Interaction between loaded barges and bed material

An explorative study to investigate flow velocities and sediment transport caused by the passage of a barge train over a small sandy shoal.

R.J. Lenselink
Interaction between loaded barges and bed material

An explorative study to investigate possible erosion effects due to the passage of a barge train over a small sandy shoal.

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Delft, August 2011

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Final version

Cover photo: Push barge combination sailing on the river Waal (Source: Google Earth)
This report is the result of my master thesis, part of the master Civil Engineering at the University of Technology in Delft. After four years of study at the HTS in Vlissingen I decided that my knowledge of Hydraulic engineering was not enough and had to increase.

Albert Einstein once said: “The more I learn, the more I realize how little I know”

Although this phrase can be found in a lot of variations the message is the same; after years of study I realize how little I actually know about this subject and how much there is still to learn. Prof. dr. ir. M. Stive once told the students during one of his lectures of Coastal Engineering that Albert Einstein advised his son to pick another study than sediment transport using the argument that it was too complicated...

I have learned a lot and I am still not satisfied with my results, since I realize that there is much more to write about. Still, I tried to give an accurate and complete description of the subject and matter.

A first word of appreciation and gratefulness for their time and effort goes out to my graduate committee in which Prof. ir. T. Vellinga, dr. ir. H.J. de Koning Gans, dr. ir. A. Sieben and ir. H.J. Verheij are seated. I am grateful for the time they spend on giving comments and providing me with information which helped me to complete this study. A special thanks to Henk de Koning Gans for the possibility to use his program DelKelv and the time and patience he had with me to explain the program and theory. I also want to thank Henk Verheij, my direct supervisor, for his time and very accurate description of improvements and correction of my work. The most important lesson I learned from Henk Verheij is that it doesn’t matter that one doesn’t have all the information or knowledge, try to make the best of the information that actually is available and hopefully you will get a satisfying result. Or in Dutch: “Als het niet kan zoals het moet, dan moet het zoals het kan”.

I would like to thank my fiancée Audry Dekker for her loving care and (mental) support which dragged me trough the tougher periods during this master thesis. And last but certainly not least I would like to thank my parents for their support and their everlasting faith in me.

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Summary

Small shallow sections in the river Waal lead to a hindrance for the users and for high maintenance costs for the administrator, Rijkswaterstaat. Dredging these small shoals is inefficient and takes time in which the users can not sail with full draught. Sailing with a limited amount of cargo leads to reduced income and users of the river Waal try to keep sailing with maximum draught.

It is known that ships in restricted waterways cause sediment transport which sometimes leads to erosion. Part of this erosion is due to propeller induced flow velocities, the other part is caused by increased flow velocities called return flow. Return flow is depending on draught, dimensions of ship and waterway, sailing speed and flow velocities in the river itself. Goal of this master thesis is to investigate the influence of the parameters influencing return flow and investigate the caused erosion by changing those parameters in model studies and flume experiments.

From these experiments it becomes clear that flow velocities significantly increase with a decreasing under keel clearance. From measurements it follows that the flow velocity under the bow of the barge increases with maximum of 1.31 times the return flow velocity as calculated using Schijf (1949). From computational model runs and experiment done in a flume at Deltares it follows that only 55% of specific discharge is left under the bow. 45% is considered to be diverged to the sides of the barge; this process is called fanning out.

Besides the model tests, two cases are taken into account: the standard operating speed (2.56m/s) of a push barge combination sailing in upstream direction and the same push barge combination but now sailing with its maximum speed possible. This maximum sailing speed (3.07m/s) is calculated regarding the installed power of a push boat and the resistance a push barge combination will encounter when sailing with a certain speed and draught.

Ship squat is calculated using Ankidunov and Schijf, these values are compared with measured values of sinkage of the barge during the experiments. Ankidunov gives higher values for the amount of squat then Schijf. Differences for the two considered sailing speeds is 0.10m for both cases. It is shown that measured values are comparable with the calculated values. It is important to predict the amount of squat to prevent accidents. A ship sailing with a speed of 2.56m/s in upstream direction will have a squat of 0.42m (10% of the original draught of 4m). With a speed of 3.07m/s, the amount of squat increases to 0.60m.

Although bed levels were measured in an accurate way, the measuring error combined with a relative high grain size leaded to unreliable data about bed level changes. Focus of the research was therefore moved to the velocity profile and defining a simple model which is able to calculate occurring flow velocities under a ship’s bow. Results from this model are compared with measurements and existing formulae.

Data gained from measurements of DelKelv show a correlation with the maximum measured flow velocities under the bow of the barge. No correlation is found with the average measured flow velocities.
Since most of the existing formulae are focused on the maximum occurring flow velocities, this lack of correlation between average flow velocities and modeled flow velocities does not matter.

From the different flow velocities gained from calculations, measurements and model runs, it becomes clear that the modified Martin and Maynord equation (equation [1-1]) and the WL DelftHydraulics formulae (equation [2-31]) predict the maximum occurring flow velocities very well.

Finally the occurring flow velocities are used as input for a sediment formula (Engelund-Hansen) and for the two considered cases the effect on the amount of sediment transport is investigated. From this it becomes clear that although the flow velocities increase significantly due to the passage of a push barge combination, the time over which this increase takes place is not long enough to induce serious sediment transport. Using push barge combinations for the removal of small sandy shoals in the river Waal does not seem to be very effective.
1 Introduction

1.1 Project definition

Rijkswaterstaat, as an administrator of the river Waal, is concerned with keeping the river at a certain fixed depth. Their goal is to maintain the channel at such a depth, that a 'neat and safe' usage of the river is guaranteed. A goal oriented contract has been signed by a subcontractor which has to fulfil two functional demands, set-up by Rijkswaterstaat:

* Maintaining a minimum depth of the channel bed of OLR -2.8m (Overeengekomen Lage Rivierstand= minimum water level which occurs during a OLA (Overeengekomen Lage Afvoer), a discharge which is, during a longterm period of several years, not exceeded with more than an average of 20 ice-free days/year)*;  

* Maintaining the channel depth for as long as possible under H-4.0m, with H being the actual water level present.*

Since fluctuations of the river bed are unpredictable and the need for a 'neat and safe' usage of the river is one of the highest priorities, Rijkswaterstaat wishes to give up-to-date information about the bed levels to the users of the river. At this moment, except for during flood seasons, the river bed is measured on a (nearly) daily base by the subcontractor of the dredging works. From these measurements the MGD (Minst Gepeilde Diepte = minimum measured depth) is determined and published. Users of the river determine their loading capacity (and therefore keel clearance) using the MGD. The MGD provides a restriction on the draft for the users of the river, therefore restricting transport capacity.

Since the MGD can change after the measurements are done each day, there is a certain risk involved in determining the loading capacity according to known values of the MGD. The MGD value is published without any safety margins and measuring errors, which means that this value actually has to be valid until the next measurement is known. A ship, loaded according to a measured MGD value, passing the location of the MGD value 12 hours after measuring, will risk grounding since the depth at the location could have changed within those 12 hours.

Figure 1-1 shows small shallow sections in the river Waal, measured using a multi-beam. In the left picture, the shallow sections are indicated with red, the right picture shows the shallow sections as orange dots.

At this moment some users of the river determine their loading capacity based on the MGD-0.10m (10 cm deeper than the minimum measured depth) which leads to a frequent ‘contact’ with the river bed. Another known value used in the IJssel river for determining the loading capacity is MGD -0.3m. These values are possible because the sailors on these ships have such knowledge of the river and its limitations that, with a given MGD value, the ship is manoeuvred in such a way that the shallow sections are avoided. In this way

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1 Source: Preventiegids voor de binnenvaart (2005)
only experience and knowledge about the river and its restrictions (and opportunities!) can lead to a ‘safe’ passage of the shallow sections.

Dredging of the shallow sections of the river, to remove the small shoals, takes time and has its influence on the capacity of the river and on its users. As a result of the planned expanding of the river a higher frequency in dredging work is expected, which will lead to a higher frequency of hindrance for the users of the river. Therefore Rijkswaterstaat created the project “nieuwe bezems, efficienter baggeren” (meaning: finding more efficient ways of dredging) which consists of two parts. Part one is a study for more efficient ways of dredging done by H. Bots, Off-shore engineering graduate. The second part is about researching the possibilities and effectiveness of using inland vessels for clearance of small sandy shoals, subject of this master thesis.

1.2 Problem description

The principle of removing small sandy shoals using inland vessels is not a new technique. During the 90s a method called ‘agitation dredging’ (Dutch: agitatiebaggeren) was used to clear shallow parts of a channel. This method is based on the propeller wash of a ships propeller. Mainly mud was removed by hovering with a ship above the shallow section, the flow in the river transported the sediment downstream and the required depth (at the former shallow section) was therefore maintained in a relative easy way. Even at this day agitation dredging is still used in e.g. Suriname, to prevent the Suriname river from silting up. At the river Waal, the river used for this master thesis, most of the sediment consists of sand and gravel (near Germany). Sand particles have a lot more weight than mud particles and are therefore less easy transported. A technical study into the transport of sand particles due to the return flow of inland vessels is therefore needed to investigate the usage of this process.
At this moment, most of the shallow sections in the river Waal are removed using dredging. These dredging ships are not always capable of removing a shallow section within the same day as the measurements. Result is that users of the river Waal will have to adapt their draft to this MGD and are therefore not able to transport at their maximum capacity.

A sketch which gives a good indication of parameters influencing flow underneath a ship is given in Figure 1-2. Stolker & Verheij (2006) developed a model and used ideas of Rigter (1989) who did a study after erosion effects at the Drechtstunnel, an immersed tube. In this picture a first indication is given of the parameters influencing flow velocities under a ship, denoted as $U_r$ m/s.

Rigter assumes a contraction at the ship’s bow which ensures a maximum velocity underneath the bow $U_{rb}$ at cross section 1 and a lower velocity $u_r$ at cross section 2. Flow at cross section 2 is bounded on two sides, the ship’s hull and the bottom of the waterway. Due to boundary layer effects (Prandtl) the velocity is zero at both locations and a laminar layer is present. The velocity profile as shown can be calculated using theory about flow between plates. Couette or Poiseuille for laminar flow and ‘the law of the wall’ by von Kármán in a turbulent situation.

![Figure 1-2 Schematic view of parameters influencing flow velocity underneath the ship (Source: Rigter)](source)

The two parameters which are the most interesting for this case study are $U_{rb}$ and $u_r$. These two velocities are of importance for the initiation of motion for sediment particles. Once these particles are moving, $U_r$ and the velocity behind the ship are determining the distance over which the particles are transported.

Figure 1-3 shows the result of a towed barge combination and the velocities occurring related to the distance from the bow of the combination. Negative velocities are opposite to the direction of the towed combination. It is clearly visible that at a small distance from the bow, the flow velocity underneath the bow, $U_{rb}$, is the largest as already assumed by Rigter. This proves that the assumptions made by Rigter are true and therefore usable for further investigation.
At this moment the existing amount of theory about flow velocities underneath a ship in shallow water and the effect on the bottom is minimal. It is therefore hard to make reliable calculations about the amount of sediment transported and the distance over which the sediment is transported due to ship induced motions. This master thesis will deal with this problem and will give a clear as possible insight in the theory known about the subject mentioned above. Using flume experiments and existing theory, an effort will be made to give the reader insight in the processes involving flow, sediment transport and effectiveness of using inland vessels.

Earlier research to calculating flow velocities underneath ships is done by Stolker and Verheij (2006). WL Delft Hydraulics (1987) and Martin & Maynord (1994) already came up with a method for predicting flow velocities, but these methods turned out to be not accurate enough when compared to data sets gained from measurements. WL Delft Hydraulics came up with the equation $U_{rb} = 1.5 - 2.0 \cdot U_r$. Stolker and Verheij came up with two equations: a physical approach based on the research done by Rigter and a modified Maynord approach, which gave the most accurate results when compared to the measurements available.

This modified Maynord equation reads:

$$\frac{U_{rb}}{V_s} = 1.07 \cdot \left( \frac{B_s}{h} \right)^{0.08} \cdot \left( \frac{T}{h} \right)^{1.82}$$  \hspace{1cm} [1-1]

In which:
- $U_{rb} =$ velocity underneath the ship's bow \hspace{1cm} [m/s]
- $V_s =$ relative speed of the ship \hspace{1cm} [m/s]
- $B_s =$ maximum width of the ship's hull \hspace{1cm} [m]
- $T =$ draft of the ship \hspace{1cm} [m]
- $h =$ water level \hspace{1cm} [m]

The flow velocity underneath the ship can be used in sediment transport formulas related to the situation (every sediment transport formula has its own restrictions for usage). A combination of the flow velocities underneath a ship and sediment transport formulas (suspended and bottom transport) is the key to predicting the amount of sediment transported due to a ship sailing over a shallow section.

The first set-up is made to be able to predict the effect of flow underneath a ship and the therefore induced sediment transport. Quantities related to this process are still unknown or inaccurate; this makes it hard to make any statement about the outcome.
On the other hand it is known that inducing sediment transport and therefore clearance of small shoals using a push barge combination is possible. Agitation dredging is (still) successfully used and proven to have its effect on removing small shallow parts in rivers. Whether the usage of inland vessels is effective and under which conditions it might be effective is at this moment unpredictable and will be investigated during this master thesis.

1.3 Research questions

Questions will have to be answered in order to solve the problem as described. The most important questions are mentioned below and will be translated to objectives and activities in the following chapters. First of all the main question, followed by questions for each topic.

- **Main question:**
  
  *Is the usage of inland vessels, for example push barge combinations, effective for removal of small sandy shoals and can this be done in a safe way?*

- **Ship**
  
  *Which ships on the river Waal are usable for removing small sandy shoals?*
  
  *Which speeds are common for the users of the river Waal?*
  
  *Which restrictions are present when using ships for this purpose, considering mobility and safety?*
  
  *What is the flow velocity underneath the ship and which parameters involve this velocity and the direction of the velocity?*
  
  *What is the best speed for the design ship regarding sediment transport taking into account safety and manoeuvrability?*

- **River**
  
  *Which river discharges are common on the river Waal and which flow velocities are related to these discharges?*
  
  *What sediment gradation is present in the river Waal?*
  
  *Which regulations or laws have to be taken into account when inducing sediment transport on the river Waal?*
  
  *Which regulations have to be taken into account regarding safety and usage of the river?*

- **Sediment transport**
  
  *Which sediment transport formulae are usable for the situation in the river Waal?*
  
  *Without the interference of a passing ship, what is the amount of transported sediment due to flow velocities of the river Waal?*
  
  *Considering a known flow velocity due to the passage of a ship, how much sediment is transported, taking into account the speed of the ship itself too?*
  
  *Considering sediment transport, where is the suspended sediment deposited again?*

- **Model testing**
  
  *Which parameters are important to measure?*
  
  *Which restrictions are present when using a model in a flume to gain data about sediment transport and flow velocities?*
  
  *What is the reliability of the data and can it be compared to other test results?*
1.4 Objective

The Main objective of this master thesis is formulated as follows:

*Show in a qualitative and quantitative way the effect of using inland vessels for clearance of small sandy shoals and determine whether this method is effective when used in a safe and responsible way.*

Sub-Objectives, following from the objective are:

- *Development of an analytical model for flow under a ship;*
- *Development of a model for sediment transport induced by ship’s currents*
- *Carrying out physical model tests*
- *Carrying out computational model tests*
- *Calibration of the analytical model with test results*
- *Validation of the calibrated models with other data*
- *Prediction of erosion for pilot tests inclusive test conditions*
1.5 Approach & Project boundaries

The master thesis can be divided in some phases, which order is not always as logical. Due to the availability of persons and facilities, this order is however unavoidable. Main goal of this thesis is to answer the research questions mentioned in chapter 1.3 as good as possible.

Phase I - Physical model tests at Deltares

Due to availability of flume no 4 and already ongoing experiments, it was decided that in the end of the summer in 2010 some physical model tests would be carried out. H.Bots, former graduate at the Off-shore department of the TU Delft, was researching different dredging methods using including a deep draught barge. This thesis started with joining the ongoing experiments to get familiar with the equipment. Decided is to measure a velocity profile under the deep draught barge and to do some runs with different speeds and draughts to get more insight in the processes and parameters determining the flow velocity under the barge.

Phase II - Literature study

During this phase, relevant information will be studied. Main topics treated in this study are:

- **Design conditions**: definition of a ship which will be used for further research, dimensions of the river Waal, parameters involving flow and discharge, parameters involving sediment and morphologic situation in the river Waal.
- **Fluid mechanics**: open channel flow, flow past objects and boundary layer development;
- **Hydrodynamics**: movement of a ship, return flow, flow under a ship, and ship squat;
- **Sediment transport**: translation of flow velocities to sediment transport, calculating amount of sediment transport, influence of ship's screw and the distance after which sediment is settled.

Phase III – Modeling

This phase is divided into three parts:

- **Computational model tests using DelKelv, a potential flow model to gather more data about flow past and under a ship.**
- **Physical model, further elaborate the data gained from the model tests in the flume at Deltares.**
- **Making and developing a theoretical model of flow under a ship and the influence of the flow on the sediment transport.**

Phase IV - Analysis and conclusions

Using all the gathered information to compare results of different methods for predicting flow and sediment transport due to the passage of a ship and processing this into a coherent text form.

Phase V - Report and presentation

Combining all the gathered information and merging it into the final report. Finally a presentation will be given to explain the thesis in public.
The report does not follow the phases as described above. The next chapter gives information about the river Waal and its (for this thesis) relevant characteristics. Besides the river Waal itself, the relevant theory from fluid mechanics, sediment transport and ship movements will be used. This chapter can be considered as a summary of the literature study. Calculations are made for the river Waal in an average condition and the capabilities of push barge combinations are shown.

Chapter 3 treats the physical model tests carried out in the flume at Deltasres. An overview of the equipment and materials is given. Also the configurations of the tests are elaborated. In section 3.5 the results of the tests are treated and analysed.

Besides the physical modeling an attempt is made to approach the results from the measurements with a simple analytical model, based on continuity equations. This is treated in chapter 4.1, results gained from this model are compared with the measurements. Section 4.2 covers the computational model DelKelv; a description of the model, model run configurations and results are treated here. First the flume experiments are simulated, to be able to compare the measurements with the data gained from DelKelv. Secondly a simulation of a prototype situation is done with a 2x2 push barge combination to gain information about occurring flow velocities in a full scale situation.

Chapter 5 uses the information gained from literature study, experiments and simulations to describe and analyze two cases. In the first case a common sailing condition of a 2x2 push barge combination is analyzed. Secondly, a maximum feasible condition is analyzed in which the 2x2 push barge combination is assumed to be sailing at its maximum speed. For these cases the return flow velocities are calculated and an indication of the occurring sediment transport is given. From these results conclusions are drawn in chapter 6.

In chapter 7 recommendations for further research are given.
2 Literature overview

2.1 River Waal

The river Rhine originates in Switzerland and divides just after passing the Dutch border into the 'Pannerdensch Kanaal' and the river Waal. About $2/3$ of the discharge of the Rhine flows through the river Waal which makes this the river with the highest water (and sediment) discharge in the Netherlands. With a length of nearly 82 km the river Waal ends near the village of Woudrichem, where it merges together with the river Maas into the Merwede. Figure 2-1 shows the catchment area of the river Waal, on the right side the bifurcation at the 'Pannerdensche Kop' and on the left side the merge with the Maas into the Merwede.

![Figure 2-1 Catchment area of the river Waal](image)

Besides the main function of transporting water to the sea, the river Waal is the main transport route from the Rotterdam harbor to Germany. Nearly 30% of the cargo transported over water passes the river Waal. Therefore the river Waal is also called the 'highway' considering the amount of users and the ship speeds compared with other rivers in the Netherlands. The combination of the river discharges and the intense traffic makes the river Waal a difficult river to navigate.

2.1.1 Hydrological information

Except for the important transport function of the river Waal, it was already mentioned that the river Waal is the largest river in the Netherlands considering river discharges. A general rule of thumb, mentioned in every book about the largest of the Dutch rivers, states that the river Waal receives about $2/3$ of the discharge of the river Rhine and the Pannerdensch Kanaal receives about $1/3$. The Pannerdensch Kanaal divides again in the IJssel and the NederRhine/Lek which both are accounted for $1/9$th and $2/9$th of the Rhine’s discharge.

Using Waterbase, a database provided by Rijkswaterstaat, discharge values were obtained from measuring locations at Lobith (river Rhine) and Tiel (river Waal). Using this data to calculate average values for the river discharges resulted in the data as shown in Table 2-1. From this dataset it follows that the river Waal receives about 72% of the discharge of the river Rhine. The graphs of these data sets are shown in Figure 2-3 River discharge at Tiel and Figure 2-2 with the dots displaying the yearly average values.
A commonly used average discharge at the river Rhine is about 2200 m³/s. In extreme cases the river should be able to handle a discharge of 16,000 m³/s.

At Tiel the flow carrying width of the river is about 260 m, for an average discharge depths vary between 4 to 5 m. Table 2-2 shows the water levels and discharges measured at Nijmegen.
Nijmegen is located at about km 875, an average bed level of NAP +3.1m is measured according to Figure 2-4. As earlier in this report already mentioned, Rijkswaterstaat tries to maintain a minimum depth of 4 m. During dry seasons, a minimum depth of 2.8 m is tried to maintain, which is not always possible.

An average width of 350-400m is nearly everywhere present in the river Waal, depths fluctuate with the discharges but are most of the time between 5-6 m. Average flow velocities therefore vary between 0.65 and 1.10m/s again depending the discharge and water levels present. During further research regarding this master thesis, a discharge of 1579m$^3$/s and an average flow velocity of 1.10 m/s will be used. This value is in range of the ones derived from the available data and is also commonly used by Rijkswaterstaat. For an average situation a depth of 5.1m will be used, this is also in range of the values derived from the available data and also commonly used by Rijkswaterstaat.

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2 Source: Rijkswaterstaat

3 Source: Rijkswaterstaat
Using above data some important parameters can be derived, first of all the hydraulic radius defined as:

\[
R = \frac{A}{O}
\]  

[2-1]

In which:

\[
O : \quad \text{wetted perimeter} \ (= B + 2 \cdot d) \quad [\text{m}]
\]

\[
A : \quad \text{wetted surface} \quad [\text{m}^2]
\]

Using an average (flow carrying) width of 260m and an average depth of 5.1m results in a hydraulic radius of 4.9m.

Also of importance is the Reynolds number, further treated in 2.2.1, indicating the type of flow and the type of stresses dominating the flow pattern. This Reynolds number is larger than \(4.2 \times 10^6\) which means that the flow is considered as turbulent.

### 2.1.2 Sediment

Sediment in the river Waal is decreasing in size with an increasing distance from the start of the river. Starting at the origin of the river Rhine in Switzerland a kilometer (km) indication follows the river to the sea. Figure 2-5 shows the development of the sediment characteristics along the river Waal. Increasing km values are downstream directed. From this figure some characteristic values can be determined.

![Figure 2-5 Sediment diameters along the river Waal](image)

Above figure shows clearly the variation in diameters, the \(D_{90}\) decreases from 18mm until nearly 1mm. The \(D_{50}\) converges to a value around 1 mm quiet soon, for the \(D_{90}\) a convergence to a value of 4-5mm is found. For further calculations a value of 1 mm for \(D_{50}\) and a value of 5mm for \(D_{90}\) will be used.
2.1.3 Transport, usage and maintainance

Being the ‘highway’ of the rivers in the Netherlands and having a discharge as large as 1,579 m$^3$/s results in a lot of regulations. The ones important for usage of inland vessels for removing shallow sections are mentioned below.

Bulk transport between Rotterdam/Antwerpen and Germany, mostly taken care of by ThyssenKrupp-Veerhaven and the Imperial-Reederij-Gruppe is done by pushed barge combinations. Most common are pushed barge combinations with 4 barges and 1 push boat. Those combinations transport about 11,000 ton of cargo during a single trip between Rotterdam/Antwerpen and Germany. The trip between the Europoort (Rotterdam, Netherlands) and Duisburg (Germany) is about 240 km long and depending on the direction the ships are sailing, takes between 26 and 12 hours\(^4\).

Upstream, the barges are loaded and the ship is sailing against the flow, downstream the barges are empty and the ship is sailing with the flow. Table 2-3 gives an overview of the average sailing speeds for both trips (upstream and downstream). This is the speed relative to the shore.

<table>
<thead>
<tr>
<th>Sailing direction</th>
<th>Distance [km]</th>
<th>Sailing time [h]</th>
<th>Average speed [km/h]</th>
<th>Average speed [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream</td>
<td>240</td>
<td>26</td>
<td>9.23</td>
<td>2.56</td>
</tr>
<tr>
<td>Downstream</td>
<td>240</td>
<td>12</td>
<td>20</td>
<td>5.56</td>
</tr>
</tbody>
</table>

Table 2-3 Travel times and average velocities

Figure 2-6 shows the general flow pattern in a river bend. Due to centrifugal forces the flowing water mass is pushed to the outside of the river bend, leading to the highest flow velocities. Accept for navigational consequences, these higher flow velocities result in erosion and thus deeper sections. During dry periods, users of the river will try to carry as much cargo as possible, taking into account, the deeper sections in the outer bends. This principle is very common and widely accepted by the users and administrator of the Waal. Figure 1-1 shows clearly the shallow sections in the inner side of the river bend as a result of the process described above.

Figure 2-6 Flow in a river bend

\(^4\) Source: Thyssen-Krupp-Veerhaven
The first and most important restriction for the usage of inland vessels for removal of small sandy shoals is safety. Most of the ships using the river Waal are loaded at their maximum capacity and are therefore not very maneuverable. Since the ship’s velocity seems to play an important role in the flow underneath a ship, squat is too. Considering the load and therefore the under keel clearance of the ships, squat is an important factor to account for.

When sailing with small under keel clearances, squat effects are of great importance. Not to forget that the presence of another ship in a narrow river will also affect squat. According to Barrass (2006) the amount of squat can double as a result of the presence of another ship, this is an important statement considering the traffic intensity on the river Waal.

The actual channel ‘width’ varies between 150m and 170m; this is not the physical width, but a maintained width. Shallow shoals, located outside this virtual channel, will not be removed using inland vessels.

During extreme discharges of 16,000 m$^3$/s water levels are high, in this situation dredging works are not necessary since the shallow sections are not shallow anymore.

A third important regulation is the dredging policy in the river Waal. Dredged material from the river Waal has to be deposited within 1,500 m of the location from where it was taken. This is done to maintain a sediment balance in the river.

At this moment two methods are used to maintain the river Waal, dredging using a trailing suction hopper dredger (TSHD) and using a plow. A contract is signed with a dredging contractor to maintain a certain depth at all time. They use the plow to remove some small shallow sections and the TSHD for the removal of the larger areas such as the inner side of a river bend.

For further details about the dredging works on the river Waal see the report of H.Bots (2011), Efficient maintenance of the Dutch fairways, a technical study into two topics; the efficiency of using TSHD in the river Waal and a study into innovative methods for dredging.
2.2 Fluid mechanics

2.2.1 Open channel flow

In open channel flow, like the river Waal, flow is gravity driven. The bed level at the upstream side of the river Waal is higher than downstream, thus creating a pressure gradient. The average slope of the bottom in the river Waal is $10^{-4}$ (Figure 2-4 Bed levels in the river Waal).

When dealing with large areas of interest for the usage of e.g. discharge calculations river flow is often considered as uniform. Uniform flow is valid when at every point in the fluid the flow velocity has the same magnitude. In this case, both flow velocity and water depth are not constant due to differences in bottom location (erosion, sedimentation) and flow profile variations. Uniformity in flow patterns is depending on spatial variations.

Steadiness of flow is on the other hand depending on variations in time. When the discharge through a certain section of the river is changing in time, flow is to be considered as un-steady flow. Since we are looking at small time scales considering the flow under and next to a moving ship, discharge will not vary significantly and we can assume steady flow. When looking at the morphology of the river one has to consider the flow as un-steady. Differences between summer discharges and winter discharges make a difference in the flow pattern of the river and are therefore not to be neglected.

More hydraulic properties can be described using Reynolds, Froude and Chezy. The Reynolds number, indicating the ratio between the flow velocity times the hydraulic radius and the kinematic viscosity of the fluid, gives information about the behavior of the flow. The higher the Reynolds number the less important the viscosity is when looking at a flow-profile. For Reynolds numbers lower than 2,300, flow is considered as laminar, flow with Reynolds values higher than 3,500 is considered as turbulent. The Reynolds number is described as:

$$ Re = \frac{U \cdot R}{\nu} $$

$U$: average flow velocity in longitudinal direction \([\text{m/s}]\)

$R$: hydraulic radius, defined as the wet cross section divided by the wet contour \([\text{m}]\)

$\nu$: kinematic viscosity, defined as the dynamic viscosity divided by the density of the fluid \([\text{m}^2/\text{s}]\)

In situations for which $B>>h$ the hydraulics radius can be replaced with the water depth, $h$. Using an average width of 260m, average depth of 5.1m, average flow velocity of 1.10 m/s and a viscosity of $1.33\times10^{-6}$ m$^2$/s, gives a Reynolds number of $>4.2\times10^6$. This indicates that the flow is turbulent and turbulent shear stresses are dominant. Another important fact about the turbulence in a flow is that shear stress is proportional to the square of the velocity gradient. (In laminar flow it is proportional to the velocity gradient)
An equation which can be used to calculate the averaged shear stress in a random cross section is equation [2-3]. In this equation the shear velocity is introduced; a parameter which isn’t an actual velocity but has the units of a velocity (m/s). Later on this shear velocity will be treated more in detail.

\[ \tau_0 = \rho \cdot u_*^2 \]  

[2-3]

In which:
- \( \tau_0 \): bottom shear stress \([\text{N/m}^2]\)
- \( \rho \): specific mass of water \([\text{kg/m}^3]\)
- \( u_* \): shear velocity \([\text{m/s}]\)

With \( u_* \) defined as:

\[ u_* = \sqrt{g \cdot R \cdot i_b} \]  

[2-4]

- \( g \): gravitational acceleration \([\text{m/s}^2]\)
- \( R \): hydraulic radius \([\text{m}]\)
- \( i_b \): bottom slope [-]

Using the data derived gives a value for the shear velocity of 0.07 m/s equal to a shear stress of 4.8 N/m².

The value of \( i_b \) can be used for equilibrium situations in which the depth is equal to the equilibrium depth, in this situation the bottom slope is equal to the slope of the energy line (caused by friction) \( i_w \).

Chezy developed a formula from which the uniform flow velocity could be derived. Since the uniform flow velocity is already known in this case, the Chezy coefficient can be calculated, which is of use for further calculations regarding resistance. The Chezy constant is defined as:

\[ C = \frac{U}{\sqrt{R \cdot i_b}} \]  

[2-5]

In which:
- \( U \): depth averaged flow velocity \([\text{m/s}]\)

With an uniform flow velocity of 1.1 m/s, a hydraulic radius of 4.9 m and a slope of 10⁻⁴ this results in a Chezy value of 49 m¹/²/s for the river Waal. The Chezy value will prove to be of importance when looking at the sediment transport and bed roughness later on in this report. Note that a larger Chezy value indicates a smoother regime.

A velocity profile in open channel flow is difficult to quantify and very different from the velocity profile which is used for simple calculations. Instead of a steady, uniform flow where the situation is described as inviscid flow, flow in an open channel consists of different velocities, mainly caused by turbulence. Also the viscosity of the water plays an important role when looking at flow velocities and sediment transport. The viscosity of the water causes a shear force due to differences in flow velocities near the bottom and above, this will be treated later on in this chapter.

### 2.2.2 Boundary layer development

Although a lot of information about pipe flow is known, most theory about flow in open channels considering energy losses is based on comparable cases in pipe flow situations.

Three main topics considering flow past (partially) immersed bodies always return:

- Development of a (laminar) boundary layer, first described by Prandtl;
Energy losses due to friction;
Vena contracta or the contraction due to a pressure drop;

These three topics are subject of many studies nowadays, but still no analytic solutions to the different problems are found. Most of the theoretical problems are solved nowadays using empiric relations, found by doing a lot of experiments.

A summary of these widely used relations are given below. Theory related to the topics will not be treated broadly but enough to get a good understanding of the subject.

Development of the boundary layer is often used in fluid mechanics and aeronautical engineering. Due to the viscosity of a fluid and a certain roughness or shape of an object, fluid passing the object will face shear forces related to those properties and a boundary layer will grow. One of the most important boundary conditions considering flow past objects is of course that the flow velocity at the object itself is zero, given an impermeable structure. This creates velocity differences which again create shear forces due to different velocities. Prandtl found out that in these situations a laminar boundary layer grows, dividing the flow and the object in the water with a laminar layer, which increases in thickness along the flow direction. This layer occurs in laminar and turbulent flows and is described by White (1998) as:

\[
\delta = \frac{5.0 \cdot x}{Re^{1/2}_x} \quad \text{[2-6]}
\]

Laminar flow

\[
\delta = \frac{0.16 \cdot x}{Re^{1/7}_x} \quad \text{[2-7]}
\]

Turbulent flow

With:

Local Reynolds number \( Re_x = \frac{U \cdot x}{\nu} \) \text{[2-8]}

\( x \): horizontal distance along the boundary [m]

Boundary layer thickness, indicated with \( \delta \), is sometimes also indicated with \( \delta_{99\%} \). This is due to the definition of the boundary layer thickness: “the locus of points where the velocity \( u \) parallel to the plate reaches 99 percent of the external velocity \( U \)” (White, 1998).

These relations are used for describing flow between plates, the most basic form in which the boundary layer theory is used. Gourlay (2006) used these equations to calculate the growth of a boundary layer underneath a ship sailing with a small under keel clearance. When applying this boundary layer theory on the four barge combination as defined in chapter 2.3.1, an increase of the boundary layer, starting from the ship’s bow can be calculated:

\[
\delta = \frac{0.16}{(U / \nu)^{1/7}} \cdot x^{6/7} \quad \text{[2-9]}
\]

\( U \): 4.00 [m/s]
\( \nu \): 1.33*10^{-6} [m²/s]

This flow velocity is the flow velocity in the river added with the ships speed since ‘the plate’ is moving with this speed along the water. Note that in this equation no roughness is taken into account (besides viscosity). This equation results in Figure 2-7 when plotting the boundary layer development with respect
to the distance $x$. On the horizontal axis one finds the distance from the start of the distortion of the flow, for instance a ship's bow ($x=0\text{m}$). Following the ship's bottom in the direction of the stern shows the nearly linear ($x^{6/7}$) growth of the boundary layer.

![Development of boundary layer](image)

**Figure 2-7 Boundary layer development related to the horizontal distance, for three different flow velocities**

When, in this situation, the boundary layer is as large as the under keel clearance, one speaks of a fully developed situation. In this situation the flow is to be considered as viscous flow. In this situation turbulent shear forces are dominant.

Turbulent shear forces, although complex to quantify, can be described using Reynolds Time averaging concept: Reynolds divided the turbulent flow velocity into an average flow velocity and a fluctuating flow velocity. In general the theory implies a separation of the flow velocity into a time averaged velocity and a fluctuation relative to the average velocity, this is denoted as:

$$ u = \bar{u} + u' $$

[2-10]

In which:

- $\bar{u}$: average velocity over a certain time [m/s]
- $u'$: velocity relative to the average velocity [m/s]

![Visualization of Reynolds time-averaging concept](image)

**Figure 2-8 Visualization of Reynolds time-averaging concept**
This way of defining velocities will be used when taking a closer look at flow past a boundary layer (wall-flow). To get an impression of the velocity profile close to a boundary, theory by von Kármán (1930 & 1933) will be treated. First it is important to make a distinction in the types of flow occurring near a boundary.

Figure 2-9 shows three sections in which the velocity profile can be divided:

- A viscous wall layer where viscosity dominates;
- An overlap layer, where both viscosity and turbulence determines the flow pattern (not treated here);
- An outer turbulent layer where turbulence dominates and the flow can be described as inviscid.

From these three sections, in this case the most interesting are the viscous wall layer and the outer turbulent layer. These will be treated separately.

![Figure 2-9 Shear stress and velocity profile near a wall (Source: White, 2006)](image)

Describing viscous wall flow is the most accurate when using the 'law of the wall'. This theory is originally founded by Theodore von Kármán in 1930 and used for describing the flow close to a wall (or impermeable boundary). Later on it became clear that the law of the wall also gives a good description of the flow pattern in natural flows (open channel flow).

Coleman (1970) defined the velocity (which is used by van Rijn since 1981) as a function of the water level, $z$, using the law of the wall as:

$$u(z) = \frac{u_s}{\kappa} \ln \frac{z}{z_0}$$  \hspace{1cm} [2-11]$$

With:

$$u_s = \left( \frac{\tau_w}{\rho} \right)^{1/2}$$  \hspace{1cm} [2-12]$$

In which:

- $u(z)$ flow velocity [m/s]
- $u_s$ shear velocity [m/s]
- $\kappa$ Kármán constant ($= 0.41$) [-]
- $z$ water level [m]
- $\tau_w$ wall shear stress [N/m²]
- $\rho$ fluid density [kg/m³]
2.3 Hydrodynamics

When looking at the behavior of moving ships in water, it is important to know the essential characteristics of a ship. Is the shape of the ship influencing the resistance or draft? Which parameters determine the different degrees of freedom, like heave and trim, and therefore influencing processes like squat? To get a clear overview the most important theory of ship movements and processes involved with ship movements are treated in this chapter.

2.3.1 Important parameters

An important dimensionless parameter is the Froude depth number, used for many different situations. The Froude depth number is defined as the ratio between velocity and the square root of the gravitational acceleration times the water depth. Or in symbols:

\[ Fr = \frac{U}{\sqrt{gh}} \]

In which:
- \( U \): average flow velocity in longitudinal direction \([\text{m/s}]\)
- \( g \): gravitational acceleration \([\text{m/s}^2]\)
- \( h \): depth \([\text{m}]\)

This dimensionless parameter represents the ration between the inertia of the water and the static weight of the water is interesting. Considering a situation with super critical flow, viscous forces are relatively small compared to the inertia forces which can be described with the Froude number. This gives important information about the type of flow occurring around objects.

When the Froude number approaches unity (\( Fr = 1 \)) the resistance of the water on the ship reaches a level which the ship in most of the cases cannot overcome, this determines its limit speed, further treated in 2.3.3. Only high speed ships or ships with enough power are able to overcome this limit speed, although this might cost a lot of fuel for ‘normal’ ships. Since the depth is also taken into account in the Froude depth number, one can determine the minimal required depth for a ship with a certain speed also by using the Froude number.

For this master thesis especially the pushed barge combinations are interesting, since they have a shape which is favorable for maintaining higher flow velocities under the keel. Normal inland navigational ships have a more V-shaped hull, this contrary to the shape of the barges which have a more U-shaped hull. Since the more water is transported underneath the ship, the more sediment transport will take place due to the increased speed, the shape of the underwater body of a ship is important. Modern slender hull ships have a shape which guides the water along the ship, considering a minimum of resistance. A pushed barge combination has a nearly flat bottom which keeps the return flow more underneath the barge’s bottom, therefore reducing the fanning out effect as shown in Figure 2-10. This fanning out effect is comparable with wave diffraction, and is depending on the shape of the ship’s hull but also on the potential differences between the flow next to the ship and the flow underneath the ship.
Important parameters and coefficients can be derived from the ship’s hull. Table 2-4 shows the dimensions of the design ‘ship’ units. One combination consists of 4 barges and 1 push boat, a schematic impression is shown in Figure 2-11.

<table>
<thead>
<tr>
<th></th>
<th>Length over all: LOA [m]</th>
<th>Draft: T [m]</th>
<th>Beam: B [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Push boat: Veerhaven VIII ‘Nijlpaard’</td>
<td>40.0</td>
<td>1.94</td>
<td>14.98</td>
</tr>
<tr>
<td>Barge: Class Europa II-a</td>
<td>76.5</td>
<td>4.0</td>
<td>11.4</td>
</tr>
<tr>
<td>Total (maximum values)</td>
<td>193</td>
<td>4.0</td>
<td>22.8</td>
</tr>
</tbody>
</table>

Table 2-4 Dimensions of ship, barges and 2x2 combination

The dimensions of the ship, barges and combination of both ship and four barges are defined in Table 2-4.

Properties like the dimensions of the ship play an important role in determining the flow around a ship. The water displacement (besides the dimensions of a restricted waterway) proves to be one of the most dominant properties considering flow velocities around a ship.

The dimensions of a rectangular box, drown around the ship, compared with the water displaced by the ship is called the block coefficient, defined in equation [2-14]

\[ C_b = \frac{V}{L_{\text{mid}} \cdot B \cdot T} \]  

[2-14]

According to Tabaczek et. al. (2007) a push-barge combination often has a block coefficient \( C_b > 0.85 \). A report from VBD (2002) commissioned by Thyssen-Krupp Veerhaven, states that the block coefficient for Europa IIa barges are varying from 0.9-0.93 for a maximum draught of 3.99m.
2.3.2 Degrees of freedom of a ship

A ship moving through water is experiencing a lot of different loads. Loads generated by waves, wind or flow, but also loads generated due to the movement of the ship itself. Finally, boundaries as the bottom or banks in a channel or a river, influence the motion of a ship. As a result of the loads the ship will move or rotate in a certain direction. The ability to move in a certain direction is due to a so called degree of freedom, from which there are six. These degrees of freedom are shown in Figure 2-12, a difference is made between rotations and translations. An example of a translation and rotation is ship squat, treated in 2.3.4, this process is defined as the sinkage and trim of a ship due to its speed.

![Figure 2-12 Six degrees of freedom in ship movement](image)

Not every degree of freedom will be treated in detail here, only the ones influencing the ship in such a way that the flow velocity under the ship is changed significantly. In this case sinkage (heave) and trim (pitch) are the most important since we are mainly interested in the draught and under keel clearance (UKC) of the ship.

2.3.3 Forces on a ship while moving

Water displacement becomes more and more important when a ship starts to move. A ship with a certain speed displaces the water at its bow which flows to the stern of the ship due to a pressure gradient as a result of water level differences. Since the water and the ship’s hull are not fully smooth, the moving water generates resistance. This resistance is best visible at the bow and stern of the ship, where two waves are noticeable (see also Figure 2-13). The ship is thus actually pushing water ahead. Due to the water flow opposite to the sailing direction, increased flow velocities occur and the water level decreases. Two topics are important (for this master thesis) when considering the water movement caused by a sailing ship: Return flow (described in chapter 2.3.5) and ship squat (described in chapter 2.3.4). These chapters describe the changes in flow around the ship and the changes in the location of the ship in relation to the bottom.
A push barge combination sailing along the river Waal has to overcome friction due to viscosity and external loads such as wind and flow in the opposite direction of the sailing direction. First of all one can state that the sailing speed of a ship is most of the time limited. This limitation is defined by the maximum velocity of a disturbance in shallow water equal to the square root of the gravitational acceleration times the water depth. From this one can state that in practice the following must hold:

\[ Fr = \frac{U}{\sqrt{gh}} < 1 \]  \[2-15\]

In the situation with a water depth of 5.1m a normal ship is not able to sail faster than 7.1m/s, normal is defined by self-propelled ships with a conventional hull shape. Sailing ships and slender body ships are under certain conditions able to sail faster, this is called sailing in plane; a state in which the water displacement is minimum and therefore the friction on the ship is low. Most of the ships sailing in the river Waal will not be able to overcome this limiting speed of 7.1m/s since the installed engine power will not be sufficient.

Schijf (1949) provides a relation for the situation in which not only the water depth is restricted but also the width of a channel or river is taken into account. This relation is defined using conservation of energy and is defined as:

\[ 1 - \frac{A_s}{A_c} + \frac{1}{2} \left( \frac{V_{\text{lim}}}{\sqrt{gh}} \right)^2 - 3 \left( \frac{V_{\text{lim}}}{\sqrt{gh}} \right) = 0 \]  \[2-16\]

In which:

- \( V_{\text{lim}} \): maximum possible sailing speed [m/s]
- \( A_s \): amidships cross section [m²]
- \( A_c \): flow carrying cross section of river [m²]

Depending on the dimensions of the ship and channel a limiting speed can be calculated which cannot be exceeded for a normal ship.

Van de Kaa (1978) divides the resistance of a sailing ship into three different resistances: frictional resistance, pressure resistance and resistance due to hydraulic phenomena. Using these components the maximum sailing speed can be calculated when the installed engine power is known.
The frictional resistance is defined as:

\[
R_f = \frac{1}{2} C_f \cdot \rho_w \cdot (V_s + U)^2 \cdot S
\]  \[2-17\]

In which:
- \( R_f \): frictional resistance  \([\text{N}]\)
- \( C_f \): friction coefficient  \([-]\)
- \( \rho_w \): specific mass of water  \([\text{kg/m}^3]\)
- \( V_s \): sailing speed  \([\text{m/s}]\)
- \( U \): average flow velocity  \([\text{m/s}]\)
- \( S \): wetted surface of ship  \([\text{m}^2]\)

In which the friction coefficient is defined by:

\[
C_f = 0.075 \cdot \left(\log\left(\frac{V_s + U}{v} \cdot L_s\right) - 2\right)^2
\]

In which:
- \( U_r \): return flow velocity as calculated using Schijf  \([\text{m/s}]\)
- \( v \): dynamic viscosity  \([\text{m}^2/\text{s}]\)
- \( L_s \): length of ship  \([\text{m}]\)

When no information is available, a safe value for \( C_f \) is 0.002\([-\cdot]\).

Pressure resistance is defined as:

\[
R_p = C_p \cdot \frac{1}{2} \cdot \rho_w \cdot V_s^2 \cdot A_s
\]  \[2-18\]

In which:
- \( R_p \): pressure resistance  \([\text{N}]\)
- \( C_p \): friction coefficient \(( \approx 0.2 \) \([-\cdot]\)

Resistance of a ship is this quadratic related to the sailing speed of a ship, doubling the sailing speed will result in a friction 4 times as large. Resistance due to hydraulic phenomena such as waves and displaced water is taken into account as a function of the water level drop down ‘\( z \)’, as defined by Schijf.

\[
R_H = \rho_w \cdot g \cdot A_s \cdot z
\]  \[2-19\]

In which:
- \( R_H \): Resistance due to hydraulic phenomena  \([-\cdot]\)
- \( z \): water level drop  \([-\cdot]\)

The total resistance is defined as the sum of all resistances:

\[
R_T = R_f + R_p + R_H
\]  \[2-20\]

\( R_T \): Total resistance  \([\text{N}]\)

To overcome this resistance a certain amount of power is needed, this is defined as:

\[
P_S = \frac{R_T \cdot (V_s + U)}{\eta_f}
\]  \[2-21\]

In which:
- \( P_S \): Power to overcome the resistance  \([\text{W}]\)
- \( \eta_f \): overall propulsive efficiency \(( \approx 0.7 \) \([-\cdot]\)
The installed engine power will most of the time determine the maximum sailing speed. Often the maximum installed power will not be used since this is not economic, the engine will use significantly more fuel which is very expensive. Now the needed power for a normal situation (draught of 4m, sailing speed of 2.56m/s, flow velocity of 1.1m/s) can be calculated. Note that the ship is sailing upstream, so the flow velocity is added to the sailing velocity.

The wetted surface of the 4 barge combination is equal to 4712m², the amidships cross section is equal to 91.2m². The resistances are now calculated as follows (numerical example in Appendix E):

<table>
<thead>
<tr>
<th>Resistance</th>
<th>Power</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction</td>
<td>63.1</td>
</tr>
<tr>
<td>Pressure</td>
<td>59.8</td>
</tr>
<tr>
<td>Hydraulic</td>
<td>223.7</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>346.6</strong></td>
</tr>
</tbody>
</table>

Table 2-5 Resistance and power for \( V_s = 2.56 \text{m/s}, T=4 \text{m} \) and \( U=1.1 \text{m/s} \)

Push boats on the river Waal commonly have three propellers each with an installed power of 1800pk which is equal to 1320kW (Source: ThyssenKrupp Veerhaven, Veerhaven VIII ‘Nijlpaard’). Total installed power is thus 3960kW, which means that under normal conditions 46% of the total power is used.

The maximum service speed is delivered at 90% of the total power. This means that in practice only 3564kW will and can be used to gain speed. Calculating the maximum speed possible for this installed power is done iteratively since the water level drop ‘\( z \)’ is also related to the sailing speed. So at first a sailing speed has to be estimated, from which the water level drop has to be calculated. Both values have to be used for calculation of the power and following from the maximum power, the value of \( V_s \) has to be decreased or increased. These calculations are done using Excel to save time, from the iterations it follows that the maximum sailing speed with the installed power is equal to 3.07m/s. For this speed a water level drop of 0.476m is calculated using equations [2-29] and [2-30]. The power needed for this sailing speed is then equal to 3537kW. These values are summarized in Table 2-6.

\[
V_s = 3.07 \quad [\text{m/s}]
\]
\[
z = 0.476 \quad [\text{m}]
\]
\[
P_f = 3537 \quad [\text{kW}]
\]

Table 2-6 Maximum velocity for 90% engine power (Veerhaven VIII)

With the existing push boats on the river Waal sailing upstream with a draught of 4m this is the maximum sailing speed that is possible. This is with a 2x2 configuration and an ambient flow velocity of 1.1m/s (opposite direction of the sailing direction).

### 2.3.4 Ship squat

Ship squat is one of the most important physical processes considering nautical safety when sailing in shallow water. Over the years, a lot of methods are designed for calculating ship squat. A lot of them are empiric, some of them more physical related. One of the founders of the modern ship squat calculations is Tuck (1966), who came up with one of the first broadly accepted physical descriptions of squat for sub-
and super critical flow situations ($0.5 < F_h < 1.5$). Also a difference was made in open water situations and bounded situations like channel and river situations. Most of the current methods for calculating ship squat are based on Tuck’s findings and are used for calculating the heave and trim of a ship due to its own speed and environmental conditions.

The water level drop can be roughly calculated using Bernoulli, who stated that the total amount of energy present at a certain location in an open channel could be described as:

$$ p + \rho gh + \frac{1}{2} \rho U^2 = C $$  \hspace{1cm} [2-22]  

Or:

$$ \frac{p}{\rho g} + h + \frac{U^2}{2g} = H $$  \hspace{1cm} [2-23]  

In which:

- $\rho$ density of water [kg/m$^3$]
- $C$ constant [-]
- $h$ depth [m]
- $H$ energy level [m]
- $p$ hydrostatic pressure [N/m$^2$]

![Figure 2-14 Decreasing water level (upstream situation)](image)

When energy losses as a result of friction are neglected, the total energy head between two locations should be constant. This results in the following balance equation:

$$ \frac{p_1}{\rho g} + h_1 + \frac{U_1^2}{2g} = \frac{p_2}{\rho g} + h_2 + \frac{U_2^2}{2g} $$  \hspace{1cm} [2-24]  

A water level drop between ($h_2$-$h_1$) as show in Figure 2-14 can now qualitatively be described. Considering continuity, the discharge between point 1 and 2 will stay the same. An increase of the flow velocity due to the passing ship will lead to an increase in the velocity head. Since the total energy head remains constant, a decrease in the water level (and with that the hydrostatic pressure) will occur.

The lowered water level results in a lowering of the ship with respect to the bottom. Archimedes’s law states that the upwards force on a ship is equal to the weight of the displaced water volume. Since the amount of water displaced by the ship will stay the same, the upwards force will too. The sinkage of the ship due to its own weight will thus stay the same, but the lowered water level will result in a decreased under keel clearance.

Ankidunov prediction for squat is one of the most thorough methods for calculating squat. No other method gives the possibility to use parameters for ship and waterway conditions in such a detailed way.
IJsebaert (2011) compared the most common methods for calculating ship squat and concluded that Ankidunov's method was the most accurate and thorough. He compared his calculated results with track data of trailer suction hopper dredgers (TSHD). This method will therefore be used in this master thesis too. The ship's total squat is defined as the midship's sinkage added with the trim of the ship. According to Briggs (2009) this method is valid for depth Froude Numbers less or equal then 0.6. In this case the depth Froude number is 0.46, the method of Ankidunov thus is valid for this case.

Figure 2-15 shows the amount of squat as calculated for the push-barge combination as defined in Table 2-4. Water depth $d=4m$, $C_b=0.95$, simulating a four barge combination sailing with an increasing speed.

For an average situation on the river Waal, Figure 2-16 and Figure 2-17 show the squat for an increasing velocity or increasing draft. Water depth is equal to 5.1m, flow velocity is equal to 1.1m/s, draft is 4m in Figure 2-16 and ship speed is equal to 2.56m/s in Figure 2-17.
Figure 2-16 Squat as a function of sailing speed for the prototype situation, constant draught of 4m (d=5.1m)

Figure 2-17 Squat as a function of draught (T) for the prototype situation, constant sailing speed of 2.56m/s (d=5.1m)
From Figure 2-16 one can derive that with a draught of 4m the push barge combination theoretically can have a maximum sailing speed of 4m/s. With that speed the barge will sink so much that the draught is equal to the water depth. In that case the push barge combination will slow down due to the friction which again will result in a decrease in squat and thus draught. From section 2.3.3 it is already known that a maximum sailing speed of 3.07m/s is possible with the configuration as described above (T=4m, U=1.1m/s). A possibility would be to sail with decreased draught to be able to gain higher sailing speed and probably higher flow velocities under the ship, but in practice the maximum draught will most of the time be used to be able to transport as much as possible. Transporting less will lead to a decrease in income and is therefore not further considered in this case.

Figure 2-17 shows the change in squat with a variable draught, considering a constant sailing speed of 2.56m/s. With a draught of 4m the squat is equal to 0.42m, a larger draught is not possible due to the dimensions of the barges. Comparing results from Figure 2-16 and Figure 2-17 shows that the influence of the (initial) draught is smaller than the influence of the sailing speed. A change in sailing speed will result in a larger change of squat than an equal change in draught. With initial draught the draught with a sailing speed of 0m/s is meant. Table 2-7 shows the amount of squat for the two considered sailing speeds from section 2.3.3.

<table>
<thead>
<tr>
<th>V_s (m/s)</th>
<th>S_{max} (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.56</td>
<td>0.42</td>
</tr>
<tr>
<td>3.07</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Table 2-7 Squat calculated using Ankidunov for two sailing speeds

2.3.5 Return flow

Theory used for calculating the return flow velocity is given by Schijf (1949) and is based upon the method of preservation of energy. Flow is simplified to a one dimensional current, which makes it possible to get an exact analytical solution. Sinkage of the ship is assumed to be equal to the water level depression, occurring due to the increased flow velocities. No trim is taken into account, the ship’s squat is assumed to be equal to the water level depression (which again is assumed constant over the width of the channel). No energy losses due to friction or whatsoever are taken into account. The ship with a prismatic amidships cross-section over the total length of the ship is assumed to be sailing in a straight, infinitely long prismatic canal section with a constant speed. Occurring return flow is assumed to be uniform over the width of the channel.

Important is the coordinate system Schijf assumes the observer to be on the ship, resulting in axes moving along with the ship. So when a ship is sailing with a speed equal to V_s, Schijf assumes the approaching flow velocity to be equal to V_s and no ship speed at all. In a waterway with a natural flow velocity like the river Waal, the average flow velocity is stated at 1.1m/s (see section 2.1.1). Note that the flow velocity is added to the sailing speed of the ship, since the ship is assumed to be sailing upstream. For conditions in which the ship is sailing down stream, the flow velocity should be substracted from the saling speed.

\[ V_s' = V_s + U \]  \hspace{1cm} [2-25]
With: \( V_s \): sailing speed related to the water \([\text{m/s}]\)

\( V_s \): sailing speed related to the banks of the channel or river \([\text{m/s}]\)

\( U_r \): flow velocity (opposite to sailing direction) \([\text{m/s}]\)

For the cross section in the undisturbed situation in front of the ship and the cross section amidships of the ship, Bernoulli (see section 2.3.4) gives:

\[
\frac{V_s^2}{2g} + h_0 = \frac{(V_s + U_r)^2}{2g} + h_0 - z \tag{2-26}
\]

\[
z = \frac{(V_s + U_r)^2}{2g} - \frac{V_s^2}{2g} = \frac{2 \cdot V_s \cdot U_r + U_r^2}{2g} \tag{2-27}
\]

\( V_s \)  
Ship’s sailing speed \([\text{m/s}]\)

\( U_r \)  
return current velocity along ship at midships section \([\text{m/s}]\)

\( h_0 \)  
water depth in undisturbed channel \([\text{m}]\)

\( g \)  
gravitational acceleration \([\text{m}^2/\text{s}]\)

\( z \)  
water level drop at midships section \([\text{m}]\)

Applying the continuity equation for the same two cross sections gives:

\[
Q_A = Q_B = V_s \cdot A_s = (V_s + U_r) \cdot (A_c - A_s - W_c \cdot z) \tag{2-28}
\]

In which: \( A_c \)  
Wet cross section of undisturbed channel \([\text{m}^2]\)

\( A_s \)  
Ship’s underwater amidships cross section \([\text{m}^2]\)

\( B_s \)  
Ship’s beam at midships section \([\text{m}]\)

\( T \)  
Draft of the ship \([\text{m}]\)

\( W_c \)  
Width at water level of undisturbed channel \([\text{m}]\)

When equations [2-27] and [2-28] are combined; they contain two unknowns, e.g. \( z \) and \( U_r \). Eliminating one of the equations results in a pair of equations with only one unknown. This way the water level drop down can be calculated and return flow velocity can be calculated. No further elaboration on these equations is given, more information is available in Verheij et. al. (2008).

When the sailing speed of a ship and the dimensions of the ship and channel are known, one can calculate the return flow velocity using equation [2-29]. Using the known values and rewriting the equation as a function of \( U_r \) results in a function with one unknown. Using Matlab or a graphical calculator it is now easy to find the zero crossing and thus the value of \( U_r \). Note that for the situation with an ambient flow velocity opposite to the sailing direction, the flow velocity should be added to the calculated return flow velocity.

\[
\frac{(V_s + U_r)^2 - V_s^2}{2 \cdot g \cdot h} - \frac{U_r}{V_s + U_r} + \frac{A_r}{A_s} = 0 \tag{2-29}
\]

With the known value of \( U_r \), the water level drop \( z \) can now be calculated using [2-30].

\[
z = \alpha \cdot \frac{(V_s + U_r)}{2 \cdot g} - \frac{V_s^2}{2 \cdot g} \tag{2-30}
\]
As a numerical example a push barge combination sailing in the river Waal with the two known velocities is given in Table 2-8.

<table>
<thead>
<tr>
<th>$V_s$ [m/s]</th>
<th>$U_r (+U)$ [m/s]</th>
<th>$z$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.56 m/s</td>
<td>1.75 (=0.65+1.10)</td>
<td>0.30</td>
</tr>
<tr>
<td>3.07 m/s</td>
<td>2.10 (=1.00+1.10)</td>
<td>0.48</td>
</tr>
</tbody>
</table>

Table 2-8 Return flow velocity and water level drop according to Schijf

An increase in sailing speed of 0.51 m/s results in an increase of return flow velocity of 0.35 m/s and an increase in water level drop of 0.18 m. In the following sections the flow velocity $U_{rb}$ (return flow under bow) is sometimes expressed in a coefficient times $U_r$ for these calculations times $U_r$ is used.

The water level drop ‘$z$’ is often used as indication for squat. Compared with the values gained from the calculations with Ankidunov (0.42 m and 0.60 m), the differences are about 0.10 m for both sailing speeds. Ankidunov will be used for further calculations since this method takes into account more influencing factors and when sailing with small under keel clearances the squat should not be underestimated.

### 2.3.6 Return flow under a ship’s bow

At this moment three ways to predict the occurring flow velocity under a ship’s bow are known. These methods are earlier described in a paper of Stolker and Verheij (2006). In 1988 WL Delft Hydraulics came up with a rule of thumb, which is derived from numerous experiments. These experiments are part of a research called ‘aantasting van dwarsprofielen in vaarwegen’ or translated to English: ‘erosion of cross sections in waterways’ which started in 1974 and lasted until 1984. From these measurements was found that the flow velocity underneath a ship’s bow was in the order of 1.5-2 times the return flow calculated using Schijf.

In symbols this is noted as:

$$U_{rb} = (1.5 - 2) \cdot (U_r + U_0)$$

[2-31]

With:
- $U_{rb}$: flow velocity under the bow of the ship [m/s]
- $U_r$: return flow velocity calculated using Schijf [m/s]
- $U_0$: flow velocity opposite to sailing direction [m/s]

Martin and Maynord derived from a number of towing tests the following equation:

$$U_{rb} = 0.16 \cdot (V_s + U_0) \cdot \left(\frac{B_s}{h}\right)^{0.54} \cdot \left(\frac{T}{h}\right)^{0.68}$$

[2-32]

$B_s$: width of the ship at midship’s section [m]
$h$: water depth [m]
$T$: draught [m]

This formula is limited in usage, the waterdepth/draught ratio should be larger than 1.6 and the blockage ratio $A_{ch}/A_s$ also used by Schijf, should be larger than 6. In the case of the Veerhaven VIII sailing in the river Waal, the $h/T$ ration is equal to 1.3, which is smaller than required. The ratio $A_{ch}/A_s$ in this case equal
to 14.5, which fulfills the requirements. For the case of the river Waal, the reliability of the results following from this equation is thus questionable.

Stolker & Verheij (2006) modified the Martin and Maynord equation after validating and calibrating it on three different data sets. This resulted in a modified Martin and Maynord equation proved to give more accurate results for different situations and is shown in equation [2-33].

\[
U_{rb} = 1.07 \cdot (V_s + U_0) \cdot \left( \frac{B_s}{h} \right)^{0.08} \cdot \left( \frac{T}{h} \right)^{1.82}
\]  

[2-33]

Differences between the modified and the original Martin and Maynord equation are visible in the powers which are used for the ratio width/depth and draught/depth. In the original equation the width of the ship divided by the water level seems to be of bigger importance, in the modified equation this ratio is less important for the flow velocity. The ratio draught/water depth on the other hand seems to be more important than Martin and Maynord originally measured.

For the sailing speeds of 2.56m/s and 3.07m/s the return flow velocities are calculated with the known equations and summarized in Table 2-9. These values will later on be compared with measurements and result from a computational model (chapter 0).

<table>
<thead>
<tr>
<th></th>
<th>2.56m/s</th>
<th>3.07m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>WL Delft Hydraulics</td>
<td>2.63-3.50</td>
<td>3.15-4.20</td>
</tr>
<tr>
<td>Martin Maynord</td>
<td>1.11</td>
<td>1.27</td>
</tr>
<tr>
<td>Modified Martin Maynord</td>
<td>2.84</td>
<td>3.23</td>
</tr>
</tbody>
</table>

Table 2-9 calculated return flow velocities under a ship's bow

Both WL Hydraulics and the modified Martin and Maynord equation give values that are somewhat equal to each other. The original Martin and Maynord equation gives very low values compared with the other two. This may be due to the restrictions as already mentioned earlier, the draught/depth ratio considered here lays outside the given boundary of T/h>1.6.
2.4 Sediment transport

2.4.1 Initiation of movement

The initiation of movement of an individual sediment particle depends on several factors which can be divided in deterministic factors and random factors.

Deterministic factors are the forces on a particle generated by gravity and the fluid flowing over the particle with a certain velocity and with certain characteristics such as viscosity and density. Figure 2-18 gives an impression of the forces acting on a grain. Gravity logically provides a downward force \( W \) related to the mass of the particle which is again depending on the specific mass \( \rho_s \) and grain size \( D \). The fluid on the other hand provides two forces on the particle which are the lift force \( F_L \) and secondly the flowing water produces a drag force on the particle due to friction between the water and the particle mainly depending on the viscosity of the water and the grain size.

Random factors are turbulence, indicated by the Reynolds number and the placement of the grain. Turbulence can be indicated and the amount of turbulence can be roughly calculated but the direction of the turbulent motions and therefore forces acting on the particle are almost impossible to predict. Secondly the placement of the particle is of importance. A particle located underneath a layer of other particles is less likely to start to move than a particle on top of all other surrounding particles.

Flow in a river provides a shear stress on the river bed which, when strong enough, initiates a particle to move. This shear stress will be enlarged as a result of the return flow of a passing ship. This increased shear stress can cause erosion which is preferable when looking at the removal of small sandy shoals. Shields (1936) came up with a parameter, called the critical Shields parameter which indicates whether a sediment particle moves or not. This critical Shields parameter is depending on the so called particle Reynolds number, \( \text{Re}_* \), which is defined as:

\[
\text{Re}_* = \frac{u_{*,cr} \cdot d}{\nu}
\]  

[2-34]

In which:

- \( u_{*,cr} \) = critical shear velocity [m/s]
- \( d \) = particle diameter [m]
- \( \nu \) = dynamic viscosity [m$^2$/s]

And the critical Shields parameter:
In which: \( u_{cr}^2 \) = The square of critical shear velocity \([\text{m}^2/\text{s}^2]\)

\[ \Delta = \frac{\rho_s - \rho_w}{\rho_w} = \text{Relative density} \quad [\text{kg/m}^3] \]

\[ g = \text{Gravitational acceleration} \quad [\text{m/s}^2] \]

\[ d = \text{Particle diameter} \quad [\text{m}] \]

This shear velocity is related to the critical bottom shear stress \( \tau_{b,cr} \) through:

\[ \tau_{b,cr} = \rho \cdot u_{cr} \cdot \left| u_r \right| \quad [2-36] \]

Finally, plotting the values of the critical Shields parameter with the values of the particle Reynolds number in a graph, results in Figure 2-19. The line drawn in the graph indicates the mobility of the particle. All particles with values of the critical Shields parameter above this line are assumed to move, all particles with values of the critical Shields parameter below this line will most likely not move.

![Figure 2-19 Shields diagram](image)

Since sediment transport has to occur to make the usage of push barge combinations for removal of small sandy shoals effective, we are interested in the values of \( u \) occurring underneath the ship. Knowledge of the shear stresses at the bed underneath the ship, will result in the insight in the amount of sediment transported due to the passage of a ship.

For the river Waal the Chezy coefficient is proven to be equal to 47\( \text{m}^{1/2}/\text{s} \), the bottom shear stress is calculated with:

\[ u_r = \frac{u \cdot \sqrt{g}}{C} = \sqrt{\frac{\tau_b}{\rho}} \quad [2-37] \]

- \( \bar{u} \): depth average velocity \([\text{m/s}]\)
- \( \tau_b \): bottom shear stress \([\text{N/m}^2]\)
- \( C \): roughness coefficient (Chezy) \([\text{m}^{1/2}/\text{s}]\)
For a depth averaged flow velocity of 1.1m/s and a Chezy value of 47m$^{1/2}$ the shear velocity will be equal to 0.073m/s and the bottom shear stress equal to 5.37N/m$^2$. A passing ship will induce an increase in depth average flow velocity and thus in an increase in shear velocity and bottom shear stress.

2.4.2 Transport

When sediment particles start to move, three types of transport are noticeable; bottom transport, suspended transport and wash load. Wash load is the transport of very fine particles which will only settle when no flow occurs. Since still water does not occur in the reference location for this master thesis, wash load will not be looked further into.

When the bottom shear stress reaches critical values, sediment particles will start to move. These particles start rolling or sliding over the bed and will make small jumps, this is called saltations. Bottom transport takes place in the river Waal due to the flow velocities present. This is clearly visible on images made by Rijkswaterstaat made at different locations along the river Waal. (Pictures of this movie are used in Figure 2-5)

Determining the type of transport that will occur in a flow is possible by using the Rouse number. Hunter Rouse developed a dimensionless parameter showing the ratio between the fall velocity of a particle and the von Kármán constant times the critical shear velocity. Depending on the magnitude of this number, the type of transport can be estimated. This is very handy since there are a lot of sediment transport formulae available and not all of them take e.g. suspended transport into account. When one knows the type of transport to be expected, the right sediment transport formula can be used.

\[ P = \frac{W_s}{\kappa \cdot u_*} \]  
\[ W_s: \text{ fall velocity of the sediment particle [m/s]} \]  
\[ \kappa: \text{ von Kármán constant (≈ 0.41)} \]  
\[ u_*: \text{ shear velocity [m/s]} \]  

Regimes for the Rouse number are defined as:

<table>
<thead>
<tr>
<th>Type of transport</th>
<th>Rouse number regime</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bed</td>
<td>$P &gt; 2.5$</td>
</tr>
<tr>
<td>Suspended (50%)</td>
<td>$1.2 &lt; P &lt; 2.5$</td>
</tr>
<tr>
<td>Suspended (100%)</td>
<td>$0.8 &lt; P &lt; 1.2$</td>
</tr>
<tr>
<td>Wash</td>
<td>$P &lt; 0.8$</td>
</tr>
</tbody>
</table>

Table 2-10 Rouse number regimes

For the river Waal it is found (see section 2.4.2) that the fall velocity is equal to 0.03m/s and the shear velocity $u_*$ is equal to 0.073, the Rouse number for this situation is then equal to 1.02. The transport in the river Waal is thus to be considered as suspended transport.

Two parameters still remain unknown, the fall velocity of a sediment particle and the critical shear velocity. The fall velocity can be calculated using the following equations:
In which:
\[ w_j = \sqrt{\frac{4(s-1) \cdot g \cdot d}{3 \cdot C_D}} \]  \[ [2-39] \]

\( w_j \): fall velocity of a sediment particle \[ [m/s] \]
\( s \): relative density \( (\rho_s / \rho) \) \[ [-] \]
\( g \): gravitational acceleration \[ [m/s^2] \]
\( d \): particle diameter \[ [m] \]
\( C_D \): drag coefficient (Table 2-11) \[ [-] \]

\( C_D \) is depending on the grain Reynolds number and divided into three regimes:

<table>
<thead>
<tr>
<th>Reynolds number</th>
<th>Value for ( C_D )</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;Stokes range&quot;</td>
<td>0.1 &lt; Re &lt; 0.5</td>
</tr>
<tr>
<td>&quot;Newton range&quot;</td>
<td>400 &lt; Re &lt; 2 \cdot 10^4</td>
</tr>
<tr>
<td></td>
<td>Re &gt; 2 \cdot 10^5</td>
</tr>
</tbody>
</table>

Table 2-11 \( C_D \) values depending on the grain Reynolds number

With the grain Reynolds number defined as:
\[ \text{Re} = \frac{w_j \cdot d}{v} \]  \[ [2-40] \]

Since in most problems in civil engineering values of the grain Reynolds number are laying between the so-called Stokes range and the Newton range, no simple expression is available for describing the relation between \( C_D \) and Re. Van Rijn (1993) came up with three empirical relations for natural sediment where the fall velocity can be estimated using one of the following relations:

\[ w_j = \frac{g d^2}{18 v} \]  \[ [2-41] \]
\[ w_j = \frac{10 d v}{d} \left( \sqrt{1 + \frac{0.01 \Delta d^2}{v^2}} - 1 \right) \]  \[ [2-42] \]
\[ w_j = 1.1 \sqrt{\Delta d d} \]  \[ [2-43] \]

Since both the \( D_{50} \) and \( D_{90} \) values of the sediment in the river Waal are located in two different regimes \((d>1000 \mu m \text{ and } 100<d<1000 \mu m)\) equation [2-42] will be used. For the river Waal the fall velocity will be \( w_j = 0.03 \text{m/s} \) (using equation [2-43] results in a fall velocity of 0.14m/s which is very high). With a \( D_{50} \) of 700 \( \mu m \) the fall velocity of a grain as used in the flume will be 0.02m/s. Using equation [2-39] results in fall velocities of 0.33m/s for the river Waal \((C_d=0.2)\) and 0.26m/s for the flume \((C_d=0.2)\). These are different from the fall velocities calculated using van Rijn and are a lot bigger. In lecture notes of river engineering both equations are used, but equation [2-44] containing the shape factor \( C_D \) is more reliable for very low Reynolds values \((<1)\). With increasing Reynolds numbers, the relation between the shape factor and Reynolds number are less reliable. Therefore the values calculated using van Rijn will be used.

In the lecture notes of river engineering is stated that for a certain conditions a sediment transport formulae may be used to calculate transport quantities directly. For a small adjustment length \( L_a \) and a
small adjustment time $T_a$, transport formulae can be used to calculate the amount of transported sediment. In all other cases the formulae give an indication of the transport capacity but not the transport itself. The adjustment length $L_a$ (m) is defined as:

$$L_a \approx \frac{u_f \cdot h}{w_f}$$  \hspace{1cm} [2-45]

In which:

- $u_f$: depth averaged flow velocity [m/s]
- $h$: water depth [m]
- $w_f$: fall velocity of a particle [m/s]

And the adjustment time $T_a$ (s) is defined as:

$$T_a \approx \frac{h}{w_f}$$  \hspace{1cm} [2-46]

In which:

- $h$: water depth [m]
- $w_f$: fall velocity of a particle [m/s]

This results for the river Waal using the values from section 2.1.2 and results from the sediment in the flume of Deltares in the following adjustment lengths and -times:

<table>
<thead>
<tr>
<th></th>
<th>$L_a$ [m]</th>
<th>$T_a$ [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>River Waal ($D_{50} = 1000 \mu m$)</td>
<td>187</td>
<td>170</td>
</tr>
<tr>
<td>Flume ($D_{50} = 700 \mu m$)</td>
<td>2.3-9.2</td>
<td>23</td>
</tr>
</tbody>
</table>

Table 2-12 Adjustment lengths and -times

The adjustment length $L_a$ will be between 1.21m for 0.1m/s and 4.84m for 0.4m/s. Adjustment time for the same velocities is 12.1s. These two parameters indicate the actual time it will take for a grain to sink from the upper water level to the bottom and the length in which it is transported in that case. Values for the river Waal are also displayed in Table 2-12.

A ship sailing in the river Waal will induce an increase of flow velocities at the bottom. The duration of this increase in flow velocities can be estimated by dividing the length of the ship by the velocity of the ship. This results in a certain time it will take for a ship to pass from bow to stern a certain location with a certain speed. For a push barge combination sailing in the river Waal the length of a 2x2 combination with Europa IIa barges is equal to 2\times76.50m=153m. For simplicity it is assumed that during the passage of the ship one constant increase in flow velocities will occur.

<table>
<thead>
<tr>
<th>Sailing speed</th>
<th>duration of passage</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.56m/s</td>
<td>$\frac{153m}{2.56m/s}=60s$</td>
</tr>
<tr>
<td>3.07m/s</td>
<td>$\frac{153m}{3.07m/s}=50s$</td>
</tr>
</tbody>
</table>

Table 2-13 Passage times for different speeds

The sediment transport formula of Engelund-Hansen is used for determining the sediment transport in the river Waal. This formula takes bottom transport and suspended transport into account and is often
used for estimating sediment transport in rivers. In the river Waal suspended transport is occurring (see section 2.4.1).

Equation [2-47] is the Engelund-Hansen equation, since the bottom shear stress, sediment diameter and roughness are known, the sediment transport in the river Waal can now be estimated.

\[ s = 0.05 \cdot \sqrt{\frac{C^2}{g} \cdot \Delta \cdot D} \cdot \left( \frac{\tau_b}{\rho \cdot g \cdot \Delta \cdot D} \right) \]  

[2-47]

In which:

- \( s \) : transport per unit of time [kg/s]
- \( C \) : roughness coefficient (Chezy) [m\(^{1/2}\)/s]
- \( \Delta \): relative density \( \left( \frac{\rho_s - \rho_w}{\rho_w} \right) \) [-]
- \( \tau_b \): bottom shear stress [N/m\(^2\)]

For a grain diameter of 0.001m, a flow velocity of 1.1 m/s, a Chezy value of 47m\(^{1/2}\)/s and bottom shear stress of 5.37N/m\(^2\) a sediment transport of 0.00027kg/s is found. The increase in sediment transport can now be calculated which gives indication of the usage of the push barge combinations for the possible removal of small sandy shoals.

2.4.3 Prediction of erosion

Sieben (2008) came up with a simplified model, describing the flow and induced sediment transport. This model is treated below. It also contains the sediment transport, which is further treated in chapter 2.4.2. For more information about sediment transport one should read this section first.

In a kick-off document called “nieuwe bezems, efficienter baggeren” Sieben came up with a method to predict the amount of erosion that can occur due to the passage of a ship. This 1-D model is based on conservation of energy and assumes steady uniform flow. Results derived from this model are not validated or calibrated, but since nearly all important parameters are taken into account is shows the influence of the different parameters very well.

Sieben defined the sailing ship as shown in Figure 2-20, this is a modified version of the picture used in the kick-off document, only the labels are changed from Dutch into English.
Consider a ship, sailing with a speed $V_s$, a draught $d_2$ and a keel clearance $d_1$. Total water depth is equal to the sum of the draught $d_1$ and the keel clearance $d_2$. By assuming uniform and steady flow, the bottom shear stress $\tau_b$ is equal to:

$$\tau_b = -\rho_w \cdot u^2$$  \[2-48\]

In which:
- $\tau_b$ : bottom shear stress  \[N/m^2\]
- $\rho_w$ : specific mass of water  \[kg/m^3\]
- $u^2$ : flow velocity  \[m/s\]

The ratio between the bottom shear stresses at location I and II can no be noted as:

$$\frac{\tau_{b,II}}{\tau_{b,I}} = \left(1 + \frac{V_s \cdot d_2}{U \cdot d_1}\right)^2 \cdot \left(\frac{d}{d_1}\right)^2$$  \[2-49\]

In which:
- $\tau_b$ : bottom shear stress  \[N/m^2\]
- $d$ : total water depth (defined in figure xx)  \[m\]
- $d_1$ : keel clearance (defined in figure xx)  \[m\]
- $d_2$ : draft (defined in figure xx)  \[m\]
- $U$ : flow velocity  \[m/s\]
- $V_s$ : sailing speed of ship  \[m/s\]

And defining the mass balance by:

$$s_{II} - s_I = V_s \cdot \Delta z$$  \[2-50\]

In which:
- $s_I$ : sediment transport through cross section I  \[m^2/s\]
- $s_{II}$ : sediment transport through cross section II  \[m^2/s\]
- $\Delta z$ : bed level change  \[m\]

Sediment transport can be estimated with the empirical relation $s \propto \tau_b^{1/2}$ which is only valid for uniform flow with equilibrium transport. This is a very big simplification compared to the real flow and transport occuring. From Garcia (1998) it follows that a lot of common used sediment transport formulas are usable for modeling sediment transport around ships.
Substituting of [2-49] in [2-50] gives:

\[
\left( \frac{\tau_{b,II}}{\tau_{b,III}} \right)^{b/2} - 1 \right) s_I = V_s \cdot \Delta z \tag{2-51}
\]

According to de Vries (1967) sediment transport is, following from the already mentioned empiric relation, equal to the displacement velocity (m/s) of bed changes:

\[ s = \frac{w \cdot d}{b} \tag{2-52} \]

- \( w \): displacement velocity [m/s]
- \( d \): water depth [m]
- \( b \): constant depending on used sediment formula [-]


\[ \frac{\Delta z}{d} = \frac{U}{V_s} \cdot \frac{w}{b \cdot U} \cdot \left( 1 + \frac{V_s \cdot d_s}{U \cdot d} \right)^b \cdot \left( \frac{d}{d_s} \right)^b \cdot 1 \tag{2-53} \]

So now we have a simplified model which describes the bed level changes as a function of the water depth, keel clearance, draft, sailing speed and bottom mobility. An indication of possible erosion effects can now be done using common values for the river Waal.

- \( b \) : 5 (Engelund-Hansen, 1967) [-]
- \( d \) : 5.1 [m]
- \( U \) : 1.1 [m/s]
- \( w \) : 2.0 [m/day]

The erosion as a function of the sailing speed for different under keel clearances is plotted in Figure 2-21. A 50% UKC means that the draft of the ship is replacing 50% of the total water depth, leaving the ship with 50% of the water depth under its keel. According to the model of Sieben, an increase in draft (so a decrease in UKC) and an increase in sailing speed, results in a significant increase in the amount of sediment transported.
Push barge combinations sailing in the river Waal are sailing in an upstream direction with an average velocity of 2.56 m/s and a maximum draught of 4 meters (UKC=21.5%). When plotting the results of the sediment transport in Figure 2-21, the lines with larger UKC percentages are not very well visible anymore. Therefore these results are plotted separately in Figure 2-22.

It is obvious that the erosion indicated by the model of Sieben is not realistic for the combination of practical values of the river Waal and the push barge combination of Veerhaven. An erosion of more than 4.5 m will not occur as a result of a passing push barge combination.
A possible explanation for this excessive erosion is the fixed water depth in the equation. In real life, after some erosion occurs, the water depth will increase as a result of this erosion. This leads to lower flow velocities and therefore less erosion. Calculation of erosion should therefore be done in an iterative way; first calculating the erosion occurring during a time interval and then adjusting the depth after which erosion during a next time interval can be calculated.

One important parameter not included in this model is for instance the grain diameter. Looking at the formula one can calculate the erosion as a function of many variables, but none of them contains information about the grain size and weight. This might be a possible reason for the overestimation of the amount of erosion.

Also Sieben assumes that all the water flowing in front of the ship, will pass underneath the ship. Fanning out due to friction is hereby neglected although model tests show a decrease of nearly 50% in specific discharge for water passing the bow of the ship. This means that the specific discharge as calculated in the model of Sieben, in reality will be nearly twice as small.

Assuming only 50% of the specific discharge flowing under the bow gives a new model, defined in [2-56].

First, the new balance is defined as:

\[
\frac{\tau_{b,II}}{\tau_{b,I}} = -\frac{\rho_w \left(0.5 \cdot \left(q + V_s \cdot d_z\right)\right)^2}{\rho_v \cdot (q / d)^2}
\]

Becomes:

\[
\frac{\tau_{b,II}}{\tau_{b,I}} = \left(0.5 \cdot \frac{0.5 \cdot V_s \cdot d_z}{U \cdot d}\right)^2 \cdot \left(\frac{d}{d_i}\right)^2
\]

And results in:

\[
\frac{\Delta z}{d} = \frac{U}{V_s} \cdot \frac{w}{b \cdot U} \cdot \left(0.5 \cdot \left(\frac{0.5 \cdot V_s \cdot d_z}{U \cdot d}\right)^b \cdot \left(\frac{d}{d_i}\right)^b - 1\right)
\]

Figure 2-23 shows the new results compared with the situation in which continuity is assumed for the 'Veerhaven case'.
The fanning out reduces the erosion significantly; with a ship speed of 2.56 m/s the erosion is three times as small as in the first approximation was calculated. The effect of fanning out should be taken into account and proves to be of importance. Note that squat is not taken into account at this stage.

To gain more insight in the sediment transport of the model of Sieben, equation [2-56] is used but now the sediment transport $s$ is not substituted into the equation as firstly done using de Vries (1967). At first the possible erosion $dz$ is calculated using equation [2-57] with the following data:

\[
\begin{align*}
V_s & : 2.56 \text{ [m/s]} \\
\bar{d} & : 5.1 \text{ [m]} \\
\bar{d}_1 & : 1.1 \text{ [m]} \\
\bar{d}_2 & : 4 \text{ [m]} \\
U & : 1.1 \text{ [m/s]}
\end{align*}
\]

Rewritten

\[
\Delta z = s \cdot \frac{1}{V_s} \left\{ \left( \frac{0.5 + \frac{0.5 \cdot V_s \cdot \bar{d}_2}{U \cdot \bar{d}}} {U \cdot \bar{d}} \right)^b \cdot \left( \frac{\bar{d}}{\bar{d}_1} \right)^b - 1 \right\} \quad [2-57]
\]

At first the sediment transport is defined as [2-52] and equal to $2.36 \times 10^{-5}$ m$^2$/s. Now the erosion after some time can be calculated, after which the new depth can be used for a new calculation. This results in a new value for the erosion which will be lower for an increased depth. Repeating this method for small time intervals until the passage time of a push barge combination is reached will give a more realistic insight in the erosion occurring due to the passage of a barge train.
Table 2-14 shows the iteration process for a time interval of 10s.

<table>
<thead>
<tr>
<th>Iteration no.</th>
<th>d [m]</th>
<th>d1 [m]</th>
<th>d2 [m]</th>
<th>dz [m]</th>
<th>t [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.1</td>
<td>1.1</td>
<td>4</td>
<td>1.3433</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>6.4433</td>
<td>2.4433</td>
<td>4</td>
<td>0.0327</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>6.4760</td>
<td>2.4760</td>
<td>4</td>
<td>0.0308</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>6.5087</td>
<td>2.5087</td>
<td>4</td>
<td>0.0290</td>
<td>40</td>
</tr>
<tr>
<td>5</td>
<td>6.5414</td>
<td>2.5414</td>
<td>4</td>
<td>0.0274</td>
<td>50</td>
</tr>
<tr>
<td>6</td>
<td>6.5741</td>
<td>2.5741</td>
<td>4</td>
<td>0.0259</td>
<td>60</td>
</tr>
<tr>
<td>7</td>
<td>6.6069</td>
<td>2.6069</td>
<td>4</td>
<td>0.0244</td>
<td>70</td>
</tr>
</tbody>
</table>

Table 2-14 erosion with iterative calculation of depth

The first iteration shows a large amount of erosion (1.34m) which is not realistic. Assume that the outcome of the first iteration can be neglected. The bed level of 6.44 should be used for further calculations. Adding up the amount of erosion during the next minute (10-70s) gives a value of the total erosion of 0.17m. The bottom level should thus be 1.34m higher, and will be equal to 5.27m.

The parameter depth should be divided into two parameters, calculation depth ‘d_cal’ and output depth ‘d_out’. Now the opportunity occurs to neglect the first iteration but give more realistic output values. When the erosion as calculated in the first iteration is neglected, the second iteration would give exactly the same value for erosion. For a ‘realistic’ calculation, the first iteration is thus needed. So let’s state:

\[ d_{out,i} = d_{cal,i} - \Delta z_0 \]  \[2-58\]

In which the subscript ‘i’ stands for the iteration number and \( \Delta z_0 \) represents the erosion as calculated with the ‘start-up’ iteration (noted as the subscript zero).

Although the calculated 0.17m intuitive looks more realistic, this value can not be validated since measurements are not available.
3 Physical model (Flume experiments)

3.1 Objective of physical model tests

Main goal of doing these experiments is gaining insight in the processes involving sediment transport due to the passage of a barge and gaining insight in the flow velocities underneath a passing barge. Effect of the passage of the barge on the flume bed will be measured and monitored to gather quantitative and qualitative data which can be used for this master thesis. Also the behaviour of a towed barge combination can be monitored which will give information about the hydrodynamics of the barge combination.

As a result of the experiments one should be able to draw a conclusion regarding the effectiveness of using a barge for removal of small shallow shoals in a river bed.

3.2 Scaling the physical model

Using scale models to gain insight in prototype situations has its limitations. Dimensions like length or width can be scaled with e.g. a factor two, but the material properties will remain the same. A cube made of concrete has the same specific gravity as the model scale cube which is two times smaller. Not to mention the volume of the cube which, using a length scale factor $N_L=2$, is $N_L^3$ times smaller than the prototype version of the cube. To be able to get results that are comparable without scaling problems there are some scaling rules defined. The best way for scaling proportions is using dimensionless numbers like the Froude or Reynolds number.

In model testing for Civil Engineering purposes, dimensionless numbers which are often used are:

Reynolds:

\[
\text{Re} = \frac{u \cdot R}{\nu}
\]  \hspace{1cm} [3-1]

Shields:

\[
\theta = \frac{u^2}{\sqrt{g \cdot d}}
\]  \hspace{1cm} [3-2]

Advantages of using dimensionless numbers for scaling purposes is that results of model scale experiments can be translated easily to full scale situations. Nevertheless one should always be careful before drawing conclusions from model scale experiments, this will be explained further on in this chapter.

Since sediment transport and the initiation of transport are important for this master thesis, the scaling of the transport conditions are one of the highest priorities. Scaling from prototype to model scale is hereby
done using the Shields parameter, a parameter giving information about the mobility of a sediment particle depending on flow conditions.

The dimensions of the ship were not easy to scale, a four barge combination has a length of more than 190 m which means that using a scale factor of 1:20 still a length of 9.5 meters has to be build. This results in a very heavy and very difficult to handle scale model. Therefore was decided to only scale the most important parameters so that the sediment transport would be similar to the prototype situation.

### 3.3 Material and Equipment

The tests were performed at Deltares, which has a number of flumes available for experiments in which certain river conditions can be simulated. Flume no.4 has the following dimensions; length: 20 m and width of 3 m. Two pumps are available and capable of producing a maximum discharge of 500 l/s. The width of the flume was decreased to 2 m to be able to maintain the maximum discharge in the flume. Figure 3-2 shows two photographs of the flume, the left one gives an overview and the right one shows clearly the narrowing of the flume made by a wooden shield.

![Figure 3-2 Flume no. 4 at Deltares](image)

Pumps as used in flume number 4 where indicated with P41 and P42. Each pump is able to produce a maximum discharge of 0.250 m$^3$s$^{-1}$ which can be regulated using valves. Each pump has three levels, Off (O), Low (L) and High (H), indicating the capacity used by the pumps. Each set of pumps is connected to a discharge meter, showing the percentage of the capacity of the pump. A 100% indication means that both of the pumps are at full capacity. Using the valve regulation and the three different levels on the pump made it possible to control the discharge of the flume in an accurate way. In appendix A more information can be found about the possible flow velocities using the different water levels and pump levels.

In this flume, dunes were constructed, representing the shallow sections in a river bed. Each dune has a total length of 2.15 m and increases in height from 0.10 m until 0.18 m related to the flume bottom. The dimensions of the dunes are on a scale of 1:10, so the height difference of 0.08 m is equal to a height difference of 0.80 m in prototype scale. Figure 3-3 shows a schematic view of the dunes and a photograph of the dunes as constructed in the flume.
A sand fraction with a D50 of 700µm was chosen by H. Bots, he made calculations to scale this diameter to gain comparable results as if it was in the river Waal. During the calculations an error was made and the actual grain diameter should have been 200µm. He calculated that the actual transport should be nearly 27 times as high when using the correct grain diameter. This has to be taken into account when analyzing the bed level changes later on.

Figure 3-3 Schematic model and photograph of the dunes as constructed in the flume

Measuring the data during the experiments is done using several different tools. All the measurements were sent to a computer on the measuring trolley. This computer received the data and saved it to a file with a frequency of 20Hz, meaning that every second 20 samples of each signal were saved. Equipment used for the flume experiments does not measure e.g. velocities or forces, it measures voltage. This voltage has to be translated to more practical units using calibration reports. Each device has its own calibration report where one can find translation regulations for translating volts to practical units. To measure all the data at the same time, 15 different signals were recorded. Not every device will be dealt with in detail, only the ones of significant importance. An impression of the equipment located on top of the measuring trolley is given in Figure 3-4.
Measuring the bed levels was done using a profile tracker. This device consists of two main elements, a probe which measures the resistance in the water and servos to move the probe in a horizontal and vertical direction. Since the horizontal movement was restricted as can be seen in the left picture of Figure 3-5 only 0.78 m of the flume width could be measured. The measuring grid was therefore chosen in such a way that on both sides of the barge about 0.10 m could be measured. Measurements done by the profile tracker were depending on the resistance of the water, which is dependent of e.g. temperature. H.Bots found out that measurements done in the morning compared to measurements of exactly the same (undisturbed) area in the evening showed differences. He concluded that measurements done during different daytimes could lead to a measuring error of 0.02 m. According to the calibration report, delivered with the probe, an accuracy of ±0.01 m is achieved when using the probe according to the manual.

Constructing the dunes used for the flume experiments was a time consuming job. At first the flume had to be emptied (only water) and the sand in the flume had to be dry. Together with the laboratory assistant, H.Bots designed a mold which has the shape of the dunes. This mold can be mounted underneath the measuring trolley and using ropes, the mold can be towed over the sand to get the dune profiles as shown in Figure 3-6. The measuring trolley was moved after each successful construction.

Some remarks have to be related to the accuracy of the dune construction. Since the mold had to be towed over the sand bed using two ropes, the mold was not always towed in the same way, got stuck a few times and had to be placed manually using the measuring trolley. This resulted in dune profiles that are in
general equal, but varying in height and length. A perfect result would be if all the dunes constructed had exactly the same dimensions, but this is not possible. Although the dimensions vary a little, this has nearly no influence on the usability of the measurements. Each dune was measured before and after the experiment. This resulted in two profiles from which the amount of sediment transport could be derived. Result was that only the difference between the two measurements were of importance, a slightly varying dimension was therefore no obstruction for a usable test result.

Measuring flow velocities under the barge is done using EMS probes. EMS stands for Electro Magnetic Sensor and operates by measuring induction between two fixed points. The change in a electromagnetic field can be measured and translated to flow velocities. For the experiments at Deltares, three EMS probes were present. Taking into account the contraction described by Rigter (1989) occurring just under the bow of a ship, it was decided that one of the adjustable probes had to be installed here, to get the most interesting data. Measuring flow velocities related to the distance from the barge makes it possible to construct a velocity profile. The EMS is able to measure flow velocities in two directions (X&Y) and flow velocities of ±2.50 m/s. Before each experiment started, the EMS had to be calibrated.

Two other locations for the remaining EMS probes are amidships of the barge and at the stern of the barge. At the stern of the barge, the non adjustable EMS was mounted.

The barge, constructed for the model experiments was has a length of 1.50m, a width of 0.60m and a total height of 0.45m. A freeboard of 0.15m was realized, under which a more smooth bow was created using polyethylene and epoxy rosin was used to fix the polyethylene on the barge. This was done to create a smooth bow, comparable with the bow of a barge.
3.4 Approach and Configurations

3.4.1 Approach

During the experiments a lot of data was gathered. As mentioned before, measuring the bed-profile is very important as it gives an indication of the amount of transported sediment and where the sediment is settled again. Also the speed of the measuring trolley, the pull force on the barge (connected to the trolley with steel wires), the X- and Y-directions of the flow velocity at three locations underneath the barge and the vertical displacement of the barge at the bow and stern were measured.

Measuring the bed profile is done by using a ‘provo’ or profile tracker (eng.), for more information about this instrument see chapter 3.3, where a detailed description of the instrument is given.

The measurable area of the Provo perpendicular to the longitudinal axis of the flume is 0.78 m, which means that, taking into account a width of the barge of 0.60 m, approximately an extra 0.10 m at each side of the barge could be measured. Figure 3-9 shows the measuring grid as used for measuring the bed profile.

Measuring the longitudinal profile, parallel to the flume’s axis, is also done using the Provo. Since the Provo was mounted to the measuring trolley, obviously the range of these measurements where depending on the trolley’s range. Decided was to measure 0.500 m in front of the first dune and 0.500 m
after the last dune. This way sediment transported over a longer distance than the assumed several centimeters could be measured too.

In the longitudinal direction cross sections were measured with a mutual distance of 0.10 m each dune has a total length of 2.15 m and six dunes were constructed in the flume, so in total a section of (0.50 m + 6 x 2.15 m + 0.50 m) x 0.78 m = 10.84 m$^2$ was measured before and after each run.

Secondly the velocities near and under the barge are important. A passing barge will result in higher flow velocities due to the reduced wet surface in the flume. At this moment there is not much information about flow velocities below ships/barges. Since the flow velocities below the barge have a large effect on the sediment transport and data about the flow velocities is scarce, these have to be measured too. At the bow of the barge and at the center of the barge flow velocities will, according to theory about the subject, be the highest. At the bow due to the bow shape, a contraction will occur, resulting in a contraction that influences the flow. Using the right equipment, we will be able to measure the size of this contraction and the flow velocities due to the contraction. These values can be compared with the values gained by at the other two locations underneath the ship's hull and will give a nice overview of the flow distribution underneath the barge.

Thirdly, the forces on and the movement of the barge itself needs to be measured. During these experiments the barge is pulled by the measuring trolley instead of, as in real life, pushed forward by a propeller. To be able to get an impression of the power needed to give the barge it's certain speed, the pulling force needs to be measured. Two steel cables connected from the barge to two force indicators are the connection between the measuring trolley and the barge. The two force indicators measure the amount of pulling force needed to pull the barge. Using available theory can lead to an estimation of the amount of force needed by a propeller to match the barge's speed. When this amount of power is calculated, an extra factor due to propeller wash can be added to the theoretical suspended sediment transport.

To get an idea about the barge's movements during the experiment two displacement indicators are attached to the barge, one to the bow and one to the stern of the barge. These displacement indicators measure the displacement of the barge during the experiment. Information gained by these indicators can be useful because at a certain speed or acceleration the barge will tilt a bit. This displacement is influencing the flow beneath the barge thus influencing the sedimentation beneath the barge. Also some
important processes like squat can be measured, this information is useful to check measured data with calculations.

Finally, except for the data gathered during the experiments by the different measuring tools, film images will be made during the experiments. Together with the measured data they will give a good indication of the processes in the flume during the experiment. For a better film quality some additional underwater lights are placed around the barge.

3.4.2 Configurations

In order to get a good overview of the effects of the different parameters and their effect on the sediment transport, different configurations will be tested and analyzed. The under keel clearance of a ship determines the flow velocity underneath the ship, which again influences the amount of sediment transported. Following from this theory experiments with varying under keel clearance will prove this. Also the flow velocity in the flume has its effect on the sediment transport

3.4.2.1 Velocity profile

To gain insight in flow velocities under the barge, a velocity profile is made. A period of three minutes is needed when measuring with a 20Hz frequency interval to gain an accurate measurement. The EMS probe location was changed after one successful measurement.

In the prototype situation, push barge combinations are sailing against or with the flow of the river. Measuring a velocity profile with a duration of three minutes for each measurement sailing against the flow direction would give the most interesting results. This is hard to model using a scale model, since the flume would have to be of significant length. A period of three minutes is needed for a reliable measurement in which the end of the flume already will be reached when using a moving barge. At first some measurements are made to gain insight in the sensitivity of the EMS. These measurements resulted in a threshold considering the distance of the EMS to the bottom of 0.08m, after which the EMS would give irregular values as a result as the earlier described sensitivity for certain materials. A UKC of 0.20m is chosen to be able to compare with measurements done with a moving barge during the tow-experiments. Velocities measured during these test are only measured at one height, so it can be interesting to compare results of the different experiments when using the same UKC.
3.4.2.2 Sedimentation-Erosion run

Sedimentation and erosion is investigated by using the barge and pulling it in the opposite direction of the flow in the flume. Decided is to vary the speed of the barge and its draught.

<table>
<thead>
<tr>
<th>Speed</th>
<th>0.15m UKC</th>
<th>0.20m UKC</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1m/s</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>0.3m/s</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>0.5m/s</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>

*Table 3-1 measured combinations for sedimentation-erosion run*

A velocity of 0.5m/s is the maximum possible for the measuring cart and since six dunes are available an amount of two dunes for each velocity is chosen. Water levels will be kept fixed at 0.54m and the flow in the flume itself is set to full. Before and after each experiment the bed levels will be measured to gain information on the changes in bed levels as the result of the passage of the barge.
3.5 Results

3.5.1 Velocity profile (static condition)

To gain insight in flow velocities under the barge, a velocity profile is measured. During a three minute interval velocities are measured at three positions underneath the barge (see Figure 3-11). Three EMS probes are used, from which only two could vary in height, the third one was fixed into one position. During the measurements the parameters in Table 3-2 were used. For more information about the EMS probes, see chapter 3.3 Material and Equipment.

<table>
<thead>
<tr>
<th></th>
<th>Water level</th>
<th>0.46m</th>
</tr>
</thead>
<tbody>
<tr>
<td>T</td>
<td>Draught</td>
<td>0.26m</td>
</tr>
<tr>
<td>UKC</td>
<td>Under Keel Clearance</td>
<td>0.20m</td>
</tr>
<tr>
<td>U</td>
<td>Uniform flow velocity</td>
<td>0.1-0.4m/s</td>
</tr>
</tbody>
</table>

Table 3-2 Parameters for velocity profile

Measurements are carried out with a static position of the barge since the length of the flume is not enough to achieve a sufficiently long measurement time for measurements with a moving barge. Important when looking at the figures showing the velocity profiles is to keep in mind that only a span of 12cm is measured, from a total of 20cm keel clearance.

Measurements gained from these experiments are also used for analyzing situations in which the barge would actually move with a certain speed. This is possible as long as some assumptions are made; these assumptions are earlier used by Schijf (1949) and are summarized and explained in chapter 2.3.5.

Data gathered is transformed into figures and tables using Matlab. Each measurement of three minutes is averaged to one value and plotted in a graph showing the velocity in relation to the distance from the barge’s bottom.

For the sake of simplicity four indices are introduced to make comparison between different locations easier. Location ‘0’ is the undisturbed situation in front of the barge representing an uniform steady flow in which the specific discharge is 100% of the original, so no dissipation or losses have taken place. Following from the first location, three other locations are defined, consecutive location ‘I’, location ‘II’ and location ‘III’ each representing a position of a EMS-probe. Figure 3-7 gives an indication of the exact location of the probes, the schematized locations are shown in Figure 3-11.

Velocity profiles measured are shown in two ways; all together with different pictures for each location, secondly every location and velocity separated. In every figure the horizontal axis represents the flow velocity, a positive flow velocity (m/s) is directed from the bow to the stern of the barge. Margins and axis
limits are kept the same for every picture to allow visual comparison. On the vertical axis the distance (in cm) to the bottom of the barge is shown, an increasing distance is thus closer to the bottom of the flume. Dotted lines represents the flow velocities in the flume, one exception is made; in the figures showing the separate flow profile, the blue dotted line represents the zero velocity point.

Figure 3-12 Measured velocities at bow of the barge

Figure 3-13 Measured velocities amidships of the barge

Figure 3-14 Measured velocities at stern of barge
Figure 3-15 velocity profiles with four different flow velocities at the barge’s bow.
EMS measurements midships (II)

Figure 3-16 velocity profiles with four different flow velocities amidships of the barge
Figure 3-17 velocity profiles with four different flow velocities at the barge's stern
Since the boundaries founded by the keel of the barge and the flume bottom are not moving, velocities are zero. Measurements of the flow velocities at the flume bottom are not possible with the EMS-probes, but the measurements at the keel of the barge should be proving this statement. When taking a closer look at the results, different flow velocities are found at the keel of the barge, almost none of them is equal to zero. A possible explanation is the size, or specifically, the thickness of the probes. The shape of the probes is an oval sphere with a thickness of approximately 1cm. In the most upward position the probe is still extended for a small distance out of the barge's bottom (approximately 1.0cm). A detail photograph of a part of the barge's keel is shown in Figure 3-18, one can clearly see the holes provided for the EMS-probes. This can explain why the flow velocities in the uppermost position of the probes are not equal to zero.

Measurements at location III, at the fixed EMS probe even showed negative measured values, this probe is flat and mounted in the keel of the barge. Since the probe is fixed and therefore measures every run at the exact same position, one should expect the same flow velocity for each run with equal flow velocities. Probes located at I and II where changed in vertical position after each run, but the probe at location III remained steady. This means that during the measurement of one profile (at 1,3,...,13) seven measurement where done which are different from each other. At first these measurements at the stern of the barge were negative. Negative flow velocities mean flow velocities opposite to the flow direction in the flume. Two reasons for these negative values can be suggested; either the probe is unreliable or the probe is oriented in the wrong direction. Assuming a wrong orientation of the probe gives at least the plausible argument to keep using the values and investigate further plausible relations between the different measured positions.

Discharge in the flume was every time set to a certain value, such that flow velocities where equal to the desired 0.1-0.4m/s. Since discharge is a function of the flow velocity times the wet surface, the depth averaged flow velocity could be checked by dividing the discharge by the water depth times the width of the flume. Comparison of the flow velocities under the barge and in front of the barge can be done using the specific discharge and by looking at the flow velocities.
When dealing with flow velocities varying with depth, one can calculate the specific discharge to gain insight in the amount of water flowing through a certain cross section. Specific discharge $q$ [m$^2$/s] is defined as the depth integrated flow velocity, or in formula form:

$$q = \int_{z_0}^{h} u(z) \, dz$$  \hspace{1cm} [3-3]

With:
- $q$ specific discharge [m$^2$/s]
- $u(z)$ flow velocity at $z$ [m/s]
- $z_0$ bed level [m]
- $h$ water level [m]

Flow velocities in front of the barge are assumed uniformly distributed. A comparison of the specific discharge between the measured locations can now be made. Flow velocities as measured during the experiments are shown in Table 3-4, Table 3-5 and Table 3-6. Using these measurements leads to the specific discharges shown in Table 3-3.

<table>
<thead>
<tr>
<th>$U_m$ [m/s]</th>
<th>$h$ [m]</th>
<th>$q \times 10^{-2}$ [m$^2$/s]</th>
<th>$q \times 10^{-2}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 [m/s]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.10</td>
<td>0.46</td>
<td>4.60</td>
</tr>
<tr>
<td>I</td>
<td>0.06</td>
<td>0.20</td>
<td>1.15</td>
</tr>
<tr>
<td>II</td>
<td>-0.01</td>
<td>0.20</td>
<td>-0.14</td>
</tr>
<tr>
<td>III</td>
<td>0.01</td>
<td>0.20</td>
<td>0.12</td>
</tr>
<tr>
<td>0.2 [m/s]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.20</td>
<td>0.46</td>
<td>9.20</td>
</tr>
<tr>
<td>I</td>
<td>0.24</td>
<td>0.20</td>
<td>4.88</td>
</tr>
<tr>
<td>II</td>
<td>0.14</td>
<td>0.20</td>
<td>2.87</td>
</tr>
<tr>
<td>III</td>
<td>0.11</td>
<td>0.20</td>
<td>2.14</td>
</tr>
<tr>
<td>0.3 [m/s]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.30</td>
<td>0.46</td>
<td>13.80</td>
</tr>
<tr>
<td>I</td>
<td>0.43</td>
<td>0.20</td>
<td>8.57</td>
</tr>
<tr>
<td>II</td>
<td>0.28</td>
<td>0.20</td>
<td>5.63</td>
</tr>
<tr>
<td>III</td>
<td>0.05</td>
<td>0.20</td>
<td>1.06</td>
</tr>
<tr>
<td>0.4 [m/s]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.40</td>
<td>0.46</td>
<td>18.40</td>
</tr>
<tr>
<td>I</td>
<td>0.53</td>
<td>0.20</td>
<td>10.59</td>
</tr>
<tr>
<td>II</td>
<td>0.41</td>
<td>0.20</td>
<td>8.20</td>
</tr>
<tr>
<td>III</td>
<td>0.11</td>
<td>0.20</td>
<td>2.10</td>
</tr>
</tbody>
</table>

Table 3-3 Calculated velocities and discharges
### Table 3-4 Flow velocities at bow (I)

<table>
<thead>
<tr>
<th>Distance to keel [m]</th>
<th>Flow velocities [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.10</td>
</tr>
<tr>
<td>0.00</td>
<td>0.102</td>
</tr>
<tr>
<td>0.02</td>
<td>0.116</td>
</tr>
<tr>
<td>0.04</td>
<td>0.122</td>
</tr>
<tr>
<td>0.06</td>
<td>0.121</td>
</tr>
<tr>
<td>0.08</td>
<td>0.117</td>
</tr>
<tr>
<td>0.10</td>
<td>0.072</td>
</tr>
<tr>
<td>0.12</td>
<td>-0.247</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.122</td>
</tr>
<tr>
<td>Mean</td>
<td>0.058</td>
</tr>
</tbody>
</table>

### Table 3-5 Flow velocities amidships (II)

<table>
<thead>
<tr>
<th>Distance to keel [m]</th>
<th>Flow velocities [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.10</td>
</tr>
<tr>
<td>0.00</td>
<td>-0.104</td>
</tr>
<tr>
<td>0.02</td>
<td>0.012</td>
</tr>
<tr>
<td>0.04</td>
<td>0.021</td>
</tr>
<tr>
<td>0.06</td>
<td>0.027</td>
</tr>
<tr>
<td>0.08</td>
<td>0.019</td>
</tr>
<tr>
<td>0.10</td>
<td>-0.009</td>
</tr>
<tr>
<td>0.12</td>
<td>-0.035</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.027</td>
</tr>
<tr>
<td>Mean</td>
<td>-0.010</td>
</tr>
</tbody>
</table>

### Table 3-6 Flow velocities at stern (III)

<table>
<thead>
<tr>
<th>Distance to keel [m]</th>
<th>Flow velocities [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.10</td>
</tr>
<tr>
<td>0.00</td>
<td>0.016</td>
</tr>
<tr>
<td>0.02</td>
<td>0.019</td>
</tr>
<tr>
<td>0.04</td>
<td>0.010</td>
</tr>
<tr>
<td>0.06</td>
<td>0.008</td>
</tr>
<tr>
<td>0.08</td>
<td>0.006</td>
</tr>
<tr>
<td>0.10</td>
<td>0.005</td>
</tr>
<tr>
<td>0.12</td>
<td>0.007</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.019</td>
</tr>
<tr>
<td>Mean</td>
<td>0.009</td>
</tr>
</tbody>
</table>
Above data is used to get a more visual impression of the differences between the four measured locations. In front of the barge, flow is undisturbed by the presence of the barge, this location is defined as I. Assuming the specific discharge at location 0 as the 100% specific discharge, the development of q along the barge is shown for all four flow velocities. On the horizontal axis the four locations and four different velocities are shown. Each velocity is displayed with in an own color, so comparisons can be made more easy. The vertical axes shows the percentage of the specific discharge in which 100% stands for the situation as described above.

Clearly visible is the reduction of the specific discharge along the barge. In the case of no friction and no flow dispersion the specific discharge between the locations should stay the same. Between the flume and the bow of the barge 50% of the water is diffused. This can be explained by the ‘fanning out’ effect, due to velocity differences between the water flowing under the barge and the water next to the barge, friction causes the water to change direction. The diffusion caused by this friction points the water to the slower regions, being the water next to the barge. This results in a decrease of discharge under the barge, although higher flow velocities are found, the total volume of water transported along one line under the barge decreases. Location III is displayed in Figure 3-19 with the original obtained measurements; in the next figure the positive values are shown. The velocity in location I measured 12cm under the keel of the barge (see Figure 3-15, U=0.1m/s) is very different from the other velocities measured. For some unknown reason the velocity is very low compared to the other measurements. It is of course possible that the value is well measured, but in that case it is hard to explain a negative flow velocity, especially when other measurements show no comparable results. Data gained from the measurement is removed and a new value for the specific discharge at location I with U=0.1m/s, is calculated by first calculating the new average velocity and then the new specific discharge. Without the measurements at 12cm underneath the barge's keel, the following results are measured and calculated:

<table>
<thead>
<tr>
<th>Location</th>
<th>( U_m ) [m]</th>
<th>h [m]</th>
<th>( q \times 10^{-2} ) [m(^2)/s]</th>
<th>( q \times 10^{-2} ) [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>U=0.10m/s</td>
<td>I 0.108 0.20</td>
<td>2.16</td>
<td>47%</td>
<td></td>
</tr>
</tbody>
</table>
Using this in the overview again results in Figure 3-20. The change in $q$ seems now more plausible when comparing with the other values at location I.

![Figure 3-20 Specific discharges (in %) with modified value](image)

Roughly one can state that 38-53% of the specific discharge diffuses before it reaches the EMS-probe at the bow. Then, when looking at the amidships placed EMS, again the specific discharge decreases with approximately 12-21% (neglecting the specific discharge measured at 0.1 m/s). And finally at the EMS at the stern of the barge more plausible results are visible, the reliability is unknown. Due to friction a decreasing discharge can be expected and is visible between locations 0-I-II. Looking at the differences of specific discharge related to the flow velocities, measurements show that the higher the flow velocity, the more water flows under the barge. Thus, the fanning out effect reduces with an increasing flow velocity. At the bow only 55% (average over location I) of the total specific discharge in front of the barge is left.

Looking at the flow velocities a certain pattern can be distinguished. Flow velocities under the bow of the barge are increasing as the flow velocity in the flume increases. Comparing the increase of flow velocities relative to the flow velocity in the flume is done in Table 3-7. $dU$ represents the difference between the flow velocity and the measured velocity.

<table>
<thead>
<tr>
<th>$U_{\text{u, max}}$ [m/s]</th>
<th>$dU$ [m/s]</th>
<th>$dU$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 m/s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>0.122</td>
<td>0.022</td>
</tr>
<tr>
<td>II</td>
<td>0.027</td>
<td>0.073</td>
</tr>
<tr>
<td>0.2 m/s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>0.262</td>
<td>0.062</td>
</tr>
<tr>
<td>II</td>
<td>0.204</td>
<td>0.004</td>
</tr>
<tr>
<td>0.3 m/s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>0.457</td>
<td>0.157</td>
</tr>
<tr>
<td>II</td>
<td>0.356</td>
<td>0.056</td>
</tr>
<tr>
<td>0.4 m/s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>0.548</td>
<td>0.148</td>
</tr>
<tr>
<td>II</td>
<td>0.485</td>
<td>0.085</td>
</tr>
</tbody>
</table>

**Table 3-7 maximum flow velocities**
This difference between velocities gives an indication of the flow pattern under the barge. Normally when all the water flowing in front of the barge, also flows under the barge itself, flow velocities have to increase since the flow carrying height decreases (Q=U*A). In practice, like here in the flume, friction and other processes like the fanning out, reduce the discharge under the barge.

As expected nearly on every location and during every flow velocity, the measured velocities increase relative to the flow velocities in front of the barge. Only at location II (amidships of the barge) during a flow velocity of 0.1 m/s, the change in flow velocity is negative. One should expect at least a flow velocity of 0.1 m/s, but in this case the flow is barely above zero.

Looking at the maximum flow velocities again the increase in flow velocity is less at location II then at location I. This is related to the decrease in specific discharge as mentioned earlier. The less water flows through a certain location, the less the flow velocity will increases at that specific location.

Comparing the velocity profiles with each other also a certain resistance is present, at some point (seems to be 0.3 m/s) the impulse delivered by the water is high enough to guide the water under the barge. Flow velocities lower than this specific point will not lead to significant higher flow velocities under the keel of the barge.

Dimensionless flow velocities are used to make a comparison between different measurements easier. In this case the dimensionless flow velocity is defined as:

\[ u_{(-)} = \frac{u(z)}{u_0} \]  \[3-4\]

In which:
- \( u_{(-)} \) : dimensionless flow velocity
- \( u(z) \) : flow velocity at level \( z \), in which \( z \) is a discrete function
- \( u_0 \) : flow velocity in undisturbed situation, at location 0

In this case the dimensionless flow velocity is defined as:

\[ u_{(-)} = \frac{u(z)}{u_0} \]  \[3-4\]

The \( u_0 \) is defined in the same way as Schijf (1949), the velocity of the barge is thus added to the flow velocity when they are opposite directed from each other.

Now the increase or decrease in flow velocity relative to the original flow velocity can be compared with different situations. Figure 3-21 shows the dimensionless flow velocities for each location. Using the available data points a second order polynomial is determined. Displayed is the so called 'R²'-value, giving an indication of accuracy of the equation of the trend line. An R²-value of 1 means a perfect fit.

In MS Excel 2007 one can vary the order of the trend line from 2nd order to 6th order which will lead to an exacter approximation of the data points. Since the measurements done in the flume are not very accurate these higher order trend lines will not lead to a better prediction of the flow velocities under the barge. Also comparison of the different trend line equations is easier for a second order equation since less coefficients have to be compared (in case of a second order equation only three; \( ax^2+bx+c \)).
Figure 3-21: Dimensionless flow velocity profiles for Location I (top), Location II (middle) and Location III (bottom). Plotting the equation of the trend lines is done in the order of increasing flow velocities. The upper equation represents thus the equation of the trend line for a flow velocity of 0.1 m/s and the lowest equation represents the trend line for a flow velocity of 0.4 m/s.
Two comparisons can now be made; at first the changes in dimensionless flow velocities between different locations and secondly the changes in dimensionless flow velocities at one location for changing flow velocity. Table 3-11 and Table 3-12 show these comparisons in trend line equations and values of the dimensionless flow velocities.

For each flow velocity, the change in average dimensionless flow velocity is listed below in Table 3-8. It shows clearly that the average dimensionless flow velocity decreases with an increasing distance from the bow of the barge.

<table>
<thead>
<tr>
<th>Location</th>
<th>difference</th>
<th>Location</th>
<th>difference</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1.15</td>
<td>II</td>
<td>1.05</td>
<td>III</td>
</tr>
<tr>
<td>0.1 [m/s]</td>
<td>1.15</td>
<td>0.10</td>
<td>0.01</td>
<td>0.09</td>
</tr>
<tr>
<td>0.2 [m/s]</td>
<td>1.22</td>
<td>0.30</td>
<td>0.92</td>
<td>0.46</td>
</tr>
<tr>
<td>0.3 [m/s]</td>
<td>1.44</td>
<td>0.39</td>
<td>1.05</td>
<td>0.81</td>
</tr>
<tr>
<td>0.4 [m/s]</td>
<td>1.32</td>
<td>0.18</td>
<td>1.14</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Table 3-8  differences in dimensionless flow velocity between locations

The amount of decrease in dimensionless flow velocity is not constant for each change in location of flow velocity. For an average flow velocity of 0.1m/s the decrease between locations I and II is for instance significantly bigger than the decrease between the same locations with an average flow velocity of 0.4 m/s. Also the reliability of the calculated average dimensionless flow velocities at location III is doubtful, since the measurements done with the EMS probe located at Location III are not reliable too.

Since Martin and Maynord give a relation of the maximum flow velocity occurring under a ship’s bow, it is good to have an overview of the maximum occurring dimensionless flow velocities. These are derived using the maximum values from Table 3-4, Table 3-5 and Table 3-6.

<table>
<thead>
<tr>
<th>Location</th>
<th>Location</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>II</td>
<td>III</td>
</tr>
<tr>
<td>0.1 [m/s]</td>
<td>1.22</td>
<td>0.27</td>
</tr>
<tr>
<td>0.2 [m/s]</td>
<td>1.31</td>
<td>1.02</td>
</tr>
<tr>
<td>0.3 [m/s]</td>
<td>1.52</td>
<td>1.19</td>
</tr>
<tr>
<td>0.4 [m/s]</td>
<td>1.37</td>
<td>1.21</td>
</tr>
</tbody>
</table>

Table 3-9  maximum dimensionless flow velocities

Under the bow of the barge the maximum flow velocities measured lay between 1.22-1.52 times the average flow velocity in front of the barge.

<table>
<thead>
<tr>
<th>Location</th>
<th>Location</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>II</td>
<td>III</td>
</tr>
<tr>
<td>(see table Table 3-7)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.1 [m/s]</td>
<td>0.122</td>
<td>0.014+0.1=0.114</td>
</tr>
<tr>
<td>0.2 [m/s]</td>
<td>0.262</td>
<td>0.031+0.2=0.231</td>
</tr>
<tr>
<td>0.3 [m/s]</td>
<td>0.457</td>
<td>0.048+0.3=0.348</td>
</tr>
<tr>
<td>0.4 [m/s]</td>
<td>0.548</td>
<td>0.064+0.4=0.464</td>
</tr>
</tbody>
</table>

Table 3-10  maximum flow velocities compared to Schijf

Table 3-10 shows the maximum dimensionless flow velocities and the same dimensionless flow velocities compared with the return flow as calculated using Schijf. Now a comparable relation as the WL Delft Hydraulics equation is gained. The coefficient derived from the measurements (1.07-1.31) is lower than the ones used by WL Delft Hydraulics (1.5-2.0). This can be due to measuring errors, but can also be due to different dimensions of the barge and flume used for the experiments. According to the measurements
the maximum increase in flow velocity under the bow of the barge is thus 1.07-1.31 times the return flow velocity as calculated using Schijf.

\[ U_{rb} = 1.07 - 1.31 \cdot U_r \]  \[3-5\]

It is safe to use these maximum flow velocities, since Martin and Maynord also calculate the maximum occurring flow velocities. The peak in Figure 1-3 is described by the formula of Martin and Maynord. Also the WL Delft Hydraulics and the modified Martin and Maynord equations calculate the maximum return current velocity (Stolker and Verheij, 2006).

Looking at the equations of the trend lines as displayed in Table 3-12 shows different factors representing the line through the calculated dimensionless flow velocities. Important to realize is that even though the function description of a polynomial can be different from each other, they still may be valid within the same range. Most practical for describing the velocity profile under the barge would be one parabolic function with a constant ‘a’ and ‘b’ for each flow velocity and only a, with the flow velocity varying, value of ‘c’ which determines the relative height of the parabolic function.
<table>
<thead>
<tr>
<th>Location I</th>
<th>a (x^2) + b (x) + c</th>
<th>R^2</th>
<th>average</th>
<th>median</th>
<th>maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 [m/s]</td>
<td>-75.7</td>
<td>9.29</td>
<td>0.94</td>
<td>0.996</td>
<td>1.15</td>
</tr>
<tr>
<td>0.2 [m/s]</td>
<td>-56.6</td>
<td>7.81</td>
<td>0.98</td>
<td>0.983</td>
<td>1.22</td>
</tr>
<tr>
<td>0.3 [m/s]</td>
<td>-56.5</td>
<td>6.28</td>
<td>1.37</td>
<td>0.853</td>
<td>1.44</td>
</tr>
<tr>
<td>0.4 [m/s]</td>
<td>-40.1</td>
<td>4.92</td>
<td>1.24</td>
<td>0.901</td>
<td>1.32</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location II</th>
<th>a (x^2) + b (x) + c</th>
<th>R^2</th>
<th>average</th>
<th>median</th>
<th>maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 [m/s]</td>
<td>-76.9</td>
<td>9.67</td>
<td>-0.17</td>
<td>0.963</td>
<td>0.10</td>
</tr>
<tr>
<td>0.2 [m/s]</td>
<td>-78.0</td>
<td>11.74</td>
<td>0.57</td>
<td>0.985</td>
<td>0.92</td>
</tr>
<tr>
<td>0.3 [m/s]</td>
<td>-109.0</td>
<td>17.86</td>
<td>0.44</td>
<td>0.859</td>
<td>1.05</td>
</tr>
<tr>
<td>0.4 [m/s]</td>
<td>-64.0</td>
<td>8.33</td>
<td>0.96</td>
<td>0.906</td>
<td>1.14</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location III</th>
<th>a (x^2) + b (x) + c</th>
<th>R^2</th>
<th>average</th>
<th>median</th>
<th>maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 [m/s]</td>
<td>11.0</td>
<td>-2.37</td>
<td>0.19</td>
<td>0.971</td>
<td>0.09</td>
</tr>
<tr>
<td>0.2 [m/s]</td>
<td>5.4</td>
<td>0.27</td>
<td>0.63</td>
<td>0.910</td>
<td>0.68</td>
</tr>
<tr>
<td>0.3 [m/s]</td>
<td>-1.1</td>
<td>2.00</td>
<td>0.10</td>
<td>0.975</td>
<td>0.24</td>
</tr>
<tr>
<td>0.4 [m/s]</td>
<td>9.7</td>
<td>-0.92</td>
<td>0.30</td>
<td>0.770</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Table 3-11 dimensionless velocities sorted by location

<table>
<thead>
<tr>
<th>Location I</th>
<th>a (x^2) + b (x) + c</th>
<th>R^2</th>
<th>average</th>
<th>median</th>
<th>maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 [m/s]</td>
<td>-75.7</td>
<td>9.29</td>
<td>0.94</td>
<td>0.996</td>
<td>1.15</td>
</tr>
<tr>
<td>Location II</td>
<td>-76.9</td>
<td>9.67</td>
<td>-0.17</td>
<td>0.963</td>
<td>0.10</td>
</tr>
<tr>
<td>Location III</td>
<td>11.0</td>
<td>-2.37</td>
<td>0.19</td>
<td>0.971</td>
<td>0.09</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location I</th>
<th>a (x^2) + b (x) + c</th>
<th>R^2</th>
<th>average</th>
<th>median</th>
<th>maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2 [m/s]</td>
<td>-56.6</td>
<td>7.81</td>
<td>1.04</td>
<td>0.983</td>
<td>1.22</td>
</tr>
<tr>
<td>Location II</td>
<td>-78.0</td>
<td>11.74</td>
<td>0.57</td>
<td>0.985</td>
<td>0.92</td>
</tr>
<tr>
<td>Location III</td>
<td>5.4</td>
<td>0.27</td>
<td>0.63</td>
<td>0.975</td>
<td>0.68</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location I</th>
<th>a (x^2) + b (x) + c</th>
<th>R^2</th>
<th>average</th>
<th>median</th>
<th>maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3 [m/s]</td>
<td>-56.5</td>
<td>6.28</td>
<td>1.37</td>
<td>0.853</td>
<td>1.44</td>
</tr>
<tr>
<td>Location II</td>
<td>-109.0</td>
<td>17.86</td>
<td>0.44</td>
<td>0.859</td>
<td>1.05</td>
</tr>
<tr>
<td>Location III</td>
<td>-1.1</td>
<td>2.00</td>
<td>0.10</td>
<td>0.975</td>
<td>0.24</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location I</th>
<th>a (x^2) + b (x) + c</th>
<th>R^2</th>
<th>average</th>
<th>median</th>
<th>maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4 [m/s]</td>
<td>-40.1</td>
<td>4.92</td>
<td>1.24</td>
<td>0.901</td>
<td>1.32</td>
</tr>
<tr>
<td>Location II</td>
<td>-64.0</td>
<td>8.33</td>
<td>0.96</td>
<td>0.906</td>
<td>1.14</td>
</tr>
<tr>
<td>Location III</td>
<td>9.7</td>
<td>0.92</td>
<td>0.30</td>
<td>0.770</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Table 3-12 dimensionless velocities sorted by mean flow velocity at location 0
Averaging the found coefficients for each location gives coefficients for a parabolic equation which doesn’t fit all the data points but at least gives an indication of the flow velocity development as a function of the distance for the barge’s keel. The ‘c’-value has to be chosen in a way that the top of the parabolic function is equal to the maximum dimensionless flow velocity for that specific situation.

For location I this results in a parabolic equation of:

\[ u_I(x) = -57.2x^2 + 7.1x + c \]

<table>
<thead>
<tr>
<th>( U_0 )</th>
<th>( u_{max} )</th>
<th>( c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1m/s</td>
<td>1.22</td>
<td>1.00</td>
</tr>
<tr>
<td>0.2m/s</td>
<td>1.31</td>
<td>1.09</td>
</tr>
<tr>
<td>0.3m/s</td>
<td>1.52</td>
<td>1.30</td>
</tr>
<tr>
<td>0.4m/s</td>
<td>1.37</td>
<td>1.15</td>
</tr>
</tbody>
</table>

For location II:

\[ u_{II}(x) = -82.0x^2 + 12.0x + c \]

<table>
<thead>
<tr>
<th>( U_0 )</th>
<th>( u_{max} )</th>
<th>( c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1m/s</td>
<td>0.13</td>
<td>-0.31</td>
</tr>
<tr>
<td>0.2m/s</td>
<td>1.02</td>
<td>0.58</td>
</tr>
<tr>
<td>0.3m/s</td>
<td>1.19</td>
<td>0.75</td>
</tr>
<tr>
<td>0.4m/s</td>
<td>1.17</td>
<td>0.73</td>
</tr>
</tbody>
</table>

And for location III:

\[ u_{III}(x) = 6.3x^2 + 0.3x + c \]

These results are very different from each other and are not relevant to use for further calculations. To gain more information about the total data set, all values were plotted in a scattered grid figure, resulting in \( R^2 \) values of 0.075, even when the odd data point were deleted out of the data set (only dimensionless flow velocities larger than 1 were taken into account) the reliability of the trendline was minimal.

Since formulas like Martin and Maynord and the modified Martin and Maynord equation use the maximum flow velocities, the data of Table 3-9 will be used for further calculations.
A final attempt is made to gain insight in a possible relation between the measured flow velocities. Figure 3-22 shows the average and median values of the dimensionless flow velocities from location I and II. The third location is intentionally left out since these measurements are unreliable and will disturb the possible trend between data points. Also the average measurements at $U_0=0.1\text{m/s}$ are left out, since they are significantly lower than the other measurements. The median values of these measurements is still used since they will contain more usable values. Although the fit is not reliable ($R^2=0.1-0.13$) the data points are not extremely scattered. The derived relation of the trendline reads (when rewrited to used parameters):

$$\frac{U}{U_0} = 0.5 \cdot U_0 + 1$$

With $U$: Measured velocity [m/s], $U_0$: Flow velocity in undisturbed situation [m/s], 0.5: Coefficient following from trendline [s/m], 1: Constant following from trendline [-]

For small values of $U_0$ this equation is usable, but with increasing average flow velocities ($U_0$) the results seem unreliable. When on the other hand the same coordinate system as Schijf is used, thus adding the sailing speed to the average flow velocity (when opposite directed) the calculated velocities are increasing rapidly. With a sailing speed of 2.56m/s and an average flow velocity of 1.1m/s, the calculated flow velocity becomes $2.56 \times (0.5 \times 2.56 + 1) = 10.4m/s$, this is nearly 4 times as large as values gained from the modified Martin and Maynord equation (2.84m/s). Therefore the reliability of this equation is doubtful for the flow velocities occurring in prototype conditions. The equation is thus only valid for $0.1m/s \leq U_0 \leq 0.4m/s$. 

Figure 3-22 Average and median dimensionless flow velocities plotted as function of flow velocity $U_0$
3.5.2 Run with small under keel clearances

Six dunes were build in the flume at Deltares, and considering the available time for the experiments, only a few runs over these dunes could be made. Therefore it was chosen to pull the barge over the dunes with three different speeds, 0.1 m/s, 0.3 m/s and 0.5 m/s. The first run was done with a keel clearance of 0.15m after which the dunes where build again for the next experiment.

<table>
<thead>
<tr>
<th>Location</th>
<th>Vs 0.1m/s</th>
<th>Vs 0.3m/s</th>
<th>Vs 0.5m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>-0.87</td>
<td>-0.47</td>
<td>-0.37</td>
</tr>
<tr>
<td>II</td>
<td>-0.11</td>
<td>0.08</td>
<td>0.25</td>
</tr>
<tr>
<td>III</td>
<td>-1.16</td>
<td>-1.17</td>
<td>-1.18</td>
</tr>
</tbody>
</table>

Table 3-13 Average flow velocities during run

Flow velocities in the flume itself were set at maximum (0.43m/s) the barge was towed against the flow direction. From this information the flow velocities in Table 3-13 seem unreliable. Looking at location I the measured flow velocity decreases while the speed of the barge (and with that the speed of the EMS probe) increases. Therefore the measured flow velocities are not to be trusted and will not further be used.

On the next two pages, results of the bed-level measurements are shown, measurements prior to- and after the experiments are plotted as a 3D surface. Positive X-directions are pointed against the flow direction, positive Y-directions are starboard directed and positive Z-directions are upward directed.
Figure 3-23 Profile before experiment (15cm UKC)

Figure 3-24 Profile after experiment (15cm UKC)
Above figures show no significant bed level changes at all. A possible reason for this may be that the sediment size is too large (earlier mentioned by Hein Bots), another reason may be that the flow velocities under the barge were too small to induce sediment transport. Figure 3-27 shows the differences in bed level gained by subtracting the bed levels of the two measurements from each other.
One can clearly distinguish the erosion at the dune tops, here indicated by small steep gullies since erosion is a negative bed level change. Integrating these values over the length and width of the flume gives a total erosion of $0.0827 m^3$, sedimentation of $0.0788 m^3$ and thus $0.0039 m^3$ transported out of the measuring area.
Six dunes where measured during the experiments, containing a total volume of 0.49 m$^3$ of sand. An erosion of 0.0827 m$^3$ means that 17% of the dunes are eroded and elsewhere deposited. Note that this is according to the measurements! In both Figure 3-28 and Figure 3-27 disturbances are visible, which are caused by measuring differences between the measurement before the experiment and after the experiment. When measuring a cross section of the dune near the top, a measuring error of 1 cm to the left or right between two measurements will cause a big difference in bed levels. When for instance the first measurement is done exactly at the top of the dune and the second one a bit to the right the ‘measured’ difference will be significant while maybe the dune top didn’t change at all. Thus the 17% of erosion will be less when including a factor for measurement errors. Bots (2011) stated in his report that due to different time of measurements already a measurement error of 0.02 m occurred. This became clear when measurements done in the morning were compared to measurements done in the evening. The amount of sediment transported due to the passage of the barge is therefore not reliable and will not be used for further considerations.

During the experiments also the sinkage of the bow of the barge was measured. This is interesting since a sinkage of the bow means a smaller keel clearance and thus has its influence on the flow velocity. A negative value means opposite directed to the flow direction, thus in the same direction of the moving barge.

<table>
<thead>
<tr>
<th>$\text{Measured}$</th>
<th>$dT$</th>
<th>$\text{Ankidunov}$</th>
<th>$dT$</th>
<th>$\text{Schijf}$</th>
<th>$dT$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 m/s</td>
<td>3.57</td>
<td>-</td>
<td>0.16</td>
<td>-</td>
<td>0.40</td>
</tr>
<tr>
<td>0.3 m/s</td>
<td>3.98</td>
<td>0.41</td>
<td>1.09</td>
<td>0.93</td>
<td>0.88</td>
</tr>
<tr>
<td>0.5 m/s</td>
<td>4.49</td>
<td>0.51</td>
<td>2.67</td>
<td>1.58</td>
<td>1.64</td>
</tr>
</tbody>
</table>

Table 3-14 Measured and calculated sinkage ($T=0.26$m)

And for the run with a draught of 31 cm, shows the calculated and measured values.

<table>
<thead>
<tr>
<th>$\text{Measured}$</th>
<th>$dT$</th>
<th>$\text{Ankidunov}$</th>
<th>$dT$</th>
<th>$\text{Schijf}$</th>
<th>$dT$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 m/s</td>
<td>6.24</td>
<td>-</td>
<td>0.22</td>
<td>-</td>
<td>0.52</td>
</tr>
<tr>
<td>0.3 m/s</td>
<td>7.39</td>
<td>1.15</td>
<td>1.41</td>
<td>1.19</td>
<td>1.05</td>
</tr>
<tr>
<td>0.5 m/s</td>
<td>8.98</td>
<td>1.59</td>
<td>3.65</td>
<td>2.24</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Table 3-15 Measured and calculated sinkage ($T=0.31$m)

The measured values of the sinkage are compared with calculated values using Ankidunov. It is important to realize that the measured values are obtained by a wire sensor, which measures the amount of wire that is pulled out of the box. The measured value itself doesn’t mean much, but the differences in measurements give an indication of the sinkage of the ship. To be able to compare these values with the calculated ones, the increase in draught (due to the sinkage) is compared. For a draught of 26 cm the measured values differ with a factor 2-3 from the calculated ones. With an increased draught of 31 cm the measured values are more comparable with Ankidunov, the difference now is about 0-1.4. Ankidunov seems to overestimate the sinkage of the barge, for a large ship this will be less, since a larger ship will react less to short distortions like the non steady acceleration of the measuring chart.
4 Model Development

4.1 Analytical model

An analytical model, able to predict flow velocities under the bow of a ship, is made. Starting with relatively easy-to-produce volume balances will give insight in the most relevant parameters considering flow velocities and sediment transport.

4.1.1 Objective of analytical model

Objective of this theoretical model is to produce a simple relation which is able to predict the amount of sediment transport due to the passage of a ship.

4.1.2 Model description and Configurations

For this analytical model situations in increasing detail will be described. Just as used in the sections about the physical model tests, four locations are important considering the flow under the barge (or a pushed barge combination). The first location, indicated with 0 is the location in the flume (or river) where the flow can be assumed to be uniform and undisturbed. Secondly, location I is the cross section right below the bow of the barge (or ship). Location II is defined as the cross section amidships and finally location III is located near the ship’s stern.

First approximation:

Assume frictionless uniform steady flow. Viscosity and friction due to roughness of materials are neglected. No energy losses are accounted for, this situation is comparable with potential flow boundary conditions.

Looking at specific discharges in the longitudinal direction of the barge, a change has to take place between location 0 and location I.

Important to keep in mind is that the volume of water displaced by the ship per unit of time equals the ship’s speed times the draught of the ship (hereby neglecting) the bow-wave

The volume balance between 0 and I reads:

\[ Q_I = Q_0 \]  \hspace{1cm} [4-1]

With:

\[ (q_I \cdot B_s - V_s \cdot B_s \cdot T) = B_s \cdot U_0 \cdot h_0 \]  \hspace{1cm} [4-2]

Dividing by \( B_s \) [m] and only leaving \( q_I \) [m²/s] on the left hand side gives:

\[ q_I = U_0 \cdot h_0 + V_s \cdot T \]  \hspace{1cm} [4-3]

Defining the return flow under the ship’s bow as:

\[ U_{rb} = \frac{q_I}{UKC} \]  \hspace{1cm} [4-4]
Results after subtraction in:

\[
U_{rb} = \frac{U_0 \cdot h_0 + V_s \cdot T}{UKC}
\]  \hspace{1cm} \text{[4-5]}

In this first approximation, \(U_{rb}\) is equal to the flow velocity amidships and at the stern since no storage or loss of volume occurs. Looking at a more realistic description of the hydrodynamic situation one should consider boundary layer development and possible development of a vena contracta due to release of the laminar boundary layer from the barge's keel. One should notice that the calculated \(U_{rb}\) represents the averaged flow velocity, in reality this velocity is not the same over the whole flow carrying height.

Comparing this simplification with the measurements done in the flume is done in Table 4-1.

<table>
<thead>
<tr>
<th>(U_0)</th>
<th>(U_{rb})</th>
<th>(U_{\text{measured}})</th>
<th>(U_{rb}/U_0)</th>
<th>(U_{\text{measured}}/U_0)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.23</td>
<td>0.11</td>
<td>2.30</td>
<td>1.10</td>
</tr>
<tr>
<td>0.2</td>
<td>0.46</td>
<td>0.26</td>
<td>2.30</td>
<td>1.30</td>
</tr>
<tr>
<td>0.3</td>
<td>0.69</td>
<td>0.46</td>
<td>2.30</td>
<td>1.53</td>
</tr>
<tr>
<td>0.4</td>
<td>0.92</td>
<td>0.55</td>
<td>2.30</td>
<td>1.38</td>
</tr>
</tbody>
</table>

Table 4-1 Measured values compared with calculated values

Clearly this first approximation overestimates the amount of water flowing under the barge's bow. In chapter 3.5.1 is shown that according to the measurements only about 50% of the water is flowing under the bow so the concept of this first approximation has to be fine-tuned by adding more details about the hydrodynamic circumstances and related properties.

As Figure 3-20 shows, the specific discharge decreases when the water flows underneath the ship. For simplicity this is blamed on the fanning out effect, which seems to decreases for a little with higher flow velocities. In general one can state that the specific discharge \(q\) under the bow is (average of location I) 55% of the original discharge and amidships only about 31% (neglecting the negative discharge) is left. When using the ‘modified’ data of the EMS at the stern, an average of 11% of the specific discharge is left.

Now again focusing on the bow of the barge, a decrease of 45% (average value of percentages as shown in Figure 3-20) in specific discharge doesn’t mean that the flow velocity also decreases with 45%, since the specific discharge \(q\) is defined as the product of the water depth and the flow velocity. The water depth (or under keel clearance in case of looking at flow under the barge) decreases since the barge is impermeable and the water has to flow under the barge.

Now using this information, the following can be stated:

\[
U_{r,b} = \frac{0.55 \cdot (U_0 \cdot h_0) + V_s \cdot T}{UKC}
\]  \hspace{1cm} \text{[4-6]}
Now calculating the flow velocities again gives:

<table>
<thead>
<tr>
<th>$U_0$ [m/s]</th>
<th>$U_{rb}$ [m/s]</th>
<th>$U_{measured}$ [m/s]</th>
<th>$U_{rb}/U_0$</th>
<th>$U_{measured}/U_0$</th>
<th>Relative error</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.127</td>
<td>0.11</td>
<td>1.27</td>
<td>1.10</td>
<td>15%</td>
</tr>
<tr>
<td>0.2</td>
<td>0.253</td>
<td>0.26</td>
<td>1.27</td>
<td>1.30</td>
<td>2%</td>
</tr>
<tr>
<td>0.3</td>
<td>0.380</td>
<td>0.46</td>
<td>1.27</td>
<td>1.53</td>
<td>17%</td>
</tr>
<tr>
<td>0.4</td>
<td>0.506</td>
<td>0.55</td>
<td>1.27</td>
<td>1.38</td>
<td>8%</td>
</tr>
</tbody>
</table>

Table 4-2 Measured values compared with calculated values

Although the calculated values are not exactly the same as the measured one, one gets a pretty accurate approximation of the average flow velocities occurring underneath the barge. Important to mention is that the ratio between the wet cross section of the ship and the channel cross section $A_s/A_c$ is equal to 0.11, which is a common value for natural waterways according to Verheij (2008). For most waterways values of 0.1-0.3 are common. With a maximum error of 17% at a velocity of 0.3m/s this approximation is accurate enough for further calculations.

<table>
<thead>
<tr>
<th>$U_0$ [m/s]</th>
<th>$U_r$ [m/s]</th>
<th>$U_r+U_0$ [m/s]</th>
<th>$(U_r+U_0)/U_0$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.014</td>
<td>0.114</td>
<td>1.14</td>
</tr>
<tr>
<td>0.2</td>
<td>0.031</td>
<td>0.231</td>
<td>1.155</td>
</tr>
<tr>
<td>0.3</td>
<td>0.048</td>
<td>0.348</td>
<td>1.16</td>
</tr>
<tr>
<td>0.4</td>
<td>0.064</td>
<td>0.464</td>
<td>1.16</td>
</tr>
</tbody>
</table>

Table 4-3 Return flow according to Schijf

For the configuration as used for the flume experiments, it follows that the return flow as calculated using Schijf is equal to $1.16*U_0$ (see also Table 4-3). This would mean that the values calculated from the analytical model would be equal to $1.27*U_0=1.27*(1.16*U_r)$ or $1.46*U_r$. This is comparable with the lower region of the WL Delft Hydraulics equation.

For further calculations this relation will be used, the return flow velocity will thus be:

$$U_{rb} = 1.46 \cdot U_r$$ [4-7]

In chapter 5, Synthesis, these values will be used for calculating sediment transport and compared with other formulae.
4.2 Computational model

DelKelv is a potential flow model, made by Dr.ir. H.J. de Koning Gans; assistant professor at the faculty of Maritime engineering. This model is often used by maritime technology students during a master course considering numerical simulation using panel methods and potential flow theory. Also de Koning Gans uses his model for research purposes. Examples of these research projects are: “Squat of ships sailing in a non centered track in a canal” (2011) and “Squat effects of very large container ships sailing in a harbor environment” (2005), for both articles DelKelv is used.

4.2.1 Objective of computational model tests

Three objectives are stated considering the usage of DelKelv:

- Reproduce the experiments done in the flume at Deltares to allow comparisons between measurements and model results.
- Simulate a pushed barge combination to gain insight in the flow and pressure differences induced by the flow around the ship.
- Be able to show the influence of the most relevant parameters. Most relevant parameters are the ones influencing the flow velocities under the ship the most.

Main advantage of using a potential flow model is that one is able to gain insight in the flow velocities and pressures in certain conditions. Input parameters can be changed and nearly instant results about the changes in flow pattern occurring due to the new conditions will be available.

The flow is assumed to be inviscid, which means that friction and processes related to friction are neglected. DelKelv is able to calculate the properties of the occurring Kelvin waves, but for this master thesis the water level will be fixed. According to H.J. de Koning Gans the influence of Kelvin waves on the bottom is negligible and therefore not needed to model. Influence of Kelvin waves on the bottom decreases with a factor $e^{-kz}$ (related to the water level and distance to the bottom) and with a depth of about 5-6m the influence of these waves will be barely noticeable.

4.2.2 Approach and Configurations

Some adjustments had to be made to the way the input was processed by DelKelv since the water level was kept fixed. For every run an input file has to be made, containing coordinates of the wet surface of the ship, the bottom and also containing some settings. These settings contained definition of directions, names of files and more. An impression of the defined barge and the needed coordinates is given in Figure 4-1. The visible part is the wet surface of the barge, so only the part of the barge that is located under water is defined since the water level is kept at a fixed level.
Model runs are divided in two parts, each containing different configurations. An overview is shown in Table 4-4.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Dune at bow</th>
<th>Flat bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barge (flume experiments)</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>15cm UKC</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>20cm UKC</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Pushed barge combination (prototype)</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>22.6%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4-4 Configurations for DelKelv runs

Under keel clearances of 15cm and 20cm for the barge are chosen to simulate the experiments done in the flume at Deltares. Prototype keel clearance is chosen to simulate a push barge combination sailing in the river Waal. Since the model is linear interpretable (according to H.J. de Koning Gans) these values are good enough for qualitative analysis. All simulations are done with a flow velocity of 1m/s and no velocity of the ship itself.

4.2.3 Results

The results are divided in a part considering the simulation of the barge and a part considering the simulation of flume experiments with the barge and a part considering the simulations with the prototype push barge combinations.

4.2.3.1 Barge (flume experiments)

On the next pages results of the simulation runs are shown. Although the relation between velocities and pressures are easy to derive from each other (Conservation of energy), both are displayed in the figures.

Graphs of velocities and pressures and a visual impression are shown in Figure 4-5, Figure 4-7 and Figure 4-9. For both keel clearances the same changes in velocities and pressures occur along the barge. At first, flow velocities are nearly 1m/s. When taking a closer look at the bow of the barge, one can see the flow velocity decrease for a bit until the passage of the bow. Right of the bow, an immediate increase in flow velocity occurs with a factor 1.2-1.3 of the original flow velocity. Amidships of the barge, the flow velocity
decreases as far as 1.1-1.2 times the original flow velocity of 1 m/s. Finally, close to the stern of the barge, the flow velocity increases again to 1.2-1.3 times the original flow velocity and after passage of the barge decreases again and then converts back to about 1 m/s. Due to the linearity of the model these factors are practical for usage with different situations. In a situation with half the flow velocity as now (comparable with the flume experiments (U=0.4 m/s), the extreme values of velocities under the bow of the barge will still be 1.2-1.3 times the velocity.

The small decrease of the flow velocity in front of- and at the stern of the barge is in the prototype situation caused by the wave at the bow and stern. Since the water level is kept fixed during these simulations, no wave can occur. Probably this drop in flow velocity is caused by the diffusion of the flow, some of the water will flow in another direction due to the way a potential flow model works. A fixed object like the barge is defined as a so-called source point, providing a boundary condition so that the water flows past this point instead of right through. Another plausible reason is the slope of the water level drop; potential flow works with equivalent-potential lines, lines where the potential is the same. Since the water depth is decreasing in a short distance from water level until it the keel clearance, a small slope will occur. This slope had thus no physical meaning.

Comparing the calculated flow velocities with the measured flow velocities is done in Table 4-5. Shown are the dimensionless flow velocities calculated by dividing the measured or simulated flow velocity by the average flow velocity in the flume. During the simulations the flow velocity was set to 1 m/s, the simulated values are thus divided by 1 m/s to get the dimensionless flow velocity as displayed. The dimensionless flow velocities gained from measurements are displayed in the four columns on the right side of the table. Again the measured values are divided by the average flow velocity (respectively 0.1, 0.2, 0.3 and 0.4 m/s).

<table>
<thead>
<tr>
<th>Location</th>
<th>Distance to bow of barge [m]</th>
<th>DelKelv</th>
<th>Measurements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>U0=1m/s</td>
<td>U0=0.1m/s</td>
</tr>
<tr>
<td></td>
<td></td>
<td>U0=0.2m/s</td>
<td>U0=0.3m/s</td>
</tr>
<tr>
<td></td>
<td></td>
<td>U0=0.4m/s</td>
<td>U0=1m/s</td>
</tr>
<tr>
<td>0</td>
<td>1.50</td>
<td>0.96</td>
<td>1.00</td>
</tr>
<tr>
<td>I</td>
<td>-0.46</td>
<td>1.19</td>
<td>1.15</td>
</tr>
<tr>
<td>II</td>
<td>-0.91</td>
<td>1.14</td>
<td>0.10</td>
</tr>
<tr>
<td>III</td>
<td>-1.37</td>
<td>1.18</td>
<td>0.09</td>
</tr>
</tbody>
</table>

Table 4-5 Comparison of simulated and calculated dimensionless flow velocities

At Location 0, the undisturbed situation in front of the barge, the dimensionless flow velocity is expected to be equal to 1 since the flow velocity should by definition be equal to the measured flow velocity. Data from DelKelv shows a dimensionless flow velocity of 0.96 (1.50 m in front of the bow of the barge), which could be the result of a numerical error or influence of the boundary on the simulated flow velocity. At Location I the dimensionless flow velocities of DelKelv are not very different from the dimensionless flow velocities as calculated from the measurements done in the flume. Since Delkelv doesn’t take friction into account one could expect the simulated dimensionless flow velocity to be higher than the ones as
calculated from the measurements in the flume. But this is not the case at this location, still the values are comparable and not very different from each other.

Location II shows comparable results with the data gained from the measurements, only the dimensionless flow velocity measured with an average flow velocity of 0.1m/s is very different from the simulated one. Dimensionless flow velocities measured with higher average flow velocities (0.2m/s-0.4m/s) are almost the same as the simulated ones. The differences are not very large, the dimensionless flow velocity seems to approach the simulated ones with increasing average flow velocity. This could be due to the already assumed decrease in fanning out effect (higher flow velocities, less fanning out) but could also be the result of measurement errors. The size of these errors is not known, so it is possible that the correlation of the increase in dimensionless flow velocity with increasing average flow velocity is just coincidence.

Location III shows as already expected no correlation at all with the simulated dimensionless flow velocities. This again proves the unreliability of the EMS probe mounted at Location III. No further comparisons will be made since the reliability proves to be insufficient for further usage.

From the simulations of DelKelv it follows that the program is able to predict flow velocities with a high reliability. Comparing the data from the simulations with the measurements at Location I and II show a large correlation with a relative small error. This data below is also shown in Figure 4-2.

DelKelv can thus be used to gain information about occurring flow velocities at different configurations. Data gained from these simulations has proven to be reliable, at least for the situation of the flume experiments. From the analytical model in chapter 4.1 it followed that the dimensionless flow velocity was equal to 1.27, which is also in range of the values found with DelKelv and measurements done.

![Figure 4-2 Comparison of DelKelv with measurements (flat bottom)]
Just as in section 3.5.1 the specific discharge is calculated for the three locations and displayed in Figure 4-4. The specific discharge at the bow of the barge is the same as measured during the experiments. 54% of the water is flowing under the center line of the barge, where the measurements were done. At locations II and III the specific discharge remains about 50% of the total discharge at location 0. This is probably due to the fact that friction is not taken into account; the fanning out effect will therefore be minimal in DelKelv results.

Flow velocities occurring in the run with a dune modeled in the bottom profile (see Figure 4-6) show a significantly higher peak velocity. The maximum occurring flow velocity is equal to 2.20*U₀ this is equal to 1.92*Uᵣ. Sailing over a shallow section will thus lead to much higher flow velocities. The value of 1.92*Uᵣ is still within the range as earlier defined by WL Delft Hydraulics.
Figure 4-5 Flow velocities at the (flat) bottom

Figure 4-6 Flow velocities at the bottom with dune

Page 86
Figure 4-7 Pressures at the (flat) bottom

Figure 4-8 Pressures at the bottom with dune
Figure 4-9 relative flow velocities for $T=15\text{cm}$, $U=0.1 \text{m/s}$, 0.2m/s, 0.3m/s, 0.4m/s
Figure 4-9 shows the changes in flow velocity starting with 0.1m/s and increasing with 0.1m/s until 0.4m/s. The legend and color bar are kept the same for each run so that visual comparison is possible. At the bow and at the stern increased flow velocities are clearly visible. Also at the dune an increase in flow velocities is clearly visible.

4.2.3.2 Prototype simulations

For the prototype simulation a four barge combination is designed. Unlike Figure 2-11 suggests, no ship is taken into account in the simulation runs. Barges, when fully loaded, have a draught up to 4m according to information from ThyssenKrupp-Veerhaven. The ship itself only has a draught of 2.5m which is significant less than the 4 meters from the barges. Therefore, keeping in mind that the ship is pushing the barges, the additional effect of the ship on the flow will not be very large.

Close to the bow of the barges a small drop in flow velocity is visible, but opposite of the situation with the modeled barge, now the drop is more significant. The flow velocity drops until approximately 0.7 times the original flow velocity ($U_0$), after which is increases until $1.49*U_0$. Amidships again the flow velocity decreases a bit but is still more than the original flow velocity. Close to the stern again the flow velocity increases until $1.4-1.5U_0$ and drops then until $0.7U_0$.

The maximum flow velocity occurring at the bow of the barges is equal to 1.49 times the average flow velocity at the undisturbed location in front of the barges. This is equal to $1.35*U_r$, with $U_r$ being the return flow velocity calculated using Schijf. From the measurements done with the barge in the flume it was concluded that the maximum flow velocity at location I is equal to 1.52 times the flow velocity. In Delkelv the input velocity in x-direction can act as a ship speed or a flow velocity or a combination of both. Due to the neglected friction (viscosity) the results will be the same when running a test with a ship speed and flow velocity and running a test with a flow velocity equal to the sum of both velocities of the first run.

Also important is the area of influence over which the flow velocities increase. From the measurements in the flume no information was gained about the area of influence, but with DelKelv this is become clearer. Looking at Figure 4-12 one can see that the maximum flow velocities only occur over about 0.8 times the width of the barges. Three cross sections are made to gain more information about the flow velocity distribution, one at the bow where the highest flow velocities occur, one amidships and one at the stern again at the highest flow velocities.
Figure 4-11 Prototype: Flow velocities at the (flat) bottom

Figure 4-12 Prototype: Visualized flow velocities at the (flat) bottom
The three cross sections are shown in Figure 4-13 where on the vertical axis the flow velocity is displayed and on the horizontal axis the distance to the centerline of the four barges is shown. Only one side from the centerline is shown in the graph, the other side is symmetric to the one shown below. Total width of the 2x2 combination is 22.80m, so in the graph the edge of the barge is at 11.40m indicated by the colored vertical line.

Amidships the flow velocity is maximal at the centerline of the barge and equal to 1.07 times the flow velocity $U_0$. This velocity decreases with increasing distance from the centerline of the barge and converges to the normal flow velocity $U_0$.

At the stern of the barges a peak just outside the centerline of the barges is visible; this maximum value is equal to 1.435m/s. At the edge of the barge, at 11.40m the flow velocity is still 1.32 times as large as $U_0$. At 30m distance from the centerline of the barge the flow velocity is nearly the same as the flow velocity amidships but now it becomes smaller than the velocity amidships. Also the velocity at the stern of the barges converges but is for a distance larger then 30m smaller than the amidships velocities.

The flow velocities at the bow are the largest, the maximum velocity is 1.49m/s as already mentioned earlier in this paragraph. The velocity decreases and is nearly equal to the velocity at the stern close to the side of the barge. Just as the flow velocity at the stern, the velocity of the bow decreases and becomes smaller than the velocity amidships at a distance of 32m from the centerline.

Barge have a large influence on the flow velocities next to the ship, but these flow velocities decrease strongly at a larger distance from the centerline of the barges. At a distance of $1*B_s (=22.80m)$ from the center line the velocities are less than 10% larger than $U_0$. 

![Figure 4-13 Flow velocity at bottom related to the distance from the centerline of barges](image-url)
Although the area over which flow velocities increase is much larger than just the width of the barge combination, the flow velocities are not constant.

The increased flow velocities are therefore divided into two sections. The first section is located under the barge combination, flow velocities are assumed to be equal to $1.43*U_0$, this is an average value of the flow velocities as shown in Figure 4-13. The second section is defined as the distance from the side of the barges until the distance of $1*B_s$ from the centerline is reached. This is thus located from $11.40m<x<22.80m$. Flow velocities are assumed to be equal to $1.18*U_0$ this is also gained from averaging dimensionless flow velocities over this section.

![Figure 4-14 Distribution of flow velocities for the prototype simulation](image)

Figure 4-14 shows this distribution in a more visual way. The orange part is assumed to contain flow velocities of $1.18*U_0$ (or $1.07*U_r$), the red part under the barges is considered to contain flow velocities of $1.43*U_0$ (or $1.3*U_r$).

For the calculation of sediment transport under the push barge combination, this should be taken into account. Now something is known about the distribution of flow velocities under the push barge combination. The flow velocity under the push barge combination is assumed to be equal to $1.3*U_r$, flow velocities occurring next to the barge combination (with a maximum distance of 22.80m from the centerline of the barges) is assumed to be equal to $1.07*U_r$. 
5 Synthesis

Flow velocities are measured, modeled and calculated using different techniques. Comparing the results is done in the first part of this synthesis. The amount of sediment transport related to these flow velocities is elaborated in the second part of this chapter. Using Engelund-Hansen and simplified relations results in a sediment transport for three different cases. Two of these cases are already mentioned in Chapter 2 and are related to the operating and maximum sailing speed of a push barge combination. The third case takes only the natural flow velocity of the river Waal into account. This way the effect of a passing push barge combination can be compared with the situation without ships.

For a simplified case, but comparable with a real situation comparable with the push barge transport in the river Waal, two different cases will be further elaborated. Occurring flow velocities and erosion will be treated with the values found during this research. This way a clear overview will be created in which the different researched parameters will be summarized.

At first, the situation using data from Thyssen Krupp Veerhaven will be sketched. Thyssen Krupp Veerhaven is one of the largest companies in the Netherlands considering transport of bulk by push barge units. They use barge of the type Europa IIa, which are 76.50m long, 11.40m wide and can have a maximum draught of 4m. During a trip from the Rotterdam harbor to Germany, where the bulk is delivered, an average sailing speed of 2.56m/s (compared to the shoreline) is normal. With this speed the trip to Germany will take 26 hours. Most of the time, the push barge units will sail in a 2x2 combination, having a length of 153m (193m with push boat included) and a total width of 22.80m.

An average situation on the river Waal between Nijmegen and Zaltbommel is considered. In this case the water depth is equal to 5.1m and the ambient flow velocity $U_0$ is equal to 1.1m/s. The push barge combination sails with a draught of 4m, which gives a keel clearance of 1.1m (22% of the water depth). The roughness coefficient, defined by the Chezy parameter is in this case equal to $47\text{m}^{1/2}/\text{s}$.

For the calculations in this chapter, the following values will be used:

- $U$ = 1.1 [m/s]
- $V_s$ = varying [m/s]
- $d$ = 5.1 [m]
- $C$ = 47 [m$^{1/2}$/s]
- $D_{50}$ = 0.001 [m]
- $L_s$ = 153 [m]
- $B_s$ = 22.80 [m]
- $T$ = varying [m]

Earlier in this report it was already mentioned that especially the sailing speed, sailing direction and the ratio draught/water depth are important for occurring flow velocities under the ship's bow. In reality it is not possible to sail with extreme velocities, this velocity is bounded by the installed engine power and physical laws. Sailing with a higher speed than normal (2.56m/s) is often not economical, captains make a choice between sailing time and fuel consumption. With the installed power it was calculated in section 2.3.3 (Table 2-6) that with the installed engine power a maximum sailing speed of 3.07m/s is possible.
The combination of sailing with higher speed in shallow water results that squat becomes important and has to be quantified to prevent unsafe situations. For the calculated sailing speeds of 2.56\(\text{m/s}\) and 3.07\(\text{m/s}\) the amount of squat is equal to 0.42m and 0.69m. In this situation the keel clearance is still large enough to prevent grounding.

Two conditions are now defined, an average situation and a maximum feasible situation considering the installed engine power. These two situations are summarized below:

<table>
<thead>
<tr>
<th></th>
<th>(V_s) [m/s]</th>
<th>(T_0) [m]</th>
<th>squat [m]</th>
<th>(T) [m]</th>
<th>UKC [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normaal</td>
<td>2.56</td>
<td>4.0</td>
<td>0.42</td>
<td>4.42</td>
<td>0.68</td>
</tr>
<tr>
<td>Maximaal</td>
<td>3.07</td>
<td>4.0</td>
<td>0.60</td>
<td>4.60</td>
<td>0.40</td>
</tr>
</tbody>
</table>

For these configurations the maximum occurring flow velocities can now be calculated. Behind each method the table or equation used is mentioned.

- **Bedrijfssnelheid** (2.56\(\text{m/s}\))
  - Martin Maynord (Table 2-9): 1.11
  - Modified Martin Maynord (Table 2-9): 2.84
  - WL Delft Hydraulics (Table 2-9): 2.63-3.50
  - Analytical model (Equation [4-7]): 2.55
  - DelKelv (1.07-1.3*\(U_r\)): 1.87-2.28
  - Measurements (Equation [3-6]): 1.87-2.29

- **Maximumsnelheid** (3.07\(\text{m/s}\))
  - Minimum: 1.11
  - Average: 2.33
  - Maximum: 3.50

<table>
<thead>
<tr>
<th></th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.11</td>
<td>2.33</td>
<td>3.50</td>
</tr>
<tr>
<td></td>
<td>1.39</td>
<td>2.82</td>
<td>4.20</td>
</tr>
</tbody>
</table>

*Table 5-1 Calculated return velocities for the prototype situation*

For further calculations the modified Martin and Maynord equation will be used. This is done because this formula is already calibrated and validated on four different data sets and the already proven. WL Delft Hydraulics formula gives average values that are equal to the ones as calculated from the modified Martin and Maynord.

Results gained from the measurements in de flume are less reliable since no sailing speed was taken into account. Using the same method as Schijf makes the results still usable but the fanning out factor (55\%) will probably change due to movement of the barge. This results in a change in ratio between the measured flow velocity and calculated flow velocity.

Adjustment length and –time are defined in Table 2-12 and equal to 187m en 170s for the river Waal. For small adjustment lengths and –times compared to the calculation time sediment transport formula can give results that can be used as actual transport. Since the adjustment length and –time are both large compared with the length of the ship (153m) and passage time(60s) the sediment transport formula of Engelund-Hansen can only be used to give a transport capacity (Source: lecture notes River Engineering).

Engelund-Hansen will be used (see section 2.4.2, equation [2-47]) for estimating the transport capacity and increase in transport capacity due to the passage of a ship. The sediment transport is calculated but represents a transport capacity, still the results are used to gain more insight in possible erosion. For a sailing speed of 2.56\(\text{m/s}\) the passage time is 60s, for a sailing speed of 3.07\(\text{m/s}\) this is 50s. as the depth
averaged flow velocity the values of $U_{rb}$ calculated with the modified Martin and Maynord equations are used. Using a porosity of 35% (Kleinhans, 2002) gives a specific mass of the sediment on the river bed of $(1-0.35)*(2650\,\text{kg/m}^3-1000\,\text{kg/m}^3)=1073\,\text{kg/m}^3$.

\[
U_{rb}\,[\text{m/s}] \quad \tau_b\,[\text{N/m}^2] \quad s\,[\text{kg/s}] \quad s\,[\text{m}^3/\text{s}]
\]

<table>
<thead>
<tr>
<th>$U_{rb}$ [m/s]</th>
<th>$\tau_b$ [N/m²]</th>
<th>$s$ [kg/s]</th>
<th>$s$ [m³/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural flow (no ship)</td>
<td>1.10</td>
<td>5.37</td>
<td>0.000091</td>
</tr>
<tr>
<td>Sailing speed=2.56m/s</td>
<td>2.84</td>
<td>29.10</td>
<td>0.010434</td>
</tr>
<tr>
<td>Sailing speed=3.07m/s</td>
<td>3.23</td>
<td>41.86</td>
<td>0.019856</td>
</tr>
</tbody>
</table>

Table 5-2 sediment transport due to occurring flow velocities

For one square meter, the amount of erosion can be estimated for one passage of a push barge combination (table 5.3.).

\[
\begin{array}{|c|c|c|}
\hline
\text{Time [s]} & \text{Erosion [m]} \\
\hline
\text{Natural flow (no ship) in 1day} & 86,400 & 0.007324 \\
\text{Sailing speed=2.56m/s} & 60 & 0.000583 \\
\text{Sailing speed=3.07m/s} & 50 & 0.000925 \\
\hline
\end{array}
\]

Table 5-3 Erosion as a result of 1 passage

Sailing with an increased speed of 3.07m/s thus results in erosion nearly twice as large as during normal sailing speeds. This is a large increase, but the amount of erosion is very low. The erosion will be the highest under the barge combination itself, next to the barges some erosion is to be expected too (Figure 4-14).

The propeller of the push boat will induce extra erosion, which is not taken into account for this subject. Probably the amount of erosion will increase drastically when propeller effects are also taken into account. In Figure 1-3 a large increase in flow velocities just after passage of the stern (of course also induced by the ship’s propellers) is visible.

In one day an average of 4-5 push barge combinations will sail in upstream direction, the erosion can be multiplied with the amount of passages when they all sail over the same shallow section. Still the amount of erosion is low, when compared with the erosion occurring in one day due to the ambient flow velocity.

A very easy assumption for the location where the sediment will be deposited again would be to use the adjustment length, which is 187m. In practice this will be smaller, since for the adjustment length the sediment particle is considered to fall from the top of the water column to the bottom. Sediment picked up by the flow under the barge will rise for a maximum height equal to the keel clearance. Higher is not possible due to the hull of the ship. The adjustment length will be equal to $1.1\text{m/s}(5.1-1.1\text{m})/0.03\text{m/s}=40.3\text{m}$. This is just an estimation, deposition of sediment is hard to predict.
6 Conclusions

From the analysis and discussion on the different topics treated during this master thesis, the following conclusions can be drawn. Conclusions are sorted by the sequence of the research questions as mentioned in section 1.3.

General

In general one can state that the usage of push barge combinations for the removal of small sandy shoals requires high flow velocities. These flow velocities can be predicted very well by the modified Martin and Maynord equation. The flow velocities induce a sediment transport of nearly 70-170 (depending on the sailing speed) times as high as would occur in an undisturbed situation in the river Waal. The duration of these increased flow velocities on the other hand results in an increase in sediment transport for only 60-50s depending on the sailing speed. The total erosion caused by the return flow of the passing ship will thus be small due to the short duration compared with erosion due to natural flow. Therefore it can be stated that the usage of push barge combinations for the removal of small sandy shoals by just using the return flow is not effective.

Navigation and Hydrodynamics

From calculations using the installed power and resistance of a ship sailing with a certain speed, it follows that the push barge combinations on the river Waal are able to sail at a maximum speed of 3.07m/s. For this speed manoeuvrability is not taken into account.

With a sailing speed of 2.56m/s and a flow velocity of 1.1m/s in opposite direction, the occurring flow velocities will vary around 2.84m/s (Modified Martin and Maynord) under the bow of the ship. For a sailing speed of 3.07m/s and the same flow velocity the occurring flow velocity will be around 3.60m/s (Martin and Maynord).

Important to mention is that the WL Delft Hydraulics formula prove to be very useful. From measurement and model testing coefficients are derived which can be multiplied by the return velocity as calculated using Schijf. Just as the coefficients in the WL Delft Hydraulics formula, these are most of the time between 1.5 and 2. Also the velocities as calculated using the modified Martin and Maynord are lying in between the \((1.5-2)U_r\).

Sediment transport

The model of Sieben seems to overestimate the amount of erosion occurring due to the passage of a push barge combination. With a lack of measurements to validate this, the value of this model should not be underestimated. Besides the quantitative output, the model gives a very good qualitative result. Relevant processes can be shown by ‘playing’ with the parameters.

A very easy assumption for the location where the sediment will be deposited again would be to use the adjustment length, which is 187m. In practice this will be smaller, since for the adjustment length the sediment particle is considered to fall from the top of the water column to the bottom. Using the keel clearance as the maximum height of the water column, the adjustment length is equal to 40.3m.
Model testing

Most important parameters are the sailing speed, the draught, the flow velocity and the water level. These are the ones influencing flow under the bow the most. Using EMS-probes to measure flow velocities with a small under keel clearance is not successful due to the interference of the bottom of the flume on the measuring device.

Flume experiments

The reliability of the data gained from measurements is not easy to quantify. When more experiments could have been carried out, this could have been checked by doing some experiments more than one time. This way consistency could have been checked. With the data gained from the experiments during this master thesis, qualitative information could be derived, but the quantitative information considering sediment transport was not easy to validate. Measurements of flow velocities on the other hand show correlation with the existing formulae.

The maximum flow velocity measured under the bow of the barge is equal to 1.52m/s which is equal to 1.31*U_r (calculated using Schijf).

Removal of sandy shoals is not proved by the experiments carried out in the flume. Since not all flow velocities measured during these test are reliable, it is not possible to make a quantitative prediction of the amount of sediment transported in case of a smaller D_50. The measuring error is nearly as big as the measured difference in bed level, which makes the measured data unreliable.

Model simulations show a negative pressure at the bow of a ship, which causes sediment to lift off from the bed. This is clearly visible at photographs taken from tests done by Hein Bots.

Computational model

DelKelv is a very good model for gaining insight in flow around objects and thus for modeling flow around a ship. The dimensionless flow velocities calculated from the model runs are comparable with the ones calculated using the measurements of the velocity profile. Also the decrease of specific discharge of 55% between the undisturbed situation and the bow was calculated using DelKelv data. This decrease in discharge is not further noticeable since DelKelv does not take friction into account.

Using DelKelv it becomes clear that the width over which the flow velocities increase is very well visible. For a 2x2 barge combination, the flow velocities at a distance of 1*B_s the flow velocities became that small that they were smaller than 1.1*U. It is now clear that the increase in flow velocity due to the passage of a push barge combination will be noticed over a total width of nearly 2*B_s (1*B_s from the centerline of the barges, to both sides of the combination)

Maximum occurring flow velocities for a flat bottom are between (1.4-1.5)*U_0, this is equal to about 1.33*U_r. For the model runs with dunes in the bottom profile, the occurring flow velocities are much higher, these are equal to 1.92*U_r. This again proves the use of the WL Delft Hydraulics formula.
7 Recommendations

Propeller induced erosion

The most important recommendation is to add the effect of the three propellers of the push boat to the increase of flow velocities. Bots (2011) already showed in the flume that the erosion of the bed can be very large when sailing with smaller draughts.

Prototype measurements

An option is to add measuring devices on the push barge units sailing in the river Waal, known is that captains of Thyssen-Krupp-Veerhaven are using a multibeam to measure actual depths when sailing in the river Waal. This is valuable data, especially when the multibeam data of the ship sailing behind another is compared with its predecessor. Erosion or accretion can then be derived when they sail the same track.

Modeling

Many different computer models are available which are able to give more information about sediment transport. Also the influence of turbulence and boundary layer development can be taken into account with some of the models.

Costs

Technical effectiveness is not equal to effectiveness in general. Money, safety and changes in environment play a role before the usage of push barge combinations can be considered applicable. Although the induced erosion might be small, the usage of push barge combinations for sailing over shallower sections can prove to be effective when the costs for extra fuel consumption are low.
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Appendix A: Flume number 4

Discharge and Flow velocity depending on the pump capacity

<table>
<thead>
<tr>
<th>Water level (m)</th>
<th>Wet surface</th>
<th>Pump P41 O/L/H</th>
<th>Pump P42 O/L/H</th>
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<th>Rate of flow (m³/s)</th>
<th>Flow velocity (m/s)</th>
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Appendix B: Ankidunov

Parameters and their values in the method of Ankidunov:

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<thead>
<tr>
<th>Variable</th>
<th>Description</th>
<th>Value</th>
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<tbody>
<tr>
<td>$A_s$</td>
<td>cross section midships</td>
<td>91.20 m$^2$</td>
</tr>
<tr>
<td>$A_c$</td>
<td>cross section channel</td>
<td>1912.5 m$^2$</td>
</tr>
<tr>
<td>$B$</td>
<td>beam (of ship)</td>
<td>22.80 m</td>
</tr>
<tr>
<td>$B_{ch}$</td>
<td>channel width</td>
<td>375 m</td>
</tr>
<tr>
<td>$B_{Tr}$</td>
<td>stern transom width (assumed $=B$ in this case)</td>
<td>22.80 m</td>
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<tr>
<td>$C_b$</td>
<td>blockage coefficient</td>
<td>0.95 [-]</td>
</tr>
<tr>
<td>$F_{nh}$</td>
<td>Froude number</td>
<td>0.36 [-]</td>
</tr>
<tr>
<td>$g$</td>
<td>gravitational acceleration</td>
<td>9.81 m/s$^2$</td>
</tr>
<tr>
<td>$h$</td>
<td>channel depth</td>
<td>5.1 m</td>
</tr>
<tr>
<td>$h_t$</td>
<td>trench height</td>
<td>5.1 m</td>
</tr>
<tr>
<td>$K_{tr}$</td>
<td>trim coefficient</td>
<td>0.64 [-]</td>
</tr>
<tr>
<td>$K_{Pt}$</td>
<td>propeller trim</td>
<td>0.20 [-]</td>
</tr>
<tr>
<td>$K_{TrT}$</td>
<td>stern transom factor</td>
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<td>$K_{TTT}$</td>
<td>initial trim factor</td>
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<tr>
<td>$K_{PS}$</td>
<td>propeller</td>
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<td>$K_{TR}$</td>
<td>trim coefficient</td>
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<tr>
<td>$L_{pp}$</td>
<td>Length between perpendiculars</td>
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<tr>
<td>$n_{Tr}$</td>
<td>trim coefficient</td>
<td>2.89 [-]</td>
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<tr>
<td>$P_{ch1}$</td>
<td>channel effects</td>
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<tr>
<td>$P_{nh}$</td>
<td>ship forward speed</td>
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<tr>
<td>$P_{hu}$</td>
<td>ship hull parameter</td>
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<td>$P_{ch2}$</td>
<td>channel effect trim correction</td>
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<tr>
<td>$P_{+h/T}$</td>
<td>water depth effects</td>
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<td>$S$</td>
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<tr>
<td>$S_h$</td>
<td>channel depth factor</td>
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<tr>
<td>$S_m$</td>
<td>midpoint sinkage</td>
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</tr>
<tr>
<td>$S_{max}$</td>
<td>maximum squat</td>
<td>0.39 m</td>
</tr>
<tr>
<td>$T$</td>
<td>draught (=$4$m)</td>
<td>variable m</td>
</tr>
<tr>
<td>$T_{ap}$</td>
<td>static draft at stern</td>
<td>4.00 m</td>
</tr>
<tr>
<td>$T_{fp}$</td>
<td>static draft at bow</td>
<td>4.01 m</td>
</tr>
<tr>
<td>Trim</td>
<td>trim of the ship</td>
<td>-0.001 [-]</td>
</tr>
<tr>
<td>$V_s$</td>
<td>sailing speed (=2.56 or 3.07m/s)</td>
<td>variable m/s</td>
</tr>
</tbody>
</table>

Below the formulas used for the calculation of the different coefficients and parameters are given.

$$S_{Max} = L_{pp} \left( S_m \mp 0.5 \cdot \text{Trim} \right)$$

$$S_m = (1 + K^S) P_{Hu} P_{Fa} P_{+h/T} P_{Ch1}$$

$$\text{Trim} = -1.7 P_{Hu} P_{Fa} P_{+h/T} K_{TrT} P_{Ch2}$$
\[ S_m = (1 + K_p^S) P_{h_T} P_{F_{gh}} P_{P_{Ch1}} \]

\[
K_p^S = \begin{cases} 
0.15 \\
0.13 
\end{cases}
\]

\[ P_{h_T} = 1.7 C_B \left( \frac{BT}{L_{pp}^2} \right) + 0.004 C_B^2 \]

\[ P_{F_{gh}} = F_{gh}^{(1.8+0.4 F_{gh})} \xrightarrow{F_{gh}} F_{gh} = \frac{V_s}{\sqrt{g \cdot h}} \]

\[ P_{Ch1} = \begin{cases} 
1.0 \\
1.0 + 10 S_h - 1.5 (1.0 + S_h) \sqrt{S_h} \xrightarrow{S_h} S_h = C_B \left( \frac{S}{h / T} \right) \left( \frac{h_T}{h} \right) \rightarrow S = \frac{A_S}{A_{Ch}} 
\end{cases} \]

\[ T_{rim} = -1.7 P_{h_T} P_{F_{gh}} P_{h_T} K_{Tr} P_{Ch2} \]

\[ P_{h_T} = 1.0 + \frac{0.35}{(h / T)^2} \]

\[ P_{F_{gh}} = F_{gh}^{(1.8+0.4 F_{gh})} \]

\[ K_{Tr} = C_B^{n_{Tr}} - (0.15 K_p^S + K_p^S) - (K_B^T + K_{Tr}^T + K_T^T) \]

\[ P_{Ch2} = \begin{cases} 
1.0 \\
1.0 - 5 S_h 
\end{cases} \]

For more information about the values used for coefficients, see Briggs (2009)
Appendix C: Resistance and Power

Calculation of resistances with used values:

\[ R_F : \quad = \frac{\sqrt{2} \cdot 0.002 \cdot 1000 \cdot (2.56 + 1.1)^2}{4712} = 63.1kN \]
\[ R_P : \quad = \frac{\sqrt{2} \cdot 0.2 \cdot 1000 \cdot 2.56^2}{91.2} = 59.8kN \]
\[ R_H : \quad = 1000 \cdot 9.81 \cdot 91.2 \cdot 0.25 = 223.7kN \]
\[ R_T : \quad = 63.1kN + 59.8kN + 223.7kN = 346.6kN \]

Calculation of power with used values:

\[ P_F : \quad = \frac{63.1 \cdot (2.56+1.1)}{0.7} = 330.0kW \]
\[ P_P : \quad = \frac{59.8 \cdot (2.56+1.1)}{0.7} = 312.7kW \]
\[ P_H : \quad = \frac{223.7 \cdot (2.56+1.1)}{0.7} = 1169.6kW \]
\[ P_T : \quad = P_F + P_P + P_H = 1812.3kW \]
Appendix D: Answers to research questions

**Navigation**

Many different ships are present on the river Waal, but the push barge units are the ones that could be used best for the removal of small sandy shoals. Stolker and Verheij (2006) already noted that the increase in flow velocity at the passage of a barge train was significantly higher than any other type of vessel from which measurements were available.

The push barge combinations are sailing with an average speed of 2.56m/s in upstream direction and with an average speed of 5.56m/s in downstream direction. Push barge combinations from Thyssen-Krupp-Veerhaven are only loaded when sailing in upstream direction since they transport bulk to Germany. This research focuses on the ones sailing in upstream direction since the flow velocities under the ship are higher and the barges are moving with a larger draft, which results in a larger water displacement.

In one day 4-5 push barge combinations will sail in upstream direction, this is only the ones from Thyssen-Krupp-Veerhaven, further information is not known.

Restrictions that have to be taken into account when using push barge combinations are summarized below:

*When sailing with an increased speed, the amount of squat increases. For ships sailing with a draught of 4m, this could lead to groundings and thus to unsafe situations. Before asking a captain to increase speed, one should always take this into account.*

*Maneuverability decreases with increased sailing speed, this is important to be aware of since the river Waal is a busy waterway.*

*Only within the ‘virtual’ waterway (with a width of 150-170m) shallow sections can be removed using the push barge combinations, outside these margins it is not safe to sail.*

**Sediment**

Engelund-Hansen is the best sediment formula for modelling sediment transport on the river Waal. It takes into account suspended transport, which is occurring on this river, and proves to give accurate results. Also according to C.Sloff (Deltares) Engelund-Hansen is one of the best sediment transport formulae when considering sediment transport on rivers.

In one day according to the theory an erosion of nearly 0.01m will occur. This is calculated using the bottom shear stress with an average flow velocity of 1.1m/s, from which the shear velocity could be derived. The passage of a ship will lead to an sediment transport that is nearly 70 times as high as during an undisturbed situation. During the passage of a ship with the maximum speed of 3.07m/s, the
sediment transport will further increase until nearly 170 times the amount of sediment transported in an undisturbed situation.

For removal of sediment in certain sections of the Waal a sediment balance should be respected, which results in restrictions on the distance over which sediment is allowed to be transported again. Since these allowed distances are much larger than the distance over which the sediment is transported in the cases of this master thesis, this subject this is not ought to be relevant.
Appendix E: Derivation model A. Sieben

1. \[
\frac{\tau_{b,II}}{\tau_{b,I}} = -\rho_w \cdot \left( \frac{q + V_z \cdot d_z}{d_1} \right)^2
\]

2. \[
\frac{\tau_{b,II}}{\tau_{b,I}} = \left( \frac{q + V_z \cdot d_z \cdot d}{q} \right)^2
\]

3. \[
\frac{\tau_{b,II}}{\tau_{b,I}} = \left( \frac{q + V_z \cdot d_z}{q} \cdot \frac{d}{d_1} \right)^2
\]

4. \[
\frac{\tau_{b,II}}{\tau_{b,I}} = \left( 1 + \frac{V_z \cdot d_z}{U \cdot d} \right)^2 \cdot \left( \frac{d}{d_1} \right)^2
\]

5. \[
s_{I\infty} \left( \frac{\tau_{b,I}}{\tau_{b,I}} \right)^{b/2}
\]

6. \[
s_{II\infty} \left( \frac{\tau_{b,II}}{\tau_{b,II}} \right)^{b/2}
\]

7. \[
\left( \frac{\tau_{b,II}}{\tau_{b,I}} \right)^{b/2} - \left( \frac{\tau_{b,I}}{\tau_{b,I}} \right)^{b/2} = V_z \cdot \Delta z
\]

8. \[
\left( \frac{\tau_{b,II}}{\tau_{b,I}} \right)^{b/2} \left( \frac{\tau_{b,I}}{\tau_{b,I}} \right)^{b/2} = V_z \cdot \Delta z
\]

9. \[
\left( \frac{\tau_{b,II}}{\tau_{b,I}} \right)^{b/2} - 1 \cdot s_I = V_z \cdot \Delta z
\]

10. \[
V_z \cdot \Delta z = \left( \left( 1 + \frac{V_z \cdot d_z}{U \cdot d} \right)^2 \cdot \left( \frac{d}{d_1} \right)^2 \right)^{b/2} - 1 \cdot \frac{w \cdot d}{b}
\]

11. \[
V_z \cdot \Delta z = \left( 1 + \frac{V_z \cdot d_z}{U \cdot d} \right)^b \cdot \left( \frac{d}{d_1} \right)^b \cdot 1 - 1 \cdot \frac{w \cdot d}{b}
\]

12. \[
\frac{\Delta z}{d} = \frac{U}{V_z} \cdot \frac{w}{b \cdot U} \cdot \left( 1 + \frac{V_z \cdot d_z}{U \cdot d} \right)^b \cdot \left( \frac{d}{d_1} \right)^b - 1
\]