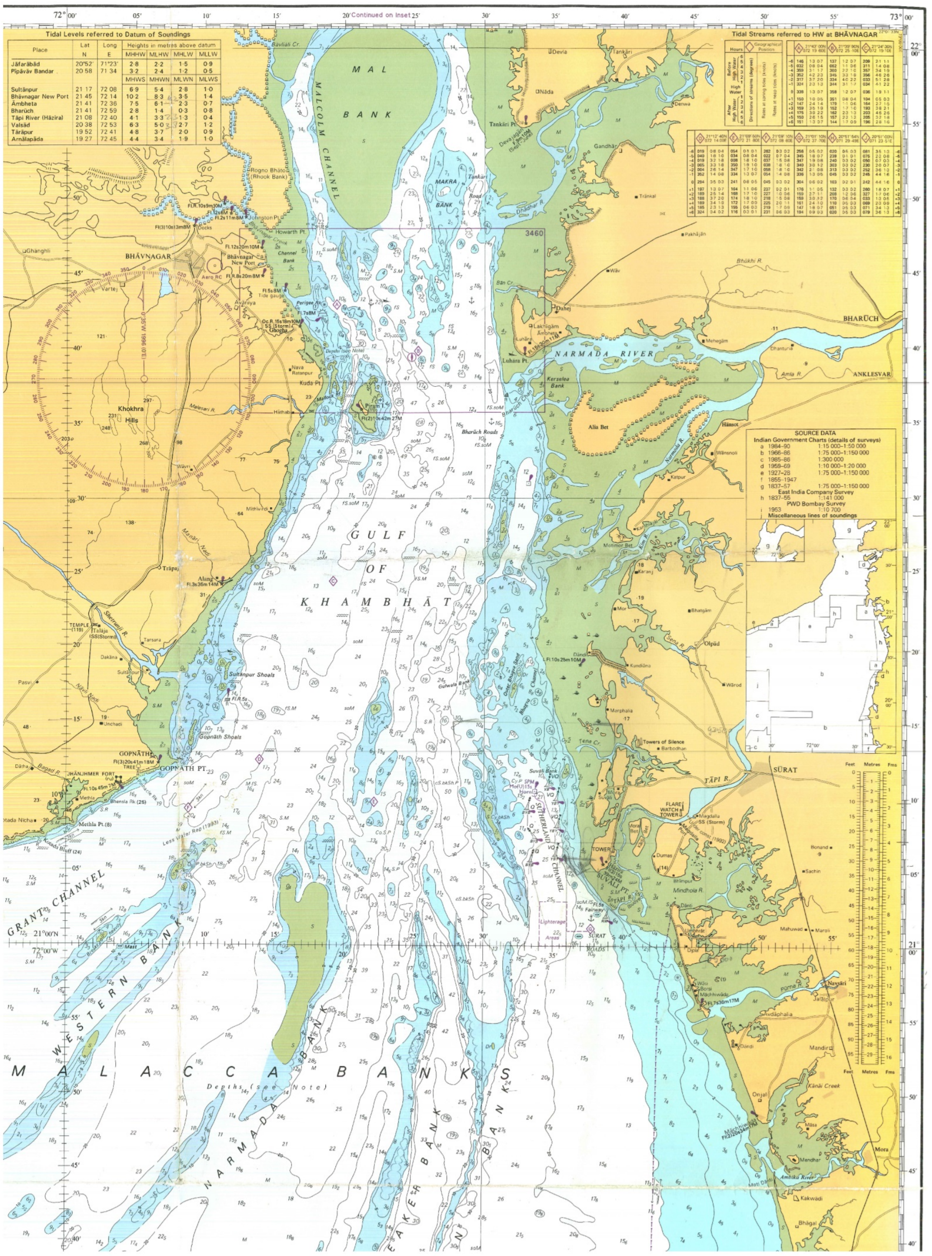


Figure G.5: Admiralty Chart 1486 - Gulf of Khambhat (British Admiralty, 1980)

G.1.3. Quarry yield curve

As was mentioned in chapter 2, the quarry yield curve represents the stone sizes that can be delivered by the local quarry. Matching the demand with the demand can save costs of overproduction. The yield curve from the quarries in Gujarat are unknown. However, a assumed yield curve based on a report "Quarry yield prediction as a tool in breakwater design" (Smarason, 2000) and the report by Broos and Wiersema can be defined. It has been decided to use an exponential function for this yield curve. The yield curve used by Broos and Wiersema is presented in figure G.6 and the yield curve used in this study is presented in figure G.7 and follows the line defined by relation G.1

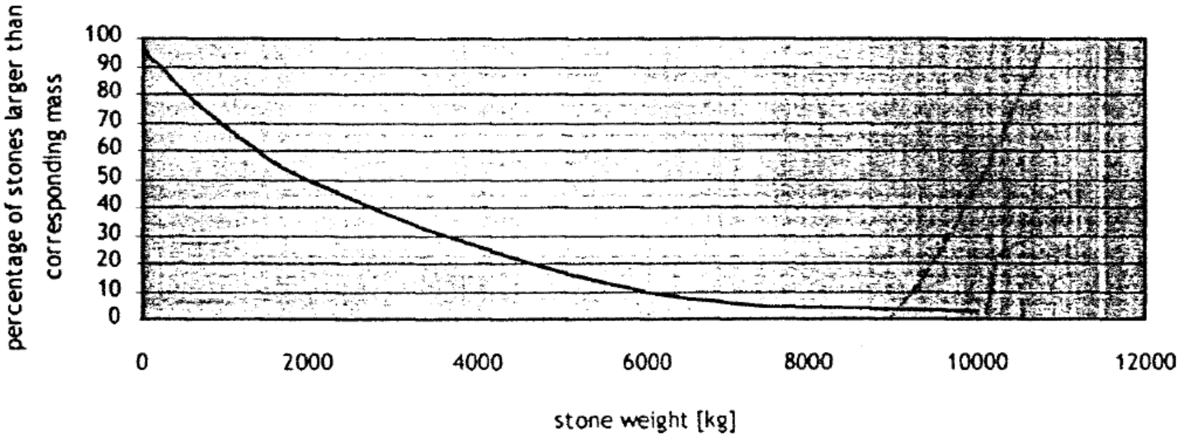


Figure G.6: Assumed quarry yield curve used by Wiersema and Broos (Wiersema and Broos, 1998)

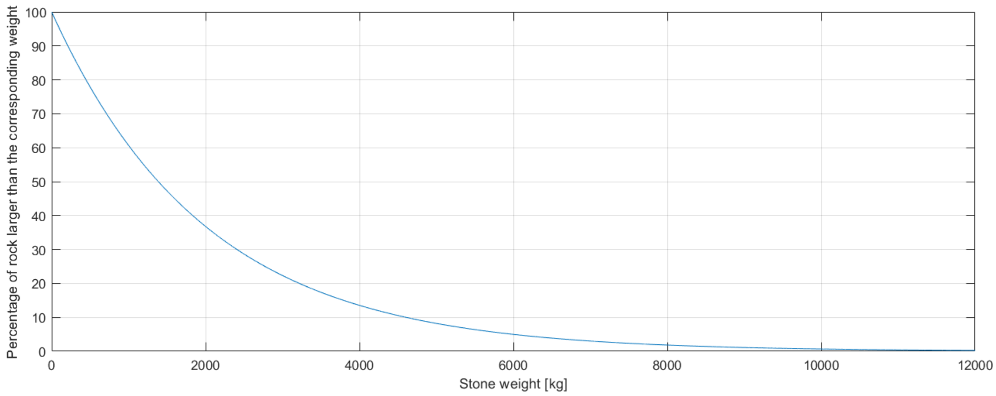


Figure G.7: Assumed quarry yield curve in this study (equation G.1)

$$P_{yield} = 100e^{-0.0005 \cdot w_s} \tag{G.1}$$

In which:

P_{yield}	=	Percentage yield larger than corresponding stone weight	[%]
w_s	=	Stone weight	$[kg]$

G.1.4. Storage area

In this section, the goal is to define the relation between the wet surface basin area above the dam line for high and low water. This relation is called a hypsometric curve for a basin (see figure G.8) and can be used to asses the total storage of the basin

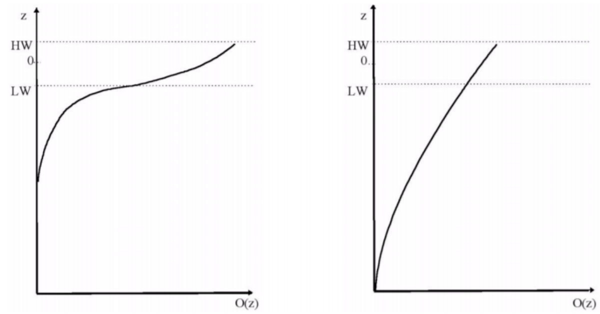


Figure G.8: Hypsometric curves for two types of tidal basin (Bosboom, 2015)

The shape of this curve is characteristic for the basin and can help define the type of tidal basin. In figure G.8, two hypsometric curves are displayed. The left figure is representative of a situation with shallow channels and large intertidal storage areas. The right situation is the opposite; deep channels and small intertidal area. The hypsometric curve can also be used to compute the storage volume and wet basin area, which is required as input for the storage model.

G.1.4.1. Required data

The hypsometric curve can be generated with use of tidal data and satellite images. The following needs to be known to compute the curve:

- Wetted area of basin with ebb (at CD)
- Wetted area of basin with flood (at CD + maximum tidal range)
- Wetted area of the basin in between flood and ebb at several water levels
- The tidal range (10 m)

G.1.4.2. Google Earth Engine

The wetted area of the basin can be computed using google earth engine. Google Earth has developed a data set of water occurrence on the surface of the earth. These data were generated using 3,066,102 scenes from Landsat 5, 7 and 8 acquired between 16 March 1984 and 10 October 2015. Each pixel is individually classified into water / non-water using an expert system and the results were collated into a monthly history for the entire time period. This means that a chart of the surface waters can be generated in which an occurrence percentage can be set.

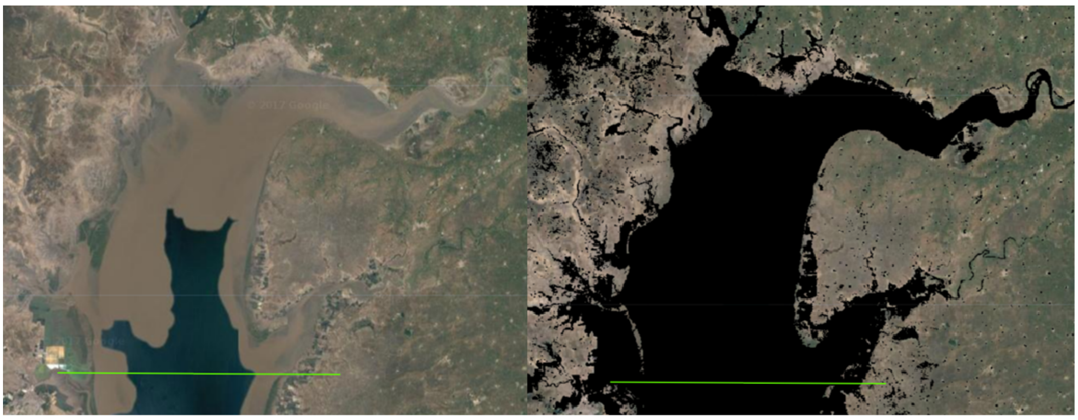


Figure G.9: Results of 99% occurrence (left) and 1% occurrence (right)

For example, if an occurrence percentage of 1 has been chosen, only 1% of the time (average over each month for 32 years) water was present on that location. This means the chart displays almost any location where water has been in the past 32 years. This means If 99 % is chosen, only the places where water is almost always (even during extreme low water) are displayed. Because the data is a monthly average over 32 years, taking these 1 and 99 % percentages as high and low water is not representable (figure G.9).

However, it is unknown what the percentages represent. furthermore, monthly averaged data is open for interpretation. Since the dataset is large (32 years) and abnormalities such as water detection in land reclaimed areas during this period can be located, the data can be used to form a good first estimate of the hypsometric curve. In this case it is the bests that can be used with lack of other wetted area data sets of the area. High and Low water can later be defined as logical points on the curve.

G.1.4.3. Computing the wet surface area

Google earth displays every pixel which is wet in black (figure G.10) define the area that is black above the dam (green line in all figures), a polygon is drawn around the black area, averaging out the complicated structure with straight lines. The area of the polygon can then be computed in Earth Engine.

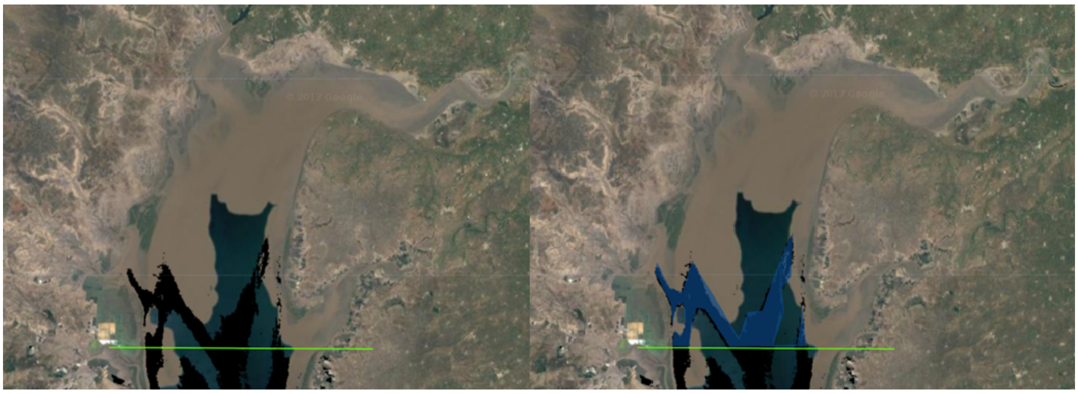


Figure G.10: Occurrence 80% (left), drawn polygon to compute this surface area (right)

G.1.4.4. Result

In figure G.11, the result is displayed for several percentages of water occurrences and plotted.

Occurrence [%]	Wetted area of the basin [km^2]
90	7
80	175
70	472
60	802
50	1067
40	1302
30	1505
20	1600
10	1704

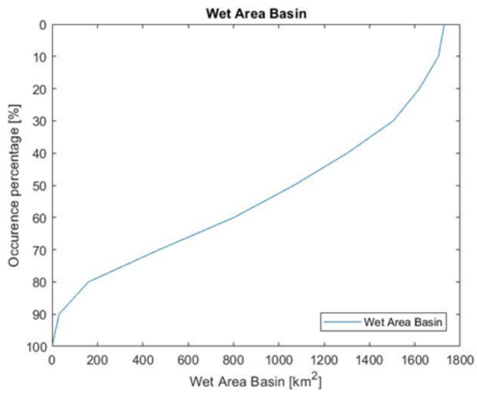


Figure G.11: Results of area computation (left), Hypsometric curve w.r.t occurence (right)

Since this is the wet basin area plotted verses the water occurrence percentage, a relation with water level is not yet defined. However, the plot in figure G.11 has about the same shape as figure G.8. Concluding that a linear relation between the occurrence frequency and water level can be assumed for now.

Since occurrence percentages need to be linked in some way to LAT and HAT, respectively the 70 % and 10 % occurrences are chosen. The reasoning behind this is as follows:

- The wet surface area of 70 % is in the same order of the area on Navionics navigational charts which displays LAT (Navionics, 2018). For the comparison see figure 15 (+/- 472 km2 vs +/-460 km2). So 70% occurrence is LAT.
- Choosing HAT at 10% occurrence is logical. This means 10% of the time water was here. 10% makes sure that it still originates from the tide. Taking 10% less than 100% excludes storm surges and bank changes that happened in the last 32 years. 3 or 5% could also be chosen here since HAT is only once a month.

In figure G.12 the result of the LAT and HAT are derived.

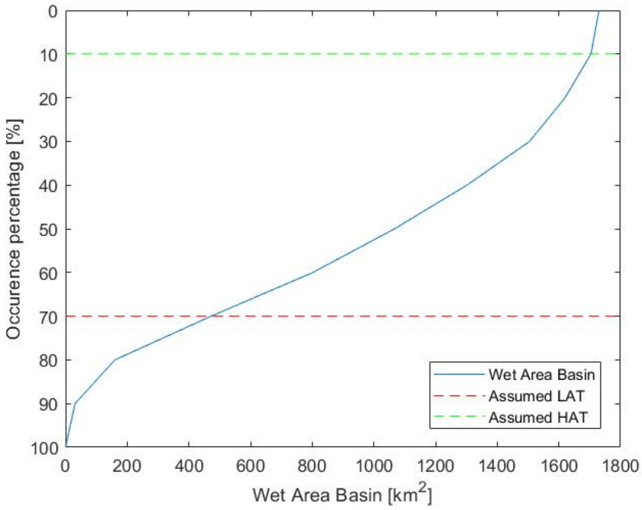


Figure G.12: Hypsometric curve with LAT and HAT plotted on the 70 and 10 % occurence line respectively

Conversion to water level

For the first basic analytical model, a tidal range of 10.4 m has been assumed between LAT and HAT. This has been deducted from four different tidal constituents. On the earlier assumption the water level is linearly related to the occurrence and assuming CD + 0 m for LAT and therefore the 70% occurrence frequency and CD + 10.4 m for the HAT and therefore the 10% occurrence frequency, the water level can be linearly interpolated. This is shown in figure G.13.

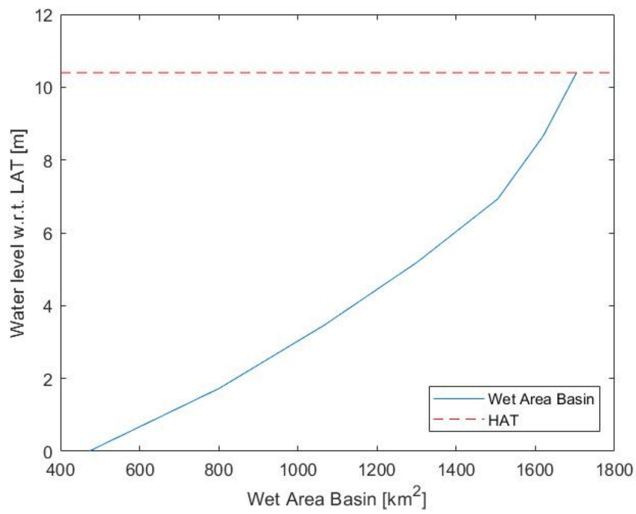


Figure G.13: Hypsometric curve translated to water levels

G.1.4.5. Storage area

The change in water level in the basin in the storage model ussually assumes a constant wet basin area at all tidal levels. However is is often not the case, since the wet area at Highest Astronomical Tide (HAT) is larger than at the Lowest Astronomical Tide (LAT) as was proven before with the hypsometric curve. With help of this relation, the definition of the wet area at every occuring water level can be seen. However, it is common to use a linear interpolation between the water levels HAT and LAT for first estimations. The wet area of the basin is therefore defined by equation G.2

$$A_b = A_{LAT} + \frac{A_{HAT} - A_{LAT}}{TR} h_b \tag{G.2}$$

In which TR is defined as the maximum astronomical tidal range. In this equation A_{LAT} and A_{HAT} are respectively 400 and 1700 m^2 , as was earlier established.

G.1.5. Dam dimensions

The closure dam is the element that will finally close the basin. The dimensions of this dam have to be kept to a minimum to reduce costs. In general a large closure dam is first closed with a small dam and then fortified and reshaped to the final design. The dimensions of this small dam depend on temporary conditions depending on the time of construction. The small dam doesn't have to withstand a 1/1.000 or a 1/10.000 year storm. For example, overflow and overtopping are not important failure mechanisms. However, it is important that it can withstand the forces that are exerted during the construction period and that the probability of failure is acceptable. The dam dimensions depend on 3 measures:

- Crest height
- Slope angle
- Crest width

The crest height of the dam is based on HAT (CD + 10.5 m) plus the significant wave height during the construction period (1.5 m) plus one meter freeboard = CD + 13 m. The angle of the inner and outer dam slope is defined by the stability (angle of repose) of the material used. A large angle is desired, since it reduces the size of the dam. The angle decreases when smaller rocks are used, but is set in this case to a slope of 1:2 which results in a an angle of 27°.
The crest width of the dam is based on several technical aspects:

- Stability of the rocks on the crest
- Discharge due to through flow
- Piping
- Transportation capacity

In the model, the transportation capacity will be the normative aspect. For all others it is assumed no extra width is required. This input consist of two parameters

- The required width of the dam during a vertical closure
- The required width of the dam during a horizontal closure

The crest width during a vertical closure is set at 0 meters. During a horizontal closure 7 meters is required to accomadate dumping trucks and on average another 1 m to create bunds every 100 meters for trucks to turn around. All dam dimension are displayed in figure G.14

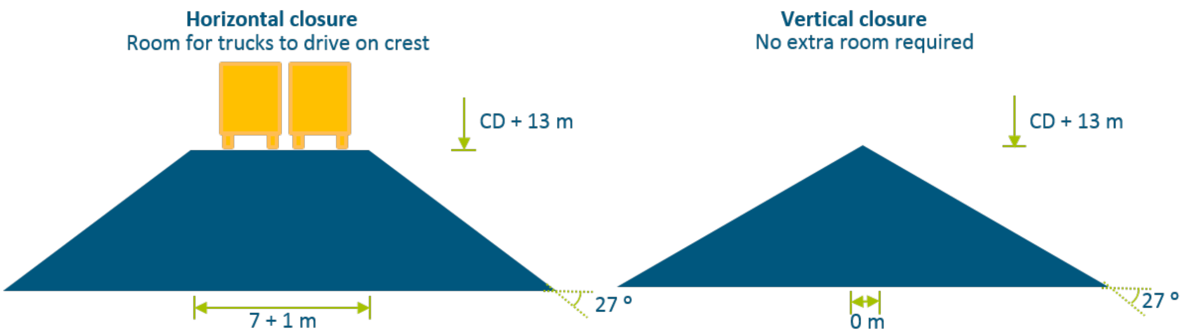


Figure G.14: Assumed dam dimensions (crest height, crest width and slope angle)

G.1.6. Bed protection parameters

The computation for the required volume of bed protection is probabilistic and therefore statistical distributions are used to define the parameters. The accepted failure probability is set at 1/1000 per section during the execution. Furthermore, the number of monte carlo simulations is set at 10.000 to have at least 10 failures in the computation for a reliable result. In table G.1 the values for all distributions is displayed . The choice of the values is explained in this section. For simplicity, a normal distribution is used to model all parameters. The standard deviation is a percentage of the mean to even further reduce the amount of parameters.

Table G.1: Reduced parameters and their representative distributions

Parameter		**Definition**	**Unit**	**Distribution**	**Parameters**	
					μ	σ
D_{n50}	=	D_{50} = Median (nominal) grainsize diameter	$[m]$	Normal	0.0015	$0.1 \cdot \mu$
Ψ_c	=	Shields parameter sand	$[-]$	Normal	0.035	$0.02{\cdot}\mu$
ρ	=	density sand	$[-]$	Normal	2600	$0.01 \cdot \mu$
$cot(\beta_f)$	=	Failure slope after failure	$[-]$	Normal	15	$0.1 \cdot \mu$
β	=	Slope of scour hole before failure	$[1/x]$	Normal	1.5	$0.1 \cdot \mu$
α_t	=	Factor for turbulence	$[m]$	Normal	1.1	$0.02 \cdot \mu$
$u_{a,s}$	=	Average soliciting flow velocity	$[m/s]$	Normal	eq. D.8	$0.03 \cdot \mu$

The shields parameter is set at 0.035 which refers to some movement of the grains with a standard deviation of 2% the range of accepted shields parameters is 0.03 and 0.04. The mean density of dry sand is set at 2600 kg/m^3 with a small standard devation of only 1% since the deviation is different types of sand is small. The mean failure slope after failure $cot(\beta_f)$ is set at 15 with a large standard deviation of 10%, since this varies with different types of soil. a mean of 15 is on the safe side, assuming the sand is not packed. The mean slope before failure β is set at 1.5 since this is the standard angle of repose for sand. A large standard devaition of 10% is still chosen to account for variations in the sand layer. The factor of turbulence α is set at 1.1 and a standard deviation of 0.02 to account for the complete spectrum of turbulence influence (α_t = 1.0-1.2). For the D_{n50}, the next paragraph describes the chosen values.

D_{n50} The soil has been surveyd by Haskoning in 1998 in several places and sediment conditions have been analyzed (Royal Haskoning, 1998a). The outcome of the survey for locations $KB1$ - $KB10$ is shown in figure G.15. The locations can be found in figure G.16. In this study, the D_{n50} is assumed to be equal to the D_{50}. Furthermore, the data is for the upper layer which on average consists of 20 m. This layer is assumed to be present in the complete soil column in the computational model. Locations $KB1$ and $KB2$ are located on the new dam location. The resulting mean D_{50} in these layers is 0.150 mm with a standard deviation of 10%

Sample		KB1	KB2	KB3	KB4	KB5	KB6	KB7	KB8	KB9	KB10
d10	[mm]	0.049	0.085	0.094	0.052	0.002	0.051	0.052	0.046	0.088	0.292
d50	[mm]	0.170	0.120	0.150	0.110	0.030	0.110	0.080	0.100	0.210	0.540
sand	[%]	85	92	95	85	83	88	77	76	92	90
silt	[%]	15	8	5	13	7	12	20	22	8	5
clay	[%]	0	0	0	2	10	0	3	2	0	0
gravel	[%]	0	0	0	0	0	0	0	0	0	5

Figure G.15: Overview of the d10, d50 and sample composition of the upper soil layer (Royal Haskoning, 1998a)

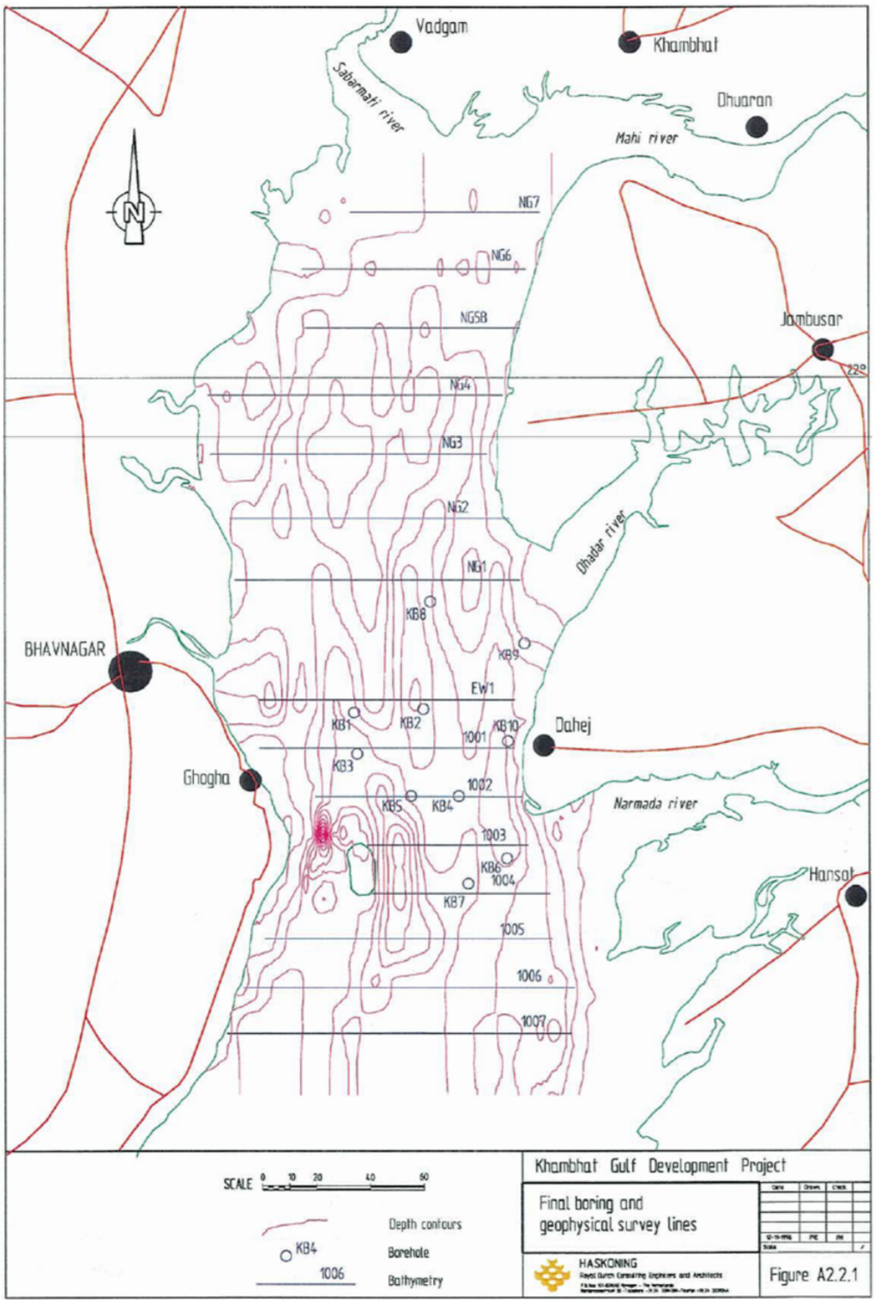


Figure G.16: Final boring and geophysical survey lines (Royal Haskoning, 1998a)

G.1.7. Rock parameters

The computation for the required rock size is probabilistic and therefore statistical distributions are used to define the parameters. The accepted failure probability is set at 1/100 per layer during the execution. Furthermore, the number of monte carlo simulations is set at 1.000 to have at least 10 failures in the computation for a reliable result. The quantity of rock parameters is reduced to the set shown in the table G.2. For simplicity normal distributions have been chosen for all parameters and the standard deviation is a percentage of the mean.

Table G.2: Reduced Rock parameters and their representative distributions

Parameter		**Definition**	**Unit**	**Distribution**	**Parameters**	
					μ	σ
C	=	Chezy coefficient for large rocks	$[m^{0.5}/s]$	Normal	35	$0.05 \cdot \mu$
ψ	=	Shields parameter for rock	[-]	Normal	0.035	$0.1 \cdot \mu$
ρ_r	=	Density rock	[-]	Normal	2600	$0.01 \cdot \mu$
α	=	Slope of the dam head	[°]	Normal	27	$0.1 \cdot \mu$
ϕ	=	Natural slope of the material	[°]	Normal	40	$0.1 \cdot \mu$
F_t, v	=	Correction factor turbulence vertical closure	[-]	Normal	0.36	$0.02 \cdot \mu$
F_t, h	=	Correction factor turbulence horizontal closure	[-]	Normal	0.64	$0.02 \cdot \mu$

Starting with the Chezy coefficient C for large rocks. Large rocks create a lot of friction and therefore are considered very rough. Usual values for this parameter are around the minimum, which is $30m^{0.5}/s$. In this case, 35 is chosen as mean with a 5% standard deviation and a minimum of 30. The shields parameter ψ is set at 0.035 which refers to some initial movement of the rocks (no loss) with a standard deviation of 2% the range of accepted shields parameters is 0.03 and 0.04 which is between the usually applied Shields parameters. The slope of the dam head α was already determined in G.1.5 and is set at 27° , due to execution mistakes a 10% standard deviation is added . The mean natural angle of repose ϕ for rocks is set at 40 °, with a standard deviation of 10%. The factors of horizontal (F_h)and vertical closure (F_v) have been derived in appendix E. However, since a probabilistic approach is chosen, even these deterministically determined parameters have a spreading. The standard deviation is therefore set to 2% of the mean.

G.1.8. Caisson threshold value

Royal Haskoning estimated in 1998 that the rocks sizes would become too large to manage if the flow velocity crossed the 6 m/s value and sluice caissons should be implemented to finish the remaining gap. In the model this is incorporated because it's one of the only strict boundary conditions which Haskoning revealed in the pre-feasibility report from 1998. This threshold value is therefore chosen to be the same in this study.

G.1.9. Equipment parameters

The equipment parameters are defined with use of known capacities of equipment and are shown in table G.3. For the dumping ships, dredging company Van Oord can deliver large split barges which can deliver 3000 tons of material in one dump (Van Oord, 2015). All truck related parameters come from the seamunguem project in Korea, where they used small dumping trucks to close the dam horizontally (Wallingford, 2005). The working time per day also came from this report. In case a cable-way has to be used to close a section vertically, the maximum capacity is 2250 tons per hour. Which more than twice the capacity of cableway used at Brouwershavense Gat in 1972. If needed, this capacity can be increased, but the cost per length measure associated with this increase should also be increased (Wiersema and Broos, 1998)

Table G.3: Equipment parameters

Parameter		**Definition**		**value**	**Unit**
C_{truck}	=	Capacity of a dumping truck	=	15	tons
C_{ship}	=	Capacity of a dumping vessel	=	3000	tons
C_{cable}	=	Capacity of a cableway or bridge system	=	2250	tons/hour
V_T	=	Velocity truck	=	40	km/h
T_{unload}	=	Time to unload a truck	=	60	s
T_{load}	=	Time to load a truck	=	300	s
T_{work}	=	Working time per day	=	22	hours

G.1.10. Elementary costs

Most parameters in the previous sections are determined with a certain accuracy or spreading and logical explanations. However, with the elementary cost parameters this is can't be done. The cost functions can be trusted to translate the required materials and equipment to cost and are comprehensive enough to give a good first estimation of the total cost. However, the discussion lies in defining the elementary cost parameters. Often these elementary cost are well kept secrets of large contracters. On the basis of experience, these values can't be acurately guessed and could be a factor 10 off. However, for the purpose of this study, the values from table G.4 have been established and have been tried to rationalize.

Table G.4: Elementary costs

Symbol	**Cost parameter**	**Cost**	**Unit**
CT	Initial cost of truck	20.000	euro
CTR	Cost of truck rent	2.000	euro/day
CS	Initial cost of ship	1.000.000	euro
CSR	Cost of ship rent	50.000	euro/day
CC	Initial cost of cableway/bridge	50.000.000	euro
CCL	Cost of cableway/bridge	50.000	euro/m
CMR	Cost of rocks	50	euro/m^3
CMG	Cost of gabions	150	euro/m^3
CMCI	Initial cost of caissons	30.000.000	euro
CMC	Cost of caissons	400	euro/m^3
CMO	Cost of overproduction	20	euro/m^3
CBB	Cost of bed protection	70	euro/m^3

G.2. Work area

In this section, the required working area is exemplified to give an indication of the size of the material storage area's (stock piles) and temporary infrastructure (access roads and closures). Some basic sketches are presented for both land connections of the dam, starting with the West side (near Bhavnaghar) in section G.2.1, followed by the east side (near Dahej) in section G.2.2.

G.2.1. West side

Initial situation In the initial situation, it can be notices that a deltaic area exist with a tidal inlet which is dry most of the time (figure G.17). The first usable area would be the area on the left (West) after the bridge. Here a large relatively high work area is presented where no current infrastructure has been built. The section potential location is closer to the Gulf (East) near the entry of the inlet. However, more temporary closures are required to build large storage spaces as well.

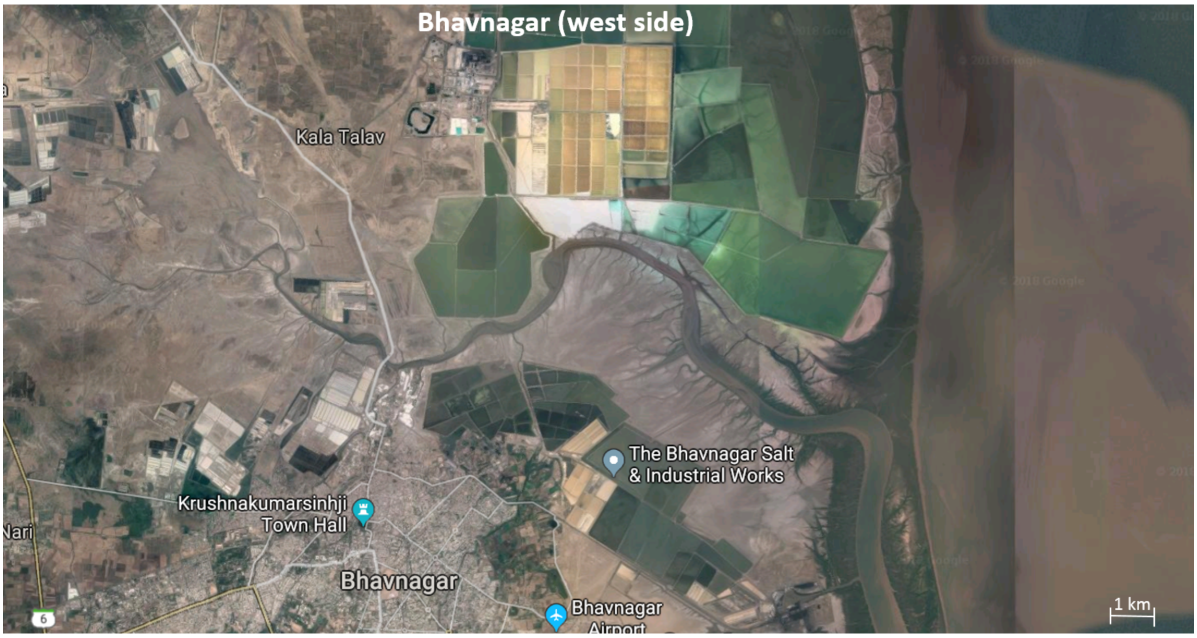


Figure G.17: Initial overview work site at the West side of the Gulf

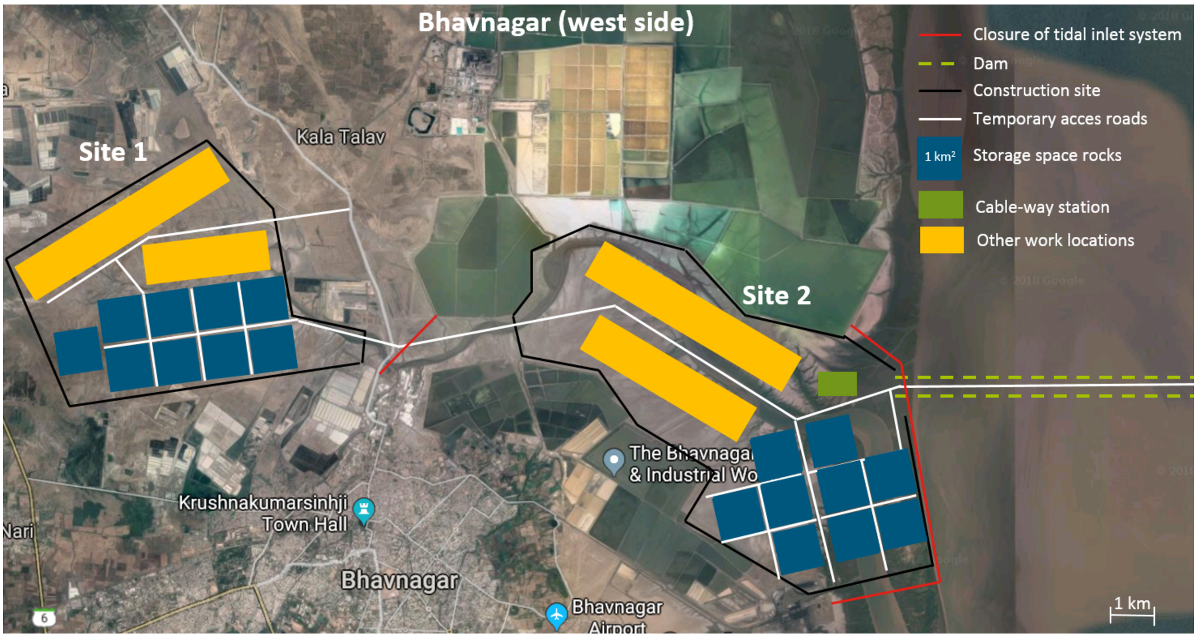


Figure G.18: Overview proposed construction site and temporary works at the West side of the Gulf

Proposed working terrains In figure G.18, both proposed construction sites have been drawn. Preference goes to the Westerly located sited, because less length of temporary closures is required. However, the specifics have been drawn in the figure for both sites. Furthermore, 36 MCM of rock is required to construct the dam. In order to store at least 50% of all material volume (assuming the other 50% is located on the other side), large stockpiles are therefore required if construction must be continuous. Not all rock has to be stockpiled. However, in this sketch it is assumed that this is required. With an average stock pile height of 2 meters, the stockpiles on both sides need to be 9 km^2 each. If 4 meters high can be reached, only half is required and so on. Therefore, several blocks of 1 km^2 have been drawn in figure G.18 to illustrate the stockpiles. Also, a possible cable-way station has been drawn together with other possible work locations. It can be argued if all this storage space is required. Keeping a constant check on quarry outputs should minimize stock pile requirements. With correct planning, using only 1 km^2 or less for stockpiling should be enough.

G.2.2. East side

On the East side, also two locations can be proposed (figures G.19 and G.20). The first location is directly behind the salt banks (large industry) on the agricultural land. These farmers will not be able to produce for several years. However, this could way up against extra cost for building a site more to the West (in the delta). This would mean building temporary dikes to close a deltaic area for construction. Again, stock pile sizes are drawn in the figure.

Figure G.19: Initial overview work site at the East side of the Gulf

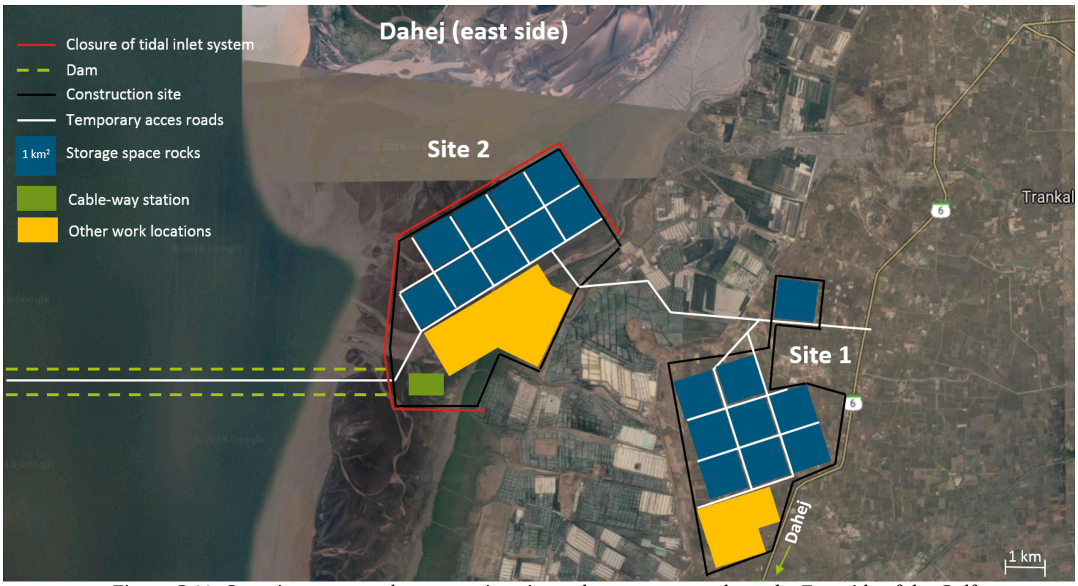


Figure G.20: Overview proposed construction site and temporary works at the East side of the Gulf

H

Results Case study 1

In this chapter, the results of case study 1 are described in detail, including all related figures.

H.1. Cases tested and summary results

In total 4 different case studies are defined which are tested with a 1, 3, 5 and 8 cross sectional model (figures H.1 to H.4).

- **Case 1:** Horizontal from shoreline to middle, remaining part vertical
- **Case 2:** Horizontal flats first, remaining part vertical
- **Case 3:** Vertical from bottom up, remaining part horizontal
- **Case 4:** Vertical channels first, remaining part horizontal

Also variation can occur in the elementary cost, so the reference case and three cost variations are tested for all options:

1. Reference case
2. Relatively higher material prices
3. Relatively higher equipment prices
4. Relatively higher bed protection prices

Within these case studies, every time 6 strategic options are tested:

- **Option 1:** Completely horizontal
- **Option 2:** 20% vertical, 80% horizontal
- **Option 3:** 40% vertical, 60% horizontal
- **Option 4:** 60% vertical, 40% horizontal
- **Option 5:** 80% vertical, 20% horizontal
- **Option 6:** Completely vertical

H.1.1. Summary results case study 1

Table H.1: Summary results case study 1: least cost options for all tested cases

Case 1: 1 Horizontal - 2 Vertical, regular	**1 CS**	**3 CS's**	**5 CS's**	**8 CS's**
Reference prices	6	5	5	5
Increased bed protection prices	6	5	5	5
Increased equipment prices	5	4/5	5	5
Increased material prices	6	5	5	5/6
Case 2: 1 Horizontal - 2 Vertical, flats first				
Reference prices	6	5	6	4
Increased bed protection prices	6	5	6	5
Increased equipment prices	5	4/5	4/5/6	4
Increased material prices	6	5	6	4/5/6
Case 3: 1 Vertical - 2 Horizontal, regular				
Reference prices	3	6	6	6
Increased bed protection prices	6	6	6	6
Increased equipment prices	3	6	6	6
Increased material prices	6	6	6	6
Case 4: 1 Vertical - 2 Horizontal, channels first				
Reference prices	3	6	6	6
Increased bed protection prices	3/6	3/6	6	6
Increased equipment prices	3	3	6	6
Increased material prices	6	6	6	6

H.1.2. Cross sectional models tested

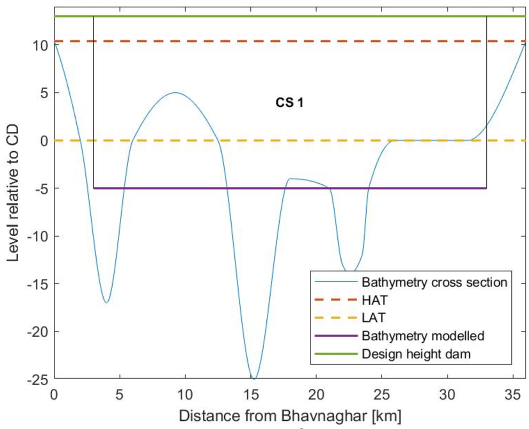


Figure H.1: Single cross section

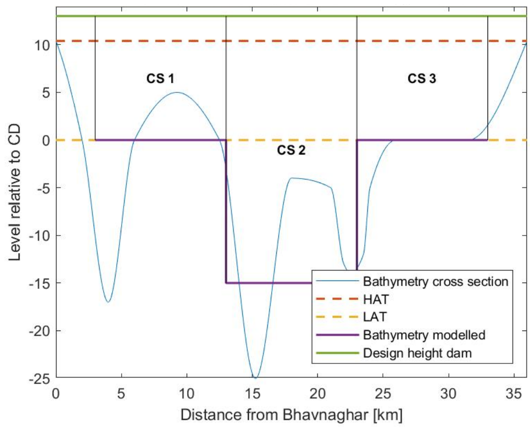


Figure H.2: Three cross sections

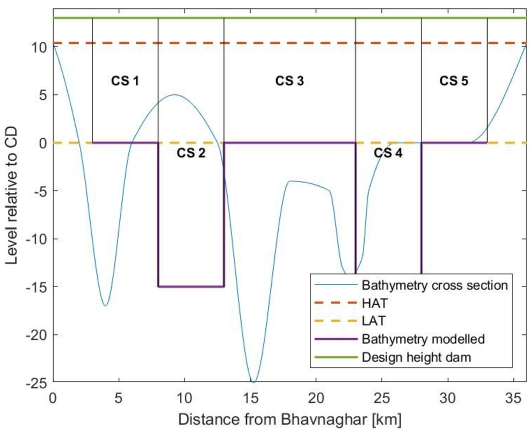


Figure H.3: Five cross sections

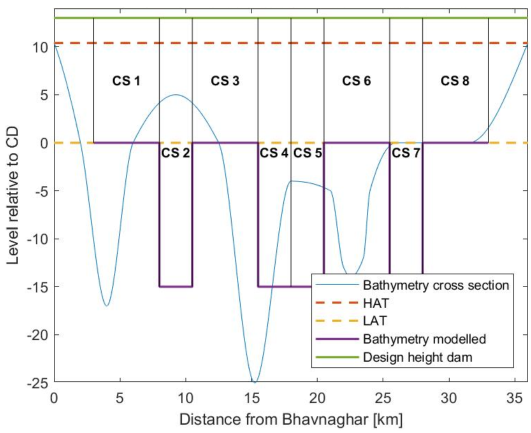


Figure H.4: Eight cross sections

H.2. Results: Case1: 1 horizontal, 2 vertical, regular

In this subsection, the results are displayed from case 1. In this case a horizontal closure is performed first after which the remaining percentage in the strategic option is closed vertically. The horizontal part is performed from the shoreline to the middle and is referred as a *regular* closure with respect to a *flats first* or *channels first* closure strategy. In figures H.5 to H.10, the resulting strategies for all six options are visualized for the 8 cross sectional model. Models with 1, 3 and 5 cross sections are executed in exactly the same way but with fewer cross sections. In these figures, red is horizontal and green vertical. The darkness of the color represents the phase and the textual inscription presents all information about the section: Phase(1..n) - closure method(H/V) - capacity ($\cdot$ 1000 m^3/day). The total costs of each strategy is diplayed in figures H.11 to H.18

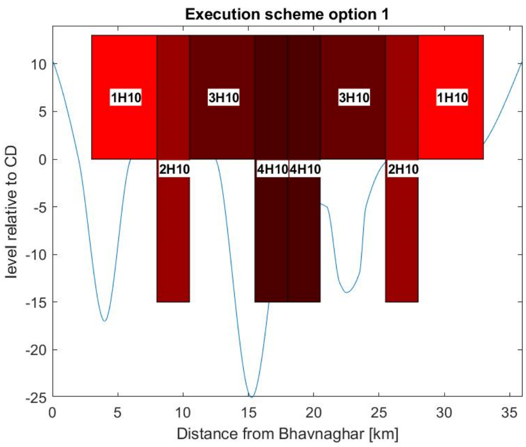


Figure H.5: Execution scheme option 1: fully horizontal

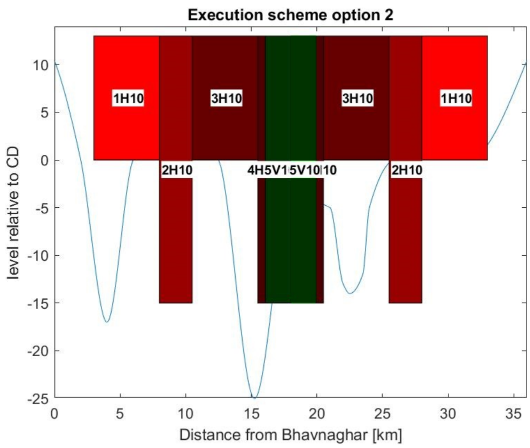


Figure H.6: Execution scheme option 2: 20% vertical

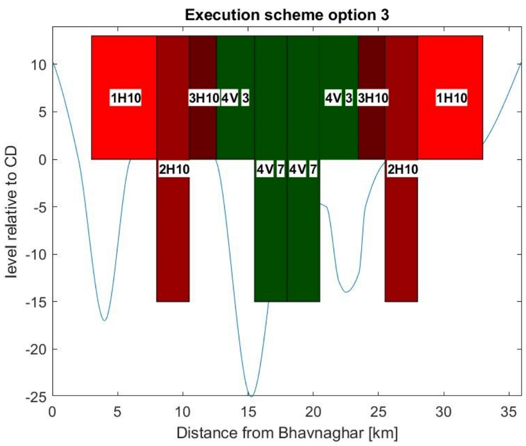


Figure H.7: Execution scheme option 3: 40% vertical

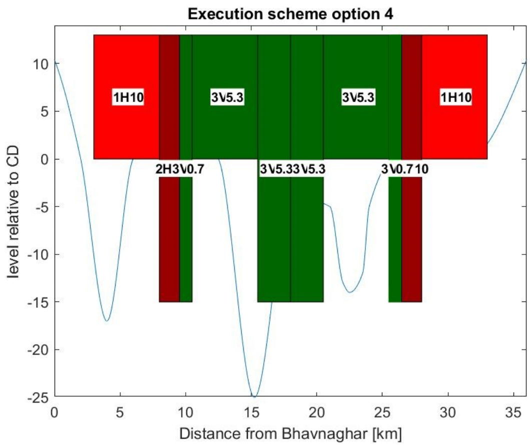


Figure H.8: Execution scheme option 4: 60% vertical

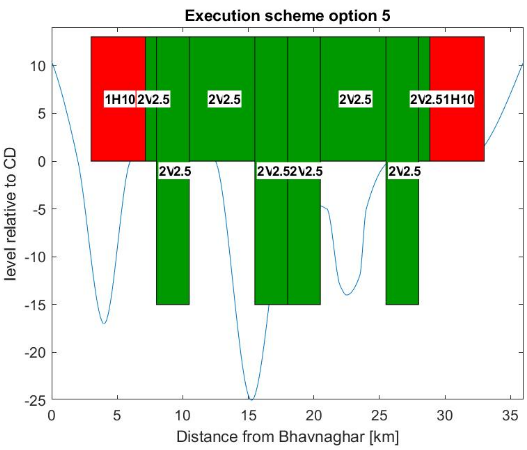


Figure H.9: Execution scheme option 5: 80% vertical

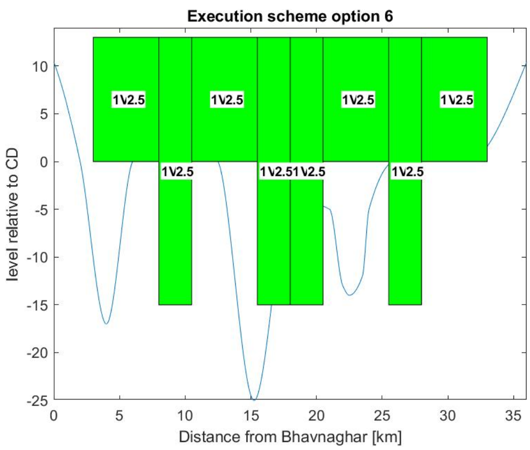


Figure H.10: Execution scheme option 6: fully vertical

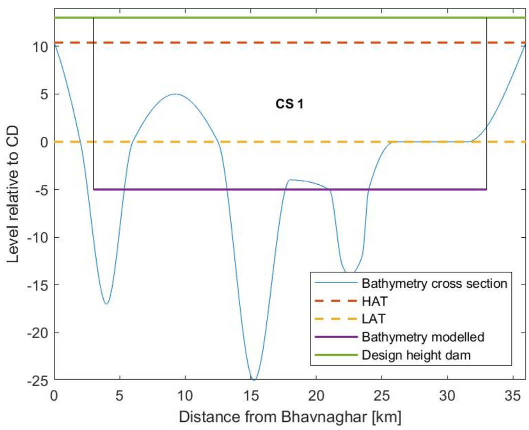


Figure H.11: Single cross section

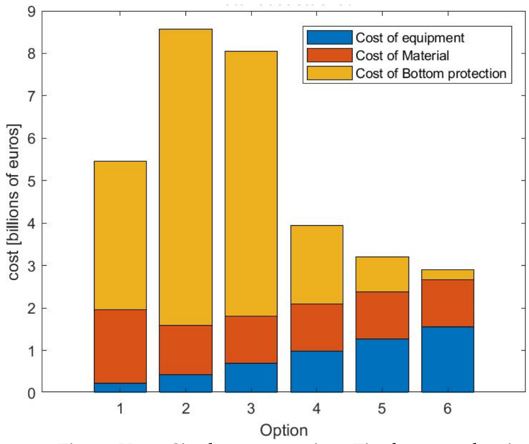


Figure H.12: Single cross section: Final cost evaluation

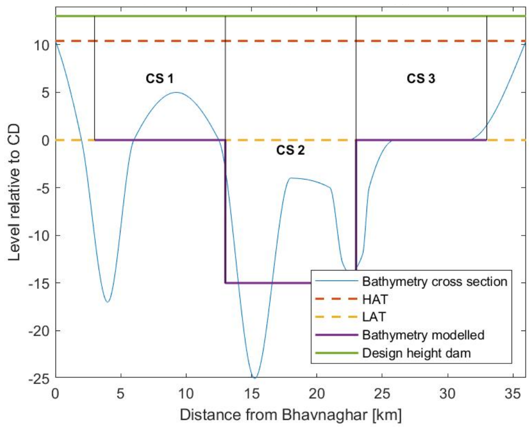


Figure H.13: Three cross sections

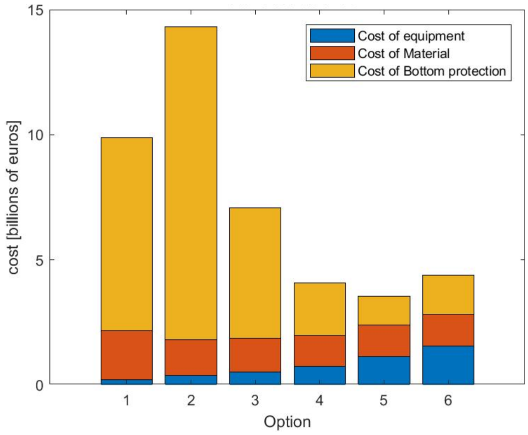


Figure H.14: Three cross sections: Final cost evaluation

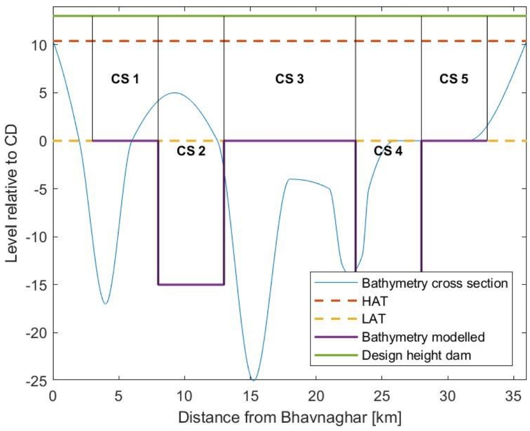


Figure H.15: Five cross sections

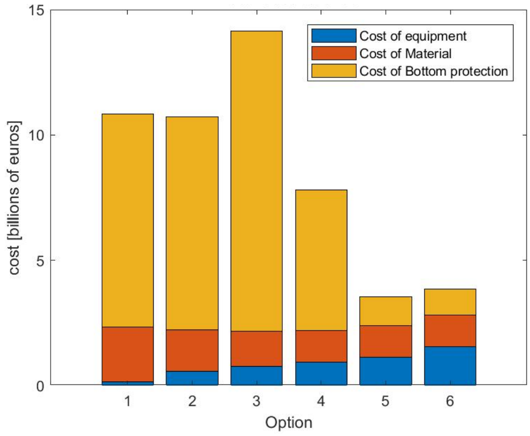


Figure H.16: Five cross sections: Final cost evaluation

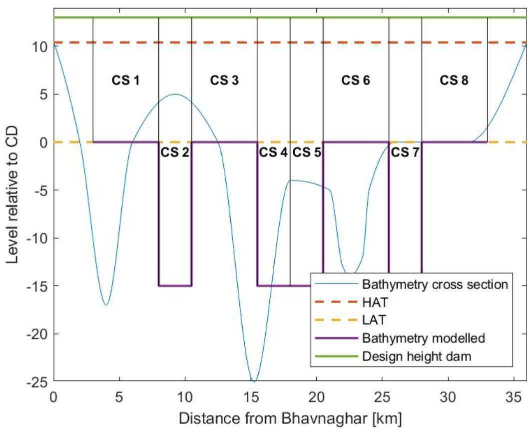


Figure H.17: Eight cross sections

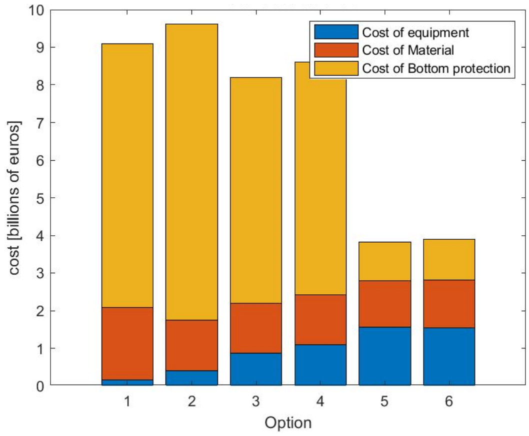


Figure H.18: Eight cross sections: Final cost evaluation

H.2.1. Discussion

The least cost option in all four cases for all cross sectional models is displayed in table H.2 based on the final evaluations presented in figures H.5 to H.18. These results are discussed in this section:

Table H.2: Least cost options case 1: 1 horizontal, 2 vertical, regular

Case 1: 1 Horizontal - 2 Vertical, regular	**1 CS**	**3 CS's**	**5 CS's**	**8 CS's**
Reference prices	6	5	5	5
Increased bed protection prices	6	5	5	5
Increased equipment prices	5	4/5	5	5
Increased material prices	6	5	5	5/6

First, it can be noticed that the required quantity of bed protection varies significantly by changing the strategy in all models. The influence of this cost factor can directly be concluded as the most important. The equipment cost also varies significantly with respect to the material cost, which is the least varying cost factor.

In all models, the cost of bed protection increases exponentially with increasing percentage of horizontal closure. However in almost all models, option 1 (100% horizontal) defies this relation because it requires less bed protection than option 2 (80% horizontal). This exception can be explained by the increased cost of bed protection at the end of the closure. The last 20% is closed vertically under high velocity circumstances. Since the length of protection required in this last section is used accros the complete width of this section, relatively more area of bed protection is required. A horizontal closure requires only half that area (although the length required could me more, the resulting area is less). In the 5 cross-sectional model, The largest peak in bed protection switched from option 2 to option 3 (60% horizontal, 40% vertical). This can again be explained by vertical closure at the end, which happens exactly in the shallow part in the middle of the bay. High flow velocities in shallow water cause large scour holes, since the discharge can't spread towards deeper parts (advantage of velocity dispersion with increasing depth through continuity of flow). The peak costs are therefore almost 15 billion euro, which compared to the single cross sectional model is a factor 1.5 larger. In the 8 cross-sectional model, options 1 to 4 experience the same problem of creating a high flow velocity on the tidal flats. Because there are more channels and flats included each option averages the final vertical closure accros a channel and a flat. The result is that they all are expensive (about 9 billion euros), but are relatively cheaper than option 2 in the 3 cross sectional model, or option 3 in the 5 cross-sectional model (15 billion euros).

The least bed protection cost is in almost all models found in option 6 (completely vertical). However, in all models with more than a single cross section, option 5 (80 % vertical and 20% horizontal) requires the same or even less bed protection. This can also be explained by the same phenomenon that was stated earlier; horizontal closures require only half the total area of bed protection with respect to the cross section width times the length required (see calculation method of the bed protection area in appendix D) . In this option, the area of bed protection is reduced at the start of the closure, where in all multi-crossectional models a horizontal closure is executed on the tidal flats. This reduces the area required without increasing the flow velocity significantly.

The result of the development of closing the flats first horizontally influences the least cost option in the 3 and 5 cross-sectional models. In some cases option 5 and even option 4 is the cheapest strategy. Bed protection needs are still low because the flow velocity has not increased significantly yet and the length of the cable-way required is reduced. This strategy was therefore also proposed by Royal Haskoning in 1998.

H.2.2. Conclusion

The conclusion with respect to the posed hypothesis is positive. Significant changes occur by increasing the amount of cross sections. However, this largely depends on the quantity of bed protection required which also shows large variations within the multi-cross sectional models. Based on only these results, the hypothesis can't be confirmed. However, the potential for possible confirmation is present.

H.3. Results: Case 2: 1 horizontal, 2 vertical, Flats first

In this subsection, the results are displayed from case 2. In this case a horizontal closure is performed first after which the remaining percentage in the strategic option is closed vertically. The horizontal part is performed from tidal flats to the channels and is referred to as a *flats first* closure with respect to a *regular* or *channels first* closure strategy. In figures H.19 to H.24, the resulting strategies for all six options are visualized for the 8 cross sectional model. Models with 1, 3 and 5 cross sections are executed in exactly the same way but with fewer cross sections. In these figures, red is horizontal and green vertical. The darkness of the color represents the phase and the textual inscription presents all information about the section: Phase(1..n) - closure method(H/V) - capacity ($\cdot$ 1000 m^3/day). The total costs of each strategy is diplayed in figures H.25 to H.32

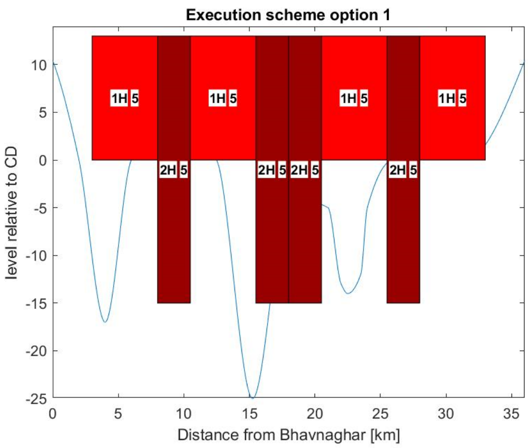


Figure H.19: Execution scheme option 1: fully horizontal

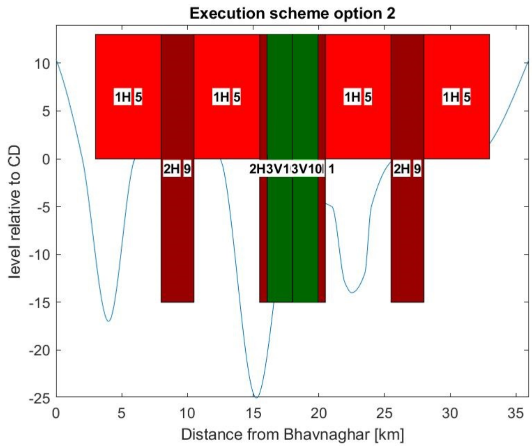


Figure H.20: Execution scheme option 2: 20% vertical

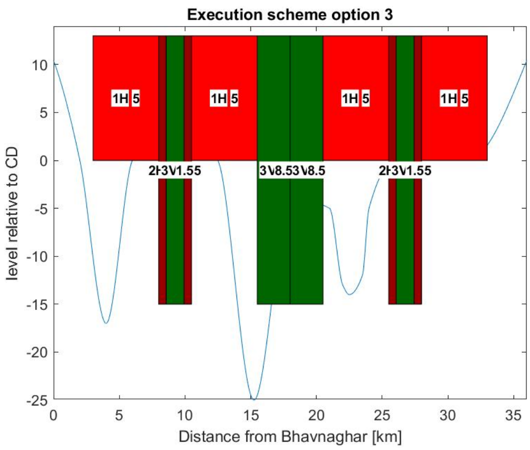


Figure H.21: Execution scheme option 3: 40% vertical

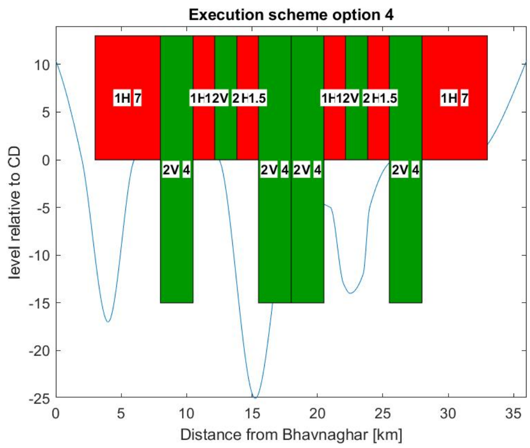


Figure H.22: Execution scheme option 4: 60% vertical

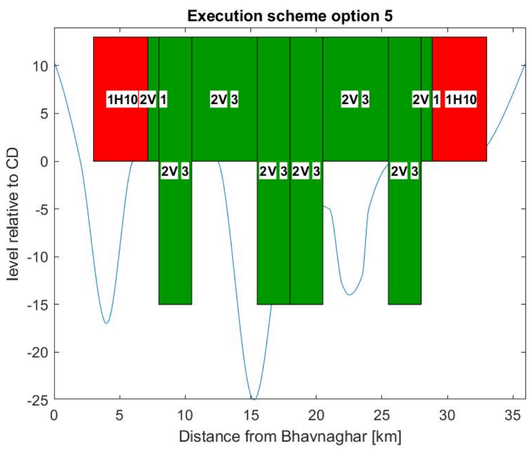


Figure H.23: Execution scheme option 5: 80% vertical

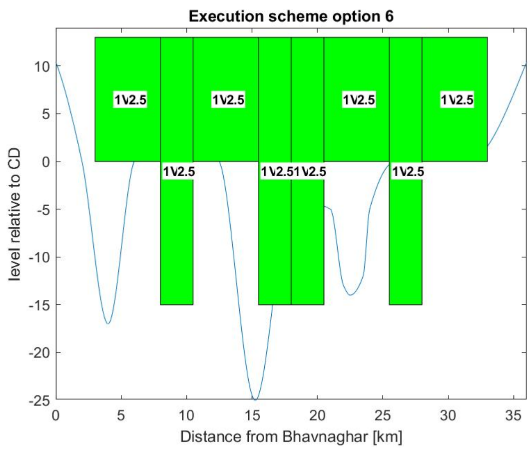


Figure H.24: Execution scheme option 6: fhlly vertical

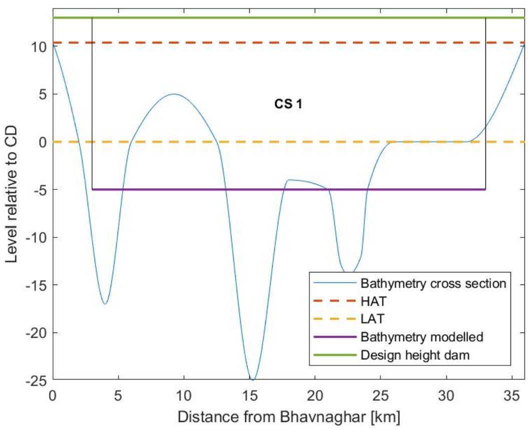


Figure H.25: Single cross section

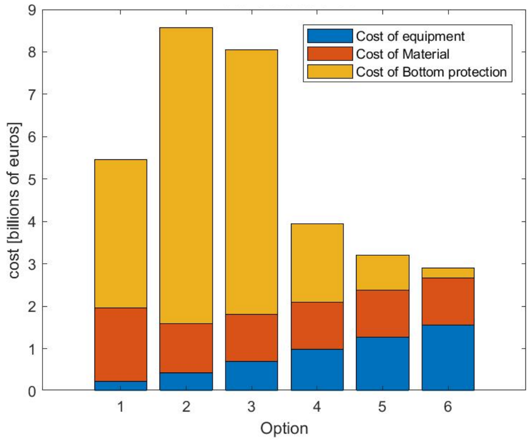


Figure H.26: Single cross section: Final cost evaluation

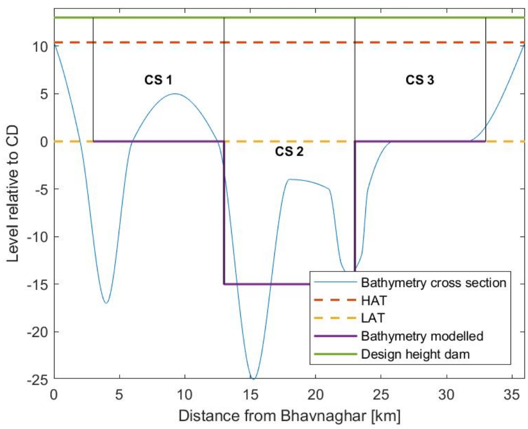


Figure H.27: Three cross sections

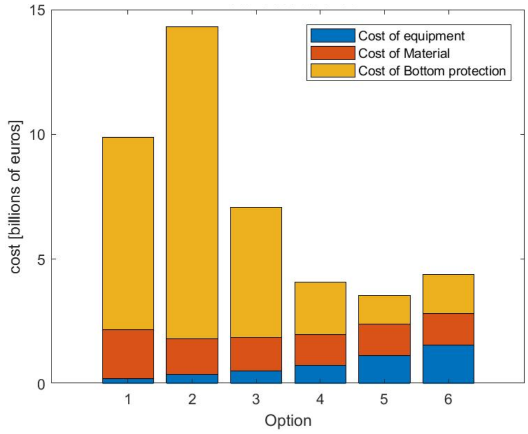


Figure H.28: Three cross sections: Final cost evaluation

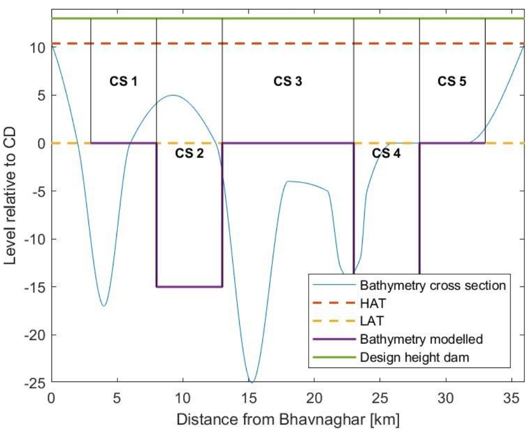


Figure H.29: Five cross sections

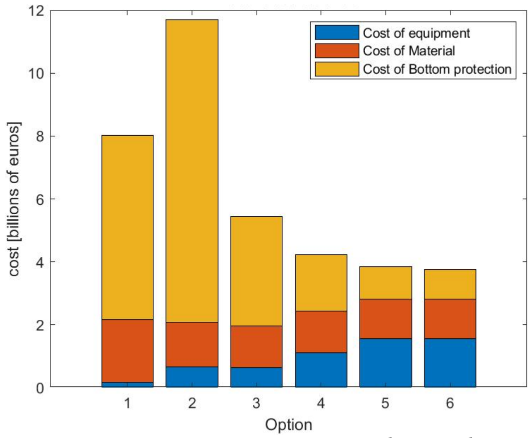


Figure H.30: Five cross sections: Final cost evaluation

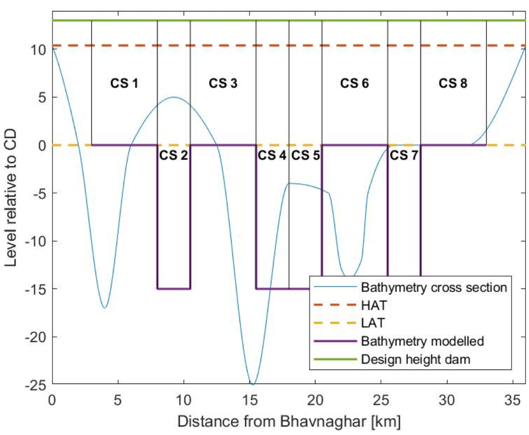


Figure H.31: Eight cross sections

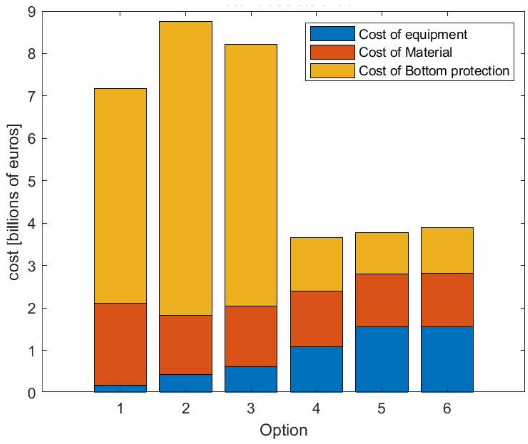


Figure H.32: Eight cross sections: Final cost evaluation

H.3.1. Discussion

The least cost option in all four cases for all cross sectional models is displayed in table H.3 based on the final evaluations presented in figures H.19 to H.32. These results are discussed in this section:

Table H.3: Least cost options case 1: 1 horizontal, 2 vertical, flats first

Case 2: 1 Horizontal - 2 Vertical, flats first	**1 CS**	**3 CS's**	**5 CS's**	**8 CS's**
Reference prices	6	5	6	4
Increased bed protection prices	6	5	6	5
Increased equipment prices	5	4/5	4/5/6	4
Increased material prices	6	5	6	4/5/6

First, it can be stated that the same conclusions can be drawn about general behavior of the model with respect to the previous case: The bed protection is still the most influential component. The previous conclusion that closing the flats first horizontally is beneficial up to some point can be confirmed again in the 3, 5 and 8 cross-sectional models. Options 4 and 5 are the cheapest because they close the flats first with minimal increase in flow velocity and reduce the size of the cableway required significantly. This is again not visible in the single cross-sectional model. The interesting question to ask here is to what point is closing the flats first beneficial and from whereon will it be more expensive.

H.3.2. Conclusion

The hypothesis can be now be confirmed for this case because all multi cross-sectional models show another least cost option than the single cross-sectional model.

H.4. Results: Case 3: 1 vertical, 2 horizontal, regular

In this subsection, the results are displayed from case 3. In this case a vertical closure is performed first after which the remaining percentage in the strategic option is closed horizontally. The vertical part is performed accros all cross sections at the same time and is referred to as a *regular* closure with respect to a *flats first* or *channels first* closure strategy. In figures H.33 to H.38, the resulting strategies for all six options are visualized for the 8 cross sectional model. Models with 1, 3 and 5 cross sections are executed in exactly the same way but with fewer cross sections. In these figures, red is horizontal and green vertical. The darkness of the color represents the phase and the textual inscription presents all information about the section: Phase(1..n) - closure method(H/V) - capacity ($\cdot$ 1000 m^3/day). The total costs of each strategy is diplayed in figures H.39 to H.46

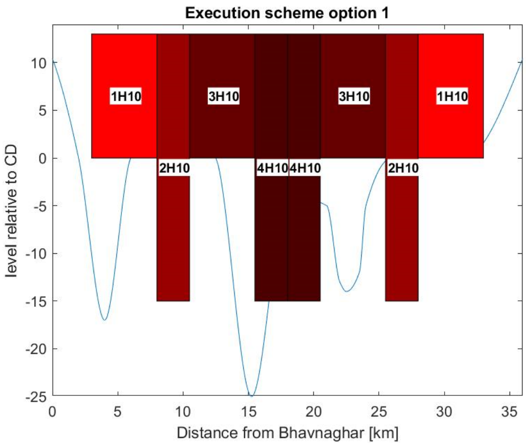


Figure H.33: Execution scheme option 1: fully horizontal

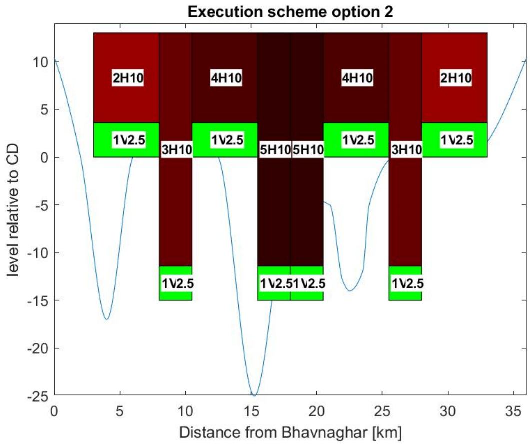


Figure H.34: Execution scheme option 2: 20% vertical

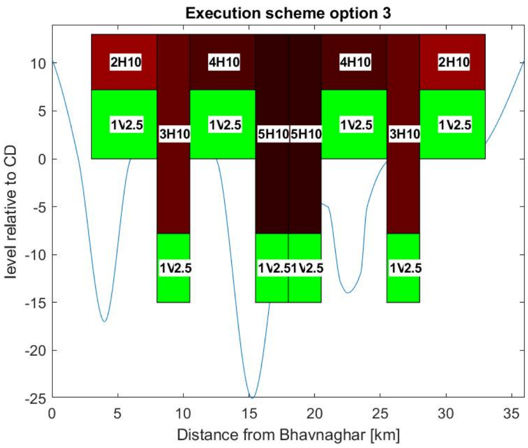


Figure H.35: Execution scheme option 3: 40% vertical

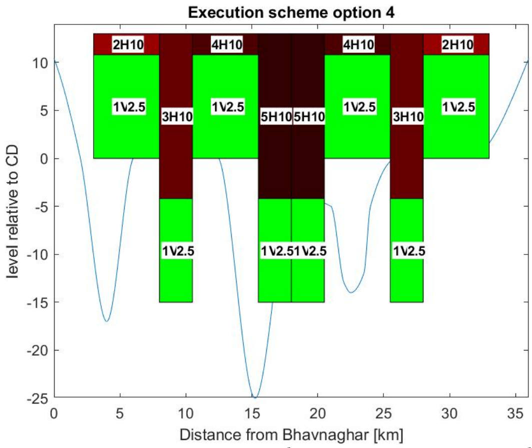


Figure H.36: Execution scheme option 4: 60% vertical

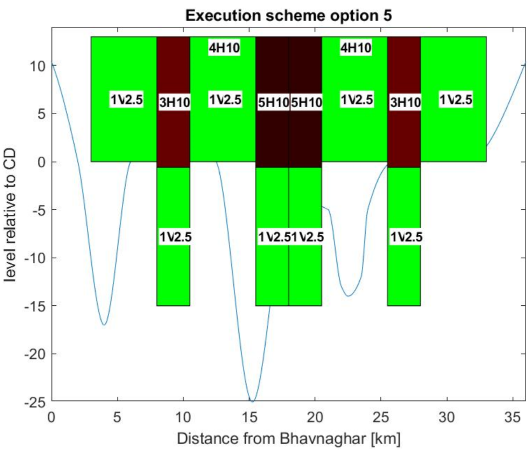


Figure H.37: Execution scheme option 5: 80% vertical

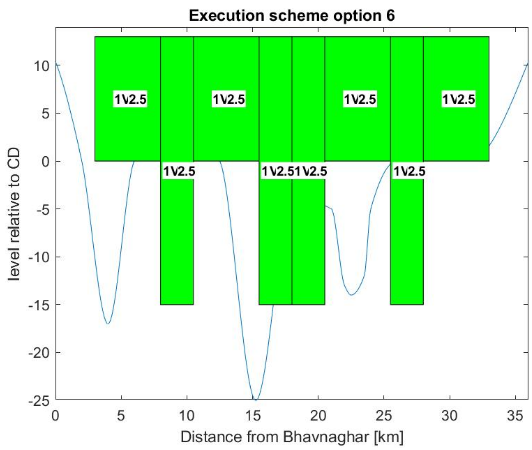


Figure H.38: Execution scheme option 6: fhlly vertical

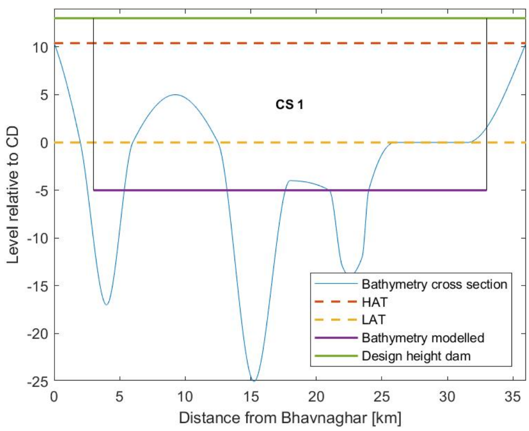


Figure H.39: Single cross section

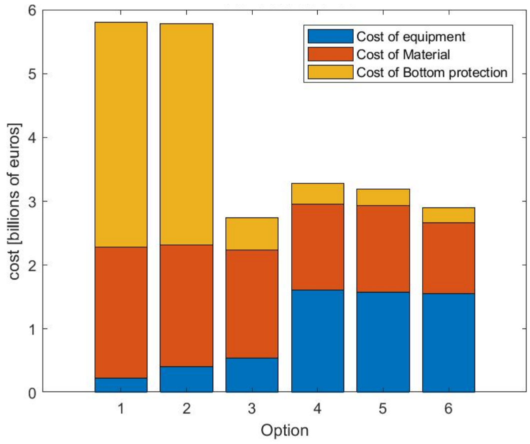


Figure H.40: Single cross section: Final cost evaluation

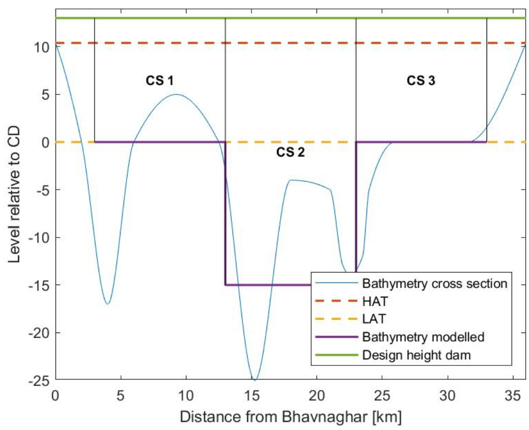


Figure H.41: Three cross sections

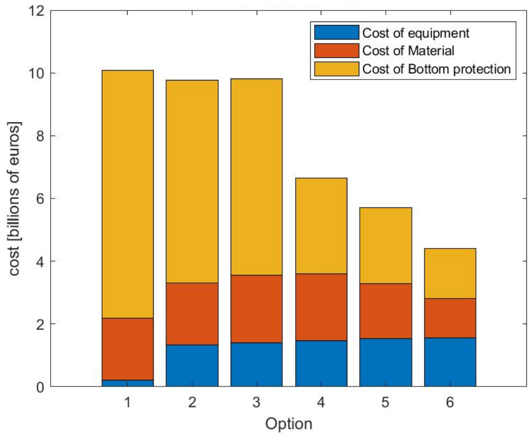


Figure H.42: Three cross sections: Final cost evaluation

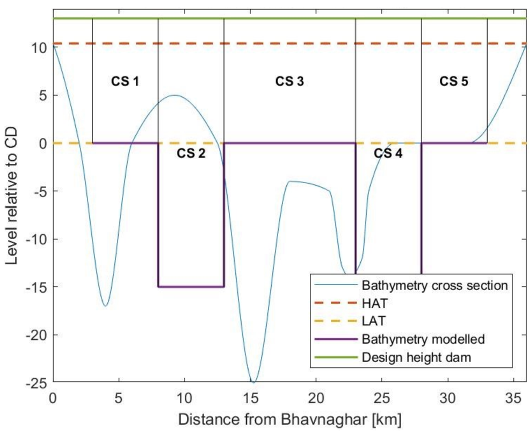


Figure H.43: Five cross sections

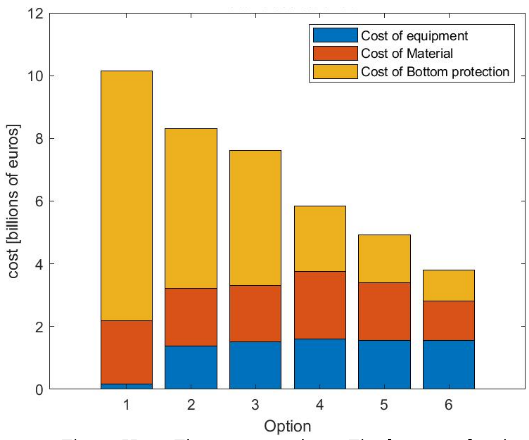


Figure H.44: Five cross sections: Final cost evaluation

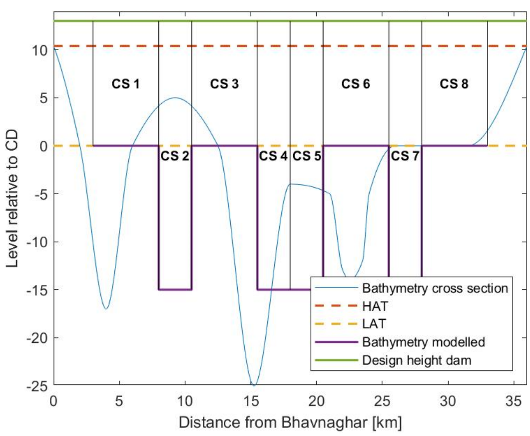


Figure H.45: Eight cross sections

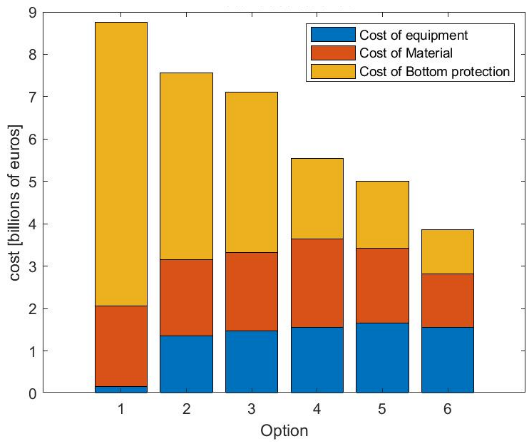


Figure H.46: Eight cross sections: Final cost evaluation

H.4.1. Discussion

The least cost option in all four cases for all cross sectional models is displayed in table H.5 based on the final evaluations presented in figures H.33 to H.46. These results are discussed in this section:

Table H.4: Least cost options case 3: 1 vertical, 2 horizontal, regular

Case 3: 1 vertical, 2 horizontal, regular	1 CS	3 CS's	5 CS's	8 CS's
Reference prices	3	6	6	6
Increased bed protection prices	6	6	6	6
Increased equipment prices	3	6	6	6
Increased material prices	6	6	6	6

In this case, it is clear that option 3 is beneficial in the single cross sectional model. Although material costs are high, equipment costs are still relatively low due to the partial vertical closure up to the point where ships can still operate. Since a partial vertical closure is always beneficial for the bed protection (continuity), this cost factor is also very small in this option. However, this changes in all other models, because the deeper channels "profit" less from the velocity dispersion through continuity, because the dam levels are relatively low compared to the depth. Furthermore, the channels are closed horizontally which takes a long time due to the increased size of the dam. During this time, the scour hole can develop and causes bed protection volumes to significantly increase with respect to the single cross-sectional model.

H.4.2. Conclusion

The hypothesis can be confirmed for this case as well, since the influence of a channel is significant on the cost of bed protection. small percentage partial vertical closures (20-40%) are therefore not recommended.

H.5. Results: Case 4: 1 vertical, 2 horizontal, channels first

In this subsection, the results are displayed from case 4. In this case a vertical closure is performed first after which the remaining percentage in the strategic option is closed horizontally. The vertical part is performed in the channel first and (if filled up) spreads to the flats of all cross sections. This type of closure is is referred to as a *channels first* closure with respect to a *flats first* or *regular* closure strategy. In figures H.47 to H.52, the resulting strategies for all six options are visualized for the 8 cross sectional model. Models with 1, 3 and 5 cross sections are executed in exactly the same way but with fewer cross sections. In these figures, red is horizontal and green vertical. The darkness of the color represents the phase and the textual inscription presents all information about the section: Phase(1..n) - closure method(H/V) - capacity ($\cdot$ 1000 m^3/day). The total costs of each strategy is diplayed in figures H.53 to H.60

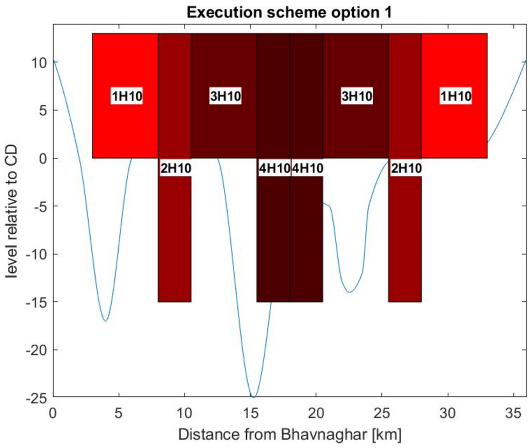


Figure H.47: Execution scheme option 1: fully horizontal

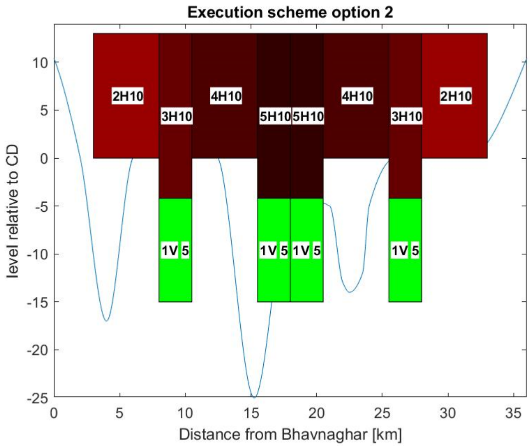


Figure H.48: Execution scheme option 2: 20% vertical

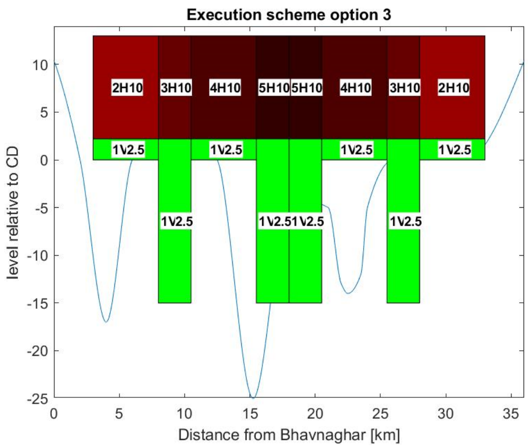


Figure H.49: Execution scheme option 3: 40% vertical

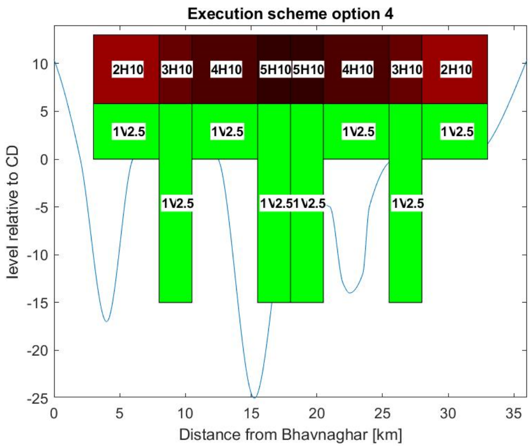


Figure H.50: Execution scheme option 4: 60% vertical

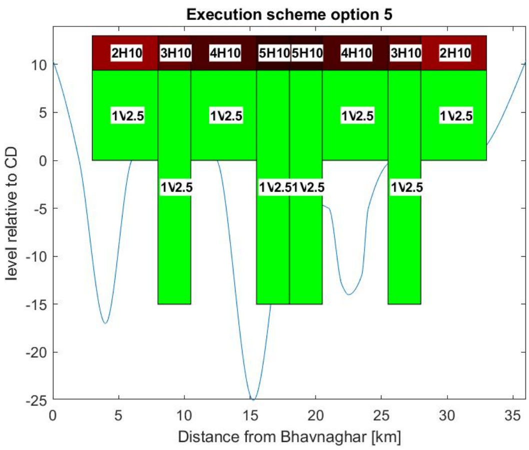


Figure H.51: Execution scheme option 5: 80% vertical

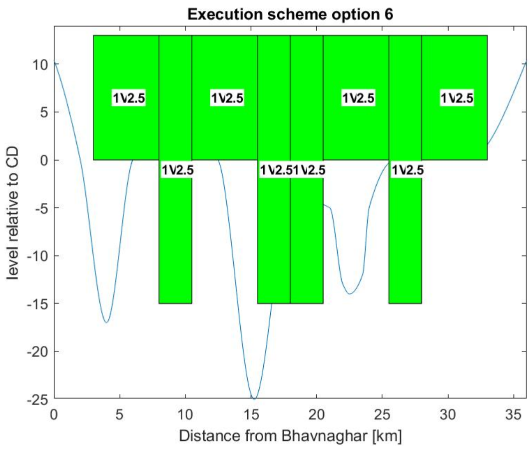


Figure H.52: Execution scheme option 6: fhlly vertical

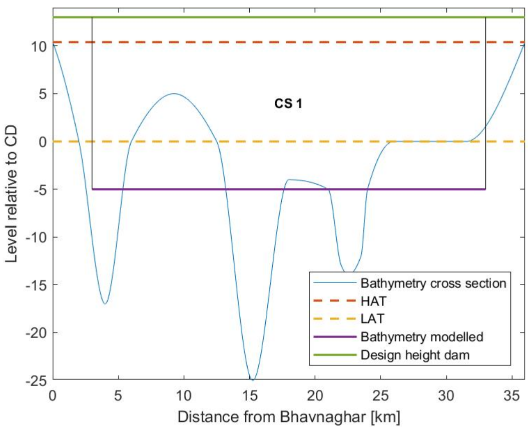


Figure H.53: Single cross section

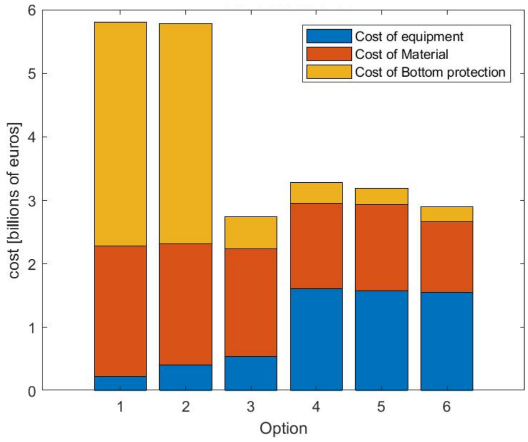


Figure H.54: Single cross section: Final cost evaluation

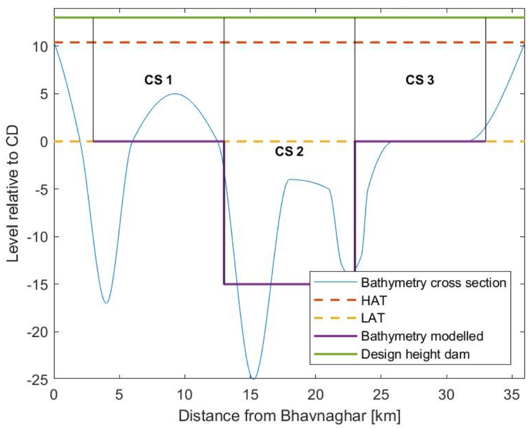


Figure H.55: Three cross sections

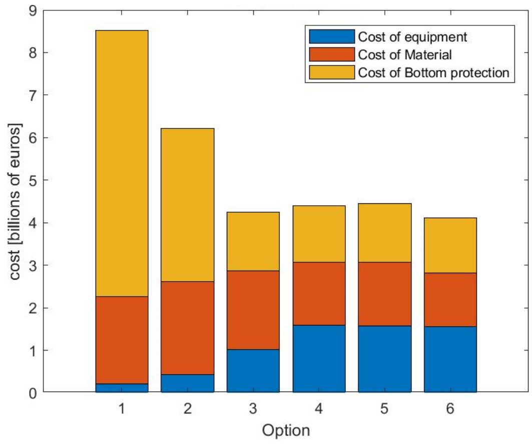


Figure H.56: Three cross sections: Final cost evaluation

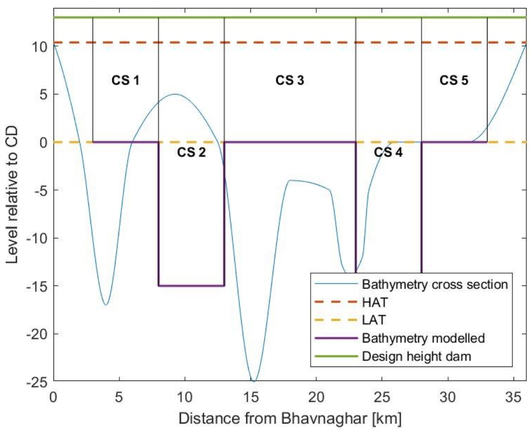


Figure H.57: Five cross sections

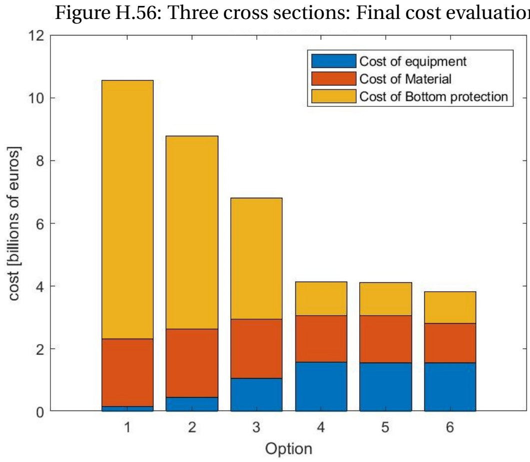


Figure H.58: Five cross sections: Final cost evaluation

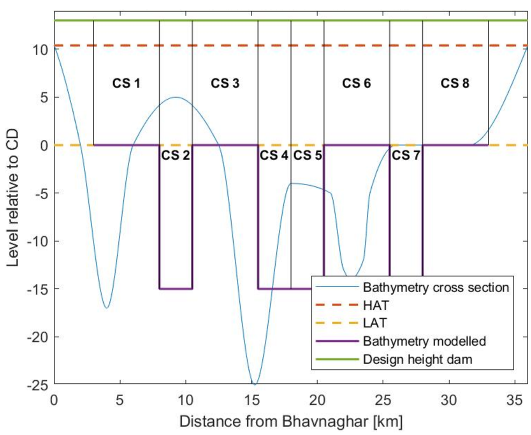


Figure H.59: Eight cross sections

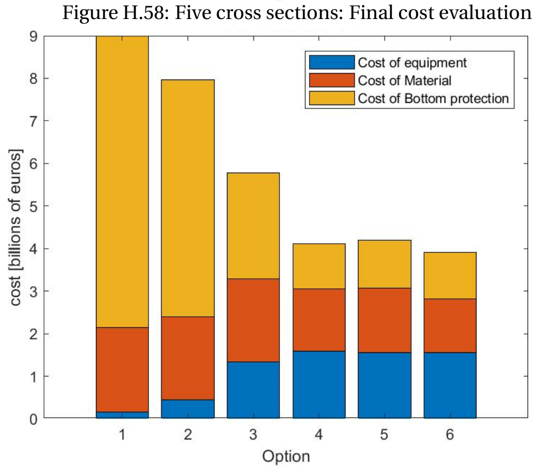


Figure H.60: Eight cross sections: Final cost evaluation

H.5.1. Discussion

The least cost option in all four cases for all cross sectional models is displayed in table H.5.

Table H.5: Least cost options case 3: 1 vertical, 2 horizontal, regular

Case 3: 1 vertical, 2 horizontal, regular	**1 CS**	**3 CS's**	**5 CS's**	**8 CS's**
Reference prices	3	6	6	6
Increased bed protection prices	3/6	3/6	6	6
Increased equipment prices	3	3	6	6
Increased material prices	6	6	6	6

First, it can be stated that the same conclusions can be drawn about general behavior of the model with respect to the previous case with respect to the single cross-sectional model. The changes in the multi cross-sectional model are significant with respect to case 3. This can be explained by the quick initial filling of the channels after which the situation is almost the same as in the single cross-sectional model. However, due to the relative small depth, the bed protection required is still larger in the multi cross-sectional models. Furthermore, with a small increase in the prices of bed protection or material, the cheapest option changes from 3 to 6. With a small increase in equipment prices, options 3 stays the cheapest and the relative difference increases. However, in the 3 cross-sectional model, option 3 also becomes the cheapest, confirming that the price differences (although small) influence the least cost option more than the amount of cross sections used in this case.

H.5.2. Conclusion

The hypothesis can't be confirmed completely in this case, since the least cost options do not change significantly enough. This is because the price differences (although small) influence the least cost option more than the amount of cross sections used in this case.