Autonomous dredging of mud

MSc thesis Rob Swart
December 2015
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Preface

This master thesis forms the final part of my master program in Hydraulic Engineering at the Delft University of Technology. It is the result of a discovery expedition through the field of dredging technology that started with experiments on a submerged dredge and ended with a model of water injection dredging.

I would like to thank Cees van Rhee for his support and advice. Also Bart van der Schrieck owes my gratitude for the many brainstorm sessions and comments without which this thesis would not be possible. I would like to thank Sape Miedema for joining the graduation committee. Also, I would like to thank Jan Kollen for giving me the opportunity to perform this research at Grontmij. Finally, I would like to thank Mark van de Zande from IHC-Merwede who kindly offered the facilities required for the physical experiments.

Rob Swart
December 2015
Summary

There are many locations where the accumulation of mud poses a problem. Examples are mud layers in lakes, which deteriorate the ecological situation, and mud in channels and basins of ports, which can obstruct traffic. Usually these mud deposits are removed periodically with regular dredging vessels. Looking at the future and considering the trend of the increasing autonomy of machines, it is not unthinkable that this trend will also cause changes in the dredging field.

This line of thinking resulted in the development of a concept for an autonomous, submerged and free floating dredging device by the engineering consultancy Grontmij. This plate-shaped device was thought to be able to remove mud by suction while hovering just above the bed. A small scale physical model was set up to examine the feasibility of this concept. This model consisted of a plate with floaters and a suction mouth connected to a dredge pump. The setup was placed in a steel water tank in which the device could move in two directions. After performing initial experiments and basic calculations, it became clear that this concept was not feasible. Due to the submerged, free floating design in combination with a relatively large suction force, it was difficult to control the stability of the device. Therefore, this concept was disregarded, but the general idea of a small and autonomous mud dredging device was still considered to be promising.

In order to find a more viable solution, a literature study on dredging processes and their application in both existing and conceptual machines was conducted. This led to a number of potential solutions for the removal of thin layers of mud. A multi criteria analysis led to a concept based on water injection dredging (WID).

WID is a dredging technology that uses water jets to fluidize the soil to be removed. This is done by injecting a high volume of water at a low pressure into the soil. After being fluidized, the soil-water mixture flows away under the force of gravity as a density current. This technique was developed at the end of the 1980s and has been applied in dredging projects ever since. Compared to regular dredging methods, WID is generally less expensive since the soil does not have to be transported. Moreover, since there is no direct contact with the soil, the machine suffers less from wear. The backside of WID is that it poses more restrictions on the soil and working location, making it less universally applicable.

A more extensive literature study on the theory behind water injection dredging followed. This mainly led to multiple 1990s TU Delft/HAM graduation reports in which it was tried to describe parts of the WID process. These reports gave interesting insights into the different parts of the WID process since they combined theoretical considerations with physical experiments. Unfortunately, a model that describes the full WID process from jetting till outflow could not be found.

Therefore, combining parts of the available models with novel insights and approximations, a new model was set up. In the 1990s reports, the WID process was usually split into three sub processes:

1. Jetting and loosening of mud
2. Entrainment and hydraulic jump
3. Outflow as density current

Multiple modelling attempts have been done to describe the three sub processes, but due to the complexity of the ongoing processes this is not an easy task and the results are sometimes questionable. Therefore it is attempted to make one model that covers the full WID process and can be tweaked by including experimental data.
The first sub process is the jetting and loosening of mud. First the intrusion depth is estimated using a perpendicular intrusion model. It is assumed that a jet intrudes into the soil up to a depth where the thrust pressure is equal to about 6 times the cohesion of the soil. Using the hauling velocity of the dredge, the mixture discharge and its density and velocity can be estimated.

The second sub process is more troublesome since it was thought that entrainment and a hydraulic jump take place after the jetting process, although these effects are not clearly visible in experiments. In these experiments a big, turbulent cloud of soil and water is visible and it is not exactly known what happens in this cloud. Probably, the only way to get more insight into the processes at this location is more advanced fluid modelling. Therefore, the previously used methods to calculate entrainment and the hydraulic jump are not applied in this report. Since it can be assumed that at least some entrainment takes place during this phase, an entrainment factor is simply applied to model the entrainment of water.

The third and last sub process deals with the outflow of the density current. In the past it was suggested to use a 2-layer density current model to describe this process. Unfortunately, this model results in an internal backwater curve, which requires a downstream boundary condition for subcritical flow. Since the actual goal is to estimate the distance over which the density current travels, this boundary condition is absent, making the 2-layer model unsuitable for this task.

To come to an estimation for the travel distance of the density current, a simple Chézy formulation is applied, altered with a term to account for the reduced effect of gravity under water. Using an estimation for the settlement velocity, the decrease of mud discharge is taken into account. This gives reasonable results for the properties of the density current and the distance over which this current propagates.

An issue is the high variability of the mud properties and local circumstances, resulting in a wide range of possible model results. The assumed settling velocity of the suspended mud has a large influence on the travel distance. Estimating this velocity is complex and depends hugely on local circumstances. Also, the slope and roughness of the bed have a considerable influence on the model output.

With the WID model we went back to the initial topic of autonomous dredging. For the AWID (autonomous WID) device itself, several options for propulsion, energy supply and form factor have been considered. In the view of the results with the submerged concept, a small and floating device seems to be the most realistic. A device with a jet beam of 4 m, 500 kW installed power, a 2000 kWh battery pack and therefore a working time of four hours is possible with currently available technology.

A case study has been done on the Botlek harbour in the Port of Rotterdam. Using available data on the yearly volume of dredged material and the distribution of this material over the basins, an estimation has been made for the production of the AWID in this harbour. Since both mud volume and travel distance play a key role, both the production of the AWID device and the amount of mud to be removed are expressed with the unit [volume*travel distance/year]. This case study shows that WID is a feasible dredging method to clean a harbour basin. Extrapolating the results to the full Port of Rotterdam results in a system of six AWID devices that each serve their own area around one of the six docks that are locations throughout the port.

Looking at WID, it can be concluded that more public research has to be carried out to increase our understanding of several aspects of this technique. Also, more sophisticated models of both the processes directly behind the jet and that during the outflow of the density current are desirable. To make these models as realistic as possible, detailed information about the mud to be dredged is required.
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1 Introduction

1.1 Background

Many situations exist where (semi)permanent dredging is required, for example in locations where a port basin, waterway or lake suffers from sedimentation. The traditional solution for this problem is to use a large surface floating dredger.

Currently, the developments in the field of autonomous cars are advancing at a high pace. Also the first trials with autonomous vessels are undertaken. These developments have led the concept of autonomous dredging vessels that are permanently available and are able to remove sediments continuously. Instead of one large vessel, multiple smaller devices can be deployed to remove the sediments.

Also the developments in the area of renewable energy sources can change the execution of dredging projects. It is very likely that in the future dredging vessels will be powered by renewable energy, for example stored in batteries, in the form of hydrogen or simply delivered with an electricity cable. Since the availability of solar and wind energy depends on natural circumstances and is therefore highly variable, it is not unthinkable to use a dredging solution that can work with a changing energy supply. Instead of short periods of high production, it is then possible to use a permanent solution that only works when energy is available.

This transition seems to be possible by replacing traditional dredging vessels by multiple autonomous devices that are relatively small and inexpensive. In this report, the possibilities for such devices are studied by combining knowledge about existing techniques and new developments.

1.2 Problem definition

The central research question of this report can be formulated as follows:

What are the possibilities for the removing of a thin layer of mud using an autonomous device?

The following requirements are chosen as boundary conditions for the research project:

1. The device should be able to operate autonomous and continuously.
2. The device should be able to remove material from a certain extended area and thus has to be able to relocate itself.
3. The device dredges mud, since this material can be found in many ports, waterways and lakes.
4. As with all dredging devices, energy efficiency is important. A solution based on renewable energy is preferred.
1.3 Structure of the report

First the problem will be described in more detail in chapter 2. The location and working circumstances for the dredge will be sketched. Also, a list of required device characteristics will be made and the main properties of mud will be given.

In chapter 3 a study of dredging processes and the application of these processes will be presented. Both existing dredging devices and concepts will be listed. At the end of this chapter three innovative solutions for the removal of mud from a harbour basin will be compared in a multi criteria analysis and the most promising concept will be chosen.

This most promising concept will be elaborated on in chapter 4. In this chapter a model will be developed that describes the ongoing processes. A rough estimation of the production of the dredging device will be made.

In chapter 5 the mathematical model will be applied to a realistic business case. Also, attention will be given to the form factor of the device and the layout of the full dredging system.

To conclude this report, in chapter 6 the main conclusions and recommendations are presented.
2 Problem description

2.1 Sedimentation and maintenance dredging

Many ports and access channels are sheltered by structures like breakwaters to protect the vessels against waves and increase the safety. A problem that often arises is that these ports and access channels suffer from sedimentation since their shelters turn them into low energy environments (LEE), as opposite to high energy environments (HEE) directly outside the ports, for example a river or coastline. In Figure 2.1 and Figure 2.2 examples are given of situations in which a low energy environment is located close to a high energy environment. In such a low energy environment sediments in the water will settle and deposit on the bed. This reduces the depth of the basin or channel and can thus obstruct traffic.

Figure 2.1: Schematic port area with sea influence. Source: PIANC (22)

Figure 2.2: Schematic port area with river and sea influence. Source: PIANC (22)
Usually, these sediments are removed by regular dredging vessels. To indicate the large volume of sedimentation, in Figure 2.3 an overview is given of the yearly amount of dredged material in the Botlek harbour in the Port of Rotterdam.

![Graph showing maintenance dredging in Botlek Harbour](image)

**Figure 2.3:** Yearly volume of maintenance dredging in the Botlek harbour. Source: El Hamdi (11)

### 2.2 Criteria for dredging device

The goal of this report is to develop an autonomous solution for the removal of the sediments in a port basin. In order to make a good selection of possible dredging devices a set of criteria is necessary that can be used to find the best solution for the described dredging case.

The desired application is the removal of thin (max. ~0.5 m) layer of mud from the bottom of reservoirs, canals and harbours. A key element of this report is the expectation that the device should be small, autonomous and able to work year-round. On the long term, it should be powered by renewable energy. Since the supply of this energy is expected not to constant, the device should be able to easily adapt its production to the available power.

From this general picture the following set of criteria emerges:

1. **Stable erosion process**
   - The erosion of mud should take place in a well-controlled, stable manner that can take place for hours without human interference.
2. **Maintainability**
   - It has to be possible to reach the device for periodical maintenance.
3. **Reliability**
   - In the system has to be reliable to minimize the risk on delays.
4. **Safety**
   - Since other vessels can be around the dredging device, safety of people and materials needs to be taken into account.
5. **Required manpower for supervision**
   - Even though a device can be called ‘autonomous’, with the current state of technology supervision is still required.
6. **Development and construction costs**
   - To be economically feasible, the system may not be too expensive.
7. **Capable of dredging mud**
   - As described before, the focus will be on the dredging of muddy material. This requires different properties compared to sand dredging.
8. Works in area with lots of debris, e.g. a harbour
   *In the water of a harbour basin many kinds of debris can be found. The device should be able to deal with this. Also the device should be able to work at depths of 10-20 m.*

9. Small device that can work year-round (workability)
   *Key element of the device that has to be found is that it can work year-round to optimize its efficiency.*

10. Can work with changing energy supply
    *Since in the future a change from fossil fuels to renewable energy is to be expected, the device should be able to cope with changing energy supplies.*

11. Influence on traffic and other users
    *Since a port areas usually experience heavy traffic, the influence of the dredging operation on this traffic should be minimal.*

### 2.3 Mud characteristics

In this chapter an overview of the main properties of mud will be presented. Several key indicators that have an influence on the dredging of mud will be explained. This information will be used to choose representative mud properties that can be used as input in the model that will be developed in this report.

In their thesis’s Veen (40), Eijnthoven (10) and Vijverberg (44) give a clear overview of the properties of mud and therefore most of the information in this paragraph is taken from their reports.

#### 2.3.1 Mud classification and composition

The term mud is used for weak soil mixtures of clay, silt, possibly sand, mixed with water. The properties of mud depend on the composition of the material. The (possible) components are:

- Non-cohesive particles: silt and sand.
- Cohesive particles: clay and organic material. The suspended clay particles group in a flocculation process, the formed flocs have different properties than the single particles.
- Gas: formed by chemical decomposition processes.
- Water: properties of water influence the flocculation process.
- Organic material and skeletons

These components can vary in space and time and depend on many different variables. The behaviour of mud is strongly dependent on these components, which makes it difficult modelling difficult. Therefore, field measurements are required to find good input values for calculations.

The colloidal fraction consists mainly of the clay minerals kaolinite, illite and montmorillonite that are to a large extent responsible for the cohesive behaviour of mud. These minerals consist of electrical charged particles that can attract other particles like water molecules, ions from dissolved salts, organic particles or other clay particles. The type of bonds between these particles can either stimulate or counteract the flocculation effect, which influences the behaviour of the mud. Large flocs have a larger settlement velocity and can therefore influence the sediment characteristics.

The properties of mud can be divided in those that are structure-dependent and thus have to be determined in-situ, and those that are structure-independent and therefore can be determined from stirred soil samples. To identify and classify mud, the structure-independent properties are mainly used. (40)
These properties are:

- Composition of solid matter: mineralogy and organic content
- Composition of pore water
- Particle properties: grain distribution, specific surface, density, electric charge
- Flocculation parameter: parameter for the occurrence of flocculation
- Attenberg limits
- Stirred rheological properties
- Water and gas content

2.3.2 Appearance in water column

Looking at the appearance of mud in a water column, Ross and Matha present a general description of a three-layer system consisting of the following layers: (9)

1. A consolidated bed with a concentration larger than 300 kg/m$^3$. This layer has accumulated over time, during which the both density and strength increased.
2. A fluid mud suspension layer with a concentration in the range of 10 to 300 kg/m$^3$. This layer can be subdivided into two sub layers:
   - Laminar (viscous) lower layer (100 to 300 kg/m$^3$)
   - Turbulent upper layer (10 to 100 kg/m$^3$)
3. A suspension layer with a concentration in the range of 0 to 10 kg/m$^3$.

These layers are indicated in Figure 2.4.

Figure 2.4: Three layer system of mud sediment in the water column and bed. Source: Ross and Matha (26)
Nasner (19) studied the properties of mud in different harbour basins at the German North-Sea coast. He measured the solid concentration, density and dynamic viscosity of the mud at different locations in the mud layer. These locations are depicted in Figure 2.5. The solid concentration, dynamic viscosity and density in the different layers are shown in respectively Figure 2.6, Figure 2.7 and Figure 2.8. These figures show that there are considerable differences between the solid concentration and viscosity in the same layer in different harbour basins.

Figure 2.5: Sketch of sample positions in the fluid mud layer. The three points marked with red crosses represent the lutocline, middle and lower layer. Source: Nasner (19)

Figure 2.6: Solid concentration in the different layers and harbours. Source: Nasner (19)
Figure 2.7: Dynamic viscosities in the different layers and harbours. Source: Nasner (19)

Figure 2.8: Densities in the different layers and harbours. Source: Nasner (19)

Wurpts and Torn (52) investigated the relationship between the yield strength and density of mud in the ports of Rotterdam and Emden. As can be seen in Figure 2.9, for mud to be fluid a threshold density of about 1100-1150 kg/m³ can be assumed.
More data on the mud density in a water column is given by Winterwerp and Van Kesteren (51), who presented a vertical density profile as measured in the access channel of the Port of Rotterdam (Figure 2.10).

2.3.3 Cohesion

Since the process of deforming and removing of the mud will take place at high speed, it can be assumed that it is a undrained process, so this undrained shear strength is applicable. Therefore, the undrained shear strength, or cohesion, $c_u$ of mud is an important property for dredging purposes.

The cohesion of the mud is a measurement of its shear strength so it indicates the resistance against breaking the coherence between its particles. To break this coherence, a shear strength is required that is at least as large as the cohesion of the mud.

The cohesion is determined by the mineralogical composition, organic material, pore water properties and rheological history of the mud.
According to PIANC (22), it can be stated that generally soil can be transformed economically into a mixture when the cohesion is below 10 kPa. Soils with higher cohesion will only partially go into suspension with remaining soil in lumps at the bottom. It is also stated that silt typically encountered in maintenance dredging projects has a shear strength far below 5 kPa.

### 2.3.4 Water content and plasticity index

Cohesive soils can be classified using the plasticity limits, also called Attenberg limits. These give the state of the soil depending on the amount of water it contains: the water content. The water content of mud is defined as the water mass divided by the grain mass:

$$ w = \frac{W_{\text{water}}}{W_{\text{grains}}} $$

The soil can have one of the following four consistencies: solid, semi-solid, plastic and liquid. The boundaries between these consistencies are called Attenberg limits:

- Shrinkage limit \( w_s \): water content for which a further reduction of the water content no shrinkage occurs.
- Plastic limit \( w_p \): water content for which the clay can just be rolled into 3 mm thick wires.
- Liquid limit \( w_l \): water content for which a V-hole in the material starts to flow after 25 beats (Casagrande method) or the water content belonging to a intrusion depth of 10/20 mm with a falling cone with a cone of 60 g - 60°/80 g - 30° (method of Head).

To make a distinction between the consistencies, a set of two standard tests has been developed by Casagrande. These are laboratory tests, so the state of the soil can be different from its original state. This has to be taken into account when considering the applicability of the test results. In Figure 2.11 an overview of the different consistencies and Attenberg limits is given.

![Figure 2.11: Soil consistencies and Attenberg limits. Source: Eijnthoven (10)](image)

Typical Attenberg limits for the clays kaolinite, illite and montmorillonite are given in Table 2.1.

<table>
<thead>
<tr>
<th>Clay type</th>
<th>Shrinkage limit ( w_s ) (%)</th>
<th>Plastic limit ( w_p ) (%)</th>
<th>Liquid limit ( w_l ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montmorillonite</td>
<td>9-15</td>
<td>50-100</td>
<td>100-900</td>
</tr>
<tr>
<td>Illite</td>
<td>15-17</td>
<td>35-60</td>
<td>60-120</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>25-29</td>
<td>25-40</td>
<td>30-110</td>
</tr>
</tbody>
</table>

**Table 2.1: Shrinkage limit, plastic limit and liquid limit for different clay types (10)**

The difference between the liquid limit and the plastic limit is called the plasticity index \( I_p \) and is used to compare different soils with each other:

$$ I_p = w_l - w_p $$
The relative position of the actual water content relative to the plasticity limit and the liquid limit is given by respectively the liquidity index $I_l$ and the consistency index $I_c$:

$$I_l = \frac{w - w_p}{w_l - w_p}$$  \hspace{1cm} (2.3)

$$I_c = \frac{w_c - w}{w_l - w_p}$$  \hspace{1cm} (2.4)

The Attenberg limits, and mainly the liquid limit, represent the influence of the soil composition on the stirred properties. In a plasticity graph as in Figure 2.12 the plasticity limit is plotted against the liquid limit. This graph can be used to classify cohesive soils based on their plasticity, which is a complex function of amongst others mineralogical composition, particle surface and organic content. The so-called A-line is an empirical line that splits the graph in a part for organic and inorganic clays. The B-line represents a upper limit for all soil types.

**Figure 2.12: Plasticity graph by Casagrande (7)**

Another important parameter is the activity parameter that can be used to predict the dominant clay type present in a soil sample. The activity is defined as the plasticity index divided by the percentage of particles smaller than 2 μm:

$$A = \frac{I_p}{\% < 2 \mu m}$$  \hspace{1cm} (2.5)

Figure 2.13 shows that different clay types have a specific value for the activity parameter.
In general the stirred shear strength of cohesive soils is clearly related to the water content. Experiments show that for a water content equal to the liquid limit the stirred shear strength is about 2 kPa. Most types of mud have a water content that is far larger than the liquid limit, so a cohesive soil can be called mud when the shear strength is lower than about 2 kPa. However, the non-stirred shear strength can be considerably higher.

Therefore Eijthoven (10) uses the following definition of mud:

Mud is a fine grained, plastic material with a stirred shear strength smaller than 2 kPa and an upper limit for the undrained shear strength of 10 kPa.

### 2.3.5 Rheology

Rheology describes the deformation of materials in relation with the forces that are exerted on these materials. The resulting relationship between the shear stress $\tau$ and shear rate $\frac{du}{dz}$ or $\dot{\gamma}$ characterizes a fluid. If this relationship is linear and starts in the origin, the fluid is called Newtonian. There are also non-Newtonian models, for example the Bingham, pseudoplastic and dilatant models, see Figure 2.14.

![Figure 2.13: Relationship between plasticity index and clay fraction. Source: Skempton and Northey (31)](image)

![Figure 2.14: Different rheological models. Source: wikipedia.org](image)
The rheological properties of mud are difficult to describe since during an in-situ experiment or the sampling process the rheological properties of the mud often change. However, the Bingham model is mostly used. Winterwerp and Van Kesteren (51) give the following formula for this model:

\[ \tau = \tau_y + K \gamma^n \] (2.6)

With \( \tau_y \) the so-called yield stress, the stress which the material starts to shear. The yield stress has a very different value for different solid concentrations. For a concentration of about 10 g/L the yield stress is in the range of 0.01-0.1 Pa. For concentrations up to 1000 g/L it can have values between 1 and 10 kPa. K varies between 0.01 to 1000 Pa·s\(^n\). The value of n is between 1/3 and 1/2. Usually, it is assumed that n=1, leading to the Bingham plastic model as shown in Figure 2.14. In this case K is the viscosity of the fluid mud.

2.3.6 Density
Density is an important characteristic of mud for dredging purposes. The main factors that determine the density of mud are:

- Mud composition
  *The gas content, the water binding capacity of the clay minerals and the possibly present organic materials influence the density of mud.*

- Deposition environment
  *The deposition environment and the type of supplied material mainly determine the initial state of the mud. When the mud layer is deposited, the particles can settle individually or grouped (floculated). These two processes mainly occur in respectively fresh and salt water environments.*

- Level of consolidation
  *Consolidation is the reduction of the pore volume of a material under the influence of the weight of the material itself or a top load. During this process the pore water flows out and water pressure is converted into grain pressure. This process is takes time and only after a certain period the full load is carried by the grain pressure.*

2.3.7 Consolidation and bed strength
Van Rijn (38) gives the following definition for the consolidation process: a process of floc compaction under the influence of gravity forces with a simultaneous expulsion of pore water and a gain in strength of the bed material. This process only occurs close to the bed. Three stages can be distinguished:

- Initial stage (days): processes of hindered settling and consolidation occur at the same time. Freshly deposited flocs are grouped in an open structure with large pore volumes. The bed surface sinks linear with time t.
- Secondary stage (weeks): the pore volume is further reduced. Pore water escapes through small drains and the bed surface level sinks with \( \sqrt{t} \) or \( \log(t) \).
- Final stage (years): flocs are broken down and the pore volume is further reduced.
Van Rijn gives indicative figures for the ranges of the sediment density during the different stages as shown in Table 2.2.

<table>
<thead>
<tr>
<th>Consolidation stage</th>
<th>Rheological behaviour</th>
<th>Wet sediment density [kg/m³]</th>
<th>Dry sediment density [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freshly consolidated (1 day)</td>
<td>Dilute fluid mud</td>
<td>1000-1050</td>
<td>0-100</td>
</tr>
<tr>
<td>Weakly consolidated (1 week)</td>
<td>Fluid mud (Bingham)</td>
<td>1050-1150</td>
<td>100-250</td>
</tr>
<tr>
<td>Medium consolidated (1 month)</td>
<td>Dense fluid mud (Bingham)</td>
<td>1150-1250</td>
<td>250-400</td>
</tr>
<tr>
<td>Highly consolidated (1 year)</td>
<td>Fluid – solid</td>
<td>1250-1350</td>
<td>400-550</td>
</tr>
<tr>
<td>Stiff mud (10 years)</td>
<td>Solid</td>
<td>1350-1400</td>
<td>550-650</td>
</tr>
<tr>
<td>Hard mud (100 years)</td>
<td>Solid</td>
<td>&gt;1400</td>
<td>&gt;650</td>
</tr>
</tbody>
</table>

Table 2.2: Density ranges of consolidated mud for several stages. Source: Van Rijn (38)

The consolidation of a layer of mud depends, amongst others, on:

- Initial layer thickness
- Initial concentration
- Permeability, which depends on sediment composition, size, organic content, salinity and temperature

It can be noted that for sediments in port areas a density of 1200 kg/m³ is assumed to be the maximum density through which vessels can sail. The depth at which the mud has this density is called the nautical depth.

The wet density can be calculated from the dry density with the following formula:

$$\rho_{\text{wet}} = \rho_{\text{dry}} + \rho_w \left(1 - \frac{\rho_{\text{dry}}}{\rho_s}\right)$$  \hspace{1cm} (2.7)

The effect of sediment concentration ($c$ in kg/m³) on fluid density ($\rho$ in kg/m³) is given by Winterwerp (50):

$$\rho(S,T,c^{(i)}) = \rho_w(S,T) + \sum_i \left(1 - \frac{\rho_w(S,T)}{\rho_s^{(i)}}\right)c^{(i)}$$  \hspace{1cm} (2.8)

With

- $\rho_w(S,T)$ = water density as function of temperature and salinity [kg/m³]
- $\rho_s^{(i)}$ = solid density of mud [kg/m³]
- $i$ = fraction [-]
- $c$ = concentration [kg/m³]
3 Comparison of concepts

The aim of this chapter is to find a device that is suitable to dredge a layer of mud autonomously from a harbour basin. In order to do so, this chapter starts with a literature study on fundamental dredging processes, the applications of these processes and existing dredging devices. Both widely used machines and concepts that never have been tested or used in practise are summed up.

After this literature study a number of new ideas and concepts will be presented that might be a good solution for the given problem. The properties of the dredging devices are compared and the three most promising new concepts are chosen. A multi-criteria analysis will be used to choose the most-promising concept. This concept will be elaborated on in the next chapters.

3.1 Categorization on three levels

In order to categorize the different types of dredging devices, an analysis will be made that groups their properties on three levels. Below a quick overview of the categorization is given and in the next paragraphs the concepts and technologies that fit into these categories are explained in more detail.

Level 1: dredging processes for loosening of soil
The first level consists of the fundamental processes for the loosening of soil. These are cutting by a mechanical means, erosion by suction under a ridge, erosion by jets and breaching.

Level 2: methods for applying dredging processes
Different dredging methods exist where the dredging processes from level 1 are applied in several combinations.

Level 3: dredging machines
The third level consists of the complete machines that execute a dredging operation. These machines combine a dredging method from level 2 with a means of repositioning.
3.2 Level 1: dredging processes

The fundamental dredging processes cutting, erosion and breaching are described in this paragraph.

Cutting by mechanical means
By moving a tool such as a blade through the material to be excavated, this material can be loosened and removed. The blades can be used in different configurations, for example on a rotating head, a bucket or a draghead. An overview of the cutting processes that play a role in different materials is given by Miedema (18). He also gives models to calculate the cutting force.

Erosion
Dredging by erosion means that the soil particles are loosened and transported by a moving fluid. In practice this is done by either suction under a small gap or by water jets.

There are many formulas available to calculate the erosion flux. The formulas that are regarded as most usable are those derived by Van Rijn (37) and Van Rhee (36).

Van Rijn did a lot of work on erosion and transport of sediment and derived a pick-up formula. This formula is derived for single particles that are eroded at flow velocities below 1 m/s. However, during dredging much higher velocities occur and instead of single grains, full layers are removed at the same time. The permeability of the soil and dilatancy effects result in hindered erosion, lowering the erosion rate. Van Rhee adapted the Van Rijn formula for these effects (36).

Both formulas make use of the dimensionless Shields parameter that is a measure for the amount of bed shear stress particles undergo. When this amount of stress exceeds a critical value, the particles will start to move.

Breaching
When undermining a soil slope with a jet, a cutting device or a suction mouth, it becomes unstable and the material starts to slide downwards under the force of gravity. A vertical wall, the ‘headwall’ retreats with a velocity that depends on the properties of the soil. The slope of this wall is steeper than the soil’s angle of repose due to dilatancy effects. This means that during the shearing of the grains the pore volume increases, creating an under pressure in the pores that causes a normal force. This normal force results in a friction force that balances the gravitational force. When the velocity of the equipment equals the headwall velocity, the slope will be vertical. For a velocity smaller than the head wall velocity, the angle will be smaller.
3.3 Level 2: dredging methods

The described dredging processes are applied in several dredging methods. In this paragraph an overview of these methods is given. Many of the pictures in this paragraph are taken from the lecture notes on dredging equipment by Vlasblom (45) and Van der Schrieck (35).

3.3.1 Plain suction head

The plain suction head is a simple suction mouth that is usually not much more than the open end of a suction pipe that is covered by a screen to prevent large objects from entering the pipe. The plain suction head is fit for non-cohesive material and it is more suitable for sand extraction than layer removal.

When the dredging process is started, the soil is only loosened by the eddies created by the water flow. When after a while a small pit has developed, the breaching process sets in and the production increases. By moving the suction tube forwards at the right speed, a stable production can be maintained.

Water jets can be used to help with the activation of the breaching process and to control the mixture formation. The latter is mainly of importance when the suction mouth is completely covered in soil. In this case the jets are located around the suction mouth.

Figure 3.1: The plain suction head. Source: Vlasblom (45)
3.3.2 Dustpan suction head

This head is derived from the plain suction head and has a relatively wide suction mouth. Compared to the plain suction mouth, it looks more like a modern draghead and it can extract sand at a higher production rate while the breaching height is relatively low. This low breaching height makes it suitable for the removal of thin layers. Nowadays, this type of mouth has been replaced by more efficient methods.

Since the breaching production is proportional to the square of the breaching height \((35)\), the suction mouth has to be wider in case of a small breaching height in order to achieve a good production rate. The width of a dustpan mouth is about 10 to 15 times the diameter of the suction pipe.

Similar to the circular head, water jets are used to stimulate the breaching process and to ensure that the mixture contains a sufficient amount of water.

![Dustpan heads](image)

Figure 3.2: Dustpan heads. Source: Vlasblom (45)
3.3.3 Draghead
The draghead is a very commonly used dredging tool. The head is dragged over the soil and makes use of multiple dredging processes for the loosening of soil. It can be used for the dredging of sand, silt and clay. Both maintenance dredging and sand production are possible applications.

The excavation can be done by hydraulic means, mechanical means or a combination of these. Hydraulic excavation is done by the dredge pump flow, water jets or both. The jets are also important for the forming of the mixture in the draghead. Mechanical excavation is done by blades that are connected to the draghead. In cohesive soils mechanical excavation can be used on its own. It can be noted that mechanical excavation, that uses the vessel's propeller power, is much less efficient than hydraulic excavation.

Next to the most commonly used IHC/Dutch dragheads and California dragheads also many other, specialized types have been developed. Although their fundamental concepts are good, other (practical) problems resulted in lower production rates. Examples are:

- The silt head, for silt and soft clays
- The active draghead
- The venturi head

![Draghead with blade](image-url)
3.3.4 Cutter head
The cutter head is derived from the plain suction head. In front of the suction inlet a rotary cutting mechanism is placed to be able to loosen harder materials such as gravel and rock. Variants on this head are the disc cutter, scoop and sweep heads, which are used for environmental dredging.

The cutter head loosens the soil before it is pumped away as a sand-water mixture. This cutter head rotates in front of the suction mouth and is fitted with cutting edges or teeth, depending on the type of material to be dredged.

![Image of the cutter head](image)

Figure 3.4: The cutter head. Source: Vlasblom (45)

3.3.5 Auger screw
The auger screw mechanism consists of two Archimedean screws that rotate in opposite direction. These screws cut the material and carry it to the centre, where a suction mouth is located. This way of dredging causes a very low level of turbidity and spillage and is therefore suited for the dredging of thin layers of contaminated materials.

A variant on the auger is the brush suction dredger, which uses a brush to be able to remove very thin layers of material (in the order of millimetres) and is applied in filtration ponds.

![Image of the auger screw](image)

Figure 3.5: Auger or Archimedean screw. Source: Van der Schrieck (35)
3.3.6 Water injection dredging (WID)

Water injection dredging is a relatively new dredging technique that can be very cost effective in specific dredging situations, especially in smaller projects. WID is a hydrodynamic dredging technique where the dredger injects a large volume of water at a low pressure into the soil layer in order to fluidise this layer. The jet nozzles are mounted on a horizontal tube that has a length between 4 and 15 meter.

To fluidise cohesive (fine) soils, the cohesion has to be overcome and to fluidize granular soils the internal friction has to be overcome. By injecting this jet of water into the bed a density flow is created that has a higher density than the surrounding water. The layer of fluidized sediment flows away under the force of gravity while remaining close to the water bed. An important point is that the dredge has to keep moving during the jetting process to prevent the water-soil mixture from reaching low densities. This would result in the mixture being dispersed over the full water column, which has an undesirable influence on the environment.

WID can only be applied to soils with specific properties. For cohesive soils, the undrained shear strength cannot be too high, because this would prevent the jet from fluidising the material. Lumps of clay will form which will quickly settle on the bed. For non-cohesive soils the average grain size cannot be too high because this will cause the grains to settle too quickly, preventing the flow from travelling over long distances.

![Diagram of Water Injection Dredger](image-url)

Figure 3.6: Water injection dredger. Source: Van der Schrieck (35)
3.3.7 Agitation dredging
Related to water injection dredging is agitation dredging. These techniques show important differences and should not be confused. Whereas with WID the generated density flow stays close to the bed and flows to lower laying areas under the force of gravity, the suspension created with agitation dredging is dispersed over the full water column and is displaced with the water flow, for example a river or tidal current.

Agitation dredging is executed with a regular trailing suction hopper dredger. After pumping up the dredged material it is not stored in the vessel's hopper, but discharged into the water. This type of dredging can be used for sand, silt, mud and clay. An important prerequisite is that a high degree of turbidity and siltation are allowed at the dredging location.

3.3.8 Mass flow excavation (flow dredging)
A common method to dig trenches for submarine pipelines is mass flow excavation or flow dredging. A jet is used to erode bed material in order to create a trench. Either a single, large diameter, low pressure jet or an array of small, high pressure jets can be used.

![Figure 3.7: Mass flow excavation. Source: Van der Schrieck (35)](image)
3.3.9 Bucket, grab and other mechanical methods

There are several mechanical dredging methods. One of the first was the bucket ladder dredger and this can be seen as the predecessor of the cutter suction dredger. Other mechanical methods are the grab, the dredge wheel, consisting of rotating buckets and the bottom disk cutter, consisting of a horizontal rotating disk.

All mechanical dredging methods use the cutting process to loosen the soil. In case of the dredge wheel and bottom disk cutter the mixture is pumped away, while the bucket ladder uses mechanical transport.

The mechanical devices can be applied for both sand, clay and rock. They are suitable to dredge harder materials and are also used in mining.

Figure 3.8: Bucket dredger. Source: Vlasblom (45)
3.4 Level 3a: widely used or prototyped dredging machines

The dredging methods that are described in the previous paragraph are applied in different types of dredging machines. In this report these machines are divided in three categories:

- Level 3a: machines that are either widely used or have been tested in practice (paragraph 3.4)
- Level 3b: machines that only exist on paper as concepts. These have been found in patent databases and other publications (paragraph 0)
- Level 3c: new unconventional concepts (paragraph 3.7)

In paragraph 3.6 a schematic overview of the different existing dredging processes, methods and machines is given.

3.4.1 Stationary pontoon

A stationary pontoon can be equipped with several dredging tools. The most common is the cutter suction dredger (CSD), see Figure 3.9. A pontoon can also be equipped with a plain suction mouth (Figure 3.10), dustpan mouth or mechanical means as bucket ladder, but these are either outdated or less common. Also, pontoons are often equipped with a grab, resulting in backhoe dredgers.

Usually the pontoon can move itself over small distances using anchor lines and spud poles. For travelling over larger distances the pontoon relies on a support vessel.

Figure 3.9: The cutter suction dredger. Source: Vlasblom (45)

Figure 3.10: The plain suction dredger. Source: Vlasblom (45)
3.4.2 Vessel without hopper
A simple, self-propelling vessel without internal storage area (a hopper) can be equipped with several dredging tools. For example an auger screw, brush suction head or an installation for water injection dredging, agitation dredging or mass flow excavation.

3.4.3 Vessel with hopper
Vessels with a hopper are often equipped with dragheads. This combination is called a trailing suction hopper dredger or TSHD (Figure 3.11). Using a dredge installation and one or two trailing drag heads it can load and unload itself. Together with the CSD this is the most common dredger type. The TSHD can also be used for agitation dredging and mass flow excavation.

Figure 3.11: The trailing suction hopper dredger. Source: Vlasblom (45)

3.4.4 ROV (free floating)
It is possible to mount a suction head on a remotely operated vehicle (ROV). Although the ROV can move freely, due to reasons of stability the device rests on the bed during the dredging process. An example of such a machine is given in Figure 3.12.

Figure 3.12: Oceaneering Millennium ROV dredge. Source: oceaneering.com
3.4.5 ROV (tracks/trencher)
Multiple ROV trenchers are available that drive on the sea bed with caterpillars. The company SMD states that its trenchers can be configured to carry multiple tools, simultaneously or in interchangeable cartridges, including rock and clay chains, jetters, dredges, educators (Venturi pump) and backfill tools to suit every soil combination (Figure 3.13). The company LWT manufactures a submersible crawler dredge for removing mud and build-up in sewers, tanks and covered lagoons (Figure 3.14).

Figure 3.13: SMD QTrencher 1000. Source: smd.co.uk

Figure 3.14: LWT Mud Cat ROV submersible crawler dredge. Source: lwtdredge.com
3.4.6 Punaise
The Punaise is a machine that has been developed and tested in the nineties by the PinPoint Dredging Company, a partnership of Damen, Nelis (currently BAM), Ballast Nedam Dredging (currently Van Oord) and Boskalis. This machine is rather unique since it is designed to stay underwater on a single location, from where it pumps sand to for example a beach.

The Punaise is placed on the sea bottom and fluidises sand with water jets. This results in a breaching process during which sand is pumped up with the central suction mouth (see Figure 3.15 and Figure 3.16). The device moves downward in the created pit and is positioned using ballast water and anchor lines. It is possible to move the device when the pit is considered deep enough, but it is also possible to dump new sand in the pit and hence on top of the Punaise.

The device is controlled from an operation station on the land. The Punaise is used to transport sand to the coast that has been dumped near shore by a vessel, but also for sand-bypasses (48). The device is not in use anymore because of several reasons. Firstly, nowadays it is more common to dump sand on the foreshore instead of the beach itself. So transport by pipelines to the beach is not as required as often as in the past. Secondly, in case of a sand bypass, the sand needs to be ‘caught’ over a certain distance from the beach in offshore direction. Since a single Punaise has only a limited range of influence, multiple Punaises have to be placed in line, making the operation very costly. A third possible reason is a conflict of interests between the involved companies.

In his thesis, Van Berk (34) has made calculations on the application of the Punaise for sand suppletions on the beach of the Kop of Goeree.

![Figure 3.15: The Punaise dredge. Source: theartofdredging.com](image)

![Figure 3.16: The Punaise dredge in use. Source: theartofdredging.com](image)
3.4.7 Walking dredge
A collaborative research project between the Memorial University of Newfoundland (Canada) and Excavation & Equipment Manufacturing (India) has resulted in the design, build and test of a small prototype of a modular walking submersible dredger for the cleaning of deep inland reservoirs (Figure 3.17). The philosophy behind this device is that legs are suitable to move a machine over unstructured terrain. For the actual dredging, the same techniques as in a cutter suction dredger are used (28).

![Figure 3.17: Prototype of the walking dredge. Source: Sarkas, Bose and Sarkar (28)](image)

3.4.8 Archimedean Screw Vehicle
An Underwater Archimedean Screw Vehicle has been developed by Harbor Branch Oceanographic Institution that propagates using a Archimedean screw and removes material with an auger scraper (Figure 3.18). The device fluidizes the bottom material with a low pressure jet. It is designed to remove thin layers of fine-grained materials with a high precision: the accuracy is between 5 and 10 cm. Remotely operated vehicle (ROV) principles are used to assist in guidance, navigation and control (17).

![Figure 3.18: Harbor Branch Underwater Archimedean Screw Vehicle. Source: McLellan and Hopman (17)](image)
3.5 Level 3b: existing unconventional concepts

In this paragraph an overview of several existing ideas for non-conventional dredging devices is given. These have been found in patent databases, master thesis’s and other publications. These concepts do only exist on paper and, as far as could be verified, have never been realized in practice. Although their practical value can be questioned, they can prove useful as a source of inspiration for new concepts.

3.5.1 Submersible dredge by Sloan
Sloan designed a submersible dredge that is occupant-operated and drives over the bed using caterpillars. It has a cutter suction mouth and a pipeline system to discharge the dredged material. This idea is patented by Sloan et al. in 1972 (32).

Figure 3.19: The Submersible dredge by Sloan (32)
3.5.2 STUMP by Batchelder
The STUMP, which stands for Submersible, Transportable Utility Marine Pump looks like a vertical submarine that has space for a small crew and can be combined with different kinds of dredging tools. It has been patented by Batchelder in 1978 (2).

The inventor claims that multiple types of dredging tools can be used, both hydraulic and mechanical means. So the processes that take place in the plain suction dredger, jets, cutter suction dredger and auger dredger are applicable. Moreover, he also mentions the possible use of a mechanical bucket dredger.

Figure 3.20: The STUMP by Batchelder (2)
3.5.3 Submersible sludge removal apparatus by Werner/Decker
A submersible sludge removal apparatus has been patented by Werner and Decker in 1987 (47). This is an apparatus for removing sludge from a settling basin, consisting of a pair of longitudinally extending pontoons with in between a submersible pump for pumping sludge. This pump can be pivoted to vary the composition of the dredged material. An external device is required to move this device by means of a cable.

Figure 3.21: The submersible sludge removal apparatus by Werner/Decker (47)
3.5.4 Mud slicer (slibschaaf) by Uiterwijk
In 1988, Uiterwijk (33) patented a device to remove thin layers of mud from the bed of a basin. This ‘mud slicer’ device is dragged along the bed and captures the mud before it is pumped out. The inventor gives attention to the fact that the device can be used in both the forward and backward direction. The slicer consists of a box that is open at the bottom side, closed at the sides and has flexible flaps at the front and rear side to allow the mud to enter.

Figure 3.22: The mud slicer by Uiterwijk (33)

3.5.5 Stationary suction pit by Biemond
In his master thesis, written in 1993, Biemond (4) compares different possibilities for a ‘rehandle’ apparatus. With this term he refers to a device that can be used to transport dredged material from a location near the shore to the shore itself. The goal is to avoid the use of a coupling buoy between a vessel and sinker line which is seen as the factor that limits the workability of the process. Biemond compares traditional dredging vessels with the Punaise (see paragraph 0), ejection pumps and a channel with fluidisation pipes. He concludes that the last option is the most promising.

The processes in and around these pipes are similar to that in the hopper of a trailing suction hopper dredger. The sand is fluidised with jet water in order to be pumped away. A fluidisation pipe is located on the sea bed, on which sand is dumped. A fluidisation channel is formed above the pipe and due to the breaching effect and fluidisation the sand will flow towards the suction head.

Figure 3.23: The stationary suction pit by Biemond (4)
3.5.6 Sediment removal system by Price
In 2002, Price (24) patented a sediment removal system that consists of two parts: a vessel floating on the water surface and a submerged crawler that includes a drive system for moving across the bottom of a water body. An auger scraper is mounted on the submerged crawler. This crawler features a suction device to remove sediments. The device is meant to be used on the bottom of settling ponds that are used for wastewater treatment.

![Figure 3.24: Sediment removal system by Price (24)](image)

3.5.7 TRIPOD by Verheul
In his thesis on dredging at high depths, written in 2004, Verheul describes a Triangle Walking Platform called the TRIPOD (22). He focuses mainly on the mechanics of the walking motion. In the concept a cutter suction head is mounted on the platform, but he states that more dredging methods are possible. This platform is connected with a vessel through a flexible pipe.

![Figure 3.25: TRIPOD by Verheul (42)](image)
3.6 Summary of techniques and solutions

In Table 3.1 a schematic summary of the dredging processes, methods and machines is given. In the top part of the table it is indicated which fundamental dredging processes (level 1) are used in the different the dredging methods (level 2). Also, the suitable soil types for every dredging method are indicated.

In the lower part of the table every cell represents a combination of a dredging method (level 2) and a repositioning method, leading to a possible machine (level 3). For every repositioning method the suitable soil types are given at the right-hand side. The numbers indicate the combinations that have been described in the preceding paragraphs. The legend below shows which machine corresponds to which number.

1. Cutter suction dredger (CSD)
2. Plain suction dredger
3. Dredge wheel/bottom disk cutter
4. Bucket ladder dredger
5. Dustpan dredger
6. Backhoe dredger
7. Auger dredger (pontoon/propelled)
8. Brush suction dredger
9. Water injection dredger (WID)
10. Agitation dredger
11. Mass flow excavation
12. Trailing suction hopper dredger (TSHD)
13. Free floating remove operated vehicle (ROV)
14. ROV on tracks (trencher)
15. Auger dredger on tracks
16. Punaise
17. Walking dredge with cutter head
18. Harbour branch Archimedean screw
19. Submersible dredge (Sloan)
20. STUMP (Batchelder)
21. Submersible sludge removal apparatus (Werner/Decker)
22. Mud slicer (Uiterwijk)
23. Stationary suction pit (Biendon)
24. Sediment removal system (Price)
25. TRIPOD (Verheul)
Table 3.1: Schematic overview of processes, dredging methods, existing machines and concepts
3.7 Level 3c: new unconventional concepts

In this paragraph several new concepts are presented that might be able to serve as an autonomous dredging device. Some ideas are more realistic than others, but like some concepts in the previous paragraphs, they are still useful to increase the scope.

3.7.1 Blobfish

Gronmij developed a concept called Blobfish: a submerged dredging apparatus that hovers just above the bed (Figure 3.26). It has the shape of a plate with in the centre a suction tube. The water-soil mixture flows from all directions towards this central suction mouth. This concept has a special position in this report since it was the incentive to start this research project. Unfortunately, after a number of scale tests it turned out that this idea is not feasible. In these experiments is has been observed that it is difficult to stabilize the device since the downward suction force is relatively strong. For the sake of completeness it is described briefly in this paragraph. In Appendix A: a more extensive description, theoretical considerations and scale tests can be found.

A key property of this concept is that the device is only connected to a flexible discharge and intended to work autonomously like a vacuum cleaner in a swimming pool. To ensure an equilibrium of vertical forces, chains are connected to multiple corners of the plate. The idea of these chains is that they stabilize the plate. When the plate moves upwards, more shackles leave the bottom, resulting in a downward force. When the plate moves down, more shackles lie down on the bottom, decreasing the downward force. This way, the chains should be able to compensate for a varying weight resulting from a varying flow density and under pressure.

Regarding free floating machines it can be noted that only dredging methods that rely on loosening methods without a cutting reaction force are suitable, since it is has proved difficult to provide a reaction force that is large enough. This means that mainly dredging methods that involve jets are suitable for mounting on a free floating device: water injection dredging, agitation dredging and mass flow excavation. These methods cause an upwards force that, although it can be small, still has to be compensated. However, this is expected to be much easier than compensating a downward suction force as is the case with the Blobfish design.

Propulsion: jets, propeller or external with cables
Dredging method: plain suction head

Figure 3.26: Artist impression Blobfish
3.7.2 Autonomous water injection dredging devices

In locations where WID can be applied it is possible to use a cloud of autonomous water injection dredging (AWID) devices. These AWID devices can operate autonomously and work together to locate and clean the areas that suffer from sedimentation. The dredged locations should be designed in such a way that there is a single location where the removed material can flow to. If this is not possible, a suction pit can be used.

There are multiple possibilities for the form factor of the autonomous WID devices: submerged and free floating like the Blobfish, on tracks or caterpillars or as a vessel like regular WID dredgers. A small vessel seems to be the most feasible choice, although conservative. When using a submerged device, it has become clear from the experiments with the Blobfish that it is very difficult to keep the device stable and in place. Also powering an underwater device with an engine will be more difficult.

In the past, a stationary suction pit in the Rotterdam Harbour has been tested. The principle was to use a pit with in the centre a stationary suction device. However, the test did not succeed since the mud quickly became too thick to dredge and a vertical canal was being formed through which almost all transport took place. It is possible to use AWID machines to keep the material in the pit in a more fluid condition such that it can be pumped away by the suction device. The Punaise, described in paragraph 0, was a stationary suction pit for sand.

**Propulsion:** vessel with propeller

**Dredging method:** water injection dredging

3.7.3 Bed supported auger scraper screw

When looking at devices that are in contact with the bed and use the bed for propulsion, first a distinction between sand, clay and mud has to be made. Almost all devices that rest on the bottom need a sufficiently stable bed and are therefore more suitable for beds consisting of sand or stiff clay. However, when a relatively thin layer of mud rests on a strong underground, it will also be possible to dredge this mud layer. It has to be taken into account that the mud does create extra friction, which increases the required propulsion power.

From practise it is known that an auger scraper screw is the most reliable method for mud dredging. This screw actively moves the mud from the ‘catch area’ towards the suction tube and prevents the system from clogging. Therefore this dredging method will be applied in this concept.

This auger can be placed on a construction that rests on the bed and moves using tracks or a system cables in combination with slides. This can be done in a linear direction, but also in the shape of an arc, similar to the way a cutter suction dredger works. In the last case, a flexible pipeline that can be put on a roll can be used to discharge the dredged material.

The version on tracks is inspired on the submersible crawler as presented in paragraph 3.4.5. A difference is that the cloud principle as discussed in the previous paragraph can also be applied. Also, the device has probably to be larger to be able to remove sufficient quantities of mud in an efficient way.

**Propulsion:** tracks or slides with cable system

**Dredging method:** auger scraper screw
3.7.4 Cloud of submarines
As an alternative to a regular hopper in a trailing suction hopper dredger, it is possible to use submarine-like devices to transport the dredged materials. An advantage of this is that the soil does not have to be transported vertically as is the case with regular TSHD’s and CSD’s. Also, the propulsion of submarines is energetically more efficient than that of surface floating vessels. The trade-off is the power required to fill and empty the submarine’s storage compartment and to control its location through buoyancy. Other disadvantages can be found in the more complex technologies required for submarines compared to vessels. Whereas the submerged position of such a device could be an advantage when looking at the hindering of vessels with a small draft, at the same time new safety issues arise. A collision between a submarine dredge and vessel can of course have large consequences.

The innovative part of this concept lays clearly the submarine way of transporting soil. For the dredging itself, it is suggested to use a regular combination of jetting and suction. Since this concept is submerged, it shows similarities with the Blobfish.

Propulsion: submarine with jets
Dredging method: suction/jetting

3.7.5 Underwater hovercraft with suction mouth
Another possibility is a hovercraft-like propulsion method that does not use air to keep a distance to the water surface (Figure 3.27), but water to keep a distance to a bed. Using chambers with a high water pressure an upward force can be established. Depending on the size of this force, any dredging method that does not require direct contact with the sub soil can be applied. Also systems that use suction are possible, since in theory the suction forces can be compensated by the upward force generated by the pressure chambers.

The drawback is that this system will probably only work with relatively strong beds. Very soft mud for example will most likely just flow away and is thus not suitable to create a closed chamber in which a high water pressure can be reached. It can be argued that this outflow of material is a positive effect if the device is viewed upon as a WID device. This way the water pressure underneath the hovercraft is both used to loosen the soil and the keep a distance between the device and bed. However, for now this concept will be regarded as a device that uses suction as dredging method.

Propulsion: underwater hovercraft with propellers
Dredging method: suction

Figure 3.27: A regular hovercraft. Source: wikipedia.org
3.8 Concept choice

In this paragraph three new concepts from the previous paragraph are chosen and based on the criteria presented in paragraph 2.2 a multi-criteria analysis will be done in order to find the concept that seems to be the most promising for the autonomous dredging of mud. This concept will be elaborated on in the next chapters.

3.8.1 Most promising concepts

Based on the ideas and concept as described in the previous paragraphs, the following concepts seem to be the most interesting:

1. Autonomous water injection dredging devices
   *The interesting point of this option is that no care has to be taken of the discharge of the dredged material, since it can flow freely away as a density current. Optionally, a stationary suction pit can be deployed.*

2. Bed supported auger scraper screw
   *This concept is interesting because it is known from practice that auger dredgers are very suitable to dredge mud. In order to make it autonomous and non-obstructing, a system with tracks or slides can be used for propulsion.*

3. Underwater hovercraft with suction mouth
   *In theory it is possible to build a hovercraft that works underwater. The hovercraft supplies a certain amount of upward force to compensate for the suction force.*

3.8.2 Multi-criteria analysis

A multi criteria analysis is carried out to select the most viable alternative. The criteria were already presented in paragraph 2.2. To take account of differences in importance between these criteria, to all of them a weight factor between 0 (lowest) and 5 (highest) is assigned. Subsequently, the three concepts receive a score between 0 and 5 for these criteria. Below the criteria are given together with an explication on their weight factor. Also the scores that are assigned to the different concepts are given and explained.

Stability

Both the erosion process and the positioning of the device itself have to be stable to assure a reliable operation. The importance of this criterion became clear during the experiments conducted on the Blobfish device and therefore it has a weight of 5.

The auger that rests directly on the bed is expected to be very stable. Also the auger scraper screw has been proved to be a stable way of removing mud. Therefore this concept gets a 5 points. The underwater hovercraft relies on a more complicated system that can have a negative impact on the stability, resulting in a score of 3. The WID solution that uses a vessel is also expected to be stable, but less than the auger that rests directly on the bed and therefore gets 4 points.

Maintainability

Any kind of mechanic device needs periodical maintenance and it is therefore important that it can be reached easily. However, the maintainability is not critical for the functioning of the device and therefore is given a weight factor of 4.

Both the auger and hovercraft solutions work close to the bed and therefore have to be lifted in order to undergo maintenance. This will take some time and effort and therefore these devices get a score of 3 on this point. The WID vessel operates on the water surface and is much easier to reach, resulting in a score of 4.
Reliability
The total dredging solution has to be reliable to minimize the risks of downtime. It should be sturdy and fool-proof. A weight factor of 4 is considered to be appropriate.

It is expected that the underwater devices are more complicated to construct and use and therefore bring larger risks with them. Also, there is less practical experience with these kind of devices and therefore the auger and hovercraft get a score of 3. WID is a proved technique and can therefore be assumed to be more reliable, resulting in a score of 4.

Safety
It is possible that other vessels or people get close to the dredging device. Because their safety has to be guaranteed, this is a very important criterion and therefore receives a weight of 5.

Contrary to the WID device, the auger and hovercraft cannot be seen from the water surface, which poses a risk. Also, the auger and hovercraft are possible equipped with a discharge line that can be hit by vessels. Since the auger is possibly moved by a system of cables that can be quite vulnerable, it receives a score of 3. The hovercraft gets 4 points and the WID concept receives a score of 5.

Required manpower
Even though a device can be called ‘autonomous’, with the current state of technology supervision is still required for safety reasons. Moreover, since there is no former experience with the studied concepts they will require extra attention. This criterion receives a weight of 3 because it is expected that in the future this factor will be of less importance.

For all concepts it is expected that at all times at least one supervisor has to be available to keep an eye on the works. Also a maintenance crew has to be available for emergency cases. There are no big differences on this point between the three concepts, so all receive a score of 3.

Costs
Another factor of importance are the costs of development and construction of the device. Since in the end all dredging works come down to a cost/benefit analysis, this is an important criterion. However, in this study the innovative aspects are more important than the costs of the devices and therefore this criterion gets a weight factor of 4.

It is expected that underwater devices are more complicated to develop and build than a vessel. Also the auger that simply rests on the bed is probably cheaper than the hovercraft solution. Therefore, the auger, hovercraft and WID receive scores of respectively 4, 3 and 5.

Dredging mud
Since the main goal of this report is to dredge relatively thin layers of mud from basins, the applicability of the concept for this material is of high importance and has a weight of 5.

A distinction has to be made between the possibility to work on a layer of mud and the effectiveness of the mud removal process itself. The auger can work on a mud layer, but the effectiveness depends on the thickness and strength of the layer. However, for removing mud an auger scraper has proved to be very effective, resulting in a 3. The hovercraft can have problems with mud since there is a chance that is will be blown away by the high water pressure. Also from an environmental point of view this solution poses risks, since it is possible that the mud is dispersed over the full water column. Altogether, a score of 2 is appropriate. The WID is very well suited for dredging of mud, in fact it is only applicable on relatively soft materials. So the WID concept receives a 5.
Debris in harbour
In a harbour it can be expected that the water and bed are spoiled with pieces of debris that possibly cause problems for dredging machines. This criterion receives a weight of 4.
A piece of rope can easily get stuck in the auger dredge and clog the machine. Therefore the auger gets a score of 2. The underwater hovercraft floats over the bed and pumps up the material. It is possible to protect the suction mouth with a grid to prevent debris from entering so it is better suited for a harbour than the auger and gets a score of 4. The WID does not suffer at all from debris in the water or on the bed and gets the maximum score of 5.

Workability
A key element of the autonomous concept is that the machine has to be able to work year round and. In other words, its workability has to be high. Therefore this criterion has a weight of 4.

It is expected that all three machines are capable of doing this, but that the underwater devices will be slightly less influenced by the weather than the WID. It has to be noted that in harbour basins the influences of the weather can be expected to be rather small. Nonetheless, the auger and hovercraft receive 4 points and the WID 3.

Energy supply
There are several types of energy sources possible to power the dredging devices. It is expected that in the future instead of fossil fuels, more and more renewable energy sources will be used. Another aspect is the ease with which the different concepts can be powered when looking at their location. Since the energy supply is not considered as a key element of this report, it receives a weight factor of 2.

In general it can be expected that powering an underwater device is more complicated than a device that is located on the water. Therefore the auger and hovercraft get a score of 3 and the WID 4.

Influence on traffic
Since port areas usually experience a large traffic load, the influence of the dredging operations on this traffic should be minimal. This criterion has considerable importance and therefore is assigned a weight of 3.

The auger and hovercraft excel on this point, since their location close to the bed does not hinder most vessels. Only for vessels with a high draft there is a risk that they enter the working area of the dredging device. This asks for good safety system to prevent collisions. The WID dredger floats on the water and can thus obstruct traffic, although the risk of accidentally bumping into it is smaller than with the other two concepts. Considering these pros and cons, all devices receive the same score of 4 points.

Conclusion
The scores as assigned above are summarized in Table 3.2. The following weighted scores follow from the analysis:
1. Auger: 347
2. Hovercraft: 333
3. AWID: 433

It can be concluded that the AWID device is the most viable solution. Therefore this concept will be studied in more detail in the next chapters.
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<th>Weight</th>
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<td>7 Dredging mud</td>
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<td>8 Debris in harbour</td>
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<td><strong>333</strong></td>
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Table 3.2: Multi criteria analysis on concepts for autonomous dredging devices
4 Water injection dredging

From the multi criteria analysis in the previous chapter it was concluded that the concept with a number of small water injection dredging devices can be regarded as the most promising. In this chapter this concept will be elaborated on in further detail. First, a short description and history of the WID technology will be given, together with some practical production figures. Hereafter the basics of jetting theory are described. What follows is a model of the full WID process that describes the processes from the jetting until the outflow phase. Important characteristics are the mud production and the height, velocity and travel distance of the generated density current.

4.1 Background of water injection dredging

Water injection dredging is a relatively new dredging method that was developed in the 1980s by engineer Van Weezenbeek, employed at HAM (now part of Van Oord). In 1987 the first water injection vessel Jetsed was constructed and the technique slowly became more and more popular for maintenance dredging. Many WID projects have been carried out around the world and nowadays WID can be regarded as a proved technology. Especially for maintaining safe navigation depth in smaller ports, WID can be an effective, economical and environmentally sound solution (12).

WID is related to agitation dredging, since both bring sediments into suspension. An important difference is that with agitation dredging the sediments are dispersed over the full water column and discharged using river or tidal currents, while with WID the fluidised sediment layer remains close to the bed (between 1 and 3 m) and is discharged as a density current.

When applying water injection dredging, a large volume of water is injected in the sediment under a low pressure through nozzles mounted on a horizontal injection bar that is being moved close to the bed. This fluidises the sediment by overcoming the cohesion in fine-grained (cohesive) soils or the internal friction in coarse-grained (granular) soils. Although fluidisation of granular material is possible, the material will settle quickly, resulting in short transport distances. Therefore, WID is best suited for removing clays, fine to coarse silts and mixtures of those materials called mud. The particles in these soil types are very fine and thus have a low settling velocity. A sediment-water mixture is formed that has a higher density than the surrounding water. Pressure differences originating from these density differences create a density current that flows to deeper water under the force of gravity, see Figure 4.1. In this density current the sediments move over a certain distance before they land on the bed again.

A density current is a moving fluidised sediment layer that is driven by forces caused by gravity and a pressure difference in the water. Also river currents, tidal forces or waves can help to transport the fluidised sediment. Since this density current can, under the right circumstances, flow out by itself, no sediment transport by a hopper, barge or discharge pipeline is necessary. This reduces the act of dredging to initiating a density current. Also there is no contact between the sediments and the machinery, reducing wear. This makes WID very cost efficient, but only for the cases where it is applicable.
4.2 Theory of water jets

Rajaratnam (25) gives descriptions of different jet layouts of which circular and plane wall jets are of interest for water injection dredging. The circular jet in free water can be used to model the vertical part of the jet flow. To model the development of the flow after it has bent into a horizontal direction, the plane wall jet theory has been used in the past and will therefore be described as well.

4.2.1 Outflow velocity and discharge of a jet

The velocity with which the water leaves the nozzle can be calculated with:

$$u_0 = \sqrt{\frac{2\Delta p}{\rho_w}}$$  \hspace{1cm} (4.1)

With

\begin{align*}
  u_0 & = \text{outflow velocity} \quad [\text{m/s}] \\
  \Delta p & = \text{pressure drop over nozzle} \quad [\text{Pa}] \\
  \rho_w & = \text{water density} \quad [\text{kg/m}^3]
\end{align*}

As an indication, an outflow velocity $u_0$ of about 10-15 m/s and a pressure drop of 100 kPa are realistic values that are given in literature.

The discharge of a single water jet can be calculated by multiplying (4.1) with the area of the jet and a nozzle discharge coefficient (35):

$$Q_n = C_d A_n u_0$$  \hspace{1cm} (4.2)

With

\begin{align*}
  Q_n & = \text{discharge of single jet} \quad [\text{m}^3/\text{s}] \\
  C_d & = \text{nozzle discharge coefficient} \quad [-] \\
  A_n & = \text{nozzle area} \quad [\text{m}^2]
\end{align*}
For regular water injection devices a discharge per meter width of 0.3 m$^3$/s is realistic (39). Also a distance of 30 cm between the centres of the jets is a value that is used in practice. These values can be used to estimate a realistic amount of nozzles $n_n$ of a certain size for a certain jet beam length. The total jet discharge is:

$$Q_0 = n_n Q_n$$  \hspace{1cm} (4.3)

### 4.2.2 Circular turbulent free jet

After leaving the nozzle, a turbulence layer is formed in which ambient water is accelerated and entrained by the jet water. The region in which the velocity is equal to the initial velocity $u_0$ becomes smaller with distance. At the position where the turbulence has penetrated till the central axis of the jet, the flow development region stops and the region of fully developed flow begins. In Figure 4.2 the used definitions are clarified.

![Image: Definition sketch of circular turbulent free jet. Source: Nobel (21)](image)

In the region of fully developed flow the velocity distribution is Gaussian and is given by:

$$u(s,r) = \sqrt{\frac{k_1 u_0 D_n}{2s}} e^{-\frac{s^2}{r^2}}$$  \hspace{1cm} (4.4)

With

- $u(s,r) =$ jet velocity at axial distance $s$ and radial distance $r$ [m/s]
- $k_1 = k_2 =$ entrainment coefficients [-]
- $u_0 =$ outflow velocity [m/s]
- $s =$ distance to nozzle along jet trajectory [m]
- $r =$ radial distance from jet axis [m]
- $D_n =$ nozzle diameter [m]
The flow is called fully developed when the velocity at the centreline is equal to the initial velocity $u_0$. This holds when $u(s = s_{sr}, r = 0) = u_0$, with $s_{sr}$ the distance to the nozzle. This leads to the following expression for $s_{sr}$:

$$s_{sr} = \frac{k D_n}{2} \approx 6.2 D_n$$ (4.5)

The value of the dimensionless entrainment coefficient $k$ is assumed to be 77 [1]. Using $\frac{77}{2} \approx 6$ and formula (4.4), the velocity in the centreline $u_{max}$ can be derived:

$$u_{max}(s) \approx 6 \frac{D_n}{s} u_0$$ (4.6)

The discharge in the fully developed flow region as a function of distance can be found by integrating (4.4). This formula has also been found by Albertson [1].

$$Q(s) \approx 0.32 \frac{s}{D_n} Q_n, \text{ for } s \geq s_{sr}$$ (4.7)

With $Q_n$ the discharge of a single nozzle. The rate of change of the discharge can be found by differentiating (4.7) with respect to the distance $s$:

$$\frac{dQ}{ds} \approx 0.081 \pi D_n u_0, \text{ for } s \geq s_{sr}$$ (4.8)

The height of the flow thickness along the jet trajectory can be approximated with (25):

$$b(s) = 0.10 s$$ (4.9)

With $b$ jet width for which $u > 0.5u_{max}$ [m]

4.2.3 Plane turbulent wall jet (smooth wall)

Rajaratnam (25) gives formulas for the flow of a plane jet along a wall. This jet model can be assumed to be more suitable to model the flow when its direction has changed from vertical to horizontal. The definitions of a smooth wall jet with a length $L_n$ and a nozzle height $h_n$ are given in Figure 4.3.

Figure 4.3: Definition sketch plane turbulent wall jet. Source: Nobel (21)
The formulas below are valid from the point where the flow has fully developed, which is:

\[ s \geq 14.7h_n \quad (4.10) \]

The decreasing velocity along the centreline of the jet, which is the maximum velocity, is given by:

\[ u_{\text{max}}(s) = 3.50 \sqrt{\frac{h_n}{s}} u_0 \quad (4.11) \]

This formula is valid for \( \frac{s}{h_n} \) up to about 100-140 (25).

Nobel (21) states that a wall jet is half a rectangular jet and derives the formula for the uniform flow velocity as a function of distance:

\[ u_u(s) = \frac{1}{2} \sqrt{2} u_{\text{max}} \quad (4.12) \]

The increasing flow height is given by:

\[ b(s) = 0.5h_n + 0.068s \quad (4.13) \]

This formula has been derived from experimental data as shown in Figure 4.4.

![Figure 4.4: Growth of the length scale for plane wall jets. Source: Rajaratnam (25)](image)

Rajaratnam gives the following expression for the growth of the discharge along the jet trajectory:

\[ Q(s) = 0.248 \frac{s}{h_n} Q_n \quad (4.14) \]
4.3 Modelling of water injection dredging

In the past, research has been done on the modelling of WID, mostly in the 1990s. Extensive investigations have been done by companies like HAM (owner of the patent, nowadays Van Oord) and the results are only partially available in the public domain. Therefore, in this report a simple model will be developed that combines the mechanisms as described in public available reports with new insights.

Usually, the WID process was divided into three sub-processes (14):
1. Jetting and loosening of mud
2. Entrainment and hydraulic jump
3. Outflow as density current

The first sub process treats the intrusion of the jet flow into the mud and the subsequent formation of a mud-water mixture. Since the flow of this initial mud-water mixture is considered to be supercritical, whereas the outflow as a density current is subcritical, somehow the flow has to transform in between. For this transformation the entrainment and hydraulic jump model has been introduced. However, the entrainment and hydraulic jump model has proved to be rather troublesome since not much is known about what happens during this intermediate phase. Therefore, this model is described in this report, but it is not used in the final model.

In the model 4 numbered locations are used as indicated in Figure 4.5:
0. At the outflow of the jets
1. Just after mixing the jet flow and the mud
2. Just before the hydraulic jump, at the end of the entrainment phase
3. Just after the hydraulic jump, at the start of the outflow phase

![Figure 4.5: Overview WID. Adapted from Kortmann (14)](image)

In the next paragraphs these three sub processes will be described in further detail. Together they form a model that can be used to describe the full water injection dredging process. This model will be implemented in a Matlab script that is used to make an estimation for the necessary bed slopes and resulting transport distance.

A stationary model will be used. This means that it is assumed that the geometry of the process does not change over time. The size and shape of the different parts of the density current are constant. This assumption is valid after a certain start-up period, since in the beginning the density current has to develop before it reaches the stationary state.
4.4 Jetting and loosening of mud

The first sub process consists of jetting the soil with a large amount of small vertical water jets. The result of this process is that the soil is loosened and mixed with the water into a suspension. When the jet flow reaches its maximum intrusion depth, the flow will bend and continues to flow in a nearly horizontal direction along the bed. At this point the flow velocity is high and the mixture flow is assumed to be supercritical.

![Figure 4.6: The jetting process. Adapted from Kortmann (14)](image)

4.4.1 Intrusion process

In previous reports by Schuurman (30) and Bronsvoort (6) estimations have been made on the intrusion and production of water jets. In these reports it is suggested to use a perpendicular intrusion model based on the thrust pressure of the jet and the cohesion of the mud. A schematic overview of this model is given in Figure 4.7. The jet water intrudes in mud up to a certain depth that depends on the characteristics of the mud and the water flow velocity.

![Figure 4.7: Intrusion of jet in mud. Adapted from Schuurman (30)](image)
Schuurman states that a jet intrudes into the soil up to a depth where the thrust pressure is equal to about 6 times the cohesion of the soil:

\[ p_{\text{intr}} \approx 6c_u \]  \hspace{1cm} (4.15)

With

\[ p_{\text{intr}} = \text{jet pressure at intrusion depth} \quad \text{[Pa]} \]
\[ c_u = \text{cohesion of soil} \quad \text{[Pa]} \]

The cohesion of the soil has to be determined for the specific soil that is dredged since it can show large differences for different soils, as described in paragraph 2.3.3.

The velocity in the centreline of a circular jet in water is given by (4.6):

\[ u_{\text{max}}(s) = 6 \frac{D_o}{s} u_0 \]  \hspace{1cm} (4.16)

The pressure at the centreline of the jet can be expressed as a function of \( u_{\text{max}}(s) \):

\[ p(s) = \frac{1}{2} \rho \left[u_{\text{max}}(s)\right]^2 \]  \hspace{1cm} (4.17)

Together with (4.16) the jet pressure can now be written as a function of the intrusion distance \( s \):

\[ p(s) = 18 \rho u_0^2 \frac{D_o^2}{s^2} \]  \hspace{1cm} (4.18)

Now the relation between the cohesion of the mud and the penetration depth can be derived by combining formulas (4.15) and (4.18):

\[ 6c_u = 18 \rho u_0^2 \frac{D_o^2}{s^2} \]  \hspace{1cm} (4.19)

This leads to an intrusion depth \( s_{\text{intr}} \) as a function of cohesion \( c_u \) and jet parameters \( u_0 \) and \( D_o \):

\[ s_{\text{intr}} = \sqrt{\frac{3 \rho u_0^2 D_o^2}{c_u}} \]  \hspace{1cm} (4.20)

In this model it is assumed that the water density does not change during the process. However, in reality this density increases due to the mixing with soil. Also it is assumed that a soil layer with thickness \( s_{\text{intr}} \) is available. In practise, it is very well possible that the soil layer is thinner and in that case the actual layer thickness has to be used to determine the production.
4.4.2 Production

Experiments by Schulting (29) and Veen (40) show that for a lower hauling velocity more sediment is removed per unit of area. For the influence of the moving velocity of the jet on the intrusion depth no complete formulation exists, except for the experience that the intrusion depth is inversely proportional to the hauling velocity. This observation implies that the soil production per unit of time does not depend on the hauling velocity.

This means that the amount of soil produced during the jetting process can be expressed in two ways:
1. The production per unit time, which has been found to be independent of the hauling velocity
2. The production per unit area, which is does depend on the hauling velocity

The production per unit time of the in-situ soil in m³/s can be calculated as the product of the intrusion depth, jetting width and the hauling velocity of the jet. Again, a soil layer with a thickness of the intrusion depth has to be available.

\[ Q_{in} = s_{inr} \cdot w_{dr} \cdot v_{dr} \]  \hspace{1cm} (4.21)

With
\[ Q_{in} = \text{in-situ production} \quad [\text{m}^3/\text{s}] \]
\[ s_{inr} = \text{intrusion depth} \quad [\text{m}] \]
\[ w_{dr} = \text{width WID beam} \quad [\text{m}] \]
\[ v_{dr} = \text{hauling velocity dredge} \quad [\text{m/s}] \]

This estimation can be improved by incorporating the widening of the jet beam when entering the mud, as done by Schuurman. However, it is questionable if this will lead to a better estimation since this way the complex flow of the jet in both water and mud is still not described accurately. For a better description the more sophisticated theory of Nobel (21) could be used. Since this theory is much more complicated, in this report the simple formulation will be used, as derived above.

Since the jets are located at a certain distance from each other, not every area of the mud is expected to undergo the same amount of erosion. Also, it is possible that mud particles are too large and heavy to flow away and will settle quickly. Therefore a jetting factor \( f_{jet} \) between 0 and 1 is introduced that reduces the production as calculated with (4.21).

\[ Q_{in} = f_{jet} \cdot s_{inr} \cdot w_{dr} \cdot v_{dr} \]  \hspace{1cm} (4.22)
4.4.3 Situation directly behind jet (location 1)
It is assumed that after reaching the intrusion depth, the flow of mud and water bends towards a horizontal direction with its bottom at the intrusion depth. The discharge from the separate jets is assumed to be spread over the full jet beam width, also including the gaps between the jets, see Figure 4.8.

Now the situation just after the jet can be calculated. This is location 1 in Figure 4.9.

Because the jets are located close to the mud layer, it is assumed that there is no entrainment of water over the intrusion distance. So after the bend the jet discharge is assumed to be still $Q_0$. The total discharge just behind the jet is equal to:

$$Q_i = Q_0 + Q_n$$ (4.23)

The mud is mixed with the jet flow that has a much higher velocity and is thus accelerated due to the horizontal momentum of the jet flow. Assuming that both components mix perfectly and that there is conservation of momentum, the velocity and flow height at location 1 can be calculated. To do so, a flow and discharge relative to the earth are used. This gives for the jet flow a velocity of $u_0 - u_{aw}$ and for the discharge $n_c C A_s (u_0 - u_{aw})$. Mud enters the system with an absolute velocity of $u_{aw}$ and a discharge of $Q_n$. Using these values leads to the following velocity directly behind the jet:

$$u_s = \frac{\rho_{aw} Q_n u_{aw} + \rho_w n_c C A_s (u_0 - u_{aw})^2}{\rho_i Q_i}$$ (4.24)

The height of the layer at location 1 becomes:

$$h_1 = \frac{q_1}{u_s}$$ (4.25)
With $q_1$ the specific discharge at location 1:

$$q_1 = \frac{Q_1}{w_{gr}} \quad (4.26)$$

The density of the fluidised soil mixture is equal to:

$$\rho_1 = \frac{Q_o \rho_{s,gr} + Q_w \rho_w}{Q_o + Q_w} \quad (4.27)$$
4.5 Entrainment and hydraulic jump

This paragraph deals with the processes that play a role between the jetting phase and the outflow phase. It is difficult to determine what exactly happens at this point. In the past it has been suggested by Kortmann (14) to model the flow as a plane wall jet that entrains water and thus increases its thickness. At a certain point, a hydraulic jump takes place during which the flow changes from supercritical to subcritical, together with an increase in flow height.

This model will be worked out, although it has to be noted that it is questionable whether it corresponds with the processes that occur in practise. Therefore, this model is not used in the final Matlab script. The sole purpose is to give insight in a possible way of modelling of this process that can be improved in future research.

At the end of this paragraph an alternative, simple way of modelling this sub process is presented. This model will be used in the Matlab script.

![Figure 4.10: The sub process entrainment and hydraulic jump. Adapted from Kortmann (14)](image)

4.5.1 Overview and purpose of sub process

Due to turbulence in the layer between the fast flowing mixture and the (almost) stationary upper layer, entrainment of water takes place. This results in an increasing layer thickness, increasing discharge and decreasing flow velocity. At some point, it is assumed that the supercritical flow turns into a subcritical flow through an internal hydraulic jump. To calculate at what distance from the jet this jump takes place, the average flow velocity, layer thickness and density will be calculated as a function of the distance from the jet beam. Using these values also a conjugated thickness can be calculated as a function of the distance. At the location where this conjugated thickness is equal to the equilibrium thickness for the outflow, the jump will occur. This equilibrium thickness can be calculated using an equilibrium between the gravity force and bed friction.

4.5.2 Entrainment during horizontal flow

When flowing away from the jet beam, the flow is instable because of turbulence in the zone between the lower mixture layer and upper water layer. This results in the flow being mixed with the non-moving water layer. During this entrainment process, the mixture flow $Q$ increases and the velocity $u$ decreases.

The formulas for the plane wall jet are used, as described in paragraph 4.2. These formulas are repeated here, with the assumption that the 'jet opening' is equal to the initial flow height $h_1$. Also, the initial discharge is assumed to be $Q$. Since at this moment the flow is nearly horizontal, except for a relatively small bottom slope, the coordinate along the jet trajectory will be called $x$ instead of $s$. 

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The increasing flow height as function of the distance $x$ can be calculated with:

$$h(x) = 0.5h_1 + 0.068x$$  \hspace{1cm} (4.28)

The decreasing velocity along the centreline, which is a maximal velocity, is given by:

$$u_{\text{max}}(x) = 3.50 \sqrt{\frac{h_1}{x}}$$  \hspace{1cm} (4.29)

As mentioned before, this formula is valid for $\frac{x}{h_1}$ at least up to 100-140 (25), so for this application it is probably used to its limit.

These formulas are valid from the point where the flow has fully developed, which is:

$$x \geq 14.7h_1$$  \hspace{1cm} (4.30)

Nobel gives for the average velocity:

$$u_{av}(x) = 2.4u_1 \sqrt{\frac{h_1}{x}}$$  \hspace{1cm} (4.31)

Rajaratnam gives the following expression for the growth of the discharge over distance:

$$Q(x) = 0.248 \frac{x}{h_1} Q_1$$  \hspace{1cm} (4.32)

Different combinations of the formulas are possible to calculate, $u$, $h$ and $q$. The following calculation method has been found to give the best results:

1. Calculate layer height with (4.28)
2. Calculate average velocity with (4.31)
3. Calculate $q$ and $Q$ using the layer height and average velocity

With these expressions the change from initial flow height $h_1$ to $h_2$, initial flow velocity $u_1$ to $u_2$ and initial discharge $Q_1$ to $Q_2$ can be calculated. In Figure 4.11 the locations 1 and 2 are indicated. In the next paragraphs the exact position of location 2 will be determined.

---

Figure 4.11: Overview of entrainment. Adapted from Kortmann (14)
4.5.3 Criteria for stability of interface layer

Between the layers in a two-layer system an interface layer exists in which the flow velocity gradually changes. Just behind the jet the difference in flow velocities between the two layers is large. Since an interface layer has not developed yet, the velocity gradient is very steep. The thickness of the interface layer will grow as the jet water moves further away from the jet. When the thickness of this interface layer does not change anymore, the flow is called stable. In this paragraph a requirement will be developed with which one can determine whether the flow is stable or not.

**Interface layer thickness**

Kranenburg (15) gives an empirical formula for the thickness of the interface layer:

\[
\delta = 0.32 \frac{\Delta U^2}{\varepsilon g}
\]  

(4.33)

With

- \( \delta \) = thickness interface layer [m]
- \( \Delta U \) = velocity difference between the layers [m/s]
- \( \varepsilon = \frac{\rho_m - \rho_w}{\rho_w} \) = relative mixture density [-]

The thickness of this interface layer should be smaller than the total water depth to be applicable. A practical rule of thumb is:

\[
\frac{\Delta U^2}{\varepsilon ga} < 1
\]  

(4.34)

**Richardson gradient**

Also the Richardson gradient is useful to determine the stability of an interface layer. The Richardson number gives the ration between the energy required to move a water particle with a certain density over a certain height, in this case the interface layer thickness, and the kinetic energy that is available to do so.

The Richardson gradient is given by:

\[
R_i = -\frac{g}{\rho \left( \frac{du}{dz} \right)} > \beta
\]  

(4.35)

Experiments have shown that for a Richardson gradient larger than a value \( \beta \) between 0.3-0.4 a stable current occurs. In case the upper layer in a two layer current is stationary, a lower layer velocity \( u_{low} \) and interface layer thickness \( \delta \), the following approximations can be made:

\[
\frac{du}{dz} \approx \frac{u_{sw}}{\delta}
\]  

(4.36)

\[
\frac{d\rho}{dz} \approx \frac{\varepsilon \rho}{\delta}
\]  

(4.37)

Together with (4.35) the stability criterion can be written as:
\[ \beta Fr_\beta^2 < \frac{\delta}{h_i} < 2 \quad (4.38) \]

With the internal Froude number:

\[ Fr_\beta = \frac{u_{low}}{\sqrt{g h_{low}}} \quad (4.39) \]

The number 2 at the right hand side of equation (4.38) gives the upper thickness of the interface layer, for which the full density current is part of the interface layer.

Now the stability criterion (4.38) can be rewritten as a limit for the internal Froude number:

\[ Fr_\beta < \frac{2}{\sqrt{\beta}} \quad (4.40) \]

So the internal Froude number should be smaller than about 2.2 \( \cdot \) 2.6 for the flow to be stable.

### 4.5.4 Internal hydraulic jump

Kranenburg (15) describes the theory of the internal hydraulic jump, which is (in this case) a transition form a subcritical to a supercritical flow. He gives an expression for the ratio of the thickness of the layer before and after the jump. The layer thickness after the jump is called the conjugated thickness of the layer thickness before the jump. Using a momentum balance the conjugated thickness can be calculated.

In the derivation a coordinate system and balance area are used that move along with the jump, so that within the balance area the shape of the jump and the quantity of mass and momentum do not change over time. Also, the following assumptions are made:

- Friction between the upper and lower layer and between the lower layer and the bottom can be neglected because of the short distance over which the jump takes place.
- External effects are not regarded.
- Only two dimensions are taken into account, changes in width are not accounted for.

In Figure 4.12 the used variables are indicated. The quantities in the top layer have index number 1 and the quantities in the bottom layer have index number 2. This differs from the notation used elsewhere to indicate the locations as Figure 4.5. Quantities after the jump are indicated with an apostrophe (\' ) mark. Because of the moving coordinate system, the velocities relative to the earth are lowered by the velocity \( c \) of the coordinate system to make them relative to the hydraulic jump. This velocity \( c \) is equal to the velocity of the dredger.
This leads to a system of equations from which a relationship between the layer thickness at both sides of the jump follows:

\[
\frac{q_{1c}^2}{\varepsilon g h_1 \frac{h_1 + h_2}{2}} + \frac{q_{2c}^2}{\varepsilon g h_2 \frac{h_1 + h_2}{2}} = 1
\]  

(4.41)

With

- \(q_{1c}\) = specific discharge upper layer per unit width rel. to moving coordinate system \([\text{m}^2/\text{s}]\)
- \(q_{2c}\) = specific discharge lower layer per unit width rel. to moving coordinate system \([\text{m}^2/\text{s}]\)
- \(h_1\) = layer thickness upper layer before jump \([\text{m}]\)
- \(h_2\) = layer thickness lower layer before jump \([\text{m}]\)
- \(h_1'\) = layer thickness upper layer after jump \([\text{m}]\)
- \(h_2'\) = layer thickness lower layer after jump \([\text{m}]\)
- \(\varepsilon\) = relative density mixture \([-\]]

The first term in (4.41) can be neglected since for low flow velocities in the upper layer it only leads to a difference of several per cent. This results in the following relation for the ratio between the thickness of the lower layer before and after the jump (16):

\[
\frac{h_1'}{h_2'} = \frac{1}{2} \left( \sqrt{1 + 8Fr^2} - 1 \right)
\]  

(4.42)

In Appendix C: the following relationship for the conjugated layer thickness is derived from formula (4.41):

\[
h_{eq} = \sqrt{\frac{(k_1 + k_2)Q^2}{\varepsilon g l_0 w_{cr}}}
\]  

(4.43)

With \(k_1\) and \(k_2\) friction factors that for turbulent flow are given by Kranenburg. These are also described in Appendix C.
4.5.5 Modelling of processes around hydraulic jump

In the modelling of the hydraulic jump, three different locations are used, numbered 1, 2 and 3. In the picture below these locations are indicated.

![Diagram of hydraulic jump](image)

**Figure 4.13: Overview entrainment and hydraulic jump. Adapted from Kortmann (14)**

Using the following calculation steps the location of the hydraulic jump can be determined:

1. Calculate layer thickness and average velocity as a function of x using respectively (4.28) and (4.31)
2. Calculate the discharge q
3. Calculate conjugated thickness as function of x using (4.42)
4. Calculate equilibrium depth as function of x with (4.43)
5. At the location where 2 and 3 are equal a hydraulic jump occurs
6. Now the discharge, velocity and height at the beginning of the density current (location 3) are known and can be used in the outflow calculation.

4.5.6 Simple entrainment model

As described in the introduction of this paragraph, the above derived method to model the entrainment and determine the location of the hydraulic jump is considered as a theoretical model that needs to be verified and improved in future research.

To come to an simple and usable model, it is assumed that the amount of entrainment that takes place after the jet flow and mud have mixed can be indicated as a multiple of the jet discharge $Q_0$. This leads to the following expression for the total discharge that can be used for the outflow model:

$$Q_z = Q_m + (1 + \text{f}_{\text{entr}})Q_0$$  \hspace{1cm} (4.44)

With

$$\text{f}_{\text{entr}} = \text{factor to taking into account entrainment} \hspace{1cm} [-]$$

With this discharge $Q_z$ also other properties of the flow after entrainment can be calculated. As stated before, the hydraulic jump theory will not be applied in the model. Instead, an outflow model will be used that gives a value for the flow height and velocity at location 3. A topic that can be looked upon further in the future is whether and how it is possible to connect a model that describes the transition from supercritical to subcritical flow and an outflow model.
4.6 Outflow as density current

After the supercritical density flow has been transformed to a subcritical flow through a hydraulic jump or other mechanism, it will flow away as a density current. At this moment, the water-sediment mixture can be modelled as a suspension. It can be assumed that the interface between the liquid mud layer and the water layer above it is calm since the velocity differences are relatively low. Therefore it is assumed that no entrainment of surrounding water takes place at this stage.

The density difference between the fluidised layer and the surrounding water forces the fluidised soil layer to flow. This flow is comparable to that of a salt water wedge that can travel along a river bed for kilometres upriver, even though the river current opposes its flow direction.

While the fluidised soil layer is moving as a density current, the soil particles will settle and the thickness of the layer decreases. This causes the flow to slow down until all particles are settled.

In literature different methods have been found to model a density current that is generated by water injection dredging. Two models are given further attention in this paragraph: the 2-layer model of Kranenburg (15) and a simple model using the Chézy equation.

Other methods have been described by Cordi (8), Verweij (43), Verhagen (41) and Van Rijn (39). These are described in Appendix B.

4.6.1 Real world values

In order to be able to judge whether the calculated values do make sense, below several values found in literature are presented.

Van Rijn (39) states that for a density current practical values are:
- \( h = 0.1 \) to \( 1 \) m
- \( u = 0.1 \) to \( 1 \) m/s
- \( c = 50 \) to \( 200 \) kg/m\(^3\)

Verhagen (41) gives the following values:
- \( h = 1 \) to \( 3 \) m
- \( v_{dr} = 0.5 \) m/s

In a report on WID in the port of Hamburg, Netzband (20) gives the following values:
- \( h = 1 \) to \( 3 \) m
- \( u = 0.3 \) to \( 1 \) m/s
- Transport distances from few meters to few kilometres

As shown in paragraph 2.3.2, fluid mud has a density below approximately 1150 kg/m\(^3\).
4.6.2 Kranenburg 2-layer model

Kranenburg (15) derives the formulas that describe a multi layer flow situation. He uses conservation of mass and momentum to formulate a system of 2n equations, with n the number of layers, to solve for 2n unknowns: the layer thickness and velocity for every layer.

For the special case of a two-layer system equations are derived. The following assumptions are made (23):

- Stable stratification: a less dense fluid above a denser fluid
- Hydrostatic pressure: the horizontal length scale is much larger than the vertical length scale
- Constant density in the layer in both space and time
- No mass transfer between layers
- Horizontal velocity constant in each layer

The situation is schematized in Figure 4.15. Here the subscripts 1 and 2 stand for the different layers, and not for the different locations as shown in Figure 4.5.

![Figure 4.15: Overview 2-layer model. Source: Pietrzak (23)](image)

For this special case of a situation with two layers, the system reduces to two continuity equations and two equations of motion. For the case of a stationary flowing density current (layer 2) and non-flowing water layer (layer 1) a backwater curve equation can be obtained. In Appendix B: a full derivation is given.

\[
\frac{\partial h_2}{\partial x} = \frac{1}{\sigma p g} \left( \frac{\tau_1 + \tau_2 - \tau_2}{h_1 + h_2} \right) + \frac{\partial h_b}{\partial x} - \frac{q_2^3}{\sigma g h_2^3} - 1
\]  \hspace{1cm} (4.45)

With

- \( h_b \) = bed height \([m]\)
- \( h_1 \) = thickness upper layer \([m]\)
- \( h_2 \) = thickness lower layer \([m]\)
- \( \tau_1 \) = friction between upper and lower layer \([Pa]\)
- \( \tau_2 \) = friction between lower layer and bed \([Pa]\)
- \( q_2 \) = specific discharge lower layer \([m^2/s]\)
This is a differential equation for the thickness of the lower layer $h_2$. With $\frac{\partial h_2}{\partial x}$ being negative for a slope that is downward in positive x-direction. Kranenburg (15) also gives this formula, but in the more general case for which $q_1 \neq 0$ (formula 4.7 in his lecture notes) and Kortmann (14) refers to it. Pietrzak (23) calls it the internal backwater curve.

Because the upper layer $h_1$ is relatively large compared to the lower layer $h_2$ and the friction between the two layers $\tau_1$ is expected to be lower than the friction $\tau_2$ between the lower layer and bed, it is reasonable to neglect the $\frac{\tau_1}{h_1}$ term in (4.45). The simplified equation thus becomes:

$$\frac{\partial h_2}{\partial x} = \frac{\tau_1 - \tau_2 + \partial h_2}{h_1 u \rho g} \frac{q_2^2}{h_2^3}$$

This equation can be used to calculate the rate of change of the thickness of the density current.

**Issues of 2-layer model for WID**

The 2-layer model has several issues which make it less suitable for application on a WID calculation.

Since this model results in a backwater curve equation, a downstream boundary condition is necessary. From this downstream point, a calculation should be made in upstream direction. This procedure is similar to that for the modelling of the water level in a river. Far upstream an equilibrium situation occurs where an equilibrium layer thickness occurs. For WID no clear downstream boundary condition can be indicated, since this position is not known by definition: it has to be calculated how far the density current will move on a bed with a certain slope. Such a boundary condition can be obtained when assuming a deep pit at a fixed location, but this is not the desired situation.

Since the goal is to calculate at what position the density current will stop or disappear, so when its height and velocity are zero, it can be argued that for the boundary condition both a zero height and velocity can be used. However, this is not possible since it would not give a result for the backwater curve.

Another problem with equation (4.46) is that when the term $\frac{q_2^2}{\rho \rho g h_2^3} = Fr \approx 1$, the denominator is close to zero and $\frac{\partial h_2}{\partial x}$ goes to infinity. This means that for a flow that is on the edge of supercritical and subcritical, no reliable backwater curve can be calculated.

Because of these reasons, it has been decided not to use this model and use a more straightforward approach to model the density current.
4.6.3 Chézy model

The outflow model as presented in the previous paragraph shows problems when it is translated into a practical mathematical model. Therefore it is proposed to use a more simplistic model, based on the Chézy formula. This approach is based on an equilibrium depth and velocity. A small bed slope results in a low velocity and with \( q = uh \) a large layer thickness. In the extreme case of a horizontal bed the layer thickness goes to infinity, which of course is unrealistic. What will happen in this case is that the material will start to flow because of the density difference compared to that of the adjacent water. Therefore, for the model to be realistic an upper limit to the layer density has to be formulated. This results in a lower limit for the bed slope.

The flow velocity can be calculated with:

\[
\begin{align*}
  u &= C^* \sqrt{\varepsilon gh_{\text{bed}}} \quad (4.47)
\end{align*}
\]

With
\[
\begin{align*}
  C^* &= \text{dimensionless Chézy coefficient} \quad [-] \\
  \varepsilon &= \text{relative mixture density} \quad [-] \\
  i_b &= \text{bed slope} \quad [-]
\end{align*}
\]

The dimensionless Chézy coefficient can be calculated from the Chézy coefficient \( C \) with:

\[
\begin{align*}
  C^* &= \frac{C}{\sqrt{g}} \quad (4.48)
\end{align*}
\]

Winterwerp and Van Kesteren (51) give for the Chézy coefficient different values for different locations, but all are within a range of 60-110 m\(^{1/2}\)/s.

The following relation holds for the discharge:

\[
\begin{align*}
  q &= uh \quad (4.49)
\end{align*}
\]

Substituting (4.49) in (4.47) results in:

\[
\begin{align*}
  u &= C^* \frac{2}{3} \left( \varepsilon g q_{\text{bed}} \right)^{\frac{1}{3}} \quad (4.50)
\end{align*}
\]

Using formula (4.50) the average velocity of the density current with a certain discharge along a certain slope can be calculated. With formula (4.49) the layer height is also known. Now, when assuming a settling velocity, the decrease of the discharge can be estimated. This leads to new, lower values for the velocity and layer height. This way, an estimation can be made for the distance over which the current travels. The exact procedure will be described in paragraph 4.7.
4.6.4 Note on the length of the cloud

The length of the cloud depends on several parameters. In this paragraph a qualitative description of these parameters is presented.

![Jet beam diagram](image)

**Figure 4.16: Outflow definitions. Adapted from Kortmann (14)**

First of all, the mixture density plays an important role. To study the influence of the hauling velocity on the mixture density, an expression for the height of the hypothetical water layer that is added per unit of area can be calculated with:

\[
h_{\text{wat}} = \frac{Q_0}{w_{dr} v_{dr}}
\]

(4.51)

With

- \(Q_0\) = total jet discharge [m\(^3\)/s]
- \(w_{dr}\) = width of the dredge beam [m]
- \(v_{dr}\) = hauling velocity of the dredge [m/s]

As described in paragraph 0, it is assumed that the amount of soil entering the system per unit of time \(Q_{in}\) is independent of the hauling velocity \(v_{dr}\).

Now the situation can be described for both a low and a high hauling velocity to get insight into the behaviour of the system for these cases. When the hauling velocity is high, a certain amount of water is spread over a large area, resulting in a lower amount of water added per unit area. Since \(Q_{in}\) does not depend on the velocity, this results in a mixture with a high density \(\rho_m\). A higher density relative to the water means that the driving force of the density current is higher, resulting in a higher flow velocity of the density current. So in case of a high hauling velocity, the length of the cloud is relatively large. It has to be noted that there is a upper limit on the hauling velocity, since the mud density should be below a threshold value of 1150 kg/m\(^3\) to be fluid.

For a small hauling velocity the opposite holds: more jet water is added per unit area, resulting in a mixture with a lower density \(\rho_m\). The density current will travel slower due to the lower driving force, resulting in a shorter cloud.

In Figure 4.17 this is visualised. Along the trajectory of the dredger at multiple points in time the trajectory of a piece of soil is given. It is assumed that this piece of soil moves with a single density current velocity \(u_{dc}\). Another assumption is that the settlement time \(t_{sett}\) is equal for both situations. Although these are all rough estimates, they help giving an quantitative indication of the cloud length.

It has to be noted that all of this contradicts Kortmann’s (14) statement that the right side of the cloud moves with the same velocity as the jet at the left side of the cloud.
Figure 4.17: Length of the cloud for a slow and fast moving jet
4.6.5 Settling, deposition and sedimentation

Under the force of gravity the particles in the mud move downwards and after a while they settle on the bed. The settling velocity of a single particle can be calculated and depends on the size and specific density of the particle and the viscosity of the mixture. Since there are many particles in fluidised mud, they prevent each other from settling. This effect is called hindered settling. Nasner (19) notes that the settling process is also slowed down by bacteria that are attached to the fine sediments and produce slimes.

The process results in both deposition and sedimentation. Winterwerp and Van Kesteren (51) give the following definition of these processes:

- Deposition: the gross flux of cohesive sediment flocs on the bed
- Sedimentation: the net increase in bed level, the deposition rate minus the erosion rate

The following elaboration on the settling velocity consists a summary of the main points on this topic from the dissertation of Winterwerp (50).

**Principles**

The net result from erosion and sedimentation leads to a certain velocity with which the bed level moves. Depending on the difference between erosion and sedimentation, this can both be an erosion or sedimentation velocity. The general expression for the velocity with which the bed moves is:

\[
v_{\text{bed}} = \frac{E - S}{\rho_s(1 - n_0 - c_b)}
\]  

(4.52)

With

- \(v_{\text{bed}}\) = erosion or sedimentation velocity bed [m/s]
- \(E\) = pick-up flux [kg/m²s]
- \(S\) = settling flux [kg/m²s]
- \(\rho_s\) = grain density [kg/m³]
- \(n_0\) = porosity of original bed [-]
- \(c_b\) = near-bed concentration [-]

However, in this report it is assumed that in the density current the deposition is far larger than the erosion and thus erosion is neglected. This leads to the following formula for the sedimentation velocity:

\[
v_{\text{sed}} = \frac{S}{\rho_s(1 - n_0 - c_b)}
\]  

(4.53)

**Settling flux**

The settling flux is caused by the sedimentation of sand grains and is given by:

\[
S = \rho_s v_{\text{set}} c_b
\]  

(4.54)

With

- \(v_{\text{set}}\) = settling velocity [m/s]
Single particle settling

Winterwerp (50) gives the following formula for the settling velocity of a single mud floc:

\[
v_{\text{set},r} = \frac{\alpha}{18\beta} \left( \frac{\rho_s - \rho_w}{\mu} \right) g D_p^{3-n_f} \frac{D_n^{-1}}{1 + 0.15 \text{Re}_p^{0.687}}
\]  \( (4.55) \)

With

\[
\begin{align*}
\alpha &= \text{particle shape coefficient} \quad [-] \\
\beta &= \text{particle shape coefficient} \quad [-] \\
\mu &= \text{dynamic viscosity} \quad [\text{Pa.s}] \\
D_p &= \text{diameter primary mud particles (typically 4 μm)} \quad [\text{m}] \\
D &= \text{floc diameter} \quad [\text{m}] \\
\text{Re}_p &= \frac{v_{\text{set},r} D}{\nu} = \text{particle Reynolds number} \quad [-] \\
\nu &= \text{kinematic viscosity} \quad [\text{m}^2/\text{s}] \\
n_f &= \text{fractal dimension (about 2)} \quad [-]
\end{align*}
\]

For spherical \((\alpha = \beta = 1)\), massive \((n_f = 3)\) particles in the Stokes regime \((\text{Re}_p \ll 1)\), the Stokes formula for settling particle is obtained:

\[
v_{\text{set},r} = \frac{(\rho_s - \rho_w) g D^2}{18 \mu}
\]  \( (4.56) \)

In Figure 4.18 observations of settling velocity are plotted together with formulas (4.55) and (4.56), referred to as respectively ‘eq. 4.12’ and ‘Stokes’. For formula (4.55) the used constants are:

\[
\begin{align*}
\alpha &= 1, \quad D_p = 4 \mu\text{m}, \quad \rho_s = 2650 \text{ kg/m}^3, \quad \rho_w = 1020 \text{ kg/m}^3 \quad \text{and} \quad \mu = 10^{-3} \text{ Pa.s}
\end{align*}
\]

It can be observed that formula (4.55) fits the data quite well for flocs smaller than a few mm.

Figure 4.18: Relation between settling velocity and floc size. Source: Winterwerp (50)
This formula is the special case for laminar flow for the following general formula combined with the appropriate drag coefficient:

\[ \nu_{set,r} = \sqrt{\frac{4\Delta gD}{3C_D}} \]  \hspace{1cm} (4.57)

With \( C_D \) the drag coefficient that can be described with different empirical formulas, depending on the flow regime.

\[
C_D = \begin{cases} 
\frac{24}{\text{Re}_p} = \frac{24\nu}{wD} & \text{Re}_p \leq 1 \\
\frac{24}{\text{Re}_p} + \frac{3}{\sqrt{\text{Re}_p}} + 0.34 & 1 \leq \text{Re}_p \leq 2000 \\
0.4 & \text{Re}_p \geq 2000 
\end{cases} \]  \hspace{1cm} (4.58, 4.59, 4.60)

**Hindered settling**

For very low mud concentrations below 1 g/l the theoretical settling velocity increases due to the flocculation effect. For higher concentrations above 10 g/l, which will occur during WID, hindered settling takes place and the settling velocity will be reduced.

Hindered settlement is the occurrence of a lowering of the settling velocity due to the water that flows upwards while particles move downwards. So the actual fall velocity is lower than that of a single grain. This effect can be described with the classical formula by Richardson and Zaki:

\[ \nu_{set} = \nu_{set,r} (1 - \phi)^n \]  \hspace{1cm} (4.61)

With

- \( \phi \) = volumetric concentration of mud flocs \([-\]
- \( n \) = exponent \([-\]

The volumetric concentration \( \phi \) is related to the volumetric concentration \( c \) [kg/m\(^3\)] with:

\[ \phi = \frac{\rho_s - \rho_w}{\rho_f - \rho_w} \frac{c}{\rho_s} \]  \hspace{1cm} (4.62)

The exponent \( n \) is a function of the particle Reynolds number \( \text{Re}_p \). Richardson and Zaki give a relation for \( n \) which is valid for \( 0.2 < \text{Re}_p < 10^5 \) and \( 0.04 < c < 0.55 \):

\[ n = \frac{4.7 + 0.41\text{Re}_p^{0.75}}{1 + 0.175\text{Re}_p^{0.75}} \]  \hspace{1cm} (4.63)

The exponent \( n \) can have values between 2.5 and 5.5 and for sand with a diameter \( 120 < D < 400 \mu m \) the it has the value 4.

Winterwerp states that formula (4.61) is not correct for cohesive sediments and introduced a more sophisticated formula that also includes the effect of return flow:
With
\[ m = \text{coefficient} \]
\[ \phi_* = \min \{1, \phi\} \]
\[ \phi_p = \text{volumetric concentration of primary particles} \]
\[ \phi = \text{volumetric concentration of mud flocs} \]
\[ m = \text{exponent for non-linear effects} (=1 \text{ in dissertation Winterwerp}) \]

In Figure 4.19 this formula is compared with experimental data. The concentration \( c_g \) is the ‘gelling point’ for which the mixture forms a space filling network for which the volumetric concentration \( \phi \) becomes unit by definition. This figure shows that a realistic range for the settling velocity is between 0.1 and 1 mm/s. Van Rijn (39) states that realistic settling velocities for mud are between 0.1 and 0.5 mm/s. Since the exact properties of the mud to be dredged are unknown, it can be concluded that a settling velocity within this range to be used in the calculations. To make a reliable calculation that applies to a specific situation, the properties of the specific mud to be dredged have to be determined.

Figure 4.19: Comparison of formula (4.64) with experimental data. Source: Winterwerp (50)
4.7 Calculation scheme

In this paragraph the steps to model the WID process as described in the previous paragraphs are put together to give an overview of the model. The model is worked out as a Matlab script that can be found in Appendix D:

![Diagram of WID process]

Figure 4.20: Overview locations of WID process. Adapted from Kortmann (14)

**Input values**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$g$</td>
<td>gravitational acceleration</td>
<td>[m/s$^2$]</td>
</tr>
<tr>
<td>$v_{dr}$</td>
<td>dredge velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$w_{dr}$</td>
<td>dredge beam width</td>
<td>[m]</td>
</tr>
<tr>
<td>$D_n$</td>
<td>nozzle diameter</td>
<td>[m]</td>
</tr>
<tr>
<td>$n_n$</td>
<td>number of nozzles</td>
<td>[-]</td>
</tr>
<tr>
<td>$\Delta p$</td>
<td>pressure drop over nozzle</td>
<td>[kPa]</td>
</tr>
<tr>
<td>$C_d$</td>
<td>nozzle discharge coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$\rho_{situ}$</td>
<td>density in-situ</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>density water</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$c_u$</td>
<td>cohesion in-situ mud</td>
<td>[kPa]</td>
</tr>
<tr>
<td>$\mu$</td>
<td>dynamic viscosity fluid mud</td>
<td>[Pa.s]</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$v_{set}$</td>
<td>settling velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$l_b$</td>
<td>bed slope (negative)</td>
<td>[-]</td>
</tr>
<tr>
<td>$C$</td>
<td>Chézy coefficient</td>
<td>[m$^{1/2}$/s]</td>
</tr>
<tr>
<td>$f_{jet}$</td>
<td>jetting factor</td>
<td>[-]</td>
</tr>
<tr>
<td>$f_{entr}$</td>
<td>entrainment factor</td>
<td>[-]</td>
</tr>
</tbody>
</table>
Calculation of jet properties

1. Jet flow velocity

\[ u_0 = \sqrt{\frac{2\Delta p}{\rho_w}} \]  

(4.65)

2. Nozzle area

\[ A_n = \pi \left( \frac{D_n}{2} \right)^2 \]  

(4.66)

3. Discharge single nozzle

\[ Q_n = C_o A_n u_0 \]  

(4.67)

4. Total jet discharge

\[ Q_j = n_n Q_n \]  

(4.68)

5. Specific discharge

\[ q_0 = \frac{Q_j}{w_{dr}} \]  

(4.69)

6. Jet power without losses

\[ P_{jet} = Q_j \Delta p \]  

(4.70)

Calculation of properties interaction jet and soil

7. Intrusion depth

\[ s_{intr} = \sqrt{\frac{3\rho_w u_0^2 D_n^2}{c_w}} \]  

(4.71)

8. In-situ soil discharge that enters process

\[ Q_{in} = s_{intr} v_{wp} w_{dr} f_{jet} \]  

(4.72)

Situation directly behind jets (location 1)

9. Discharge mixture

\[ Q_i = Q_{in} + Q_j \]  

(4.73)
10. Specific discharge

\[ q_1 = \frac{Q_0}{w_{gw}} \]  

(4.74)

11. Density mixture

\[ \rho_1 = \frac{Q_s \rho_{gw} + Q_0 \rho_w}{Q_s + Q_0} \]  

(4.75)

12. Relative density

\[ \varepsilon_1 = \frac{\rho_1 - \rho_w}{\rho_w} \]  

(4.76)

13. Flow velocity using conservation of momentum

\[ u_1 = \rho_{gw} Q_s v_{gw} + \rho_w n_c C s (u_0 - v_{gw})^2 \]  

\[ \frac{\rho_1 Q_1}{\rho_1 Q_1} \]  

(4.77)

14. Flow height

\[ h_1 = \frac{q_1}{u_1} \]  

(4.78)

15. Froude number

\[ Fr_1 = \sqrt{\frac{u_1^2}{h_1 g \varepsilon_1}} \]  

(4.79)

**Situation after entrainment of water (location 2)**

16. Discharge including certain amount of entrainment. This step is added since in the ‘black box’ area between the jet and the start of the density current, probably some extra water is entrained.

\[ Q_2 = Q_s + \frac{f_{entr} Q_0}{Q_s} \]  

(4.80)

17. Specific discharge

\[ q_2 = \frac{Q_2}{w_{gw}} \]  

(4.81)

18. Density

\[ \rho_2 = \frac{Q_s \rho_{gw} + f_{entr} Q_0 \rho_w}{Q_s + f_{entr} Q_0} \]  

(4.82)
19. Relative density

\[ \varepsilon_2 = \frac{\rho_s - \rho_w}{\rho_w} \]  

(4.83)

**Script to calculate u, h, Fr, Re and q as function of the distance (from location 3 onwards)**

u(i), h(i), Fr(i), Re(i), q(i) are vectors, with i the distance from the starting location of the density current.

20. Calculate initial values for q, u and h. The expression for the velocity is obtained by substituting 

\[ q = uh \]  

in the Chézy formula  

\[ u = C^* \sqrt{\frac{\Delta \rho}{\rho} g h_i \text{bed}} \] . Since the gravitational acceleration is in the square root, the dimensionless Chézy coefficient \( C^* \) has to be used:  

\[ C^* = \frac{C}{\sqrt{g}} \]

\[ q(1) = q_2 \]  

(4.84)

\[ u(1) = C^{*2/3} (-\varepsilon_2 q_1 g_j \text{bed})^{1/3} \]  

(4.85)

\[ h(1) = \frac{q(1)}{u(1)} \]  

(4.86)

21. Set script parameters. A while loop is used that calculates u, h, Fr, Re and q until the height h<0.1 m.

\[ dx = 1 \]

\[ i = 1 \]  

(4.87)

while h>0.1

22. Velocity

\[ u(i) = C^{*2/3} (-\varepsilon_2 q(i) g_j \text{bed})^{1/3} \]  

(4.88)

23. Flow height

\[ h(i) = \frac{q(i)}{u(i)} \]  

(4.89)

24. Froude number

\[ Fr(i) = \frac{u(i)^2}{\sqrt{h(i) g \varepsilon_2}} \]  

(4.90)
25. Reynolds number

\[ \text{Re}(i) = \frac{\rho u(i)h(i)}{\mu} \]  
(4.91)

26. Time to travel distance \( dx \)

\[ t_{\text{step}}(i) = \frac{dx}{u(i)} \]  
(4.92)

27. Settling distance in time \( t_{\text{step}} \)

\[ h_{\text{set}}(i) = t_{\text{step}}(i)\nu_{\text{set}} \]  
(4.93)

28. New discharge

\[ q(i + 1) = u(i)\left[h(i) - h_{\text{set}}(i)\right] \]  
(4.94)

29. Increase \( i \)

\[ i = i + 1 \]  
(4.95)

30. Back to step 21

Now \( q, u, h, \text{Fr} \) and \( \text{Re} \) can be plotted as a function of the distance. The results of this calculation model are presented in the next paragraph.
4.8 Model results

In this paragraph the model is executed for a realistic set of parameters and the results are presented. For a number of parameters a description of the used value is given. It is noted that for many parameters a range of acceptable values is known. In many cases a value within this range is chosen without an in-depth justification for the simple reason that this is impossible without practical data.

Input values

**General**
\[ g = \text{gravitational acceleration} = 9.81 \, [\text{m/s}^2] \]

**Dredge**
\[ v_{dr} = \text{dredge velocity} = 0.5 \, [\text{m/s}] \]

*From practice it is known that WID devices move at a hauling velocity between 0.5 and 1 m/s. Since the AWID device is relatively small, a low velocity is chosen.*

\[ w_{dr} = \text{dredge beam width} = 4 \, [\text{m}] \]

*Taken into account that the autonomous device will be smaller than a regular WID vessel, a beam width of 4 m is assumed to be reasonable. See paragraph 5.2.2.*

**Jets**
\[ D_n = \text{nozzle diameter} = 0.08 \, [\text{m}] \]

*In literature a nozzle diameter of about 0.08 m has been suggested as realistic for a sufficiently high discharge.*

\[ n_n = \text{number of nozzles} = 15 \, [-] \]

*It is known from practice that a heart-to-heart distance between the nozzles of 30 cm is reasonable. With a beam width of 4 m, this results in about 15 nozzles.*

\[ \Delta p = \text{pressure drop over nozzle} = 100 \, [\text{kPa}] \]

*In literature the value of 100 kPa is often mentioned.*

\[ C_d = \text{nozzle discharge coefficient} = 0.96 \, [-] \]

*Van der Schrieck (35) gives discharge coefficients for multiple nozzle setups and gives the value of 0.96 for simple nozzle with a straight outlet.*

**Water and soil**
\[ \rho_{\text{situ}} = \text{density in-situ} = 1300 \, [\text{kg/m}^3] \]

*As shown in Figure 2.10, in-situ densities between 1200 and 1300 kg/m$^3$ can be found in the Port of Rotterdam.*

\[ \rho_w = \text{density water} = 1000 \, [\text{kg/m}^3] \]

\[ c_u = \text{cohesion in-situ mud} = 10 \, [\text{kPa}] \]

*As described in paragraph 0, a maximum value for the in-situ cohesion is 10 kPa. This value is used for a conservative calculation.*
\[ \mu = \text{dynamic viscosity fluid mud} = 0.1 \text{ [Pa.s]} \]

As shown in Figure 2.7 the viscosity in a fluid mud layer is about 0.1 Pa.s.

**Main parameters for outflow**

\[ \nu_{\text{set}} = \text{settling velocity} = 0.15 \text{ [mm/s]} \]

As described in paragraph 4.6.5, settling velocities in a range of 0.1-1 mm/s have been observed, depending on the mud properties and local circumstances.

\[ i_b = \text{bed slope} = -0.001 \text{ [-]} \]

Wurpts and Torn (52) state that a bed slope of 0.001 has been used in the past and it is even possible for a density current to travel many kilometers over a slope of only 1:10000.

\[ C = \text{Chézy coefficient} = 80 \text{ [m}^{1/2}/\text{s]} \]

Winterwerp and Van Kesteren (51) give for the Chézy coefficient different values for different locations, but all are within a range of 60-110 m\(^{1/2}\)/s.

\[ f_{\text{jet}} = \text{jetting efficiency factor} = 0.5 \text{ [-]} \]

As described in 0 a production efficiency factor between 0 and 1 is applied to take into account the fact that not all soil is being removed. As a first estimation a factor 0.5 is used, since the calculated intrusion depth is rather high.

\[ f_{\text{entr}} = \text{entrainment factor} = 1.2 \text{ [-]} \]

To take into account the entrainment of water behind the jet, a factor of 1.2 is assumed to be acceptable.
Output values

\[ u_0 = \text{jet flow velocity} = 14 \text{ [m/s]} \]
\[ Q_n = \text{single nozzle discharge} = 0.068 \text{ [m}^3\text{/s]} \]
\[ Q_0 = \text{jet discharge} = 1.0 \text{ [m}^3\text{/s]} \]
\[ q_0 = \text{specific jet discharge} = 0.26 \text{ [m}^2\text{/s]} \]
\[ s_{intr} = \text{intrusion depth} = 0.62 \text{ [m]} \]
\[ P_{jet} = \text{jet power} = 100 \text{ [kW]} \]
\[ Q_m = \text{mud discharge} = 0.62 \text{ [m}^3\text{/s]} \]

<table>
<thead>
<tr>
<th></th>
<th>Location 1</th>
<th>Location 2/3</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Q )</td>
<td>discharge mixture</td>
<td>[m}^3\text{/s]}</td>
</tr>
<tr>
<td>( q )</td>
<td>specific discharge mixture</td>
<td>[m}^3\text{/s]}</td>
</tr>
<tr>
<td>( h )</td>
<td>layer thickness</td>
<td>[m]</td>
</tr>
<tr>
<td>( u )</td>
<td>velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>( \rho )</td>
<td>mixture density</td>
<td>[kg/m}^3\text{]}</td>
</tr>
<tr>
<td>( Fr )</td>
<td>Froude number</td>
<td>[-]</td>
</tr>
</tbody>
</table>

Table 4.1: Model results at location 1 and 2

In Figure 4.21 the height, velocity, discharge, Froude number and Reynolds number are plotted as a function of the distance.
Intrusion depth

It can be observed that the intrusion depth is quite high and it is questionable whether this result is realistic. Since the actual intrusion depth is limited by the mud layer thickness, which is in many cases probably smaller, this theoretical maximum intrusion depth does not play a very big role. However, since the mud production in the model depends on the intrusion depth, a low jetting coefficient $f_{jet}$ of 0.5 is used to account for this issue.

Dependence of output parameters on input parameters

A number of input parameters can be controlled by the design and usage of the dredge. However, there is also a number of parameters that depends on the specific location and can vary widely, resulting in very different model results.

In Table 4.2 it is indicated what the influence of the these input parameters is on several output parameters. It is indicated how every output parameter changes if one of the input parameters increases (↑): with an increase (↑) or decrease (↓).
### Table 4.2: Dependence output on input parameters

The main output parameter of interest is the distance over which the density current travels. It can be observed that the decrease of discharge over distance, so actually the settling velocity, determines this transport distance. In paragraph 4.6.5 it is shown that a settling velocity in the range of 0.1-1 mm/s is realistic. These values result in a wide range travel distances between 5000 and 5000 m. Without more details on the mud type it is difficult to say what the real settling velocity and thus travel distance will be. It is clear that the distance will be in the order of kilometres.

The bed slope and Chézy coefficient both have the same kind of influence on the output parameters. A large bed slope or Chézy coefficient results in a high flow velocity, which leads to a smaller flow height. With the input ranges as described as given in Table 4.2, sensitivity for the bed slope and Chézy coefficient is much smaller than that for the settling velocity. For the initial velocity this range is 0.3-0.8 m/s and for the initial height this range is 0.6-1.8 m.

A larger entrainment coefficient leads to a larger flow height and a slightly larger travel distance. For example, with the used parameters an initial flow height of 2 m is reached with an entrainment coefficient of 5. This means that for the flow height to become unrealistic, a very large amount of entrainment has to take place.

### Practical travel distance to use in calculations

For the production estimation it is useful to have a single travel distance for the density current. As can be seen in Figure 4.21, in practise the current slowly dies out and the mud will be dispersed over a certain distance. Since the decrease of the discharge is assumed to be linear, on average the mud settles halfway the transport distance. Therefore, this distance can be used in further calculations as ‘the’ travel distance. It is possible to model the displacement of mud in a more precise way, but for a rough estimation this is assumed to be unnecessary. With the variables as presented above, this results in an average travel distance of about 1.5 km. Therefore, this distance will be used in the calculations on the case study.

### Required bed slope

From the model practical values follow for the bed slope on which the density current will flow. A slope of 1/1000 seems to be a good rule of thumb.
5 Case study

In chapter 4 a theoretical model of the water injection dredging process has been developed. In this chapter the results from this model are applied to a realistic case of a port basin. The case will be elaborated on in further detail and the possible characteristics of the autonomous water injection dredging (AWID) device will be explored. With this information, a first design can be made for the autonomous dredging system.

5.1 Port of Rotterdam

The Port of Rotterdam is the largest port in Europe as measured by annual cargo throughput. It suffers from mud sedimentation and constant dredging is required. This makes the Port of Rotterdam an interesting location to apply the AWID concept.

The Port of Rotterdam is an open, deep-water port in the Maas river estuary. In Figure 5.1 an overview of the port area is given. Since both a tidal current and a river discharge cause heavy siltation in the port, continuous maintenance dredging is required to keep the port navigable. Every year approximately 15 million m$^3$ of sediment is dredged (11). Usually this dredging is done by conventional trailing suction hopper dredger (TSHD) vessels.

![Figure 5.1: Overview Port of Rotterdam. Source: Google Earth](image)

The Port of Rotterdam is becoming more and more a high tech, automatic port and has the ambition to be the smartest port of the world. Unmanned terminals are already in use and most likely in the future the navigating and mooring of vessels will be highly automated (46). An automatic way of removing mud from the basins fits well within the goal of being the smartest port in the world.

Since there is public data available on the Botlek harbour (11) this part of the harbour will be used as an example case. In Figure 5.2 the yearly volumes of dredged material in this harbour are given and in Figure 5.3 the distribution of this material over the different basins is shown. It becomes clear that the amount of sedimentation in a certain basin varies widely: between (almost) zero and 4.5 meter per year.
Since nowadays the sediments that settle in the port are relatively clean, the mud does not have to be deposited in a special area for contaminated material, like the existing ‘Slufter’ area. Therefore it assumed that the sediment can be brought back into the system to flow away to the sea.

Figure 5.2: Yearly volume of maintenance dredging in the Botlek harbour. Source: El Hamdi (11)

Figure 5.3: Thickness of yearly dredged layer. Source: El Hamdi (11)

The aim of this report is not to give a detailed working method, but to explore the possibilities for dredging methods that can be part of an autonomous dredging solution. Therefore, rough figures will be used for the estimation of the production of the device.
5.2 AWID device properties

As described in chapter 2, the objective of this thesis is to find a way to use multiple small dredging devices that work together and navigate autonomously through the port in order to remove the mud from different basins.

In this report the basic requirements and properties for the AWID device are explored. Matters as safety, detailed ship design, ship management and maintenance are beyond the scope of this report and can be interesting topics for further study from a mechanical or ship engineering perspective.

5.2.1 Current fleet of WID

As of June 2010, 22 Water Injection Dredgers were active. An overview is given in Table 5.1 (22). The power of the used jet pump diesel engines varied from 75 kW in the Baldur to 1250 kW in the Maasmond. The possible dredging depths vary between 0 and 30 m and the working speeds are between 1 and 2 knots. This shows that the WID technique can be applied on different scales.

<table>
<thead>
<tr>
<th>Vessel</th>
<th>Maximum dredge depth [m]</th>
<th>Jet bar width [m]</th>
<th>Jet pump diesel engine power [kW]</th>
<th>Company</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maasmond</td>
<td>21</td>
<td>12.0</td>
<td>1250</td>
<td>Van de Kamp</td>
</tr>
<tr>
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<td>Draga Rio Madeira</td>
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<td>Meyer &amp; Van de Kamp</td>
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<td>19</td>
<td>8.8</td>
<td>440</td>
<td>Boskalis</td>
</tr>
<tr>
<td>Hol Deep</td>
<td>18</td>
<td>8.1</td>
<td>346</td>
<td>Bremen Ports</td>
</tr>
<tr>
<td>Odin</td>
<td>10</td>
<td>4.4</td>
<td>220</td>
<td>Van Oord</td>
</tr>
<tr>
<td>Baldur</td>
<td>7</td>
<td>2.5</td>
<td>75</td>
<td>Van Oord</td>
</tr>
</tbody>
</table>

Table 5.1: Overview of WID dredgers as per June 2010. Adapted from PIANC (22)
5.2.2 Main specifications AWID

Most regular WID devices have a beam that is about 10-13 m long. In theory these regular, relatively large WID vessels can be made autonomous by installing a control system. However, it is expected that the use of multiple, smaller devices is better suited for an autonomous system. Safety and cost reasons play a role in this expectation. Also the limited travel range, as will be described in the next paragraph, asks for multiple devices that are spread over the port. Therefore a small device with a beam of 4 m seems reasonable. This beam length was already used in the calculations in the previous chapter.

Looking at Table 5.1, an existing WID dredger that has a jet beam of about 4 m is the Odin. The specifications of this dredger are used as a reference for the AWID dredger. Its main specifications are given in Table 5.2.

<table>
<thead>
<tr>
<th>Specification</th>
<th>Odin 1</th>
<th>kW</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jetting power</td>
<td>220</td>
<td></td>
</tr>
<tr>
<td>Propulsion power</td>
<td>2*89</td>
<td></td>
</tr>
<tr>
<td>Total installed power</td>
<td>410</td>
<td></td>
</tr>
<tr>
<td>Length o.a.</td>
<td>17.5</td>
<td>m</td>
</tr>
<tr>
<td>Width</td>
<td>4.5</td>
<td>m</td>
</tr>
<tr>
<td>Depth</td>
<td>1.8</td>
<td>m</td>
</tr>
<tr>
<td>Draft (loaded)</td>
<td>1.45</td>
<td>m</td>
</tr>
<tr>
<td>Jet bar width</td>
<td>4.4</td>
<td>m</td>
</tr>
<tr>
<td>Maximum dredge depth</td>
<td>10</td>
<td>m</td>
</tr>
</tbody>
</table>

Table 5.2: Specifications WID dredger Odin. Source: vanoord.com/dravosa.com

The Odin has a maximum dredge depth of 10 m. This is too low to clean the deepest basins of the Port of Rotterdam, which have a depth of 24 m (27). It is assumed that a similar device with a longer beam is feasible when the installed power is higher. Therefore a total installed power of 500 kW is assumed to be realistic.

5.2.3 Power source and storage

The power required for both jetting and sailing can be supplied in multiple ways. Nowadays, the most common power source is an engine on fossil fuel. However, taking into account the environmental footprint of the system, other energy solutions can also be considered. Possible options are:

- Electric with cable connected to grid
  It is possible to connect an electric powered vessel to the electricity grid with a cable. This way energy from renewable sources can be used. Drawbacks are the low flexibility of the system and the complexities and risks that underwater cables bring.

- Electric with battery pack
  Looking at the current developments in the field of electric cars, a similar system can be used for vessels. An advantage is that the vessel does not need to be connected to the grid, giving it more freedom to move. Disadvantages of the current generation of batteries are the low energy density, relatively high price and low lifetime.

- Hydrogen
  The first trials with hydrogen powered vessels are being undertaken (13). The energy density of hydrogen is higher than that of batteries. A disadvantage is that the technology is still expensive.
Hydraulic pressure cable

A cable that supplies water under a high pressure can be used to power both the engines for sailing and jetting. Compared to electric cables this solution is expected to be safer and it makes the electric systems on board less complicated. However, there are no examples from practice so such a system will be highly experimental.

With the specific energy and energy density of different energy storage solutions it can be calculated what volume and mass are required per hour to power the AWID device, as is done in Table 5.3. It has to be noted that the used figures are rough and that the efficiency of the engines and pumps is not taken into account. Therefore the presented figures only serve as an first indication.

<table>
<thead>
<tr>
<th>Storage material</th>
<th>Specific energy (MJ/kg)</th>
<th>Energy density (MJ/L)</th>
<th>kg/hr</th>
<th>m³/hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrogen (compressed)</td>
<td>142</td>
<td>6</td>
<td>16</td>
<td>0.3</td>
</tr>
<tr>
<td>Diesel/fuel oil</td>
<td>48</td>
<td>36</td>
<td>38</td>
<td>0.05</td>
</tr>
<tr>
<td>LPG/propane/butane</td>
<td>46</td>
<td>26</td>
<td>40</td>
<td>0.07</td>
</tr>
<tr>
<td>Lithium-ion battery</td>
<td>0.36-0.88</td>
<td>0.9-2.6</td>
<td>2000-5000</td>
<td>0.7-2</td>
</tr>
</tbody>
</table>

Table 5.3: Specific energy and energy density of different fuels (source: wikipedia.org) and the mass and volume required per hour for a total installed power of 500 kW.

The table shows that compared to fossil fuels, hydrogen has a higher energy density and batteries have a much lower energy density. This means that hydrogen can be a viable option if this technology becomes more mature and economic. Comparing a storage system with batteries to regular fossil fuel, the mass is 50-130 times higher and the volume 14-40 times higher. Battery packs are currently commercially available and their costs are decreasing. During dredging the AWID generally moves at a slow speed of 0.5 m/s and therefore large and heavy battery pack probably is not a big issue. Therefore it can be concluded that a battery powered AWID system is a good solution.

It has to be noted that the densities of the lithium-ion batteries in Table 5.3 only include the battery cells themselves and do not take into account auxiliary equipment as the cooling system, frame, casing and electronics. In order to make a realistic estimation of the battery system, as a reference the Tesla PowerWall is used. This battery is introduced in 2015 and is intended as a low cost power storage system for home use, for example in combination with solar panels. Multiple units can be combined to form a system with a larger capacity. The main specifications of the PowerWall are given in Table 5.4. It has to be noted that the volume of the battery is not given by Tesla and is therefore simply calculated using the available dimensions, assuming the unit has the shape of a rectangular cube.

<table>
<thead>
<tr>
<th>Capacity</th>
<th>10 kWh</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensions</td>
<td>1.3x0.86x0.18 m</td>
</tr>
<tr>
<td>Volume</td>
<td>0.2 m³</td>
</tr>
<tr>
<td>Mass</td>
<td>100 kg</td>
</tr>
<tr>
<td>Price</td>
<td>3500 $</td>
</tr>
</tbody>
</table>

Table 5.4: Specifications Tesla PowerWall. Source: teslamotors.com

Tesla also offers a system that is geared towards commercial use. This system, called Powerpack, consists of 100 kWh units and is highly scalable. However, no detailed information on this system is available at the moment of writing.

From the specifications of the PowerWall it follows that the specific energy is 0.36 MJ/kg and the energy density is 0.18 MJ/L. When looking at Table 5.3, it can be concluded that especially the energy density is rather low, even when taking into account the auxiliary equipment. In practice a higher...
density can probably be reached due to scale advantages. Therefore using the PowerWall will give a conservative estimation for the required battery volume.

It is assumed that the AWID has to be able to work for four hours on a single battery load. With a propagation velocity during dredging of 0.5 m/s or 1.8 km/h the range is about 7 km.

The total required battery capacity is 2000 kWh (500 kW*4 hr). Therefore 200 PowerWall units are needed, with a volume of 40 m³ and a mass of 20 metric tons. As mentioned before, the volume will most likely be lower in reality.

5.2.4 Design

Two options for an initial design are regarded:

1. A battery pack mounted underneath a regular WID vessel
2. Two pontoons connected with trusses

It is possible to mount a battery pack underneath a regular WID vessel, covering its full underwater surface (Figure 5.4). For example for the Odin, with a length of 17.5 m and a width of 4.5 m, this would result in a battery pack with a thickness of about 0.5 m. A disadvantage of this solution is that the battery is difficult to reach.

![Figure 5.4: Regular WID pontoon with battery pack](image)

Another possibility is use two separate pontoons to minimize the size and mass of the AWID (Figure 5.5). To increase the stability and ability to support a long WID beam to reach high depths, the pontoons are spaced apart with a number of trusses. One pontoon contains the 40 m³ battery pack and the other the engine, propulsion system, jet pump. This second beam also is connected to the WID beam.

Possible dimensions for the pontoons are:

- Width: 6 m
- Length: 4 m
- Height: 2 m
Each pontoon thus has a volume of 48 m$^3$. In between the pontoons multiple trusses are placed that have a length of about 10-15 m. This way the length of the full AWID becomes 18-23 m. The battery pack pontoon has a system to support the jetting beam. It is possible to design a system so that the full battery pack can be replaced at once, or that the battery pack stays in place and the full pontoon needs to wait at the dock until the batteries are recharged. An advantage is that the battery pack can be reached easily. It has to be investigated whether the relatively small area provides enough buoyancy.

Figure 5.5: Two pontoons connected with trusses

For maintenance reasons it is possible that the AWID device has a modular design that allows quick replacement of parts that need repair or maintenance. Spare parts can be maintained during normal working hours and possibly be replaced fully automated at any necessary time.

For example, the AWID vessel can consist of four parts:

1. A propagation module that contains an engine, propellers and navigation systems.
2. A water injection module that contains the jetting beam and a pump system. This module can be powered by its own engine or by the engine from the propagation module.
3. An energy module that contains the batteries.
4. A survey module that can be mounted on the AWID device that works in an area that needs new survey data. This way the costly survey equipment can be used on multiple vessels.

These design considerations require further attention from a marine engineering or ship design point of view.
5.2.5 Survey and control
Since the boats have to work together to make optimal use of their capabilities, a central control centre is required that knows the exact location of all vessels and that sends them to the right location. Also, maintenance and repair can be scheduled and coordinated centrally. In case of a battery pack or diesel engine, it is possible to automate the process of sailing back to the dock and charging or changing the battery pack.

Another point of interest is the survey of the works. Nowadays, dedicated survey vessels are used to produce an image of the state of the harbour bed. In theory it is possible to equip the AWID vessels with survey equipment so that the control room knows exactly which part of the harbour needs dredging. Based on this info the schedule can be changed. As described above, it is possible to mount the survey equipment in a module that can easily moved from one dredge to another.

5.3 Production
In order to make a design of the AWID system, both the production capacity of the device and the required production in the Botlek harbour have to be estimated.

The production can be expressed as a combination of two quantities: the mud volume and the distance over which this volume has to be transported. The distance is important since, as has been observed in the previous chapter, the mud only travels over a limited distance. This leads to a figure for the yearly production that has the unit m³*km/year.

5.3.1 Production capacity AWID
In past experiments it has been observed that it is often necessary to dredge a certain area twice in order to remove all sediments. Therefore it is assumed that two ‘sweeps’ are require to clean a certain area. This corresponds to the jetting factor of 0.5 as used in the previous chapter. It can be assumed that the soil-water mixture starts to flow when its density is lower than 1100 kg/m³, which poses a requirement on the hauling speed, jet discharge and entrainment coefficient.

The AWID production in cubic meter per hour can be estimated with:

\[ Pr = f_{jet} s_{intr} w_{dr} v_{dr} * 3600 \]  \hspace{1cm} (5.1)

With

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pr</td>
<td>production AWID</td>
<td>m³/hour</td>
</tr>
<tr>
<td>( f_{jet} )</td>
<td>jetting factor</td>
<td>-</td>
</tr>
<tr>
<td>( s_{intr} )</td>
<td>effective intrusion depth</td>
<td>m</td>
</tr>
<tr>
<td>( w_{dr} )</td>
<td>width beam</td>
<td>m</td>
</tr>
<tr>
<td>( v_{dr} )</td>
<td>hauling velocity</td>
<td>m/s</td>
</tr>
</tbody>
</table>

The effective intrusion depth of 0.3 m is an estimation that describes the limited amount of available mud. As described in paragraph 4.4.1, this depth is smaller than the theoretical maximum intrusion depth. This results in a instantaneous production of about 1100 m³/hour.

In practise, the dredge will not reach this production since time is required for navigating, positioning, turning, refuelling/recharging, maintenance and repair. Also bad weather conditions can have a negative influence on the workability. Finally, it is possible that due to regulations the AWID device is not allowed to work 24/7.

In Table 5.5 several realistic production rates from different WID projects are given. These rates take into account all production limiting factors. The table is adopted by PIANC from Wilson (49). It shows
that for finer grained soils generally higher production rates have been reached. As a rough indication, a production rate between 500 and 1000 m³/hour is realistic for a full size WID dredger.

<table>
<thead>
<tr>
<th>Project name</th>
<th>Soil description</th>
<th>Volume [m³]</th>
<th>Duration [hours]</th>
<th>Production rate [m³/hr]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epon Harbour, Delfzijl, The Netherlands</td>
<td>Silt &amp; sand D&lt;sub&gt;50&lt;/sub&gt; 0.3 mm</td>
<td>160000</td>
<td>200</td>
<td>800</td>
</tr>
<tr>
<td>Haringvliet harbour, The Netherlands</td>
<td>Silt/clay</td>
<td>121000</td>
<td>252</td>
<td>480</td>
</tr>
<tr>
<td>Crouch River, United Kingdom</td>
<td>Clayey silt</td>
<td>6200</td>
<td>12</td>
<td>540</td>
</tr>
<tr>
<td>Upper Mississippi River 1992</td>
<td>Sand 0.3-0.4 mm</td>
<td>6200</td>
<td>44</td>
<td>140</td>
</tr>
<tr>
<td>Calumet 1994</td>
<td>Silt 0.004-0.05 mm</td>
<td>12000</td>
<td>24</td>
<td>500</td>
</tr>
<tr>
<td>East and West Calumet floodgates</td>
<td>Silt 0.004-0.05 mm</td>
<td>17900</td>
<td>17</td>
<td>1080</td>
</tr>
<tr>
<td>Michoud 2002</td>
<td>Silt 0.06 mm</td>
<td>179000</td>
<td>96</td>
<td>1900</td>
</tr>
<tr>
<td>Mississippi River Gulf Outlet (MRGO) 2003</td>
<td>Silt</td>
<td>269000</td>
<td>96</td>
<td>2800</td>
</tr>
<tr>
<td>Weser Estuary, Germany, 2009</td>
<td>Sand 0.6 mm (per year)</td>
<td>650000</td>
<td>1200</td>
<td>550</td>
</tr>
<tr>
<td>Elbe Estuary, Germany, 2009</td>
<td>Sand and silt 0.05-0.6 mm (per year)</td>
<td>1500000</td>
<td>2000</td>
<td>750</td>
</tr>
</tbody>
</table>

Table 5.5: Production rates during WID projects. Source: PIANC (22)

Looking at the information in Table 5.5, a 50% production efficiency factor is suggested to take into account all production limiting factors. This leads to a net production of 550 m³/hour.

As described before, the transport distance has also to be taken into account in the production estimation. Depending on the chosen parameters in the model, it is reasonable to assume that all the material has been deposited at distance of 3 km from the initial location. Since the settling process is linear, the bulk of the material travels a distance of 1.5 km (see paragraph 0). This distance will be used as the transport distance. For distances larger than this 1.5 km, it is necessary to dredge at multiple locations along the outflow route to bring the settled material again in suspension. An issue of this method is that the required work to remove mud at distances smaller than 1.5 km is underestimated. This leads to the following formula for the production with the unit m³*km/hour:

\[
Pr_{tot} = f_{eff} s_{sett} W_{aw} r_{dr} l_{outflow} * 3600
\]  

(5.2)

With

- \( Pr_{tot} \) = total production AWID [m³*km/hour]
- \( f_{eff} \) = production efficiency factor = 0.5 [-]
- \( l_{outflow} \) = outflow distance = 1.5 [km]

This results in a production of 800 m³*km/hour or 0.019 million m³*km/day.
5.3.2 Required production
The main figures of the Botlek harbour are:
- Total area: 3 million m$^2$ (estimation using Google Earth)
- Average yearly dredging volume: 1.5 million m$^3$ (from figure Figure 5.2)

This results in an average height of the yearly dredged mud layer of 0.5 m. Figure 5.3 shows that there are large differences in the sedimentation between the different basins. Therefore the Botlek harbour has been divided into three main areas, as is indicated in the same figure. For these areas an estimation is made of the area and average yearly sedimentation height, resulting in a yearly sediment volume per area (Table 5.6).

![Figure 5.6: Three areas with different amount of sedimentation. Adapted from El Hamdi (11)](image)

<table>
<thead>
<tr>
<th>Area</th>
<th>Sedimentation volume [million m$^3$/year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. High amount of sedimentation</td>
<td>1</td>
</tr>
<tr>
<td>2. Medium amount of sedimentation</td>
<td>0.4</td>
</tr>
<tr>
<td>3. Low amount of sedimentation</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 5.6: Area and sedimentation for each of the three areas

It is assumed that after every set of two ‘sweeps’ 30 cm of material is removed. Using this assumption and the yearly sedimentation height, an estimation of the dredge frequency can be made for each of the three areas. This is indicated in Table 5.7.

<table>
<thead>
<tr>
<th>Sedimentation [m/year]</th>
<th>Dredging frequency [/year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. High amount of sedimentation</td>
<td>7</td>
</tr>
<tr>
<td>2. Medium amount of sedimentation</td>
<td>2</td>
</tr>
<tr>
<td>3. Low amount of sedimentation</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 5.7: Dredging frequency per area

Now the only difference between the areas is the average distance over which the material has to be transported. For each of the three areas an average required transport distance can be estimated. To come to a reasonable estimation of the distance to the river for each of the three areas, a weighted averaged is calculated for each of the basins. In Figure 5.7 the basins are numbered and in Table 5.8 this weighted average of the distance is calculated for each of the three areas. The results are presented in Table 5.9.
Figure 5.7: Numbered basins for calculation transport distance to river. Adapted from El Hamdi (11)

<table>
<thead>
<tr>
<th>Area 1</th>
<th>Basin nr</th>
<th>Area [x1000 m$^3$]</th>
<th>Av. distance to river [km]</th>
<th>Area*distance [1000 m$^3$km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>130</td>
<td>0,05</td>
<td>6,5</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>120</td>
<td>0,2</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>0,7</td>
<td>140</td>
<td></td>
</tr>
<tr>
<td>total</td>
<td>450</td>
<td></td>
<td>170,5</td>
<td></td>
</tr>
<tr>
<td>Weighted average distance</td>
<td></td>
<td></td>
<td>0,4</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Area 2</th>
<th>Basin nr</th>
<th>Area [x1000 m$^3$]</th>
<th>Av. distance to river [km]</th>
<th>Area*distance [1000 m$^3$km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>270</td>
<td>0,8</td>
<td>216</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>250</td>
<td>1,7</td>
<td>425</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>300</td>
<td>1,7</td>
<td>510</td>
<td></td>
</tr>
<tr>
<td>total</td>
<td>820</td>
<td></td>
<td>1151</td>
<td></td>
</tr>
<tr>
<td>Weighted average distance</td>
<td></td>
<td></td>
<td>1,4</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Area 3</th>
<th>Basin nr</th>
<th>Area [x1000 m$^3$]</th>
<th>Av. distance to river [km]</th>
<th>Area*distance [1000 m$^3$km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>180</td>
<td>1,4</td>
<td>252</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>220</td>
<td>1,8</td>
<td>396</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>90</td>
<td>2,1</td>
<td>189</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>200</td>
<td>2,1</td>
<td>420</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>350</td>
<td>2,1</td>
<td>735</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>300</td>
<td>3</td>
<td>900</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>270</td>
<td>3,3</td>
<td>891</td>
<td></td>
</tr>
<tr>
<td>total</td>
<td>1610</td>
<td></td>
<td>3783</td>
<td></td>
</tr>
<tr>
<td>Weighted average distance</td>
<td></td>
<td></td>
<td>2,3</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.8: Calculation of weighted average distance of basins to the river
Average travel distance [km]

1. High amount of sedimentation 0.4
2. Medium amount of sedimentation 1.4
3. Low amount of sedimentation 2.4

Table 5.9: Average travel distance per area

By multiplying the distances from Table 5.9 with the yearly sediment volume from Table 5.6, a rough estimation of the required production can be made with the unit m³·km/year. This is presented in Table 5.10.

<table>
<thead>
<tr>
<th>Sediment volume [million m³/year]</th>
<th>Travel distance [km]</th>
<th>Required production [million m³·km/year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. High amount of sedimentation</td>
<td>1</td>
<td>0.4</td>
</tr>
<tr>
<td>2. Medium amount of sedimentation</td>
<td>0.4</td>
<td>1.4</td>
</tr>
<tr>
<td>3. Low amount of sedimentation</td>
<td>0.2</td>
<td>2.3</td>
</tr>
</tbody>
</table>

Table 5.10: Yearly sediment volume, travel distance and required production per area

An issue with this method is that the travel distance of mud that is used in the production of the AWID is always 1.5 km, also when the area is closer to the river than this distance. In order to make a realistic calculation, the travel distances in Table 5.10 that are smaller than 1.5 km have to be changed to this minimum value. This is shown in Table 5.11.

<table>
<thead>
<tr>
<th>Sediment volume [million m³/year]</th>
<th>Travel distance [km]</th>
<th>Required production [million m³·km/year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. High amount of sedimentation</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>2. Medium amount of sedimentation</td>
<td>1.5</td>
<td>0.6</td>
</tr>
<tr>
<td>3. Low amount of sedimentation</td>
<td>2.3</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 5.11: Corrected travel distances for minimum distance of 1.5 km

Dividing the yearly production by the AWID production capacity as calculated in paragraph 5.3.1 (0.019 million m³·km/day) results in an estimation for the duration of the dredging works in the basin, as shown in Table 5.12. It can be concluded that it takes 137 days per year to keep the Botlek harbour free of mud. This value makes sense, because when not taking into account the travel distances the required time would be:

\[
\text{time} = \frac{\text{yearly volume}}{\text{production}} = \frac{1.5 \times 10^8 \text{ m}^3 / \text{year}}{550 \text{ m}^3 / \text{hour}} = 2700 \text{ hours} = 114 \text{ days} \quad (5.3)
\]

The extra 23 days are required since some parts are located at a large distance from the river and the mud has to be dredge a second time along the outflow route. It can be argued that these extra days are not required, since a first sweep would move all the material from area 3 to area 1. Subsequently, this small amount of material can easily be removed during the more frequent cleaning of area 1. However, looking at the uncertainties in the presented calculations, this issue is not of great importance.

<table>
<thead>
<tr>
<th>Required production [million m³·km/year]</th>
<th>Yearly dredging [days/year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. High amount of sedimentation</td>
<td>1.5</td>
</tr>
<tr>
<td>2. Medium amount of sedimentation</td>
<td>0.6</td>
</tr>
<tr>
<td>3. Low amount of sedimentation</td>
<td>0.5</td>
</tr>
<tr>
<td>Total</td>
<td>2.6</td>
</tr>
</tbody>
</table>

Table 5.12: Required production and required amount of working days per year
5.3.3 Summary
In Table 5.13 the results from all the calculation steps are summarized.

<table>
<thead>
<tr>
<th></th>
<th>1. High</th>
<th>2. Medium</th>
<th>3. Low</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area [million m²]</td>
<td>0.5</td>
<td>0.8</td>
<td>1.6</td>
<td>2.9</td>
</tr>
<tr>
<td>Sedimentation [m/year]</td>
<td>2</td>
<td>0.5</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>Dredging frequency [/year]</td>
<td>7</td>
<td>2</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Yearly mud volume [million m³/year]</td>
<td>1</td>
<td>0.4</td>
<td>0.2</td>
<td>1.6</td>
</tr>
<tr>
<td>Average distance [km]</td>
<td>0.4</td>
<td>1.4</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>Corrected av. distance [km]</td>
<td>1.5</td>
<td>1.5</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>Yearly production [million m³*km/year]</td>
<td>0.4</td>
<td>0.6</td>
<td>0.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Corrected yearly prod. [million m³*km/year]</td>
<td>1.5</td>
<td>0.6</td>
<td>0.5</td>
<td>2.6</td>
</tr>
<tr>
<td>Yearly dredging time [days/year]</td>
<td>79</td>
<td>32</td>
<td>26</td>
<td>137</td>
</tr>
</tbody>
</table>

Table 5.13: Summary of estimation of duration dredging

5.4 The AWID system
It has been estimated that it takes about 137 days per year to keep the Botlek harbour free of mud. To design the full AWID system for the Port of Rotterdam, this value has to be extrapolated. This leads to a rough estimation, since all port basins will have characteristics that differ from the Botlek harbour.

The total water area of the Port of Rotterdam amounts 4800 ha or 48 million m² (27). This is 16 times the area of the Botlek harbour. Now a simple multiplication leads to 16*137=2192 days of dredging per year for the full port. This means that 6 AWID devices should be able to keep the Port of Rotterdam free of mud.

One central docking site for maintenance and repair is required. In addition to this central site, 5 smaller docking sites have to be located in different parts of the harbour since the travel range of an AWID is limited by the battery capacity. At the small docks the AWID devices can recharge their batteries. Also, these docks can be used to store the devices when they are not in use. By spreading the AWIDs over the port, every device has its own working area. The basins in the Port of Rotterdam are spread over a length of about 36 km. This means that every AWID works along a distance of about 6 km, which seems realistic. In Figure 5.8 a possible distribution of the docks over the port is given.
The main characteristics of the AWID system as described in the previous paragraphs are:

- A bed slope of about 1/1000 is required for the density current
- 6 AWID devices with a 4 m jet beam and net production of 550 m³/hour.
- 1 main docking site for maintenance and repair
- 5 smaller docking sites for charging and storage

These are all first indications and all these aspects should be studied in more detail. For example, it is possible to increase the beam length from 4 to 6 m, which results in a higher production per device. The fact that less AWIDs are required can be an advantage, but looking at the limited sailing range this can also introduce a problem. When developing this concept in more detail, the optimum size of the AWIDs has to be estimated. Factors to take into account are:

- Production per AWID
- Workability
- Working method in basins
- Battery capacity
- Vessel characteristics
- Harbour layout
- Distribution of sedimentation
6 Conclusions and recommendations

In this final part of this thesis the main conclusions are given and recommendations are made for further research and development.

6.1 Conclusions

It can be concluded that the initial concept of a submerged, free floating dredging device that uses suction is not feasible. Due to the relatively large suction force it is very difficult to control the stability of the device. Attempts to improve the design all led towards a device that at least partly rests on the bed. Adding the fact that jets are in most cases required to loosen the soil, this brings us to a regular TSHD draghead.

From a more practical point of view, it can be concluded that starting this project with physical model tests instead of theoretical considerations and a literature study was not an ideal way of working. However, the model tests were of use since they helped in developing a practical sense of the ongoing processes.

The more abstract idea of an autonomous dredge to remove mud from lakes or harbour basins can be regarded as a promising concept. Different possibilities have been compared, amongst which an auger dredge on tracks, an underwater hovercraft and a floating water injection device. The water injection dredging (WID) device is considered as the most suitable dredging method for an autonomous device. Advantages of this technique are that no direct contact with the bed is required and that the dredged material flows out as a density current, eliminating the need of a discharge system with pipelines or a vessel with a storage area.

Concerning the processes that play a role in WID, it can be stated that no comprehensive model is publicly available. A considerable amount of research on sub processes was done in the past, but often the results are not made public. Therefore, in this thesis public available knowledge has been combined into a model that covers the full WID process.

For the jetting part of the water injection process a vertical intrusion model can be used to estimate an intrusion distance. Since not all material will be put into suspension, a production reducing factor is applied. Using the propagation velocity of the dredge, the fluid mud discharge can be calculated. After applying a factor to model the entrainment of water directly behind the jet, an input discharge and density for the outflow model are obtained.

In the past it has been suggested to use a 2-layer model for the flow of a density current generated by WID. This model leads to a backwater curve equation, which requires a downstream boundary condition. Since this boundary condition is not known in the current case, this model is not applicable. Therefore a model based on the Chézy formula is used. The model is sensitive for a several parameters that are dependent on the specific situation of the dredging project. The resulting travel distance of the density current lies between 0.5 and 5 km, depending on the local circumstances.

With the current state of battery technology, it is possible to design a AWID (autonomous WID) with reasonable dimensions that is able to work for several hours. A design with a 4 m jet beam, 500 kW installed power and 2000 kWh battery pack is feasible. This leads to a vessel that can dredge for four hours before it has to return to the charging dock.
In the final part of this thesis a quick calculation has been made on the autonomous removal of mud from the Port of Rotterdam using WID. First the Botlek harbour is looked upon and afterwards the results are extrapolated to the full port. By dividing the Botlek harbour in three areas and estimating for each area the amount of material that has to be transported over a certain distance, a rough production figure with the unit \( m^3 \cdot km/\text{year} \) has been determined. Using the estimated production of the AWID device, it follows that it takes about 5 months per year to keep the Botlek harbour free of mud.

Since no detailed dredging data on other parts of the Port of Rotterdam was available, a rough interpolation using the total water area of the port led to a system with 6 AWID devices. These devices are combined with six docks that are located in different parts of the harbour so that each of the devices serves its own area and travel times are minimized. One dock serves as the main dock where maintenance and repair can take place and the other five are for recharging the battery pack and sheltering the AWIDs when they are not in use.

A last general observation that appeared throughout this graduation project is the always present difficulty when trying to apply theoretical models on practical applications. In this thesis on multiple occasions input parameters have been chosen and extra coefficients have been introduced in such a way that the calculation results reasonably agree with available practical data. In other words, to ensure that the results make sense. Also, several times a simple rule of thumb proved to be at least as useful as a complicated model.

### 6.2 Recommendations

Not much is known about the processes in the direct vicinity of the water jet during WID. Although a considerable amount of research has been conducted on the intrusion of a jet in clayey materials, it was found to be very difficult to apply this knowledge to the situation in this thesis. After intrusion, a highly turbulent mixture has developed that somehow will obtain a certain height. Using more advanced modelling, it is probably possible to make a better estimation of the thus reached mixture height. This height and velocity could serve as input values for an outflow model.

Considering the outflow model, also a more advanced model can be developed. Especially the concept of settling needs more attention, since this factor has a high influence on the travel distance of the density current. As with jetting, quite some research has been conducted on this topic, but the translation to a practical application in WID needs more attention.

Another topic of interest is the relationship between the properties of the mud and other local circumstances on the results of the WID process. The available field data on WID shows an enormous variation in for example production and travel distance. When developing more advanced models, the influence of the input parameters needs to be known in more detail in order to obtain feasible results. Therefore it would be good to compare model results with practical data.

Looking at the concept of autonomous dredging, a further and more detailed study on the desired properties of autonomous dredging devices is desirable. The presented figures are based on rough estimations, so with more detailed data it is possible to optimize the initial design as presented in this report. Not only a civil engineering perspective is required, but also mechanical, maritime and electrical engineering play an important role. Examples of interesting topics are the exact form factor of the device, the power supply system and the control system to enable the device to clean a certain area as efficient as possible. A collaboration with the Port of Rotterdam or another port authority seems favourable to make such a study a success.
References

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<tr>
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<th>Reference</th>
</tr>
</thead>
</table>

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46. VPRO Tegenlicht (Dutch television), *The smartest port in the world* (2015).


**List of symbols**

In this list the symbols as used in the main text are given.

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<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<td>$A_n$</td>
<td>nozzle area</td>
<td>$[m^2]$</td>
</tr>
<tr>
<td>$b$</td>
<td>jet width for which $u &gt; 0.5u_{\text{max}}$</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$c$</td>
<td>mass concentration</td>
<td>$[\text{kg/m}^3]$</td>
</tr>
<tr>
<td>$c_u$</td>
<td>undrained cohesion soil</td>
<td>$[\text{Pa}]$</td>
</tr>
<tr>
<td>$C$</td>
<td>Chézy coefficient</td>
<td>$[m^{1/2}/s]$</td>
</tr>
<tr>
<td>$C^*$</td>
<td>dimensionless Chézy coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$c_d$</td>
<td>nozzle discharge coefficient</td>
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<td>$d_{\text{wat}}$</td>
<td>water depth</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$D_{\text{jump}}$</td>
<td>distance between jets and hydraulic jump</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$d_n$</td>
<td>nozzle diameter</td>
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<td>$f_{\text{eff}}$</td>
<td>production efficiency factor</td>
<td>[-]</td>
</tr>
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<td>$f_{\text{jet}}$</td>
<td>jetting efficiency factor</td>
<td>[-]</td>
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<td>$f_{\text{entr}}$</td>
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<td>gravitational acceleration</td>
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<td>flow thickness</td>
<td>[-]</td>
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<tr>
<td>$h$</td>
<td>bed level</td>
<td>$[m]$</td>
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<td>$h_n$</td>
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<td>$[m]$</td>
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<td>thickness upper layer</td>
<td>$[m]$</td>
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<tr>
<td>$h_2$</td>
<td>thickness lower layer</td>
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<td>[-]</td>
</tr>
<tr>
<td>$i_b$</td>
<td>bed slope</td>
<td>[-]</td>
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<tr>
<td>$k_1 = k_2$</td>
<td>entrainment coefficient for jetting</td>
<td>[-]</td>
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<td>$L_{dc}$</td>
<td>travel distance density current</td>
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<td>$n_n$</td>
<td>number of nozzles</td>
<td>[-]</td>
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<td>$p$</td>
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<td>$P_{\text{jet}}$</td>
<td>jet power</td>
<td>$[\text{kW}]$</td>
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<td>specific discharge</td>
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<td>$Q$</td>
<td>discharge</td>
<td>$[\text{m}^3/\text{s}]$</td>
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<td>$Q_{\text{in}}$</td>
<td>soil discharge entering system</td>
<td>$[\text{m}^3/\text{s}]$</td>
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<tr>
<td>$Q_n$</td>
<td>discharge single nozzle</td>
<td>$[\text{m}^3/\text{s}]$</td>
</tr>
<tr>
<td>$r$</td>
<td>radial distance from jet axis</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$s$</td>
<td>coordinate along jet trajectory</td>
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</tr>
<tr>
<td>$s_{\text{dr}}$</td>
<td>distance to nozzle where flow fully developed</td>
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<td>$s_{\text{intr}}$</td>
<td>intrusion depth jet</td>
<td>$[m]$</td>
</tr>
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<td>time step</td>
<td>$[\text{s}]$</td>
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<td>Description</td>
<td>Unit</td>
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<tr>
<td>--------</td>
<td>----------------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>$u$</td>
<td>velocity (density) flow</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_0$</td>
<td>outflow velocity jet</td>
<td>[m/s]</td>
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<tr>
<td>$u_{\text{max}}$</td>
<td>velocity at centreline jet</td>
<td>[m/s]</td>
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<td>hauling velocity of dredge</td>
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<td>$v_{\text{set}}$</td>
<td>settling velocity</td>
<td>[m/s]</td>
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<tr>
<td>$w$</td>
<td>water content</td>
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<tr>
<td>$w_{\text{dr}}$</td>
<td>width of WID beam</td>
<td>[m]</td>
</tr>
<tr>
<td>$x$</td>
<td>horizontal distance</td>
<td>[m]</td>
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<tr>
<td>$\delta$</td>
<td>thickness interface layer</td>
<td>[m]</td>
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<td>$\varepsilon$</td>
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<tr>
<td>$\phi$</td>
<td>volumetric concentration</td>
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</tr>
<tr>
<td>$\mu$</td>
<td>dynamic viscosity</td>
<td>[Pa.s]</td>
</tr>
<tr>
<td>$\Delta p$</td>
<td>pressure drop over jet</td>
<td>[Pa]</td>
</tr>
<tr>
<td>$\rho$</td>
<td>density</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>density solids</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\rho_{\text{situ}}$</td>
<td>density in-situ material</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>density water</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\tau$</td>
<td>friction between layers</td>
<td>[Pa]</td>
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<th>Description</th>
<th>Source</th>
</tr>
</thead>
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</tr>
<tr>
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<td>Source: PIANC (22)</td>
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<td>Figure 3.9:</td>
<td>The cutter suction dredger.</td>
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<td>Figure 3.10:</td>
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<td>Source: oceaneering.com</td>
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<td>SMD QTrencher 1000.</td>
<td>Source: smd.co.uk</td>
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<td>LWT Mud Cat ROV submersible crawler dredge.</td>
<td>Source: lwtdredge.com</td>
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<td>Figure 3.15:</td>
<td>The Punaise dredge.</td>
<td>Source: theartofdredging.com</td>
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<td>Source: theartofdredging.com</td>
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<td>Source: Sarkas, Bose and Sarkar (28)</td>
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<td>The Submersible dredge by Sloan (32)</td>
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<td>Figure 3.20:</td>
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<td>Figure 3.23:</td>
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Appendix A: Blobfish

The incentive of this graduation research project was given by the concept called Blobfish. Grontmij developed this concept as a part of an innovative system for the autonomous dredging of mud in a lake. Since an application in other locations was also expected to be possible, the initial goal of this research project was to study the Blobfish concept in more detail. After both theoretical and experimental research had been carried out, it turned out that the Blobfish concept was not feasible. However, the general idea of a small, autonomous dredging device to remove mud was still considered to be an interesting topic for a research project. In this appendix the study on the Blobfish device is presented for future reference.

A-1 Introduction

Grontmij developed a concept called Blobfish, a submerged dredging apparatus that hovers just above the bed. The prototype model has the shape of a flat plate with in the centre a suction tube. The desired way of functioning is that soil is removed by the water flow under the plate. This way, a sand-water mixture flows from all directions to the suction mouth. The desired application is the dredging of thin layers of material. Both sand and mud are considered to be soil types that can be removed with this device.

To reach an equilibrium state in the vertical direction, at different locations chains are connected to the bottom side of the plate. The purpose of these chains is that they stabilize the plate. When the plate moves upwards, more shackles leave the bottom, resulting in a downward force. When the plate moves down, more shackles will lie down on the bottom, decreasing the downward force. This way, the chains should be able to compensate for a varying vertical forces resulting from a varying flow density and under pressure.

There are multiple ways possible to move the device, for instance by means of a cable system or propellers. These options have not been worked out since the focus of this research is on the suction system itself. In the initial design the pump is located at an external location, but in real life, the pumping installation has to be mounted on the device itself.

![Figure A.1: Artist impression Blobfish](image-url)
A-2 General considerations

In this paragraph the key processes that play a role in the functioning of the Blobfish device are discussed. The properties of the initial flat plate design are described together with suggestions to improve the design.

A-2.1 Radial versus linear flow
Underneath the plate of the Blobfish a radial flow pattern will occur. A flow velocity of 5 m/s in a pipeline with a diameter of 0.5 m results in a flow velocity of 1 m$^3$/s in the pipe. When assuming a constant distance of 0.2 m between the plate and bed, radial flow results in a flow velocity pattern under the plate as shown in Figure A.2. From the figure it becomes obvious that with an increasing distance from the suction pipe the flow velocity quickly becomes very small, leading to a small erosion velocity.

![Radial flow velocity](image)

Figure A.2: Radial flow velocity

It is questionable whether the use of radial flow is a good solution. A regular linear suction head, in practise perhaps consisting of a row of circular suction heads, could be a better solution.

A-2.2 Flow and erosion processes
To determine the flow and erosion processes, first the water flow field has to be investigated and on the basis of the found flow velocities the erosion rates can be calculated. The erosion process itself changes the morphology and thus influences the flow field and velocity pattern, introducing a feedback mechanism.

Assuming a uniform erosion profile and a uniform distance between bed and plate, there is radial flow underneath the plate, as described in the previous paragraph. When the erosion is not uniform, which is a highly probable situation, there is no simple radial flow. The flow and erosion pattern depend on the shape of the plate. For a flat plate, there are two locations where the flow velocity is probably larger than elsewhere, resulting in a large erosion: directly underneath the suction pipe and at the sides of the plate. This is indicated in Figure A.3. A possibility is to give the plate a shape in such a way that the flow and thus erosion velocity are more constant.

At the sides of the plate, where the water flow enters the system, the area through which the water flows is suddenly reduced. This situation can be compared to one half of a Borda pipe. The Borda effect entails that the flow is not able to follow the 180 degree bend into the area underneath the plate. This creates a circulating flow pattern at the entrance of the narrower pipe. The flow expansion and deceleration after a certain distance result in energy losses. Because of the contraction higher velocities are expected at the edges of the plate, resulting in higher flow velocities and thus an...
increased erosion rate. The energy losses can be minimized by streamlining the water inflow area at the sides of the plate by increasing the distance to the bed locally. Also if the plate has a large size and therefore a low flow velocity at the border, this effect will be reduced.

Figure A.3: Cross section of the erosion pattern under the Blobfish

In Figure A.4 the results from both the erosion formula by Van Rijn and Van Rhee are presented, together with experimental data (5). It follows that erosion of sand starts to take place at a flow velocity of about 0.5-1 m/s, although the erosion rate will be low. Figure A.2 shows that this velocity can be observed at a radial distance of about 1-2 meter.

Figure A.4: Erosion velocity as a function of flow velocity. Source: Bisschop, Visser, Van Rhee and Verhagen (5)
A-2.3 Plate stability and supporting forces
The plate is expected to be pushed towards the bed due to the suction effect. This results in a smaller gap between the plate and bed, increasing the flow velocity and thus increasing the suction force. This is obviously an unstable situation. However, it is possible that due to the increased flow velocity the erosion velocity also increases, resulting in a deeper erosion hole, which leads to lower flow velocities. It is difficult to tell which of these two effects will dominate.

At the bottom side of the plate a number of chains is mounted to stabilize the equilibrium. To ensure that the plate will not get stuck, ‘legs’ or bumpers can be used. The location of these devices should be such that they are not resting on areas that are being eroded because the soil in these areas will be unstable. Also, the forces and bearing capacity of the soil have to be taken into account to ensure that the legs or bumpers provide a force that is sufficient for stabilizing the device.

In order to minimize the friction force when moving the plate, the contact area between plate and bed should be as small as possible. It has to be investigated whether a fully floating system is realistic. Also a suitable design for the legs has to be found that works well when the device is moving. When using very simple legs, it is possible that the torque becomes too large.

Alternatively it is possible to support the device on the bed. Similar to what is done in regular drag heads, the edges of the plate can be made flexible. These ‘flaps’ are hinged to the plate and dragged over the bed.

Another way to lower the under pressure below the plate is to use valves that allow water to enter from above. This is a commonly used technique in regular TSHD heads. But although this way the pressure can be lowered, the extra amount of clear water that is pumped makes the system less energy efficient.

A-2.4 Dimensioning the chains
The chains have to compensate a variable vertical force that has two main components:
1. The suction force
2. The mixture density in the part of the pipe that rests on the plate

In order to make initial calculations suction force, it is assumed an average flow velocity of 1 m/s occurs over the full plate area. A plate diameter of 6 m is assumed. This causes a pressure difference of:

\[
p = \frac{\rho v^2}{2} = \frac{1000 \cdot 1^2}{2} = 0.5 \text{kPa} \tag{A.1}
\]

The plate area is:

\[
A_{\text{plate}} = \pi r^2 = \pi \cdot 3^2 = 28 \text{m}^2 \tag{A.2}
\]

Now, the total downward force due to suction is:

\[
F_{\text{suction}} = p \cdot A_{\text{plate}} = 14 \text{kN} \tag{A.3}
\]

The variable mixture density in the suction pipe exerts a variable force on the plate. It is assumed that the part of the pipe that exerts a force on the plate has the following properties:
- A length of 2 times the plate radius of 3 m: L=6 m
- Pipe radius \( r = 0.25 \text{ m} \)
This results in a volume of this pipe section of:

\[ V = \pi r^2 = 6\pi \times 0.25^2 = 1.2m^3 \quad (A.4) \]

When the device is not working, it is assumed that the pipe is filled with water, so it exerts not extra force on the plate. When working with a low density mixture, a density of 1200 kg/m\(^3\) is assumed, and for a high density mixture a density of 1700 kg/m\(^3\). In these two cases the pipe exerts an extra downward force on the plate.

For the high density mixture:

\[ F_{pipe, high} = V \cdot \Delta \rho \cdot g = 1.2 \cdot 700 \cdot 9.81 = 8kN \quad (A.5) \]

For the low density mixture:

\[ F_{pipe, low} = V \cdot \Delta \rho \cdot g = 1.2 \cdot 2 \cdot 9.81 = 2kN \quad (A.6) \]

To calculate the properties of the chains, the following 3 situations have to be taken into account.

1. **Working with high density mixture**

When working with a high density mixture, the downward force is at its maximum and all chains rest on the bed. In order to create a vertical equilibrium, an upward buoyant force is required.

This buoyant force can be calculated using the following assumptions:

- The mass of the device is 2000 kg, resulting in a gravity force \( F_z = 20kN \).
- The force due to the pipe is \( F_{pipe, high} = 8kN \).
- The suction force is \( F_{suction} = 14kN \).

\[ F_{buoy} = F_z + F_{pipe, high} + F_{suction} \quad (A.7) \]

This results in a required buoyancy of \( F_{buoy} = 42kN \).

![Figure A.5: Situation with high density mixture](image)
2. **Working with low density mixture**

In this case a number of chains is not on the bed since the mixture density is lower. The downward force causes by the chains can be calculated with the following assumptions:

- The mass of the device is 2000 kg, resulting in a gravity force \( F_g = 20kN \).
- The force due to the pipe is \( F_{pipe,low} = 2kN \).
- The suction force is \( F_{suction} = 14kN \).
- The buoyancy force \( F_{buoy} = 42kN \).

The following equilibrium equation holds in this situation:

\[
F_{buoy} = F_g + F_{chain} + F_{pipe,low} + F_{suction}
\]

\((A.8)\)

It follows that in this case \( F_{chain} = 6\) kN.

![Figure A.6: Situation with low density mixture](image)

3. **The device is not working**

When the device is not working there is no suction force and it is assumed that the pipeline is filled with water. The full amount of chains is now required to keep the device in equilibrium.

The downward force caused by the chains can be calculated with the following assumptions:

- Estimating the mass of the device to be equal to 2000 kg, resulting in a gravity force \( F_g = 20kN \).
- The buoyancy force \( F_{buoy} = 44kN \)

The following equilibrium equation holds in this situation:

\[
F_{buoy} = F_g + F_{chain}
\]

\((A.9)\)

This results in a required chain force \( F_{chain} = 24\) kN.
Chain length
Assuming that the total steel cross section of the chain is 1 dm² and using a steel density of 8000 kg/m³ it follows that in total 30 m of chain is required. Looking at the low density mixture case it follows that now 7.5 m of chain has to be in the water. Assuming a distance between the device and bed of 20 cm, it follows that 38 chains are required. For a plate with a diameter of 6 meter, this is 1 chain every 50 cm. This will certainly have an effect on the flow and does not seem to be feasible.

Pontoon
Another possibility is to use an pontoon to compensate for the downward forces. For example, a pontoon of 5*5 m² that has to compensate the same force of 42 kN lowers into the water over the following distance:

\[ \Delta h_{\text{pontoon}} = \frac{F}{A_{\text{pontoon}} \rho w g} = \frac{42000}{25 \cdot 1000 \cdot 9.81} = 0.17m \quad (A.10) \]

Conclusions
- From simple calculations it follows that in order to compensate the suction forces a high number of chains is required.
- It seems to be difficult to mount this number of chains on the device.
- The distance between the plate and bed should be only a few centimetres, this is not enough for a proper weight force compensation.
A-2.5 Layer thickness (stationary and moving)
The resulting thickness of the removed layer at different locations can be investigated both for a stationary and a moving plate. When the plate is flat and stationary, the hole is expected to be deeper at the centre and narrower at the sides due to different flow velocities.

When moving the plate, some sections will be more eroded than others, resulting in an uneven erosion pattern. The centre of the hole will be deeper than the sides because erosion has taken place for a longer time and moreover at a higher rate. This effect is clarified in Figure A.8, where at the left side a sketch of the erosion pattern as function of the radial distance is given. This pattern follows from the radial velocity in combination with the relationship between the erosion and flow rate. At the left side of the sketch the shaded areas give an indication of the time a particular area is being eroded.

At the back side of the plate (the direction where the plate comes from) there will be a hole, and the resistance will be low, resulting in the inflow of big quantities of clear water: in Dutch known as ‘vals water’. In commonly used drag heads this effect is minimized by using a movable visor with flaps to close the gap between the bottom and the head as much as possible.

![Erosion pattern](image)

Figure A.8: Side view and top view erosion pattern

![Possible shape to equalize erosion](image)

Figure A.9: Possible shape to equalize erosion
A-3 Experimental results

In this paragraph the setup and results of the small scale experiments are presented. The experiments have been carried out in a steel water tank at the IHC Innovation Lab in Rotterdam. The aim of the experiments was to get a feeling for the processes and forces that play a role. Therefore only very basic tests have been done.

A-3.1 Experimental setup

An overview of the setup is given in Figure A.10. The setup consists of the following parts:

- Blobfish scale model (Figure A.11 and Figure A.12)
- Steel box with dimensions 4x0.8x0.8 m³ (Figure A.13)
- V-notch weir to measure the discharge (measured leakage of 0.5 m³/hr)
- Saer BP 5 pump ($P_{\text{max}} = 2.6$ kW and $P_{\text{nom}} = 1.5$ kW) with characteristics as given in Figure A.14
- Valve to control the discharge. In Figure A.15 the relation between the valve setting and discharge is given.
- Suction pipe with inner diameter of 4 cm and 2 floatation devices
- Discharge pipe with inner diameter of 8 cm
- Geotextile bag for sand and weight measurement device

On page 122 a visual impression of the full setup is presented.

The Blobfish scale model is based on a hexagonal plate with diameter of 0.65 m. In the centre a suction tube is mounted. Three plastic boxes filled with air are used as floatation devices. Weights can be added to change the buoyant force of these boxes.

The weights of the different components are given in Table A.1.

<table>
<thead>
<tr>
<th>Component</th>
<th>Force [N]</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perspex plate</td>
<td>6</td>
<td>Down</td>
</tr>
<tr>
<td>Discharge pipe (filled)</td>
<td>5</td>
<td>Down</td>
</tr>
<tr>
<td>Suction</td>
<td>88</td>
<td>Down</td>
</tr>
<tr>
<td>Floaters (3 pieces)</td>
<td>8</td>
<td>Up</td>
</tr>
<tr>
<td>Resulting on supports</td>
<td>91</td>
<td>Down</td>
</tr>
</tbody>
</table>

Table A.1: Weights of components Blobfish

Figure A.10: Schematic overview setup
Figure A.11: Scale model Blobfish

Figure A.12: Schematic picture Blobfish

Figure A.13: Steel box used for experiment
Figure A.14: Pump characteristics Saer BP 5. Source: saerelettropompe.com

Figure A.15: Relation between the setting of the pump valve (as measured in mm displacement of the screw) and the discharge as measured with the V-notch weir
A-3.2 Results
The following parameters have been used during the experiments:

- Water height: 40 cm
- Sand layer of about 2 cm
- Pump discharge: 10 m³/hr
- Grain size sand: 100 μm. Origin and shape are unknown. It can be assumed that locally the sand is packed very densely.
- Supporting feet: 3 pieces with a height of 25 mm
- Suction head extends about 10-25 mm from the plate

The experiments made clear that a very small distance between the plate and bed is needed to be able to remove the sand. This means that it is difficult to use chains to compensate for the variable vertical forces since these need a longer distance.

The Blobfish can be stabilized reasonably when it is not pumping and the chains seem to have stabilizing effect. However, the weight of discharge pipe has a big effect on the equilibrium since it is relatively heavy compared to the other parts of the system.

When the Blobfish is close to the bottom during pumping, as is required to reach sufficiently high flow velocities, the 3 feet are pressed towards the bed due to the suction force. This results in a considerable drag force. Moving the device is possible and the drag force is about 20 N. In fact, the device is being dragged over the bed. Without pumping the required force to move the device is about 10 N.

Without first loosening the sand by hand there is no production. The very fine sand that was available had been in place for a few weeks and therefore reached a considerable level of consolidation. Therefore the soil had to be loosened to be able to be dredged.

In a time of 2 minutes about 13 kg of wet sand was produced. It has to be noted that the compaction state of the sand is unknown and the sand is locally probably very dense. With a wet density of 2000 kg/m³ the found production equals the following volume:

\[
V_{\text{with pores}} = \frac{13 \text{kg}}{2000 \text{ kg/m}^3} = 0.007 \text{m}^3
\]  
\((A.11)\)

During 2 minutes pumping at 10 m³/hr, the following amount of water flows through the suction head:

\[
V_{\text{water}} = \frac{2}{60} \text{hr} \cdot 10 \text{ m}^3/\text{hr} = 0.17 \text{ m}^3
\]  
\((A.12)\)

This results in a concentration of:

\[
C = \frac{V_{\text{with pores}}}{V_{\text{water}}} = 0.04
\]  
\((A.13)\)
A-3.3 Conclusions

- Even for a the used small circular head and low flow velocities the device is pushed into the bed and is difficult to control. The expectation of a free floating suction device is unrealistic.
- Instead of a circular system, a linear head is more realistic.
- Another way to compensate for the suction forces is required since the chains do not function well in practise.
- Dredging by erosion with a low flow velocity results in low productions. Probably jets are required to increase the production.

These results all point to a regular TSHD draghead. This makes sense when looking at the amount of development this common type of draghead has undergone during the past decades. Therefore, after these experiments it was decided to stop the research on the Blobfish and change the scope of the research project.
Figure A.16: Photo impression Blobfish experiments
A-4 Calculations on experiment

In this paragraph calculations are done on the experimental setup to check whether the found results make sense from a theoretical point of view.

The following parameters are used, in accordance with the experimental parameters:

\[
\begin{align*}
 r_{pipe} &= 0.02 \text{ m} & \text{radius suction pipe} \\
 Q &= 10 \text{ m}^3/\text{hr} = 0.0028 \text{ m}^3/\text{s} & \text{discharge} \\
 d_{gap} &= 0.02 \text{ m} & \text{distance between plate and bed} \\
 \nu_{pipe} &= 2.3 \text{ m/s} & \text{flow velocity in pipe}
\end{align*}
\]

A-4.1 Flow velocity pattern

When the changing bed form is not taken into account, a radial flow pattern can be expected. From the experiments it followed that most of the erosion took place directly underneath the suction mouth. Therefore the flow velocity at this location deserves extra attention. Below two different assumptions are used to calculate the erosion pattern.

Assumption 1

A first possible assumption is that that below the suction pipe the velocity is equal to that in the pipe and that starting at the edge of the pipe the velocity decreases in radial direction. This results in the flow velocity pattern as shown in Figure A.17. This is assumption is rather rough and more insight in the flow pattern is required to be able to estimate the flow velocity underneath the suction mouth in a better way. Using this simplification, it is clear that the biggest part of the erosion will take place underneath the suction mouth.

![Radial flow velocity](image)

Figure A.17: Radial flow velocity assumption 1
**Assumption 2**
A second possible assumption is that the flow velocity below the suction pipe is equal to that at the edge of the pipe. The resulting flow velocity pattern is given in Figure A.18.

![Radial flow velocity](image)

**Figure A.18: Radial flow velocity assumption 2**
Probably the flow pattern is more complex since turbulent effects will play a role. It is to be expected that due to this turbulence there will be erosion under the full suction head. But for a first estimate, the two assumptions can be used as upper and lower bounds. That is, a velocity between 1.1 and 2.3 m/s below the suction mouth and a decreasing velocity in radial distance from the suction mouth.

### A-4.2 Total suction force
In this paragraph an estimation will be made for the downward force caused by the suction effect. To convert a velocity profile to a pressure profile the following formula can be used

\[ p = \frac{\rho v^2}{2} \]  

(A.14)

Applying this on the velocity profile found under assumption 1 in the previous paragraph gives the pressure profile as presented in Figure A.19. The unrealistic high velocities lead to an unrealistic pressure. However, since the area is very small, even in this case the force under the mouth itself will be small.

![Pressure](image)

**Figure A.19: Suction force under plate**
For a radius larger than the pipe diameter, the flow velocity can be expressed as a function of the radius \( r \) with:

\[
\nu(r) = \frac{Q}{2\pi rd_{\text{gap}}} \quad (A.15)
\]

Combining formulas (A.14) and (A.15) gives:

\[
p(r) = \frac{\rho Q^2}{4\pi^2 r^2 d_{\text{gap}}^2} = C \frac{1}{r^2} \quad (A.16)
\]

With \( C \) a constant:

\[
C = \frac{\rho Q^2}{4\pi^2 d_{\text{gap}}^2} \quad (A.17)
\]

The total force consists of the force under the mouth and the force under the rest of the plate:

\[
F_{\text{tot}} = F_{\text{mouth}} + F_{\text{rest}} \quad (A.18)
\]

With the force underneath the mouth:

\[
F_{\text{mouth}} = pA = \frac{\rho Q^2}{2} \frac{r_{\text{pipe}}^2}{\pi r_{\text{pipe}}^2} = 3N \quad (A.19)
\]

And a force under the rest of the plate:

\[
F_{\text{rest}} = \int_{r_{\text{gap}}}^{r_{\text{pipe}}} \rho \cdot dr \cdot d\varphi = 2\pi \int_{r_{\text{gap}}}^{r_{\text{pipe}}} \rho \cdot dr = 2\pi C \int_{r_{\text{gap}}}^{r_{\text{pipe}}} \frac{1}{r} dr = 2\pi C \left[ -\frac{1}{r} \right]_{r_{\text{gap}}}^{r_{\text{pipe}}} = 88N \quad (A.20)
\]

This leads to a total force \( F_{\text{tot}} = 88N \). It is striking that the suction underneath the mouth itself generates only a very small part of the total force.

This force is valid during the period when the distance between the plate and bed is still equal to the initial distance of 2 cm. It is to be expected that this distance increases due to erosion, leading to a smaller velocity and thus a smaller force. So the studied situation gives the highest expectable force, assuming that the distance of 2 cm can be maintained.

**A-4.3 Force on supports**

Assuming that during dredging the Blobfish rests on the bed, the total downward force of 88 N has to be counteracted by the three supports. It can be expected that the supports need to have a certain area due to the limited bearing capacity of the soil.

The Brinch-Hansen formula can be used to calculate the bearing capacity of the bed. However, it is questionable whether the results of this formula are valid for the small areas and depths that are used in this case. In Figure A.20 a relation between the depth in the soil and the bearing capacity is given for a square support of \( 2\times2 \text{ cm}^2 \).
Figure A.20: Relation between bearing capacity and depth of support in soil

With these values it can be calculated how many supports are required to compensate the downward force of 88 N. This is done for supports of 1x1, 2x2, 5x5 and 10x10 cm\(^2\). The calculation has been executed for the case the supports do not protrude into the soil and for the case that they protrude 2 cm into the soil. The results are given in the tables below.

<table>
<thead>
<tr>
<th>Size of support</th>
<th>Bearing capacity not in soil [kPa]</th>
<th>Required support area [m(^2)]</th>
<th>Required number of supports</th>
</tr>
</thead>
<tbody>
<tr>
<td>1x1</td>
<td>0.75</td>
<td>0.12</td>
<td>1173</td>
</tr>
<tr>
<td>2x2</td>
<td>1.5</td>
<td>0.06</td>
<td>150</td>
</tr>
<tr>
<td>4x4</td>
<td>3</td>
<td>0.03</td>
<td>19</td>
</tr>
<tr>
<td>8x8</td>
<td>6</td>
<td>0.015</td>
<td>2</td>
</tr>
</tbody>
</table>

Table A.2: Supports that do no protrude into the soil

<table>
<thead>
<tr>
<th>Size of support</th>
<th>Bearing capacity 2 cm in soil [kPa]</th>
<th>Required support area [m(^2)]</th>
<th>Required number of supports</th>
</tr>
</thead>
<tbody>
<tr>
<td>1x1</td>
<td>3</td>
<td>0.03</td>
<td>293</td>
</tr>
<tr>
<td>2x2</td>
<td>4</td>
<td>0.022</td>
<td>55</td>
</tr>
<tr>
<td>4x4</td>
<td>5</td>
<td>0.018</td>
<td>11</td>
</tr>
<tr>
<td>8x8</td>
<td>8</td>
<td>0.011</td>
<td>1</td>
</tr>
</tbody>
</table>

Table A.3: Supports that protrude 2 cm into the soil

These tables show that according to the Brinch-Hansen formula the supports can only deliver small supporting force. If these numbers are right, the tables show that, unless a substantial amount of rather large supports is used, the supports will be buried completely into the ground. This means that the full plate rests on the bed.
A-4.4 Drag/friction forces due to support

In order to calculate the forces that are required to move the device, several assumptions can be made, leading to different models.

Assumption 1
Assuming the supports are resting on top of the soil, the friction between the three supports and the soil can be calculated.

Assumption 2
A second possibility is that the supports protrude into the soil over a certain depth and that the soil in front of the supports exerts an additional force on the supports.

Assumption 3
If the calculations on the supports in the previous paragraph were right, the used supports were far too small to enable the device to be on top of the sand. Hence there would be friction between the full plate and the soil. Since the supports are inside the soil, this adds another friction component.

Since in the experiment a force of only 20 N was required to move the device, the first assumption is used. The formula for friction force can be used:

\[ F_{\text{friction}} = \mu F_N \]  \hspace{1cm} (A.21)

With

- \( F_{\text{friction}} \) = Friction force \[ \text{[N]} \]
- \( \mu \) = Friction coefficient \[ \text{[-]} \]
- \( F_N \) = Normal force \[ \text{[N]} \]

With \( \mu = 0.4 \) a friction force of 20 N leads to a normal force of 50 N. This suggests that the calculated suction force is higher than the suction force that occurred in practise.

Assumption 2 and 3 lead to even lower normal (and thus suction) forces due to the extra component of the part of the support that is inside the soil. This suggests that the supports do not penetrate into the soil, which is in contradiction with the findings in the previous paragraph.

The forces exerted on the supports that slide through the soil can be modelled using the theory for the cutting of sand. This is a very rough estimation but gives a first indication. The formula for the force required for cutting in wet sand as given by Miedema (18) is:

\[ F_h = \frac{c_l \rho_s g v h^2 c w}{k_m} \]  \hspace{1cm} (A.22)

In which:

\[ \varepsilon = \frac{n_{\text{max}} - n_i}{1 - n_{\text{max}}} \]  \hspace{1cm} (A.23)

\[ k_m = \frac{1}{2} k_i + \frac{1}{2} k_{\text{max}} \]  \hspace{1cm} (A.24)
With
\[
\begin{align*}
v_c &= 0.1 \text{ m/s} & \text{velocity} \\
h_b &= 0.02 \text{ m} & \text{height ‘blade’} \\
h_i &= 0.02 \text{ m} & \text{height cut layer} \\
w &= 0.02 \text{ m} & \text{width} \\
n_i &= 42 \% & \text{initial porosity} \\
n_{\text{max}} &= 50 \% & \text{maximum porosity} \\
k_i &= 0.00002 \text{ m/s} & \text{initial permeability} \\
k_{\text{max}} &= 0.0002 \text{ m/s} & \text{maximal permeability}
\end{align*}
\]

The factor $c_1$ depends on the following angles:
\[
\begin{align*}
\alpha &= 60^\circ & \text{blade angle} \\
\delta &= 30^\circ & \text{soil/interface friction angle} \\
\phi &= 42^\circ & \text{angle of internal friction}
\end{align*}
\]

For this combination of angles, Miedema gives:
\[
c_1 = 0.9 - \text{constant}
\]

This results in a horizontal force $F_h = 10N$ per support. So for the 3 supports this would total $F_{h,\text{tot}} = 30N$. This is a very rough estimate, since the formula is geared to cutting blades and the parameters are not verified, but it gives an first estimation of the force required to drag the supports through 2 cm of soil.

When the supports are only 1 cm in the soil, so with $h_b = h_i = 0.01m$ the total force is only $F_{h,\text{tot}} = 8N$.

Apparently, since the measured force was low, the supports did not penetrate very deep into the soil.

**A-4.5 Soil production estimation**

For the production the velocity is important, since the faster the device moves, the less time there is to deepen a certain stretch. Using $v = 0.1 \text{ m/s}$ and the mouth diameter of $d = 0.04 \text{ m}$, this implies that there will be erosion underneath the mouth during 0.4 s. A simple model has been made in Matlab that calculates the erosion depth after a certain time. The script works as following:

1. Create an array with radial distances and the initial distance between plate and bed at each location (2 cm)
2. Calculate the flow velocity at each radial location
3. Calculate the erosion velocity at each radial location using Van Rijn/Van Rhee
4. Calculate the eroded distance during time step $dt$ at each location
5. Add the eroded distance to the previous bed-plate distance
6. Back to 2

This results in a depth of 0.027 m after 0.4 s. Subtracting the initial plate-bed distance of 0.02 m, this means that 7 mm of sand has been eroded. The erosion profile is shown in Figure A.21.
Figure A.21: Estimation erosion profile under plate

The shape of this erosion pattern is unrealistic but gives a first indication. A shape that is more realistic and easier for calculations would be a cone, as indicated in Figure A.22.

Figure A.22: Schematic profile erosion profile

Modelling the erosion as a cone width a diameter of 0.06 m and a height of 0.028-0.02=0.008 m, results in the following volume:

$$V_{cone} = \frac{1}{3} \pi r^2 h = 3 \cdot 10^{-5} m^3$$  \hspace{1cm} (A.25)

Assuming this cone is eroded in 0.4 s, the production is

$$Pr = \frac{V_{cone}}{t} = \frac{3 \cdot 10^{-5}}{0.4} = 7.5 \cdot 10^{-5} m^3/s$$ \hspace{1cm} (A.26)
During a test of 2 minutes this means that 9 litre, of 18 kg of wet material ($\rho=2000 \text{ kg/m}^3$) can be dredged. This agrees quite well with the result from the experiment (13 kg).

2 points of attention:
1. Van Rijn erosion formula is used, which overestimates the erosion velocity according to Bisschop, Visser, Van Rhee and Verhagen (5)
2. A certain location is during 0.4 s under the suction mouth, but next to the suction mouth there is also erosion taking place. So in fact the bed is eroded during a longer time period. This means that the production is underestimated.

### A-4.6 Production example linear mouth

Since it is expected that a linear suction mouth is more practical, in this paragraph basic calculations are done to estimate the production of such a mouth.

To make a first estimation of the production using a plate with a linear suction mouth, the following figures can be used:

$D = 0.15 \text{ m}$  pipe diameter
$u = 5 \text{ m/s}$  flow velocity in pipe
$c = 25 \%$  concentration
$w_{dr} = 5 \text{ m}$  width of mouth
$v_{dr} = 0.1 \text{ m/s}$  sailing velocity

This leads to a total discharge $Q=0.09 \text{ m}^3/\text{s}$ of which $0.02 \text{ m}^3/\text{s}$ is sand. The mass of this sand is that is discharged per second is equal to:

$$M_{\text{sand}} = Q_{\text{sand}} \frac{\rho_{\text{situ}} - \rho_{\text{water}}}{\rho_{\text{particle}} - \rho_{\text{water}}} \rho_{\text{particle}} = 36 \frac{\text{kg}}{\text{s}}$$  \(A.27\)

With
\[
\begin{align*}
\rho_{\text{water}} &= 1000 \text{ kg/m}^3 \quad \text{density water} \\
\rho_{\text{particle}} &= 2650 \text{ kg/m}^3 \quad \text{density particles} \\
\rho_{\text{situ}} &= 2000 \text{ kg/m}^3 \text{ (n=0.4)} \quad \text{density in-situ}
\end{align*}
\]

The required pump power for this production is:

$$P = \rho_{\text{m}} g Q H = 6.2 \text{kW}$$  \(A.28\)

With
\[
\begin{align*}
\rho_{\text{m}} &= 1400 \text{ kg/m}^3 \quad \text{density mixture} \\
H &= 5 \text{ m} \quad \text{head}
\end{align*}
\]

The efficiency is:

$$\eta = \frac{P}{M_{\text{sand}}} = 0.17 \frac{\text{kJ}}{\text{kg}}$$  \(A.29\)

In order to be able to make a 2D calculation, it is assumed that the side of the mouth are closed. First the required erosion depth for the given discharge, concentration, plate width and sailing velocity has to be calculated.
\[ \text{production} = \text{width} \times \text{depth} \times \frac{\text{length}}{\text{second}} \]

\[ 5 \times d \times 0.1 = 0.02 \quad \text{(A.30)} \]

\[ d = 0.01 \text{ m} \]

Assuming that the length of the mouth in the direction of movement is 0.5 m, the time that every area is being eroded can be calculated with:

\[ t_{\text{erosion}} = \frac{L_{\text{mouth}}}{v_{\text{dr}}} = \frac{0.5}{0.1} = 5 \text{ s} \quad \text{(A.31)} \]

The following erosion velocity is required:

\[ v_e = \frac{d}{t_{\text{erosion}}} = \frac{0.01}{5} = 0.002 \text{ m/s} \quad \text{(A.32)} \]

According to Van Rijn a flow velocity of about \( u = 1 \) m/s is necessary to reach this erosion velocity. The under pressure becomes:

\[ p = \frac{\rho v^2}{2} = \frac{1000 \cdot 1^2}{2} = 500 \text{ Pa} \quad \text{(A.33)} \]

And the total force on the plate is:

\[ F = pA = 500 \cdot 5 \cdot 0.5 = 1250 \text{ N} \quad \text{(A.34)} \]

The distance required between the plate and bed to have discharge \( Q \) for a mouth width \( B \) is:

\[ d = \frac{Q}{w_{\text{dr}}u} = \frac{0.09}{5 \times 1} = 0.018 \text{ m} \quad \text{(A.35)} \]
Appendix B: Modelling of density currents

After water injection dredging has taken place, entrainment has occurred and possibly an internal hydraulic jump has taken place, the mixture has a certain thickness and will start flowing under the influence of gravity as a density current. In this paragraph four different approaches to model a density current are presented.

2. Cordi (1994)
5. Van Rijn (2012)

B-1 Method Kranenburg (1988)

Kranenburg (15) derives the formulas that describe a multi layer flow situation. He uses conservation of mass and momentum to formulate a system of 2n equations, with n the number of layers, to solve for 2n unknowns: the layer thickness and velocity for every layer.

For the special case of a two-layer system equations are derived. The following assumptions are used (23):

- Stable stratification: a less dense fluid above a denser fluid
- Hydrostatic pressure: the horizontal length scale is much larger than the vertical length scale
- Constant density in the layer in both space and time
- No mass transfer between layers
- Horizontal velocity constant in each layer

The situation is schematized as shown in Figure B.1. Here the subscripts 1 and 2 stand for the different layers, and not for the different locations as shown in Figure 4.5.

![Figure B.1: Overview 2-layer model. Source: Pietrzak (23)](image-url)
For this special case of a situation with two layers, the system reduces to two continuity equations and two equations of motion. The continuity equations for respectively the upper and lower layer are:

\[
\frac{\partial h_1}{\partial t} + \frac{\partial}{\partial x} h_1 u_1 = 0 \\
\frac{\partial h_2}{\partial t} + \frac{\partial}{\partial x} h_2 u_2 = 0
\] (B.1)

The equations of motion for the upper and lower layer are:

\[
\frac{\partial u_1}{\partial t} + u_1 \frac{\partial u_1}{\partial x} + g \frac{\partial}{\partial x} (h_b + h_1 + h_2) = -\frac{\tau_0 - \tau_1}{\rho_1 h_1} \\
\frac{\partial u_2}{\partial t} + u_2 \frac{\partial u_2}{\partial x} + \frac{\rho_2}{\rho_2} \frac{\partial h_2}{\partial x} + g \frac{\partial}{\partial x} (h_b + h_2) = -\frac{\tau_1 - \tau_2}{\rho_2 h_2}
\] (B.3)

With

- \( h_b \) = bed height \[\text{[m]}\]
- \( h_1 \) = thickness upper layer \[\text{[m]}\]
- \( h_2 \) = thickness lower layer \[\text{[m]}\]
- \( \tau_0 \) = friction between upper air \[\text{[Pa]}\]
- \( \tau_1 \) = friction between upper and lower layer \[\text{[Pa]}\]
- \( \tau_2 \) = friction between lower layer and bed \[\text{[Pa]}\]
- \( q_2 \) = specific discharge lower layer \[\text{[m}^2/\text{s}]\]
- \( u_1 \) = velocity upper layer \[\text{[m/s]}\]
- \( u_2 \) = velocity lower layer \[\text{[m/s]}\]

The four unknowns in this system are \( u_1 \), \( u_2 \), \( h_1 \), and \( h_2 \). The factors \( \tau \) indicate shear stress due to friction. The shear stress \( \tau_0 \) accounts for the friction between the upper water layer (layer 1) and the wind. Without any wind, this stress is equal to zero. These friction coefficients are described in appendix C-3.

For the case of a stationary flowing density current (layer 2) and non-flowing water layer (layer 1) equations (B.1) to (B.4) can be written as:

\[
h_1 u_1 = q_1 = 0
\] (B.5)

\[
h_2 u_2 = q_2 = \text{constant}
\] (B.6)

\[
g \frac{\partial}{\partial x} (h_b + h_1 + h_2) = -\frac{\tau_0 - \tau_1}{\rho_1 h_1}
\] (B.7)

\[
u_2 \frac{\partial u_2}{\partial x} + \frac{\rho_2}{\rho_2} g \frac{\partial h_2}{\partial x} + g \frac{\partial}{\partial x} (h_b + h_2) = -\frac{\tau_1 - \tau_2}{\rho_2 h_2}
\] (B.8)
Eliminating $u_2$ and $h_1$ from (B.8) using (B.6) and (B.7) gives an equation that describes the gradient of the thickness of the density current. The derivation of this equation is given below.

First (B.5) is rewritten as:

$$u_2 = \frac{q_2}{h_2} \quad (B.9)$$

Its derivative to $x$ can be written as:

$$\frac{\partial u_2}{\partial x} = \frac{\partial u_2}{\partial h_2} \frac{\partial h_2}{\partial x} = -\frac{q_2}{h_2^2} \frac{\partial h_2}{\partial x} \quad (B.10)$$

Formula (B.7) can be rewritten as:

$$\frac{\partial h_1}{\partial x} = -\frac{\tau_0 - \tau_1}{g \rho_1 h_1} - \frac{\partial}{\partial x}(h_0 + h_2) \quad (B.11)$$

Filling in (B.9), (B.10) and (B.11) into (B.8) gives the following equation:

$$-\frac{q_2}{h_2} \frac{\partial h_2}{\partial x} + \rho_2 g \left( -\frac{\tau_0 - \tau_1}{g \rho_1 h_1} - \frac{\partial}{\partial x}(h_0 + h_2) \right) + g \frac{\partial}{\partial x}(h_0 + h_2) = -\frac{\tau_1 - \tau_2}{\rho_2 h_2} \quad (B.12)$$

Which can be simplified using the following steps:

$$-\frac{q_2^2}{h_2^3} \frac{\partial h_2}{\partial x} - \frac{\rho_1}{\rho_2} g \frac{\tau_0 - \tau_1}{\rho_1 h_1} - \frac{\rho_1}{\rho_2} g \frac{\partial}{\partial x}(h_0 + h_2) + g \frac{\partial}{\partial x}(h_0 + h_2) = -\frac{\tau_1 - \tau_2}{\rho_2 h_2} \quad (B.13)$$

$$-\frac{q_2^2}{h_2^3} \frac{\partial h_2}{\partial x} - \frac{\rho_1}{\rho_2} g \frac{\partial}{\partial x}(h_0 + h_2) + g \frac{\partial}{\partial x}(h_0 + h_2) = -\frac{\tau_1 - \tau_2}{\rho_2 h_2} + \frac{\rho_1}{\rho_2} g \frac{\tau_0 - \tau_1}{g \rho_1 h_1} \quad (B.14)$$

$$-\frac{q_2^2}{h_2^3} \frac{\partial h_2}{\partial x} + (1 - \frac{\rho_1}{\rho_2}) g \frac{\partial}{\partial x}(h_0 + h_2) = -\frac{\tau_1 - \tau_2}{\rho_2 h_2} + \frac{\rho_1}{\rho_2} g \frac{\tau_0 - \tau_1}{g \rho_1 h_1} \quad (B.15)$$

$$-\frac{q_2^2}{h_2^3} \frac{\partial h_2}{\partial x} + \frac{\partial}{\partial x}(h_0 + h_2) = -\frac{\tau_1 - \tau_2}{\rho_2 h_2} + \frac{\rho_1}{\rho_2} g \frac{\tau_0 - \tau_1}{g \rho_1 h_1} \quad (1 - \frac{\rho_1}{\rho_2}) g \quad (B.16)$$

$$\left(1 - \frac{\rho_1}{\rho_2} \right) g \frac{\partial h_2}{\partial x} + \frac{\partial}{\partial x}(h_0 + h_2) = -\frac{\tau_1 - \tau_2}{\rho_2 h_2} + \frac{\rho_1}{\rho_2} g \frac{\tau_0 - \tau_1}{g \rho_1 h_1} \quad (B.17)$$
Now in the model the rate of change of the height of layer 2 can be calculated with:

\[
\frac{\partial h_2}{\partial x} = \frac{\frac{r_1 - r_2}{\rho_1 h_1} + \frac{r_0 - r_1}{\rho_2 h_2}}{\left(1 - \frac{q_1}{\rho_2}\right) g h_2^3} - \frac{\partial h_b}{\partial x} - (B.18)
\]

Which can be simplified to:

\[
\frac{\partial h_2}{\partial x} = \frac{1}{\varepsilon g h_2^3 - 1} \left( \frac{r_1 + r_2 - r_2}{h_1 h_2} + \frac{\partial h_b}{\partial x} \right) (B.19)
\]

This is a differential equation for the thickness of the lower layer \( h_2 \). With \( \frac{\partial h_b}{\partial x} \) being negative for a slope that is downward in positive \( x \)-direction. Kranenburg (15) also gives this formula, but in the more general case for which \( q_1 \neq 0 \) (formula 4.7 in his lecture notes) and Kortmann (14) refers to it. Pietrzak (23) calls it the internal backwater curve.

Because the upper layer \( h_1 \) is relatively large compared to the lower layer \( h_2 \) and the friction between the two layers \( r_1 \) is expected to be lower than the friction \( r_2 \) between the lower layer and bed, it is reasonable to neglect the \( \frac{r_1}{h_1} \) term in (4.45). The simplified equation thus becomes:

\[
\frac{\partial h_2}{\partial x} = \frac{\frac{r_1 - r_2}{h_1 \varepsilon g h_2^3} + \frac{\partial h_b}{\partial x}}{\varepsilon g h_2^3 - 1} (B.20)
\]

This equation can be used to calculate the rate of change of the thickness of the density current. However, since this is a backwater curve, a downstream boundary condition is necessary. From this downstream location, the density current height can be calculated in upstream direction. As described in paragraph 4.6.2, this poses problems when applying this method to WID.
B-2 Method Cordi (1994)

Cordi (8) has done physical experiments to study the flow of density currents on an inclined bed. He compares his experimental results with theoretical formulations. For the body of the density current uses a momentum, mass and volume balance. He states that these equations fit his experimental results quite well. The momentum balance is derived from a force balance of the density current and is given by:

\[
S_{g}g'h\sin(\theta) - \frac{\rho_{s}}{\rho_{w}}C_{d}U^2 - \frac{\tau_{sw}}{\rho_{w}} = \frac{1}{2} \frac{d(S_{g}h^2g')}{dx} \cos(\theta) + \frac{1}{\rho_{w}} \frac{d(U'h\rho)}{dx}
\]  \(\text{(B.21)}\)

For the head of a density current two approaches are possible: one using the Chézy equation (Verhagen also uses this) and one using the method by Bittter and Linden (1980).

Chézy equation method

This method uses the Chézy equation:

\[
U_{f} = C_{c}\sqrt{g'fH_{f}}
\]  \(\text{(B.22)}\)

With

\[
g_{f} = \frac{\Delta\rho}{\rho}g
\]  \(\text{(B.23)}\)

Middleton and Turner propose a value of 0.75 and d’Altinakar and Denton propose 0.63 for \(C_{c}\). Experiments comply with this value as can be seen in the picture below.

![Figure B.2: Relationships with different values for \(C_{c}\). Source: Cordi (8)]](image)

Figure B.2: Relationships with different values for \(C_{c}\). Source: Cordi (8)
In the table below values are given for the velocity according to this formula using (B.22) with:

\[
\begin{align*}
\Delta \rho & = 70 \text{ [kg/m}^3]\text{]} \\
\rho & = 1070 \text{ [kg/m}^3]\text{]} \\
g & = 9.81 \text{ [m/s}^2]\text{]} \\
C_c & = 0.75 [-]
\end{align*}
\]

<table>
<thead>
<tr>
<th>Layer thickness (m)</th>
<th>0.5</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity (m/s)</td>
<td>0.43</td>
<td>0.62</td>
<td>0.88</td>
<td>1.08</td>
</tr>
</tbody>
</table>

**Table B.1: Relation layer thickness and velocity**

In his experiments, Cordi used slopes of 1:20 (3°) and 1:40 (1.5°). The velocities from this formula comply quite well with the experimental results.

A simple Matlab script with Cordi formula gives the following results. In this script for every space step the setting distance is calculated using a given settling velocity, reducing the height and thus velocity of the density current.

![Figure B.3: Height density current as function of distance](image)

**Bitter and Linden method**

The second method, developed by Bitter and Linden (1980), uses a non-dimensional head velocity and is applicable for currents with low density (below 1200 kg/m3):

\[
\frac{U_f}{\sqrt{B_0}} = S_2 \left( \frac{\cos(\theta)}{\alpha} + \frac{\alpha \sin(\theta)}{2(E_w + C_0)} \right) \left( \frac{\sin \theta}{E_w + C_0} \right)^{\frac{2}{3}}
\]

(B.24)

With

\[
\begin{align*}
U_f & = \text{velocity of front} \\
B_0 & = g \frac{\rho_o - \rho_w}{\rho_w} q_o \\
S_2 & = \text{coefficient}
\end{align*}
\]
\[ \alpha = \frac{U_{\text{max}}}{U} \]

\[ E_w = \text{entrainment coefficient of water} \]

\[ C_o = \text{drag coefficient} \]

**B-3 Method Verweij (1997)**

Verweij \( (43) \) uses balance equations given by Parker (1986), Turner (1973) and Chu (1979). He integrates the equations over the layer thickness. This gives equations equal to those of Van Rijn (paragraph B-5).

He gives equations for stationary \((d/dt=0)\) density currents as derived by Parker (1986). These equations give the change in layer density, sediment transport, velocity and turbulent energy.

In this stationary and uniform model, an equilibrium exists between gravity and friction forces at bottom and interface. Sedimentation, erosion and entrainment are described, but a complete model is left out of the report.


Verhagen \( (41) \) uses a very simple method to calculate velocity and production. This model does not include erosion, sedimentation and entrainment. He uses the Chézy formula (thus assuming a stationary and uniform situation), which is a simplification of the more extensive momentum equation:

\[ v_{fm} = \alpha \sqrt{\frac{\Delta \rho}{\rho}} gh_{fm} \] \( (B.25) \)

With

\[ v_{fm} = \text{velocity of the fluid mud layer} \quad [\text{m/s}] \]

\[ h_{fm} = \text{thickness of the fluid mud layer} \quad [\text{m}] \]

\[ \alpha = \text{coefficient with a value of about 0.9} \quad [-] \]

The water balance is given by:

\[ q = h_{fm}(v_{fm} + v_{dr}) - h_{cm}v_{dr} \] \( (B.26) \)

And the sediment balance:

\[ h_{cm}v_{dr}c_m = h_{fm}(v_{fm} + v_{dr})c_{fm} \] \( (B.27) \)

The used symbols are given in Figure B.4.
Combining equations (B.25) and (B.26) leads to:

$$\frac{\Delta P}{\rho} - g \alpha^2 (q + h_{cm} v_{cm}) = v_{fm}^2 + v_{vm}^2 v_{dr}$$

(B.28)

Now \( h_{fm} \) and \( v_{fm} \) can be calculated analytically and afterwards \( c_{fm} \) can be calculated with (B.27). The production of the dredge is given by:

$$P = v_{dr} h_{cm} w$$

(B.29)

### B-5 Method Van Rijn (2012)

Van Rijn studies density currents in a paper (2012) and in a book (2005). He presents an extensive model that includes erosion, sedimentation and entrainment.

When the density current is flowing, there are two possibilities:

1. The Froude number becomes smaller than 1, which indicates a subcritical flow. The density current will become thinner.

2. The Froude number stays larger than 1, so the flow stays supercritical. This takes place when the bed slope is sufficiently large. The density current will become thicker and material can be transported over large distances.

When the bed slope is relatively large, the supercritical flow will accelerates. This causes an avalanche type supercritical mud flow that erodes the bed. The layer thickness will increase and can travel over a long distance.

Van Rijn states that for mud the critical bed slope is about 0.2 to 0.4 degrees (1/300-1/150) for an initial concentration of 50 kg/m\(^3\), an initial layer thickness of 1 m and a settling velocity of 0.1 mm/s. This indicates that for harbours, where the slope is often much smaller, an hydraulic jump will occur close to the WID device and the flow will turn subcritical.

When the bed slope is small the gravity pull is small as well, the thickness of the mud flow will increase due to the bed friction forces. The flow becomes subcritical (Fr<1) after a hydraulic jump. When there is no deeper channel close to the density current (as is to be expected with WID), the thickness of the mud flow will decrease because the mud deposits and the flow will die out.
Strangely, Van Rijn presents a model for a situation with supercritical flow, while he suggests that during WID usually subcritical flow occurs.

The Froude number of the mud layer is defined as (39):

$$Fr_2 = \frac{u_2}{\sqrt{\frac{\Delta \rho_2}{\rho_2} gh_2}}$$  \hspace{1cm} (B.30)

With

- $u_2$ = mud flow velocity \[\text{[m/s]}\]
- $h_2$ = mud flow layer thickness \[\text{[m]}\]
- $c_2$ = mud concentration \[\text{[kg/m}^3\]\]
- $\Delta \rho_2 = \rho_2 - \rho_w = (\rho_s - \rho_w)c_2$
- $\rho_2 = \rho_s c_2 + (1 - c_2)\rho_w$

If $Fr_2 > 1$ the mud layer is supercritical and the thickness of the mud layer $h_2$ is smaller than the critical depth, given by:

$$h_{2,cr} = \left[ \frac{q^2}{\frac{\rho_2 - \rho_w}{\rho_2} c_s g \cos(\beta)} \right]^{\frac{1}{3}}$$  \hspace{1cm} (B.31)

With

- $q = u_2 h_2$ = flow rate in mud layer \[\text{[m}^2/\text{s]}\]
- $\beta$ = bed slope angle \[\text{[rad]}\]

The flow velocity of a supercritical mud flow can be expressed as:

$$u_2 = Fr_2 \sqrt{\frac{\Delta \rho_2}{\rho_2} gh_2}$$  \hspace{1cm} (B.32)

Van Rijn models the density current for supercritical flow with the following balances. Momentum balance of mixture in lower layer in s-direction:

$$\frac{\partial (u_2 h_2)}{\partial s} + (\rho_s - \rho_w)h_2 c_2 \left[ g \cos(\beta) \frac{\partial h_2}{\partial s} - g \sin(\beta) \right] + (\rho_s - \rho_w) \left( \frac{1}{2} gh_2^2 \cos(\beta) + u_2^2 h_2 \right) \frac{\partial c_2}{\partial s} + (\tau_i + \tau_s) = 0$$  \hspace{1cm} (B.33)

Mass balance for fluid in lower layer:

$$\frac{\partial (u_2 h_2 (1 - c_2))}{\partial s} - W_i - W_s = 0$$  \hspace{1cm} (B.34)

Mass balance for sediment in lower layer:
\[
\frac{\partial(u_i c_i h_i)}{\partial s} - S_i - S_b = 0 \tag{B.35}
\]

With

\[
\begin{align*}
    h_i &= \text{thickness upper layer} \\
    h_2 &= \text{thickness lower layer} \\
    c_i &= \text{depth-av. vol. sed. conc. upper layer} \\
    c_2 &= \text{depth-av. vol. sed. conc. lower layer} \\
    u_i &= \frac{q_i}{h_i} = \text{velocity upper layer} \\
    u_2 &= \frac{q_2}{h_2} = \text{velocity lower layer} \\
    W_i &= \text{exchange of fluid at interface} \\
    W_b &= \text{exchange of fluid at bed} \\
    S_i &= \text{exchange of sediment at interface} \\
    S_b &= \text{exchange of sediment at bed} \\
    \rho_2 &= \text{mixture density lower layer} \\
    \rho_w &= \text{fluid density} \\
    \rho_s &= \text{sediment density} \\
    \tau_i &= \text{shear stress at interface} \\
    \tau_b &= \frac{g}{C^2} = \text{bed shear stress} \\
    C &= \text{Chézy coefficient} \\
    \beta &= \text{bed slope angle in s-direction} \\
    s &= \text{coordinate along bed slope}
\end{align*}
\]

These 3 equations are a set of equations with 3 unknowns: \(u_2\), \(h_2\) and \(c_2\). The change of the thickness of the mud layer can be rewritten as:

\[
\frac{\partial h_2}{\partial s} = \frac{\sin(\beta)}{\cos(\beta)} \left[ 1 - \left( \frac{h_{2,cr}}{h_2} \right)^3 - \alpha_1 \left( \frac{W_i + W_b}{u_2} - \frac{\alpha_1 h_2}{h_2} \frac{\partial c_2}{\partial s} \right) \right] \frac{\partial c_2}{\partial s} 
\]

\[
\tag{B.36}
\]

With:

\[
\alpha_1 = \frac{2 \rho u_i^2}{(\rho_s - \rho_w)(1 - c_i)h_2 c_i g \sin(\beta)} \tag{B.37}
\]

\[
\alpha_2 = \frac{2 \rho_s u_i^2 + (\rho_s - \rho_w)(1 - c_i)u_i^2 + \frac{1}{2} (\rho_s - \rho_w)(1 - c_i)h_2 g \cos(\beta)}{(\rho_s - \rho_w)(1 - c_i) c_i h_2 g \sin(\beta)} \tag{B.38}
\]

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Shear stress turbulent flow:

\[ h_{1,cr} = \left( \frac{q^2}{(\rho_s - \rho_w) c_s g \cos(\beta)} \right)^{\frac{1}{3}} \]  

(B.39)

\[ h_{2,eq,turb} = \left( \frac{(C_d + C_a)q^2}{(\rho_s - \rho_w) c_s g \sin(\beta)} \right)^{\frac{1}{3}} \]  

(B.40)

\[ h_{2,eq,\text{Jam}} = \left( \frac{(\tau_i + \tau_s)h_i^2}{(\rho_s - \rho_w) c_s g \sin(\beta)} \right)^{\frac{1}{3}} \]  

(B.41)

Van Rijn is solving this equation in a spreadsheet. These equations are only valid for supercritical density currents. For a subcritical flow Van Rijn gives another (simpler) set of formulas. For subcritical flow the velocities are relatively small and the particles will settle quickly so that the density flow will gradually die out. Now the mass balances for fluid and sediment become:

\[ \frac{\partial q}{\partial S} = \frac{\partial q}{\partial s} + (W_i + W_s) \]  

(B.44)

\[ \frac{\partial q}{\partial S} = S_b + S_j \]  

(B.45)

With:

\[ u_s = \frac{q}{h_i} \]  

(B.46)

\[ c_s = \frac{q_i}{q} \]  

(B.47)

With a density difference between the sediment suspension and clear water given by:

\[ \Delta \rho_2 = \rho_2 - \rho_w = [\rho_s c_s + (1-c_s) \rho_w] - \rho_w = c_s (\rho_s - \rho_w) \]  

(B.48)

And a relative density difference:

\[ \frac{\Delta \rho_2}{\rho_2} = \frac{c_s (\rho_s - \rho_w)}{c_s (\rho_s - \rho_w) + \rho_w} = \frac{c_s (s-1)}{c_s (s-1) + 1} \]  

(B.49)
With
\[ \rho_s = \rho_s c_z + (1 - c_z) \rho_w \] = density of fluid-sediment mixture of lower mud layer
\[ c_z = \frac{\rho_s - \rho_w}{\rho_s - \rho_w} \] = volume concentration of lower layer
\[ \rho_w \] = fluid density
\[ \rho_s \] = sediment density
\[ s = \frac{\rho_s}{\rho_w} \] = relative density

During the propagation of the density current the particles will settle. Therefore the settling velocity has to be known in order to determine the distance over which the density current will travel.

For the laminar Stokes regime, the settling velocity can be described with:
\[ w_s = (1 - c_w) \phi \frac{\Delta \rho D^2_s \phi \rho}{18 \nu} \] \hspace{1cm} (B.50)

The setting in a subcritical flow is modelled by Van Rijn in the sheet SED-TUBE. He gives settling velocities for mud between 0.1 and 0.5 mm/s. Assuming flocculated mud with an effective settling velocity of 0.5 mm/s, the settling length is in the order of \( x/h_0 = 1000 \) (so the particles settle 1 m over a distance of 1000 m).

Figure B.5: Results model Van Rijn (39)
Appendix C: Hydraulic jump

Kranenburg (15) describes the theory of an internal hydraulic jump, which is (in this case) a transition form a subcritical to a supercritical flow. He gives an expression for the ratio of the thickness of the layer before and after the jump. The layer thickness after the jump is called the conjugated thickness of the layer thickness before the jump. Using a momentum balance the conjugated thickness can be calculated.

In the derivation a coordinate system and balance area are used that move along with the jump, so that within the balance area the shape of the jump and the quantity of mass and momentum do not change over time. Also, the following assumptions are made:

- Friction between the upper and lower layer and between the lower layer and the bottom can be neglected because of the short distance over which the jump takes place.
- External effects are not regarded.
- Only two dimensions are taken into account, changes in width are not accounted for.

In Figure 4.12 the used variables are indicated. The quantities in the top layer have index number 1 and the quantities in the bottom layer have index number 2. Quantities after the jump are indicated with an apostrophe (') mark. Because of the moving coordinate system, the velocities relative to the earth are lowered by the velocity $c$ of the coordinate system to make them relative to the hydraulic jump. This velocity $c$ is equal to the velocity of the dredger.

This leads to a system of equations from which a relationship between the layer thickness at both sides of the jump follows:

$$
\frac{q_1^2}{\varepsilon g h_1 \left( \frac{h_1 + h_2}{2} \right)} + \frac{q_2^2}{\varepsilon g h_2 \left( \frac{h_2 + h_1}{2} \right)} = 1 \quad (C.1)
$$

With

- $q_1c$ = specific discharge upper layer per unit width rel. to moving coordinate system [m$^2$/s]
- $q_2c$ = specific discharge lower layer per unit width rel. to moving coordinate system [m$^2$/s]
- $h_1$ = layer thickness upper layer before jump [m]
\( h'_1 \) = layer thickness upper layer after jump \([\text{m}]\)
\( h_2 \) = layer thickness lower layer before jump \([\text{m}]\)
\( h'_2 \) = layer thickness lower layer after jump \([\text{m}]\)
\( \varepsilon \) = relative density mixture \([-]\)

**C-1  Conjugated layer thickness**

The first term in (C.1) can be neglected since for low flow velocities in the upper layer it only leads to a difference of several per cent. This results in the following relation for the ratio between the thickness of the lower layer before and after the jump (16):

\[
\frac{h'_2}{h_2} = \frac{1}{2} \left( \sqrt{1 + 8Fr^2} - 1 \right)
\]  

(C.2)

The internal Froude number should be smaller than 1 before the jump (supercritical flow) and larger than 1 after the jump (subcritical flow). The internal Froude number can be calculated with:

\[
Fr^2 = \frac{q_1^2}{\varepsilon_1 gh'_1^3} + \frac{q_2^2}{\varepsilon_2 gh_2^3}
\]  

(C.3)

If there is no entrainment during the jump, the relative density differences before and after the jump are equal: \( \varepsilon_1 = \varepsilon_2 \). When assuming that the flow velocity in the top layer is small compared to the flow in the lower layer (\( q_1 \ll q_2 \)), the internal Froude number can be approximated with:

\[
Fr^2 = \frac{U^2}{\varepsilon g h_2}
\]  

(C.4)

In which \( U \) is the velocity difference between the two layers.

**C-2  Equilibrium thickness**

In case after the hydraulic jump the velocity in the upper layer is assumed to be zero, the rate of change of the lower layer thickness can be calculated with the following equation for an internal backwater curve. This equation has already been derived in Appendix B:

\[
\frac{\partial h_1}{\partial x} = \frac{1}{\varepsilon \rho g} \left( \frac{\tau_1}{h_1} + \frac{\tau_2}{h_2} \right) + \frac{\partial h_2}{\partial x} \left( \frac{q_2^2}{\varepsilon g h_2^3} - 1 \right)
\]  

(C.5)

In this formula \( \tau_1 \) and \( \tau_2 \) are the friction forces between respectively the water layer and the density current and between the density current and the bed. In the next paragraph these coefficients will be studied in more detail.

For a uniform flow the thickness does not change, so the teller in (C.5) is equal to zero, resulting in:
\[
\frac{1}{\varphi \rho g \left( \frac{r_1}{h_1} + \frac{r_1 - r_2}{h_2} \right)} + \frac{\partial h_b}{\partial x} = 0 \quad (C.6)
\]

The physical explication of this formula is that the gravity causes a force along the slope that is equal to the bed friction. Using this formula the equilibrium thickness of the lower layer after the hydraulic jump can be found for every discharge. At the location where this thickness is equal to the conjugated depth as calculated with (C.2), the hydraulic jump will take place.

### C-3 Friction coefficients

The coefficients \( r_1 \) and \( r_2 \) are the friction forces between respectively the water layer and the density current and between the density current and the bed.

The friction between the bed and bottom layer can be modelled in two ways: using a turbulent model or a laminar model. In previous reports the turbulent model has been used, mostly in combination with the \( k \)-factors given by Kranenburg. However, since the flow after the jump most probably has a laminar character, it is more correct to use a laminar model for the bed friction at this point. In the next paragraphs both the turbulent and laminar friction models are explained.

#### C-3.1 Laminar and turbulent flow

The Reynolds number is used to determine whether a flow is laminar or turbulent:

\[
Re = \frac{\rho_m u h}{\mu} \quad (C.7)
\]

With:

- \( \rho_m \) = density of the mixture \([\text{kg/m}^3]\)
- \( u \) = velocity of the mixture \([\text{m/s}]\)
- \( h \) = layer height \([\text{m}]\)
- \( \mu \) = dynamic viscosity \([\text{Pa.s}]\)

A flow in a pipe is called turbulent when the Reynolds number is larger than 4000, it is laminar for \( Re < 2100 \) and in between these values the flow state is called transitional. A difficulty is the value for the dynamic viscosity that has to be used. For water at 20\(^\circ\) the dynamic viscosity is about \( 10^{-3} \). However, the mud-water mixture is assumed to be less viscous than water. A good starting point are the values for the so-called lutocline layer as presented in paragraph 2.3.2. This lutocline layer, which is the upper layer and has lowest density, shows probably the highest similarity to the mud in the density current, since its density is below 1100 kg/m\(^3\). For this mud layer the measured dynamic viscosity is in the order of 0.1 Pa.s.
C-3.2  Turbulent flow
In case of turbulent flow, the friction model as presented by Kranenburg (15) can be applied. The friction between the two layers is given by shear stress $\tau_1$ (15):

$$\tau_1 = -k_1 \rho_1 |u_1 - u_2| (u_1 - u_2)$$  \hfill (C.8)

Since it can be assumed that $u_1 \ll u_2$ this can be simplified as:

$$\tau_1 = k_1 \rho_1 u_2^2$$  \hfill (C.9)

For the case of a flowing lower layer, a non-flowing upper layer and no wind, the factor $k$ is given by:

$$k_1 = 15 \cdot 10^{-4}$$  \hfill (C.10)

The shear stress $\tau_1$ accounts for the friction between the bottom and the lower layer (layer 2). Kranenburg assumes that this friction is also proportional to the velocity squared:

$$\tau_2 = -k_2 \rho_2 u_2^2$$  \hfill (C.11)

With the friction coefficient:

$$k_2 = \frac{g}{C^2} = 30 - 40 \cdot 10^{-4}$$  \hfill (C.12)

With $C$ the Chézy coefficient. Note that these figures have probably been found for a salt wedge. To improve the model results, a friction coefficient between in-situ mud and the liquid mud layer is required.

C-3.3  Laminar flow
To find the friction between the bottom and density current for laminar flow, an analogy with laminar flow through a pipeline can be used. Battjes and Labeur (3) derived the relationship between the friction and flow velocity for a laminar current in a pipeline. Here the same methodology will be used to derive such a relationship for laminar flow on a flat bed.

With a roto-visco test (also called stationary rheometer or Couette viscometer) the relationship between friction and a velocity gradient can be determined (35):

$$\tau = \eta \frac{dv}{dh}$$  \hfill (C.13)

With

- $\eta$ = dynamic viscosity \hspace{1cm} [Pa.s]
- $\frac{dv}{dh}$ = velocity gradient \hspace{1cm} [m/s²]
The friction decreases from $\tau_o$ at the surface area till 0 at a distance $h_o$ from the surface, see Figure C.2. This can be described with:

$$\tau = \frac{(h_o - h)\tau_o}{h_o} \quad (C.14)$$

![Figure C.2: Definitions coordinate system and friction](image)

Combining (C.13) and (C.14) gives:

$$\frac{dv}{dh} = \frac{\tau_o}{\eta h_o} (h_o - h) \quad (C.15)$$

Integration of (C.15) from $h=0$ till $h=h_o$ with the condition $u(0)=0$ leads to the total discharge:

$$Q = \int_{h=0}^{h_o} \frac{\tau_o}{\eta h_o} (h_o - h) dh = \left[ \frac{\tau_o}{\eta h_o} \left( h_o h - \frac{1}{2} h_o^2 \right) \right]_{h=0}^{h_o} = \frac{\tau_o h_o}{2\eta} \quad (C.16)$$

The average velocity can be calculated by dividing the discharge by the area:

$$\bar{u} = \frac{Q}{h_o b} = \frac{\tau_o}{2\eta b} \quad (C.17)$$

Rewriting (C.17) gives:

$$\tau_o = \frac{2\eta Q}{h_o} = 2\eta b \bar{u} \quad (C.18)$$

It can be observed that in this case the friction is linear proportional to the flow velocity. Since the viscosity $\eta$ will be different for different mud densities, using a roto-visco test a relationship between the friction and mud density can be derived. This is illustrated in Figure C.3, where the left graph shows a possible relation between the friction $\tau$ and velocity gradient $\frac{dv}{dh}$ for a certain mud density $\rho$ that can be found with a roto-visco test. Performing this test for different mud densities and keeping the velocity gradient $\frac{dv}{dh}$ constant, the friction $\tau$ as a function of mud density $\rho$ can be derived, as
depicted in the right graph in Figure C.3. It is expected that above a certain critical density $\rho_{\text{crit}}$ the friction will increase rapidly, so it is advantageous to keep the density below this value.

![Graph](image)

**Figure C.3:** Left: results of a roto-visco test for a certain mud density. Right: friction as a function of mud density for a certain velocity gradient

### C-3.4 Conclusion

It can be concluded that in a turbulent situation the friction is proportional to the square of the velocity $\tau \propto u^2$ whereas in a laminar situation it is linearly proportional to the velocity $\tau \propto u$.

It is suggested to use the turbulent flow model for the flow before the jump and the laminar flow model for the flow after the jump.

When neglecting the term in (C.6) with the thickness of the upper layer $a_1$ and substituting (C.9) and (C.11), the equilibrium thickness can be calculated with:

$$h_{\text{eq}} = \sqrt{\frac{-(k_1 + k_2)Q^2}{\varepsilon g l_b w_{cr}}}$$

(C.19)
Appendix D: Matlab script model

## Input

```matlab
% General
g=9.81; % gravitational acceleration [m/s^2]

% Dredge
v_dr=0.5; % velocity dredge [m/s], range 0.5-1
w_dr=4; % width of WID beam [m]

% Jets
D_n=0.08; % nozzle diameter [m]
n_n=15; % amount of nozzles [-]
dp=100000; % pressure drop [Pa]
c_d=0.96; % nozzle discharge coefficient [-]

% Water and soil
rho_situ=1300; % density in-situ [kg/m^3]
rho_w=1000; % density water [kg/m^3]
c_u=10000; % cohesion of mud [Pa]
mu=0.1; % dynamic viscosity [Pa.s], range 0.1-1 according to report Nasner

% Main parameters for outflow
v_set_mm=0.15; % settling velocity [mm/s], range 0.1-1
v_set=v_set_mm/1000; % settling velocity [m/s]
i_b=-0.001; % bed slope [-], range 1/1000-1/100
C=80; % Chézy coefficient [m^(1/2)/s], range 60 (rough)-110(smooth)
f_jet=0.5; % production efficiency factor [-]
f_entr=1.2; % factor to determine entrainment between jet and start outflow, as multiple of jet volume Q_0 [-]

Jetting and loosening of mud

```matlab
% Jet properties
heart_dist=w_dr/n_n; % heart-to-heart distance between jets [m]

u_0=sqrt(2*dp/rho_w); % jet flow velocity [m/s]
A_n=pi*(D_n/2)^2; % nozzle area [m^2]
Q_n=c_d*A_n*u_0; % single nozzle discharge [m^3/s]
Q_0=n_n*Q_n; % total discharge nozzles [m^3/s]
q_0=Q_0/w_dr; % specific discharge [m^2/s] Verhagen and Van Rijn: should be about 0.3 m^3/s/m

P_jet=Q_0*dp; % jet power [W]

% Soil
s_intr=sqrt(3*rho_w*u_0^2*D_n^2/c_u); % intrusion depth using model Schuurman [m]
Q_in=s_intr*v_dr*w_dr*f_jet; % in-situ soil that enters process [m^3/s]

% Situation directly behind jet (location 1)
Q_1=Q_in+Q_0; % discharge mixture [m^3/s]
```
\[ q_1 = \frac{Q_1}{w_{dr}}; \] % specific discharge mixture [m²/s]

\[ \rho_1 = \frac{Q_{in} \rho_{situ} + Q_0 \rho_w}{Q_{in} + Q_0}; \] % density mixture [kg/m³]

\[ \epsilon_1 = \frac{\rho_1 - \rho_w}{\rho_w}; \] % relative density [-]

\[ u_1 = \frac{(\rho_{situ} Q_{in} v_{dr} + \rho_w n_n C_d A_n (u_0 - v_{dr})^2)}{\rho_1 Q_1}; \] % velocity using conservation of momentum [m/s]

\[ h_1 = \frac{q_1}{u_1}; \] % flow height [m]

\[ Fr_1 = \sqrt{\frac{u_1^2}{h_1 g \epsilon_1}}; \] % Froude nr [-]

### Situation after entrainment (location 2)

\[ Q_2 = Q_{in} + f_{ent} Q_0; \] % discharge including entrainment [m³/s]

\[ q_2 = \frac{Q_2}{w_{dr}}; \] % specific discharge [m²/s]

\[ \rho_2 = \frac{Q_{in} \rho_{situ} + f_{ent} Q_0 \rho_w}{Q_{in} + f_{ent} Q_0}; \] % mixture density [kg/m³]

\[ \epsilon_2 = \frac{\rho_2 - \rho_w}{\rho_w}; \] % relative density [-]

### Outflow as density current with Chézy formula (from location 3 onwards)

\[ c_{star} = c \sqrt{g}; \] % dimensionless Chezy coefficient [-]

\[ q(1) = \frac{q_2}{u(1)}; \] % initial specific discharge [m²/s]

\[ u(1) = c_{star}^{2/3} \left( -\epsilon_2 g q(1) i_b \right)^{1/3}; \] % initial velocity [m/s]

\[ h(1) = \frac{q(1)}{u(1)}; \] % initial height [m]

\[ dx = 1; \] % step size [m]

\[ i = 1; \] % first step [-]

\[ \text{while } h > 0.1; \] % while-loop to calculate flow until its height is below 0.1 m

\[ u(i) = c_{star}^{2/3} \left( -\epsilon_2 g q(i) i_b \right)^{1/3}; \] % velocity using Chézy [m/s]

\[ h(i) = \frac{q(i)}{u(i)}; \] % flow height [m]

\[ Fr(i) = \sqrt{\frac{u(i)^2}{h(i) g \epsilon_2}}; \] % Froude nr [-]

\[ Re(i) = \rho_2 u(i) h(i) / \mu; \] % Reynolds nr [-]

\[ t_{step(i)} = dx / u(i); \] % time step to travel dx [s]

\[ h_{set(i)} = t_{step(i)} v_{set}; \] % settling distance [m]

\[ q(i+1) = u(i) (h(i) - h_{set(i)}); \] % new, decreased discharge [m²/s]

\[ i = i + 1; \] % go to next step

end

\[ [N, J] = \text{min}(|h|); \] % distance at which h=0.1 m [m]

\[ \text{travel distance dc} = J; \] % distance at which h=0.1 m [m]
Plots

```matlab
subplot(5,1,1)
plot(h)
title('Height density current')
xlabel('Distance [m]')
ylabel('Height h [m]')
axis([-inf inf 0 1])

subplot(5,1,2)
plot(u)
title('Velocity density current')
xlabel('Distance [m]')
ylabel('Velocity u [m/s]')
axis([-inf inf 0 1])

subplot(5,1,3)
plot(q)
title('Discharge density current')
xlabel('Distance [m]')
ylabel('Discharge q [m2/s]')
axis([-inf inf 0 0.5])

subplot(5,1,4)
plot(Fr)
title('Froude number density current')
xlabel('Distance [m]')
ylabel('Froude number [-]')
axis([-inf inf 0 1])

subplot(5,1,5)
plot(Re)
title('Reynolds number density current')
xlabel('Distance [m]')
ylabel('Reynolds number [-]')
axis([-inf inf 0 6000])
```