Preparing a long term management plan for the future of the Slufter

An analysis of the functional, spatial and geotechnical possibilities

Master of Science Thesis

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Summary

The Slufter is a large scale disposal facility for contaminated dredged material from the Dutch rivers, channels and harbor basins. It is located in the Rotterdam harbor, on the southwestern tip of the Maasvlakte. Since the Slufter was built in 1987, the supply of dredged material has decreased significantly. As a result, the prognosis is that it will take a long time before its full capacity of 150 million cubic meters is utilized. At the moment, approximately 50% of the basin has been filled.

The overcapacity and surplus of space raises questions on the applicability of this area for purposes other than storage of dredged material. Therefore, the Port of Rotterdam Authority is in need of a long term management plan in order to move towards an early dismantlement of the basin and in the meantime make full use of the area’s potential. In this research, preparations are made for the establishment of a long term management plan by analyzing the functional, spatial and geotechnical possibilities of the Slufter. The aim has been to find potential future functions and ways to create land in the Slufter.

For the functional analysis a brainstorm session has been organized to generate ideas and to formulate possible future functions for the Slufter. A criteria analysis has been done to downsize the large list of functions generated during the brainstorm session, and the remaining functions have been further analyzed and evaluated.

For the short term the best opportunities can be seen for renewable energy projects, such as solar panels, windmills and algae breeding. However, fish breeding, greenhouse cultivation and a location to accommodate Bevi-companies (handling and storage of dangerous substances) are also functions with potential. In addition, the opportunities for a nature and/or birds reserve are clearly present.

For the long term, after complete dismantlement of the Slufter as a disposal facility, harbor functions such as empty depot or storage of chemicals provide the best opportunities since these types of functions offer more certainty of a positive return on investment.

In the spatial and geotechnical analysis two alternatives for creating land have been considered; the first alternative analyzes the feasibility of a retaining structure, and the second alternative focusses on the natural process of ripening clay.

The purpose of a retaining structure is to provide a physical boundary between the storage basin and the new function(s). The geotechnical feasibility of a retaining structure built on top of the 30m thick layer of dredged material, in the shape of an embankment, has been proven. However, the embankment would require large dimensions and thereby large quantities of construction material, and thus a large investment. Moreover, the large dimensions
limit the amount of surface area that can be obtained. There are techniques available with which an embankment with steeper slopes can be realized. One of these techniques is described in this report; an embankment built with a reinforcement of geosynthetic material. As a result of the steeper slopes less construction material is needed, and the construction costs can be decreased considerably. However, additional research on the geotechnical feasibility and the execution method of this type of structure is required.

A more cost effective solution is the clay ripening alternative. The geotechnical possibilities for building on top of a layer of ripened clay are limited in the short term. Therefore, a long term approach is recommended. This requires a shift from wet storage to dry storage. The current sheet of water on top of the dredged material has to be lowered in order to initiate a process of ripening of the top layer. Designated areas can be elevated further by spraying the incoming supply of contaminated dredged material on top of the first crust of ripened clay. Once the layer of ripened clay has become thick enough and provides sufficient bearing capacity, the area can be made accessible to construction equipment and can be further developed. An obvious decision would be to develop the shallow parts of the basin first, along the inner slope of the ring-dike.

The results from the functional, spatial and geotechnical analysis have been combined in order to prepare a long term management plan. A decision making tool has been developed to give the Port Authority an overview of the short and long term possibilities, and to serve as a guideline for selecting an alternative approach on storage handling and use of space. The approach that is recommended follows the principle of the clay ripening alternative in combination with dry storage, for the remaining use of the Slufter as a disposal facility. It combines short term opportunities for the application of small scale functions (such as solar panels or algae breeding) with a long term, sustainable solution towards an early dismantlement of the Slufter. In addition, this approach if permanent implicates that the high surrounding dike structure of the Slufter is no longer needed and can therefore be (partly) removed. The obtained space can then also be used for other purposes.

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1By dry storage is meant that the incoming dredged material is still discharged into the basin as a soil/water mixture. However, once inside, the clay and silt particles rest on top of the ripened dredged material whilst the excess water evaporates or percolates to deeper layers.

Port of Rotterdam Authority
This is the final report of the MSc Thesis ‘Preparing a long term management plan for the future of the Slufter’, with the subtitle ‘An analysis of the functional, spatial and geotechnical possibilities’. It is the graduation work of Roderik Heerema for the Hydraulic Engineering specialization ‘Ports & Waterways’ and commissioned by the Port of Rotterdam Authority. The report represents the work done from the beginning of the research in early November 2010 until the end of June 2011.

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**The Slufter** was built in ’86/’87 and is located at the southwestern tip of the Maasvlakte. Its function is to store contaminated dredged material that has been dredged from the Rotterdam harbor shipping channels and basins, from rivers and inland waterways in the Netherlands under governance of Rijkswaterstaat and to a lesser extent from third party projects within the Netherlands.

In Chapter 1 the thesis is described in terms of the research motive and the problem definition, leading to the main thesis objectives. Chapter 2 describes the chosen approach in order to elaborate the objectives. The functional, spatial and geotechnical analysis are elaborated in Chapters 3, 4 and 5, respectively. In Chapter 6 the results of the analyses are used to prepare a long term management plan for the future of the Slufter. Finally, the conclusions are drawn and recommendations are given in Chapter 7. The appendices provide extra information. For a quick overview of the results of the analyses chapters is referred to Sections 3.4, 4.6 and 5.5 ‘Evaluation’.
Figure 1: The location of the Slufter [Google Earth, 2005]

Figure 2: The Slufter
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Chapter 1

Thesis description

In this chapter, first some background information is given on the history of the Slufter. Then, the research motive is explained and the problem is defined. This ultimately leads to the main objectives of the thesis.

1.1 Background information

Maintenance dredging is an everlasting activity in the port of Rotterdam. Because tidal motions and river runoff bring in large amounts of sediment, the harbor basins continuously have to be kept to depth to allow for ship navigation. Due to the unrestricted discharge of excess waste water from industries along the entire river, over the years the Rhine had become highly polluted. Since the river flow velocity decreases significantly in the Rhine Delta, and a sharp salinity gradient is present, the port of Rotterdam acts as a sink for the industry induced contaminated sediments. By the 1980s the Port of Rotterdam Authority (from now on referred to as the Port Authority) and Rijkswaterstaat were dredging large amounts of contaminated dredged material which could not be disposed into the North Sea. Therefore, plans were made for the design of a large scale disposal facility (the Slufter); whilst in the meantime the “Project Onderzoek Rijn” (Rhine Research Project, POR) was initiated by the Port Authority.

The POR mainly focused on those industries along the Rhine that were apparent polluters, such as the steel and chemical manufacturers. Their goal was to convince these companies to decrease their discharge of harmful substances by 70 to 90%. They gathered evidence on the quantities and origins of the pollutants and thereby held a strong legal case. The companies with the largest contribution to the contaminations concurred with the POR’s conditions and made an agreement to reduce their discharge. The project was more successful than was expected and has now resulted in a significant decrease in contaminated dredged material in the Rotterdam harbor.

For this reason and others, the way in which the Slufter has been used since the completion has changed. The actual filling progress has been different than the prognosis made in 1987. The following events have contributed to this change:
CHAPTER 1. THESIS DESCRIPTION

Figure 1.1: Cumulative amount of dredged material stored in the Slufter since 1987 [20]

- As mentioned above, the degree of contamination of dredged material in the harbor basins, channels and rivers has decreased significantly resulting in less storage in the Slufter.
- Dredged material that meets certain requirements nowadays has to be processed to useful (building) materials, which led to a slight reduction of the quantities stored in the Slufter.
- Initially, the intended users of the Slufter were well-specified, namely the Port Authority and Rijkswaterstaat. However, over the years the facility was made more and more accessible to third parties. The supply of dredged material thereby increased again.
- The Slufter’s acceptance criteria for types of contaminated sediment have become less strict, with the consequence that storage of the highly contaminated dredged material type IV (see Appendix A for the classification of materials) was made possible. This also led to larger quantities of sediment to be stored in the Slufter.
- The Slufter experiences competition from smaller commercial facilities, which are sometimes closer to the dredging activities. Therefore, transportation costs are lower, and storage at the Slufter is not always the obvious choice.

Some of the above mentioned events have resulted in a decrease in supply of dredged material. In anticipation, decisions have been made on storage handling and management in order to increase the supply and guarantee the Slufter’s storage function (these are the above mentioned events that resulted in an increase of dredged material supply). Nonetheless, prospects are that it will take a long time before the entire basin is filled.

1.2 Research motive

When the Slufter was designed it was expected that the basin would reach full capacity within the first 15 years of its use. However, the reality is that after 23 years, no more than...
50% of the available space has been occupied. The steady decrease in supply of dredged material can be seen in Figure[1.1]. It is expected that the supply of contaminated dredged material will only decrease further in the future. The Slufter is increasingly losing its function as a storage facility. Overcapacity and the subsequent surplus of space creates opportunities for other functions that can be accommodated in the Slufter area. In addition, a long term strategy is required for the dismantlement of the Slufter as a disposal facility.

1.3 Problem definition

The Port Authority is in need of a long term management plan for the future of the Slufter. In future plans the interests of Rijkswaterstaat, as a joint owner, have to be taken into account as well. However, the interests of Rijkswaterstaat and the mutual ownership relation between the Port Authority and Rijkswaterstaat is beyond the scope of this research.

The problem can be defined by sub-dividing it into two simple main questions: what can be done and how can it be done? On the subject of what can be done, questions such as the following arise:

What other types of functions can be accommodated in the Slufter area? Which requirements do these functions have to meet? Which functions are technically and economically most feasible?

On the subject of what can be done, questions that will be dealt with are:

Can large parts of the basin be reclaimed in order to create land? What are the possibilities for use of land in the short term? What are the opportunities of the entire area in the long term? Is the proposed method for land reclamation technically and economically feasible?

All these questions lead to a formulation of the thesis objectives.

1.4 Thesis objectives

The objectives of this thesis are threefold: The first objective signifies the problem of finding a new and suitable function for the Slufter. The second objective signifies the problem of how to put the future function into practice. The third objective is to prepare a long term management plan for the future of the Slufter.

Objective 1: In search of the future function(s)

It is important to define what is needed and what is wished for. Many ideas have risen and plenty of opportunities can be seen, but not everything is realistic or technically and economically feasible. During the research these ideas will be gathered and investigated on their potential as a new function of the Slufter. Finally, the functions which are most feasible and have the most positive impact on harbor or landscape development will protrude.
CHAPTER 1. THESIS DESCRIPTION

Objective 2: Creating land in the Slufter

The dredged material in the basin, which for the larger part consists of fine clay and silt particles, has a very low bearing capacity. Therefore measures have to be taken in order to be able to build anything within the Slufter basin. This is a geotechnical problem.

In addition, the storage function partly has to be maintained. Therefore specific locations within the Slufter’s perimeter have to be selected to provide space for the future function, depending on the prognosis for the required remaining storage capacity. This objective focusses on the possible methods to create land in the Slufter, the geotechnical feasibility and the efficient use of the space.

Objective 3: Preparing a long term management plan

Ultimately, the proposed methods for creating land will be combined with the most suitable future functions. This leads to the preparation of a long term management plan for the future of the Slufter, incorporating the results from the first two objectives. The final report may serve as starting base for future development of the area, a multi-functional use of space, and a sustainable long term solution for the dismantlement of the Slufter as a disposal facility.
Chapter 2

Applied approach

The problem is approached by dividing it into four main points of attention: future functions, use of space, technical feasibility and a long term management plan. Section 2.1 describes how the potential future functions will be found and considered. In Section 2.2 the way in the spatial opportunities of the Slufter will be analyzed is explained. The approach on the analysis of the technical feasibility will be described in Section 2.3. Section 2.4 elaborates on the way the results of the analysis in this research are combined and how a long term management plan is prepared.

A final remark should be made on the chosen approach. Initially, no boundary conditions or requirements were posed by the Port Authority for what the future of the Slufter might behold. Therefore, this research is elaborated from a broad perspective, and aims to map the possibilities of this area as good as possible. In due course, any additional boundary condition or requirement that is relevant will present itself as a result of the analyses.

2.1 Functional analysis

The followed approach in the functional analysis is according to the following steps:

1. A brainstorm session
2. A criteria analysis
3. Further analysis of the remaining functions
4. Evaluation

The ideas for potential future functions are generated by means of a brainstorm session. The session is divided into a divergence and convergence phase, beginning with a long list of generated ideas, and ending with a short list of ideas which are considered the most desirable or feasible.

In order to not do away with the long list immediately after the brainstorm, a basic criteria analysis is done. A number of criteria are posed and the ideas are weighed per criterion by giving it a score between -2 and +2. After this analysis the 20 ideas remain that have the
highest score when adding up the score per criterion\textsuperscript{[1]}

The top 20 of ideas are translated to actual future functions. Each of these functions is investigated further and accepted or rejected based on qualitative and quantitative arguments. After evaluation, the functions with the highest potential remain.

The remaining functions will be combined with the use of space and tested on the technical feasibility with respect to the stability and settlement of the subsoil.

During the further analysis of functions (step 3) it is assumed that for all functions land reclamation is required. Therefore, the spatial and geotechnical analyses focus on finding methods to reclaim land from the Slufter basin. In the following sections this is explained in more detail.

\section{Spatial analysis}

In the spatial analysis, the following steps will be followed in order to determine the influence of various alternatives on the use of space:

1. Analysis of the basin geometry
2. Analysis of the level of dredged material
3. Estimation of the required remaining storage capacity
4. Analysis of the retaining structure alternative
5. Analysis of the clay ripening alternative

The possibilities for the use of space are investigated by first looking into the geometry of the basin and by determining the current level of dredged material.

The Port Authority requires a minimum volume to be maintained for the storage of dredged material. This is a boundary condition for the area to be developed for new functions. Therefore, the remaining storage capacity is estimated.

Two main alternatives are proposed as methods to reclaim land from the Slufter basin. The first alternative is presented by a water/soil retaining structure that can separate the Slufter’s storage function from the new function(s). In the second alternative the possibilities of land reclamation are investigated based on the natural process of ripening of dredged material. Figure\textsuperscript{2.1} shows the principle of the retaining structure alternative.

An efficient use of space is a compromise between maximizing the new function area and minimizing the use of materials and thereby costs. For both alternatives the amount of space that can be obtained is estimated.

In addition to maximizing the new function area, various locations in the Slufter that are suitable for development are selected according to the type of function that might be suitable there.

\textsuperscript{[1]the difference between a Multi-Criteria Analysis (MCA) and the basic criteria analysis applied here is that in an MCA the criteria are weighed as well, which means that one criterion is more important than the other. This approach is not followed here, because of the magnitude of the list of ideas. In addition, the MCA is a more advanced selection tool, but that is not required at this stage.}
2.3. GEOTECHNICAL ANALYSIS

Figure 2.1: Schematization of the retaining structure alternative principle

Additional note on floating structures  A third alternative to create a usable area in the Slufter is by means of floating structures. A material that is used more and more in floating foundations is EPS (styrofoam). The current state of technology is such that multiple types of functions can be accommodated on floating EPS structures. However, due to the high purchase and installation costs it’s economic viability is questionable. The costs for EPS foundations are momentarily estimated to be €50 to €90 per square meter. Taking the average, the investment costs per hectare are estimated at €700,000.

An analysis of the feasibility of floating structures as an alternative to a retaining structure or clay ripening is beyond the scope of this project. However, it should be mentioned as one of the possibilities to create land.

2.3 Geotechnical analysis

The dredged material stored in the Slufter is considered as sediment with a very low bearing capacity. In order to create land that is suitable for any kind of function, measures can and have to be taken. In this research a distinction is made between two approaches that are worth investigating; building a retaining structure or allow for clay ripening. To be able to ultimately proclaim that either of these solutions is technically feasible, logical steps have to be followed. The approach is described below and the steps to be taken are elaborated in the geotechnical analysis.

Building a retaining structure

The following describes the steps that are taken to analyze the feasibility of building a retaining structure.

1. Formulation of the initial conditions
2. Formulation of the geotechnical requirements
3. Selection of the type of retaining structure
4. Description of the execution method
5. Analysis of the geotechnical feasibility
6. Estimation of the execution costs and evaluation

The first step is to map the current geotechnical condition of the Slufter’s dredged material. The required information is derived from reports on the geotechnical research done at the Slufter in the past, in terms of the geotechnical properties that are relevant to this research. These properties are presented in the geotechnical analysis as initial conditions.

In the second step the geotechnical requirements are defined, which provide the necessary guidelines to test the technical feasibility of the selected type of retaining structure.

In the third step a decision is made on the most suitable type of retaining structure. The selection is made based on practical arguments.

The fourth step is to come up with a construction method. Here, use of materials and equipment are decisive. In addition, a method to accelerate consolidation is selected. Acceleration of the consolidation process results in a decrease of the pore water pressures and thereby improves the geotechnical properties of the soil. This is an essential part of building on dredged material with a low bearing capacity.

The fifth and most extensive step is to test the proposed solution on the main geotechnical failure mechanisms or, in other words the structure’s safety with respect to stability and settlement. A distinction is made between the construction phase and the users phase. The construction phase is the most critical part, since the build-up has to be executed slowly and the stability and settlement constantly monitored. In the users phase the eventual structure is combined with the possible future functions and tested on the loads imposed by those functions.

The necessary calculations in the fifth step are done by means of the geotechnical analysis programs MStab and MSettle. MStab is a program that can analyze stability problems and MSettle is a program that can analyze settlement problems. Alongside the calculations done by computer hand calculations are done to verify the results according to the same theoretical background of soil mechanics.

The sixth and final step is estimate the execution costs and to evaluate the economical and geotechnical feasibility of the retaining structure.

Clay Ripening

The aim of looking at clay ripening is to provide an alternative to building a retaining structure. The advantage is that natural processes are used instead of laborious construction methods in order to provide a surface that can possibly be made suitable to build on. The required knowledge on this matter is derived from the report on Physical Processes [15], which was a supplement to the Environmental Impact Assessment (Dutch: MER) that was done before the Slufter could be built. Based on the findings in this report a judgment is made on the geotechnical feasibility of clay ripening and what supplementary measures are necessary to combine it with the future functions. These extra measures may imply load conditions that can be tested by exploratory calculations. Step 1 and step 2 of the retaining structure alternative also apply to the clay ripening alternative.
2.4 Preparing a long term management plan

The ultimate goal of this research is to prepare a long term management plan for the future of the Slufter. This will be done according to the following steps:

1. Combine the results of the functional, spatial and geotechnical analysis
2. Develop a tool to give an overview of the possibilities
3. Recommend a long term management plan

In the first step the most important conclusions that protrude from the functional, spatial and geotechnical analysis are summed up and combined. A tool will be developed that comprises all the possibilities for future development. This tool may function as a guideline for further steps towards the formulation of a long term management plan. Finally, a recommendation is given on which plan is economically and technically most feasible.
Chapter 3

Functional analysis

Before the Slufter was built the area where the basin is now located was part of the foreshore. The Slufter has thus been reclaimed from the sea. In the regional planning Rijnmond (Dutch: ‘streekplan’) dating from 1974, the directive for the area west of the Europaweg was nature. Therefore, the general aim was to maintain that function. However, the regional planning scheme was only legitimate for the boundaries of the area; for the policy on the area itself it could only give an indication. Then and even now there is no official zoning plan (Dutch: ‘bestemmingsplan’) for the area where the Slufter is located. In ‘Nota 4’ of the integral revision of the regional planning Rijnmond is posed that the depot surface has to be developed into an area with nature and recreational functions [27].

Another planning boundary is set by the demarcation line, which separates the industrial activities on the Maasvlakte from the nature reserve on the coast of Voorne. The boundary aims to restrict the air pollution and noise from industry. The demarcation line has been extended since the first realization in 1964. It now runs along the southern side of the Maasvlakte up to the bend in the Europaweg and then along the southern side of the Slufter ring-dike. So the Slufter lies within the area marked as industry.

The intended future of the Slufter thus beholds nature and recreation, but this is not set down in a zoning plan. Moreover, the definition of the demarcation line would allow industrial functions. This creates the opportunity to consider and look into other functions as well, which is done in this chapter.

The functional analysis begins with the results of a brainstorm session on possible future functions and continues with the selection of functions in a criteria analysis. A remaining list of functions is discussed in a further analysis of the functions. Finally, the remaining functions are evaluated.

3.1 Brainstorm session

To generate ideas and to consider any kind of future function a brainstorm session was organized. The 10 participants were all employees from the Port of Rotterdam, but with various backgrounds and expertise. This was done to ensure that the overall view of the
Port Authority on what the future of the Slufter should look like was supported. During the brainstorm the participants were first asked to come up with as many ideas as possible (the divergence phase). Next, the large list of functions was brought back to a few, by votes and selection of the participants’ favorite ideas (the convergence phase). The choices were made based on the ideas’ feasibility and positive impact on the Port of Rotterdam. Finally, the participants were divided into four groups and asked to ‘design’ their Slufter of the future on a map by including the following features to the function; a location and size, a construction method, added value, and the timing of implementation. Overall consensus was reached on the following ideas for the future of the Slufter:

- Development of renewable energy sources
- Storage of other type of products or materials

The possibilities for renewable energy are: solar energy, wind energy, biomass production, algae breeding and hydropower. The possibilities for storage are: temporary storage of soil from projects (TOP), storage of chemicals, empty containers, dry contaminated soil and storage of hazardous substances.

Other important remarks made during the session were:

- The storage of dredged material function of the Slufter has to be sufficiently maintained
- The new functions can have a temporary as well as permanent character
- A combination of different functions (for example nature, storage and renewable energy) is an opportunity

### 3.2 Criteria Analysis

The brainstorm resulted in a list of 43 ideas. Although a first selection was made based on feasibility and positive impact on the Port of Rotterdam, all these ideas were considered once more in a criteria analysis. Each idea was assessed on twelve criteria and given a score between -2 and +2. A score of -2 means that it is a serious risk with respect to that criterion, -1 means there is a negative attitude towards it, 0 is neutral, +1 is a positive attitude and +2 is a serious opportunity. The twelve criteria are listed and explained briefly below.

**a) Efficient use of space**

The efficient use of space depends on the space required by the new function. If the new function requires a large surface, consequently, less space is available for maintaining the storage of dredged material.

**b) Vicinity support**

The people in the communities surrounding the Slufter, such as ‘gemeente Westvoorne’, are the closest living in the vicinity of the disposal facility. Therefore they will be interested in the future development of the Slufter and how this change will affect their lives. It is likely that they will be more in favor of some specific new functions than others.

**c) Social support**
3.2. CRITERIA ANALYSIS

The new function is more likely to be implemented if there is sufficient social support. Some functions are related to popular topics such as renewable energy and are therefore likely to find more social support.

d) Port of Rotterdam Authority support
Some functions add more value to the port of Rotterdam than others. The Port Authority might benefit from the new function(s) if it is a showpiece to the outside world, even when it is financially unattractive.

e) Permits
Depending on what the new function will be and what operations will take place at that location, it might either be difficult or relatively easy to obtain the required permits. The likeliness of being able to obtain the permits may speed up the development process, making those new functions more desirable.

f) Flexibility
The flexibility indicates the temporary character of a specific function. In other words, if a function can be easily transformed into other functions or terminated after implementation and use it is considered flexible.

g) Costs/ Benefits
This criteria balances the investment costs over the benefits that will be obtained from the new function. It might be possible to compensate high investment costs by a high return on investment. Costs and benefits are important for both the Port Authority and potential investors.

h) Harbor committed
Since the Slufter is a part of the port of Rotterdam, it is favorable to combine its function with a harbor related function.

i) Multi-functional
In some cases it is possible to combine different functions in the same area. It is considered beneficial if specific functions do not restrict multi-functional purposes.

j) QSHE
Specific functions might pose risks to Quality, Safety, Health and Environment. On the other hand, some functions might be exposed to hazards related to the Slufter’s storage function. The score on QSHE gives an indication of the function’s ability to overcome these problems.

k) Sustainability
The new function will be given extra value if it is a sustainable solution. Efficient use of materials, equipment and energy are important in that sense, as well as design lifetime and maintenance.

l) Technical feasibility
Initially, it is assumed that all functions are technically feasible. However, some functions are expected to be more realistic, because it has been done before or it requires less drastic
measures.

In Appendix C the results of the criteria analysis are displayed. The analysis has resulted in the following top 20 of functions that will be further investigated.

1. Solar energy
2. Temporary storage of dry soil (TOP)
3. Birds reserve
4. Algae breeding farm
5. International (EU) depot
6. Windmills
7. Soil cleansing and recycling
8. Empty depot
9. Forest on dredged material
10. Nature reserve
11. Only dredged material deposit in the Netherlands
12. Biomass production
13. Service center
14. Soil investigation area
15. Greenhouse cultivation
16. Fish breeding farm
17. Golf course
18. Storage of chemicals
19. Location ‘Bevi’-companies (for a description, see below)
20. Hydropower station

The functions related to renewable energy and storage are clearly present, and thus in accordance with the results of the brainstorm session. Each of the top 20 functions will be considered and discussed below.

### 3.3 Further analysis of functions

To give an overview of the 20 functions they are categorized in Table 3.1. In the subsequent paragraphs the functions will be further analyzed. The information used has partly been derived from documents, partly from the internet and partly from interviews with Port Authority or external specialists.

<table>
<thead>
<tr>
<th>Energy</th>
<th>Storage</th>
<th>Nature &amp; Recreation</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solar energy</td>
<td>Temporary storage</td>
<td>Birds reserve</td>
<td>Soil cleansing &amp; recycling</td>
</tr>
<tr>
<td>Algae breeding farm</td>
<td>International depot</td>
<td>Forest on dredged material</td>
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<tr>
<td>Biomass production</td>
<td>Only deposit in NL</td>
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<td>Greenhouse cultivation</td>
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<tr>
<td>Hydropower station</td>
<td>Chemicals</td>
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<td>Fish breeding farm</td>
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<tr>
<td></td>
<td>Bevi-companies</td>
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</tr>
</tbody>
</table>

A few of these functions will not be analyzed further, since it is either not necessary or beyond the scope of this research. To begin with, the functions of International (EU)
3.3. FURTHER ANALYSIS OF FUNCTIONS

depot and Only deposit in the Netherlands are much hoped-for, because that would immediately solve the problem of overcapacity without having to alter the current Slufter configuration. However, this would require a major shift in the policy on the handling of dredged material in the Netherlands or in the European Union. The chances are small that such a shift will happen within the near future and there is no use for the Port Authority to wait in anticipation.

The other function that can be set aside is the Soil investigation area. The idea of a soil investigation area is to use the Slufter as a test facility for research on all kinds of geotechnical or chemical processes. It is discarded in the further analysis, because it has already been done in the past, can be done now and will still be possible in the future.

In the further analysis of the remaining 17 functions, it is assumed that each of these functions, depending on the amount of space it occupies, requires a smaller or larger stretch of land that is to be reclaimed from the Slufter basin.

Solar energy

Solar energy is said to be one of the most promising renewable energy sources of today. The current state of technology is such that purchase and use of solar panels can be cost-effective. Installation of solar panels could be an opportunity in the Slufter area, either on the ring-dike or on areas reclaimed from the basin. A downside is the frequent lack of sunshine in the Netherlands.

![Solar panels on a pasture](image)

Figure 3.1: Solar panels on a pasture

The maximum power of a solar module in ideal circumstances is expressed in Wattpeak. 1 Wattpeak produces approximately 0.8kWh per year in the Netherlands. The accompanying solar panels take up 0.8m$^2$ of space per Wattpeak, so 1m$^2$ produces 100 kWh per year.

Suppose an area of 25,000m$^2$ (2.5ha) is realized at the Slufter and covered with solar panels, then 2500MWh per year is produced. An average family in the Netherlands uses approximately 3500kWh per year, so this amount of solar panels can provide 714 families per year with power.

At the moment, people that have solar panels installed receive around 21 euro cent per kWh when delivering the excess power back to the national grid. Suppose the life span of the
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panels is 25 years and the expected price-rise 2% per year, then the average price over this period is 26.7 euro cent per kWh. In that case the revenue is $2500 \times 10^3 \times 26.7 = €667,500$ per year.

The costs for solar panels including the installation varies from €3 to €5 per Wattpeak, depending on the type of solar cells used on the panels. A system based on mono-crystalline silicon is more expensive than a system with poly-crystalline cells. Poly-crystalline panels however need a more space per Wattpeak.

Based on these numbers the investment costs for 2.5ha of solar panels can vary from €9.4 to €15.6 million. However, the investment costs may decrease considerably if it is project on large scale.

Suppose 2.5ha of solar panels is realized with a budget of €10 million, then the investment is recovered after approximately 15 years. In the remaining 10 years of the panels’ life span another 6 to 7 million Euro can be gained as profit. It should be noted however that the land costs are not included here. The operator of the solar panels has to take these costs into account. For example, the rent for land with an empty depot function is $7-10€/m^2$ per year. If a similar price range is held for 2.5ha of solar panels, the costs per year would be €175,000-250,000. In that case the profit after 25 years would be 1 to 2 million Euro.

The information on solar energy and solar panels used in this paragraph has been derived from reference [9].

Temporary storage of soil (TOP)

In the port of Rotterdam many projects take place where slightly contaminated soil is excavated which is waiting for a new purpose, because the situation does not allow for an immediate reuse. A possible location for storage of this excess soil is TOP-Europoort. In other cases temporary storage locations have to be improvised. However, TOP Europoort also has the function of warehouse. Various types of construction material can be brought to TOP-Europoort, such as debris, rocky material and ripened dredged material. At the TOP, tests can be done to determine the materials’ properties, and contaminated soil can be processed to improve its quality. Eventually, a new destination will be sought for the materials. The Slufter area could also provide such a location.

It is expected that TOP-Europoort will have to move within a couple of years, because of new plans to develop this area. Therefore, the current situation of TOP-Europoort was evaluated by the engineering office of Gemeentewerken Rotterdam, and a vision was given on how exploitation of TOP should continue in the future at another location [33]. The Slufter was among the six alternative locations that were considered. In a Multi-Criteria Analysis the Slufter was evaluated as the fourth best location. The main reason for this low score is the remoteness of the Slufter. Preferably the new location is near to the projects so that the exchange of excess soil can be realized with minimum transport distance.

The distance and transportation costs are a major criterion for a TOP location. If a TOP is established at the Slufter, the costs for transportation to and from the projects would outgrow the benefits of the reuse of the soil. Instead the contractor will decide to find or deposit soil at a closer location.
A parallel can be drawn with the sand that is currently being recovered from the separation basins at the Slufter (also see Appendix A.5). It is often difficult to find a buyer for this sand. In some cases contractors will not even consider to pick it up free of charge, because of the transportation distance.

In the evaluation of Gemeentewerken Rotterdam the best location would be in the area of ‘Vondelingenplaat/Eemhavengebied/Waalhavengebied’. These areas are located more in the heart of the port of Rotterdam and are therefore nearer to the projects dealing with large quantities of contaminated soil.

**Birds reserve**

The Slufter area is known to attract various species of birds. This value could be exploited further by designating and developing specific areas where circumstances for bird life can be made even more attractive.

The Slufter is specifically suitable, because it provides shelter and quiet, and it is near by sources of food. In fact, there are not many places left along the coastline with such good conditions for bird life. The risks for birds that stay at the Slufter are posed by the contaminations of the dredged material. At the moment, the various species of birds mainly forage outside the Slufter, and rest within. Despite the contaminations, it seems that the birds are doing very well.

If the Slufter is partly transformed into a birds reserve, breeding and foraging must be possible without disturbance. Therefore, vacationers must be kept away and the area must provide a lake or pond with a constant water level, preferably protected from the contaminants.

This function may also solve the harbor’s sea-gulls problem. There is quite a large colony of sea-gulls that persists to live on those areas on the Maasvlakte where no activity takes
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place. Every time one of those areas is developed the colony moves to the next area. Some areas are constantly being swept to prevent them from staying there. A bird’s reserve in the Slufter may attract the colony and give them a permanent residence.

Algae breeding farm

Algae can be bred in a farm to produce bio-fuel or to provide products for the food- and pharmaceutical industry. The production of bio-fuel from algae is in a stage of development, and has not been practised on a large scale yet. At the moment, the prize for a liter of biodiesel is much higher than for regular diesel, and therefore still commercially unattractive. This may change in the future as a result of a higher efficiency of the algae production on a large scale. However, for the moment sufficient production can be achieved to fill the need of algae for the food- and pharmaceutical industry.

The algae can be produced in open ponds or in closed loop systems (consisting of vertical tubes). The closed system is considered better, because it prevents other algae species, diseases or competing bacteria to enter the system. It is however, more expensive than open pond farming. The main ingredients for the growth of algae are sunlight and CO2. At the moment, up to 20,000 liters per hectare per year can be produced and the aim is to reach 40,000l/ha or more in the near future [12]. An optimal production is a trade-off between the size of the type of algae and the growth speed. Small algae species tend to grow fast, but are more difficult to harvest.

Figure 3.3: Closed loop system

The Port Authority is, in collaboration with the Rotterdam Climate Initiative, momentarily working on a plan to establish a pilot for an algae breeding farm at the Slufter. The general idea is to realize this within the basin itself, using the current sheet of water on top of the dredged material. The reason why the Slufter might be suitable for this production is that algae are said to thrive well in salty and even in contaminated or waste water. Moreover, the Slufter is near to heavy industries such as the coal-fired power plant on the Maasvlakte, so there is sufficient supply of electricity and CO2. It thereby also contributes to the reduction of CO2 in the atmosphere. LED lamps can replace sunlight at night to allow for a 24/7 production (as done in greenhouses).

There are of course some practical problems such as the way in which the algae are contained within the basin, the way of harvesting and the way in which electricity and CO2 are supplied.
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If those problems prove difficult to overcome, the production farm (or at least the pilot project) can be realized on dry land or on a floating structure.

Various chemical companies and research institutions have shown interest in investing in this project.

**Windmills**

Even though the efficiency of windmills is a continuing discussion, research and investment is still ongoing. Generating electricity by wind is nowadays the most widespread form of renewable energy.

Currently, 17 windmills are located on the crest of the ring-dike, with 1.5MW turbines. There are plans to replace these by larger 4-6MW windmills in the near future.

There are certain requirements with respect to the placement of windmills. For instance, the minimum distance between two consecutive windmills has to be approximately 5 times the diameter of the rotor blade [10]. This is to ensure maximum efficiency for each turbine and no mutual influence. The existing windmills at the Slufter are placed approximately 210m apart. Since they are placed along the coastline, the shaft and the rotor blades are smaller than an inland windmill.

![Figure 3.4: Windmills along the Slufter's ring-dike](image)

A 1.5MW turbine has an annual production of approximately 13,000MWh. In comparison, 2.5ha of solar panels produces 2500MWh per year, as described earlier in this section.

The wind turbines can not be placed too close together. Therefore, each turbine at the Slufter takes up approximately 14ha of space. Approximately that same amount of space would be required for solar panels in order to reach the same production. However, the investment costs of one wind turbine is significantly less than a field of solar panels covering the same area. A wind turbine with a capacity of 1.5MW requires an investment in the order of 2 million Euro [10].

Windmills are heavy structures and have to endure large wind forces. Therefore, they need a
strong foundation. Either a foundation on piles or a good subsoil for a shallow foundation is required. A suitable place within the Slufter area for the placement of windmills is, besides on top of the ring-dike, in the higher parts of the inner dike. The layer of dredged material is thin here due to the gentle slopes. If this area is elevated to a height where the wind is strong enough or windmills with a higher shaft are built, more could be placed. This area reaches out over a length of approximately 170m, from the crest of the ring-dike to the ‘beach’ of the Slufter basin. If the distance between two existing windmills is 210m, then the maximum distance for a new windmill in between is \( \sqrt{\left(\frac{210}{2}\right)^2 + 170^2} = 200m \). Thus, there is insufficient space to install more 1.5MW turbines in these regions. Moreover, if the larger 4-6MW windmills are to be placed, they have to be placed even further apart. For instance, if the rotor blades have a diameter of 60m then the required interspace is 300m. The consequence is that less windmills can be placed.

There are more opportunities for windmills in the Slufter in the long term, when the storage basin is partly or completely decommissioned. However, for the nearer future a realistic option for the placement of windmills is along the southern side of the ring-dike. A stretch of approximately 1500m provides space for the placement of five 3MW or four 5MW windmills. This was also one of the conclusions from a report made by Bosch & van Rijn, Consultants in renewable energy and planning [4], where a ‘quick scan’ was done of all the potential locations for the placement of windmills in the port of Rotterdam. The southern side of the Slufter’s ring-dike is considered a location with good opportunities, under the following conditions:

- It must be proven that there is no unacceptable influence on the ecological main structure of the area south of the location
- It must be in consultation with the community ‘Westvoorne’. This has been agreed in the covenant “Overgangszone Brielse Maasmond” (21 July 1964), which ensures the involvement of the community in development plans of this transition zone between the community and the harbor
- The turbines have to be placed in such a way that there is no nuisance for the shipping radar

Soil cleansing and recycling

The Slufter’s dredged material can be cleansed and recycled by means of specialized installations. This function does not require a large operational area and can contribute to the early dismantlement of the Slufter as well as increase the capacity, i.e. the recycled soil can be used to reclaim other areas, but creates capacity in the meantime.

In order to obtain dry, clean soil the dredged material undergoes a process of de-watering and thermal cleansing (or immobilization). Small installations have a processing capacity of 100 tons per hour and take up an area of 3000m\(^2\). The largest installations can process up to 500 tons per hour and take up 5000m\(^2\).

It is unlikely that the processed clean soil can be sold commercially as a construction material. To begin with, the transportation costs are high because of the remote location of the Slufter. Moreover, the dry soil will have to compete with clean sand that is freshly recovered from the...
sea, is cheaper and has better geotechnical properties. Soil cleansing contractors in principle only process dry contaminated soil, because the Dutch laws and regulations prescribe that the companies that handle the soil are responsible for the contaminations and therefore the cleansing and recycling. In general, all dredged material in the harbor basins, rivers and channels is the problem of regional and local governments. In case of the Netherlands, the policy has been to store contaminated dredged material rather than cleaning it, simply because it is cheaper for such large quantities.

For these reasons, a soil cleansing installation should only operate at the Slufter if the aim is to reuse the processed dry soil within the Slufter area itself, for instance as an elevation material to realize land. Moreover, it depends on the requirements of the processed soil.

The costs for the de-watering of dredged material are approximately 20€/in-situ m$^3$ and for the thermal immobilization 42€/in-situ m$^3$ [29]. In-situ m$^3$ refers to the volume of the material as it was, undisturbed in the place of origin. In comparison, the storage of dredged material in a subaquatic confined disposal facility such as the Slufter is approximately 10€/in-situ m$^3$. Immobilization is therefore not a very desirable option. If the soil only has to be de-watered and there is no time pressure than ripening is a cheaper option. In addition, the costs for excavation, transportation and placement of sand within the Slufter area itself is in the order of 5€/m$^3$. Therefore, sand has better opportunities for reuse in the short than processed dredged material.

Empty depot

The Port Authority stores a significant amount of empty containers for which various locations are assigned. An area of reasonable size could be made available in the Slufter in order to provide more storage space.

Suppose 50ha could be used for the storage of empty containers. The costs for the rent of this type of area in the harbor are between €7 and €10 per square meter per year. The profits are thus 3.5 to 5 million Euros per year.

The reality is that quite a large area on Maasvlakte 1 and Maasvlakte 2 has already been reserved for empty depot. An area of 43ha is designated for this function to provide enough storage for the coming years. Moreover, another 29ha is reserved to provide enough storage space to last until 2030. These areas are located nearer to the current and the future container terminals than the Slufter.

![Figure 3.5: Empty depot in the Rotterdam harbor](R.N. Heerema 39 Delft University of Technology)
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Another major requirement of empty depots is that the transfer to and from the terminal must go via an internal road. This is a straight road that only allows for container transfer and does not interfere with public infrastructure. The road also has to be horizontal.

An internal road from the Slufter to the container terminals on Maasvlakte 1 and Maasvlakte 2 may interfere with the opening up infrastructure of the future Maasvlakte 2. Moreover, the ground level of Maasvlakte 1 is at NAP +5m, which means that the Slufter’s ring-dike and the supposed empty depot has to be lowered to this same level. Drastic measures have to be taken to realize this.

Forest on dredged material

The growing of trees on dredged material has already been done in the past. A reference project is the ‘Broekpolder’ in the 70s. This area, west of the Rotterdam city center, was elevated by a 7m thick layer of contaminated dredged material, covered by a layer of sand. Afterwards trees were planted and their growth and health were monitored. The conclusion was that, despite of the contaminations present in the subsoil, the trees were doing very well. Therefore, growing trees at the Slufter should not be a problem either.

This can be either a maritime forest, with species tolerant of near-constant sea breezes and occasional salt spray with nature purpose, or a forest for the production of timber or timber products. A forest to be used for nature development or recreational purposes should be treated similarly as a nature reserve.

Nature reserve

Although a nature reserve can not easily be combined with other functions, it is still an elegant solution for the surplus of space. Nature development in the harbor can give the Port of Rotterdam a positive image and provide an attractive spot for nature enthusiasts.

The Port Authority manages the presence of nature in the harbor next to the industrial activities. The policy on the presence of nature and nature development in and around the harbor is approached according to the following three issues:

- ‘Natura 2000’ areas
- The code of conduct with regard to endangered species
- Temporary nature locations

Natura 2000 is a European network of protected nature areas in the European Union member countries. This network forms the basis of the European policy for conservation and restoration of bio-diversity. Near the harbor there are three Natura 2000 areas; one north of the ‘Nieuwe Waterweg’, the second is the delta coast west and southwest of Maasvlakte II (Dutch: ‘Voordelta’) and the third is the dune area south of the Slufter in the community of Westvoorne (Dutch: ‘Voornse duinen’). The influence of the industrial activity on these areas is constantly monitored and taken into consideration during harbor development.

The code of conduct with regard to endangered species is set up by the Port Authority and gives directions on how to cope with endangered species when encountered in areas to be developed.

Port of Rotterdam Authority
Most open areas within the harbor have a nature function that is temporary. This means that the possibility is kept open to develop and transform that area for industrial purposes in the future. This implies specific management options when endangered species are involved.

Especially the third point is relevant for the Slufter. If it is decided that the Slufter has to fulfill a nature function it also has to be decided whether this is permanent or temporary. A temporary nature function provides more flexibility for future development. The Slufter can also function as a type of reservoir for nature values in the harbor, to compensate for future development of industrial activities. That way it might be more easy to transform a specific area on Maasvlakte I or II, which has gained nature values as a result of inactivity, into an area with harbor functions.

Biomass production

Storage and processing of biomass already takes place in the harbor at various locations, and the market is growing. In fact, the port of Rotterdam is one of the main hubs for storage and transport of bio-chemicals. The main products that are shipped in are palm oil, corn and pellets (compressed wood). Biomass can be used to generate electricity or to produce bio-fuels.

The actual production of biomass in the Netherlands takes place on a very small scale. A biomass production farm has to be significantly large in order to be economically viable (consider for example the size of the palm tree farms in Asia). The production of biomass is unlikely to take place at the Slufter, because the amount produced would be insignificantly small compared to what is shipped in.

If more biomass processing facilities will be built in the future then the Slufter could be a possible location. However, there are more areas on Maasvlakte I and in the future on Maasvlakte II that do not have a zoning plan yet and are probably more suitable.

Service center

A service center is a central location for public services such as a police, fire and medical department. Depending on the already existing services in the harbor, the position of the Slufter on Maasvlakte I and next to Maasvlakte II might be a suitable central location.

A service center in the port of Rotterdam mainly consists of a fire department, sometimes supplemented with a medical department. The harbor police mainly operates on the navigation channels and the police department is therefore located centrally and along the water. The main fire departments which provide their services to the harbor industry and the active companies are located in the Botlek (Botlekweg), Europoort (Elbeweg) and the Maasvlakte (Coloradoweg). Their locations are chosen in such a way that they can reach any area in the harbor within 15 minutes.

Because eventually the Yangtzehaven will be extended to provide the entrance to Maasvlakte II, the emergency route from the Coloradoweg to the outer tip of Maasvlakte I is cut off. Therefore, an extra service will be provided on Maasvlakte II. The Slufter is nearer to Maasvlakte II and could therefore be considered as a possible location for a fire department.
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However, this function does not require a lot of space and on Maasvlakte II there will be sufficient space. Therefore, it is more likely that such a service will be accommodated on Maasvlakte II itself. The time it will take to travel from the Slufter to the furthest point of Maasvlakte II is approximately 10 to 15 minutes with an average speed of 80km/h.

Greenhouse cultivation

In the Netherlands the price for agricultural land is high, because of the scarcity. The costs per hectare differ per region and the applicability of the land. An hectare of agricultural land in the west of the Netherlands varied from €30,000 to €40,000 in 2007. The average size of a greenhouse company was 1.25ha in 2006. However, there are also companies that take up areas of 50 to 100ha or more [2].

The advantages of greenhouse cultivation over open-air cultivation are that the growth can be better controlled, production can take place all year round and plants and flowers that do not thrive well in cold conditions can be grown. The downside is that greenhouse cultivation consumes a lot of energy. There are however many innovative initiatives to make greenhouses more energy efficient, for example by storage and reuse of heat and the use of LED-lamps at night.

The basic necessities for the growth of plants or flowers in a greenhouse are sunlight, heat, water and CO2. In case an area within the Slufter is made accessible for greenhouse cultivation, then the industries on the Maasvlakte could provide sufficient heat and CO2. The greenhouses can be realized on a small or larger scale, depending on the investment costs and what is to be produced.

Fish breeding farm

The types of fish that are being bred in the Netherlands are mainly eels and European catfish, and to a lesser extent trouts and sea fish. The breeding takes place in a recirculation system, which constantly filters and purifies the water so that it can be reused and the waste products can be broken down. Recirculation systems require a water temperature of 20 degrees Celsius for the fish to grow and therefore most breeding farms are inside a hall or shed.

In general, the future perspective for the fish breeding market is favorable. There is an increasing demand for fish and the products from the breeding farms are healthy and safe. The Netherlands has a head start with knowledge and experience on the recirculation systems. However, the investment costs are high. Therefore, the aim should be to produce more expensive fish species to remain competitive.

A major issue of fish breeding is that the fish food to enable growth partly consists of fish products such as fish meal and fish oil. It takes one to two fish that is caught in the wild to produce a full grown fish in a breeding farm. Therefore, on the long run, the breeding of fish on a farm does not contribute to a decrease in overfishing of the oceans, rivers and lakes.

The opportunities for the Slufter are that there is sufficient knowledge available on how to
breed fish and make it economically attractive\footnote{The general idea is not to breed fish in the water of the Slufter basin itself, but in small pools established on land.}. With the recirculation system not a lot of space is required for a reasonable production. It might even be possible to combine a fish breeding farm with an algae breeding farm, because algae are a substitute for conventional fish food. The European Union supports fish breeding farms and grants subsidies.

Problems that may arise for a fish breeding farm at the Slufter are the strict requirements with respect to the production method, environment, animal welfare and food safety. In addition, the existing Dutch fish breeding farms do not cooperate well enough to enhance their competitive position and they encounter quite a lot of competition from abroad.

The information used in this paragraph has been derived from references\textsuperscript{26} and\textsuperscript{30}.

**Golf course**

Golf is the fastest growing sport in the Netherlands. Since a golf course does not pose large loads on the subsoil and provides a nice scenery, it is worth taking into consideration.

There are approximately 40 golf courses in the province of Zuid-Holland. On average that is 90,000 inhabitants per golf course. The nearest golf courses to the Slufter are 12, 30 and 45km away, in the proximity of Brielle, Spijkenisse and the Rotterdam city center, respectively. The communities near to the Slufter (Westvoorne, Brielle and Hellevoetsluis) together have approximately 70,000 inhabitants and one golf course, so that is in accordance with the average.

A 9-hole golf course requires a surface area of approximately 50ha and an 18-hole golf course twice as much. The investment costs of a golf course are high, because it has to be built according to the design, with an attractive layout, a club house and a good water control system. In addition, the maintenance costs are high.

Suppose the golf course would have 700 members, who pay a fee of €1000 a year, then the yearly revenue is €700,000. In western Europe, the costs for the construction of one hole can be estimated to be around €160,000\textsuperscript{23}. A golf course of 18 holes thus costs 2.9
CHAPTER 3. FUNCTIONAL ANALYSIS

million euros (without club house). In that case it would take over four years to recover that investment. However, that is without land acquisition and maintenance costs.

Storage of chemicals

The port of Rotterdam is a major hub for fuel. The harbor acts as a pricing center, because a lot of trade takes place here. All liquid bulk transported, processed and stored in the Rotterdam harbor can be subdivided into three categories of products:

- Mineral oil products
- Chemical products
- Edible oils, fats and oleochemicals

Mineral oil products are products such as fuel oil and diesel. The largest storage tanks for these products are 100,000m$^3$. Chemical products are products such as styrene, biodiesel and bio-ethanol. In the third category palm oil is one of the main products. In general, storage tanks for chemicals and edible oils are smaller, with capacities up to 20,000m$^3$.

The throughput of oil products has grown substantially over the past ten years and with it, the storage capacity. This growth is more related to trade rather than economic growth. By clever timing and transfer of products, traders anticipate the rising oil prices, which results in an increase the throughput in the harbor. Although it is difficult to predict, the expectation is that this growth will stagnate. Momentarily, an area of 60ha at the ‘Kop van de Beer’ (along the ‘Beerkanaal’) is being granted for the development of a new mineral oil tank terminal. For this type of products enough capacity is foreseen for at least the coming ten years.

![Figure 3.7: Storage tanks](image)

The development of throughput and storage of chemicals and edible oils is better related to economic growth. Therefore, a decrease in the throughput of these products could be seen during the financial crisis. At the moment, the situation has somewhat stabilized, but growth is expected. Recent development of new palm oil refineries induces a growth in throughput of this product. It is possible that more storage capacity is required in the future.
No specific areas are appointed for tank terminals for chemicals or edible oils yet, but some areas on Maasvlakte II seem suitable. Especially the area west of the future end of the Europaweg may be appointed. This location has not been granted yet and is near the navigation channel. Moreover, it is next to existing chemical industry (Euromax Terminal) and therefore close to the existing pipeline infrastructure.

Because of the expected growth in the throughput of chemicals, the Slufter may be qualified for the development of a tank terminal. However, tanks for the storage of chemicals are generally smaller than mineral oil tanks. Therefore, they preferably have short pipeline transport distances for a quick exchange of the liquid bulk. The quays closest to the Slufter are the Hartelhaven and the future Princes Amaliahaven (approximately 2km), but these quays are and will not be accessible to liquid bulk carriers. The pipeline transport distance would become too large if the discharge takes place even further away.

In a practical sense, there are better opportunities for the storage of mineral oil products, because these storage tanks are much larger and therefore a large transport distance can be acceptable. The downside is that no extra storage space is required for the coming 10 to 15 years.

A possibility is to realize a pipeline from the Slufter to the North Sea, to allow for mineral oil discharge from ships with a draught that is too large for navigation in the harbor. In that case only one type of mineral oil can be stored at that location, because a pipeline in the sea can not be cleaned. Another option could be to set up a location for strategic long term storage of oil products. Vopak is currently establishing such a location at the Eemshaven in Groningen, so it is questionable whether there is a demand for this type of storage space in the near future.

There are also strict technical requirements to the construction of a storage tank. Most tanks in the harbor have pile foundations. Because of the low bearing capacity of the Slufter’s dredged material piles will most likely be necessary. And in that case only sites near the ring-dike, where the layer of dredged material is thin, may qualify. The confined and isolated location of the Slufter does make it safe for the handling of flammable products.

Land on Maasvlakte I, with the purpose of storage of chemicals or oil products, is issued by the Port Authority for a rent of 6-9\(¥/m^2\). If 50ha is issued, the revenue can be 3-4.5 million Euro per year. The investment for making the area suitable for storage can probably be recovered within a couple of years. However, this will only become interesting in the long term, if storage space is scarce.

**Location Bevi-companies**

Bevi stands for ‘Besluit externe veiligheid inrichtingen’ or in English, the act on the external safety of company layouts. This act applies to companies that work with and store dangerous substances. It makes sure that the company has the correct permits and operates in accordance with the regulations regarding risks and emissions. Many companies in the Rotterdam harbor are bevi-companies. Imaginary risk contour lines have been drawn around these companies’ layouts to indicate areas where the probability of the loss of a human life as a result of a calamity is one in a million per year \(10^{-6}\).

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The province of Zuid-Holland aims to reduce the risks by clustering these companies in one area and to combine risky activities responsibly. These aims will be included in the zoning plans for the future of these areas. For the Port Authority this means that space for risky companies is concentrated on the premises of Maasvlakte I and II, Europoort and Botlek-Vondelingenplaats. However, there is a lack of space. In fact, at the moment the demand for space is larger than the available space, including Maasvlakte II [25].

The isolated and confined location of the Slufter might provide an opportunity to accommodate some of these companies here. There are few people in the direct vicinity, practically no vulnerable people (children, elderly or sick people) and good opportunities for escape.

Opposition may arise from the people in the community of Westvoorne, who prefer a ‘green’ function over another function which deals with industrial products and waste. Actual realization of a location for bevi-companies at the Slufter depends on the type of storage, layout size and the exact location that is to be made available. Some types require more space and more construction and safety measures than others. It also depends on the revenues the Port Authority may generate from the rent of the issued land.

Hydropower station

The idea of a hydropower station in the Slufter originates from the ‘Plan Lievense’ [7]. This plan proposes a way to store energy by increasing the water level in a lake during an overcapacity of energy, for example produced by windmills. The increased head difference could in turn be used to allow for water flow through turbines and thereby generate electricity in times of higher demand. The same principle could be applied in the Slufter basin, either by exchange of water with the sea or internally by exchange between two basins with varying heads. The already existing windmills would then provide the energy to be stored (see Figure 3.8).

![Figure 3.8: Schematization of the principle to generate hydropower using the head difference in the Slufter basin](image)

The amount of energy that can be stored in one cycle of filling and emptying is given by the following formula:

\[ E = \frac{1}{2} \times \rho \times g \times A \times H^2 \]
where $A$ is the average surface area of the lake and $H$ is the head difference. Suppose this can be applied to the Slufter. The average surface area from the top of the dredged material to the crest of the ring-dike is approximately 288ha. The head difference from the top of the ring-dike to NAP level is 24m. Assuming some losses due to pipeline distance, friction and bends, the effective head difference is 20m. The amount of energy stored is then:

$$E = \frac{1}{2} \times 1025 \times 10 \times 2.88 \times 10^6 \times 20^2 = 5900\text{MJ}$$

Electric energy is measured in kWh, so if 1kWh equals 3.6MJ, then 1640kWh is stored in one cycle. An average Dutch family consumes 9kWh per day. If the basin is filled in one day, then 182 families can be provided with power daily. In that case pumps, pipelines and turbines are needed that can discharge 670m$^3$ of water per second. The flow velocity through the pipeline is $\sqrt{2gH} = 20\text{m/s}$. This comes down to a pipe diameter of 6.5m, which in the Slufter’s case is unrealistically large. Multiple pipelines with smaller diameters would be needed.

Another problem of this idea is that the water exchange in and out of the basin is between contaminated water from the Slufter and clean seawater. Additional measures have to be taken to constantly filter the exchanged water. This problem could be solved if the exchange is kept within the basin. In that case a separating dike in the middle is needed to realize the head difference. This dike would then need a height equal to the existing ring-dike and allow for filling and emptying twice a day to obtain the same amount of energy.

A fluctuating groundwater level as a result of the filling and emptying may result in problems for the surrounding areas. Obviously many other obstacles can be seen, such as investment costs and the structural challenges of the power plant. It would be a prestigious project, but not very realistic.
3.4 Evaluation

In the previous section all the remaining potential future functions have been further analyzed. In Table 3.2 the pros and cons are summarized for each function. On the subsequent pages, the functions are evaluated on their potential as the future function of Slufter. This is done by category: Energy, Storage, Nature & Recreation, and Other functions. At the end of this section, an overview is given of the short and long term possibilities.

<table>
<thead>
<tr>
<th>Function</th>
<th>Pros</th>
<th>Cons</th>
</tr>
</thead>
</table>
| 1. Solar energy | - Renewable energy source  
- Light weight and efficient use of space  
- Excess power transferred to the national power grid | - Investment not economically viable in the short term  
- Frequent lack of sunshine in the Netherlands |
| 2. Temporary storage of soil (TOP) | - Sufficient space  
- In line with current function Slufter | - Remote location/ High transportation costs |
| 3. Birds reserve | - Enough shelter, food and quiet  
- Potential permanent residence for sea-gulls | - Temporary nature exemption difficult to obtain  
- Contaminations might pose a risk |
| 4. Algae breeding farm | - Renewable energy source  
- There is a market (food and pharmaceutical industry)  
- Might be possible without too much adaptation of the area  
- Supply of CO2 and electricity nearby | - No bio-fuels |
| 5. Windmills | - Renewable energy source  
- Technically feasible in the ring-dike | - Limited options for expansion  
- Opposition Westvoorne community |
| 6. Soil cleansing and recycling | - Creates more storage space  
- The soil can be used as elevation material | - Soil can not be sold commercially |
| 7. Empty depot | - High revenues from the rent | - Enough space already reserved on Maasvlakte I  
- Straight and horizontal internal road required |
| 8. Forest on dredged material | - Proven that trees grow well on dredged material  
- Has a nature function | - Too small scale for production of timber or timber products |
### 3.4. EVALUATION

<table>
<thead>
<tr>
<th>Function</th>
<th>Pros</th>
<th>Cons</th>
</tr>
</thead>
</table>
| 9. Nature reserve         | - Can be established with temporary nature exemption  
- Compensates for industry, now and in the future  
- Might evolve into a permanent nature reserve | - Difficult to move endangered species if converted to industry                                                                                                                                                         |
| 10. Biomass production    | - Renewable energy source                                                                                                                                                                               | - Unsuitable climate conditions to grow products fit for biomass processing  
- Production too small scale to be economically viable  
- Other locations more suitable for biochemical processing plant (Maasvlakte I/II) |
| 11. Service center        | - Provides fire department services to the area  
- Does not take up a lot of space                                                                                                                                                                            | - Maasvlakte II provides enough space and has better connectivity in the future                                                                                                                                       |
| 12. Greenhouse cultivation| - Agricultural land is valuable  
- Heat and CO2 sources nearby                                                                                                                                                                              |                                                                                                                                                                                                                         |
| 13. Fish breeding farm    | - Growing market  
- EU subsidies  
- Knowledge and experience available  
- Can be executed on small scale  
- Combinable with algae breeding (fish food substitute)                                                                                                                                                               | - Influences overfishing negatively due to fish food  
- Strict requirements to the quality  
- Weak competitive position Dutch fish breeding market                                                                                                                                                    |
| 14. Golf course           | - Fast growing sport  
- Nice scenery  
- Opportunities for bird- and wildlife                                                                                                                                                                       | - High investment and maintenance costs  
- Requires a lot of space  
- Already one other golf course in the vicinity                                                                                                                                                    |
| 15. Storage of chemicals  | - High revenues from the rent and throughput  
- Confined and isolated location                                                                                                                                                                           | - Better opportunities for mineral oil products  
- Already enough capacity for the coming 10 to 15 years  
- Large pipeline transport distance  
- Pile foundations required                                                                                                                  |
| 16. Location Bevi- compa-| - Remote and confined area  
- Demand for space for these type of companies                                                                                                                                                          | - Opposition community Westvoorne  
- Uncertainty about type of companies and scale                                                                                             |
| nies                      |                                                                                                                                                                                                                                                                     |                                                                                                                                                                                                                         |
| 17. Hydropower station    | - Method to store renewable energy  
- Prestigious                                                                                                                                                                                               | - Many challenges to overcome  
- Not very realistic                                                                                                                                    |
Energy  Plenty of opportunities can be seen for the development of renewable energy at the Slufter. Especially solar energy and wind energy are functions that are applicable. The breeding of algae is also a promising option, be it not to produce bio-fuels, but to provide ingredients for the food and pharmaceutical industry.

The actual production of biomass has to take place on a large scale, larger than the Slufter can provide. A biomass processing plant could be realized at the Slufter, but there are other undeveloped locations on Maasvlakte I that are more suitable in the short term. Therefore there is no surplus value of such a plant at the Slufter.

A hydropower station to store renewable energy from the sun or the wind is a prestigious idea, but many challenges would have to be overcome. Therefore this idea does not seem very realistic at this stage.

Storage  During this analysis the Slufter has not proven to be a very suitable location for the storage of other types of materials. Temporary storage of soil is possible, but there are other places in the harbor, nearer to the projects, that are more favorable. Empty depot may generate large revenues, but enough space is already reserved and the fact that an internal road is required does not make it worth taking into consideration as an alternative. The same goes for storage of chemicals. Other locations are more suitable in the short term and the measures to be taken at the Slufter drastic. It is an option if the demand for space is very high.

The Slufter as a potential location for Bevi-companies that need remote and confined storage space is a function worth looking into further.

Nature & Recreation  Besides the fact that the proposed future of the Slufter has always been nature, it seems that a nature and recreational function has a positive impact on the port of Rotterdam in multiple ways. A forest on dredged material can be combined with a nature reserve and the circumstances are excellent for the establishment of a birds reserve. Moreover, any type of nature function is a compensation for any industrial expansion, now and in the future.

A golf course is not the obvious choice for the future of the Slufter, because of the large amount of space required, the large investment costs and the fact that there is already another golf course in the vicinity.

Other functions  Of the other types of functions discussed, the greenhouse cultivation and the fish breeding farm are worth looking into further. Lack of agricultural land in the Netherlands and the availability of the basic necessities for greenhouse cultivation near the Slufter makes it an interesting alternative. The fish breeding farm alternative has many pros, especially if a combination with algae breeding is possible. A soil cleansing and recycling facility can be installed at the Slufter to produce dried sediment that can be used as an elevation material. However, this would be more expensive than using clean sand which is available in the vicinity.

A service center at the Slufter has proven to be unnecessary.
To sum up, the best opportunities in the short term can be seen for wind and solar energy, an algae breeding farm and for a nature and birds reserve. However, a closer look at greenhouse cultivation and a fish breeding farm is recommended. The Slufter might also be a suitable location to accommodate Bevi-companies. In the long term, when the Slufter is no longer needed as disposal facility, the area can be converted into industry and given harbor related functions.

Short term options:
- Solar energy
- Algae breeding farm
- Windmills
- Greenhouse cultivation
- Fish breeding farm
- Location for Bevi-companies
- Nature/ birds reserve

Long term options:
- Empty depot
- Storage of oil products or chemicals
- Biomass power plant
- Nature/ birds reserve

Less favorable options:
- Temporary storage of soil (TOP)
- Soil cleansing and recycling
- Service center
- Golf course
- Hydropower station
Chapter 4

Spatial analysis

In this chapter the geometry of the Slufter and the possibilities for the use of space will be analyzed. In the first three sections the required information is gathered and explained. This information is used to analyze the two alternatives in Sections 4.4 and 4.5.

The Slufter basin is enclosed by a ring-dike with a circumference of approximately 6km. Of the 260ha surface area of the Slufter, approximately 60ha is taken up by the enclosing dike at NAP +24m and 200ha by the basin. With a maximum depth at NAP -29m, the volume of the Slufter basin is approximately 100 million m$^3$ and the storage capacity is approximately 150 million m$^3$. The storage capacity is larger than the volume because consolidation of the dredged material will take place once inside the basin, resulting in a volume reduction.

4.1 Geometry

The geometry of the Slufter determines the spatial possibilities of the area. In itself, the Slufter is awkwardly shaped, with varying geometry over length, width and depth. In the 1997 soil investigation [18] the geometry of the Slufter was approximated by estimating the surface area at each level where the slope angle changes over depth $z$, as shown in Table 4.1.

<table>
<thead>
<tr>
<th>Average surface area in ha</th>
<th>Height $z$ [m] from bottom at NAP-29m</th>
</tr>
</thead>
<tbody>
<tr>
<td>90+(148-90)$z$/28</td>
<td>0&lt;$z$&lt;28</td>
</tr>
<tr>
<td>148+(178-148)(z-28)/2</td>
<td>28&lt;$z$&lt;30</td>
</tr>
<tr>
<td>178+(264-178)(z-30)/1</td>
<td>30&lt;$z$&lt;31</td>
</tr>
<tr>
<td>264+(312-264)(z-31)/20</td>
<td>31&lt;$z$&lt;51</td>
</tr>
</tbody>
</table>

According to the table the crest of the ring-dike is at NAP +22m. This geometry dates from before the heightening of the ring-dike from NAP +22m to a level of NAP +24m. For further analysis with respect to the spatial possibilities of the area this extra height can be considered irrelevant, since no retaining structure or soil body will be elevated to that height.

Based on the numbers in Table 4.1 the total volume of the basin is 93.3 million m$^3$. 

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CHAPTER 4. SPATIAL ANALYSIS

The shape and dimensions of the Slufter are presented schematically in Figure 4.1. Note that the horizontal dimensions in this figure are represented in hectares to signify the surface area of the different parts.

The central part of the basin runs from NAP -29m up to NAP -1m, with a surface area of approximately 149ha. From NAP -1m up to NAP+24m there is a gentle slope and a steeper slope in the ring-dike. These parts make up for an area of 164ha. Future functions which require pile foundations could be applied in these areas because of the better geotechnical properties of the sand in the ring-dike. The layer of dredged material here is thinner and therefore less settlement can be expected.

4.2 The level of dredged material

The actual bottom profile as a result of the past 23 years of filling varies from place to place within the Slufter basin. The most recent soundings show a bottom level varying from NAP +0.1m in the central sections to NAP +4.5m near the banks, marking the points that are still accessible to the survey pontoon. The bottom profile can be said to be roughly sloping from the center towards the ring-dike. Figure 4.2 shows a recent 3-dimensional presentation of the bottom profile. The red and orange parts indicate the deeper sections and the blue parts are the shallower sections. A few irregularities and shoals can be discerned. On average, the bottom profile is at approximately NAP +2m.

4.3 The remaining storage capacity

In 2008, FFact Management Consultants has, in collaboration with the Slufter's advisory group, made a strategic analysis for the optimal use of the Slufter's remaining capacity [11]. In 2007, the Slufter was filled for 48%, which corresponds to a remaining capacity of 44.8 million m$^3$. 

Figure 4.1: Schematization of the Slufter
4.3. THE REMAINING STORAGE CAPACITY

Recent developments make it difficult to estimate exactly how much capacity is required for the future. It is uncertain how much capacity is required for the future. It is uncertain how much the supply from third parties will develop, especially the quantities brought in from abroad. Furthermore, it is unclear how Rijkswaterstaat will use its capacity and what the long term vision of the ministry of ‘Infrastructuur & Milieu’ is (the ministry of infrastructure and environment). Figure 4.4 shows the quantities of dredged material per year stored in the Slufter since 2006. It should be noted that these quantities are expressed in chargeable cubic meters, which includes transportation water. The actual load on the storage capacity is less. With respect to 2006, the supply of dredged material has decreased significantly. However, the supply slightly increased again from 2009 to 2010 and is expected to increase further by the end of 2011. Nevertheless, it is difficult to predict what the supply will be in the future. In general, it is expected to decrease.

To cover these uncertainties and make a reasonable estimation of the required remaining capacity a distinction is made between an optimistic and a realistic scenario. In the optimistic scenario 1/4 of the total capacity of the Slufter can be used for future development or, in other words 3/4 has to be used for the storage of dredged material. This corresponds to a volume of 70Mm$^3$, however, the major part has already been filled. The current dredged material volume, derived from the schematization of the Slufter, presented in Figure 4.1 is 38.8 million m$^3$, which is 42% of the total basin volume and thus an underestimation of the filling found in the report of FFact. The remaining capacity thus needs to be:

$$70 - 38.8 = 31.2Mm^3$$

The realistic scenario leaves room for more uncertainties and therefore more space. Here,

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1The supply of dredged material brought in from abroad is a recent development. The first quantities have come in from Bremerhaven, Germany, in early 2011. Since then, other harbors have shown interest in storing their dredged material at the Slufter.
approximately 4/5 of the total capacity has to be used for the storage of dredged material, which corresponds to a volume of 74.6Mm$^3$. Here the remaining capacity has to be:

$$74.6 - 38.8 = 35.8Mm^3$$

![Figure 4.3](image)

**Figure 4.3:** Remaining capacity of the Slufter in million m$^3$ in-situ per year

![Figure 4.4](image)

**Figure 4.4:** Annual quantities of dredged material stored in the Slufter since 2006
4.4 Alternative 1: A retaining structure

The size of the surface area that can be developed and used for new functions depends on the location and height of the retaining structure. If a vertical retaining structure is applied, it is fairly easy to determine what this size will be. However, when an embankment is chosen, the configuration of the slope has to be taken into account as well. The spatial analysis here focusses on a retaining structure in the shape of an embankment.

The bottom profile is assumed to be level and at NAP +2m, as shown also in Figure 4.1. This level is derived from the 2010 soundings (see Figure 4.2).

Separation of a part of the basin is a function of the height of the embankment and the surface area. To calculate the optimal configuration, the upper part of the basin (from NAP +2m) is schematized as shown in Figure 4.5.

![Figure 4.5](image)

**Figure 4.5:** Schematization for the calculation of the remaining capacity

With the shape of a trapezium, the volume of the remaining capacity can be approximated by:

\[ V = \frac{(rz + x + 1.2z + x)x}{2} = \left( \frac{1}{2}r + 0.6 \right)z^2 + xz \]

where \( r \) is a correction factor for obtaining the equivalent hectares of the surface area above the slope of the embankment. Therefore, \( r \) depends on the chosen slope. It should be noted that the values in the calculation are in hectares.

If the surface \( x \) is calculated all the values are known, including the resulting hectares of land. By playing with different slopes and elevation heights the spatial possibilities can be investigated.

Suppose the embankment will have an elevation of 15m above the dredged material level, then the surface area in the optimistic and realistic scenario can be obtained. For various slopes the resulting hectares are presented in Table 4.2 and shown in Figure 4.6.

From this analysis can be concluded that there is a significant difference in the area of land that can be obtained when comparing the optimistic scenario with the realistic scenario. In the realistic scenario either a steeper slope or a higher elevation is required in order to obtain enough land. In the optimistic scenario a reasonable area can be obtained with gentle slopes.

However, there is more to be aware of. Suppose an embankment is built with an elevation of 15m, a crest width of 10m and a slope of 1:10 on both sides (see Figure 4.7). In this
CHAPTER 4. SPATIAL ANALYSIS

Table 4.2: Obtained surface area in hectares when \( z = 15m \)

<table>
<thead>
<tr>
<th>Slope i [-]</th>
<th>Optimistic [ha]</th>
<th>Realistic [ha]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>83</td>
<td>52</td>
</tr>
<tr>
<td>5</td>
<td>68</td>
<td>37</td>
</tr>
<tr>
<td>10</td>
<td>53</td>
<td>22</td>
</tr>
<tr>
<td>15</td>
<td>38</td>
<td>7</td>
</tr>
<tr>
<td>20</td>
<td>23</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 4.6: Analysis of the obtainable surface area in the optimistic and realistic scenario if the elevation height \( z = 15m \)

example, the crest and the inner slope take up 16ha of created space. Therefore the remaining horizontal area reduces to:

Optimistic: 53 - 16 = 37ha
Realistic: 22 - 16 = 6ha

The minimum amount of sand required for the embankment per running meter is the 2400m

(\text{without taking the settlement into account}). Suppose the length of the embankment is

1000m, then 2.4 million m

of sand is required.

More horizontal land can be gained if the space between the ring-dike and the embankment is elevated. However, another option is to lower the outer ring-dike to a certain level. The result is a level area of land bounded by the embankment with its crest at NAP +17m, as shown in Figure 4.8.

Port of Rotterdam Authority
4.4. ALTERNATIVE 1: A RETAINING STRUCTURE

Figure 4.7: Exemplary schematization of the use of space in case an embankment is built

Suppose the ring-dike is lowered to a level of NAP +7m along the entire circumference. The sand is used to create the embankment and to fill up the area in between.

Figure 4.8: Schematization of the use of space when the original ring-dike is lowered to a level at NAP +7m

The cross-section of the ring-dike is not the same everywhere; the wider part includes the access road to the beach and runs approximately from the southeastern tip along the southern section up to the northwestern tip. Its length is approximately 3200m on average. The remaining northern part has a thinner cross-section, since there is less infrastructure. Here the length is 2400m on average.

Lowering the wider part results in a supply of sand of $1235 m^3/m^3$ and the corresponding horizontal width at NAP +7m is 131.5m. Over a length of 3200m the supply of sand then becomes $4 M m^3$ and the surface area 42ha.

Similarly, the supply of sand in the thinner part is $576.5 m^3/m^3$ and the horizontal width at NAP +7m is 109m. Over a length of 2400m the supply of sand becomes $1.4 M m^3$ and the surface area 26ha.

The total supply of sand is thus:

$$V_t = 4 + 1.4 = 5.4 M m^3$$

And the surface area is:

Optimistic: $42 + 26 + 37 = 105ha$

Realistic: $42 + 26 + 6 = 74ha$

Quite a large area can thus be realized by separating a part of the basin by means of a retaining structure. Now, the real remaining storage capacity and obtained surface area

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depends on the actual configuration of the embankment within the Slufter basin. In Figure 4.9 the embankment with a crest height at NAP +17m is drawn at a random place. Its length is approximately 1.5km.

![Figure 4.9: An embankment randomly placed within the Slufter basin](image)

Assuming an average bed level of the dredged material at NAP +2m, the surface area at the bottom is 115ha and the surface area at NAP +17m is 155ha. In this case the estimated remaining capacity is 19.7Mm$^3$, which is not enough according to the remaining storage capacity requirement in this analysis. However, suppose that the average supply of dredged material in the future is 300,000m$^3$ per year, then 19.7Mm$^3$ provide storage space for another 65 years. The surface area between the crest of the embankment and the ring dike is 77ha (see Figure 4.10).

If this solution for creating land is technically and economically feasible then other configurations have to be looked at to ensure that there is enough remaining capacity and to see what the actual obtained surface area of land is. The technical and economical feasibility is elucidated in Chapter 5.

Allowing a process of clay ripening requires a different approach, which might be a good alternative. This is investigated in the next section.
4.5 Alternative 2: Clay ripening

The alternative of clay ripening is based on lowering the water table in the basin. In this situation the bed level of the dredged material is approximated more according to the actual situation, sloping from the center towards the ring-dike. This is schematized in Figure 4.11.

The bed level is sloping from NAP +0m at the center to NAP +6.5m at the ring-dike. The water table is assumed to be at NAP +6m. To determine the resulting surface area when changing the water level, the schematization in Figure 4.12 is used.

Assuming the shape of a cone with the base at the water table and the tip at the dredged material in the center of the basin, the remaining volume can be approximated by:

\[ V = \frac{1}{3} \times x \times z = \frac{1}{3} \times 42.3z^2 \]

with \( x \) in hectares and \( z \) in meters. Applying this approximation for the optimistic and the realistic scenario results in \( z = 11.5m \) and \( z = 12.9m \). This is impossible, since \( z \) is bounded by NAP +0m and NAP +6.5m. Therefore it can be concluded that an extra retaining structure is required surrounding the area that remains for the storage of dredged material. However, the water table can still be lowered to allow for the ripening process to begin. In that case the storage capacity will be smaller temporarily, but possibly still large enough to meet the supply. Suppose the water table is lowered by 2m, then \( z = 4m \) and \( x = 169.2ha \). The area where clay ripening can take place then becomes:

\[ 274.8 - 169.2 = 105.6ha \]
and the remaining storage capacity becomes:

\[ V = \frac{1}{3} \times 42.3 \times 4^2 = 225.6 \quad \text{which corresponds to 2.26 million m}^3. \]

Suppose an annual supply of 400,000m³, then the basin could be exploited in this way for another 5 to 6 years. It depends on the ripening process whether or not this is sufficient, because this determines the bearing capacity and thus the possibilities for development.

The spatial analysis is now taken a step further. The required remaining capacity of the Slufter is 31.2Mm³ in the optimistic and 35.8Mm³ in the realistic scenario. Taking the optimistic scenario as a starting basis, the spatial possibilities of a second, inner ring-dike are analyzed. If the slopes of the inner dike are 1:10 on both sides and the crest width is 10m, then the required elevation height in order to obtain enough remaining capacity is 13m. This example is schematized in Figure 4.13.

The consequence of an inner dike with these dimensions is that there is an overlap of 56ha with the existing ring-dike on both sides. Moreover, this solution requires a very large amount.
4.5. ALTERNATIVE 2: CLAY RIPENING

There are four other possibilities for the clay ripening alternative, with a different approach. Each of these approaches will be discussed in the following paragraphs.

The first approach is similar to the retaining structure alternative. In this case the water table is lowered to allow for the ripening process to begin. After a specific number of years a crest layer of ripened clay has formed which is accessible by construction equipment and from thereon development can take place. In the meantime dry storage of dredged material can continue. An embankment can be erected up to the previously calculated height of NAP +15m in the optimistic scenario, to allow for wet storage for the remaining exploitation period of the Slufter. The embankment can be built using the sand from the old ring-dike.

Suppose the original ring-dike is lowered to a level of NAP +7m, similar to what was done in Section 4.4. If the inner ring-dike has 1:10 slopes on either side, it will take up approximately 220ha of surface area. However, the space available at NAP +7m is 26 + 42 = 68ha plus the 105.6ha if \( z = 4m \) (see Figure 4.12 and the previous paragraphs). In other words, no horizontal surface area remains in this case and therefore it does not present a solution.

The second approach is to vouch for dry storage of dredged material for the rest of the Slufter’s exploitation period. That way a lower embankment may suffice.

If the dredged material is stored dry, the capacity increases by a factor 1.5, since the cubic meters are used more effectively. The required remaining capacity therefore becomes 20.8Mm\(^3\) in the optimistic and 23.9Mm\(^3\) in the realistic scenario. For this dry storage alternative the same approach is followed to determine the required crest height and the obtained area of land. The results of the analysis are summarized in Table 4.3. It shows the required amount of sand per running meter and the resulting hectares of land that can be obtained. For comparison, this has also been determined for an embankment with slopes of 1:3. The amount of hectares can be increased considerably with steeper slopes. However, at this stage it is unclear whether this is technically feasible. Wet storage in combination with clay ripening is therefore not an option, because no horizontal surface remains. If it is technically feasible to build an embankment on top of the layer of ripened clay, dry storage provides a solution.

The third approach could be to focus on the shallow parts of the basin, near the ring-dike. It might be that the realization of an embankment in these parts is more feasible, because of...
CHAPTER 4. SPATIAL ANALYSIS

Table 4.3: Summary of the spatial possibilities of an embankment on ripened clay, for the first and second approach

<table>
<thead>
<tr>
<th></th>
<th>V_remaining [m$^3$]</th>
<th>Elevation height [m]</th>
<th>V_sand [m$^3$/m']</th>
<th>Area [ha]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Realistic</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wet</td>
<td>35.8</td>
<td>15</td>
<td>2400</td>
<td>1880</td>
</tr>
<tr>
<td>Dry</td>
<td>23.9</td>
<td>10.5</td>
<td>1208</td>
<td>950</td>
</tr>
<tr>
<td><strong>Optimistic</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wet</td>
<td>31.2</td>
<td>13</td>
<td>1820</td>
<td>1500</td>
</tr>
<tr>
<td>Dry</td>
<td>20.8</td>
<td>9</td>
<td>906</td>
<td>743</td>
</tr>
</tbody>
</table>


A thinner layer of dredged material. Assuming the bed level of the dredged material again at NAP +2m, then the height of an embankment would have to be 10m high in the realistic scenario. Slopes of 1:10 and a crest width of 10m gives a structure with a width of 210m at the base. The width of the shallow part is only 100m. So in this case the embankment would also intersect with the existing ring-dike. However, suppose that the existing ring-dike is lowered along the entire circumference to a level of NAP +12m, then the obtained area is approximately 100ha. A volume of 650m$^3$ per running meter would then be required.

![Figure 4.14: Schematization of the third approach: lower and widen the ring-dike](image)

The fourth approach proposes the dry storage of dredged material in the entire basin for the remaining life span of the Slufter as a disposal facility. This principle is based on an old storage method, where the dredged material was spread over a designated area up to a certain elevation. The ‘Broekpolder’ and ‘Oost Abtspolder’ near Rotterdam are examples of areas where this method has been applied.

Once the first crust of ripened dredged material has formed, more layers can be sprayed on top, in the same sequence as the quantities to be stored. For instance, suppose the annual supply of dredged material is 300,000m$^3$/year and the desired elevation speed is one meter per year, then 300,000m$^3$ or 30ha is elevated. After a few years the area is accessible and can be further developed. The obtained land can then be further expanded in the same manner until either the supply has become insignificantly small or the entire surface has been elevated. If by that time more storage capacity is needed a smaller inner ring-dike can be realized to extend the Slufter’s storage function. The reclaimed areas near the deepest part of the basin can be further developed, for instance by locally installing a system of...
4.5. ALTERNATIVE 2: CLAY RIPENING

vertical drains.

When applying this approach, the Slufter’s ring-dike is no longer needed or at least not in its current stature. Suppose the wish is to still contain the ripened dredged material between dikes, then the crest of the lowered ring-dike would have to be at NAP +9.6m in the optimistic scenario and NAP +10.7m in the realistic scenario. However, this alternative is flexible, i.e. the choice to lower the ring-dike can be made according to the change in supply of dredged material and required remaining capacity. In case it is decided that the ring-dike is no longer needed the full 260ha of land occupied by the basin can be developed. The volume of sand that becomes available can be used as surcharge material to enhance the consolidation process of the deeper layers of dredged material, and it may function as a cover for the contaminations.

This fourth approach combines a long term solution for the entire basin with short term development of small areas in phases.

![Schematization of the fourth approach: dry storage and layer by layer ripening of incoming dredged material](image)

**Figure 4.15:** Schematization of the fourth approach: dry storage and layer by layer ripening of incoming dredged material

In the short term, designated areas can be elevated and developed first. Logically these areas are the shallow parts near the ring-dike, because the layer of dredged material is thin here. The width of the shallow parts at NAP +2m is approximately 100m. Figure 4.16 shows an exemplary strip of land along the shallow part of the basin. In this example a surface area of 22ha is obtained. Various functions can be accommodated here.
4.6 Evaluation

In this analysis, the level of dredged material is assumed to be horizontal and at approximately NAP +2m. The required remaining storage capacity is estimated for an optimistic and a realistic future filling scenario. In the optimistic scenario 3/4 of the basin capacity has to be used for storage of dredged material, which corresponds to a remaining storage capacity of 31.2Mm³. In the realistic scenario 4/5 of the basin capacity has to be used for storage of dredged material, which corresponds to a remaining storage capacity of 35.8Mm³.

The possibilities for the use of space in case of the retaining structure alternative depend on the structure’s elevation height and its slope. The elevation height in particular determines the new storage capacity. The Slufter’s ring-dike can then be lowered to that same level.

Because of the low bearing capacity of the dredged material it may be expected that a retaining structure in the shape of an embankment will require gentle slopes. An embankment with an elevation height of 15m and slopes of 1:10 results in 37ha of land in the optimistic scenario and only 6ha in the realistic scenario. More land can be obtained if the ring-dike is lowered and the area in between the embankment and the ring-dike elevated.

The basic approach followed in the clay ripening alternative is to remove the water sheet and lower the ground water level in the basin. Thereby, a process of clay ripening is initiated. Supposedly, a second inner ring-dike could then be realized on top of a layer of ripened
dredged material. However, the analysis shows that because of the optimistic and realistic remaining storage capacity requirement, no horizontal land remains if the inner ring-dike has an elevation height of 13m and slopes of 1:10. Four different approaches have been analyzed to find a suitable alternative to the basic approach. These approaches have been listed below, together with an estimation of the reclaimable hectares of land.

1. Allow five years of clay ripening and dry storage, then realize an embankment and maintain wet storage: [0ha]
2. Allow five years of clay ripening and dry storage, then realize an embankment and maintain dry storage: [21ha optimistic, 37ha realistic]
3. Focus on the shallow parts of the basin near the ring-dike; lower and widen the ring-dike, and maintain dry storage: [100ha]
4. Initiate clay ripening and dry storage, and slowly elevate consecutive parts of the basin until a thick layer of ripened clay makes further development possible. The lowering of the ring-dike is optional: [with an elevation height of 1m per year, approximately 30ha per year]

Thus, dry storage presents better opportunities for the use of space. The second and third approach are short term options. The fourth approach provides a solution for the entire basin in the long term, whilst designated areas can be elevated in the short term. The viability of approach 3 and 4 depends on the geotechnical feasibility, which is analyzed in the next chapter.

The possibilities of a retaining structure in the shape of an embankment are minimal, because of the remaining storage capacity requirement. However, suppose the average annual supply of dredged material for the remaining lifespan of the Slufter as a disposal facility is 300,000m$^3$ (which is not unlikely) then storage can continue for another 100 years in the optimistic scenario. This gives the impression that the remaining storage capacity requirement is too strict. If this requirement is less strict there will be better opportunities for the retaining structure alternative.
Chapter 5

Geotechnical analysis

First, the initial conditions indicate what the current geotechnical condition of the dredged material is. Then the geotechnical requirements are posed to serve as a guideline for the calculations. In Section 5.3 the feasibility of a retaining structure is analyzed, and in Section 5.4 the possibilities of clay ripening are investigated as an alternative. Finally, the results of the analysis of the two main alternatives are evaluated.

5.1 Initial conditions

The initial conditions are given by the geotechnical properties of the Slufter’s dredged material. These properties will serve as the input parameters for the geotechnical calculations. All parameters are derived from the 1997 soil investigations [18]. Appendix E gives the underlying thoughts of how each parameter has been derived. The required parameters are summarized in Table 5.1.

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
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<td>Mixture density</td>
<td>$\rho_m$</td>
<td>1300-1400</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td>Solid density</td>
<td>$\rho_k$</td>
<td>2500</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td>Water density</td>
<td>$\rho_w$</td>
<td>1005</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>$c_u$</td>
<td>1-5</td>
<td>kPa</td>
</tr>
<tr>
<td>Primary compression constant</td>
<td>$C_p$</td>
<td>20.1</td>
<td>[-]</td>
</tr>
<tr>
<td>Secondary compression constant</td>
<td>$C_s$</td>
<td>481</td>
<td>[-]</td>
</tr>
<tr>
<td>Consolidation coefficient</td>
<td>$c_v$</td>
<td>$1.26\times10^{-7}$</td>
<td>m$^2$/s</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>$I_p$</td>
<td>24</td>
<td>[%]</td>
</tr>
</tbody>
</table>

The mixture density varies over depth from approximately 1300kg/m$^3$ at the top layers to slightly more dense at the bottom (1400kg/m$^3$). The undrained shear strength varies from approximately 1kPa to 5kPa over depth and per location. A lower limit value of 1kPa is used in the calculations to be safe.
CHAPTER 5. GEOTECHNICAL ANALYSIS

5.2 Geotechnical requirements

The three main geotechnical requirements are with respect to:

- Bearing capacity: the subsoil must be able to bear the load applied by equipment in the construction phase and the load of the future function in the users phase
- Slope stability: the slopes of the retaining structure must be sufficiently safe against sliding
- Settlement: for every function there is a maximum allowable amount of settlement

In order to check whether a conceptual design is feasible with respect to the bearing capacity and the slope stability, the strength of the structure has to be sufficient to withstand the load. The conditional statement shows that the value of the load has to be smaller than or equal to the value of the strength:

\[ S \leq R \]  \hspace{1cm} (5.1)

where:

\[ S = \text{the load} \]
\[ R = \text{the strength} \]

Equation (5.1) can also be written as:

\[ \frac{R}{S} \geq SF \]  \hspace{1cm} (5.2)

where SF represents the Stability Factor, which determines whether the structure is dimensioned sufficiently safe under the given load conditions. In conventional geotechnical engineering a value for the SF of 2 was chosen. This value is conservative and might result in over-dimensionalized structures. Nowadays, geotechnical structures are designed using common standards such as those provided by NEN or CUR to optimize structure safety and use of building materials. To arrive at the design values used in calculations, the representative values for the load and the strength are multiplied and divided by a partial factor, respectively. However, to study the feasibility of the technical solution for the Slufter, it suffices to use an SF of 2 as a starting point.

A distinction is made between load conditions in the construction phase and in the users’ phase. The configuration of the soil structure in the construction phase has to withstand the temporary loads applied by the equipment and the consequences of failure are less severe. Therefore, an SF of 1.2 is considered sufficiently safe. In the users’ phase long term strength and stability have to be ensured and failure is not an option. Here an SF of 2 is required.

Another distinction is made between the acceptable amount of settlement as a result of the applied load. In some situations settlement is not an issue whereas in other situations the settlement has to be kept as small as possible.

The geotechnical requirements that are in line with the aims of this research are established by the potential future functions of the Slufter.
5.3. ALTERNATIVE 1: BUILDING A RETAINING STRUCTURE

A subdivision is made in light, medium and heavy weight operations and structures. Light weight includes recreational or nature type of functions. Medium weight functions have more strict requirements with respect to geotechnical safety, because of the deployment of function-related structures, such as solar energy generation or greenhouse cultivation. Heavy weight functions deal with large load conditions and/or hazardous materials, such as storage of dry soil or chemicals.

The geotechnical analysis is limited to functions with shallow foundations or functions where the load condition can be schematized as a shallow foundation. Function-related buildings or structures that require a pile foundation will not be considered here, because of the uneconomic length of the piles that would be required in that case. Pile foundations can be applied in the Slufter, but preferably near or on top of the ring-dike.

5.3 Alternative 1: Building a retaining structure

This part of the geotechnical analysis investigates the possibility to build a retaining structure in the Slufter basin to separate the storage function from the future function. The first two sections explain the choice of the type of retaining structure and the execution method. This is followed by a study of the geotechnical feasibility of the proposed solution, based on the exploratory calculations described in Appendix [D]. In the calculations a distinction is made between the construction phase and the users phase. Finally, the execution costs of the retaining structure are estimated and the proposed method to create land in the Slufter is evaluated. In the last part of the retaining structure alternative, a section is dedicated to a different retaining structure, which makes use of geosynthetic materials.

5.3.1 Analysis and selection of the type of retaining structure

Retaining structures can either retain a body of soil, a body of water or both. In case of the Slufter, the retaining structure will have fulfill two purposes. The first is to separate the storage function from the future function, the second is to retain the dredged soil-water mixture, now and in the future. The following types of retaining structures can be discerned:

1. (a) Gravity structures
   (b) Vertical retaining structures
   (c) Composite structures
   (d) A dike or embankment
   (e) A reinforced embankment

1.a) Gravity structuresGravity structures rely on their weight to transfer the lateral loads to the foundation by means of a friction force. They are therefore usually executed as concrete caissons. A gravity structure needs a stable subsoil to rest on, however, in case of the Slufter it will simply sink through the dredged material layer. A gravity structure penetrating the dredged material and placed on top of the pleistocene sand layer would require very large dimensions, very large quantities of concrete and a very innovative construction method.

1.b) Vertical retaining structuresCommon slender vertical retaining structures are wooden, steel or concrete sheet piles, combi-walls and diaphragm walls ('D-walls'). Vertical retaining
structures have to transfer the lateral load through the soil to deeper layers by means of bending of the wall and friction between the soil and the wall and the soil and soil. Therefore they must be applied in soil with favorable geotechnical properties. The thickness of the layer of dredged material in the Slufter is approximately 30m. The dredged material is considered very weak with unfavorable geotechnical properties. This implies that vertical retaining structures have to penetrate the dredged material layer in order to reach the bearing stratum at NAP -28m. Slender vertical retaining structures require a penetration depth into the bearing stratum of 2/3 of the entire pile length when they are not anchored. This means that piles with a minimum length of 90m are required, which is exceptionally long, and then you would still only be at ground level.

An option could be to build a cofferdam, which is a structure consisting of two sheet pile walls with an anchorage between the two walls, which ensures its stability. This way the required penetration depth through the bearing stratum is 1/3 of the structure’s height. In Figure 5.1 a schematization of the principle of a cofferdam applied in the Slufter is presented. Suppose sheet piles of 60m are required to reach a level of NAP +17m (in accordance with what was found in the spatial analysis). Sheet piles with this length are not used on a regular basis and therefore not standardized. It is possible however to weld two or more sheet piles with standard lengths together. One could then estimate what the costs will be per running meter of sheet pile. If the weight per square meter of sheet pile is 500kg, then \( 2 \times 60 \times 500 = 60,000 \text{kg/m}^2 \) is needed, or 60 tons/m’. Suppose the costs for manufacturing, delivery and installation of sheet piles is 1500€/ton, then the costs for a cofferdam are 90,000€/m’.

![Figure 5.1: Schematic of a cofferdam in the Slufter basin](image)

1.c) Composite structures   Composite structures are a combination of gravity and vertical retaining structures. Therefore, they have the same advantages of both types, but also the same disadvantages. Because of the earlier mentioned disadvantages of both types of retaining structures, composite structures do not provide a viable option for the Slufter.

1.d) A dike or embankment   In a sand dike or embankment the lateral load is transferred by friction forces induced by interaction between the soil particles and interaction between the different layers (in case of a system with different soil characteristics per layer).

In comparison with a cofferdam, for a sand dike with an elevation height of 15m, a crest width of 10m and slopes of 1:10, 2400 m³ per running meter is required (as derived in Section 4.4).
If the costs per cubic meter of sand are 5\(\,\text{€}\), then the costs for a sand dike are 12,000\(\,\text{€/m}^3\). In a first estimation, a cofferdam is thus a factor 7.5 more expensive than a sand dike.

Building a dike or embankment is the only reasonable option for the Slufter, for economical as well as practical reasons. In order to realize this a body of soil has to be built on top of the existing dredged material layer. However, because of the unfavorable geotechnical properties, the elevation material should preferably be as light as possible. Blast furnace slag and pumice are much lighter than sand, have good geotechnical properties and are therefore occasionally used as elevation materials on weak subsoil. The advantage is that the dike can be realized with steeper slopes and in less time. However, still large quantities will be required and the costs per cubic meter are much higher than for sand. Moreover, these materials are not readily available so the transportation costs are also high.

Sand is in a first consideration the most favored elevation material, because a very large amount of sand from the Slufter’s ring-dike can be used. In that case, the costs for using blast furnace slag or pumice are in the order of a factor 5 higher than for sand. The realization of a sand embankment will probably require gentle slopes and a careful execution method in order to ensure stability in all construction phases.

1.e) A reinforced embankment A reinforced embankment has the same retaining characteristics as a conventional embankment, but is reinforced with geosynthetic materials. Thereby, it may be possible to realize an embankment with steeper slopes, which saves the use of construction material. In this chapter, an embankment is analyzed that is reinforced with Geocells (a product of the company Tensar). A Geocell provides a firm base of the embankment on the dredged material.

Thus, the selected type of retaining structure is an embankment on top of the dredged material. A reinforced embankment is included in the analysis to serve as an alternative to the conventional embankment.

5.3.2 Execution method

Building on unconsolidated dredged material is not common practice. In most projects dealing with soil with unfavorable properties the weak layer is removed entirely and replaced by construction sand. In case of the Slufter however, this is not desirable because of the large quantities of dredged material. The construction of an embankment on top of the dredged material therefore requires a subtle approach. The execution method consists of the following phases:

Phase 1 Thin layers of sand are sprayed on top of the dredged material by a chute mounted on top of a pontoon floating on the Slufter’s sheet of water. This technique is derived from land reclamation projects done in the Rotterdam harbor in the past, and more recently at the ‘Ijburg’ project near Amsterdam [8], where a similar first layer had to be realized on soft subsoil. The thickness of the sprayed layer of sand has to be sufficient to provide enough bearing capacity for the construction equipment.

Phase 2 In case a layer with sufficient thickness can be sprayed in Phase 1, the sheet
of water is to be removed and the area will be accessible to the equipment. In this phase a technique is required to accelerate the consolidation process. In 1994, Gemeentewerken Rotterdam has done a literature study on the applicability and costs of various available consolidation techniques [17]. This study was initiated to investigate the possibility of increasing the storage capacity of the Slufter. Among the techniques that were investigated were vertical and horizontal drains, electro-osmosis, a sandwich structure and the evaporation of dredged material. Based on case studies the various techniques were evaluated. In these case studies the vertical and horizontal drain systems were combined with vacuum consolidation.

Vertical drainage in combination with vacuum consolidation proved to be the most cost effective alternative when compared to the other techniques. By applying a vacuum to the system of vertical drains no (or less) extra surcharge is needed and consolidation can take place faster. However, compared to a system without a vacuum, where consolidation is induced by a surcharge of soil, it is more expensive.

The vertical drains consist of a quasi-rigid plastic sheet protected by a geotextile that acts as a filter and separator. The drain is driven into the soil layer through a hollow pile by specialized equipment, and retracted after anchoring the drain, see Figure 5.2.

![Figure 5.2: The installation procedure of a system of vertical drains, provided by Cofra](6)

In this analysis the vertical drainage technique is applied without vacuum, because the area will have to be elevated to create an embankment and because there is no time pressure.

---

TRANSLATION OF THE INSTALLATION PROCEDURE:

1. Uitvoeren drain locations
2. Plaatsen anchorplate of anchorplaat
3. Bring the pile to depth
4. Retract pile, drain is held by anchorplate
5. Cut drain when the pile is above the ground
6. Continue at 2

---

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Phase 3  After the vertical drains are installed a surcharge of sand is deposited into the area where the embankment will be erected, in a staged construction method. This also enhances consolidation and induces settlement.

After the third phase the embankment and the created land behind it is ready to be developed. The execution of this further development will not be considered in this research.

5.3.3 The geotechnical feasibility of an embankment

The exploratory calculations that are done in order to prove the geotechnical feasibility of the retaining structure can be found in Appendix D. In this section the most important discoveries of the analysis will be summed up and explained, so as to allow for a more directed use of the report. These discoveries are the outcome of exploratory calculations done by hand and by means of the GeoDelft software MStab and MSettle. The input parameters used in the MStab simulations are presented in Table D.1.

A distinction is made between the construction phase and the users phase. The description of the construction phase follows the three phases as explained in the previous section.

Construction phase

The three different construction phases are the spraying of sand, the installation of vertical drains and finally the further elevation of the dry area in order to realize the embankment. The final embankment has an elevation of NAP +17m (15m above the level of dredged material), a crest width of 10m and slopes of 1:10.

Phase 1  (see also Appendix D.1.1)

The elevation by spraying has to be done layer by layer to ensure that the obtained slope does not collapse. A careful approach is essential, because initially the dredged material is undrained, i.e. the pores are filled with water and the body of soil is fully saturated. Immediately after placement of the layer of sand the situation is therefore undrained and most critical. The load is carried by the pore water pressure, because the water can not flow out sideways or downwards. At that moment the angle of internal friction has no effect \( \phi' = 0 \) and stability has to be guaranteed by the undrained shear strength \( c_u \).

According to the hand calculations the bearing capacity of the dredged material, with an undrained shear strength of 1kPa, is sufficient to spray thin layers of sand with a thickness of 10 to 15cm. With a spraying pontoon this accuracy can be achieved.

In the desired situation it is possible to spray a layer of sand that is thick enough to bear the vertical drainage installation equipment once the water sheet has been removed. A hand calculation of the bearing capacity shows that a layer of 3.3m is required to allow for equipment with a distributed load of 92kN/m². This is the load of a CAT345, which is the type of vehicle that is used to install vertical drains.

An attempt was made to simulate this scenario in MStab. However, the simulation showed that the sprayed sand body fails on slope stability once the layer becomes too thick, even
with a gentle slope of 1:10. The reason for these instabilities is that hardly any consolidation takes place as a result of the sand load, so the strength of the dredged material does not increase. This is even the case in a staged construction with 100 days of waiting time in between consecutive layers.

More simulations have been done with different configurations of the sprayed layers. The analysis shows that it is possible to increase the slope stability by creating an interspace between two consecutive layers. However, because the effective weight of the soil under water is lower than above water, the soil structure fails once the water level is lowered. A stable situation may be obtained if a soil structure is sprayed in a staged construction, with interspaces of 60m wide, and with elevations of 0.5m every 100 days up to a level of NAP +4m. Once the water level is lowered to NAP +3.5m the stability factor is 1.25. The structure, however, would then have a width at the base of 670m. This is about half the distance of one side of the Slufter’s ring-dike to the other side and therefore unrealistic.

From this analysis must be concluded that a system of vertical drains has to be installed from a pontoon, in order to improve the geotechnical properties of the soil. In that case a first sand layer of approximately 1m has to be sprayed to make sure that the vertical drains stay in place at the top. A soil structure with elevations of 0.5m every 100 days, with an interspace of 20m, gives a stability factor of 1.35. This will form the base of the embankment.

**Phase 2** (see also Appendix D.1.2)

When the system of vertical drains is installed a period of waiting is allowed to initiate the consolidation process. In this period the state of the layer of dredged material changes from undrained to drained.

The only direction in which transport of water can take place is upwards through the aquiferous sand layer. Because of the low permeability of the dredged material this process will take time. The effect is that gradually the pore water pressures will decrease and thereby the effective stresses will increase. This ultimately leads to an increase of the strength of the soil and the stability of the overlying sand structure. As a result of the consolidation process the situation becomes drained and values for $\phi'$ and $c'$ can be chosen.

In MSettle, vertical drains can be included in the simulation. With a drain installation depth at NAP -27m and a drain center-to-center distance of 2m, the degree of consolidation after 100 days is 73%. The settled geometry from MSettle can be imported to MStab to check the slope stability. In the drained situation, with a degree of consolidation of 73%, the stability factor increases considerably. It is now sufficiently safe to lower the water level and to allow construction equipment to access the area.

In this case, the chosen equipment is a large excavator. In the extreme load condition, where the weight of the vehicle and the filled bucket is carried by one track, the soil structure fails on bearing capacity. In the simulation, under the load of 182kN/m², it will sink through the sand layer. The proposed solution would be to use lighter vehicles for the elevation of the first few layers, or to wait longer so as to have a higher degree of consolidation.

The influence of the vertical drain installation depth and center-to-center distance on the degree of consolidation has been analyzed using MSettle. The results are presented in Figures D.8 and D.9. A smaller drain depth or center-to-center distance means a slower progress.
of the degree of consolidation. The choice of depth and distance therefore depends on the available time, the cost reduction and the structural safety.

Phase 3  (see also Appendix \[D.1.3\])

When the structure is sufficiently safe for the equipment, the area can be elevated further. In MSettle and MStab this is simulated in a staged construction, where every 100 days a surcharge of sand of approximately 5m is placed, at a slope of 1:10. After three elevations and a superelevation, which compensates for the settlement, the embankment is finished. If the execution is done according to these steps, the SF for the slope in the final situation is 1.32.

The settlement after 400 days is approximately 3.5m. The residual settlement (after 10,000 days) is another 30cm. However, it should be noted that there is an uncertainty on the chosen parameters in this analysis. It is possible that more settlement will take place in reality. In case an embankment is to be realized, further research of the geotechnical properties, such as the consolidation coefficient and the compression parameters, is recommended.

If the structure is safe at the moment of completion than it will also be safe in the users phase, because the stability factor will increase over time as settlement continues.

During the simulations a vertical drain installation depth at NAP -27m and a center-to-center distance of 2m was chosen as a starting basis. In addition to the demonstration of the geotechnical feasibility of the embankment, the influence of drain depth and distance to the stability of the soil structure in the different phases was analyzed. This was done for installations depths at NAP +20m and NAP +10m, and drain distances of 2.5 and 3m. The analysis proves that a stable structure can also be achieved with either a drain depth at NAP +20m or a drain distance of 2.5m, within the same time frame. It is therefore advisable to optimize drain depth, distance, time and costs if an embankment is realized.

Users phase

(see also Appendix \[D.2\])

In the users phase three different future scenarios of the use of land in the Slufter are discerned. The first scenario simulates the load condition of an algae breeding pond and represents a medium-weight function. The second scenario represents the load case of an empty depot. In the third scenario, temporary storage of soil (TOP), the load of a pile of sand is simulated. Scenarios 2 and 3 are considered heavy-weight functions.

Because the functions will be accommodated on a horizontal area realized between the existing ring-dike and the embankment, slope stability is not the critical failure mechanism, but bearing capacity. In addition, the settlement as a result of the load must be checked.

In the calculations the elevation of the obtained land is NAP +7m. The bearing capacity under the above mentioned load conditions has first been determined by hand calculations. However, these calculations are very conservative, mainly because the contribution of the sand layer to the shear resistance is not included. MStab applies Bishop’s method \[32\] to determine whether there is sufficient bearing capacity.
CHAPTER 5. GEOTECHNICAL ANALYSIS

Simulation of Scenario 1 in MStab and MSettle shows that it is absolutely no problem to accommodate an algae breeding farm on the created land. The settlements are practically zero. From this can be concluded that light-weight functions, such as a field of solar panels, greenhouses or a fish breeding farm, can safely be deployed at the Slufter.

The soil structure is also able to bear the weight of a row of ten empty containers stacked up eight high. The residual settlement as a result of this load is only 3.5cm.

The pile of sand simulated in Scenario 3 poses a large load on the subsoil, however stable. Problems may arise if the pile has to be larger. In addition, the residual settlement is 80cm, which is unacceptable. Measures have to be taken to distribute the load over a larger area.

To conclude, if the sand layer on top of the dredged material is sufficiently thick, then any kind of function with a shallow or no foundation can take place at the Slufter.

5.3.4 Execution costs

In this section the execution costs of the embankment are roughly estimated. The soil structure consists of three main cost components, which can also be recognized in the construction phases:

1. The spraying of the first sand layers from a pontoon
2. The installation of a system of vertical drains
3. The further elevation of the embankment by staged surcharges of sand

Indicative costs per unit, required quantities of materials and the execution costs are presented in Table 5.2. These execution costs are based on the preliminary design as elaborated in the previous sections. The embankment has slopes of 1:10, has a width at the base of 390m, a crest width of 10m and is elevated up to NAP +17m. The system of vertical drains is installed at a depth of NAP -27m and a center-to-center distance of 2m.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Costs/unit</th>
<th>Quantity/m³</th>
<th>Costs/m³</th>
<th>Average costs/m³</th>
<th>Av. costs (mln) if L=1000m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spraying</td>
<td>10-16€/m³</td>
<td>360m³</td>
<td>€3600-€5760</td>
<td>€4680</td>
<td>€4.7</td>
</tr>
<tr>
<td>Vertical drains</td>
<td>2-2.5€/m</td>
<td>2632.5m</td>
<td>€5265-€6581</td>
<td>€5923</td>
<td>€5.9</td>
</tr>
<tr>
<td>Completion</td>
<td>5-8€/m³</td>
<td>2685m³</td>
<td>€13,425-€21,480</td>
<td>€17,453</td>
<td>€17.5</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>€22,290-€40,430</td>
<td>€28,056</td>
<td>€28.1</td>
</tr>
</tbody>
</table>

Thus, an embankment with these dimensions and a length of 1km will cost approximately 28 million Euro. If there are 100 days between every elevation phase, and the contractor can install vertical drains at a speed of 8000m per day (10 hours), then the total duration of such a project is approximately 900 days. So in two and a half years time an embankment can be realized in the Slufter. However, the investment costs are very high. In fact, it is unlikely that the Port Authority will invest in such a project, unless the value of land becomes unexpectedly large.

It is possible to reduce the costs by optimizing the vertical drain installation depth and/or the drain distance. Because of a slower consolidation process, the construction is spread
over a longer period of time. In the geotechnical analysis (Appendix D) was shown that a stability factor of 1.2 can be obtained if either the drain depth is reduced to NAP -20m or the drain distance increased to 2.5m. The smaller drain depth will result in a cost reduction of 5-6% and a larger drain distance in a cost reduction of 4-5%. This comes down to 1 to 2 million Euro on the entire project.

The total costs of the embankment per running meter as a function of the installation depth are given in Figure 5.3 for the three different drain distances. The costs increase more rapidly with the drain depth if the center-to-center distance is small. Optimizing drain depth and distance may therefore result in a reasonable decrease of the total costs. However, the share of the excavation, transportation and deposition of soil is so large that such a project requires a disproportionate investment. The costs can be decreased significantly if the embankment is smaller or has steeper slopes. Therefore, an alternative is presented in the following section; an embankment built with Tensar Geocells.

![Figure 5.3: The increase of the total costs of the embankment per running meter as a function of the drain installation depth, for three different drain distances](image)

### 5.3.5 An embankment built with Tensar Geocells

The company Tensar is specialized in the development and production of high quality geosynthetics for reinforced soil and foundation reinforcement. As a supplement to this research Tensar has provided a proposing document for an embankment in the Slufter with the use of their geosynthetic products; Geogrid and Geocell [13]. Figure 5.4 shows the principle of an embankment with the use of geocells. A geocell construction is a 1m high vertical honeycomb...
structure filled with coarse granular material. A filled geocell is a stiff, strong foundation for the soil structure and provides a workplace for the heavy equipment. In addition, it functions as a drainage layer for consolidation water.

![Diagram of a geocell structure](image)

**Figure 5.4:** The principle of Tensar geocells [13]

Any potential slip circle goes through the geocell and is thereby forced into the deeper layers. The critical failure condition is now posed by the plastic deformation of the weak dredged material layer, i.e. the squeezing out of the dredged material sideways. The rough bottom of the geocell makes use of the shear capacity of the dredged material at the interface. However, below this interface, the resistance against deformation still depends on the undrained shear strength of the dredged material.

According to Tensar it is possible to build an embankment in the Slufter, with a height of 15m and slopes of 1:1.5. The proposed construction method is according to the following five steps:

1. Lower the water level to ground level. Apply a sand-tight geotextile and a base of geogrid. The area now becomes accessible to persons.
2. Assembly of the geogrid
3. Filling of the geocell structure with coarse granular material
4. Install vertical drains through the geocell structure
5. Finish the embankment in elevations of 2m at a time

In the NEN6744 [22] a formula is given to determine the bearing capacity of a cohesive layer under the failure condition of squeezing, when loaded by a shallow strip foundation. The effective stress is given by:

\[
\sigma_{sq; d}^\prime = \left(\pi + 2 \right) + \frac{b'}{h_{sq}} \times c_{ud} + \sigma_{v; z}^\prime
\]

(5.3)

where \(b'\) is the effective width of the foundation [m], \(h_{sq}\) is the thickness of the cohesive layer [m], \(c_{ud}\) is the undrained shear strength of the cohesive layer [kPa], and \(\sigma_{v; z}^\prime\) is the vertical effective stress at depth \(z\) at the top of the cohesive layer [kPa] as a result of the soil next to the strip foundation. The value for the bearing capacity is obtained by multiplying the effective stress with the effective surface area of the foundation.
For an exploratory calculation of the bearing capacity, this formula can be applied to the embankment. However, in this case $\sigma'_{vzd}$ is taken to be zero, because there is no soil layer next to the embankment that contributes to the strength.

Two load situations in the construction phase are considered; in the first situation the load is only the weight of the geocell structure filled with granular material, and in the second situation the load is the weight of the entire embankment.

The weight of the geocell structure  The value of the undrained shear strength in this situation is 1kPa. The embankment has a width at the base of 47m and the thickness of the layer of dredged material is 30m. The effective stress then becomes:

$$\sigma'_{sqd} = \left( \pi + 2 \right) + \frac{47}{30} \times 1 = 16 \text{kPa}$$

And the strength per running meter thus becomes:

$$R = 16 \times 47 = 752 \text{kN/m'}$$

The horizontal force that induces squeezing is equal to the weight of the geocell structure. The volumetric weight of the granular material in the geocell is 18.5kN/m$^3$, and the height of the geocell is 1m. The pressure of the geocell structure on the dredged material is therefore 18.5kN/m$^2$. The vertical load is transferred to a horizontal squeezing force, distributed over the height of the layer of dredged material, and thus becomes:

$$S = 18.5 \times 30 = 555 \text{kN/m'}$$

This results in a stability factor of 1.35, which gives the impression that the bearing capacity is sufficient with respect to squeezing. This calculation method, however, looks at the structure as a whole. If one looks at the force equilibrium at the sides of the embankment, as shown in Figure 5.5, then the resistance against squeezing is provided by the horizontal components of the undrained shear stress underneath the slope, over the same length at the bottom of the dredged material layer, and along the assumed circular slip plane next to the embankment.

If the undrained shear strength at the top is 1kPa and at the bottom 3kPa, and the slopes are 1:1.5, then the strength can be approximated by:

$$R = 1.5 \times (1 + 3) + 0.5\pi \times 30 \times 1 = 53 \text{kN/m'}$$

With a load of 555kN/m', the strength is not sufficient to prevent squeezing.

The resisting shear stress can be increased by a gentler slope. From the force equilibrium, the required length of the slope can be determined:

$$555 = a \times (1 + 3) + 0.5\pi \times 30 \times 1$$

$$\rightarrow a = 127m$$

A slope of 1:127 is unrealistic. In addition, it should be noted that the weight of the material beneath the slope also contributes to squeezing. This is not taken into account in this calculation.

The weight of the entire embankment  Suppose the geocell structure has in some way successfully been placed, the squeezing has not resulted in failure of the base structure, and a
system of vertical drains has been installed. After consolidation, the geotechnical properties of the soil have improved resulting in a new value of the undrained shear strength. The embankment can then be elevated up to a height of 15m with slopes of 1:1.5.

Again, first the bearing capacity of the structure as a whole is determined, following Equation 5.3. The new value of the undrained shear strength is determined with Skempton’s empirical relation [16]:

\[ c_u = 0.2\sigma' \]

With a volumetric weight of the elevation sand of 17kN/m², the effective stress at the top of the dredged material is:

\[ \sigma' = 14 \times 17 + 1 \times 18.5 = 256.5kPa \]

If the water level is at the top of the dredged material layer, then the effective shear stress at the bottom is:

\[ \sigma' = 14 \times 17 + 1 \times 18.5 + 30 \times 13 - 30 \times 10 = 346.5kPa \]

The undrained shear strength at the top thus becomes 51.3kPa and at the bottom 69.3kPa. On average, the undrained shear strength over the depth is 60.3kPa. The effective stress that contributes to the bearing capacity then becomes:

\[ \sigma_{sqd}' = \left( \pi + 2 + \frac{47}{90} \right) \times 60.3 = 486kPa \]

and the bearing capacity is:

\[ R = 486 \times 47 = 22829kN/m' \]

The load is induced by the weight of the embankment, distributed over the height of the dredged material layer:

---

**Figure 5.5:** Schematization of the squeezing failure mechanism under influence of the load of the geocell
5.3. ALTERNATIVE 1: BUILDING A RETAINING STRUCTURE

\[ S = (14 \times 17 + 18.5) \times 30 = 7695 kN/m' \]

This results in a stability factor of 3, which again gives the impression that there is no risk of squeezing for the structure as a whole.

If one looks at the force equilibrium at the sides of the embankment, the strength is determined by the undrained shear stress along the top and bottom of the dredged material underneath the slope, and the assumed circular slip plane next to the embankment (see Figure 5.6). The undrained shear strength is 51.3 kPa at the top and 69.3 at the bottom of the dredged material. Along the sides, the undrained shear stress still has a value of 1 kPa, because no consolidation under influence of vertical drains has taken place here.

Figure 5.6: Schematization of the squeezing failure mechanism under influence of the load of the entire embankment

With a length at the base underneath the slopes of 22.5 m, the strength can be approximated by:

\[ R = 22.5 \times (51.3 + 69.3) + 0.5\pi \times 30 \times 1 = 2760.6 kN/m' \]

The load is equal to the previously calculated load and 7695 kN/m’. Again, the strength is insufficient to prevent squeezing. From the force equilibrium can be determined what horizontal length underneath the slope is required in order to have a stable structure:

\[ 7695 = a(51.3 + 69.3) + 0.5\pi \times 30 \times 1 \]

\[ \rightarrow a = 63.4 m \]

Over a height of 15 m, this comes down to a slope of 1:4.5.

According to the NEN6744 calculation method there is no risk of squeezing when looking at the structure as a whole. This method, however, applies to shallow strip foundations. Its applicability to this situation is therefore questionable. When looking at the force equilibrium along the edges of the embankment, squeezing will definitely take place. The first phase, with only the load of the geocell structure, is critical. In case an embankment with geocells is
to be realized, the geotechnical feasibility of this critical phase has to be further investigated. Moreover, a gentler slope than 1:1.5 is required.

**Execution costs** The execution costs of an embankment with slopes of 1:4.5 and an elevation height of 15m are determined and presented in Table 5.3. For this embankment, with a crest width of 5m and a width at the base of 140m, 952 m³ of sand is required. Tensar can deliver the geocell structure for 24 €/m², which comes down to 3360 €/m³. The system of vertical drains has an installation depth of NAP -27m and a center-to-center distance of 2m. However, the drains will now be installed on land. The indicative price for delivery and installation is 0.80 € per installed meter.

**Table 5.3:** Execution costs of an embankment with 1:4.5 slopes and a base of geocells

<table>
<thead>
<tr>
<th>Operation</th>
<th>Costs/unit</th>
<th>Quantity/m³</th>
<th>Costs/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delivery Geosynthetics</td>
<td>24 €/m²</td>
<td>140m</td>
<td>3,360</td>
</tr>
<tr>
<td>Assembly geocell and filling</td>
<td>15 €/m²</td>
<td>140m</td>
<td>2,100</td>
</tr>
<tr>
<td>Coarse granular material</td>
<td>18.5 €/m²</td>
<td>140m</td>
<td>2,590</td>
</tr>
<tr>
<td>Vertical drains</td>
<td>0.80 €/m</td>
<td>945m</td>
<td>756</td>
</tr>
<tr>
<td>Sand elevation</td>
<td>6.50 €/m³</td>
<td>952m³</td>
<td>6,188</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>14,994</strong></td>
</tr>
</tbody>
</table>

The costs of the large embankment, discussed in the previous section, are 28,056 €/m³. An embankment with geocells, with the here proposed construction method, is thus a factor 2 cheaper. However, a careful approach is needed in the construction phase when the geocell is placed and filled with granular material. Because of the low value of the undrained shear strength squeezing is expected and may result in unacceptable deformations. In addition, more problems may arise when the geocell is loaded by the vertical drains installation equipment.

From this preliminary analysis can be concluded that it is difficult to realize this embankment, because of the critical phase of placement of the geocell structure. A different approach, which is based on executing the critical phases from a pontoon, seems to be more viable:

1. Spray a base layer of sand (0.5m should be sufficient) on top of the dredged material from a pontoon. This layer provides a counter pressure against squeezing, next to the geocell structure
2. Assemble the geocell on land
3. Place the geocell from a pontoon and fill it with sand by spraying (coarse granular material can also be used, but this will have to be placed from a barge)
4. Install vertical drains through the geocell structure, from a pontoon
5. Lower the water level, allow for consolidation and finish the embankment in elevations of 2m at a time

The choice of using sand in the geocell instead of coarse granular material will certainly reduce the stability of the embankment, but the geocell structure will still provide a good reinforcement. The costs will increase because many construction phases require operations from a pontoon. A first estimation of the costs is given in Table 5.4.

The reinforced embankment is in this case still a lot cheaper than the conventional embankment. This is solely due to the fact that the slopes are steeper, and therefore less construction material is required. However, if this embankment is to be realized, further research has to
be done on the geotechnical feasibility and the execution method.

Table 5.4: Execution costs of an embankment with 1:4.5 slopes and a base of geocells; alternative execution method

<table>
<thead>
<tr>
<th>Operation</th>
<th>Costs/unit</th>
<th>Quantity/m³</th>
<th>Costs/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spraying of 0.5m layer of sand</td>
<td>13€/m³</td>
<td>100m³</td>
<td>€1300</td>
</tr>
<tr>
<td>Delivery Geosynthetics</td>
<td>24€/m²</td>
<td>140m</td>
<td>€3360</td>
</tr>
<tr>
<td>Assembly geocell, placement and filling by spraying</td>
<td>30€/m²</td>
<td>140m</td>
<td>€4200</td>
</tr>
<tr>
<td>Vertical drains</td>
<td>2.25€/m</td>
<td>945m</td>
<td>€2126</td>
</tr>
<tr>
<td>Sand elevation</td>
<td>6.50€/m³</td>
<td>952m³</td>
<td>€6188</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>€17,174</strong></td>
</tr>
</tbody>
</table>

5.4 Alternative 2: Clay ripening

An alternative for the realization of land might be presented by clay ripening in combination with dry storage of dredged material. As a result of the process of ripening clay, over time a hard crust develops, which has some bearing capacity. This section of the geotechnical analysis focusses on the possibilities of clay ripening to alter the Slufter’s layout in the future.

In 1984, a research was done on the physical processes of the dredged material to be stored in the Slufter, as a supplementary document to the Environmental Impact Assessment (‘Milieu Effecten Rapportage’ MER [15]). A part of the research was dedicated to the process of ripening of the top layer and its bearing capacity. The composition and thickness of the ripened clay layer increases over time and thereby the bearing capacity. This is also shown in Figure 5.7.

In the first five years the top layer dries up and cracks develop, in the figure schematized as independent columns. As a result of erosion, frost, precipitation and/or mechanical influences the columns crumble and the soil becomes more homogeneous (in the period of 5 to 10 years). After more than ten years, the columns have transformed into one layer of dried clay. The parameter \( k \) in the figure represents the influence of roots growing in the layer. This can increase the bearing capacity considerably. In case \( k=0 \) there is no influence from roots and if \( k=1 \) there is maximum influence from roots. The actual development of the ripening clay will probably lie somewhere between lines b and c in the figure.

It is expected that already after the first summer the area is accessible by the amphirol, an amphibious vehicle propelled by screw-shaped rotating cylinders (see Figure 5.9). This vehicle creates shallow trenches that can help to accelerate the ripening process.

After five years the expected thickness of the ripened top layer is 0.6 to 1m and the bearing capacity is approximately 10kN/m². In the research the possibility of elevations, for instance for the realization of a road, was briefly investigated. The analysis, depicted in Figure 5.8 has shown that a sand elevation of 0.5m results in a stability factor of 1.

This result has been investigated and verified in MStab, with the same parameters as were used in the simulations of the embankment and presented in Table D.1 in the Appendix. The ripened clay has a cohesion \( c' = 2.5 \)kPa and an angle of internal friction \( \phi' = 25^\circ \), the
dredged material has an undrained shear strength distribution of 1kPa at the top and 3kPa at the bottom. Once the area is elevated by a second layer of sand the structure fails, even if the ripened top layer has a thickness of 1m. Various simulations have been done, with gentle slopes, interspaces between consecutive surcharges of sand and sandwich structures. Sandwich structures have sand layers alternated by clay layers, to decrease the weight of the elevation. However, all of these structures have a stability factor lower than the required 1.2.

A better option for the clay ripening alternative might be to consider only those areas in the Slufter near the Ring-dike, where the dredged material layer is thinner. On average, this area stretches approximately 170m from the crest of the ring-dike to the ‘beach’; the transition from a 1:40 slope to a 1:4 slope. The parts with the gentle slopes have a length of 100m. Along the entire circumference the surface area increases from 116ha at the part with gentle slopes to 164ha at the crest of the ring-dike (see also Section 4.5).

The water level in the Slufter is momentarily at NAP+6m. The dredged material protrudes approximately 0.5m above the water level along the edges of the ring-dike and gently slopes towards the central parts of the basin. This situation has been the input for the geometry in MSettle and MStab, as shown in Figure 5.10.

Again, the resulting 1m thick layer of ripened clay is loaded by an elevation of 0.5m sand with slopes of 1:10. Unfortunately, the situation does not improve much compared to the an elevation in the middle of the Slufter (see Figure 5.11). The layer of undrained dredged material here is 5 to 6m thick, which apparently is still too much.

The layer of dredged material thus has to be thinner in order to be able to use this space. However, there are other issues that might influence a possible future usage and layout of these parts:
5.4. ALTERNATIVE 2: CLAY RIPENING

1. Once the water level is lowered the dredged material at the higher edges might flow towards the center of the basin, which automatically reduces the thickness at the beach. It is unclear how the dredged material will actually take shape.
2. The dredged material near the edges might have better geotechnical properties because of sand fractions that settle near the discharge points.

From the previous must be concluded that fast development of the area by means of ripening clay within the first 5 years is inconceivable. Therefore, a more long term solution is proposed. In this solution, the water level in the basin is lowered permanently and a process of ripening initiated. After a few years, thin layers of dredged material can be sprayed on top of the first crusted layer and the area slowly elevated in that manner. A similar approach was followed in the ‘Broekpolder’ area near Rotterdam in the 70s. The thickness of, and time interval between each consecutive layer depends on the speed of the ripening process. This determines the time it takes to significantly increase the elevation height.

When eventually enough layers have been sprayed and a thick layer of ripened, dry dredged
CHAPTER 5. GEOTECHNICAL ANALYSIS

Figure 5.10: Overview of the geometry used in MSettle for the purpose of the clay ripening alternative

Figure 5.11: The critical slip circle for an elevation of 0.5m sand on top of a 1m thick layer of ripened clay. The green rectangles along the slip circle represent the magnitude of the shear stress per slice (see also Appendix F.2). The ripened dredged material has a larger contribution to the shear resistance per slice than the unconsolidated dredged material
material has formed, the area becomes accessible to heavy equipment, such as vertical
drainage installation vehicles. A system of vertical drains can accelerate the consolidation
process of the weak dredged material below the ripened clay layer, and thereby increase the
strength of the soil.

After sufficient consolidation, a second, inner ring-dike on top of the ripened clay is optional.
This inner ring-dike can ensure enough storage capacity for the remaining usage of the Slufter
as a disposal facility.

To determine what the required thickness of the ripened clay layer has to be in order to
provide enough bearing capacity for the equipment, this scenario has been simulated in
MStab. The simulation shows that the thickness has to be 3m in the shallow parts near the
ring-dike and 4.5m in the deep parts of the basin. For the shallow part this is presented in
Figure 5.12.

Thus, the layer has to be quite thick to make the area accessible. The first layer of 1m takes
5 years to develop. The elevation of another two meters over the entire basin comes down to
approximately 5.6Mm³. Suppose the annual supply of dredged material is 400,000m³, then
this height would be reached after 14 years. However, it is also possible to transport dredged
material from one side of the basin to the other or to only deposit new supplies in one place.
Thereby elevation of specific areas can be realized faster.

The realization of an embankment after consolidation has also been simulated in MSottle
and MStab. Figure 5.13 shows an overview of the simulation, with the contour of the Slufter
basin, the ripened layer and an embankment. The embankment has a height of 4m and
slopes of 1:3. It can be either be built with sand, with clay or a combination of both. Figure
5.14 shows that a sand embankment is stable, with an SF of 3.7.

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Figure 5.12: The ripened clay layer, loaded by vertical drain installation equipment in the shallow
part of the basin, and the distribution of the shear stresses
CHAPTER 5. GEOTECHNICAL ANALYSIS

Figure 5.13: Overview of the ripened surface of the Slufter with an embankment

Figure 5.14: The critical slip circle of a sand embankment built on top of ripened clay, and the distribution of the shear stresses

To conclude, clay ripening provides a good alternative to the retaining structure, because it does not require a large investment in the short term. However, it is a long term solution because of the slow ripening process and the large required layer thickness. Once the sub-
lying weak dredged material layer has consolidated, there are good opportunities for the development of functions with no or shallow foundations.

5.5 Evaluation

Two main alternatives have been analyzed in this chapter:

1. A retaining structure
2. Clay ripening

The most commonly used types of retaining structures are:

1. (a) Gravity structures
   (b) Vertical retaining structures
   (c) Composite structures
   (d) A dike or embankment
   (e) A reinforced embankment

Of these types of retaining structures 1(d) and 1(e) were further analyzed. The other types were discarded, based on their practical and economical constraints with respect to the Slufter's situation.

The geotechnical feasibility of the conventional embankment, with an elevation height of 15m and slopes of 1:10, was proven. However, the execution costs, estimated to be €28,000 per running meter embankment, are very high. This is mainly due to the delicate execution method and the large amount of construction material required. The execution method consists of the following phases:

1. Spraying of sand from a pontoon
2. Installation of a system of vertical drains from a pontoon
3. Lowering of the water level and finalization of the embankment by phased surcharges of sand

A reinforced embankment, built with Tensar geocells, is a factor 1.5 to 2 cheaper than a conventional embankment, depending on the execution method. Because of the unfavorable geotechnical properties of the dredged material it is likely that plastic deformation by squeezing will take place. The placement and filling of the geocell structure is the most critical construction phase. Therefore, the embankment has to be realized with the execution method that is expected to be more feasible. The method proposed in this chapter is based on executing multiple phases from a pontoon:

1. Spraying of sand from a pontoon
2. Assembly of geocell on land
3. Placement of geocell and filling with sand by spraying
4. Installation of a system of vertical drains from a pontoon
5. Lowering of the water level and finalization of the embankment by phased surcharges of sand

The geotechnical feasibility of this embankment and the execution method has to be further investigated.
During the analysis of alternative 2, clay ripening, it was shown that the bearing capacity of a top layer of ripened clay is insufficient for short term development of the area. Therefore, clay ripening is a long term solution for land reclamation. However, in case of the Slufter, time is not a major issue. Once the layer of ripened clay is thick enough, it becomes accessible to equipment and can be further developed. For the shallow parts of the basin the thickness of the ripened clay layer has to be approximately 3m, and for the deep parts 4.5m, to provide sufficient bearing capacity. Clay ripening presents a more cost effective option, because the required investment is practically zero compared to the embankment alternative.
Chapter 6

Preparing a long term management plan

In this chapter, the results from the functional, spatial and geotechnical analysis are combined in order to provide a practical basis of a long term management plan for the future of the Slufter.

The aim is to gather all the relevant information from the analyses so that it gives a quick overview of the possibilities. Thereby, a tool is developed which is used to combine functional elements with efficient use of space and land reclamation methods. To actually develop a long term management plan decisions have to be made on the management strategy. The first step is to select a preferred function (or multiple functions). The second step is to select a method to reclaim land from the basin.

Subsequently, based on the experience and knowledge gained during the elaboration of this research, a strategy is recommended for the long term management plan. This is concluded by a visualization of the proposed solution.

Step 1: Select function(s)

The best opportunities in the short term can be seen for the following functions:

- Solar energy
- Algae breeding
- Windmills
- Greenhouse cultivation
- Fish breeding
- Location for Bevi-companies
- Nature/ birds reserve

The functions that are more viable in the long term are:

- Empty depot
- Biomass processing plant
- Storage of oil products or chemicals
CHAPTER 6. PREPARING A LONG TERM MANAGEMENT PLAN

• Nature/ birds reserve

Long term refers to the situation where the Slufter seizes to exist as a disposal facility and is entirely dismantled. Most of the long term functions are industrial or harbor related functions. The main reason why these functions are not realistic in the short term is that other areas on Maasvlakte I and Maasvlakte II, which are more suitable, have not been issued yet (i.e. do not have a predefined function yet). These areas provide sufficient space for at least the coming 20 years.

A nature or birds reserve provides both short and long term opportunities for the use of space.

The pros and cons of the eleven functions mentioned here are once more summarized in Table 6.1. This helps to make a choice on which function(s) is/are to be investigated further.

If it is decided that additional functions will be given to the Slufter area, the following criteria can be used to judge their viability (see Section 3.2 for an explanation per criterion):

- Efficient use of space
- Vicinity support
- Social support
- Port Authority support
- Permits
- Flexibility
- Costs/ benefits
- Harbor committed
- Multi-functionality
- Quality, Safety, Health and Environment
- Sustainability
- Technical feasibility

Step 2: Select a method to reclaim land

The flow charts presented on the last three pages of this chapter (Figures 6.3, 6.4 and 6.5) give an overview of the possibilities to create land in the Slufter basin. This flow chart can be used in the decision making process, when developing a long term management plan.

Explanation of the flow charts
At the top of the chart there are two paths to follow; either something will be changed in the short term or something will be changed in the long term. The long term path on the left presents the decision for realization of a flat area of land once the Slufter is dismantled (i.e. no longer functions as a disposal facility). If only a flat horizontal area is to remain after dismantlement, then the ring-dike has to be lowered.

The short term path on the right presents the choice for realization of a flat area of land in the short term. If this is wished for then land can either be reclaimed with a retaining structure or by means of clay ripening. If this is not preferred then there is the option of

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1The enlisted functions in this chapter are the result of the research process of this project and provide a guideline for the possibilities. However, this does not imply that any other function, that has not been included in the research, can not be taken into consideration.
floating structures. In case floating structures do not present a suitable solution to create a usable area, then the situation remains the same. Otherwise, the floating structure procedure can be started (the floating structures procedure is beyond the scope of this project).

If a standard embankment provides the desired type of retaining structure, then the standard embankment procedure can be started. If not, then the other option is to start the reinforced embankment procedure. Conversely, if the decision is made not to create land by means of a retaining structure, then land should be reclaimed by clay ripening. This is done by starting the short term clay ripening procedure.

The procedures (Figures 6.4 and 6.5) refer to the execution and implementation method of the alternatives.

P1: The standard embankment procedure This procedure follows the construction phases as explained in Section 5.3.2. The estimated building time is 2.5 years. After 2.5 years the procedure for the realization of land in between the embankment and the ring-dike can be started. The same execution method is applied for these areas. Finally, after 4 to 5 years the land can be prepared for its predefined functions.

P2-a: The short term clay ripening procedure The short term clay ripening procedure focuses on the shallow parts of the basin, near the ring-dike. In Phase 1, the water sheet is removed and the ground water level is lowered. After approximately 2 years of clay ripening, incoming supplies of dredged material can be deposited in designated areas according to a predefined elevation plan (Phase 2). Within 5 years time, approximately 30ha of the shallow parts can be elevated to the desired level and covered by a layer of sand (Phase 3). These parts can then be prepared for the predefined functions. One of the necessary preparation measures may be to install a system of vertical drains to enhance consolidation.

P2-b: The long term clay ripening procedure If it is decided to continue the clay ripening method for the rest of the basin as well, then the corresponding procedure can be started. The long term procedure is the same as the short term procedure, and first continues to elevate the remainder of the shallow parts of the basin. At this point it is also possible to elevate the deeper parts of the basin. The difference with the shallow parts is that it takes longer to elevate these areas. This is because the area is bigger and a thicker layer of ripened clay is needed to provide enough bearing capacity. It is estimated that after approximately 20 years 120ha of land is ready for further development.

P3: The reinforced embankment procedure This procedure follows the construction phases as explained in Section 5.3.5. If the problem of squeezing (geotechnical failure mechanism) can be overcome, then this type of embankment can be realized faster than the standard embankment. Once the embankment is finished the procedure for realization of land in between the embankment and the ring-dike can be started.

All procedures follow the path to the last two boxes of the flow chart. It is proposed to start using the reclaimed land for the selected functions and in the meantime continue storage of dredged material. Finally, it should be checked whether revision of the long term management plan is required.
CHAPTER 6. PREPARING A LONG TERM MANAGEMENT PLAN

Recommended functions

At the moment, there is no function in particular that can be regarded as the best solution. This is mainly because the investigated functions require a considerable investment, whilst revenues in the short term are uncertain.

The Port Authority is recommended to investigate the desirability and feasibility of solar energy, and to initiate a pilot project. Similar initiatives can be taken for fish breeding, greenhouse cultivation and Bevi-companies. It should be noted, however, that partners have to be found who are willing to invest in such projects. These types of functions require a considerable investment. The Port Authority could offer the area at a lower rent in order to make it more attractive. However, that also depends on the required measures to be taken to make the area suitable for development. On the other hand, these functions do not require a large amount of space.

Nature will always provide a surplus value to the port of Rotterdam, because it compensates for industry, now and in the future.

Recommended path in the flow chart

The requirement of the remaining storage capacity restricts the opportunities of a retaining structure in the shape of an embankment. The structure will have to be high, with gentle slopes and will therefore take up a lot of space. In the geotechnical analysis, the feasibility of an embankment was proven, but the construction costs are very high. It is unlikely that the Port Authority is willing to invest such a large sum for separating a part of the basin by means of an embankment. This alternative will become more feasible if the required remaining storage capacity is less. In that case a smaller embankment may suffice.

The most cost effective option for the Slufter is to allow for ripening of the dredged material, in combination with dry storage \(^2\). Thereby, the supply of dredged material is used as elevation material, while in the meantime the Slufter’s storage function is maintained. By using the incoming quantities of dredged material the basin can be partly dismantled over time. Once the layer of ripened clay has become thick enough, the area is accessible to heavy equipment and ready for further development. The geotechnical properties of the dredged material beneath the ripened clay can be improved by installing of a system of vertical drains.

The thick black line in the flow chart displays the recommended path.

Visualization of the proposed solution

In this section, the proposed solution, based on the clay ripening alternative, is visualized. With respect to the use of space, the Slufter can be divided into three main areas where functions can be accommodated: The crest of the ring-dike, the shallow part of the basin, and the deep part of the basin (see Figure 6.1).

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\(^2\)By dry storage is meant that the incoming dredged material is still discharged into the basin as a soil/water mixture. However, once inside, the clay and silt particles rest on top of the ripened dredged material whilst the excess water evaporates or percolates to deeper layers.
The crest of the ring-dike has a surface area of approximately 4ha. At the moment, this area is used for infrastructure and windmills. The options for this area in the future are:

- Increase surface area by lowering the ring-dike
- Remove windmills permanently
- Replace existing windmills
- Install additional windmills along the southern section of the ring-dike

Figure 6.1: The Slufter can be divided into three main areas where functions can be accommodated: The crest of the ring-dike, the shallow parts of the basin, and the deep parts of the basin.

The second area is the shallow part of the basin (60ha), and the third area is the deep part of the basin (200ha). If the clay ripening alternative is applied, first, the shallow parts of the basin near the ring-dike can be developed.

Suppose the average annual supply of dredged material in the future is 300,000m\(^3\) and the elevation speed of the surcharges of ripening clay is 1m/year, then 30ha of the shallow parts can be elevated by one meter in one year. In the geotechnical analysis was shown that a layer of ripened clay with a thickness of 3m provides enough bearing capacity for further development. So it takes 3 years to elevate 30ha of land. With a predefined plan for the deposition of dredged material it is estimated that the obtained land can be prepared for the selected functions after 5 to 6 years. The remaining 30ha of the shallow parts can be elevated and prepared in the subsequent 3 years.

In the deep parts, the layer of ripened clay has to be 4.5m thick in order to provide enough bearing capacity. Therefore it takes 4.5 years to elevate 30ha in these parts. If the clay ripening approach is followed for the deep parts as well and the procedure is started in 2012,
then approximately 120ha can be reclaimed by the year 2030. Figure 6.2 shows how fast land can be reclaimed by clay ripening. The progress is staged, because an area can only be regarded as usable land when the layer of ripened clay is sufficiently thick, i.e. 3 years for the shallow parts and 4.5 years for the deep parts.

![Figure 6.2: Progress of land reclamation by means of clay ripening if the procedure is started in 2012](image)

The obtained hectares can be used to accommodate functions such as algae breeding, solar energy, fish breeding, greenhouse cultivation or a combination of those.

The volumes of sand from the ring-dike, which no longer requires an elevation height of NAP +24m, can be used for the cover layer on top of the ripened dredged material. In the meantime further dismantling of parts of the basin can continue. The obtained land can temporarily be given nature values. Designated sections can be made suitable for breeding and foraging birds, and plants can be grown to improve the bearing capacity of the ripened clay.

After 25 to 30 years years either the entire basin is covered by a thick layer of ripened clay or the supply of contaminated dredged material has reduced to insignificantly small quantities. The time has come to dismantle the entire basin. Based on the knowledge and circumstances of the day, the area can either remain a buffer for nature, or be developed to accommodate industrial functions.

Table 6.2 gives an overview of the possible future functions and where they can be applied. As shown, most functions can be accommodated in the shallow parts in the short term and in the deeper parts in the long term, although some functions will require large pile foundations.
The ring-dike can be used for windmills and solar energy in the short term.

The proposed phased steps for the early dismantlement of the Slufter as a disposal facility combines short term opportunities with a long term, sustainable solution for the decrease in dredged material supply and usage of the surplus of space. It can be realized by altering the handling of dredged material from wet storage to dry storage. This requires a shift in the way the basin is managed.
### Table 6.1: Summary of pros and cons of the remaining functions

<table>
<thead>
<tr>
<th>Function</th>
<th>Pros</th>
<th>Cons</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solar energy</td>
<td>- Renewable energy source</td>
<td>- Investment not economically viable in the short term</td>
</tr>
<tr>
<td></td>
<td>- Light weight and efficient use of space</td>
<td>- Frequent lack of sunshine in the Netherlands</td>
</tr>
<tr>
<td></td>
<td>- Excess power transferred to the national power grid</td>
<td></td>
</tr>
<tr>
<td>Algae breeding farm</td>
<td>- Renewable energy source</td>
<td>- No bio-fuels</td>
</tr>
<tr>
<td></td>
<td>- There is a market (food and pharmaceutical industry)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Might be possible without too much adaptation of the area</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Supply of CO2 and electricity nearby</td>
<td></td>
</tr>
<tr>
<td>Windmills</td>
<td>- Renewable energy source</td>
<td>- Limited options for expansion</td>
</tr>
<tr>
<td></td>
<td>- Technically feasible in the ring-dike</td>
<td>- Opposition Westvoorne community</td>
</tr>
<tr>
<td>Greenhouse cultivation</td>
<td>- Agricultural land is valuable</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Heat and CO2 sources nearby</td>
<td></td>
</tr>
<tr>
<td>Fish breeding farm</td>
<td>- Knowledge and experience available</td>
<td>- Influences overfishing negatively due to fish food</td>
</tr>
<tr>
<td></td>
<td>- Combinable with algae breeding (fish food substitute)</td>
<td>- Strict requirements to the quality</td>
</tr>
<tr>
<td></td>
<td>- Growing market</td>
<td>- Weak competitive position Dutch fish breeding market</td>
</tr>
<tr>
<td></td>
<td>- Can be executed on small scale</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- EU subsidies</td>
<td></td>
</tr>
<tr>
<td>Nature reserve</td>
<td>- Can be established with temporary nature exemption</td>
<td>- Difficult to move endangered species if converted to industry</td>
</tr>
<tr>
<td></td>
<td>- Compensates for industry, now and in the future</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Might evolve into a permanent nature reserve</td>
<td></td>
</tr>
<tr>
<td>Birds reserve</td>
<td>- Enough shelter, food and quiet</td>
<td>- Temporary nature exemption difficult to obtain</td>
</tr>
<tr>
<td></td>
<td>- Potential permanent residence for sea-gulls</td>
<td>- Contaminations might pose a risk</td>
</tr>
<tr>
<td>Empty depot</td>
<td>- High revenues from the rent</td>
<td>- Enough space already reserved on Maasvlakte I</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Straight and horizontal internal road required</td>
</tr>
<tr>
<td>Biomass processing plant</td>
<td>- Renewable energy source</td>
<td>- Other locations more suitable (Maasvlakte I/II)</td>
</tr>
<tr>
<td>Storage of oil products or chemicals</td>
<td>- High revenues from the rent and throughput</td>
<td>- Large pipeline transport distance</td>
</tr>
<tr>
<td></td>
<td>- Confined and isolated location</td>
<td>- Already enough capacity for the coming 10 to 15 years</td>
</tr>
<tr>
<td>Location Bevi- companies</td>
<td>- Remote and confined area</td>
<td>- Opposition community Westvoorne</td>
</tr>
<tr>
<td></td>
<td>- Demand for space for these type of companies</td>
<td>- Uncertainty about type of companies and scale</td>
</tr>
</tbody>
</table>
Table 6.2: Future functions and location options

<table>
<thead>
<tr>
<th>Short term functions</th>
<th>Ring-dike</th>
<th>Shallow parts</th>
<th>Deep parts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solar energy</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Algae breeding</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Windmills</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Greenhouse cultivation</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Fish breeding farm</td>
<td>x</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Bevi-companies</td>
<td>x</td>
<td>(x)</td>
<td></td>
</tr>
<tr>
<td>Nature</td>
<td>x</td>
<td></td>
<td>x</td>
</tr>
</tbody>
</table>

| Long term functions        |           |               |            |
| Empty depot                |           |               | x          |
| Biomass processing plant   | x         |               | (x)        |
| Storage of oil products or chemicals | x | (x) |

(* Large pile foundations required)
CHAPTER 6. PREPARING A LONG TERM MANAGEMENT PLAN

LONG TERM MANAGEMENT PLAN SLUFTER

- Do not lower ring-dike
- Lower ring-dike

Realize a flat area in the long term

Reclaim land by clay ripening & dry storage

Start short term clay ripening procedure

Continue with long term clay ripening procedure

Start long term clay ripening procedure

Start using reclaimed land for selected functions & maintain storage function

Check if revision of the long term management plan is required

Floating structures

Beyond the scope of this project

Figure 6.3: A flow chart for the preparation of a long term management plan for the future of the Slufter

Port of Rotterdam Authority 102
Procedure of standard embankment:

**Phase 1:** Spraying of sand from pontoon

**Phase 2:** Installation of vertical drains from pontoon

**Phase 3:** Finish embankment by surcharges of sand

Procedure of reinforced embankment:

**Phase 1:** Spraying of sand from pontoon

**Phase 2:** Assemble geocell on land

**Phase 3:** Place geocell & fill by spraying sand

**Phase 4:** Installation of vertical drains from pontoon

**Phase 5:** Finish embankment by surcharges of sand

Preparation for land in between embankment and ring-dike:

**Phase 1:** Spraying of sand from pontoon

**Phase 2:** Installation of vertical drains from pontoon

**Phase 3:** Elevate area between embankment and ring-dike

**Phase 4:** Prepare land for predefined function(s)

**Phase 5:** Return to main procedure

**Figure 6.4:** A flow chart for the standard embankment and the reinforced embankment procedure, including an indicative time line

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Chapter 6. Preparing a Long Term Management Plan

Start short term clay ripening procedure

Phase 1: Remove water sheet & lower ground water level

Phase 2: Deposit dredged material according to predefined plan

Phase 3: Gradually cover ripened clay layers with sand

Prepare land for predefined function(s)

Return to main procedure

Indicative timeline

Year 0
Year 1
Year 2
Year 3
Year 4
Year 5 [30ha]

Start long term clay ripening procedure

Phase 1: Deposit dredged material according to predefined plan

Phase 2: Gradually cover ripened clay layers with sand

Prepare land for predefined function(s)

Return to main procedure

Year 5 [30ha]
Year 10 [60ha]
Year 15 [90ha]
Year 20 [120ha]

Figure 6.5: A flow chart for the clay ripening procedure, including an indicative timeline
Chapter 7

Conclusions & Recommendations

During the elaboration of this thesis a long term management plan has been prepared based on an analysis of the functional, spatial and geotechnical possibilities of dredged material disposal facility, the Slufter. The functional analysis focusses on the possible future functions that can be accommodated at the Slufter. The spatial analysis focusses on the Slufter’s layout, the level of stored dredged material and the efficient use of space. The geotechnical analysis focusses on the possibilities for building on top of dredged material. In the spatial and geotechnical analysis two alternatives are considered; the first alternative involves the separation of a part of the basin by means of a retaining structure, the second alternative is based on a process of ripening clay.

In this chapter conclusions are drawn on the results of the analyses and recommendations are given on the additional research that has to be done in order to test the viability of the prepared long term management plan.

7.1 Conclusions

Initially, requirements were posed by the Port Authority to give directions to a suitable solution. In the elaboration process various requirements have presented themselves, of which the following are the most important:

- A complete dismantlement of the Slufter as a disposal facility within the near future is not an option. Although the annual supply of dredged material is small, contaminated dredged material is still a problem and therefore it is necessary to maintain sufficient storage capacity
- No large quantities of already deposited dredged material shall be removed or transported
- The confinement function of the Slufter has to be maintained. Therefore, a breach of the ring-dike to connect the area with open water is not an option (lowering of the ring-dike is possible though)
- The presence of contaminated dredged material restricts the applicability of the functions. Therefore, alternatives without land reclamation have been discarded in this
research. Once parts of the basin have been reclaimed, these parts should have a cover layer of clean sand to ensure that the contaminations are sealed off.

The conclusions are drawn by looking into the most important results from the functional, spatial and geotechnical analysis. Finally, concluding remarks are given on the proposed long term management plan for the future of the Slufter.

**Functional analysis**

In the short term, the best opportunities for new functions accommodated at the Slufter can be found in renewable energy and nature. The types of renewable energy considered in this report are solar energy, wind energy and algae breeding.

The current state of technology is such that purchase and use of solar panels can be cost effective, and this is improving constantly. A reasonable amount of energy can be generated on a relatively small scale. A windmill takes up approximately the same amount of space as a field covered with solar panels to obtain the same amount of energy. Although the investment costs of solar panels are much higher than for windmills. Whereas the placement of windmills is restricted by rotor diameter, height and landscape pollution, solar panels restrict the applicability of the area it consumes. However, the surplus of space at the Slufter provides opportunities for a solar energy project initiative in the Netherlands.

Expansion of wind energy at the Slufter is possible on the southern section of the ring-dike. Moreover, there are plans to replace the current 17 windmills by windmills with a larger capacity.

In general, the Port Authority values and encourages initiatives to develop renewable energy in the harbor. Partners have to be found who are willing to invest in these types of projects. This might prove difficult, because the investment costs are high.

In collaboration with the Rotterdam Climate Initiative, the Port Authority is looking into the possibilities of a pilot project for algae breeding at the Slufter. Algae breeding does not take place at such a large scale yet so that it can significantly contribute to the production of biochemicals and biofuels. However, production on a relatively small scale can supply algae for application in the food and pharmaceutical industry.

Nature will always provide a surplus value to the port of Rotterdam, because it compensates for industrial growth, now and in the future. The circumstances for birds are ideal, because it is one of the few places left along the Dutch coastline where enough food, shelter and quiet can be found. Once the Port Authority decides to designate an area for nature, a temporary nature exemption may hold doors open for a conversion to industry in the long term.

The Slufter does not seem to be a very suitable location for harbor related functions or industry, such as biomass plants, empty depots and storage of chemicals. The main reason for this is that many places on Maasvlakte I and Maasvlakte II, that have not been issued yet, are more suitable and provide sufficient space at least for the coming 20 years. This is unfortunate for the Slufter since these types of functions are likely to have a positive return on investment within a reasonable amount of time. It is therefore supposed that harbor related functions can be accommodated at the Slufter in the long term after complete dismantlement of the basin, depending on the developments of the day.
7.1. CONCLUSIONS

Of the other functions considered, fish breeding, greenhouse cultivation and a location for Bevi-companies (handling and storage of dangerous substances) are also potential functions applicable at the Slufter in the short term. The market for artificially produced fish is growing, is subsidized by the European Union and can be executed on a relatively small scale. Agricultural land is decreasing in the Netherlands so greenhouses at the Slufter might provide an alternative. Moreover, the required sources for the breeding of fish and the growing of crops, such as heat, CO2 and electricity are nearby (industries on the Maasvlakte). Fish breeding can be combined with algae breeding, because algae provide an alternative to conventional fish food (which consists of fish caught in the ocean).

Temporary storage of soil (TOP), soil cleansing and recycling, a service center, a golf course and a hydropower station are functions that are considered unsuitable for the Slufter.

Spatial Analysis

The possibilities for the use of space are restricted by the remaining storage capacity requirement, which is based on the filling prognosis. A distinction is made between an optimistic and a realistic scenario, where either 3/4 or 4/5 of the entire basin volume has to be used for the storage of dredged material, respectively.

Two main alternatives have been investigated; the retaining structure alternative and the clay ripening alternative.

In the retaining structure alternative, the obtainable area of land is a function of the remaining storage capacity, the elevation height of the structure and its slope. However, if the retaining structure is an embankment the obtained area reduces because of slopes on either side. Gentle slopes of 1:10 are necessary because of the low bearing capacity of the dredged material, and a considerable elevation height of 15m is required. In that case the remaining horizontal land becomes 37ha in the optimistic scenario and only 6ha in the realistic scenario. More horizontal land can be realized if the existing ring-dike is lowered. That way, another 68ha can be gained.

The clay ripening alternative is in a first approach based on lowering the water level in the basin, continue wet storage in the middle of the basin and realize a second inner ring-dike to ensure enough storage capacity in the future. The analysis shows that with the required remaining storage capacity, the inner dike has to be executed with such large dimensions that it intersects with the existing ring-dike. Moreover, large quantities of elevation material would be required.

Four alternative approaches have been proposed:

1. Allow five years of clay ripening and dry storage, then realize an embankment and maintain wet storage
2. Allow five years of clay ripening and dry storage, then realize an embankment and maintain dry storage

\[1\] By dry storage is meant that the incoming dredged material is still discharged into the basin as a soil/water mixture. However, once inside, the clay and silt particles rest on top of the ripened dredged material whilst the excess water evaporates or percolates to deeper layers.

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3. Focus on the shallow parts of the basin near the ring-dike; lower and widen the ring-
dike, and maintain dry storage  
4. Initiate clay ripening and dry storage, and slowly elevate consecutive parts of the basin  
   until a thick layer of ripened clay makes further development possible. The lowering  
   of the ring-dike is optional  

In case of dry storage the remaining storage capacity requirement decreases, because the  
cubic meters of storage space are used more effectively. The results from the analysis show  
that in the first approach no horizontal land remains if the embankment has a height of 15m  
and a slope of 1:10. In case of the second approach 37ha can be obtained in the optimistic  
and 21ha in the realistic scenario, with lower embankments. Thus, dry storage provides  
better opportunities for the use of space. However, the feasibility of this approach depends  
on whether it is possible to build an embankment on ripened clay, and the execution costs.  

In the third approach, 100ha can be obtained in the realistic scenario if the ring-dike is  
lowered to a level of NAP +12m and widened by 30m, with an inner slope of 1:10 running  
along the shallow part of the basin.  

All of the above mentioned approaches involve considerable measures with respect to an  
alteration of the layout, and the movement of large quantities of soil. They provide a short  
term solution for the reclamation of a part of the Slufter, however, at great expense.  

The fourth approach is by far the most cost effective option, because the supply of dredged  
material and the natural process of ripening clay is used as the elevation method. However,  
it is a slow process. It combines a long term solution for the entire basin with short term  
development of small areas in phases.  

Geotechnical analysis  

A distinction is made between the retaining structure alternative and the clay ripening  
alternative. In the retaining structure alternative analysis, first a selection is made of the  
type of retaining structure. For the Slufter, an embankment provides the most suitable  
option. The other viable option, a cofferdam, is a factor 7 to 8 more expensive, mainly due  
to the large quantities of sheet pile required.  

The geotechnical feasibility of an embankment, with an elevation height of 15m and slopes  
of 1:10, has been analyzed by looking into the different phases of the execution method.  
Three main construction phases are discerned: First a layer of sand is sprayed on top of the  
dredged material from a pontoon, then a system of vertical drains is installed to accelerate  
the consolidation process, and finally the area is elevated further in order to finalize the  
embankment.  

Exploratory calculations by hand and in MStab and MSettle have shown that it is possible to  
spray thin layers of sand on top of the dredged material. However, once the layer becomes too  
 thick the slope along the edges becomes unstable. It is very difficult to create a soil structure  
that is stable. In particular, the lowering of the water level and loading by vertical drainage  
installation equipment leads to unacceptable instabilities of the soil structure. Therefore, the  
vertical drains must be installed from a pontoon. A sprayed layer of 1m provides a stable  
basis for the embankment, and keeps the vertical drains in place at the top.
7.1. CONCLUSIONS

After a sufficient degree of consolidation the water level can be lowered and the area accessed by excavators. This is a critical moment in the construction phase, because the soil structure may have insufficient bearing capacity under extreme load conditions. The practical solution to this problem is either to wait longer and allow for a higher degree of consolidation or to elevate the first few layers with lighter equipment.

Following this construction method an embankment can be realized within 2.5 years time. The execution costs are approximately €28,000 per running meter, which is very high. In the analysis a drain installation depth at NAP -27m and a drain center-to-center distance of 2m was used in the calculations. It is possible to reduce the execution costs by optimizing drain depth and distance. The analysis has shown that a stable embankment can still be realized within the same construction time, with a drain depth at NAP -20m or a drain distance of 2.5m.

In an analysis of the users phase, the loads imposed by the weight of function related elements have been simulated. The results have shown that functions with no or shallow foundations can be applied if the layer of sand that covers the dredged material is sufficiently thick. If the residual settlements are too large, extra measures have to be taken. Facilities on pile foundations are recommended to be placed near the ring-dike, where the layer of dredged material is thin.

Because the execution costs for the conventional embankment are very high it is worth looking into an alternative. The proposed alternative is an embankment reinforced with Tensar geocells. This type of embankment has a firm base, consisting of a 1m high geocell structure filled with granular material, which enables steeper slopes and thereby the use of less construction material.

The critical failure mechanism for this type of embankment is squeezing. Because of the low value of the undrained shear strength, squeezing is expected. For the same reason, it is very likely that the execution method as proposed by Tensar is not applicable at the Slufter. Therefore, a different method is proposed, which is based on executing multiple phases from a pontoon. Additional research is required to prove the geotechnical feasibility of the reinforced embankment.

The execution costs of an embankment built with geocells with a height of 15m and slopes of 1:4.5 are a factor 1.5 to 2 less than the conventional embankment.

Alternative 2 for land reclamation is based on the natural process of ripening clay. An analysis in MStab and MSettle has shown that the possibilities for building a soil structure on top of a 1m thick layer of ripened clay are limited, because of insufficient bearing capacity. Therefore, fast development of the area by means of clay ripening is not feasible in the short term.

An approach is proposed, where designated areas within the basin are slowly elevated by dry storage of dredged material in combination with clay ripening. When the layer of ripened clay has become sufficiently thick it is accessible by vertical drainage installation equipment and can be further developed. According to the simulations in MStab, the required thickness of the ripened clay has to be 3m in the shallow parts of the basin, near the ring-dike, and 4.5m in the deeper parts. After sufficient consolidation of the sub-lying weak layer of dredged...
material, it is possible to realize a small inner ring-dike on top of the ripened clay. Thereby, the life span of the Slufter as a disposal facility can be expanded if necessary.

Preparing a long term management plan

The results of the functional, spatial and geotechnical analyses have been combined by preparing a long term management plan for the future of the Slufter. The possibilities for the Slufter in the short term are limited, mainly for two reasons: Firstly, many places on Maasvlakte I and Maasvlakte II have not been issued yet. These areas are more suitable for the development of harbor related functions and industry, because they have a better connectivity and are already prepared for development. Secondly, the separation of a part of the Slufter basin by means of an embankment to obtain land for other functions is technically feasible, but the investment costs are very high. Therefore, a long term management plan is needed, combined with short term opportunities.

The most cost effective option to obtain land is to make a shift from wet storage of dredged material to dry storage, and to initiate a process of ripening clay. The supply of contaminated sediment provides the elevation material. This way, first the shallow parts of the basin near the ring-dike can be developed and later on the rest of the basin.

Functions that do not require a lot of space, such as algae breeding, solar energy and fish breeding can be accommodated in these earlier developed shallow parts. In the meantime, areas that do not have any purpose yet can be given nature values. By the time the entire basin can be dismantled either the presence of nature can be enhanced or the Slufter can accommodate industrial functions.

Suppose this plan is set in motion today, then the shallow parts of the basin, with a surface area of 60ha, can be elevated and developed by the year 2020. After ten more years, another 60ha of land can be developed by reclaiming central parts of the basin.

A shift from wet storage to dry storage also implicates that the high surrounding dike of the Slufter is no longer needed. This ring-dike can therefore be lowered, and thereby land can be obtained that can also be used for other purposes.

A decision making tool has been developed, which gives an overview of the short and long term possibilities for the Slufter, and serves as a guideline in the process of establishing a long term management plan.

7.2 Recommendations

If the long term management plan, as proposed in this report, is to be realized then the recommendations given in this section are considerations for the further steps to be taken.

First of all, the Port Authority will have to assess what a shift from wet to dry storage beholds. Any potential hazards from contaminations or consequences on storage handling will have to be expressed and validated in order to prove the viability of dry storage.

The potential functions that can be accommodated at the Slufter in the short term will have to be investigated further on their feasibility and desirability. These functions are
algae breeding, solar panels, fish breeding, greenhouse cultivation and a location for Bevi-companies. The Port Authority can try to stimulate entrepreneurship and find partners who are willing to invest in these types of projects. As a first step, this can be done by initiating pilot projects. Alongside, a plan can be written for the development of parts of the Slufter as a nature or birds reserve. The Port Authority will have to decide whether or not a temporary nature exemption is necessary.

If the decision is made to slowly dismantle the Slufter by clay ripening, areas have to be appointed where development will have to take place first. A plan needs to be established for the deposition of incoming quantities of dredged material in designated areas. In addition, further research has to be done on the clay ripening process in order to estimate the speed by which the clay ripens. This, and the frequency with which supplies of dredged material are brought to the Slufter, determines how fast land can be reclaimed.

Floating structures should be further investigated as an alternative to land reclamation. At the moment, the costs of a floating structure with a base of EPS are approximately €50-90 per square meter. The investment costs are therefore high.

A final remark is made on the retaining structure alternative. Its possibilities are limited because of the remaining storage capacity requirement. If less storage space is required then this alternative may become more feasible, because a smaller embankment is needed and therefore less material. In the spatial analysis the required remaining storage capacity is 31.2Mm$^3$ in the optimistic scenario, and 35.8Mm$^3$ in the realistic scenario. Suppose the average annual supply of dredged material in the future is 300,000m$^3$, which is plausible, then storage can continue for another 104 and 119 years, respectively. This is an unnecessarily long time.

With a smaller required remaining storage capacity, it is worth to further investigate the economical and technical feasibility of a reinforced embankment. In case a retaining structure does provide the desired solution, then additional research has to be done on the geotechnical properties of the dredged material, by means of field and laboratory tests.
Bibliography


Appendix A

Background information Slufter

This Appendix gives a brief description of the most important elements and characteristics of the Slufter, starting with its location and exploitation history. The organizational structure shows how the Slufter is managed and the different laws and acts that the facility has to comply with are described. The next section gives an overview of the existing installations and system infrastructure, followed by storage information. The final parts go into the environmental factors and recent innovations and developments.

A.1 Location and history

The Slufter is a large scale facility for storage of dredged material from the Netherlands’ downstream river regions. Dredged material that is too contaminated for spreading in the North Sea or for handling and re-use on land can be contained within the Slufter basin. The Slufter is founded in 1987 and is situated at the southwestern tip of the Maasvlakte (see Figure A.1).

Initially, the basin’s capacity was fully reserved for the Port Authority and Rijkswaterstaat projects in the province of Zuid-Holland. However, since the dredged material supply from the intended sources was lower than expected, Rijkswaterstaat decided in 2004 to allow for dredged material from their projects outside the Rhine-Maas river mouth regions. For the same reason, from 2006 onwards, the Slufter was also made more accessible for third parties. In the year 2007 approx. 1.4 million m$^3$ of dredged material was stored in the Slufter. This is a significant decrease compared to the peak storage in 1990 with 5 million m$^3$ of dredged material. In 1990, approx. 80% of the material came from the port of Rotterdam, 15% from Rijkswaterstaat projects and 5% from third parties. In 2007, this ratio was approx. 40-50-10%. Since the start of operation up until 2007, approximately 50% of the storage capacity had been used. In 2004, it was expected that with a yearly deposit of 3 to 5 million m$^3$ the Slufter would be completely full within 10 to 15 years. The most recent soundings show a filling of approximately 52%.

In the starting phase of exploitation, the Slufter was only meant for storage of class II and class III dredged material. However, the rules and regulations with regard to the storage
of contaminated dredged material have become more pliable over the years. Therefore, the highly contaminated class IV material is also accepted.

**A.2 Organizational structure**

The exploitation of the Slufter is done by the Port Authority in collaboration with Rijkswaterstaat Zuid-Holland, since 2004 called the ‘Beheersorganisatie Slufter’ (BoS). The organizational structure of the BoS is given in Figure A.2.

![Organizational structure of the BoS](image)

**Figure A.2:** Organizational structure of the BoS

MT- Slufter: Management Team Slufter  
HbR MI LNDD: Havenbedrijf Rotterdam Management Infrastructuur, Unit Landdepots  
HbR BNI: Havenbedrijf Rotterdam, Havens en Vaarwegen  
RWS ZH AVN: Rijkswaterstaat Zuid-Holland, Waterdistrict Nieuwe Waterweg
A.3 Laws and Acts

The handling and storage of contaminated and hazardous dredged material involves a number of laws and acts to be complied with. This section shortly describes the laws and acts that are most relevant.

Besluit Bodemkwaliteit (Bbk)

This amendment declares the criteria for the classification and the aims for handling of dredged material. In the Bbk the different dredged material is classified according to its applicability for handling and re-use [20]. The classes are: Unlimitedly applicable, class A, class B, not applicable and never applicable. However, the Slufter’s permit still refers to the former classification with a range from class I to class IV, where classes II, III and IV qualify for storage in the Slufter. In the near future the permit will be renewed with the consequential new classification, where all material starting from class B is qualified for storage in the Slufter (see Figure A.3).

Figure A.3: Classification of contaminated dredged material (above: the old classification still adhered by the BoS, below: the new classification). Note: the distribution in this figure is purely schematic and does not give an indication of parameters.

Minimale Verwerkingsstandaard (MVS)

The purpose of the MVS is to stimulate the processing of dredged material to reusable materials such as clean sand or clay, in order to save scarce storage capacity and to limit the use of raw materials [20]. The act does not apply to salty dredged material, because this particular material can not be re-used inside the dike. However, ample other applications can be found in the Slufter’s vicinity and therefore salty dredged material with a high sand content is also processed.
APPENDIX A. BACKGROUND INFORMATION SLUFTER

Wet Milieubeheer (Wm)

In the Wm all permits that are concerned with the environment are issued. Its aim is to prevent nuisance and limit environmental impacts. The ‘Milieu Effecten Rapportage’ (MER) is included in the Wm and is used in the process of granting permits \[20\].

Wet verontreiniging oppervlaktewater (Wvo)

The part of the process water that is not re-circulated and excess surface water in the Slufter basin is discharged into the Mississippi haven. The quality of the water is monitored and the permits involved are included within the Wvo \[20\]. The Wvo aims to act upon and prevent pollution of surface water.

A.4 Installations and system infrastructure

This section covers all the existing installations and infrastructure within the Slufter’s area.

Piping infrastructure

From the eastern side of the basin two discharge pipelines are directed to the north and south along the basin’s perimeter. Dredged material is shipped in through the Mississippi harbor, where the hopper is berthed and connected to the discharge pipeline. The soil-water mixture is discharged (partly underground) through the pipeline towards the Slufter where it is distributed over the basin. The pipelines along the ring-dike have eleven discharge points in total that can be controlled by opening or closing the valves. The network is provided with return flow pipelines and a recirculation pump and basin in order to decrease the discharge of process excess water in the Mississippi harbor.

At various locations in and around the Slufter basin there are altogether 17 sounding pipes that function as the groundwater monitoring system. By sampling and analyzing the quality of the groundwater on a regular basis, the system examines whether the basins isolating function is guaranteed and no harm is done to the environment through groundwater flow. As a result of the filling of the basin and the subsequent surface water rise, the accessibility of the sounding pipes near the inner bank of the basin had become limited. Therefore, these pipes have been re-installed under an angle, slanting towards the lower soil layers.

Pontoon

A pontoon and a floating pipeline installation inside the basin provide the opportunity to discharge dredged material centrally. This methodology was applied for the storage of the most contaminated dredged volumes class IV, maximizing containment and minimizing risk. However, recent developments have shown that the entire basin is safe enough to contain these materials and discharge near the banks is possible without risk. Another pontoon on
the bank of the ‘Mississippihaven’ facilitates the installation for connecting the discharge pipelines to the hopper.

Earth fills

The enclosing ring-dike of the Slufter basin is the primary structure for maintaining the storage function. The crest level varies from NAP +24m to NAP +25m and the lowest point of the basin is at NAP -28m. The basic dimensions are shown in Figure A.5.

Since 1993, specific parts within the Slufter were designated for separating sand and maturing clay in order to save basin capacity and provide alternatives for primary building materials. These areas were located along the northern and eastern edge of the basin. When the dredged material and subsequent surface water sheet had become level with the crests of the sand and clay processing basins, areas outside the ring-dike were designated for these
purposes. The old fields can still be seen, as they are partly above the water level. The current sand and clay processing fields lie outside the eastern side of the basin. At the moment the clay ripening field is fairly inactive. This area will be taken up and occupied by facilities that complement to the Maasvlakte 2.

South of the sand separation basins is an area exploited by A&G, a company that specializes in the handling and storage of dry contaminated dredged material.

Road infrastructure and nature facilities

The access road to the Slufter comes in from the east. Along the crest of the ring-dike runs an asphalt road with a width of approximately 3m. The eastern edge of the basin is made accessible for truck unloading by means of an entrance road and ramps. The entrance of the facility is provided with an office for reception and inspection.

In the east of the area, between the old and the new sand separation basins, is the so-called bird’s valley. This area will entirely be taken up by the future infrastructural footprint of Maasvlakte 2. The intention is to provide a new bird’s valley at the location where the old inactive sand separation basin is situated.

South of the Slufter is the ‘Oostvoornse Meer’ and a dune and forest area which together function as a nature reserve.

Water control systems

The water level in the Slufter basin is kept at a constant level of approximately NAP +6.5m at the moment. At the small cove in the east of the basin a weir is situated between two soil projections that extend from the dike’s inner slope. The excess water due to rainfall and slurry discharge is caught in a small basin behind the weir, from where the relatively clean water is pumped and discharged into the Mississippihaven or used in the recirculation system. The aim is to be able to control the basin’s water level and to keep the water sheet as small as possible, but accessible for the operation boat.

In 2006, the water level was brought to NAP +8.5m, which led to an undesired rise of the groundwater level in the surrounding areas, because of percolating water through the core of the dike [20]. Lowering the water level to NAP +6.5 was not drastic enough and therefore 7 drainage wells and pumps were installed along the outside of eastern dike section. At the moment, the groundwater level is under control and the drainage system is not used for this purpose anymore. The installation is still active though; the neighboring company A&G benefits from a lower groundwater level and therefore leases the drainage system.

Structural facilities

The Slufter building, which is the main working office, is situated at the northern tip of the ring-dike. It is provided with parking space. Along the perimeter of the facility is a fence. There’s an entrance facility with a weighbridge and a materials storage facility and
A.5 Storage information

Basin fill pattern

In the starting phase of the Slufter's exploitation the basin was constantly filled with water up to approximately NAP level. This phase is called the 'underwater phase' and lasted until 2002, when the stored dredged material was nearing NAP level. The underwater phase was followed by the 'extended underwater phase' until storage by means of a diffuser on a pontoon was no longer possible. From this point onwards the 'above water phase' began, which involved a new handling strategy. It was decided to continue the 'wet' storage of dredged material, which finally led to the increase of the water surface level to NAP +6.5m. The above water phase was characterized by the newly installed pipeline discharge system.

Initially, only class II and III dredged material were suited for storage in the Slufter. Therefore, the lower part of the Slufter, up to a level of approximately NAP -2m, is filled with this material. When the Slufter was made accessible to class IV material as well, it was decided to store this highly contaminated dredged material centrally. In 2006, the class IV storage facility 'Papegaaienbek' was dismantled, its stored volumes transferred to the Slufter and also placed centrally [20].

Figure A.6: Floating pipeline for central storage, and in the background the Slufter office
Contaminated dredged material

The quality of the dredged material to be stored is represented by four main parameters; arsenic, fluoranthene, dieldrin and PCB. These parameters are representative for the four groups of substances that play an important role in the contamination of the sediments, respectively:

- Heavy metals and arsenic
- Polycyclic Aromatic Hydrocarbons (PAHs)
- Organochloride pesticides
- Polychlorinated biphenyl (PCBs)

Heavy metals have a potential toxicity and when appearing in high concentrations they can have a negative influence on growth and metabolism of organisms. The PAHs, pesticides and PCBs can be characterised by a strong capability to adsorb to silt particles, high accumulation with organisms and a low degradability.

As it enters the basin, the dredged material is a mixture of water, solids and gasses. The micro-contaminations in this mixture, categorized by the above mentioned groups, can be found in solid or dissolved phase. Although the dissolved substances are relevant for the diffusion of contaminants by groundwater flow, the concentration of micro-contaminations is much higher in the solid phase than in the dissolved phase. The contaminations can be present by either adsorption to the solid phase (physical binding) or as deposits (chemical
A.5. STORAGE INFORMATION

Figure A.8: Schematic presentation of the fill pattern [24] (BAGA stands for ‘Besluit Aanwijzing Gevaarlijke Afvalstoffen’ and is part of the ‘Wet Milieubeheer’, see paragraph A.3)

binding). Organic micro-contaminations are mainly adsorbed to the organic material present in the slurry, whilst the inorganic contaminations (or metals) occur as insoluble substances. The physical-chemical conditions that influence the degree of adsorption and are indicative for the state of an sediment system are:

- The percentage of silt and organic material
- The acidity (pH-value)
- The redox-potential, indicating the oxygen content

The ion-strength, alkality, and temperature influence the solubility of a substance.

Sand separation and clay ripening

Dredged material with a sand content larger than 60% is discharged into the sand separation basins where the sand fraction is separated from the contaminated finer fraction by a sedimentation process. The technique is applicable for the class II and III types of dredged material and provides sand that can be reused. The former separation basin was situated inside the ring-dike on the eastern bank, but ceased to exist when the above water phase began. In 2006 three new basins were established outside the Slufter, with a potential productivity of 160.000m$^3$ per circulation [20]. The main value of sand separation is to save the Slufter’s storage capacity and the obtained volumes have so far only been applied within the Slufter area. The obligation to process contaminated dredged material with a high sand content for re-use is included in the act “Minimale Verwerkingsstandaard” (MVS).

By storing dredged material in a dry basin for a longer period of time a process of de-watering and oxygen flow takes place, called clay ripening. Class II and III clayey material, with a sand content of less than 50% and low organic matter content qualifies for this technique. The possibilities for re-use are limited by a value of 500mg/kg mineral oil content and the presence of sulphate and chloride in salty material [28].

When clay ripening took place within the Slufter’s ring-dike, practically all the obtained clay was re-used for Slufter purposes. The clay ripening field that momentarily lies along the eastern side of the area will be lost to the footprint of Maasvlakte 2. The prospects for
clay ripening activities in the Slufter area are adverse, because of the lack of space in the near future, the long processing time and the low commercial appeal.

A.6 Environmental factors

The storage of contaminated dredged material involves considerations with respect to nuisance and environmental impacts. However, opportunities for nature can also be found. The relevant subjects are discussed in this section.

Emissions

The emissions that have been investigated in the framework of permit requests are hydraulic emissions, smell and noise. The hydraulic emissions involve the process water that is discharged on the surface water and groundwater flow through the dike by percolation. These emissions are monitored and controlled according to the regulations.

An investigation on smell has been done in 2007 for the dredged material handling facilities, because the dredged material is relatively exposed here. The investigation concluded that non-sandy dredged material was perceptible in the vicinity of the installation and sandy material was not [20]. The nuisance of smell from the basin is limited, because of the water sheet on top of the stored material.

The impact of noise was investigated in 2004 by Royal Haskoning [14]. From the results was concluded that the emissions were below the limits.

Bird life

The places within the Slufter basin that have fallen dry can be an attractive breeding spot for summer birds visiting the coastal zones. Particularly avocets, plovers and terns can be found occasionally. Bird life in the Slufter area is encouraged by plenty of food, space and quiet. A sudden change in the water level of the Slufter can flood the lower lying dry parts, which poses risks to eggs in the breeding season. Therefore, the water level is well-managed and takes this particular season into account. It is a known fact that the Slufter’s foraging birds take in chemical pollutants through food and sediments, but so far there are no signs of reproductive abnormalities [19].

A.7 Innovative considerations and recent developments

In the past years the BoS has considered the applicability of various innovative techniques to the Slufter. Some of these techniques have been applied while others were not, but they can still have a potential value for this research. A few innovations are shortly described in this section, together with recent developments that have to be taken into account during the elaboration of this thesis.
A sandwich of clay and sand

A study has been done in 2003 on the technical properties of ripened clayey dredged material, which can be applied for the elevation of terrains and noise barriers. Tests have been done where a sandwich construction, consisting of alternating layers of clay and sand, is compared to a layer of clay. The layers have been tested on the workability by equipment. The conclusion was that the sandwich construction is a good option when clay with higher water content than usual has to be applied in elevation works. The sandwich technique has been applied in the construction of the sedimentation basins [20].

Geotubes

Geotubes provide a method for large scale de-watering of dredged material. The dredged material is discharged into a long permeable geo-textile ‘sausage’, through which the excess water can escape. The geotube can also have functional applications as a breakwater, noise barrier, bank revetment, dike-shoulder widening, dike reinforcement or dike elevation. Plans to do test with geotubes in the Slufter were dismissed based on a costs and necessity assessment [20].

A&G combi

A&G combi is a bound foundation material, consisting of four components: soil (possibly ripened dredged material), a binding agent to improve the technical properties, AVI bottom ash (residue of waste burning, functioning as a sand replacement) and an additive to stabilise the contaminations. It can be used as foundation or elevation material and has been applied at the Slufter in the sedimentation basins, the truck load dumping ground and along the slope of the ring-dike [20].

Beaudrain

Beaudrain is a vertical drainage technique developed by Boskalis. By means of these drains, the consolidation process of saturated compressible soil layers can be accelerated from decades to half a year or less. The Beaudrain technique has a range of advantages, but at the moment it is unclear to what extent and under which conditions it is applicable to the Slufter’s dredged material [20].

Floating bird island

A floating bird island has been realized in the Slufter basin to encourage the breeding of terns. The construction consists of polystyrene foam and a concrete floor. Large concrete block suspended from each corner by chains keep the island in place [5]. The construction was made by Dura Vermeer, a company that specializes in the development of floating constructions. The island is still in place and successfully hosts terns.
Algae production farm

At the moment there is an ongoing initiative by the Port Authority to develop an algae farm in the Slufter, for the production of bio fuels or chemicals. The intention is to use 50ha of the Slufter’s water surface for the farm, by means of a floating foundation. Potential partners are Exxonmobil and BASF. The progress and influence of this initiative will be monitored and taken into account during this research.

Windmills renewal

It is likely that in the near future the current 1.5MW windmills will be replaced by larger windmills with a capacity of 5MW.
Appendix B

Brainstorm session

On the following pages the slides of the presentation, used during the brainstorm session, are presented. A scan of one of the groups’ designs, resulting from the convergence phase, is also added.
APPENDIX B. BRAINSTORM SESSION

Brainstormsessie over de toekomst van de Slufter

Dinsdag 11 januari 2011
Roderik Heerema
RN.Heerema@portofrotterdam.com
T +31 (0)10 252 1508 M +31 (0)6 24692755

Introductie
Programma
9:00 – 9:30 Introductie
9:30 – 10:20 Divergentie: zoveel mogelijk ideeën genereren
10:20 – 10:35 Pauze
10:35 – 11:40 Convergentie: ideeën terugbrengen naar de meest kansrijke en een aantal functies uitwerken
11:40 – 12:00 Afronding

Introductie
Deelnemers
• In welk opzicht heb je WEL met de Slufter te maken?
• In welk opzicht heb je totaal NIET met de Slufter te maken?
• Wat verwacht je van deze ochtend?

Introductie
Toelichting
• Aanleiding: ‘Die bak komt nooit vol!’
• Vraagstelling: ‘Wat kan er, wat wil je er en hoe krijg je dat voor elkaar?’
• Doel: ‘Een top 10 van ideeën en 4 verder uitgewerkte functies’
• De spelregels...

Divergentie
De Spontane Braindump
• Schrijf in kernwoorden zo veel mogelijk ideeën op
• Je hebt 10 minuten de tijd
• Alias is mogelijk

Divergentie
De Actieheld...
• Bedenk een actieheld
• Noteer van deze held twee kenmerkende eigenschappen
• 5 minuutjes

Figure B.1: Presentation Brainstorm session
Divergentie

De Actieheld analogie
- Stel je voor wat deze held met de Slufter zou doen
- Schrijf opnieuw je ideeën op
- Nu is écht alles mogelijk
- Je hebt wederom 10 minuten de tijd

Convergentie

Selectie van ideeën
- Kies 2 van je favoriete ideeën uit

Pauze...

Convergentie

Stemmen
- Er zijn nog ... ideeën over
- Verdeel de stickertjes
  - Blauw = meest haalbaar
  - Geel = grootste positieve impact
- Wat staat er in de Top 10?

Convergentie

‘Brain Writing’
- 4 groepen van 3 personen
- Elke groep kiest van de 10 overgebleven ideeën een favoriete toekomstige nieuwe functie voor de Slufter

Convergentie

‘Brain Writing’ – Ronde 1
- Geef je gekozen functie vorm door grof een locatie en afmetingen binnen de Slufter te schetsen
- Je hebt 10 minuten de tijd

Convergentie

‘Brain Writing’ – Ronde 2
- Rouleer je resultaat door naar de volgende groep
- Bedenk nu een methode om de functie te implementeren
- Je hebt wederom 10 minuten de tijd

Figure B.2: Presentation Brainstorm session
**APPENDIX B. BRAINSTORM SESSION**

**Convergentie**

**'Brain Writing’ – Ronde 3**
- Rouleer je resultaat wederom door naar de volgende groep
- Bepaal grof de timing voor toepassing van de functie
- Geef je waarde oordeel: benoem plus- en minpunten
- Je hebt wederom 10 minuten de tijd

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**Afronding**

**Discussie en Reflectie**
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**Vragen of opmerkingen?**
- Hartelijk dank voor jullie komst en inbreng!

---

**Figure B.3:** Presentation Brainstorm session
Figure B.4: One of the resulting designs in the convergence phase
Appendix C

Criteria analysis

The results of the criteria analysis are presented in Figure C.1 on the next page.
### APPENDIX C. CRITERIA ANALYSIS

#### Figure C.1: The results of the criteria analysis

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<td>-1</td>
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<td>-1</td>
<td>-1</td>
<td>-2</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
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<td>33 Marina</td>
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<td>1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
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<td>1</td>
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<tr>
<td>42 Waste incineration</td>
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<td>1</td>
<td>-1</td>
<td>0</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

| 13 Ice skating rink | 1 | 0 | 0 | -1 | 0 | -1 | 0 | -2 | -1 | 2 | 0 | 1 | 1 | 4 |
| 27 Floating prison | 1 | -2 | 0 | 0 | -2 | 1 | -1 | -1 | -1 | 0 | 1 | 1 | 3 |
| 19 Outdoor swimming pool | 1 | 0 | 1 | -1 | -1 | 0 | -2 | 0 | -1 | 0 | 0 | 4 |
| 33 CO2 storage | 0 | -1 | 1 | 1 | -2 | -1 | -1 | 0 | 0 | -1 | 2 | 2 | 4 |
| 22 Airport | 2 | -2 | 0 | 2 | -2 | -2 | -2 | 1 | 2 | 1 | 0 | 1 | 1 | 3 |
| 24 Military practice terrain | -1 | -1 | 0 | 0 | 1 | 0 | -2 | 2 | -1 | -1 | 2 | 5 |
| 10 Watersports park | 0 | -1 | 1 | -1 | 1 | -1 | 1 | 0 | -1 | 1 | -1 | 0 | -1 | 0 |
| 5 Nuclear energy | -1 | -2 | 0 | 0 | -2 | -2 | 1 | 1 | -1 | 0 | 0 | -1 | -1 | 7 |
| 18 Race circuit | 2 | -2 | 1 | 1 | -1 | 3 | 2 | 0 | -1 | 1 | 0 | 1 | -1 | 7 |
| 16 Good Sea | 0 | 0 | 0 | 1 | -2 | -2 | -2 | 0 | -2 | 0 | 0 | 2 | 2 | 9 |
| 49 Zoop | 0 | 1 | 0 | 0 | -2 | 1 | -2 | 2 | -1 | -1 | 0 | 0 | 0 | 8 |
| 36 Submarine base | 0 | 0 | 0 | 0 | -1 | -1 | -2 | 0 | -1 | 0 | 0 | -1 | -2 | 8 |

---

**Figure C.1:** The results of the criteria analysis
Appendix D

Exploratory geotechnical calculations

In this exploratory analysis basic calculations are made on situations in the construction phase and the users phase, for the retaining structure alternative as elaborated in Section 5.3. By means of this analysis a first insight can be gained on the behavior of the dredged material with respect to bearing capacity, slope stability and settlement. The bearing capacity will be tested by hand calculations and verified by calculations in MStab. No hand calculations will be done for slope stability analysis, since these calculations are very toilsome, but the stability will be tested by calculations in MStab. The expected settlement will also be tested by hand calculations as well as calculations in MSettle.

The basic scenario simulated here is the one where a piece of land bounded by an embankment is erected from the top of the dredged material layer up to a desired level. First, a layer of sand is sprayed over the Slufter bed from a pontoon to provide a layer with sufficient bearing capacity to resist the load of the vertical drainage installation equipment. After vertical drainage has been installed, an extra sand layer can be added to function as overburden to accelerate the consolidation process, discharge of water and settlement. After some time the soil structure may be elevated further to reach the desired top level. Finally, the obtained soil structure has to be able to bear the Slufter’s future functions, according to the requirements posed in Section 5.2.

In the calculations the following assumptions are made:

1. The bed level of the dredged material is horizontal
2. The calculations are done using the parameters listed in Section 5.1
3. The dredged material is homogeneous and isotropic
4. Only static loads are considered

D.1 Construction phase

The construction phase can be subdivided into three different phases. In Phase 1 thin layers of sand are sprayed on top of the dredged material in a staged construction. For this phase
will be determined whether an uneven distribution of sand by the spraying poses any risk to instabilities, and the required thickness of the sprayed sand layer which has to withstand the load of the equipment will be calculated.

In case Phase 1 is successful, the water sheet is removed in Phase 2 and a system of vertical drains is applied. The influence of vertical drains on the consolidation process will be determined and compared to a situation without drains.

In Phase 3 the area is elevated by subsequent surcharges of sand to reach the desired crest level of the embankment.

D.1.1 Phase 1: Spraying of sand

Analysis of the risk of uneven spraying

The spraying pontoon can deposit sand on the bed with reasonable accuracy. However, the first few centimeters of a new layer are always critical, because this causes uneven loads on the subsoil. Therefore, it has to be determined whether there is sufficient bearing capacity to withstand the load of a layer of sand with a thickness of 10cm. This layer thickness corresponds to the accuracy that can be guaranteed by the contractor.

The bearing capacity can be calculated with the formula of Brinch Hansen. For a theoretical background on this method is referred to Verruijt’s book on soil mechanics [32] or Appendix F.1. Immediately after the applied load the situation is undrained and most critical. In that case the dredged material has no share of the angle of internal friction ($\phi'=0$), and the formula of Brinch Hansen reduces to the solution of Prandtl:

$$ p = c_u N_c + q $$

where $p$ represents the failure pressure, $c_u$ is the undrained shear strength of the dredged material and $q$ is the added load next to the loaded strip, which contributes to the bearing capacity. $N_c$ is a dimensionless constant for which Prandtl found $N_c = \pi/2$ if $\phi = 0$ [32]. This value represents the circular shape of the failure mechanism which is used to describe the principle of bearing capacity. It has been derived from the equilibrium of stresses working on a body of soil.

Suppose a first strip of sand is sprayed on top of the dredged material with a thickness of 10cm, as schematized in Figure D.1. The slip surface shows the way in which failure might take place as a result of the sand induced load. The failure pressure then becomes:

$$ p = 1 \times (\pi/2) = 5.14kPa $$

where the value $c_u = 1$ is derived from Table 5.1. The sand has a weight under water of 10kN/m$^3$. The load then becomes:

$$ s = 0.1 \times 10 = 1kPa $$

resulting in a SF of approximately 5 and thus there is sufficient bearing capacity. It can be concluded that for the first layer a less strict accuracy can be allowed. An accuracy of 25cm, which results in an SF of 2, should also be sufficiently safe.
D.1. CONSTRUCTION PHASE

Now suppose the first 10cm thick layer of sand is covered by a second layer of 10cm over a distance of 10m, as schematized in Figure D.2. All along the slip surface the undrained shear strength contributes to the strength.

The strength $R$ and the load $S$ are now calculated in kN per running meter, according to:

$$R = p \times B \quad \text{and} \quad S = s \times B$$

The strength then becomes:

$$R = (1 \times (\pi + 2) + 10 \times 0.1) \times 10 = 61.4 \text{kN/m}'$$

The load includes the thickness of the first two layers together and thus becomes:

$$S = 10 \times 0.2 \times 10 = 20 \text{kN/m}'$$

resulting in a SF of 3, so there is sufficient bearing capacity here as well. An accuracy of 15cm may also suffice, since that results in a SF of 2.

Determination of the required thickness of the sprayed layer

The dredged material is covered by a layer of sand. After the water sheet has been removed, the soil structure has to be accessible to the vertical drainage installation equipment (see Figure D.3, the excavator is illustrative). The load imposed by the equipment determines the required thickness of the sprayed sand layer.

Here, a CAT345 is chosen as the representative equipment. For the bearing capacity the dimensions of the caterpillar tracks are normative. The tracks have a length on the ground of 4.36m and a width of 0.75m. The weight of the caterpillar itself is approximately 50,000kg and the weight of the installation crane is another 10,000kg. This weight is carried by the two tracks, resulting in a distributed load of:

---

Figure D.1: Schematization of the load of a strip of sand sprayed on top of the dredged material, and the failure mechanism

---
Figure D.2: Schematization of the load of a second strip of sand sprayed on top of the first 10cm, and the failure mechanism

\[
\frac{60,000 \times 10}{2 \times 4.36 \times 0.75 \times 1000} = 92 \text{kN/m}^2
\]

The thickness of the dredged material layer is 30m, the thickness of the sand layer determines the required bearing capacity and has to be calculated. Due to transport of water from the dredged material layer to the sand layer (as a result of the consolidation process), the sand layer is assumed to be fully saturated. The volumetric weight of the wet sand layer \(\gamma_{s;\text{sat}} = 20 \text{kN/m}^3\).

The schematization in Figure D.3 shows the failure mechanism for the load situation. The sand layer contributes to the bearing capacity by increasing the characteristic influence area of the load and thereby the radius of the slip circle. The spread through the sand layer can be approximated by 8° on either side, according to NEN6740 [21].

This results in the following equation for the bearing capacity (or strength):

\[
R = p \times B = (1 \times (\pi + 2) + 20\Delta h) \times (0.75 + 2\Delta h \tan 8^\circ) = 5.62\Delta h^2 + 16.44\Delta h + 3.9 \quad [\text{kN/m}^2]
\]

The load consists of the weight of the equipment and the weight of the soil beneath the equipment. The equation for the load \(S\) becomes:

\[
S = s \times B = (92 + 20\Delta h) \times 0.75 \quad [\text{kN/m}^2]
\]

Now, by stating \(R = S\) one acquires the minimum value for the height of the sand layer:

\(\Delta h = 3.3m\)
This is quite a thick layer to be sprayed, which takes time and increases the risk of stability problems. Moreover, the stability factor $SF$ in this case of course is 1, however, a higher $SF$ is required. This would result in an ever thicker layer. Another problem is that if a thicker sand layer has to be penetrated by the crane to install the drains, also heavier equipment is needed, resulting in higher loads and again in an even thicker sand layer.

It is also possible to install the vertical drains from a pontoon, however this method is more expensive.

The above first approximation does not take the shear stress through the sand layer into account, which has a positive effect on the bearing capacity. Therefore the situation will be simulated in MStab to see whether the thickness of the sprayed layer can be smaller.

**Determination of the required layer thickness using MSettle and MStab**  

The first step is define the geometry of the soil structure in MStab and the relevant soil material properties of the different layers. The basic soil structure is the layer of dredged material with a thickness of 30m, which is on top of the pleistocene sand layer. The slope stability calculations made with MStab are done according to the method of Bishop (see [32] and Appendix F.2). The calculation model for the settlement selected in MSettle is the 2-dimensional NEN-Koppejan model with the option to include vertical drains. For the consolidation in MSettle the model of Terzaghi is used.

In the calculation options of MSettle the top of each layer is selected ‘drained’ and the bottom of each layer ‘undrained’, so that transport of water only takes place in upward flow. This is in accordance with the actual situation at the bottom of the dredged material layer. Here, the interface between the dredged material and the pleistocene sand has become well-nigh impermeable as a result of the fine clay and silt particles that have filled up the pores in the sand.

Table D.1 gives an overview of the chosen input parameters.
APPENDIX D. EXPLORATORY GEOTECHNICAL CALCULATIONS

Table D.1: Input parameters for the calculations in MSettle and MStab

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Dense sand</th>
<th>Dredged material</th>
<th>Loose sand</th>
</tr>
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<tbody>
<tr>
<td>$\gamma$</td>
<td>[kN/m$^3$]</td>
<td>19</td>
<td>13</td>
<td>17</td>
</tr>
<tr>
<td>$\gamma_{sat}$</td>
<td>[kN/m$^3$]</td>
<td>21</td>
<td>13</td>
<td>19</td>
</tr>
<tr>
<td>$c_u$ top</td>
<td>[kPa]</td>
<td>-</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>$c_u$ bottom</td>
<td>[kPa]</td>
<td>-</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>$c'$</td>
<td>[kPa]</td>
<td>0</td>
<td>2.5 (drained)</td>
<td>0</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>[$^\circ$]</td>
<td>35</td>
<td>25 (drained)</td>
<td>30</td>
</tr>
<tr>
<td>$c_v$</td>
<td>[m$^2$/s]</td>
<td>n.a.</td>
<td>1.26 * 10$^{-7}$</td>
<td>n.a.</td>
</tr>
<tr>
<td>$C_p$</td>
<td>[-]</td>
<td>$\infty$</td>
<td>20.1</td>
<td>30</td>
</tr>
<tr>
<td>$C_s$</td>
<td>[-]</td>
<td>$\infty$</td>
<td>481</td>
<td>1000</td>
</tr>
</tbody>
</table>

The values of the dense sand belong to the pleistocene layer beneath the Slufter basin. The compression constants have been chosen infinitely large, because it can be assumed that no settlement will take place in this layer. The sand layers have an effective angle of internal friction $\phi'$ and the effective cohesion $c'$ is zero. For the dredged material, in the undrained situation the undrained shear strength is normative for the stability calculations and after consolidation the situation is drained and the given values for $c'$ and $\phi'$ can be used.

The dredged material is loaded by a layer of sand. First, it is determined how much sand can be sprayed on top of the dredged material instantaneously (i.e. in thin layers at a time, but in one construction phase). In this situation, the sand is initially carried by the pore water pressure. Therefore, only the undrained shear strength contributes to the strength of the dredged material. The undrained shear strength increases from the top of the dredged material at NAP +2m to the bottom at NAP -28m. Because there is uncertainty on the exact distribution of the values over the depth as well as per location in the Slufter basin, multiple distributions are possible. In an unfavorable situation $c_u=1$kPa at the top and $c_u=3$kPa at the bottom of the dredged material layer can be used as a first approximation. In case the situation is more favorable a value $c_u=1$kPa at the top and a higher value than 3 at the bottom may occur. It is assumed that the undrained shear strength increases from top to bottom linearly.

It should be noted that the above mentioned unfavorable situation as well as a more favorable situation may occur in reality, because of the variability of dredged material itself and the place where it has been discharged. It is plausible to say that heavier soil particles and sand fractions settle nearer to the discharge points. This means that the situation may be more favorable around these points and more unfavorable further away. However, large quantities have also been deposited in the central parts by means of a floating pipeline. In case further studies are done on the feasibility of this alternative, cone penetration tests (CPT’s) have to be carried out to map the actual conditions over the depth and per location. For now, the values for the undrained shear strength in the unfavorable situation are used as a starting basis.

Two situations have been simulated; one where a layer of 0.5m and one where a layer of 1m is sprayed on top of the dredged material instantaneously. The elevation has slopes of 1:10, which is the gentlest slope the contractor can deliver intentionally. The results of the calculation are presented in Table D.3.
Table D.2: Stability factors for instantaneous elevations

<table>
<thead>
<tr>
<th>Elevation [m]</th>
<th>SF</th>
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<tbody>
<tr>
<td>0.5</td>
<td>2.03</td>
</tr>
<tr>
<td>1</td>
<td>0.87</td>
</tr>
</tbody>
</table>

From these results can be concluded that a first layer of sand with a thickness of 0.5m can be sprayed, but a layer of 1m is already insufficiently safe. The alternative is to try and further elevate the structure in a staged construction, i.e. layer by layer elevation with predefined time intervals. Suppose a first layer of 0.5m is sprayed and after 100 days a second layer of 0.5m is sprayed on top of the first layer.

The settled geometry from MSettle can be imported into MStab and used to calculate the stability immediately after placement of the second layer. This results in a SF of 0.97 (see Figure D.4). The reason for the low stability factor is that hardly any consolidation of the dredged material takes place as a result of this load.

**Figure D.4:** The critical slip circle for a staged elevation of 0.5m every 100 days up to a level of 1m. The green rectangles along the slip circle represent the magnitude of the shear stress per slice (see also Appendix F). The stability factor can be seen at the bottom of the figure, as well as the maximum shear stress. In this case the maximum stress is 1.56kN/m², which is small.

It is possible to improve the safety of the elevations by extending the sub-lying sand layers, so that more slip circles intersect these layers twice and therefore have a larger contribution of the shear stresses. The structure will in that case be built up stepwise, for instance with time intervals of 100 days. This has been simulated for a staged construction of the first two layers of 0.5m each and horizontal interspaces of 10m, 15m and 20m. The results for the lowest stability factors found are presented in Table D.3.
Figure D.5: The critical slip circle for a staged elevation of 0.5m every 100 days up to a level of 1m, with 20m interspace every step

Table D.3: Stability factors for a staged construction up to 1m

<table>
<thead>
<tr>
<th>Extension [m]</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1.17</td>
</tr>
<tr>
<td>15</td>
<td>1.20</td>
</tr>
<tr>
<td>20</td>
<td>1.35</td>
</tr>
</tbody>
</table>

The results show that the interspaces do improve the situation. An interspace of at least 20m is needed to significantly improve the safety. However, the moment the water level is lowered, the structure fails. For example, if the water level is lowered to 0.5m above the dredged material, the SF becomes 0.87. This is the result of a sudden increase in the weight of the soil, because it is no longer submerged. In addition, it is likely that a thicker layer than 1m will be needed to allow access to the equipment.

Therefore a variety of simulations has been done, to see whether it is possible to obtain a layer of reasonable thickness with sufficient safety. A staged construction is simulated with subsequent layers of 0.5m every 100 days, up to a level of 2m. The structure is built up with slopes of 1:10 and interspaces of 30 or 60m. For each simulation the slope stability is checked in case the water level is lowered from NAP +6m to NAP +3m or NAP +3.5m. The geometry and results of the stability analysis are summarized in Figure D.6.

It is possible to realize a stable situation when the interspaces are 60m wide and the water level is lowered to NAP +3.5m. Unfortunately, the situation becomes unstable again when the load of the vertical drainage equipment is added. The equipment must be able to come near to the slopes, but the consequence is that the system will fail.
From the above analysis can be concluded that it is very difficult to obtain a soil structure that is stable by mere spraying of sand and waiting, let alone making the terrain accessible to vertical drainage installation equipment. Moreover, the simulated structure with interspaces of 60m would result in an embankment with a width on the ground of 670m. In comparison, the longest distance from one side of the Slufter’s ring-dike to the opposite side is 1.8km. An embankment with these dimensions would thus take up a very large part of the Slufter basin.

Therefore, it is recommended to install vertical drains from a pontoon. However, still the first layer of sand must be sprayed so that the vertical drains can be positioned and anchored at the top. Previously it was determined that a staged construction of 0.5m of sand sprayed every 100 days, up to a level of 1m with interspaces of 20m results in an SF of 1.35. This will form the basis of the embankment.

The subsequent construction steps to be taken and the feasibility of the soil structure in that case is analyzed with the help of MStab and MSettle in Phase 3 below.

The following continues on the construction phase, unregarded of the method for the installation of the vertical drains.

### D.1.2 Phase 2: Installation of vertical drains

Analysis of the effect of vertical drains A layer of sand has been sprayed on top of the dredged material bed and the water sheet has been removed. Either vertical drainage has to be installed or the area has to be elevated to enhance consolidation without drains. First, the situation is considered where no special technique is applied and consolidation has to take place under the influence of a surcharge of sand on top of the dredged material. This will be done by determining the degree of consolidation and the settlement.
APPENDIX D. EXPLORATORY GEOTECHNICAL CALCULATIONS

In the Netherlands a commonly used formula to derive the settlement is Koppejan for the strain:

\[ \varepsilon = U \left[ \frac{1}{C_p} + \frac{1}{C_s} \log \left( \frac{t}{t_1} \right) \right] \ln \left( \frac{\sigma}{\sigma_1} \right) \]  

(D.2)

where \( U \) is the degree of consolidation [%], \( C_p \) and \( C_s \) are the primary and secondary compression constants [-], respectively, \( t \) is the loading time in days and \( t_1 \) is the reference time (\( t_1 = 1 \) day). \( \sigma_1 \) is the initial effective stress [kPa] and \( \sigma \) is the new effective stress under load conditions. The degree of consolidation can be approximated by:

\[ U \approx \frac{2}{\sqrt{\pi}} \sqrt{\frac{c_v t}{h^2}} \text{ if } U < 0.5 \]  

(D.3)

\[ U \approx 1 - \frac{8}{\pi^2} \exp \left( -\frac{\pi^2 c_v t}{4 h^2} \right) \text{ if } U > 0.5 \]  

(D.4)

where \( c_v \) is the consolidation coefficient \([\text{m}^2/\text{s}]\), \( t \) is the consolidation time in seconds and \( h \) is the layer thickness over which consolidation has to take place (or the consolidation length). The actual settlement can be estimated by multiplying the layer thickness with the strain:

\[ \Delta z = \varepsilon h \]  

(D.5)

For a more comprehensive explanation of these equations is referred to [32] or Appendix F.3.

Suppose the area is elevated with 10m of sand from the top of the dredged material layer and the consolidation time is 1000 days (which corresponds to 2.7 years). Using the equations above and the input parameters in Table 5.1, the settlement is approximately 20cm. The degree of consolidation after that time is only 12%.

The time required to arrive at a degree of consolidation of 99% can be approximated by:

\[ t_{99\%} \approx \frac{1.8h^2}{c_v} \]  

(D.6)

With a height of 30m and \( c_v = 1.26 \times 10^{-7} \text{m}^2/\text{s} \) the consolidation time is approximately 149,000 days (or 408 years). The corresponding settlement is 1.7m. It is obvious that under these circumstances it will take far too long for the geotechnical properties to improve. Therefore the consolidation process will be accelerated by installing vertical drains.

The influence of vertical drains is translated to the strain and settlement by adding a characteristic value of the degree of consolidation to Equation (D.2), given by:

\[ \overline{U}_h = 1 - e^{-\frac{8T_h}{\pi}} \]  

(D.7)

where \( \overline{U}_h \) is the average degree of consolidation for saturated soil. The factor \( \mu \) and the time factor \( T_h \) are given by:

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\[
\mu = \frac{n^2}{n^2 - 1} \left[ \ln(n) - \frac{3}{4} + \frac{1}{n^2} \left(1 - \frac{1}{4n^2}\right) \right]
\]

(D.8)

\[
T_h = \frac{c_h t}{D^2}
\]

(D.9)

c\textsubscript{h} is the horizontal consolidation coefficient and \( D \) is the equivalent diameter of the soil cylinder that each vertical drain has to de-water. The factor \( n \) equals \( D/d \) where \( d \) is given by:

\[
d = \frac{2(b + t)}{\pi}
\]

(D.10)

where \( b \) and \( t \) are the width and thickness of the drain, respectively.

By means of these equations the settlement in time has been derived for various load conditions, with the following input parameters:

- The horizontal consolidation coefficient, assumed to be equal to the vertical consolidation coefficient
- A center to center distance of the drains of \( D = 2m \)
- A width and thickness of the drain; \( b = 0.3m \) and \( t = 0.004m \)

The results are depicted in Figure D.7, showing the settlement in time as a result of an elevation of 5, 10 and 15m. The progress of the degree of consolidation in time, which is equal for all load conditions in combination with vertical drains, is also shown. In the figure can be seen that 99% of the consolidation has taken place after somewhere between 500 to 1000 days. For comparison, the settlement is shown for a situation where no drains are applied and the surcharge is 15m of sand. The difference is significant.

**Influence of the installation depth and center-to-center distance of vertical drains, in MSettle**

The choice of the installation depth and center-to-center distance of a system of vertical drains is a trade-off between minimizing costs and optimizing structural safety. The degree of consolidation is proportional to the structural safety, because it signifies which percentage of the load is carried by the effective stress and which percentage is still carried by the pore water pressure. A higher degree of consolidation means more stability. In MSettle, vertical drains can be included in the simulations. Figures D.8 and D.9 show the influence of the installation depth and center-to-center distance on the degree of consolidation, respectively.

The simulation of the installation depth has been done with a constant center-to-center distance of 2m, and the simulation of the center-to-center distance with a constant installation depth of NAP -27m. The dark blue lines in the graph are therefore the same. From the figures can be derived that altering the installation depth to NAP -20m or the center-to-center distance to 2.5m almost has the same effect. An installation depth of NAP -10m results in a significantly lower progress of the degree of consolidation. In general, the consequence is that one has to wait longer in order to obtain the same effect. In that case it depends on the available time, the cost reduction and the structural safety. The last part of the construction phase deals with these issues.
Figure D.7: The progress of the settlement in time under various load conditions, and the degree of consolidation

Figure D.8: The influence of the installation depth, for three different depths
Two layers of 0.5m have been sprayed with interspaces of 20m and a system of vertical drains is installed from a pontoon. The installation depth of the vertical drains is NAP -27m and the center to center distance is 2m. A period of 100 days of waiting is allowed for the consolidation process. According to the simulation in MSettle, the degree of consolidation is 73%. The condition of the dredged material now changes from undrained to drained, and the values of $c'$ and $\phi'$ in Table D.1 can be used in MStab.

As a result of the consolidation process the slope stability of the submerged soil structure increases considerably, from SF=1.35 in the undrained situation to SF=5.16 in the drained situation. When the water level is lowered to just above the original level of dredged material, at $\sim$ NAP+2.1m, the slope is still stable, with an SF of 3.63. Now the obtained area can be accessed by construction equipment.

Suppose a similar vehicle is used for the further elevation of the area, but this time an excavator. The normative load condition is given by the vertical force equilibrium of the weight of the vehicle and the weight of a filled bucket at maximum reach. If the weight of the empty bucket is 1760kg and the capacity of the bucket is 3.8m$^3$, then its vertical force is 94kN. The excavator weighs 50tons or 500kN. In the limit condition the vertical load of the vehicle and the load of the filled bucket is carried by solely one track. This is the case when the bucket is at a distance of 7.45m from the vehicle. From the vertical force equilibrium results that the load on the track is 594kN. This is distributed over a track length of 4.36m and a width of 0.75m. The representative distributed load is thus 182kN/m$^2$. In Figure D.10 the simulation of the load condition is presented. It shows that the structure fails on bearing capacity. Two practical options for solving this problem are either to wait longer and to allow for a higher degree of consolidation, or to apply lighter equipment for the first few sand surcharge layers.
D.1.3 Phase 3: Finalization of the embankment

In the previous phase, the vertical drains were installed and their effect on the consolidation process was analyzed and compared to a situation without drains. Now, it is assumed that the embankment has to be built up to a level of 15m above the layer of dredged material, in accordance with the required elevation found in Section 4.4 of the spatial analysis. The construction is done by elevating the area by surcharges of sand. An analysis in MStab and MSettle will have to prove whether the realization of an embankment is feasible.

Analysis of the realization of the embankment in MStab and MSettle When the structure is sufficiently safe for the equipment, the area can be elevated further. In MSettle and MStab this is simulated in a staged construction, where every 100 days a surcharge of sand of approximately 5m is placed, at a slope of 1:10. After three elevations and a superelevation, which compensates for the settlement, the embankment is finished. If the execution is done according to these steps, the SF for the slope in the final situation is 1.32 (see Figure D.11).

If the structure is safe at the moment of completion than it will also be safe in the users phase, because the stability factor will increase over time as settlement continues.

All the construction steps are once more summarized in Table D.4.

The influence of drain depth and center-to-center distance to the structural safety In the previous paragraphs the geotechnical feasibility of an embankment in the Slufter has been analyzed and proven. The geotechnical properties of the dredged material were improved by means of a system of vertical drains with an installation depth at NAP -27m and a
D.1. CONSTRUCTION PHASE

Figure D.11: Completion of the embankment

Table D.4: Summary of the phases and stability factors for the execution of an embankment in the Slufter

<table>
<thead>
<tr>
<th>Phase Description</th>
<th>t = 0, 0.5m spraying</th>
<th>SF = 2.03</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Wet phase, undrained</td>
<td>t = 100 days, 0.5m spraying</td>
<td>SF = 1.35</td>
</tr>
<tr>
<td></td>
<td>t = 200 days, installation of vertical drains</td>
<td></td>
</tr>
<tr>
<td></td>
<td>t = 300 days, U = 73%</td>
<td></td>
</tr>
<tr>
<td>2. Wet phase, drained</td>
<td>Slope stability submerged</td>
<td>SF = 5.16</td>
</tr>
<tr>
<td>3. Dry phase, lowered water level</td>
<td>Slope stability if water level at ∼NAP+2.1m</td>
<td>SF = 3.63</td>
</tr>
<tr>
<td>4. Dry phase, loaded by equipment</td>
<td>Bearing capacity under loading</td>
<td>SF = 1.18</td>
</tr>
<tr>
<td>5. Dry phase, completion</td>
<td>Slope stability completed embankment</td>
<td>SF = 1.32</td>
</tr>
</tbody>
</table>

It might be possible to reduce the drain depth and/or the center-to-center distance and still obtain a safe structure. This will reduce the installation costs considerably. Therefore, the construction phases are followed once more for three different drain depths and center-to-center distances. The results are summarized in Table D.5 and D.6. In the table the same phases are followed as explained in Table D.4.
APPENDIX D. EXPLORATORY GEOTECHNICAL CALCULATIONS

Table D.5: Comparison of the influence of drain installation depth to the structural safety (with center-to-center distance 2m)

<table>
<thead>
<tr>
<th>Depth</th>
<th>Phase</th>
<th>U</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>-27m</td>
<td>2</td>
<td>73%</td>
<td>5.16</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3.63</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.18</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>90%</td>
<td>1.32</td>
</tr>
<tr>
<td>-20m</td>
<td>2</td>
<td>54%</td>
<td>5.00</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>87%</td>
<td>1.2</td>
</tr>
<tr>
<td>-10m</td>
<td>2</td>
<td>23%</td>
<td>4.21</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>2.64</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.45</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>70%</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table D.6: Comparison of the influence of drain center-to-center distance to the structural safety (with drain depth NAP -27m)

<table>
<thead>
<tr>
<th>Distance</th>
<th>Phase</th>
<th>U</th>
<th>SF</th>
</tr>
</thead>
<tbody>
<tr>
<td>2m</td>
<td>2</td>
<td>73%</td>
<td>5.16</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3.63</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.18</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>90%</td>
<td>1.32</td>
</tr>
<tr>
<td>2.5m</td>
<td>2</td>
<td>53%</td>
<td>5.00</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3.40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>87%</td>
<td>1.2</td>
</tr>
<tr>
<td>3m</td>
<td>2</td>
<td>53%</td>
<td>4.45</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3.40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.61</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>80%</td>
<td>1.12</td>
</tr>
</tbody>
</table>

What primarily can be seen is that the stability factor decreases as a result of a smaller drain depth or larger center-to-center distance. In a staged construction with the same time frame, a drain depth of NAP -20m gives an acceptable SF of 1.2. The same goes for a drain distance of 2.5m.

A depth of NAP -10m or a distance of 3m results in an SF lower than 1.2. However, the safety can be increased if it is decided to allow for a longer waiting time between different construction phases. For drain depths of NAP -20m and NAP -10m this would imply an additional waiting time of 230 and 500 days, respectively, to obtain the same effect. For drain distances of 2.5m and 3m an additional waiting time of 230 and 380 days is required, respectively. It therefore depends on the time frame in which the embankment has to be delivered and the period over which the investment is spread.
D.2 Users phase

In the users phase the area between the obtained embankment and the existing ring-dike will be elevated and loaded by various functions. A distinction is made in light weight, medium weight and heavy weight functions, as explained in Section 5.2. Since light weight functions pose no extra load to the structure, the calculations are limited to medium and heavy weight structures.

Three scenarios are tested; in Scenario 1 the load of an algae breeding pond is simulated, in Scenario 2 the load is a stack of eight empty containers, and in Scenario 3 the load is presented by a pile of stored dry sand. The stability of the soil structure and the settlement of the dredged material as a result of the loading are determined by means of hand calculations and in simulations in MStab and MSettle. The hand calculations for stability are restricted to an estimation of the bearing capacity, using the solution of Prandtl (Equation (F.1)). To be able to apply this method, the dredged material is assumed undrained, so that only the undrained shear strength \( c_u \) contributes to the stability. Furthermore, it is assumed that the sand layer on top of the dredged material has no contribution to the shear stress in the failure plane.

These assumptions make the hand calculations a very rough estimation, but a first indication of what will happen in reality. The scenarios will therefore also be simulated in MStab and MSettle. Here, the application of vertical drains will result in a drained situation and the dredged material will have both cohesion and an angle of internal friction. Moreover, the shear stresses in the sand layer that contribute to the stability are taken into account.

D.2.1 Scenario 1: An algae breeding pond

The load of a medium weight function. The function considered here is an algae breeding farm and the load is presented by a breeding pond (see Figure D.12). The pond is typically shaped with four adjacent rectangular pools flowing into each other at the tips like a cascade. Typical dimensions of each pool are a width of 5m, a length of 100m and a depth of 0.3m, which corresponds to a volume of 540m\(^3\) for the four pools together.

If the water density is 1000kg/m\(^3\) the calculation value of the distributed load becomes:

\[
\frac{540 \times 1000 \times 10}{4 \times 5 \times 100} = 2.7kN/m^2
\]

The aim is to determine if the underlying dredged material and sand layer provide sufficient bearing capacity to withstand this load. Therefore the schematization is similar to the one in Construction Phase 1 and the strength \( R \) determined according to the reduced formula of Brinch Hansen, Equation (D.1) (as \( \phi = 0 \)).

The feasibility of an algae breeding farm if the elevation is up to NAP +7m. Suppose an elevation up to NAP +7m is realized on top of the dredged material layer and this area provides the space for the new function. It is also assumed that the algae breeding farm will not be realized until approximately 2.7 years after the elevation of the area began. Thus, the consolidation time is 1000 days.
The first step is to determine the new value of the undrained shear strength. The new value for the undrained shear strength can be derived by means of the following empirical relationship found by Skempton [16]:

\[
\frac{c_u}{\sigma'} = 0.11 + 0.0037 I_p
\]

where \( I_p \) is the average Plasticity Index. With an average Plasticity Index of 24% the undrained shear strength can be written as:

\[
c_u = 0.2\sigma'_v
\]

With the original dredged material level at NAP +2m an elevation height of 6.2m results in a settlement of 1.2m after 1000 days and a 100% degree of consolidation. The final height is thus at NAP +7m.

The width of the algae breeding farm is 20m. The characteristic width beneath the sand layer therefore is 20 + 2 \times 6.2 \tan 8^\circ = 21.7m and the slip circle reaches to a depth of approximately 21.7m from the top of the dredged material. The effective stress at this depth is:

\[
\sigma' = 5.2 \times 17 + 1 \times 20 + 21.7 \times 13.5 - 22.7 \times 10 = 174.4 kPa
\]

The new value for the undrained shear strength then becomes:

\[
c_u = 0.2 \times 174.4 = 34.9 kPa
\]

With this new value for the undrained shear strength, it can be determined whether there is sufficient bearing capacity to withstand the load imposed by the algae breeding farm.

The strength becomes:

\[
R = (34.9 \times (\pi + 2) + 17 \times 5.2 + 20 \times 1) \times 21.7 = 6246 kN/m' 
\]

The load is:
\[ S = (2.7 + 17 \times 5.2 + 20 \times 1) \times 20 = 2562 \text{kN/m}' \]

This results in a stability factor:

\[ SF = 2.4 \]

which means that the dredged material layer and the sand layer provide sufficient bearing capacity.

Under these circumstances and in case the vertical drainage still functions, the uneven settlement with respect to the surrounding area as a result of the load is only 2cm after 10,000 days.

**Verification with MSettle and MStab** The elevation of the area between the embankment and the existing ring-dike has been simulated in a staged construction in MSettle and the settled geometry is imported in MStab, as shown in Figure [D.13]. For the elevation the same construction steps have been followed as for the embankment, with initially two layers of sand of 0.5m sprayed on top of the dredged material, vertical drains installed from a pontoon and eventually, after lowering the water level, elevation up to NAP+7m. Another execution method might also be possible for this area.

![Figure D.13: Creating land between the embankment and the existing ring-dike](image)

In the users phase the load of an algae breeding pond is simulated in MStab. The stability factor is determined in the most unfavorable situation, when the load is applied instantaneously and is first carried by the excess pore pressures. The dredged material layer therefore behaves undrained. The result is presented in Figure [D.14]. As can be seen, with a stability
factor of 10.1 there is absolutely no risk of failure of the soil structure. In addition, there is no settlement as a result of this load. The stability will only increase over time, as the pore pressures decrease under influence of consolidation.

From this analysis can be concluded that light-weight structures such as algae breeding ponds, solar energy panels, fish breeding ponds or greenhouses can safely be applied on land created on top of the Slufter’s dredged material.

D.2.2 Scenario 2: An empty depot

The load of a heavy weight function. In this scenario an empty depot function is considered where the load is presented by the weight of 8 empty containers stacked on top of each other, as shown in Figure D.15.

A 40ft TEU has a length of 12.192m, a width of 2.438m and a height of 2.591m. The weight of one TEU is 3800kg. The calculation value for the load thus becomes:

$$\frac{3800 \times 10}{12.192 \times 2.438} = 1.28kN/m^2$$

The feasibility of an empty container terminal if the elevation is up to NAP +7m Similar to Scenario 1, the area is elevated up to a level of NAP +7m. First, the representative value of the undrained shear strength has to be determined for this situation. The characteristic
width below the sand layer and the radius of the slip circle is $2.438 + 2 \times 6.2 \tan 8^\circ = 4.2\text{m}$. The effective stress at this depth is then:

$$\sigma' = 5.2 \times 17 + 1 \times 20 + 4.2 \times 13.5 - 5.2 \times 10 = 113kPa$$

resulting in the new value of the undrained shear strength:

$$c_u = 0.2 \times 113 = 22.6kPa$$

Following the same approach the strength becomes:

$$R = (22.6 \times (\pi + 2) + 17 \times 5.2 + 20 \times 1) \times 4.2 = 943kN/m'$$

And the load is:

$$S = (1.28 \times 8 + 17 \times 5.2 + 20 \times 1) \times 2.438 = 289kN/m'$$

Leading to a stability factor $SF = 3.2$, which means that there is sufficient bearing capacity for one stack of eight containers. However, in reality a row of ten containers may be placed next to each other. The calculation above is repeated for this situation. The characteristic width below the sand layer and the radius of the slip circle is $2.438 \times 10 + 2 \times 6.2 \tan 8^\circ = 26.1\text{m}$.

The effective stress at this depth is:

$$\sigma' = 5.2 \times 17 + 1 \times 20 + 26.1 \times 13.5 - 27.1 \times 10 = 190kPa$$

and the undrained shear strength:

$$c_u = 0.2 \times 190 = 38kPa$$

The strength then becomes:

$$R = (38 \times (\pi + 2) + 17 \times 5.2 + 20 \times 1) \times 26.1 = 7935kN/m'$$

---

**Figure D.15:** Scenario 2
and the load is:

\[ S = (1.28 \times 8 + 17 \times 5.2 + 20 \times 1) \times 2.438 \times 10 = 2892kN/m' \]

The resulting stability factor is 2.7, so the safety is somewhat smaller than with a single row of containers, but there is still sufficient bearing capacity.

The uneven settlement with respect to the surrounding area as a result of a stack of 8 containers after 10,000 days is approximately 8cm, which is acceptable.

Verification with Msettle and MStab

The load case of a row of 10 empties stacked up 8 high is simulated in MSettle and MStab, as shown in Figure D.16.

![Figure D.16: The critical slip circle for the load case of a row of 10 empties stacked up 8 high](image)

In the simulation, the load of the empty containers is applied instantaneously and therefore unfavorable. In this situation, the dredged material layer behaves undrained and the load is first carried by the excess pore pressures. The stability factor is 3.3, so the soil structure is able to bear the load of an empty depot. Over time, the stability factor will increase as a result of consolidation.

The stability factor is in the same order as what was found in the hand calculations. However, in this case it is shown that the soil structure derives the largest part of its strength from the sand layers’ shear resistance. This underlines the importance of the contribution of the sand layer to the stability of the structure. As mentioned in the beginning of this section, the hand calculations are very conservative.
According to MSettle, the residual settlement as a result of the load of the row of stacked empties is only 3.5cm.

**D.2.3 Scenario 3: Temporary storage of soil**

The load of a heavy weight function. Here a function is considered where the load is presented by the storage of dry soil (such as TOP Europoort). Imagine a pile of sand with on the ground a width of 24m, a length of 100m and a height of 8m. The pile is schematized as if it has a perfectly triangular shape (see Figure D.17). The value for the load per running meter is:

\[ 0.5 \times 24 \times 8 \times 17 = 1632 \text{kN/m}' \]

![Figure D.17: Scenario 3](image)

The feasibility of dry soil storage if the elevation is up to NAP +7m. Similar to Scenarios 1 and 2, the area is elevated up to a height of NAP +7m, which corresponds to a surcharge of 6.2m and a settlement of 1.2m. Again, the first step is to determine the value of the undrained shear strength for this load condition. The characteristic width below the sand layer and the radius of the slip circle through the dredged material is \( 24 + 2 \times 6.2 \tan 8^\circ = 25.7 \text{m} \). The effective stress at this point thus becomes:

\[ \sigma' = 5.2 \times 17 + 1 \times 20 + 25.7 \times 13.5 - 26.7 \times 10 = 188.4 \text{kPa} \]

and the new value of the shear strength is:

\[ c_u = 0.2 \times 188.4 = 37.7 \text{kPa} \]

Again, following the same approach the strength becomes:

\[ R = (37.7 \times (\pi + 2) + 17 \times 5.2 + 20 \times 1) \times 25.7 = 7763.6 \text{kN/m}' \]

The load becomes:

\[ S = 1632 + (17 \times 5.2 + 20 \times 1) \times 24 = 4233.6 \text{kN/m}' \]

And the stability factor becomes \( SF = 1.8 \) which means that the situation is stable, but the stability factor is lower than 2.
With a calculation value of the distributed load of 128kN/m², the uneven settlement with respect to the surrounding area as a result of the pile of sand after 10,000 days becomes approximately 80cm, which is unacceptable.

Verification with MSettle and MStab The load case has been simulated in MStab. The critical slip circle is presented in Figure D.18.

![Critical slip circle](image)

**Figure D.18:** The critical slip circle for the load case of the storage of a pile of dry soil

A stability factor of 0.56 is found, which shows that storing a pile of sand with a height of 8m and a width of 24m in the area leads to instabilities. The load in this simulation has been applied instantaneously. Therefore, its weight is first carried by the excess pore pressures. In that case, the shear resistance of the dredged material layer is only provided by the undrained shear strength.

The analysis shows that heavy weight functions such as TOP can not be accommodated at the Slufter immediately after the elevation of the area. The practical solution is to pre-load the area by another layer of sand, which has the same weight as the intended function. Thereby, consolidation is enhanced and the geotechnical properties of the dredged material will improve. The pre-load will have to be removed before applying the function.

Simulation in MSettle shows that the residual settlement as a result of this load is 80cm (77cm after 1000 days). Extra measures would have to be taken in order to prevent this. A layer of asphalt or concrete on top of the sand layer can distribute the load over a larger area.
Appendix E

Explanation of the used soil parameters

The last extensive geotechnical research of the Slufter’s soil conditions dates from 1997 and was done by the engineering office of Gemeentewerken Rotterdam. The reason for the research was to estimate the remaining duration of the Slufter’s use. In order to make this prognosis an extensive investigation was done on the consolidation process of the stored dredged material. The investigation included various field tests and laboratory experiments on samples to derive the soil characteristics and parameters required for the prognosis calculations. A similar research was done in 1993.

Since the research was done in 1997, more than 13 years have passed with consecutive storage of more dredged material. This has changed the fill pattern and has probably changed the dredged material characteristics to some extent. However, with a lack of recent data, the current condition of the Slufter’s soil has to be based on the findings in these reports and the current bottom profile.

The most important parameters derived from former research and required for the geotechnical research in this thesis will be discussed here.

E.1 Density

In the research of 1997 the mixture density was measured at 16 different locations within the Slufter basin. The measurements show varying densities, depending on location, depth and measuring method. The variations can be caused by the varying mixture densities of different bulk transferred to the Slufter and/or as a result of the consolidation and settlement process. In general, the density tends to increase over depth. The measurements were carried out in two ways: locally by means of backscatter and in the laboratory by weighing the samples obtained from the slibsampler. In Figure E.1, examples are given of the density profiles at a measuring point in the southern part and the northern part of the basin, for both the investigations of 1993 and 1997.

Although there is a relatively large spread in the values of the density over depth, on average,
APPENDIX E. EXPLANATION OF THE USED SOIL PARAMETERS

Figure E.1: Density profiles. Top left, measurements North 1993; Top right, measurements South 1993; Bottom left, measurements North 1997; Bottom right, measurements South 1997.

Figure E.2: Density profiles. Left, backscatter measurements North 1997; Right, backscatter measurements South 1997.

the density increases with depth. It can also be seen that the values obtained with the slibsampler are consistently lower than the backscatter results. The difference is in the order of approximately 100kg/m³. Why this difference occurs is unclear, as well as which measurement technique is most accurate. The report recommends further investigation on the matter, however, in 1993 it was suggested that the slibsampler takes the presence of gas
E.1. DENSITY

Figure E.3: Extrapolation of the density profile in order to obtain an estimation for the current situation into account causing the lower values of the density.

On average when following the lines in these graphs three general layers can be discerned. The top layer shows the constant density of the water sheet, in reference [18] taken to be a value of 1005 kg/m$^3$ for the calculations. Then the water sheet reaches a transition zone where the density increases up to the bed level. This transition layer has a thickness of approximately 1m and consists of a soil/water mixture with light particles in suspension. Below the transition zone the higher density of the settled soil marks the point that can be recognized as the bed level. From the top of the settled soil layer to the deeper parts, the density increases gradually. Figure E.2 shows a similar pattern in a compilation of backscatter measurements carried out four times in the period from 1993 to 1997, for both the northern and southern location. In 1993, the average bed level was at NAP -7m and in 1997 at NAP -2m. In 2010, the bed level was at approximately NAP +2m. So in the first 6 years of operation dredged material was stored to a height of approximately 21m, in the following four years another 5m and in the past four years 4m.

In Figure E.2 a new line has been drawn that averages the density over the depth per layer. Since the density profile shows a similar pattern after years of raising the bed level, this pattern can be extrapolated to the current situation, as done in Figure E.3. The averaged 1997 profile is simply shifted 4m upward. The dredged material thus has a density of approximately 1300 kg/m$^3$ at the top layer increasing to approximately 1400 kg/m$^3$ in the deeper layers.

The solid density $\rho_k$ of the clayey dredged material present in the Slufter is 2500 kg/m$^3$. 

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E.2 The undrained shear strength

The shear strength is a measure for the strength of the soil before failure takes place under influence of a load. A body of soil may collapse when the maximum shear stress is reached. In situations where no drainage and consolidation has taken place one refers to the undrained shear strength. In the 1997 research the measured shear strength is assumed to be undrained, because in the Slufter hardly any consolidation takes place.

The shear strength has been determined in various ways, one of which is by means of the in-situ vane test. The other tests were done in the laboratory. Figure E.4 shows the results from the tests and measurements by plotting the wet density against the shear stress. The vane-test shows a slight increase of the shear stress with increasing density, with values varying from 1 to 10 kPa.

![Figure E.4: The undrained shear strength versus the wet density measured with various techniques](image)

Figure E.4 shows the static and dynamic shear stresses over the depth, measured at the same northern and southern locations as the density discussed previously. In the light of this research, the static shear strengths are the most interesting, because they describe the undisturbed soil condition. This eventually determines the soil layer's response to static loading.

The shear stress profiles have also been determined at the 14 other measuring point in the Slufter basin. Although the values show large deviations, in general, the undrained shear strength varies from 1 to 5 kPa. Relatively high values may be caused by the occasional presence of sand fractions.
Shear strengths of 1-5kPa are considered small, compared to a layer of sand. This indicates that the initial bearing capacity of the dredged material is also small.

For the analysis of strength and stability of the soil the top layers are normative. The current condition of the top layer will not be very different from the condition of the top layer in 1997. Therefore, the assumption is made that the values for the shear strength found in 1997 apply to the present-day situation and are in the order of 1-5kPa.

**E.3 Compression parameters \( C_p \) & \( C_s \)**

In order to describe the compression of a soil layer under loading the formula of Koppejan for the strain is used:

\[
\varepsilon = U \left( \frac{1}{C_p} + \frac{t}{C_s} \log \left( \frac{t}{t_1} \right) \right) \ln \left( \frac{\sigma}{\sigma_1} \right)
\]

where:

- \( \varepsilon \) = the strain [-]
- \( U \) = degree of consolidation [-]
- \( C_p \) = primary compression constant [-]
- \( C_s \) = secondary compression constant [-]
- \( t \) = time [days]
- \( t_1 \) = reference time = 1 day
- \( \sigma_1 \) = initial effective stress [Pa]
- \( \sigma \) = new effective stress [Pa]

The compression constants are material characteristics and derived from compression tests in the laboratory. The primary compression constant describes the behavior of the dredged material as a result of the consolidation process. The secondary compression constant accounts for the secular effect, i.e. the phenomenon that a soil layer (especially soil with a low permeability such as clay and peat) continues to settle after the practical end of the consolidation process. It is also known by the term ‘creep’.

The compression constants are important parameters to know when describing the behavior...
of the soil under influence of an extra load. In both the researches of 1993 and 1997, these constants have been determined during compression tests in the laboratory. The two averaged values obtained from multiple tests with samples from the northern and southern measurement locations in the Slufter basin were added to the value found in 1993 and again averaged. Subsequently, this value was used for the prognosis calculations. It is assumed that these values comply with the present day material characteristics and can be used in calculations in this research. The value for $C_p = 20.1$ and for $C_s = 481$.

### E.4 The consolidation coefficient $c_v$

The consolidation coefficient indicates the speed by which consolidation takes place. Its unit in the formula of the degree of consolidation is meters squared per second [$m^2/s$]. The consolidation coefficient is proportional to the permeability and inversely proportional to the compressibility of the soil. Because the compressibility of the soil changes over time as a result of loading, the consolidation coefficient is also time dependent. However, in geotechnical design, the consolidation coefficient is often assumed constant. The value found in the 1997 soil investigation, which is also used in this research, is $1,26 \times 10^{-7} m/s^2$.

### E.5 The Plasticity Index $I_p$

The plasticity index is defined as the liquid limit of clay subtracted by the plastic limit. It is a good measure for the usability of clay as a building material. The higher the plasticity index, the better. In this research, the plasticity index is used in the formula of Skempton (see Appendix Section D.2.1). This formula is used in the exploratory calculations by hand to estimate the new value of the undrained shear strength after consolidation. The used value of the plasticity index has also been derived from the 1997 soil investigation and is 24%.
Appendix F

Theory on soil mechanics

F.1 Bearing capacity

Sufficient bearing capacity ensures that the soil layer can bear the loads applied to the soil body. To estimate the soil behavior Prandtl’s solution can be used, which determines the maximum bearing capacity of a shallow strip foundation, as schematized in Figure F.1. However, the schematization may apply to any load condition where a rectangular shaped weight is placed on the soil body.

![Figure F.1: Failure mechanism of a shallow foundation](image)

In Prandtl’s solution the soil body is divided in three regions. In Region I the horizontal normal stress is larger than the vertical normal stress and therefore has a positive share on the bearing capacity. In region III the vertical normal stress is large and equal to the load of the shallow foundation. The transition zone is shaped by region II which is bound by a logarithmic spiral. Prandtl’s solution can be described by Equation (F.1), representing the strength \( p \) of the soil body:

\[
p = cN_c + qN_q
\]

where \( c \) is the cohesion and \( q \) is the added load next to the loaded strip. \( N_c \) and \( N_q \) are dimensionless constants for which Prandtl found:

\[
N_q = \frac{1 + \sin \phi}{1 - \sin \phi} \exp(\pi \tan \phi)
\]

(F.2)
\[ N_c = (N_q - 1) \cot \phi \]  

where \( \phi \) is the angle of internal friction. By Keverling Buisman, Caquot and Terzaghi, and Brinch Hansen the equation was expanded to finally arrive at the equation of Brinch Hansen, which includes all relevant factors under all circumstances in one formula:

\[ p = i_c s_c c N_c + i_q s_q q N_q + i_\gamma s_\gamma \frac{1}{2} \gamma B N_\gamma \]  

where

\[ N_\gamma = 2(N_q - 1) \tan \phi \]  

The third term of the right-hand-side of Equation (F.4) represents the dead weight of the soil, where \( B \) is the width of the loaded strip and \( \gamma \) is the volumetric weight of the soil. The coefficients \( i_c \) and \( i_q \) are correction factors for a possible inclined direction of the load (inclination factors), and \( s_c \) and \( s_q \) are correction factors for the shape of the loaded surface area (shape factors). Commonly used values for the inclination factors are:

\[ i_c = 1 - \frac{t}{c + p \tan \phi} \]  

\[ i_q = i_c^2 \]  

\[ i_\gamma = i_c^3 \]  

where \( p \) and \( t \) are the vertical and horizontal shifts as a result of the inclination, respectively. The values for the shape factors are often determined by:

\[ s_c = 1 + 0.2 \frac{B}{L} \]  

\[ s_q = 1 + \frac{B}{L} \sin \phi \]  

\[ s_\gamma = 1 - 0.3 \frac{B}{L} \]  

where \( B \) is the width and \( L \) is the length of the load.

Sufficient bearing capacity is guaranteed when the strength \( p \) is larger than or equal to the calculation value of the applied load.

When determining the bearing capacity of a saturated layer of dredged material, the situation is undrained and therefore \( \phi = 0 \). Assuming a load case without inclination and shape factors Equation (F.4) reduces to the simple form:
F.2. SLOPE STABILITY

\[ p = cN_c \quad \text{(F.12)} \]

With \( \phi = 0 \), \( N_c \) becomes \( \pi + 2 \), giving an upper limit \( p_c \) for Prandtl’s solution:

\[ p_c \geq (\pi + 2)c \approx 5.14c \quad \text{(F.13)} \]

**F.2 Slope stability**

The most widely used method for the analysis of slope stability is the method of slices. The failure mechanism is schematized as a circular sliding surface which is divided into a number of vertical slices (see Figure F.2).

Along the part of the slip circle that runs through the soil body there is a shear stress \( \tau \), which has to be a factor \( F \) smaller than the maximum possible shear stress, analogous with the failure criterion of Coulomb:

\[ \tau = \frac{1}{F} (c + \sigma'_n \tan \phi) \quad \text{(F.14)} \]

where \( c \) is the cohesion and \( \sigma'_n \) is the effective normal stress. The first assumption made here is that the stability factor \( F \) is equal for all slices. The second assumption is that the angle of repose is equal to the angle of internal friction (\( \delta = \phi \)), which is a rather inaccurate and unsafe approximation. The failure criterion of Coulomb and the accompanying \( \delta \) are only based on the normal stress \( (\sigma_n) \) and therefore only the horizontal shear plane is checked for failure. The failure criterion of Mohr-Coulomb and the accompanying \( \phi \) are based on the two principal stresses \( (\sigma_1 \text{ and } \sigma_3) \), so that the soil is checked for failure in all directions.

From the moment equilibrium with respect to the center of the slip circle follows that:

\[ \sum \gamma hbR \sin \alpha = \sum \frac{\tau bR}{\cos \alpha} \quad \text{(F.15)} \]
where $h$ and $b$ are the height and width, $\gamma$ is the volumetric weight of the slice and $R$ is the circle radius. If all the slices have an equal width, combination of Equation \((F.14)\) and Equation \((F.15)\) gives:

$$F = \sum \left[ \frac{(c + \sigma'_n \tan \phi) / \cos \alpha}{\gamma h \sin \alpha} \right]$$  \(F.16\)

which represents the basic principle for many stability calculations. One of the most preferred methods to analyze slope stability is Bishop’s method. This method describes the forces and moment equilibrium of a single slice as shown in Figure \(F.2\). In accordance with the moment equilibrium (Equation \((F.15)\)) and with $\sigma'_n = \sigma_n - p$ follows for the vertical equilibrium of a slice:

$$\gamma h = \sigma_n + \tau \frac{\sin \alpha}{\cos \alpha} = \sigma'_n + p + \tau \frac{\sin \alpha}{\cos \alpha}$$ \(F.17\)

In combination with Equation \((F.14)\) this can be rewritten to obtain:

$$\sigma'_n \left(1 + \frac{\tan \alpha \tan \phi}{F} \right) = \gamma h - p - \frac{c}{F} \tan \alpha \quad \text{\(F.18\)$$

By substitution of $\sigma'_n$ into Equation \((F.16)\) this finally leads to:

$$F = \sum \left[ \frac{c + (\gamma h - p) \tan \phi}{\cos \alpha [1 + \tan \alpha \tan \phi / F]} \right] \frac{\gamma h \sin \alpha}{\sum \gamma h \sin \alpha}$$ \(F.19\)

Because the stability factor $F$ is also present on the right-hand-side its value has to be determined iteratively. In general, slope stability analysis by means of the method of slices is done by computer, because many slices and critical slip circles have to be investigated. Designated stability analysis programs can also include the influence of different soil layers on the overall stability of the soil structure.

## F.3 Settlement

Common practice in the Netherlands to determine the settlement of a soil body under influence of a load and the subsequent consolidation process is by the formula of Koppejan for the strain:

$$\varepsilon = U \left( \frac{1}{C_p} + \frac{1}{C_s} \log \left( \frac{t}{t_1} \right) \right) \left( \frac{\sigma}{\sigma_1} \right)$$ \(F.20\)

$C_p$ is the primary compression constant and takes the primary strain into account, which is the strain as a result of the consolidation process. The secular compression constant $C_s$ represents the share of the secular strain, which takes place after the soil is consolidated.
Various theoretical explanations can be given for the secular effect, but it is commonly known by the term *creep* and can be described by a semi-logarithmic relation.

The parameter $t$ is the consolidation time, for which often is taken $t = 10,000$ days. For the reference time has been agreed that $t_1 = 1$ day. $\sigma_1$ is the initial effective stress and $\sigma$ is the new effective stress under load conditions, which can also be written as $\sigma = \sigma_1 + \Delta \sigma$.

The factor $U$ is the degree of consolidation, which describes the settlement process in time. This is a dimensionless quantity, varying between 0 (for $t = 0$) and 1 (for $t = \infty$). The degree of consolidation can be analytically solved by the following equation:

$$U = 1 - \frac{8}{\pi^2} \sum_{j=1}^{\infty} \frac{1}{(2j-1)^2} \exp[-(2j-1)^2 \frac{\pi^2 c_v t}{4 h^2}] \tag{F.21}$$

which is a function of only the time parameter $c_v t / h^2$ where $c_v$ is the consolidation coefficient and $h$ is the layer thickness over which consolidation has to take place. For the derivation of the analytical solution is referred to Verruijt [32]. If the time parameter is very small, one has to generate many terms in order to obtain sufficient accuracy. This can easily be done by computer. However, for a first estimations of the degree of consolidation, (F.21) can be approximated by the two following equations:

$$U \approx \frac{2}{\sqrt{\pi}} \sqrt{\frac{c_v t}{h^2}} \quad \text{if} \quad U < 0.5 \tag{F.22}$$

$$U \approx 1 - \frac{8}{\pi^2} \exp(-\frac{\pi^2 c_v t}{4 h^2}) \quad \text{if} \quad U > 0.5 \tag{F.23}$$

The first equation holds for the first half of consolidation and the second equation for the second half of consolidation.

The consolidation coefficient $c_v$ is given by:

$$c_v = \frac{k}{\gamma_w (m_v + n\beta)} \tag{F.24}$$

$c_v$ is therefore a function of the permeability $k$, the compressibility of the soil $m_v$ and $n\beta$, which takes the compressibility of the water in the pores into account. If there is also air present in the pores $\beta$ can be much larger, because air is far more compressible than water. $\gamma_w$ is the volumetric weight of the water.

In this research, instead of using Equation (F.24) the calculation value of the $c_v$ is derived from the results of laboratory experiments and field test done for the Slufter as presented in [18].

The actual settlement can be calculated from the strain by:

$$\varepsilon = \frac{\Delta V}{V} \tag{F.25}$$
APPENDIX F. THEORY ON SOIL MECHANICS

Since $V$ is proportional the layer thickness $h$, the decrease in height can be estimated by:

$$\Delta z = \varepsilon h \quad (F.26)$$

In theory the consolidation process is infinitely long. However, in practice it is sufficient if the last term in the infinite in Equation (F.21) is 0.01, because then 99% of the eventual settlement is reached. In that case one may find the following useful approximation for the consolidation time:

$$t_{99\%} \approx \frac{1.8h^2}{c_v} \quad (F.27)$$

The consolidation process can be accelerated significantly by means of vertical drainage. In [31] is explained how to make calculations with the application of vertical drains. The principle for including vertical drains in the calculations of the consolidation process is to include a characteristic value for the degree of consolidation in the Koppejan formula, Equation (F.20). The relevant formulas are described below:

$$U_h = 1 - e^{-8T_h} \quad (F.28)$$

where $U_h$ is the average degree of consolidation for saturated soil. The factor $\mu$ and the time factor $T_h$ are given by:

$$\mu = \frac{n^2}{n^2 - 1} \left[ \ln(n) - \frac{3}{4} + \frac{1}{n^2} \left( 1 - \frac{1}{4n^2} \right) \right] \quad (F.29)$$

$$T_h = \frac{c_h t}{D^2} \quad (F.30)$$

$c_h$ is the horizontal consolidation coefficient and $D$ is the equivalent diameter of the soil cylinder that each vertical drain has to de-water. The factor $n$ equals $D/d$ where $d$ is given by:

$$d = \frac{2(b + t)}{\pi} \quad (F.31)$$

where $b$ and $t$ are the width and thickness of the drain, respectively.

Equation (F.28) can also be written in terms of the consolidation time:

$$t = \frac{D^2 \mu}{8c_h} \ln \left( \frac{1}{1 - U_h} \right) \quad (F.32)$$

The ultimate goal of a fully consolidated soil layer is to improve its geotechnical properties. Vertical drainage ensures a significant decrease of the excess pore water pressure and makes compaction of the soil as a result of loading possible. In order to determine the undrained
shear strength in the new situation, Skempton found the following empirical relationship for weak clayey soil layers:

$$\frac{c_u}{\sigma} = 0.11 + 0.0037I_p$$  \hspace{1cm} (F.33)

where $I_p$ is the Plasticity Index \cite{16}. Although this relationship has been proven not to hold under all circumstances, it can be used for first estimations.