Thesis Report

Feasibility Study of Implementing a Central Suction-WID System in a Tide-Dominated Channel

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Feasibility Study of Implementing a Central Suction-WID System in a Tide-Dominated Channel

by

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Preface

This research has been conducted as a part of the master program of hydraulic engineering at TU Delft. It marks the final stage of my study in this program. I would like to express my gratitude to the people that have provided me with their support and guidance through the ups and downs during the progress of this research.

This research has been done in collaboration with Port of Rotterdam. The combination of theoretical knowledge from TU Delft and practical knowledge from Port of Rotterdam has provided me an excellent environment and guidance to finish this thesis.

I would like to thank my family for their endless support during this step of my life, the little boy has grown up now. I also would like to thank Alex Kirichek as a chairman and the daily supervisor, who is also the person who introduce me to the *water injection dredging world*. Also big thanks to Mike Hoek, Andre van Hassent, and Edwin Hupkes who always provided a lot of guidance and expertise about dredging inside a port during our regular meeting on Monday morning, what a way to start a week. To Patricia Buffon, big thanks to you for helping me during our discussions and help me to improve my writing skill. Thanks to Bram van Prooijen, as the first person that give me a lot of advices to choose a thesis topic and as a lecturer who introduces me to sediment. I also want to thank Thijs van Kessel for helping me out in terms of using a numerical model.

Last but not least, thanks to all of fellow friends who support me during the process of finishing this thesis and help me to cope with all of the pressure. To all of the readers, I hope you can learn something by reading this thesis. I wish this thesis can also contribute for future research and development in the dredging industry. *Veel plezier!*

S.W. Kurniawan

Delft, 18 October 2021

Contents

Abstract

The feasibility of installing a suction system, which consists of a pump and pipeline for transporting sediment from the sediment trap to the reallocation site to complement the water injection dredging (WID) method is accessed in this thesis. The combination of a suction system and a water injection dredger is called central suction-WID system. This thesis analyses the sediment return volume, necessary pump power, greenhouse gases production and necessary costs for implementing a central suction-WID system. Three different reallocation sites with different distances for central suction-WID are assessed: Inside of Nieuwe Waterweg, Outside of Nieuwe Waterweg, and Verdiepte Loswal..

Delft3D-Flow numerical model is used for modelling the hydrodynamic and the sediment return volume. The simulation is run for 14 days period in order to simulate spring and neap period. The numerical model result shows that discharging the sediment at the inside of Nieuwe Waterweg leads to the most sediment return, while discharging at the Verdiepte Loswal leads to the least sediment return volume. Increasing the sediment volume or reducing the particle size does not affect the sediment return rate.

The necessary pump power and energy consumption for reallocating the sediment to each location are assessed. The calculation shows that the major contributor to the energy consumption is the friction between the fluid mud and the pipe wall. Reallocating the sediment to the inside of Nieuwe Waterweg consumes the least energy while transporting it to Verdiepte Loswal needs more energy as it is the furthest reallocation site alternative.

Using a WID vessel produces fewer greenhouse gases than hopper dredger as it consumes much less fuel. Costs analysis shows that using a central suction-WID system is more expensive compared to hopper dredger. The costs of using a central suction-WID system increases as the distance between the sediment trap and the reallocation site is higher. However, combining it with another dredging method can be an alternative as it reduces the operational costs.

1 Introduction

1.1 Background and Motivation

Ports are subject to continuous change in order to maintain a strong market position. For instance, increasing vessel sizes and traffic volumes require larger water depths at the harbour channels. This leads to regular maintenance dredging needed to maintain the depth. The cost and the pollution of maintenance dredging can be relatively high. Port authorities must adapt dredging methods to more technological, socio-economic, and environmental alternatives.

As the busiest port in Europe (Gardham, 2022), Port of Rotterdam (PoR) also performs maintenance dredging. The volumes of dredged sediment have substantially increased in the Port of Rotterdam over the past years. In order to keep the port channels and waterways accessible, more than 11 million m^3 of deposited sediment were dredged in 2019 as it is shown at Figure 1.1.

The port authorities are seeking for tailor-made solutions that can help to reduce maintenance costs and CO_2 emissions while guaranteeing safe navigation in the port. Currently, maintenance dredging is performed every month at the location shown in Figure 1.2, called T-Shirt. Recently, a water injection dredging (WID) pilot was conducted by Port of Rotterdam at this location. During this WID pilot, the fluid mud produced by a WID was collected by a sediment trap. Trailer suction hopper dredging (TSHD) is then used for transporting fluid mud from the sediment trap to the reallocation site (Kirichek & Rutgers, 2019).

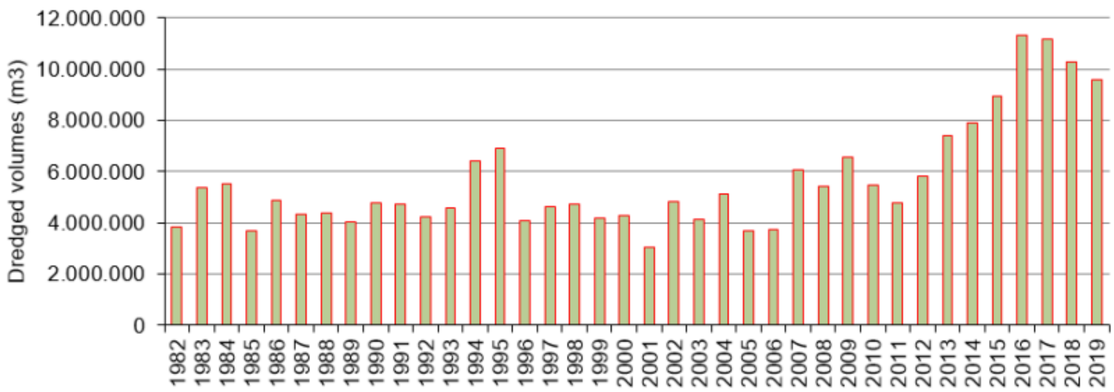

Figure 1.1 Dredged sediment volumes at Port of Rotterdam from 1987 to 2019 (Kirichek et al., 2021)

Figure 1.2 Study location (T-shirt) at the Port of Rotterdam is indicatd by red dashed lines. Water injection dredging is regularly conducted in this area. Yellow coloured area shows the location of the sediment trap.

WID pilot project has been done at the Calandkanaal at the Port of Rotterdam (Kirichek & Rutgers, 2019), where the sediment characteristics is the same as at the T-Shirt. The position of the sediment trap can be seen at Figure 1.2. Sediment trap in this project is about 1 m deeper than the bed level.

In this research, suction system will be installed at the sediment trap to collect fluid mud from the sediment trap and transport it to the reallocation site. A suction pump is used to pump the sediment through a pipeline. The suction system is expected to consume less energy and more sustainable compared to transporting the material using a hopper dredger. Combination of dredging using WID, sediment trap, and the suction system is called central suction-WID system. The benefits of using a suction system to reallocate the sediment is that it can be operated by electric power, which is in line with the energy transition movement of maritime sector. Electric power is seen as a more sustainable energy compared to marine fuel as it can be supplied from a renewable energy source. In this thesis, wind turbine is chosen as the main energy source for the suction system as wind turbines can be installed at the port area. Illustration of a central suction-WID system can be seen at Figure 1.3. This thesis assess the feasibility study of implementing the central suction-WID system as a maintenance dredging method.

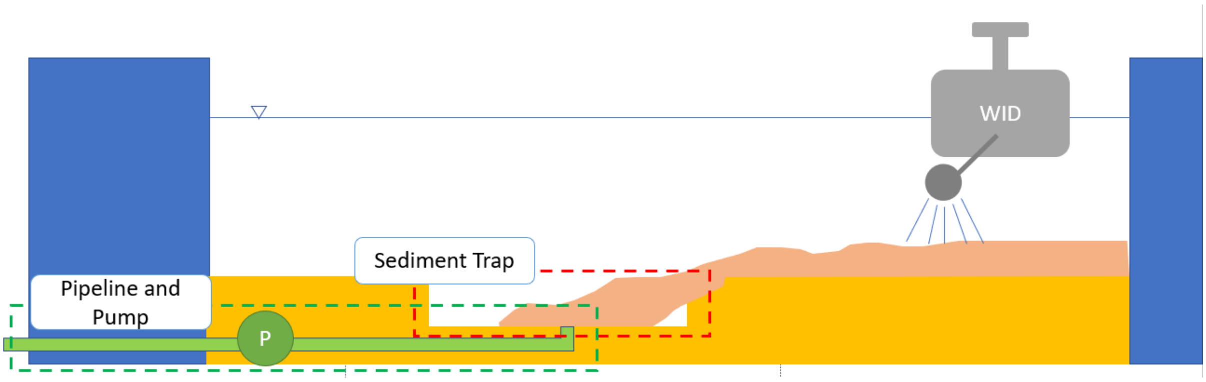

Figure 1.3 Illustration of central suction-WID system. The WID vessel fluidizes sediment and create a fluid mud layer that is transported to the suction system. After fluid mud is collected by the sediment trap, a suction system consisting of a pipeline and pump transports the sediment to the reallocation site.

1.2 Research Questions and Objectives

This thesis aims to conduct a feasibility study of a central suction-WID system by assessing its advantages and disadvantages. There are few aspects of central suction-WID system assessed in this thesis. A few scenarios with different sediment reallocation sites are tested in term of the percentage of sediment returning to the harbour area, pipe length, and the necessary pump power to operate the suction system. The feasibility study also compares the cost and greenhouse gases production of central suction-WID system to a conventional dredging method using a TSHD, which is currently used by Port of Rotterdam. The main research question of the thesis can be summarised as:

What are the advantages and disadvantages of a central suction water injection dredging system at a tide-dominated channel?

To answer the main research question, sub-research questions were defined with their corresponding objectives:

R.Q.1: What is the sediment return rate after being discharged to the reallocation site?

a. Define the natural hydrodynamic forces that affect the sediment transport and identify the sediment transport patterns;

b. Identify possible locations for sediment reallocation considering practical constraints;

c. Determine the ideal hydrodynamic conditions to discharge sediment;

d. Estimate the sediment return rate.

R.Q.2: What is the pump power necessary to reallocate the sediment?

e. Determine the pump power necessary to transport the sediment from the central suction system to the reallocation site;

f. Determine the annual energy consumption and how many wind turbines needed for operating the pump for central suction-WID system.

R.Q.3: What is the performance of the central suction-WID System compared to a conventional maintenance dredging method?

g. Estimate the greenhouse gases emissions produced by the central suction-WID system and for TSHD, and compare both;

h. Estimate the cost necessary for a central suction-WID system and for TSHD, and compare both.

1.3 Outline

This research is organized in six chapters as follows. The first and the last chapters are an introduction and conclusion of the thesis, respectively. Chapter 2 is intended to provide available information related with this research. Each chapter from chapter 3 to chapter 5 is related to the research questions above.

- Chapter 2 Literature Study: this chapter focuses on providing the necessary knowledge and information related to this thesis. Information about the hydrodynamics and sediment transport at the investigated area, the numerical model used, and information about WID are discussed in this chapter;
- Chapter 3 Hydrodynamics and Sediment Return Rate Analysis: in this chapter, hydrodynamics and the sediment return rate are assessed by using a Delft3D-Flow model. The hydrodynamics simulation results are qualitatively compared to the literature from chapter 2 in order to check whether the model represents the real conditions. There are a few alternatives for sediment reallocation site based on the hydrodynamic conditions and practicability;

- Chapter 4 Pump Power Necessary for Transporting Sediment: calculation of the pump power necessary for transporting the fluid mud from the sediment trap to the sediment reallocation site and the total energy consumption to operate the central suction-WID system.
- Chapter 5 Greenhouse Gases Emissions and Cost Analysis: this chapter compares the implementation of the central suction-WID system to a hopper dredger in terms of greenhouse gases production and the necessary cost.
- Chapter 6 Conclusions and Recommendations: in this chapter, the main conclusions and recommendations derived from this thesis are summarized.

Figure 1.4 illustrates the outline of this thesis.

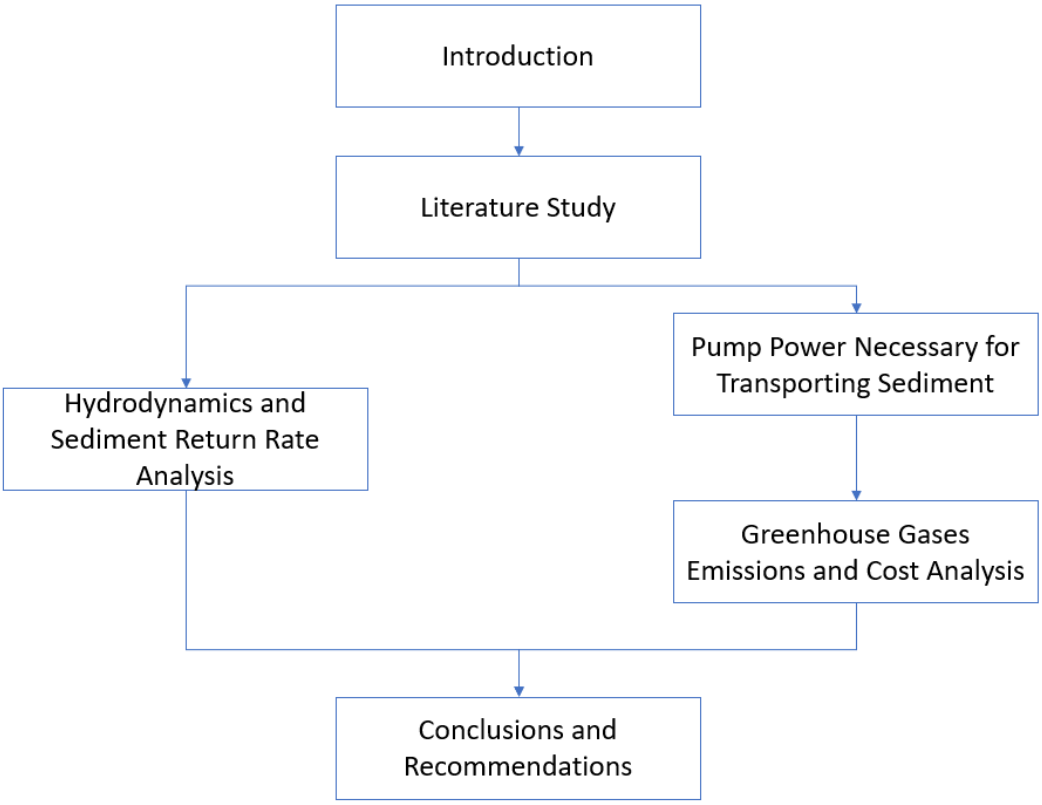

Figure 1.4 The scheme shows how this thesis is structured and the relationship between each chapter.

2 Literature Study

In this chapter, an open-access literature and available knowledge related to this thesis is presented. This information is used for making a qualitative comparison with the results of this study and for providing a reasonable background for the analysis in this thesis.

2.1 System Knowledge

Port of Rotterdam area is located at an estuarine area. The detailed information about the estuarine systems is provided in Appendix A. This region is highly influenced by the freshwater outflow of the Rhine-Meuse estuary through the Rotterdam Waterway (de Groot, 2018). Nieuwe Waterweg is located at the downstream of the Rotterdam Waterway which is connected to the Maas river. The influence of the river discharge influences the hydrodynamic in this location.

Tidal asymmetry is an asymmetry of the horizontal or the vertical tide (current or water level) and leads to a difference in magnitude and duration of a tidal component. Tidal asymmetry at the Port of Rotterdam area is mainly caused by the distortion of the tidal wave as it propagates into shallow waters such as the North Sea or the Rotterdam Waterway (Deckers, 2020). In combination with the freshwater discharge, this leads to a longer ebb than flood period. At the downstream of Rotterdam Waterway generally the upper part of the water column is ebb-dominated, while close to the bed is flood-dominated (de Nijs, 2012). Real time water level and flow velocity measurement and prediction at the Port of Rotterdam area can be accessed at the website of Port of Rotterdam (https://weather-tide.portofrotterdam.com/desktop/). Figure 2.1 shows that at the investigated location, the maximum flood current is faster than the maximum ebb current.

On the harbour basins lining the Maasmond (Maasvlakte, Europort), sediment that is suspended at sea is advected into the basins by the tidal and baroclinic currents (Verlaan et al., 2000; Winterwerp & van Kessel, 2003). De Nijs (2012) mentioned that marine sediments enter the Maasmond area as near-bed suspensions. These near-bed marine suspensions do not propagate into the Rotterdam Waterway because the Maasmond area is much deeper than the Rotterdam Waterway. De Nijs (2012) did a contaminant concentrations measurement in dredging areas in the Port of Rotterdam and it is found that the sediment at the Beerkanaal and Calandkanaal comes from the North Sea. In the Port of Rotterdam area, marine sediment predominantly consists of silt (Ebbens, 2013) is mainly found in the harbour basins while sand is found on the rivers as it is shown at Figure 2.2.

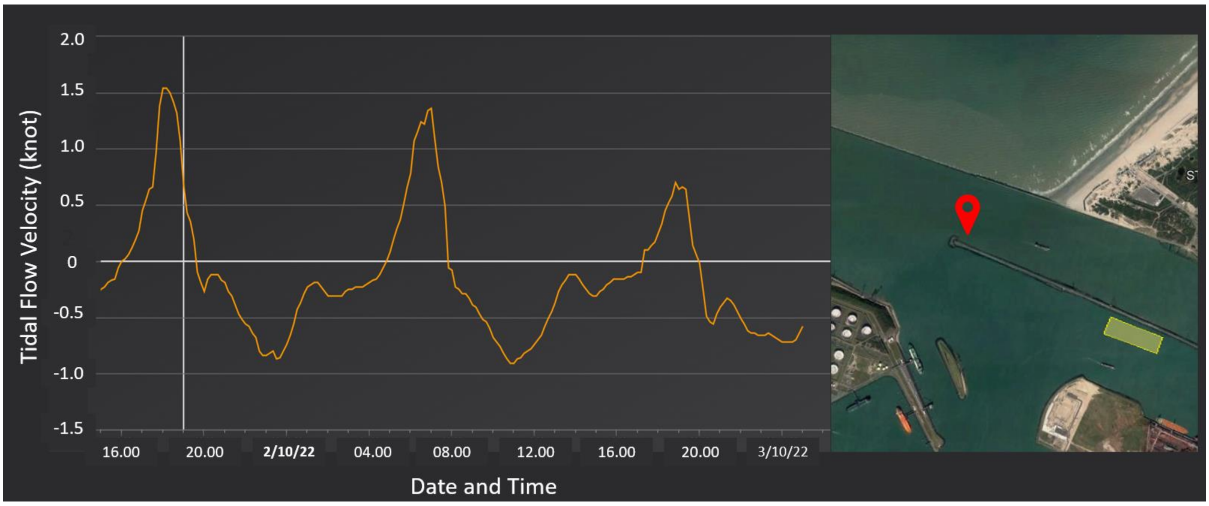

Figure 2.1 Flow velocity measured at the investigated location (red pin). Positive value means the water is flowing to the flood direction. It is observed that the flood peak velocity is faster than the ebb current and the flood period is shorter than the ebb period. This measurement and prediction is accessed from https://weather-tide.portofrotterdam.com/desktop/ at 1 October 2022, 18:56.

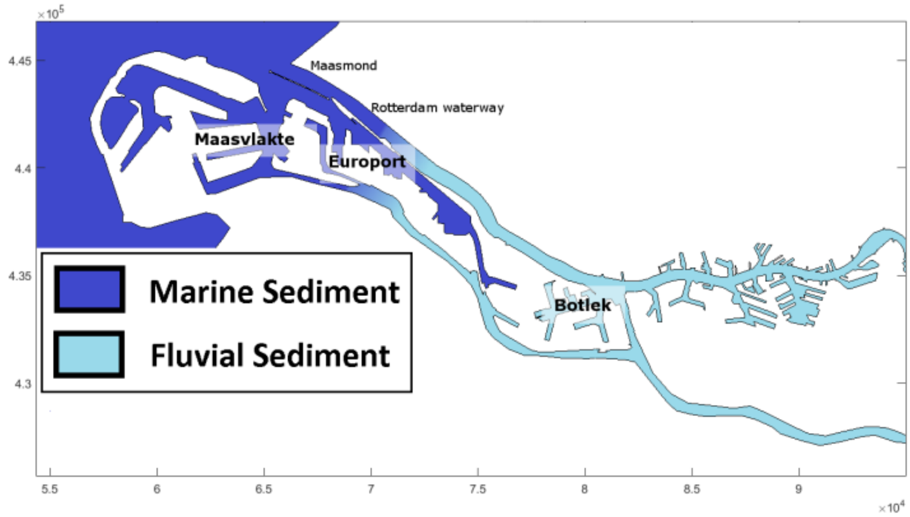

Figure 2.2 Sediment source at the Port of Rotterdam area. Marine sediments are mainly found at the harbor basin area. (de Groot, 2018)

The sediment availability from the North Sea is affected by seasonal variation. The availability of the sediment at the sea boundary is higher during autumn and winter than spring and summer values (de Nijs, 2012) due to rougher meteorological conditions. During and after rough weather periods, massive sedimentation events take place in the Maasmond. The siltation rates during such events can amount to 0.5 Mton within a week (de Nijs, 2012).

2.2 Delft3D-Flow Numerical model

A numerical model is used for simulating the sediment transport at the study location in a smaller scale as it simulates the hydrodynamic at the investigated area with the resolution of up to 40 m. A Delft3D-Flow module from Delft3D-4 numerical model, developed by Deltares, is used to simulate the hydrodynamic condition at the area of study (Figure 1.2). *The Operationeel Stromingsmodel Rotterdam* (OSR) from the Port of Rotterdam is used in this thesis to simulate the hydrodynamic and sediment transport. This model has been frequently used for calculating flow velocities and salinity concentrations for operational use in the Rhine Meuse Delta (Cronin et al., 2019). The model is capable of modelling the sediment transport at Rhine Meuse Delta, including the area of Port of Rotterdam.

The same hydrodynamic model is then used for analysing the sediment return rate. To simulate the outtake of the suction system, a discharge with sediment at the reallocation site is added to the model. At the end of simulation period, the sediment transport volume passing through each cross-sections is compared with the result of hydrodynamic simulation. The difference of sediment transport volume between two scenarios is considered as a sediment return rate.

The Delft3D-Flow is run for the period of 1 May 2016 to 16 May 2016, a period with average river discharge (2200 m^3/s) and no storm events (Cronin et al., 2019). It is important to at least model one full spring neap cycle, which is 2 weeks minimum as the hydrodynamic and sediment transport can be different during spring and neap tide. For this period, boundary conditions from the harbour model were readily available from a PRISMA project (Kirichek, et al., 2021).

The model has a different computational grid size, ranging from 200x200 m to 40x40 m. Each computational grid is divided into 10 layers with an equal depth to represent different hydrodynamic conditions in the vertical direction. Horizontally, the computation domain extends about 17 km off-shore from the coast of Goeree-Overflakke South to just before the sand engine North. Upstream, the grid reaches up to the towns of Gouda (Hollandsche Ijssel), Lekkerkerk (Lek), Papendrecht (Lower Merwede), Dordrecht (Dordtsche Kil), and to (but not including) the Haringvliet (Spui) as shown at Figure 2.3. The Maasvlakte II is represented in the model by using a dry points and thin dams features, which means there are no hydrodynamic computations at this area.

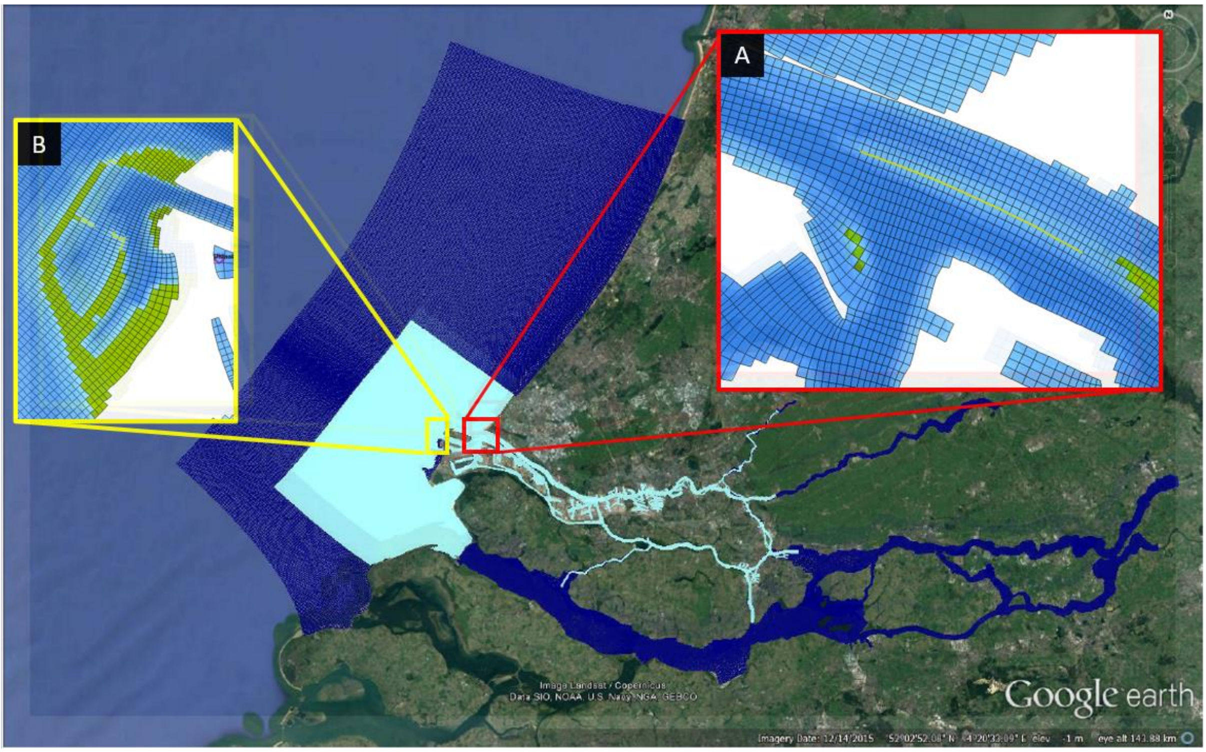

Figure 2.3 The coverage area of the numerical model is marked with the light blue colour. Picture A shows the grids at the investigated area and Picture B shows the Maasvlakte II in the model. Yellow grids on picture A and B means there is no calculation in the specific grids.

Three different sediment fractions are implemented in the model, shown at Table 2.1. Based on the study of Cronin, et al. (2019), these three different sediment settling velocities were calibrated and validate for fine sediment dynamics along the Dutch coastline. These velocities has been proven to give realistic simulations with the reality as sensitivity analysis of the settling velocity has been done by Cronin, et al (2019). Sediment settling velocities also represents flocs settling velocities from the PRISMA report (Kirichek et al., 2021) because the mud samples mainly consists of flocs and there are almost no primary particles.

Table 2.1 Sediment fractions used in the model. The settling velocities of the sediments that are used as an input in the model.

Sediment Class	Settling velocity (mm/s)
Sediment1	2
Sediment2	1
Sediment3	0.5

The initial concentrations of the sediment is part of the boundary conditions and different at each locations. Different bed shear stress are implemented as the numerical model consists of two different bed layers. The fluff layer represents the soft layer located at the top of the bed. The critical bed shear stress of the fluff layer is 0.2 N/m^2. The deeper layer represents a sandy bed which sediment may be buried has a higher critical shear stress, 0.8 N/m^2.

2.3 Water Injection Dredging

Water injection dredging (WID) is a dredging technique in which high volumes of water are injected into the soil (or sediment) with low pressure pumps through the nozzles of a horizontal injection bar (PIANC, 2013). The fluidised soil layer flows under the influence of pressure difference and gravitational forces (Figure 2.4). This density current will continue as long as the physical conditions for flow are maintained. The transport distance is dependent upon site specific conditions. If a deeper area exists around the dredged area, the mud flow will slowly accelerate and flow to the deeper area due to the bed slope (van Rijn, n.d.). During the WID process, fluidized sediment is not taken from the bed by the WID vessel, but it is loaded into a hopper or transported through discharge pipelines to an allocated placement area.

A WID pilot project was conducted by Port of Rotterdam. A sediment trap was used to trap the WID-induced fluid mud. The size of the sediment trap was 600 m over 120 m. The over depth of the sediment trap varies form 1 m to 1.3 m (Kirichek & Rutgers, 2019).

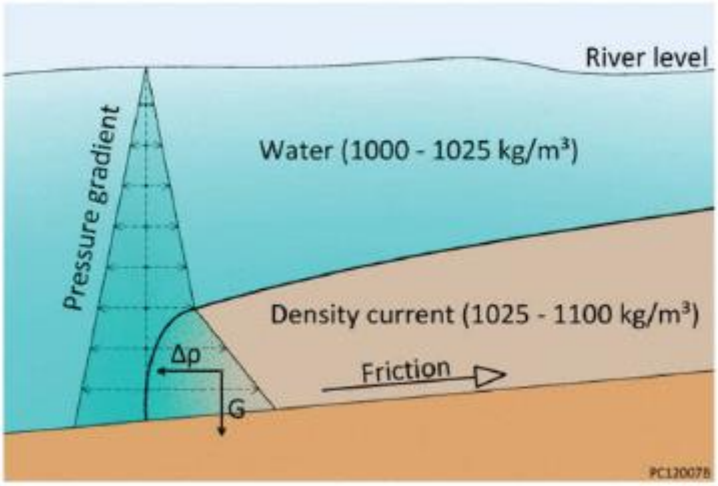

Figure 2.4 The driving force of the fluidized soil layer produced by WID is the hydrostatic pressure difference between the water and the fluidized soil layer and the resisting force is the friction of the layer. As long as the driving force is stronger, the fluid mud will keep flowing. (PIANC, 2013)

3 Hydrodynamics and Sediment Return Rate Analysis

In this chapter, numerical model is used to simulate the hydrodynamic and the sediment transport. Qualitative comparison is then made between the simulation results and the literature. After that, simulation of the suction system is done to determine the sediment return rate. The analysis is done to provide an answer for research question R.Q.1.

3.1 Methodology

Hydrodynamics at the investigated area is simulated by using a numerical model. The model is then qualitatively compared with the information from the literature review in order to determine whether the model represents the real situation. This hydrodynamics simulation results is referred as base scenario.

The discharge from the suction system's outtake is then added to the model as a discharge and the amount of sediment returns to the harbour is calculated. Three different reallocation sites are assessed. The sediment transport of the simulations with discharge is compared with the base scenario, and the difference in sediment volume entering the harbour area between the two scenarios is counted as the sediment return.

Sensitivity analysis is done in order to analyses the influence of some parameters to the model results.

3.2 Hydrodynamic Simulation Results

3.2.1 Water Level

Water level at Port of Rotterdam area is affected by tide Figure 3.1. Between 4 May 2016 and 12 May 2016 occurs the spring period so the tidal range is higher compared to 1-3 May and 13-16 May. Figure 3.2 indicates that the water level elevation is spatially uniform in all observed locations, so there is no a difference hydraulic pressure head between the intake and the outtake of the suction system.

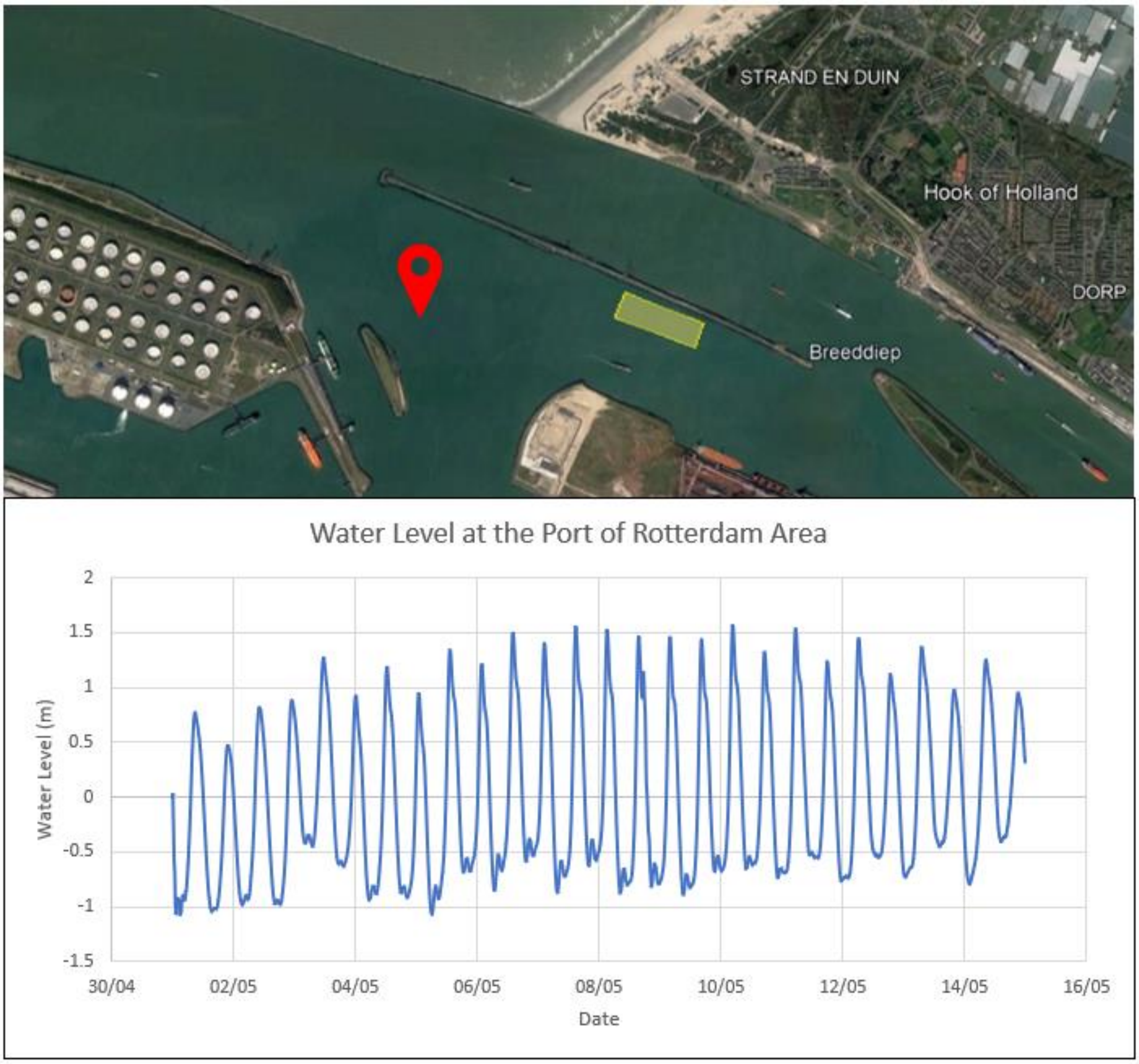

Figure 3.1 Water level at the harbour basin of Port of Rotterdam. Spring tide occurs at 4-12 May leads to higher tidal range on this period.

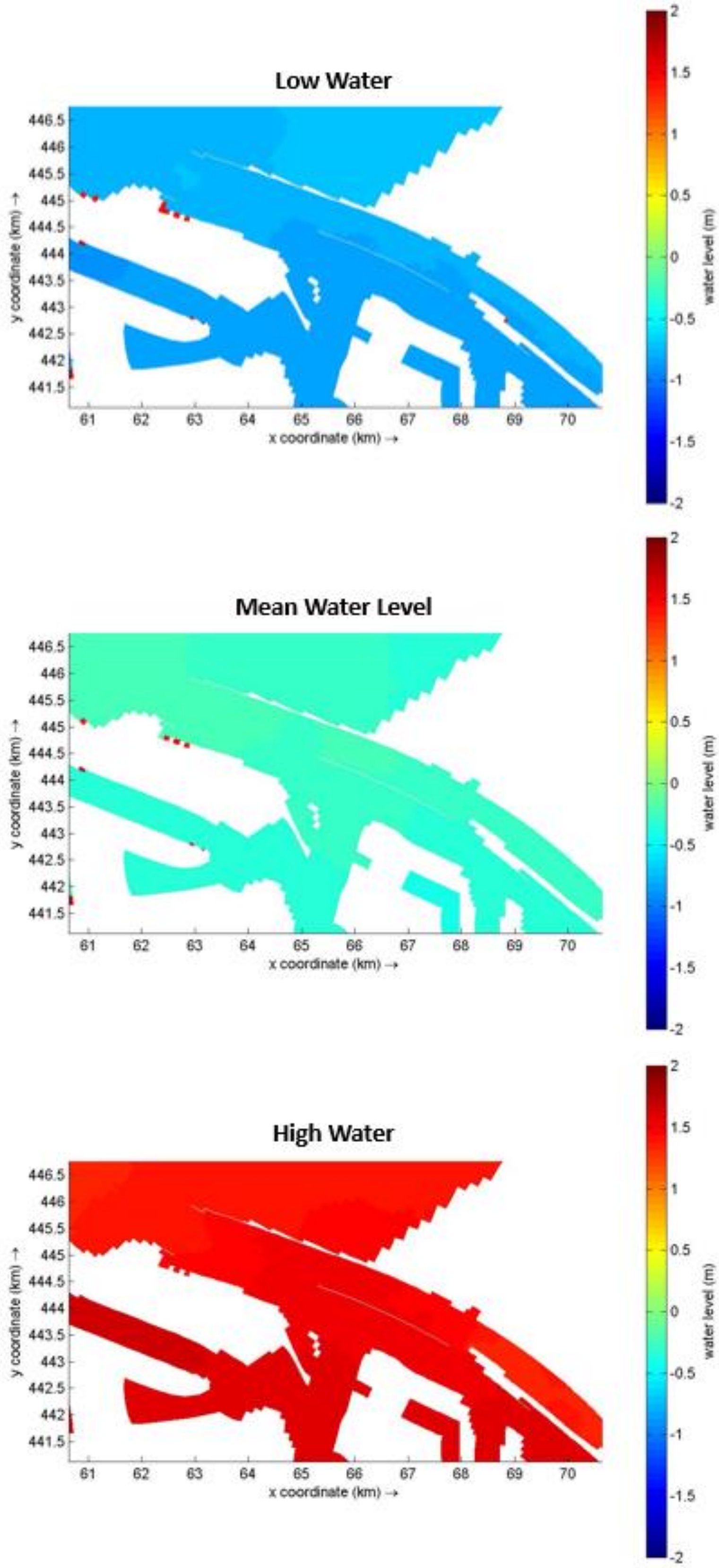

Figure 3.2 Water level at Port of Rotterdam area during low water, mean water level, and high water. There is no significant spatial different on water levels for every events.

3.2.2 Flow

The vertical flow velocity profile is different between at the Nieuwe Waterweg and at the harbour area represented by the Calandkanaal. Figure 3.3 shows the velocity profile at the Nieuwe Waterweg (A) and the Calandkanaal (B).

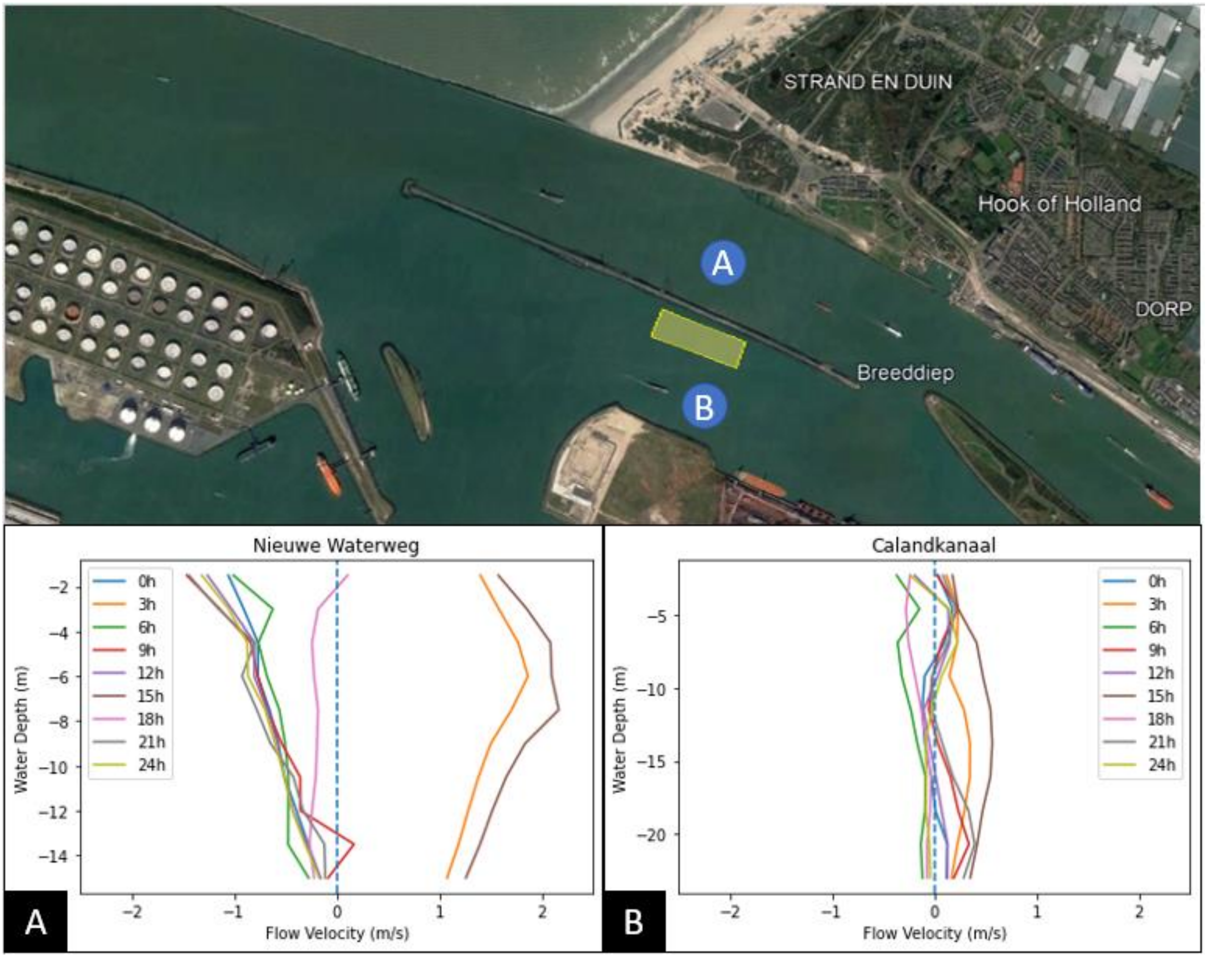

Figure 3.3 The flow velocity profile at the Nieuwe Waterweg (location A) and Calandkanaal (location B). Positive value represent to the upstream and negative value means the flow is directed to the North Sea. The flow at the Nieuwe Waterweg is ebb-dominated at the surface and flow-dominated at the bottom, while the flow at the Calandkanaal is flow-dominated at all elevations.

Based on Figure 3.3, the current is flowing to the ebb direction most of the time at Nieuwe Waterweg while the flood current is faster than the ebb current. This condition is in accordance with the finding of de Nijs (2012). Figure 3.4 shows the flow velocity direction distribution at the Nieuwe Waterweg and it is shown that at the surface, the flow is ebb-dominated and the flow near the bed is flood-dominated, similar to the finding of de Nijs (2012).

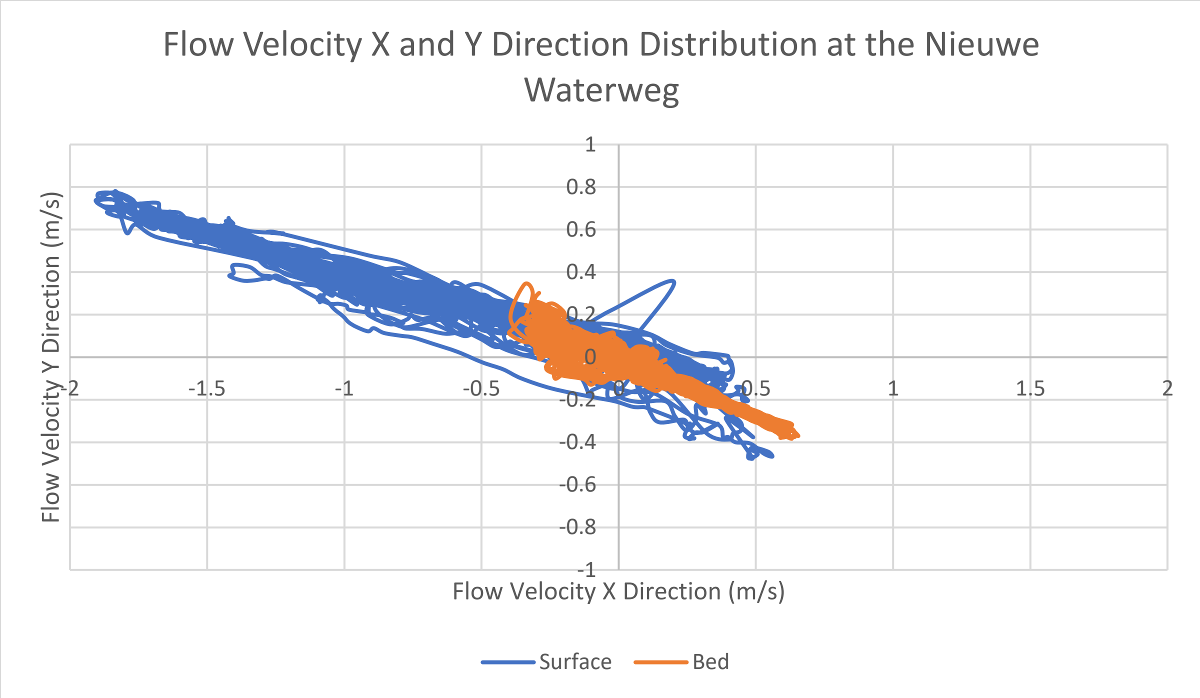

Figure 3.4 X and Y direction flow velocity distribution at the Nieuwe Waterweg (Figure 3.5 Location A). This graph is ebb-dominated flow is observed at the surface (blue line) while the flow near the bed is flood-dominated.

3.2.3 Bed Shear Stress

Bed shear stress indicates whether sediment will be deposited or transported to specific locations. The bed shear stress is related with flow velocity with a proportional to the squared velocity (Gatto, 2017). Results from the numerical simulation shows that there is no corelation between water level and bed shear stress. The bed shear stress is high when the flow velocity is high regardless of directions. Figure 3.5 shows the map of bed shear stress at the harbour area during the three different water levels. The exact value of bed shear stresses in a few locations are shown in Appendix B.

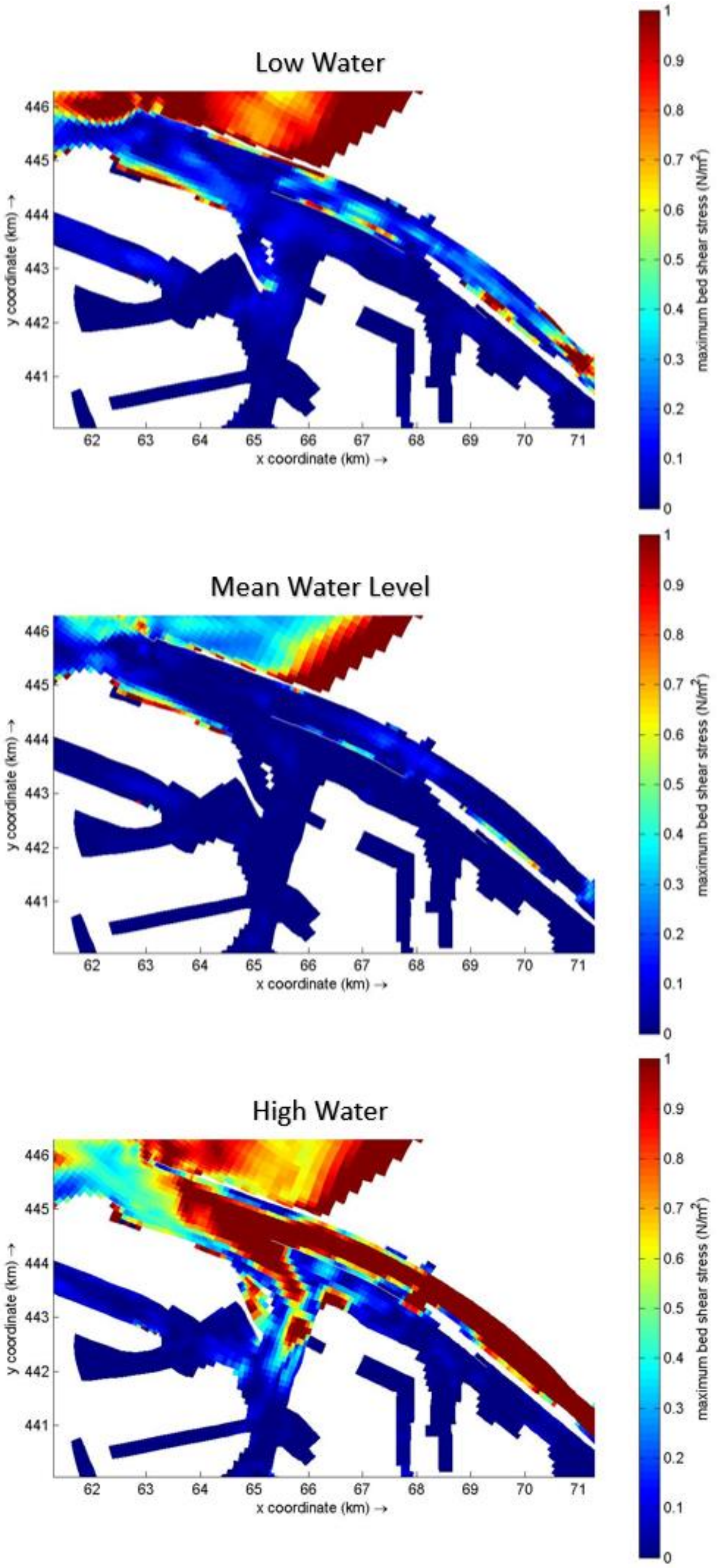

Figure 3.5 Bed shear stress at the harbour area during different water levels. There is no exact correlation between the water level and the bed shear stress. It was observed that the bed shear stress at the Nieuwe Waterweg and Maasmond is higher than the harbour basin.

Bed shear stress at the Maasmond and T-Shirt is relative higher than the harbour basin. It seems likely that the high bed shear stress at the T-Shirt is caused by flow deceleration and acceleration caused by flow expansion at the intersection. As it goes further away of the T-Shirt, the bed shear stress decreases, observed at the Calandkanaal and Beerkanaal, so the sediment will possibly settle in these locations.

Based on the flood-dominated conditions and the bed shear stress, it is expected that sediment will be transported towards the harbour basins and deposited at the locations with relatively lower bed shear stress like at the Beerkanaal and the Calandkanaal. The latter is the location where Port of Rotterdam built a sediment trap where the sediment is expected to be deposited. (Figure 3.6).

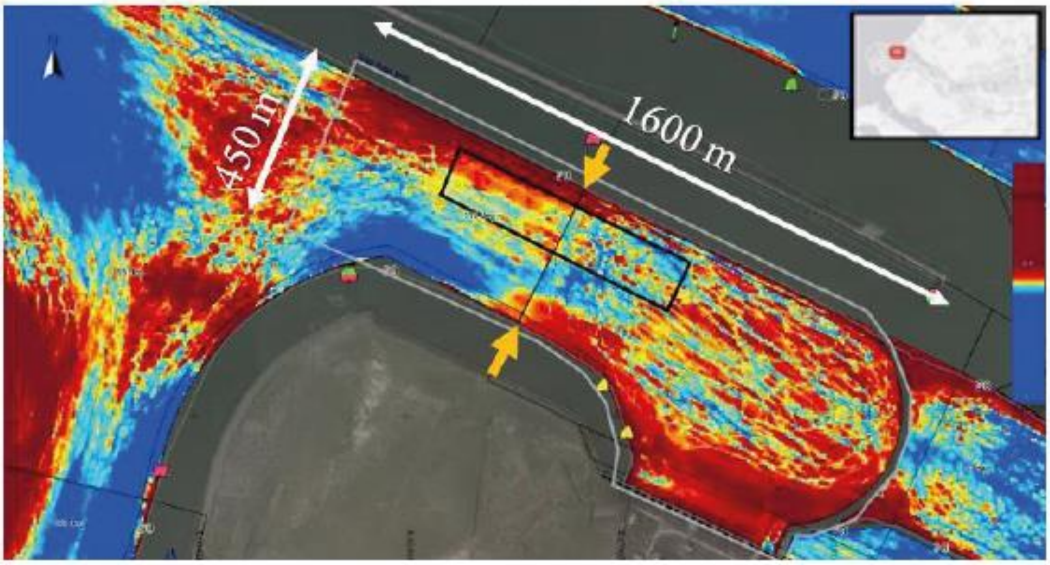

Figure 3.6 Location of the sediment trap at the Calandkanaal, where the bed shear stress is relatively low and the sediment is expected to be deposited. Based on the bed sheear stress map from Figure 3.5., it was observed that the sediment trap is located at the location where the bed shear stress is relatively lower compared to the surrounding area. (Kirichek & Rutgers, 2019)

3.2.4 Sediment Transport

Sediment transport is analyzed by creating cross-sections in the model in some locations (Figure 3.7):

1. Maasmond : at the meeting point of the Maas River and the North Sea.
2. Harbour Entrance : at the east boundary of the harbour area.
3. Breeddiep : at a channel connecting the harbour area and Nieuwe Waterweg.
4. NW West : at the downstream of Nieuwe Waterweg.
5. NW East : at the east side of Nieuwe Waterweg, which indicates the upstream of Nieuwe Waterweg.

These cross-sections are created to monitor the volume and the direction of sediment transported through the locations. After two weeks simulation, the volume of sediment passing each cross-sections can be seen at Table 3.1.

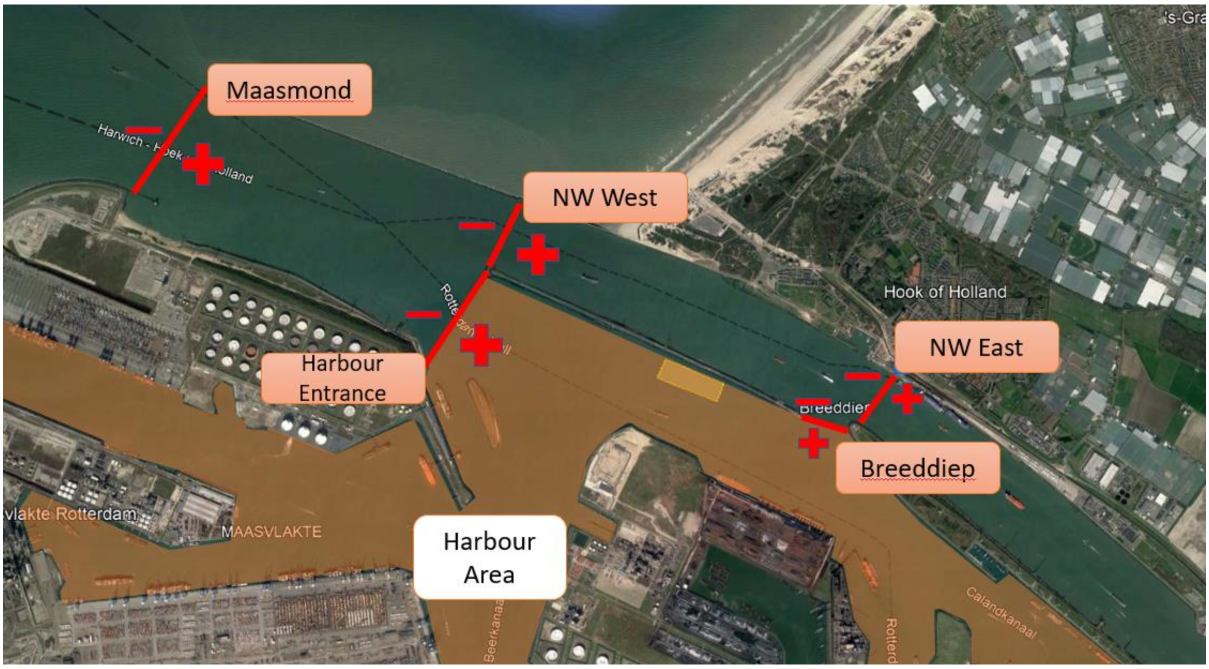

Figure 3.7 Locations of the Cross-sections. The positive and negative sign indicates the direction of sediment transport through cross-sections. Brown coloured area indicates the harbour area in this thesis, where sediments transporting back to this area is counted as a sediment return.

Table 3.1 Sediment transport volume passing through each cross-sections. The positive and negative sign shows the direction of the sediment transport flux. It was observed that the sediment transported to the flood direction from the North Sea to Maasmond, and then transported to the harbour basin via Harbour Entrance or to the Nieuwe Waterweg via NW West. All numbers are in m^3.

Cross-section	Sediment Transport Volume
Maasmond	53703
Harbour Entrance	35372
Breeddiep	-595
NW West	12884
NW East	9177

The positive value at the Maasmond cross-section means sediment is transported from the North Sea to the port area. The sediment then transported more upstream with the majority of them is transported to the harbour area via Harbour Entrance cross-section and the rest towards the Nieuwe Waterweg via NW West cross-section. This condition suits the finding of de Nijs (2012), which the bed elevation at the Nieuwe Waterweg is higher than the bed level at the Maasmond and at the harbour area and reduce the bedload transport volume to the Nieuwe Waterweg. This condition leads to a smaller amount of bed load transport to the Nieuwe Waterweg in comparison to the harbour area.

From Nieuwe Waterweg, the sediment is largely transported to the upstream to the from NW West cross-section. There is a small amount of sediment transported from the harbour area to Nieuwe Waterweg via Breediep. While the flow velocity is relatively high at Breeddiep, there is a hump on the bed which interfere the bedload transport.

In a tide-dominated channel, tidal asymmetry cause an imbalance sediment transport between ebb and flood. A difference between maximum ebb velocity and maximum flood velocity induces an asymmetry in sediment transport as the sediment transport capacity is proportional to a power of 3 for cohesive fractions and power of 4 for non-cohesive fractions (Gatto, 2017). Figure 3.8 shows the relation of the flow velocity at the bed of Nieuwe Waterweg (Figure 3.3 location A) and the cumulative sediment transport volume at the NW West cross-section. The sediment volume tends to increase during flood and decrease during ebb. The trend of the sediment volume is going up means there is more sediment to the flood direction than ebb direction, proves the sediment transport asymmetry in this location.

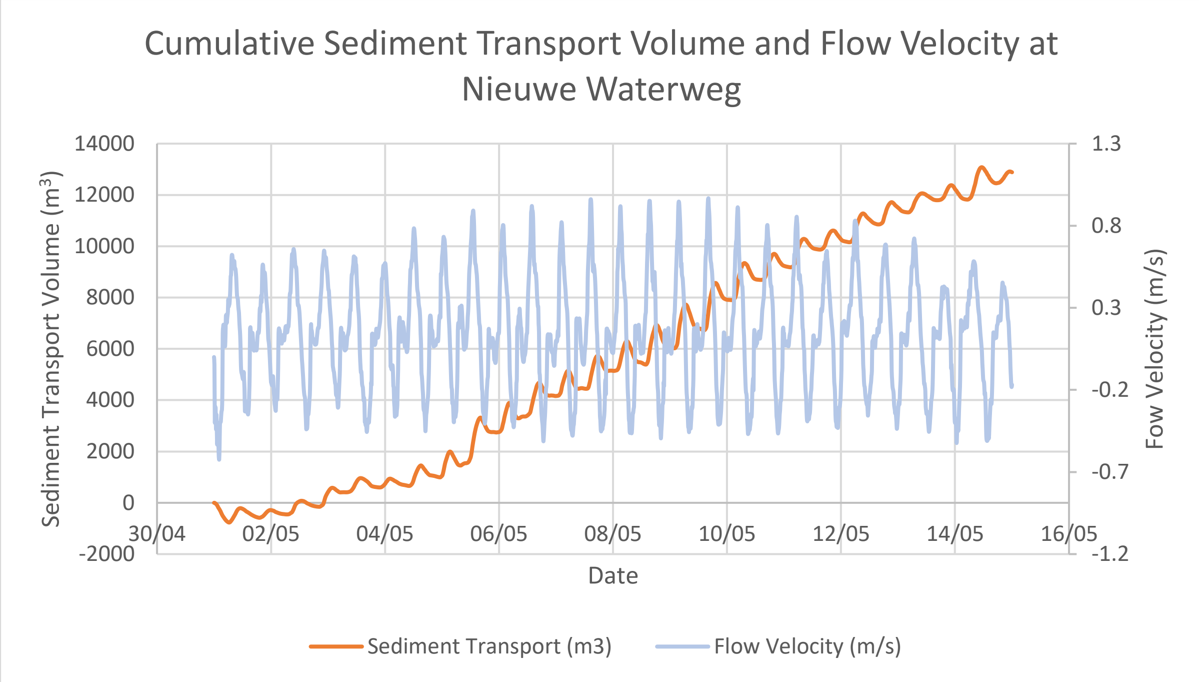

Figure 3.8 The correlation between flow velocity and the cumulative sediment volume. It was found that during flood currents (positive flow velocity), sediment tends to move upstream (positive sediment transport volume). The flow velocity is measured at the bed (layer 10) at the Nieuwe Waterweg. The cumulative sediment volume is measured at the NW-West cross-section.

3.3 Sediment Return Analysis

3.3.1 Sediment Reallocation Site Alternatives

Three alternatives for the sediment reallocation site (Figure 3.9):

1. Inside of Nieuwe Waterweg
2. Outside of Nieuwe Waterweg
3. Verdiepte Loswal

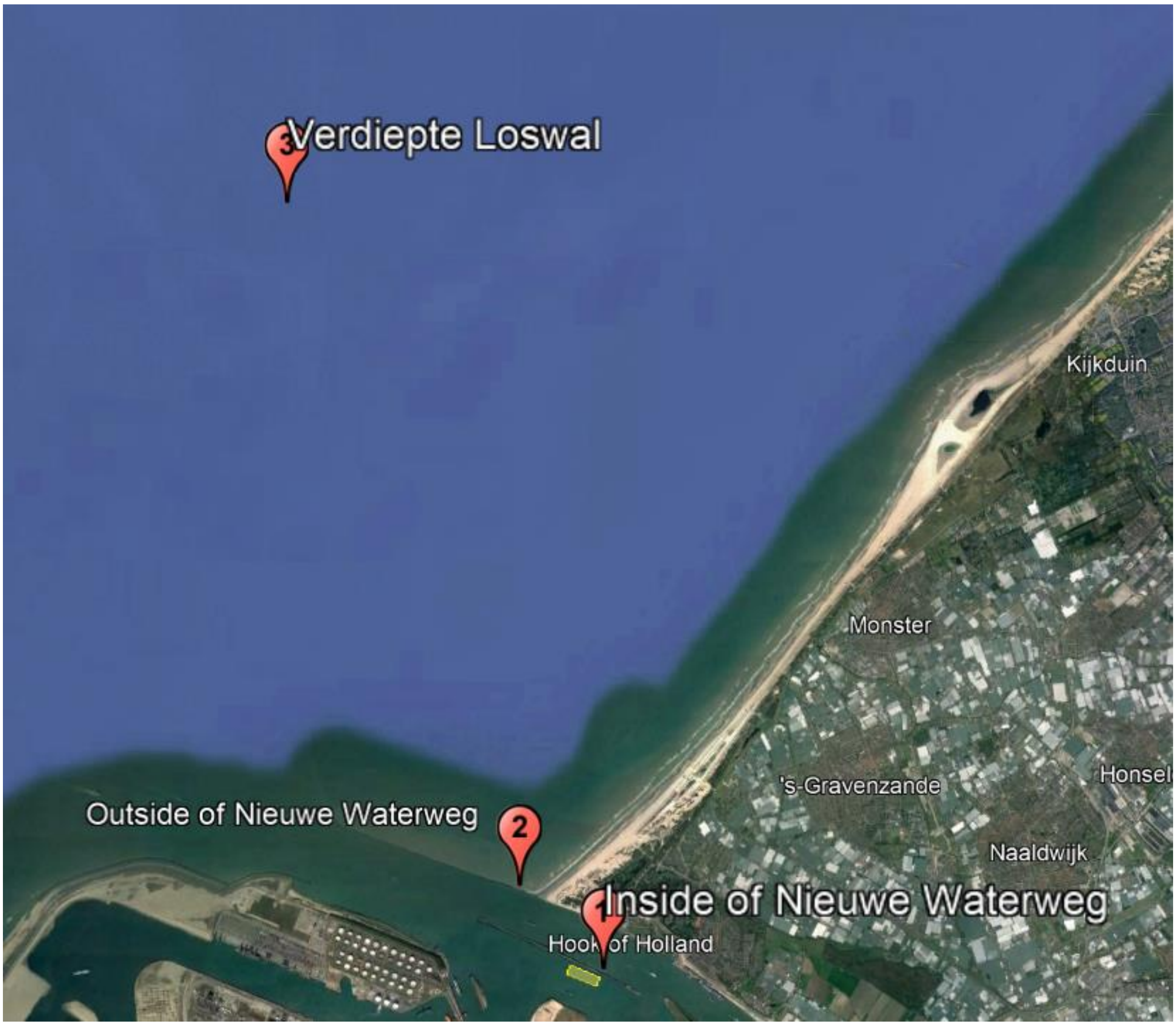

Figure 3.9 Location of the sediment reallocation site alternatives: Inside of Nieuwe Waterweg (1), Outside of Nieuwe Waterweg (2), and Verdiepte Loswal (3). The sediment trap is represented by a yellow coloured box.

3.3.1.1 Inside of Nieuwe Waterweg

Nieuwe Waterweg (New Waterway) is a ship canal that runs from Hoek van Holland to the Maeslantkering. The position of this reallocation site is inside at the Nieuwe Waterweg, 500 m from the sediment trap (Figure 3.9 location 1). The waterway is connected with Rhine river which leads to a residual current flowing to the sea due to the river discharge. This seaward current is expected to transport the sediment to the North Sea. Due to its position as a channel, the tidal flow direction at Nieuwe Waterweg can be distinguished as a downstream or a upstream flow. Two scenarios is simulated at Nieuwe Waterweg. The first scenario is if the discharge is done without considering the flow direction, which is done for 24 hours non-stop. The second scenario is only discharging on few ebb tide period but sums to 24 hour at the end.

3.3.1.2 Outside of Nieuwe Waterweg

Another alternative is to reallocate the sediment at the outside of Nieuwe Waterweg, which is at the north side of Noorderpier, Hoek van Holland (Figure 3.9 location 2). The distance to the sediment trap is 2000 m.

The existence of Noorderpier structure is expected to prevent the sediment transported back to the harbour area once it is discharged. Two simulations are done at this location with different discharge elevation, at the surface and near the sea bed.

In this thesis, this location is assessed for didactical reasons (to analyze an intermediate distance between the previous and the next one).The practicability of this location is not a topic of this thesis, but should carefully be taken into consideration given its sensitive environment-socio-economic location.

3.3.1.3 Verdiepte Loswal

Verdiepte Loswal (Lowered Relocation Area) is a location used by Port of Rotterdam to reallocate the sediment from maintenance dredging (Figure 3.9 location 3). According to the past studies, this is a location where experience and research have shown that the relocation activity is not harmful for nature and does not obstruct other functions of use (Hendriks & Schuuman, 2017).

3.3.2 Numerical Model Setup

Discharge operation feature at Delft3D-Flow is used to simulate the pipeline outtake for each location. The central suction is assumed to be pumping the sediment once the fluid mud enters the sediment trap. The pipe discharge is assumed to be 1 m^3/s. The density of fluid mud is 1150 kg/m^3 or equivalent to approximately 200 kg/m^3 sediment concentration. This concentration is used as the characteristic of the discharge in the model with half of the sediment is the sediment2 and the other half is sediment3 (see Table 2.1 for the settling velocity and the particle size for each sediment fraction class). Sediment1 is assumed to be not contained in the discharge as the particle size is considered too big for a fluid mud produced by WID. The discharge is set to start operating during ebb and continues for 24 hour non-stop (Figure 3.10).

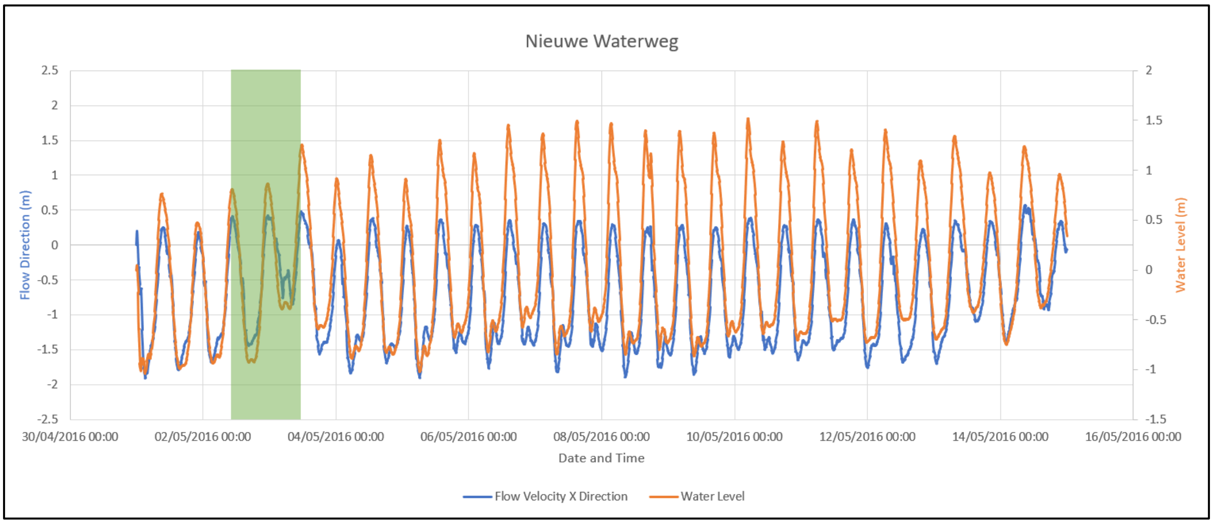

Figure 3.10 Flow Velocity and Water Level at Nieuwe Waterweg. The green coloured area represent a period where the suction system is discharging. The discharge operates for 24 hours non stop, which means it covered one daily tidal cycle.

3.3.3 Results

3.3.3.1 Discharge at the Inside of Nieuwe Waterweg

The discharge is modelled next to the separating breakwater (Figure 3.11) between the harbour basin where the sediment trap is located, and the waterway. The discharge is put at the surface (assumed to be 1 meter deep from the mean water level) as the ebb flow velocity is higher on the surface. There are 2 ways for the discharged sediment to return to the harbour area. The first is that the discharged sediment is transported towards the ebb direction by the flow at Nieuwe Waterweg until it reaches the location where the river discharge is no longer dominant, the sediment then can be transported towards harbour are by the flood current. The discharged sediment also can enter the harbour area via Breeddiep. (see Figure 3.7)

Based on the results of the numerical simulations, 52.7% of the reallocated sediment is transported to the west direction passing NW West cross-section, 25.4% is transported to the NW East cross-section, and 6.4% of the sediment transported to the harbour area via Breeddiep.

There is also 15.6% sediment deposited at the Nieuwe Waterweg channel. After 2 weeks of simulation, 24.1% of sediment is transported to the harbour basin via harbour entrance cross-section. Finally, adding this value to the amount of sediment from Breeddiep, it means that 30.4% of the sediment discharged by the suction system returns to the harbour basin. These results is summarized in Figure 3.11.

Figure 3.11 Percentage of sediment transported to each direction after discharged at Nieuwe Waterweg, considering continuous discharge over 24 hours. It is shown that 100% of the sediment is discharged at the discharge location (orange box) and then transported toward the directions represented by the white arrows. 15.6% of sediment is deposited at Nieuwe Waterweg (white box).

Figure 3.12 shows that at the beginning of the simulation, the difference in cumulative sediment transport volume is 0, it is the period before discharging. Once the suction system start discharging, the sediment transport volume differences increase gradually. A few days after the discharging , the graphs are flattening, indicating that only a small amount of the discharged sediment still being transported through the cross-sections. It can be assumed that the system has reached an equilibrium condition and that if the simulation time were longer, there would be no significant increase of the total sediment return volume.

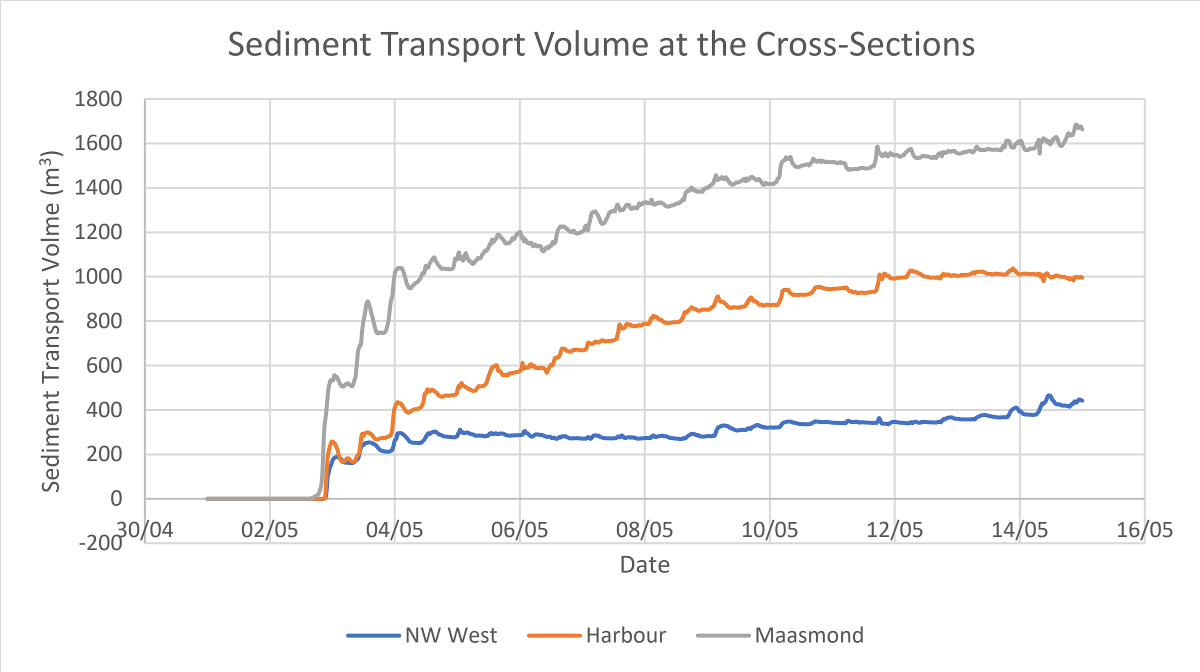

Figure 3.12 Sediment transport volume through a different cross-sections after discharged at the inside of Nieuwe Waterweg. It is observed that the graphs is flattening at the end of the simulation period, which means that no significant difference is expected if the simulation time is longer.

When the sediment is discharged at the surface, the sediment is floating at the water column and transported as a suspended sediment. During the ebb flow, the sediment is transported downstream towards the Maasmond area via NW West cross-section. If the sediment is still at the water column during flood tide, it is transported more upstream either to the NW East cross-section or towards Breediep. Even there is a elevation difference at the Breeddiep, suspended sediment can still transported to the harbour area.

The sediment that is settled at the Maasmond will be transported upstream by the flood tide. According to de Nijs (2012), majority of the sediment is transported as a bed-load transport and the bed level difference between the Nieuwe Waterweg and the Maasmond creates a situation where most of the sediment is transported toward the harbour area (see section 3.2.4), and considered as a sediment return.

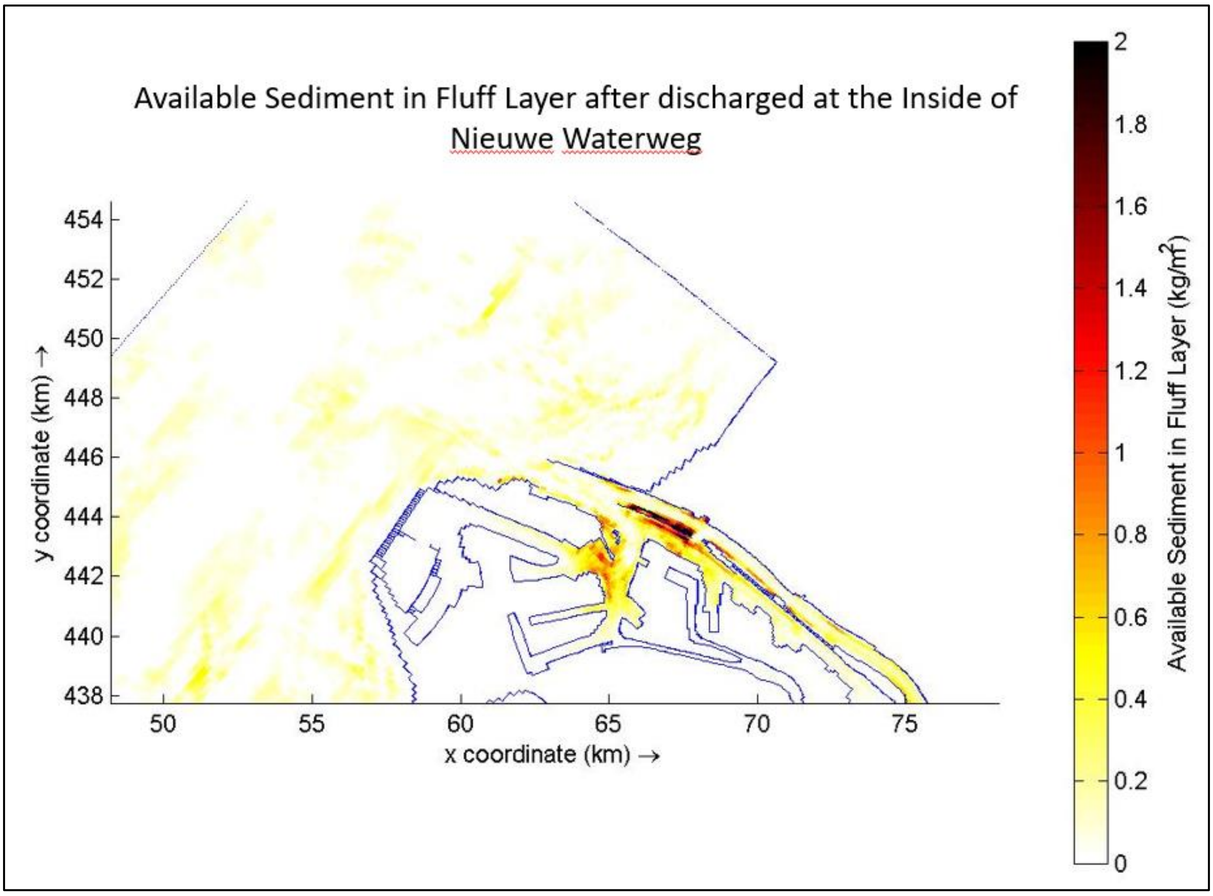

Figure 3.13 Return sediment after being discharged at Nieuwe Waterweg. Most sediment is settled either at the Nieuwe Waterweg or at the harbour area. There is no huge amount of sediment spotted at the North Sea.

Figure 3.13 shows locations where sediment was deposited after 14 days. In the Nieuwe Waterweg, sediments are spread along the channel. At the harbour area, it is observed that sediments settle at Beerkanaal and Calandkanaal, where the bed shear stress is relatively low. Figure 3.14 shows the comparison between settling locations and dredging heat map, which is map showing hot spots of dredging activity by Port of Rotterdam to validate the result of the numerical models. The dredging heat map has the same pattern with the locations where sediments settle in the numerical model. This comparison result shows that the sediment return deposited at the hot spot of dredging area, which are the locations where more sediments are deposited.

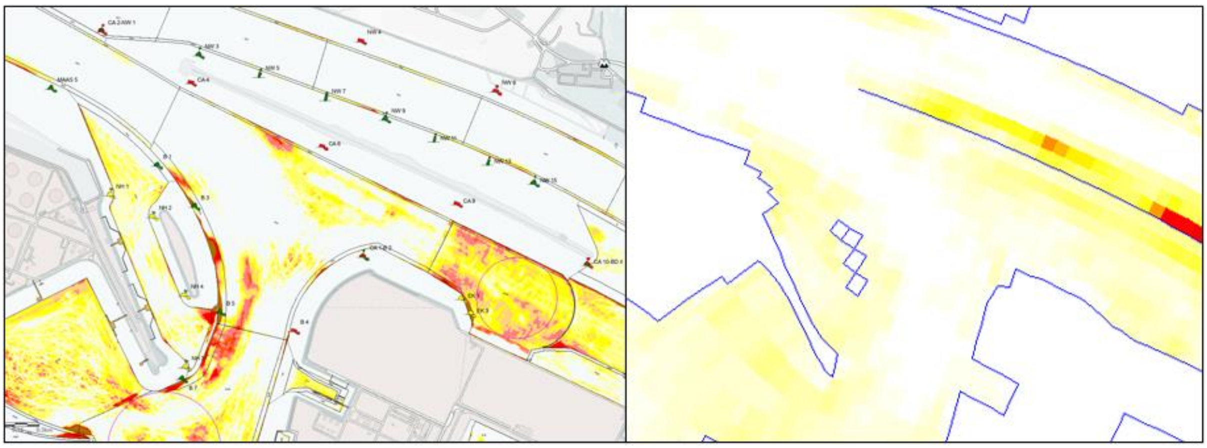

Figure 3.14 Dredging heat map (left) and the sedimentation after 2 weeks of discharging based on the numerical model (right). Red coloured means more sediment is deposited at this location. It is observed that the sedimentation pattern is similar between the map from Port of Rotterdam and the simulation results.

Different discharging schedule is simulated in the same location. Instead of discharging sediment for 24 hours without stopping, simulation of discharging only during ebb tide is done at Nieuwe Waterweg. Sediment is discharged at four different periods with duration of 6 hours each (Figure 3.15). The discharge has the same magnitude as the previous case (1 m^3/s). Discharging during ebb only creates a favorable scenario in which sediment is discharged when the flow is predominant to the downstream direction (North Sea).

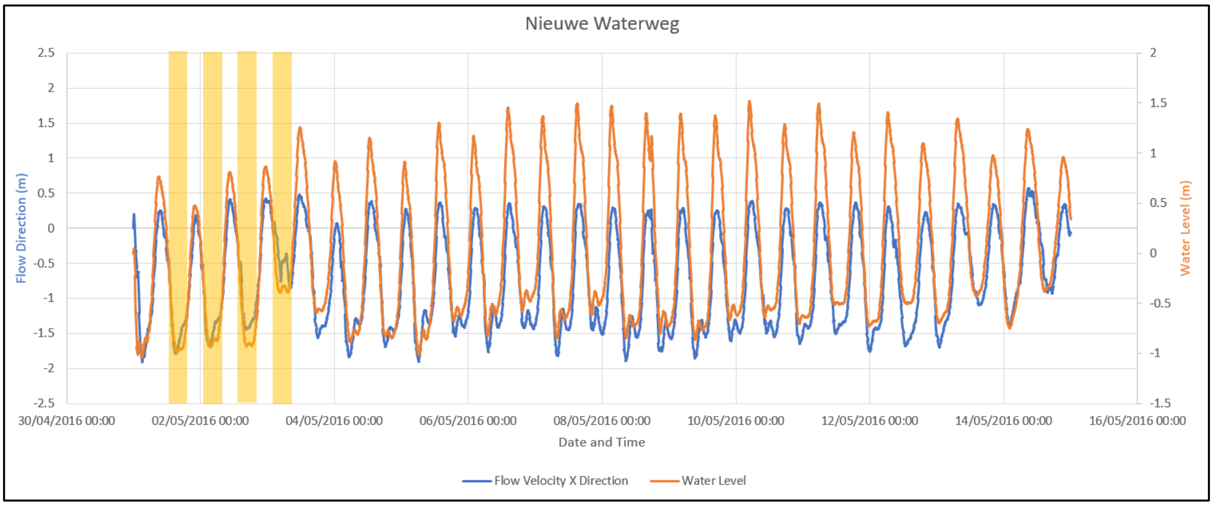

Figure 3.15 Discharging period during ebb tide only, when the flow velocity is negative. The yellow colored areas are the discharging period, each discharging period consists of 6 hours, which in total is 24 hours discharge time.

Discharging only during ebb tide leads to 30.27% sediment returning to the harbour area (Table 3.2, sums up of Harbour Entrance and Breediep) which is approximately the same as discharging during a period only (30.42%). However, the sediment volume that is deposited at Nieuwe Waterweg and transported upstream (via NW East) is lower compared to a scenario with one time discharge. To summarize, discharging during ebb tide only doesn't impact the amount of sediment return to the harbour area, but reduces sediment that settles in Nieuwe Waterweg. Table 3.2. shows this comparison

The sediment volume returning to the harbour via the harbour entrance is lower when discharging during ebb only. However, the amount of sediment transported to the harbour area via Breeddiep is higher. Since the discharging point is located on the surface (layer 1), the sediment stays at the water column once it is discharged as it needs time for the sediment to be deposited. Discharging only during ebb tide reduce the transported sediment to the upstream as the ebb current is much faster near the surface (see Figure 3.3), which is proven by lower sediment volume passes through Nieuwe Waterweg East cross-section. Once the sediment settles and reach the depth where the flood-dominated flow occurs, sediment will be transported to the flow direction. If the sediment settles when it has been transported to the Maasmond, then the most of the sediment will be transported to the harbour area due to a flood directed sediment transport flux. It's suspected that the ebb current at the Nieuwe Waterweg is not strong enough for the sediment to reach the North Sea

Since there is no significant reduction to the sediment return rate by discharging only during ebb, discharging one-time only would probably be preferred as it is easier to implement.

Table 3.2 Sediment return percentage comparison between discharging for one time and discharging only during ebb tide. It was observed that discharging only during ebb tide does not affect the sediment return.

Location	Ebb Only		1 Time Discharge	
	Vol (m^3)	%	Vol (m^3)	%
Deposited in Nieuwe Waterweg	841	12.7	1035	15.6
Nieuwe Waterweg West	4303	64.7	3502	52.7
Nieuwe Waterweg East	1017	15.3	1687	25.4
Breeddiep	485	7.3	422	6.3
Total	**6646**	**100%**	**6646**	**100%**

3.3.3.2 Discharge at the Outside of Nieuwe Waterweg

Two cases are tested in this location with the different position of the discharge: 1) at the surface (layer 1) and 2)at the sea bed (layer 10).

In case 1), after being discharged at the surface, the sediment first moved to the North Sea, and then to the harbour area. Approximately a quarter of the discharged sediment enters the Maasmond area, 15% of the sediment entered the harbour area and 6.6% sediments transported to the Nieuwe Waterweg. The percentage of sediment return volume is illustrated on Figure 3.16.

Figure 3.16 Return sediment percentage after being discharged at the outside of Nieuwe Waterweg. Location of the discharge is marked by orange box (100% sediment). After 14 days, 6.6% of the sediment is transported to Nieuwe Waterweg via NW West, 15% to the harbour area via harbour entrance. White box represent sediment that is deposited in Maasmond (3.4%) or distributed at the North Sea (75%).

Figure 3.17 shows that at the end of simulation, the trend of sediment volume is still slightly increasing, so possibly the sediment transport volume is higher if the simulation time is longer. The scheme on Figure 3.18 shows that the sediment is spread at the North Sea. There is sediments settle near the beach and around the Noorderpier.

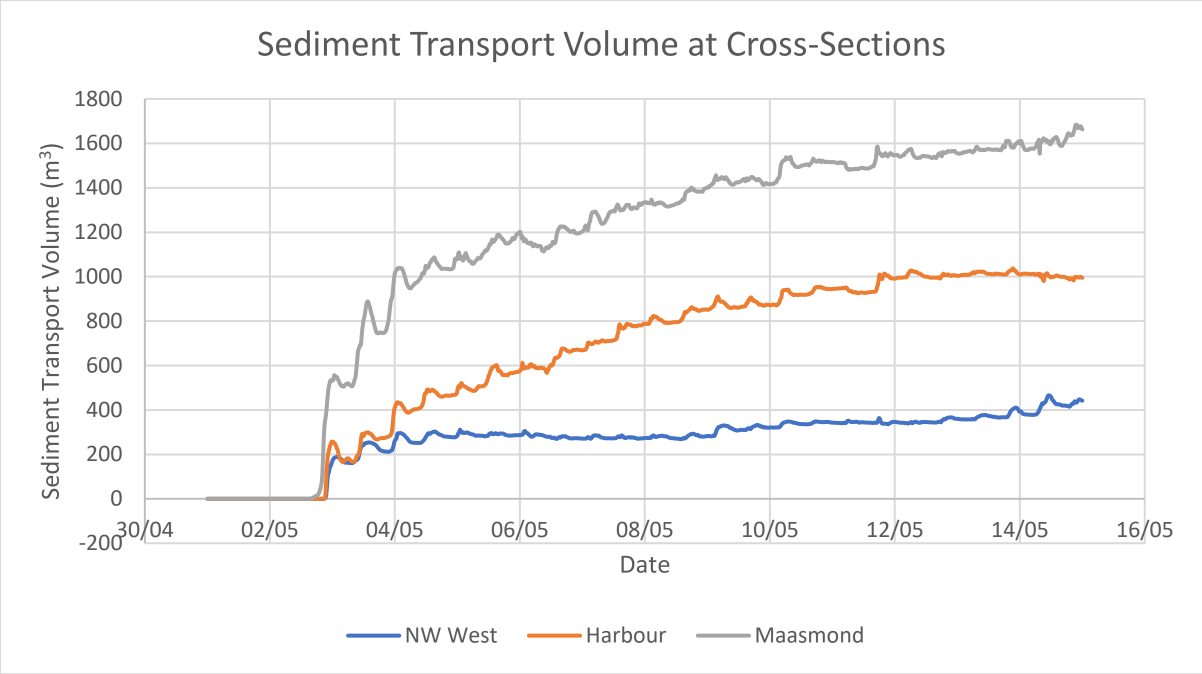

Figure 3.17 Sediment transport volume through a cross-section after discharged at the outside of Nieuwe Waterweg. The lines are still slightly increasing, especially at the Maasmond cross-section, which means there is a possibility that the sediment return volume can be higher if the simulation time is longer than 14 days.

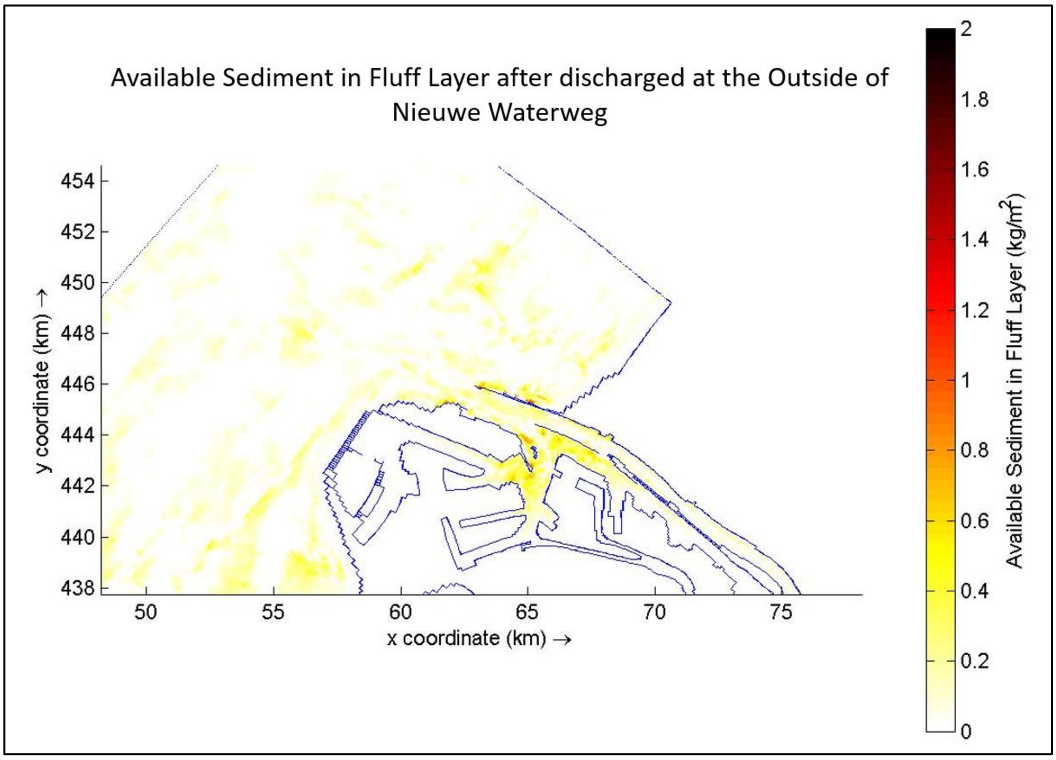

Figure 3.18 Return sediment after being discharged at the outside of Nieuwe Waterweg. A lot of sediment is spread at the North Sea, which means not all of the sediment goes to the harbour area.

The comparison between sediment transport volume on case 1) and case 2) is presented by Table 3.3. It is observed that in case 2), more sediment moves back to the harbour area and Nieuwe Waterweg. When the sediment is discharged at this location, the sediment has to be transported offshore until it moves further than tips of the Noorderpier Structure in order to be transported to the Maasmond by the flood current (illustrated in Figure 3.19). Discharging the sediment at the bottom means less time for the sediment to be transported to the sea due to the offshore directed return current at the surf zone (Bosboom, 2019). As the sediment transported offshore exceeding the tip of Noorderpier, it acts as a sediment supply from the North Sea and transported to the harbour area by the flood current. The trajectory of the sediment to enter the Maasmond cross-section is illustrated at Figure 3.19.

Table 3.3 The sediment return rate after being discharged at the surface and at the seabed of the outside of Nieuwe Waterweg

Cross-section	Discharge at Surface (case 1)		Discharge at Seabed (case 2)	
	Volume (m^3)	Percentage (%)	Volume (m^3)	Percentage (%)
Maasmond	1663	25.0	1787	26.9
NW West	441	6.6	411	6.2
NW East	327	4.9	342	5.2
Harbour Entrance	995	15.0	1101	16.6
Breeddiep	10	0.2	25	0.4

Figure 3.19 Trajectory of sediment movement in order to enters the Maasmond area. The Noorderpier structure (red dotted lines) prevent the sediment from move directly to the harbour area. After being discharged, the sediment is transported offshore until it passes the edge of the Noorderpier structure and return to the harbour area.

3.3.3.3 Discharge at Verdiepte Loswal

The suction system outtake at Verdiepte Loswal is placed at the bottom near the sea bed to reduce sediment settling time since it is the location where sediment is intended to be deposited.

At the end time of the simulation, it is found out that most of the sediments discharged are spread at the North Sea and only small part reentered the harbour area. The results from numerical model indicates that 5% from the discharged sediment pass through the Maasmond cross-section and 4.1% sediment return to the harbour via harbour entrance cross-section. There is small portion, 0.2% sediment transported to the Nieuwe Waterweg. Based on 2 weeks simulation, Figure 3.21 shows the sediment return rate is relatively going up. It means that there is a possibility the sediment return rate is higher if the simulation time is longer.

A past study indicates dumping fine sediment at around the Loswal area leads to 36% sediment return rate (Hendriks & Schuurman, 2017). The past study used a different numerical grid size and hydrodynamic boundary condition. In this study by (Hendriks & Schuurman, 2017), the simulation was done for 1 year with 1 million cubic metres of finer sediments were discharged every month. With longer simulation period, there is more time for the sediments to reach the harbour area (Figure 3.21 shows that after 14 days of simulation, the sediment return is still increasing). Hendriks & Schuurman (2017) used a finer sediment, represented by the smaller settling velocity (0.001 mm/s, 0.125 mm/s, and 1 mm/s). Finer sediment is easier to be transported, so the sediment is easier to be brought to the harbour area instead of deposited at the reallocation site or at the North Sea due to a favourable hydrodynamics for importing sediment to the harbour area. Higher amount of sediment also can contributes to the volume of sediment returning form the Verdiepte Loswal as the location act as a sediment trap. If the capacity of the pit is exceeded, more sediment is expected to return to the harbour instead of be trapped at the sediment trap.

Storm conditions are not considered in this thesis, which according to (de Nijs, 2012) such an event can leads to 0.5 Mton sediment transported to the harbour area. The period of May 2016 is chosen as it represents the average condition (Kronin, 2019), which means that seasonal variation is not considered in this thesis. De Nijs (2012) mentioned that rougher conditions during autumn and winter can increase the sediment transport towards the harbour area.

Figure 3.20 Return sediment percentage after being discharged at Verdiepte Loswal. Most of the sediments (95%) is spreaded at the North Sea while 0.7% of the discharged sediment deposits at the Maasmond. 4.1% of the sediment is transported back to the harbour area as sediment return while 0.2% of the sediment is transported to Nieuwe Waterweg.

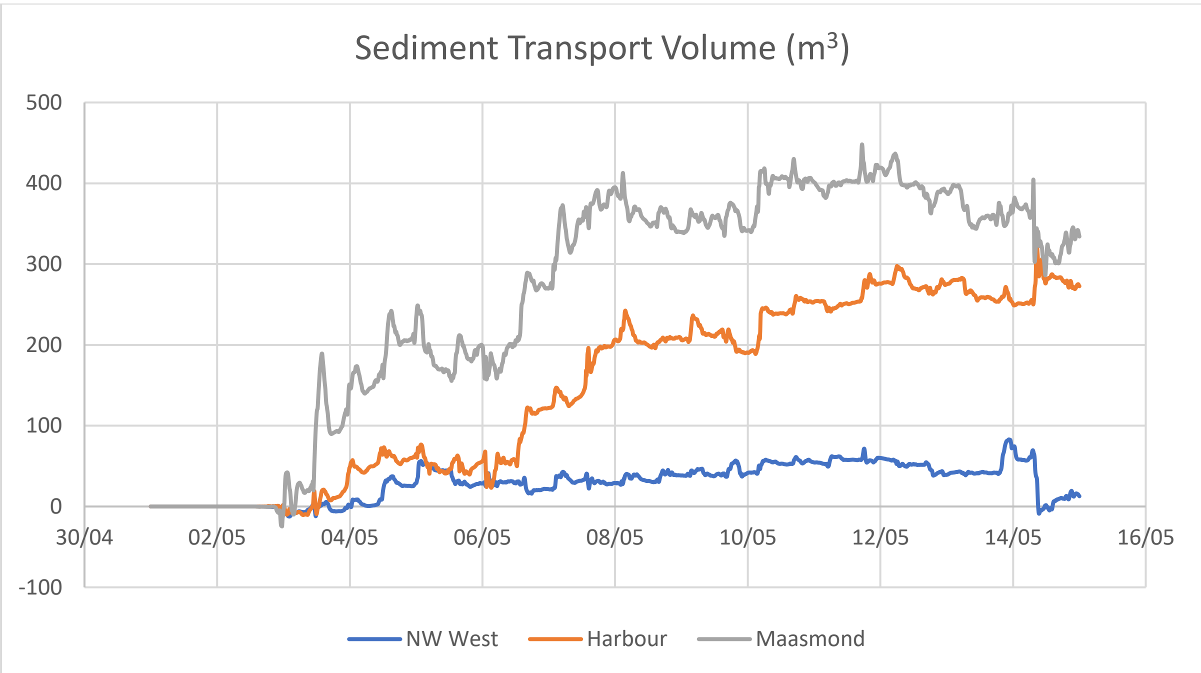

Figure 3.21 Sediment transport volume through a cross-section after discharged at Verdiepte Loswal. It can be seen that the sediment transport volume is still fluctuating at the end of the simulation

period, which means that there is a possibility the volume is higher if the simulation period is extended.

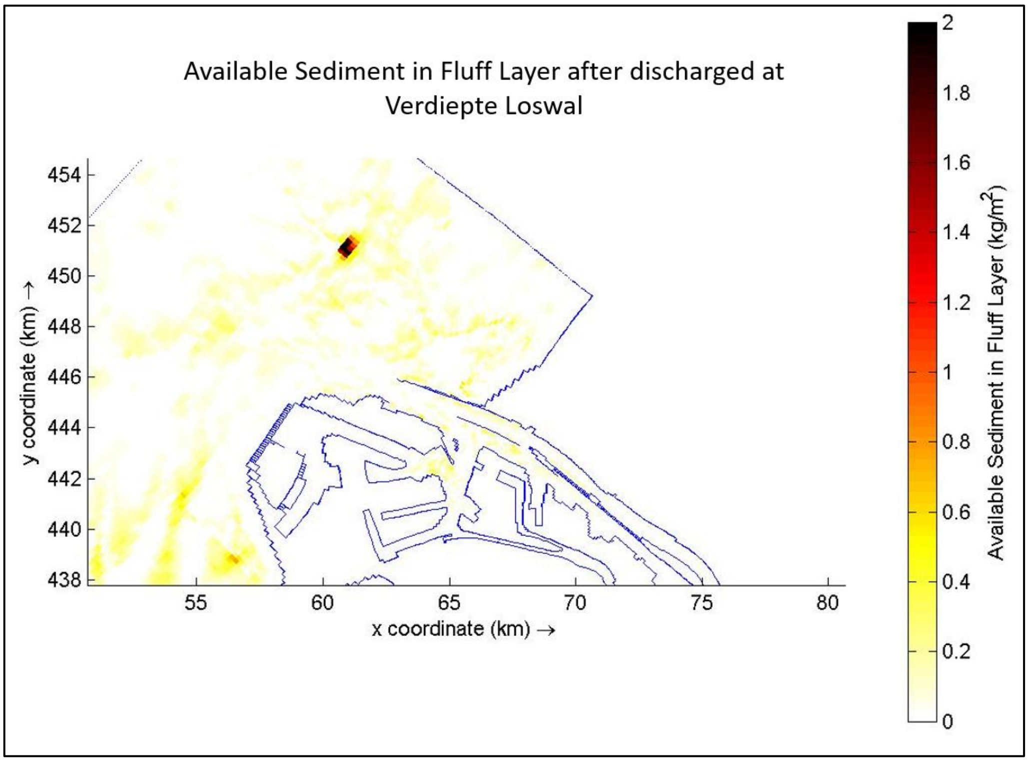

Figure 3.22 Return sediment after being discharged at Verdiepte Loswal. The sediment is concentrated at the Verdiepte Loswal, which acts as a sediment trap. Less sediment is observed return to the harbour area as most of it is spread at the North Sea.

3.3.4 Sensitivity Analysis

Sensitivity analysis are done for the scenario of discharging at the inside of Nieuwe Waterweg. This location is chosen due to its practicability compare to the other alternatives. In this sensitivity analysis, three different variables were tested:

1. Increase the discharge rate, keeping sediment concentration.
2. Increase the sediment concentration, keeping the discharge rate.
3. Reduce the sediment settling velocity.

3.3.4.1 Increase the Discharge rate

Simulation with the same setup as the one time discharge simulation at the section 3.3.1.1 is done with the discharge rate is increased to 2.5 m^3/s. Higher discharge rate means more sediment are discharged at the inside of Nieuwe Waterweg. The effect of different momentum due to different discharge rate is not included in the model. A comparison of the sediment return rate is presented at Table 3.4. It is found that increasing the discharge rate from 1 m^3/s to 2.5 m^3/s does not significantly affects the sediment return rate.

Table 3.4 Sensitivity of the sediment return rate percentage to the different discharge rate. The results shows no significant difference if the water discharge is increased.

Cross-section	Q=1	Q=2.5
Breeddiep	6.4%	6.1%
Harbour Entrance	24.1%	23.7%
Total Sediment Return	30.5%	29.8%

3.3.4.2 Increase the Sediment Concentration

The sediment concentration is doubled from 200 kg/m^3 to 400 kg/m^3, which means the volume of the discharged sediment is higher. Table 3.5 shows the sediment return rate comparison. It is observed that increasing the sediment concentration did not significantly change the sediment return rate.

Table 3.5 Sensitivity of the sediment return rate percentage to the different sediment concentration. The results shows no significant difference if the sediment concentration is increased.

Cross-section	Sed Concentration	
	200 kg/m3	400 kg/m3
Breeddiep	6.4%	6.3%
Harbour Entrance	24.1%	24.5%
Total	30.5%	30.8%

3.3.4.3 Decreasing the Sediment Particle Size

The discharge on the default setting with two fraction of sediments as it is shown at section 3.4. For the sensitivity analysis, the discharge has only one fraction of sediment, which is 200 kg/m^3 of sediment3. It means the discharge has the same amount of sediment, but with smaller grain sizes. Table 3.6 shows the effect of lower sediment settling velocity to the sediment return rate.

Table 3.6 Sensitivity of the sediment return rate percentage to a lower sediment settling velocity. The results shows no significant difference if the particle size is reduced until 24 microns.

Location	**Volume (m^3)**		**Percentage**	
	Default	**Low Set**	**Default**	**Low Set**
Breeddiep	422	338	6.4 %	5.1 %
Harbour Entrance	1600	1748	24.1 %	26.3 %
Total	2022	2086	30.5 %	31.4 %

4 Pump Power Necessary for Transporting Sediment

This chapter determines the necessary pump power and energy consumption for the suction system. The results from this chapter answer the research question R.Q.2

4.1 Methodology

A suction pump and pipeline is used for transporting fluid mud produced by WID from the sediment trap to the reallocation site. The intake of the pipe is located at the sediment trap and the outtake is located at the reallocation site (see section 3.3.1). Figure 4.1 shows an illustration of the suction system. The necessary pump power is determined by calculating the total of energy loss inside the pipeline, which are major loss, minor loss, and head loss from a vertical pipeline.

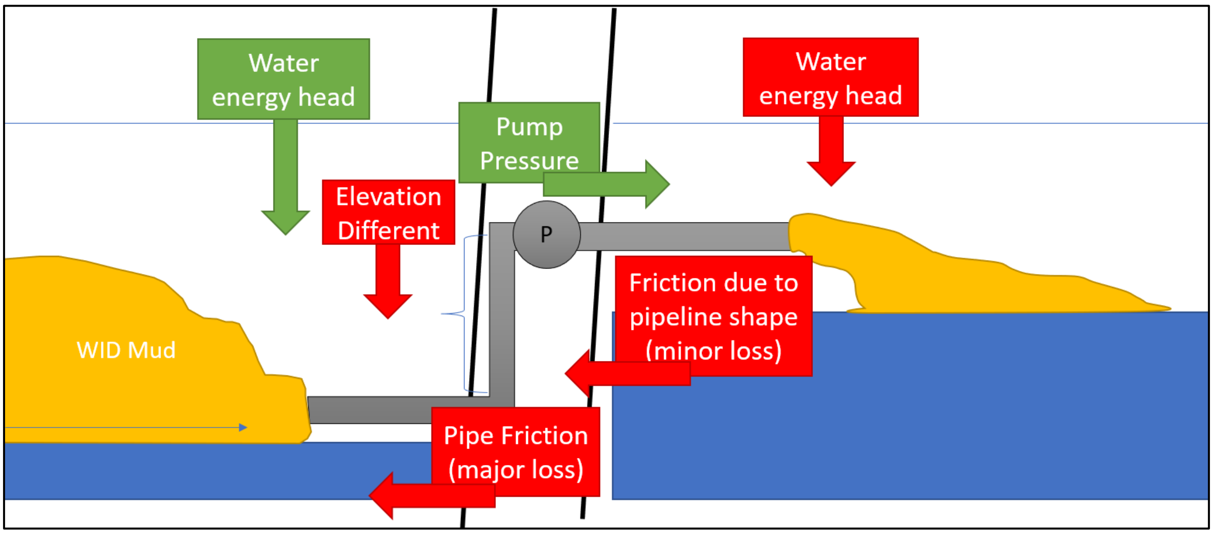

Figure 4.1 Illustration of a suction system scheme. Green coloured boxes indicate pressures that provide energy to transport the mixture to the reallocation site and the red coloured boxes represent pressure losses transporting sediment to the reallocation site and the red coloured pressures are pressure that preventing the fluid from flowing to the reallocation site

The calculation method in this section refers to the lecture notes for Dredging Pump and Slurry Transport course held by TU Delft on period 2020-2021 (Matousek, 2004).

The suction system transports the sediment from the sediment trap to the reallocation site by using a pump and pipeline system. The pump power necessary is determined by calculating the energy head loss inside the pipeline. Depends on the velocity of the flow, a laminar or a turbulence flow can occur during transporting a fluid inside a pipeline. Bingham Reynolds number is used as the parameter for determining whether a flow in a pipeline is in a laminar or turbulence regime. The threshold for laminar flow is similar with the normal Reynolds number for an open flow, which should be lower than 2100. Bingham Reynolds number is used as it also considers the effect of the wall pipe to the flow. Bingham Reynolds number can be calculated by using the equation (4.1).

$$Re_B = \frac{\rho_m V_m D}{\eta_B \left(1 + \frac{\tau_y D}{6\eta_B V_m}\right)} \tag{4.1}$$

Where:

- Re_B : Bingham Reynolds number [-]
- ρ_m : Mixture density [kg/m^3]
- V_m : mean mixture velocity in a pipe [m/s]
- D : Pipe diameter [m]
- η_B : tangential viscosity of Bingham plastic mixture [Pa.s]
- τ_y : yield stress [Pa]

The threshold value for the Bingham Reynolds number as a laminar flow is lower than 2100, similar to Reynolds number for a Newtonian flow. From the Bingham Reynolds number, an empirical approach is used for calculating the transition velocity threshold, which is the value of flow velocity at the transition between laminar and turbulence flow, can be calculated by equation (4.2) (Matousek, 2004).

$$V_T \approx 26 \sqrt{\frac{\tau_y}{\rho_m}} \tag{4.2}$$

Where:

- V_T : transition velocity [m/s]

Knowledge of a flow pattern in a dredging pipeline is important for the design and prediction of operational parameters of a dredging pipeline such as the flow velocity, production rate, and energy consumption. The flow pattern is determined by looking at the stratification between the solid materials and the carrier liquid. A tendency of solid particles to settle in a flow carrying liquid and a tendency of a flowing carrier to suspend solid particles are the most important indicators of a pattern of a flow of solid-liquid mixture in a pipeline (Matousek, 2004). The mixture flow is considered fully stratified if intensity of turbulence of a carrier flow is not sufficient to suspend any solid particle in a pipeline. The opposite extreme of the fully-stratified flow is a fully suspended flow in which all solid particles are suspended within a stream of a carrying liquid. In this case, there is no granular bed occurs in a pipeline. The fully suspended flow may be considered pseudo-homogeneous if a distribution of solid particles across a cross-section of a stream is almost uniform. This is usually the case of solid particles of silt or clay size are transported in a pipeline. An intermediate flow pattern, the partially stratified flow, is most usual during dredging operations. A mixture flow exhibits a considerable concentration gradient across a pipeline across-section indicating an accumulation of a portion of solids near the bottom of a pipeline and a non-uniform distributing of the rest of solids across the rest of a pipeline cross-sectional area is also known as a heterogeneous flow. The illustration of each flow type can be seen at Figure 3.10.

Fluid mud produced by a WID consists of mainly flocs. Mixture consists of flocs is practically non-settling and the pseudo-homogeneous character of the mixture flow is maintained at all operational velocities in a pipeline. Pseudo-homogeneous mixture flows (Figure 3.10, case IV) experience no (or at least very weak) accumulation of particles near the bottom of a pipeline, thus the deposition-limit velocity is an irrelevant parameter to predict (Matousek, 2004) as the pipe blockage due to stationary bed caused by deposited solids is not expected to happen. If the suction pipeline is used for transporting mixtures of coarser sediment like sands and gravels, a deposit velocity threshold, a minimum velocity to prevent the sediment from settling and create a stationary bed, should be analysed as it can cause a pipe blockage.

4.1.1 Head Loss in Straight Pipeline (Major Loss)

Major losses are the head lost due to flow friction in a straight pipe. The frictional head loss in water flow is determined using Darcy-Weisbach equation (Matousek, 2004). For mixture flows, the frictional losses can be determined by predicting the hydraulic gradient and this is interpreted as the head lost along a pipeline of a specific length. The major head loss, $H_{maj,}$ can be calculated by equation (4.3).

$$H_{maj} = I_m L \tag{4.3}$$

Where:

- H_{major} : Head loss due to friction of water in a straight pipe [m]
- I_m : Hydraulic gradient in mixture flow according to a suitable model [-]
- L : length of a pipe [m]

4.1.2 Head Loss in Flow Through Fittings (Minor Loss)

Fittings as bends, joint balls, expansions and contractions of a discharge area, valves, and measuring instrument act as obstructions to the flow. The pipeline inlet and outlet are also sources of local losses. Disturbances cause flow separation and an induced mixing process in the separated zones dissipates mechanical energy. This energy dissipation due to a presence of fittings is usually considerably smaller than frictional losses in straight pipes.

The minor losses is equivalent with quadratic of the flow velocity through a fitting and the density of the mixture (Matousek, 2004). A correct determination of minor losses for mixture flows is complicated, particularly for stratified flows. In a dredging practice, a simple assumption is often applied that mixture density alone sufficiently represents an effect of solids on the minor loss. The minor head loss, H_{min}, can be determined by equation (4.4).

$$H_{min} = \xi \frac{V_m^2}{2g} \frac{\rho_m}{\rho_f} \tag{4.4}$$

Where:

- H_{minor} : Minor head loss[m]
- ξ : Coefficient of minor losses [-]
- V_m : Flow velocity [m/s]

- g : Gravity acceleration [m/s^2]
- ρ_m : Mixture density [kg/m^3]
- ρ_f : Fluid density [kg/m^3]

Values of the coefficient ξ may vary between zero and one for different fittings. More details about the minor loss coefficient can be seen at Appendix D.

4.1.3 Head Loss from the Vertical Pipeline

A change of a pipeline elevation gives to arise to the static pressure differential caused by a pressure exerted by a mixture column of a height given a vertical distance between the begin and the end of a pipeline section. In a vertical pipeline, uniform distribution of solids across a pipeline cross-section, especially with a very fine particles like silt and clay. The "equivalent liquid" method is used to predict the head loss , which means the mixture has the density of the mixture but other properties remain the same as in the carrying liquid alone, in this case is water. With this condition, Darcy-Weisbach approach can be used to predict the head loss, as presented by equation (4.5)

$$H_{vert} = \lambda_f \frac{\Delta h}{D} \frac{v_m^2}{2g} \tag{4.5}$$

Where:

- H_{vert} : Head loss from a vertical pipeline [-]
- λ_f : Friction factor [-]
- Δh : Elevation difference [m]

4.1.4 Total Pressure Loss

The total frictional head loss is a sum of head losses due to friction in a straight pipeline sections and in fittings mounted to a pipeline. Total pressure losses, Δp, can be calculated by equation (4.6).

$$\Delta p = \rho_m g\left(H_{maj} + H_{min} + H_{vert}\right) \tag{4.6}$$

Where:

- Δp = Total pressure loss [pa]

4.1.5 Pump Power and Energy Consumption

The pump installed has to be capable of exceeding the pressure losses in order to keep the mixture flowing, which means it has to provide enough power to overcome the pressure losses. The necessary pump power can be calculated by equation (4.7).

$$P = \frac{Q\,\Delta p}{\eta} \tag{4.7}$$

Where:

- P : Necessary pump power [watt]
- Q : Discharge rate [m^3/s]
- η : Pump efficiency [-]

Total energy consumption can be determined by equation (4.8).

$$E = P\,t \tag{4.8}$$

Where:

- E : Total energy consumption [kWh]
- t : Operating time [hour]

4.2 Results

Some assumptions are needed for the feasibility design in order to make the calculations as the real specifications are still unknown. In the real project, these assumptions has to be replaced by the real specifications. The assumptions made are:

- Annual dredging volume is assumed to be 1.8 million tonnes dry solid (tds) which equals to around 5.45 million m^3. This number is chosen based on the annual dredging volume history at the investigated location as it relatively a huge dredging volume.

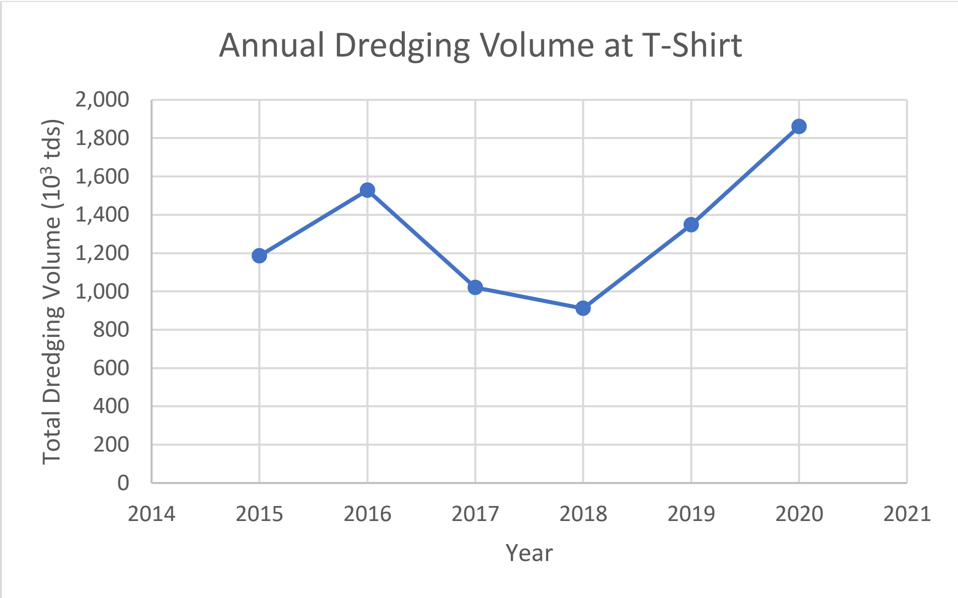

Figure 4.2 Annual dredging volume at the T-Shirt, which is the investigated area. Based on this data, 1.8 million tds dredging volume is used in this thesis. (A. van Hassent, 2021. Personal Communication)

- The flow inside the pipeline is assumed to be a laminar flow, which means energy losses due to a turbulence are neglected. Turbulence flow is not assessed in this thesis as it is complicated to be predicted. Physical research is recommended to be done for a turbulence flow analysis.
- Pipe diameter is assumed to be 1.5 meter with discharge of 2.5 m^3/s. This is considered so that the system can reallocate 1.8 million ton dry solids (tds) sediment in within a year. It takes 7636 hours per year to pump the total amount of the fluid mud from the harbour area.
- The density of fluid mud is assumed to be constant at 1150 kg/m^3 and the yield stress is 10 Pa. This assumption is made based on the characteristic of mud produced by a WID 1 week after the dredging works (Kirichek & Rutgers, 2019).
- Whether a laminar or turbulence flow occurred in a pipeline is determined by calculating the transitional velocity, which is the threshold velocity between turbulence

and laminar flow (Matousek, 2004). Reynolds number of the pipeline is also calculated. For a flow in a pipeline, Bingham Reynolds Number, Re_B is calculated. Discharge rate of 2.5 m^3/s is calculated as a laminar flow. Detailed calculation can be found at Appendix D. The Bingham Reynolds number with this fluid characteristic is calculated as 1353, which is lower than 2100, the threshold of laminar flow.

- Pump efficiency is assumed to be 0.6. In the real project, the pump performance has to be analysed in order to know the efficiency of the pump.
- Minor loss coefficient is determined by the configuration of the pipeline. Since the material and the configurations of the pipeline is hard to predict, it is assumed that the minor loss coefficient is 2. This number can be overestimate compared to the real plan.

Detailed calculation of pump power can be found at Appendix D. Table 4.1 shows the total frictional head loss and the pump power necessary to transport the fluid mud.

Table 4.1 Pump power, energy, and wind turbine necessary for each discharge location. The calculations shows that the major losses is the main cause of energy loss and it increases as the pipe length is longer.

Discharge Location	Pipe Length (m)	Elevation Difference (m)	Head Loss (m)			Pump Power (kW)
			Major	Minor	Vertical	
Inside of Nieuwe Waterweg	500	21	1.9	0.23	0.16	92
Outside of Nieuwe Waterweg	2000	21	7.3	0.23	0.16	319
Verdiepte Loswal	25000	0	90.2	0.23	0.00	3789

Pump is planned to be powered up by using an offshore wind turbines. One wind turbine can produce approximately 6 million kW per year (EWEA, n.d.). Total energy consumption and the wind turbine needed to provide energy for the pump is shown at Table 4.2.

Table 4.2 The pump power, annual energy consumption, and the wind turbines needed for powering the suction system. Each wind turbine is assumed to be able of supplying 6 MW energy annually.

Discharge Location	**Pump Power (kW)**	**Energy (MWh)**	**Wind Turbine**
Inside of Nieuwe Waterweg	92	701	1
Outside of Nieuwe Waterweg	319	2,433	1
Verdiepte Loswal	3789	28,932	5

5 Greenhouse Gases Production and Cost Analysis

This chapter is intended to provide answer for R.Q.3 In this chapter, the greenhouse gases emitted during the operation of the central suction-WID system and the necessary costs are assessed. Then, comparison is made between the central suction-WID system and TSHD which is regularly used for maintenance dredging by Port of Rotterdam at the investigated area.

5.1 Methodology

5.1.1 Greenhouse Gases Production Calculation

In this thesis, only the greenhouse gases comes from the dredging project are analysed. Any emissions produced outside from the dredging project, for example during the production phase of the wind turbine are not analysed. The pump on central suction-WID system is powered by electricity from renewable energy source (wind turbines) which produces no greenhouse gases on its operation.

The methodology used of greenhouse gases production analysis refers to van der Kurk (2021). The greenhouse gases production is equivalent to the total fuel consumed by the dredgers. In this thesis, only methane (CH_4), sulphur dioxide (SO_2), carbon dioxide (CO_2), and nitrogen dioxide (NO_2). These types of gases are selected due to their volume, atmospheric lifetime, and global warming potential. Different types of three marine fuels are assessed in this report:

- Marine Gas Oil (MGO), is a fossil fuel that is very similar to diesel.
- Hydroteated Vegetable Oil (HVO), is a type of biofuel that is often used for dredging activities. HVO is produced from animal oils.
- Liquified Natural Gas (LNG), is a fossil fuel obtained by cooling natural gas and turning it into a liquid.

For MGO and HVO, a distinction is made between different engine types that are typically installed on dredgers. Dredger that is built before the introduction 1997 (approximately the median of the Dutch dredging fleet), is referred as pre-tier 1. More modern dredgers built after that are referred as tier 2. These two engines types were differed because of the difference environmental impact. The greenhouse produced per 1 ton of fuel is shown at Table 5.1.

Table 5.1 Greenhouse gases consumption per ton marine fuel based on the fuel type and the dredging classification. Pre Tier I is used for dredgers built before 1997 and Tier II is for dredgers built afterwards. All numbers are in kg. (van der Kurk, 2021)

Gas/FuelType	MGO		HVO		LNG
	Pre Tier I	Tier II	Pre Tier I	Tier II	
Methane (CH4)	0	0	0.112	0.089	38.9
Sulfur Dioxyde (SO2)	2.57	2.63	0.03	0.03	0.09
Carbon Dioxyde (CO2)	3181.6	3182.8	3109.3	3109.3	2679.1
Nitrogen Oxides (NO2)	59.8	49	52.6	43.1	10.7

TSHD consumes 0.5 ton fuel per hour and the water injection dredger consume 0.16 ton fuel per hour (A. van Hassent, personal communication, 2022). The TSHD used is assumed to have a cycle time of 129 minutes and 4100 m^3 hopper capacity, based on the typical vessel size used by Port of Rotterdam for maintenance dredging. These numbers are then translated to 1907 m^3/hour of TSHD production. The production of the WID is assumed to be 2000 m^3/hour (A. van Hassent, personal communication, 2022). Based on the production rate and the hourly fuel consumption rate, it was found that TSHD consumes 0.26 kg/m^3 and WID consumes 0.08 kg/m^3, which is 69% lower than TSHD.

Annual dredging volume is assumed to be 1.8 million ton dry solids (tds), which is equals to 5.45 million m^3 (1 tds equals to 0.33 m^3 sediment). This number is chosen based on annual dredging volume at the investigated area (see Section 4.1).

5.1.2 Cost Analysis

Cost analysis is done to compare the cost needed for both central suction-WID system and dredging using TSHD. The analysis are only a rough estimate to compare each scenario of central suction-WID system and using a TSHD. The analysis is done for 25 years period.

Inflation is taken into account for the cost analysis, which is 3.3% per year based on the average of inflation rate in the Netherlands 1960-2021 (Worlddata, 2022).

For central suction-WID system, the cost consists of installation cost of the suction system and the operational costs while for dredging using a TSHD, only operational costs are considered as there are no additional things that has to be installed.

Investment cost for central suction-WID system consists of pipeline installation cost including the material costs and the capital cost for the pump. Operational cost of central suction-WID system is an hourly rate of renting a WID to do a dredging at a harbour basin and electricity cost used by the pump. Operational cost needed for TSHD is price per dredging volume.

5.2 Greenhouse Gases Production Analysis Results

The greenhouse gases production from fuel consumption analysis is based on (van der Kruk & Bolech, 2021). Three marine fuel types that are commonly used for dredgers in the North Sea are analyzed:

5.2.1 Emissions from WID

Production of water injection dredger for fine sediment is assumed to be 2000 m^3/hour (A. van Hassent, personal communication, 2022). This number has been confirmed by Port of Rotterdam. Table 5.2 shows the total greenhouse gases emissions produced by WID for annual dredging volume.

Table 5.2 Greenhouse gases produced by WID for 1.8 million m^3 of dredged sediment. All numbers are in kg.

Fuel Type	**MGO**		**HVO**		**LNG**
	Pre Tier I	**Tier II**	**Pre Tier I**	**Tier II**	
Methane (CH4)	0	0	49	39	16975
Sulfur Dioxyde (SO2)	1121	1148	13	13	39
Carbon Dioxyde (CO2)	1388335	1388858	1356785	1356785	1169062
Nitrogen Oxides (NO2)	26095	21382	22953	18807	4669

5.2.2 Emissions from TSHD

In this thesis, it is assumed that the TSHD used has 4100 m^3 hopper capacity and the sediment is reallocated at the Verdiepte Loswal (A. van Hassent, personal communication, 2022). The cycle time for dredging activity at the investigated area is 129 minutes and the fuel consumption per hour is 0.5 ton. Annual greenhouse gases produced by TSHD can be seen at Table 5.3.

Table 5.3 Greenhouse gases produced by TSHD for 1.8 m3 million sediment. All numbers are in kg.

Fuel Type	HVO		HVO		LNG
	Pre Tier I	Tier II	Pre Tier I	Tier II	
Methane (CH4)	0	0	160	127	55633
Sulfur Dioxyde (SO2)	3675	3761	43	43	129
Carbon Dioxyde (CO2)	4550182	4551898	4446782	4446782	3831529
Nitrogen Oxides (NO2)	85523	70078	75226	61640	15303

5.2.3 Greenhouse Gases Reduction

It is found that WID produces less greenhouse gases by 69% as it consumes less fuel to operate. Total greenhouse gases reduction by using WID instead of TSHD can be seen at Table 5.4.

Table 5.4 Greenhouse gases reduction but using a WID instead of TSHD for dredging 1.8 million m^3 sediment. All numbers are in kg.

Gas/FuelType	MGO		HVO		LNG
	Pre Tier I	Tier II	Pre Tier I	Tier II	
Methane (CH4)	0	0	111	88	38658
Sulfur Dioxyde (SO2)	2554	2614	30	30	89
Carbon Dioxyde (CO2)	3161847	3163040	3089996	3089996	2662467
Nitrogen Oxides (NO2)	59429	48696	52273	42832	10634

5.3 Cost Analysis Results

5.3.1 Cost Analysis for Central Suction-WID System

The investment cost to implement the suction system depends on the distance from the sediment trap to the reallocation site. As the distance is longer, longer pipeline is used, which leads to more material needed for the pipeline. Longer distance also leads to a higher pump power, which affect the specification of the pump.

The hourly rate for a WID is assumed to be €1250 (A. van Hassent, personal communication, 2022). With the production of 2000 m^3/hour and dredging volume of 1.8 million tds, the WID works for 2727 hour per year and the annual cost for dredging using WID is 3.41 million for every different reallocation sites as the dredging volume is the same. The electricity cost is assumed to be 0.31 per kwh based on the electricity price for a business scale on March 2022 (*Netherlands Electricity Prices*, 2022). The CO_2 annual reduction is monetized and counted as €100 profit per tonne of reduced CO_2. The amount of CO_2 reduction is 2662 ton using LNG, which is the marine fuel that is usually used by Port of Rotterdam (see Table 5.4). The annual cost per year for each reallocation sites of central suction-WID system are shown Table 5.5.

Table 5.5 Annual Operational Cost for Central Suction-WID System. All numbers are in million euro. The positive value means expenses while the negative value means profit which reduces the necessary expenses.

Cost	**Inside Nieuwe Waterweg**	**Outside Nieuwe Waterweg**	**Verdiepte Loswal**
Pump Electricity	0.2	0.7	8.9
WID cost	3.4	3.4	3.4
CO_2 reduction	-0.3	-0.3	-0.3
Total	**3.3**	**3.8**	**12.1**

5.3.2 Cost Analysis for TSHD

The cost of dredging using a TSHD is assumed to be €0.5 per m^3 of dredged sediment (A. van Hassent, personal communication, 2022). Annual cost for dredging 1.8 million tds sediment is €2,7 million per year.

5.3.3 Cost Analysis Summary

Total cost for 25 years for every scenario is shown at Table 5.6. Detailed cost per year can be found at Appendix E.

Table 5.6 Total Cost Needed for 25 Years. The cost of Central Suction-WID system is compared with the cost of TSHD. The cost for using a central suction-WID system is more expensive compared to TSHD.

Discharging Location	Cost	% more than TSHD
Inside Nieuwe Waterweg	€ 128 million	20%
Outside Nieuwe Waterweg	€ 151 million	41%
Verdiepte Loswal	€ 448 million	320%
TSHD	€ 106 million	-

The cost analysis indicates dredging using TSHD is cheaper than using a central suction-WID system. The cheapest alternative for central suction-WID system still costs 20% more than a TSHD. Reallocating to the Verdiepte Loswal costs more than three times of TSHD. Distances between the sediment trap and the reallocation site affect the cost of central suction-WID system as the cost is higher when the distance increases due to higher electricity cost for the pump. The result also shows that for central suction-WID system, the high proportion of the cost is from the operational cost.

6 Conclusion and Recommendations

In this chapter, conclusions from every chapters are made. The results from all of the analyses is used to determine the advantages and disadvantages of the central suction-WID system compared to TSHD. Recommendations are made based on the analyses in this thesis about how the implementation of central suction-WID system

6.1 Conclusions

It is confirmed that tidal asymmetry leads to sediment transport asymmetry. In a flood-dominated channel as Port of Rotterdam area, flood-directed sediment transport is expected to happen. As it was stated by de Nijs (2012), marine sediments are imported from the North Sea towards the Maasmond and transported more upstream either to the Nieuwe Waterweg or the harbour area of Port of Rotterdam with the bedload transport as a major sediment transport method. Less sediment is transported to the Nieuwe Waterweg as the bed level elevation there is higher which intervenes the bedload transport. To maintain the channel depth, maintenance dredging is performed and one of the alternatives to do it is by using a WID and the suction system, that is referred as the central suction-WID system in this study.

The sediment return from three different reallocation site alternatives is investigated for the central suction-WID system by using a Delft3D-Flow numerical model. These 3 locations are located at different distances from the sediment trap, where the intake of the suction system is located. The first option is installing the outtake of the suction system at the inside of Nieuwe Waterweg, which is close to the sediment trap. The relative downstream current due to the river discharge is expected to transport the sediment downstream. It was found that 30% of sediment returns to the harbour area after 14 days. Sensitivity analysis results showed that increasing the discharge rate from 1 m3/s to 2.5 m3/s, doubling the sediment concentration, and decreasing the particle to 24 microns didn't affect the sediment return percentage.

Another location that is assessed is the outside of Nieuwe Waterweg, which is approximately 2.5 km from the sediment trap. In this location, the existence of the Noorderpier structure is expected to prevent the sediment from entering the harbour area. The numerical simulation showed that 15% of the sediment returns to the harbour area after 14 days. The last alternative is to discharge the sediment at the Verdiepte Loswal, where Port of Rotterdam usually reallocates the sediment from their dredging project. Discharging at the Verdiepte Loswal, which is 25 km away from the sediment trap leads to 4% of sediment returns to the harbour area after 14 days.

The pump power and energy needed to operate the central suction-WID system is assessed. The results indicate that the major contributor to the necessary pump power is the pipe length. In this way, the pump power necessary for Verdiepte Loswal discharge is higher than discharging the other alternatives. The outtake of the suction system for discharging at the inside or outside of Nieuewe Waterweg is located near the water surface, which is assumed 22 m above the elevation of the intake, but even this head loss is considerably smaller than the loss proportional to the pipeline length. In conclusion, the necessary pump power and energy consumption can be reduced by decreasing the pipeline length, which depends on the distance to the reallocation site.

A comparison between the central suction-WID system and TSHD in terms of greenhouse gases production and costs is carried out. The produced greenhouse gases produced are estimated from the fuel consumption. A water injection dredger consumes 69% less fuel than a TSHD vessel, so it also produces less greenhouse gases at a similar rate. Economically, installing and using a central suction-WID for a period of 25 years long is more expensive than using TSHD during a similar period. The cheapest central suction-WID system assessed in this thesis is the one reallocating sediment inside of Nieuwe Waterweg, and it costs 20% more than TSHD. Transporting the sediment to Verdiepte Loswal, where Port of Rotterdam usually reallocates dredged material, costs more than three times than using TSHD. The high cost of the central suction-WID system is caused mainly by higher operational costs, especially the WID rental cost. Reallocating the sediment to a further away consumes more energy which leads to a more expensive electricity costs, and also produces more greenhouse gases emissions.

To summarize all of the analysis, Table 6.1 shows the comparison between each scenario. ‘Sediment return’ is the percentage of sediment that has been discharged to the reallocation site and returned to the harbour area. ‘Cost’ represents the necessary cost (capital and operational) for operating the dredging method for 25 years. ‘Fuel’ represents the fuel consumed by the dredger per 1 m^3 dredged sediment, which in equivalent to the quantity of greenhouse gases produced. Energy is the sum of installed power, which is for the dredger and for the pump of the central suction-WID system, and only the installed power for the dredging vessel, in the case of TSHD.

Table 6.1 Comparison between Central Suction-WID System and TSHD

	CSWID – Inside of Nieuwe Waterweg	CSWID – Outside of Nieuwe Waterweg	CSWID – Verdiepte Loswal	TSHD
Sediment Return	30%	15%	4%	4% (Loswaal)
Cost	€ 128 million	€ 151 million	€ 448 million	€ 106 million
Fuel (GH Gases)	0.08 kg/m3	0.08 kg/m3	0.08 kg/m3	0.26 kg/m3
Energy	2063 kW*	2390 kW*	5760 kW*	6542 kW**

**The power installed of the WID is 1971 kW, based on the WID Jetsed (Van Oord, 2022).*

***The power installed on the TSHD is based on TSHD Volvox Olympia, which is comparable to the dredger that is regularly used by Port of Rotterdam (Dredgepoint, 2021)*

The results from Table 6.1- indicate that the advantages of using a central suction-WID system are lower greenhouse gases emissions and less energy consumption. The disadvantage of using a central suction-WID system is that it costs more than TSHD. Reallocating the sediment at Verdiepte Loswal is better than the other alternatives in terms of sediment return, but the cost is higher and the installation of a pipeline - 25 km long seems unrealistic from a practical point of view. Reallocating the sediment at the inside or outside of Nieuwe Waterweg leads to higher sediment return rate than Verdiepte Loswal where Port of Rotterdam usually reallocate the dredged material to, so there is a possibility that it will increase the maintenance dredging volume at the port area as more sediment return and deposited at the harbour basin.

6.2 Recommendations

The recommendations are divided into two parts. The first part is the recommendations to improve the technical aspects of this study. The second part is the recommendations to the port authorities about reflexions on the implementation of a central suction-WID system.

6.2.1 Recommendations for Future Work

These recommendations are suggested to improve the quality of the research outputs:

- Increasing the simulation period can improve the estimation of sediment return percentages by including seasonal variations and storm conditions. The numerical model in this thesis runs for 14 days and the long term result of every scenario is not known. There is a possibility that the sediment return rate is higher after a longer period. Storm conditions are not included in this thesis, which can be a huge factor transporting sediment to the harbour area as it mentioned my de Nijs (2012). The study of Hendriks and Schuuman (2017) indicates a higher sediment return rate for Verdiepte Loswal after a period of 1 year.
- Running the numerical simulation for lower settling velocities. Smaller particles take more time to be deposited in the bed and are more easily transported, so this can affect the sediment return rate. Sediment return comparison can be made for numerical simulation with lower sediment settling velocities to the results of Hendriks & Schuuman (2017).
- The current numerical model is not capable of simulating the behaviour of a non-Newtonian flow while the fluid mud produced by WID can exhibit a non-Newtonian flow behaviour. A near-field model from Kirichek, et al. (2021) can be implemented in order to predict the behaviour of fluid mud.
- This thesis considers that all fluid mud produced by WID with an assumption that fluid mud is flowing to the sediment trap, which is not always the case. In reality, fluid mud can flow to the other areas depending on the local hydrodynamic and bathymetry conditions. Therefore, more WID pilots should be conducted in order to assess the trapping efficiency of sediment traps.
- In this thesis, the discharging rate of the suction system is limited in order to have laminar flow in the pipeline. Increasing the discharge can lead to a turbulent flow condition which consumes more energy as there is energy loss due to turbulence. However, it could reduce the operating time of the suction system, which could lead to a lower operational cost.

6.2.2 Recommendations for the Implementation of Central Suction-WID System

Recommendations are made for Port of Rotterdam, the stakeholder responsible for the study locations:

- The feasibility of design and construction method of Central Suction-WID System is not analyzed in this thesis. Installing a pipeline under an existing structures (separator between harbour area and Nieuwe Waterweg, or the Noorderpier structure at the outside of Nieuwe Waterweg) can compromise the stability of the structure. Drilling through an existing structure have its own challenges in terms of construction methods and they are out of the scope of this thesis
- It is assumed in this thesis that the dredging is uses one type of dredging vessel. In reality, a combination of two different dredging vessels could be considered, like combining WID and a TSHD, and this could reduce the operation time. The exact procedure of this operation can be analysed further.
- In this thesis, WID is used for producing fluid mud as it is considered as a more practical method due to its mobility. Other method can be used as an alternative to WID, for example by installing a stationary water jet. Further analysis has to be done in order to assess whether fluid mud produced by any dredging method flows to the sediment trap.
- Reallocating the sediment to other reallocation site assessed in this thesis than Verdiepte Loswal, which is the location currently used by Port of Rotterdam, leads to a higher sediment return rate (at least in the short term). Higher sediment return will contribute to the dredging volume of the next maintenance dredging. It means that the maintenance dredging volume at the next period is expected to be higher since more sediment deposited in the harbour area.
- Using only central suction-WID system for maintenance dredging is more expensive than TSHD. To reduce the cost, the system could be combined. A study case about combining the central suction-WID system with a bed leveller, for example can be an alternative. Detailed information about this combinaton can be found at Appendix F.

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Appendix A: Estuarine System

Estuarine System

Estuaries are bodies of water where freshwater from inland meets saltwater in the marine environment (de Nijs, 2012). They are of global significance as they are regions in which exchange of water substances and suspended particulate matter (SPM) takes place between inland freshwater systems, such as rivers, and marine environments, for example coastal or shelf seas. Many major cities in the world are located on estuaries and as a result, most estuaries are modified by human intervention. These functions are often closely tied to port infrastructure with costly and intensive dredging operations to maintain basins and channels to safeguard shipping. Keeping estuaries at artificial depths requires understanding of the system and the consequences of certain measures, which change the sediment balance in the system.

Since Port of Rotterdam is located at the estuarine area, fresh river run-off contributes an important input of buoyancy within an estuarine system. The Region of Freshwater Influence (ROFI) system experiences constantly alternating behaviour due to variations in tidal cycles, density different, river discharge, and wind forcings, and waves. Understanding the hydrodynamic forcings within the ROFI is of paramount importance to predict the behaviour of sediment and substances within the system.

ROFIs are known to have strong density gradients in the cross-shore direction, but also density variations over the vertical are generally present. Together with the wind, wave and tidal forcing, flow stratification is established (de Boer, 2009), these forcings govern the water motion. Various forcings vary over time such as differences in tidal strength between spring and neap tides, differences in freshwater discharge between freshets and droughts, and the occurrence of surges.

Because of many influencing factors including their fluctuations, the system can have a strong stochastic character. The horizontal and vertical density variations combined with gravity govern buoyant forces, namely the baroclinic pressure gradient and stratification. These buoyancy forces have a distinctive influence on the vertical variation of the turbulent forces and transport of SPM. Stratification gives rise to baroclinic density currents. Due to the horizontal and vertical density variations combined with gravity, an along-channel baroclinic pressure gradient governs a dense saltwater flow within the estuary near the bottom.

The barotropic tidal forcing advects the saline structure back and forth. This phenomenon is referred to as the salt wedge (De Nijs, 2012) and its penetration in harbour are can vary depending on various factors, such as the fresh water discharge, storms at sea and tidal strength due to spring-neap cycles. The degree of stratification can vary from highly mixed conditions to strongly stratified conditions. In a strong stratification, vertical fluctuations are dampened by the density gradient. This limits the turbulent mixing length. Stratification therefore induces the dampening of turbulence, resulting in less turbulent kinetic energy near the pycnocline. This results in a reduction of upward turbulent forces, causing the SPM to settle through the pycnocline. SPM cumulates in the tip of the salt wedge, where it is able to settle during small flow velocities such as slack water tide. During larger flow velocities, the tip of the salt wedge picks up sediment while eroding the channel bed. The accumulation of the SPM in this salt water tip is called the Estuarine Turbidity Maximum (ETM). The ETM is weakened in concentration due to the exchange with harbour basins. On top of the turbulence damping at a pycnocline, concentrated substances such as SPM and salinity in the fluid increase the fluid density according to the Equations of State. The increase in density due to these substances increases the density gradient, which in turn increases the turbulence dampening. This effect can be seen as a positive feedback mechanism.

Hydrodynamic forces at estuaries can be very dynamic due to its location at the meeting point between saltwater from sea and freshwater from river. Hydrodynamic forces that affecting the estuarine system, especially at the harbour area are discussed in the following section.

<u>Tides</u>

Vertical tide or simply ide is the vertical rise and fall of the water level (Bosboom, 2021). The rising period is the time it takes for the water level to get from the lowest elevation to the highest elevation, the falling period is the time it subsequently takes to reach the lowest level. The associated horizontal movement back and forward is the horizontal tide or tidal current. Flood current occurs when the current velocity is in the tidal wave propagation direction. Ebb currents are directed against the propagation direction.

Slack water is the name used for tidal flow reversal. High water slack around high around high water occurs for flow reversal from flood to ebb. Low water slack is the reversal from ebb to flood that during low water. The slack water period are the duration of slack water or the period of time during which current velocities are below some threshold.

In the frictionless case and without reflection from the head, the vertical and horizontal tides are in phase. As a result, the horizontal tide displays similar saw-tooth asymmetry as the vertical tide. The flood duration is identical to the ebb duration and the ebb velocities are equal to the flood velocities. Due to the effect of friction, a phase sift between vertical and horizontal tide that increases inland is expected. In the absence of other nontidal forcing or river discharge, the net tide averaged discharge should be zero even though the tidal current is asymmetric. Therefore, a shorter flood duration means that the maximum flood velocities are higher than the maximum ebb velocities. Systems in which the maximum flood velocities are higher than the maximum ebb velocities are called flood-dominance. Ebb-dominance systems are systems with the maximum ebb velocities that higher than the maximum flood velocities. Flood-dominance can be expected for a large tide (or more precisely a large ratio of tidal amplitude over water depth). In that case, the propagation of high water is faster than of low water and thus the rising period is shorter than the falling period. The existence of intertidal storage areas counteracts flood-dominance. In many tidal basins intertidal flats are present, which fall dry and are flooded again during the tidal cycle. The small water depths on these intertidal flats cause the high tide to propagate slower than the low tide. Ebb-dominance is further enhanced by the fact that the water level averaged over the flood period is generally higher than that averaged over the ebb period. Since during ebb the discharge is similar to the discharge during flood, the ebb velocities must on average be larger because of the smaller flow cross-section. This shows that the basin geometry controls the tidal distortion.

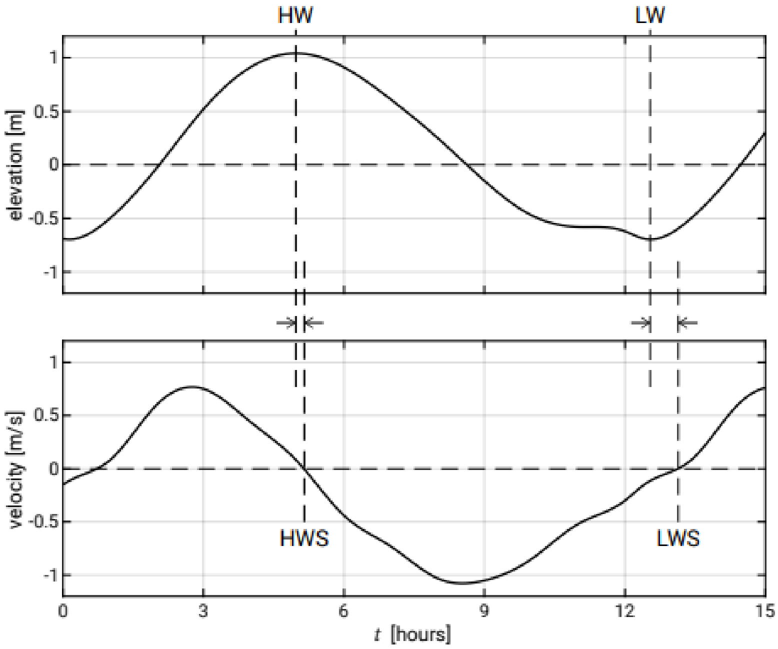


Figure A. 1 Vertical and horizontal tide in Rotterdam, where High Water Slack (HWS, flow reversal from food to ebb) occurs shortly after HW, and Low Water Slack (LWS, fow reversal from ebb to flood) follows LW with a slightly larger time difference. (Bosboom, 2021)

River Discharge

River discharge is the volume of water flowing through a river channel; measured at any given point in cubic metres per second (ESOTC, 2022). River discharge varies along the year depends on the season. Lowest river discharges occurred during spring, and throughout the mid-summer to early autumn months. Highest river discharges usually occurs at the end of the winter to early spring.

Wind Generated Waves

A wave is the generic term for any (periodic) fluctuation in water height, velocity or pressure (Schiereck, 2019). Waves at the ocean can be differed by their wave lengths or wave period. Different types of ocean surface waves based on its periods can be seen at Figure A. 2 Frequencies and periods of the vertical motions of the ocean surface. (Hothuijsen, 2007.

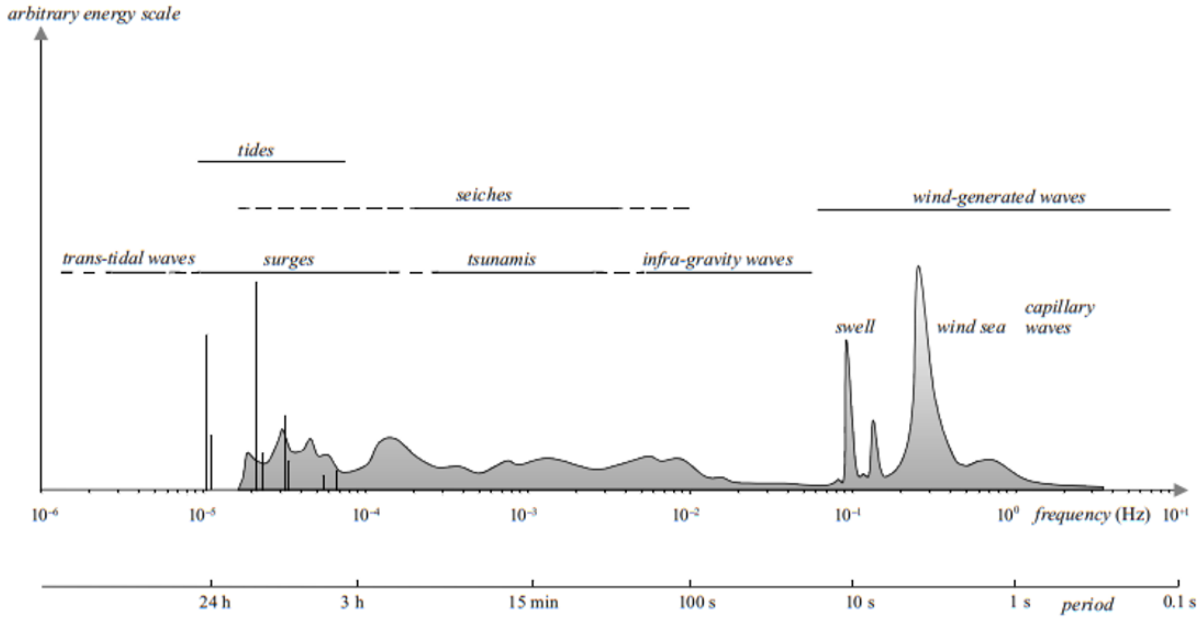


Figure A. 2 Frequencies and periods of the vertical motions of the ocean surface. (Hothuijsen, 2007)

Seiches is the standing waves with a frequency equal to the resonance frequency of the basin in which they occur. In a harbour, the amplitude of a seiche may be large enough to flood low-lying areas of the harbour, break anchor lines and otherwise disrupt harbour activities. These waves are usually generated by waves from the open sea.

Waves come for the sea are usually wind-generated waves. The period of wind-generated waves is shorter than 30 seconds. Wind sea are the waves that generated by the local wind. They are irregular and short-crested. When they leave the generation area, they take on a regular long-crested appearance and are called swell/ Waves with periods shorter than ¼ second (wave lengths shorter than about 10 cm), are affected by surface tension and are called capillary waves.

Sediment Transport

In the sediment transport modelling, the exchange processes between water column and bottom layer are often assumed to be governed by the critical bed shear stress of the bottom material and the actual occurring bed shear stresses.

$$\tau_b > \tau_{bc} \rightarrow Erosion$$

$$\tau_b < \tau_{bc} \rightarrow Sedimentation$$

For cohesive material where there is a strong binding force between the particles, the critical bed shear stress for erosion is larger than the critical bed shear stress for sedimentation. This implies that for a range of actual bed shear stresses, there is no exchange with the bottom layer in the case of cohesive sediments.

$$\tau_b > \tau_{bc_{er}} \rightarrow Erosion$$

$$\tau_{bc_{sed}} < \tau_b < \tau_{bc_{er}} \rightarrow No\ exchange\ between\ bottom\ layer$$

$$\tau_b < \tau_{bc_{sed}} \rightarrow Sedimentation$$

Bed shear stress in laminar flow is proportional with the flow velocity. In turbulent flow, the relation between bed shear stress and flow velocity becomes quadratic. In practice, the flow is always turbulent.

$$\tau_b = \rho\, {u^*}^2$$

$$u^* = \bar{\bar{u}}\frac{\sqrt{g}}{C}$$

Where:

- τ_b : Bed shear stress [N/m^2]
- u* : Shear velocity [m/s]
- $\bar{\bar{u}}$: Average flow velocity [m/s]
- g : Gravity acceleration [m/s^2]
- C : Smoothness coefficient (in the range of 40 to 60) [√m/s]

When the bed shear stress exceeds the critical bed shear stress, sediments are entrained and lifted to the water column. Sediment at the water column will be transported by the current caused by hydrodynamic forces. Three cases are considered to determine the critical bed shear stress at tidal channels with mud-sand bed mixtures (van Rijn, 2020*)*:

- Tidal channels with a dynamic bed consisting of low-density mud (weakly consolidated; p_{fines} >0.7; dry density < 400 kg/m^3) mostly occur in a regime with relatively high velocities (>0.7 m/s) resulting in significant reworking of the bed surface during the tidal cycle. The critical bed-shear for surface erosion for this type of conditions is about 0.2 to 0.4 N/m^2.

- Tidal channels with a bed consisting of high-density mud (firmly consolidated; p_{fines}>0.7; dry density > 800 kg/m^3) are mostly present in quiescent tidal environments with relatively low velocities (<0.3 m/s) where the fine sediments can deposit and consolidate. The critical bed-shear for surface erosion for this type of conditions is about 0.8 N/m^2 or larger.
- Tidal channel beds with intermediate bulk densities (dry density of 400-800 kg/m^3) generally occur along the banks in the transition zones to the intertidal mud-sand flats. The critical bed-shear stress of the fine fraction and the sand fraction is in the range of 0.4 to 0.8 N/m^2.

Tides

Tidal flow results in large gross fluxes of sediment entering and leaving the basin. The net fluxes are generally at least an order of magnitude smaller. The asymmetries of tide are of importance to the net sediment transport. Floor/ebb dominance causes a different maximum flow velocities during flood and ebb. This kind of asymmetry is called peak velocity asymmetry. For instance, if the maximum flood velocity exceeds the ebb velocity, a residual sediment transport in the flood direction is likely to happen, since sediment transport responds non-linearly to the velocity. Hence, flood-dominance implies a residual transport in the flood direction. For medium to coarse sediment, this is the dominant effect.

Another type of asymmetry is acceleration/deceleration asymmetry or slack-water asymmetry, which is the difference between durations of flow acceleration/deceleration or slack period. A longer slack after high water than after low water implies that sediment will have more time to settle at the end of flood than the end of ebb. Slack-water asymmetry affects the residual transport for fine sediment in inlets and basins. This is because fines need time to settle; fine sediments are allowed to deposit if the slack duration is long enough.

River Discharge

River discharge create a flow from upstream to downstream. Port basin that is connected to river can have a seaward directed residual flow. If this residual flow is strong enough to travel sediments, river discharge can create a seaward direction net sediment flux

Waves

The propagation of waves form the outer sea is retrained by the geometry and morphology of semi-enclosed basins. However, most waves in tidal basins are locally generated. Wave growth is limited by both shallow depths and the fetch length, which varies during the tidal cycle due to emerging shoals. Nevertheless, even during fair weather, the enhancement of bed-shear stresses by small amplitude waves is crucial for the sediment dynamics of shallow channels and tidal flats. There tidal velocities maybe insufficient to exceed the threshold of motion. Sediments on the bed can be stirred up by waves to the water column and transported by the current.

Mud Characteristics

Mud is a cohesive material consisting of a mixture of clay minerals, water, organic matters, and some amounts of sand. The particles forming the fluid mud layer to be found at the bottom of some port channels may be kept in suspension by tidal in combination with wave motion generated by wind, ship motion, human activities like dredging and fishing, or bioturbation. Fluid mud is usually defined as a fluid having a density withing the range of 1030-1300 kg/m^3.

Mud particles are characterized by very low settling velocities, and as a result they are predominantly transported as suspended load. Particles are suspended from the bed by bottom shear stresses and are kept in suspension by turbulent mixing withing the water column. Once an area with sufficiently low turbulent kinetic energy is reached, particles settle to the bed. If calm conditions persist long enough, water is slowly squeezed out of the pores by the weight of the particles, and the particles consolidate into the bed. From there, material may be eroded again if sufficiently high bottom shear stresses occurs.

Appendix B: Bed Shear Stress

Bed shear stress at five locations are assessed:

1. T-Shirt : at the middle of the T-Shirt.
2. Beerkanal Mouth : at the boundary between T-Shirt and Beerkanaal.
3. Maasvlakte Entrance : at the Beerkanaal, the entrance of Maasvlakte terminal.
4. Calandkanaal : at the Calandkanaal, in front of the berth.
5. Breeddiep : at the Calandkanaal, near Breeddiep.

Figure B. 1 shows the positions of the locations and Figure B. 2 to Figure B. 6 the bed shear stress and flow velocity at each location.

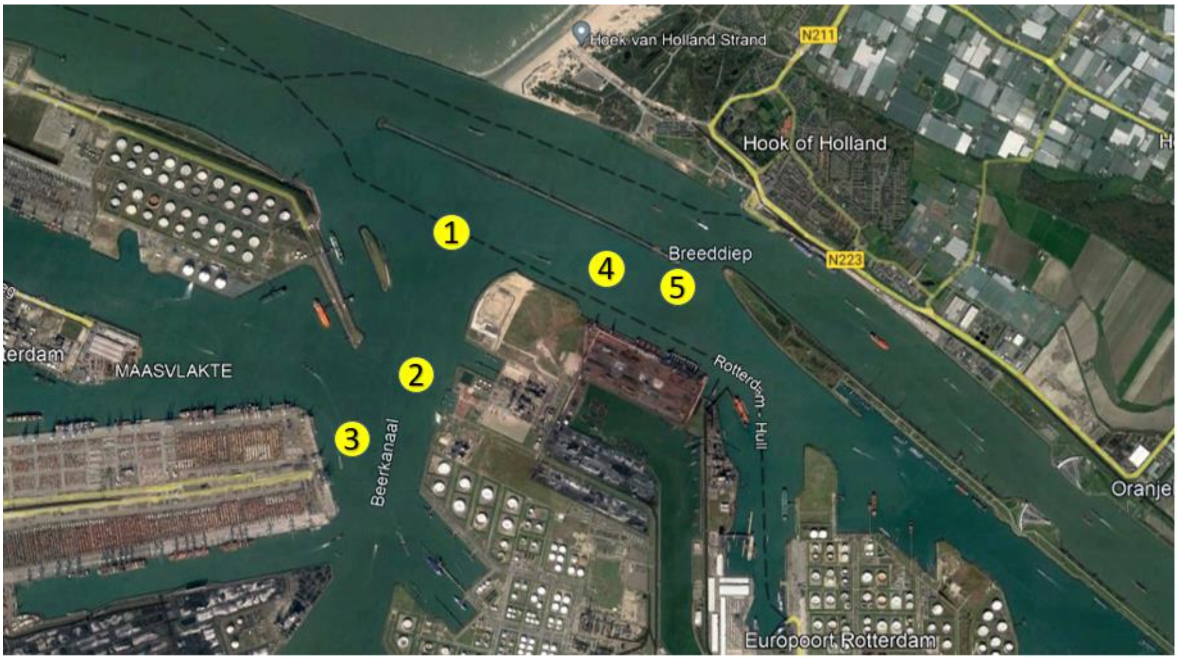


Figure B. 1 Locations of Bed Shear Stress Measurement

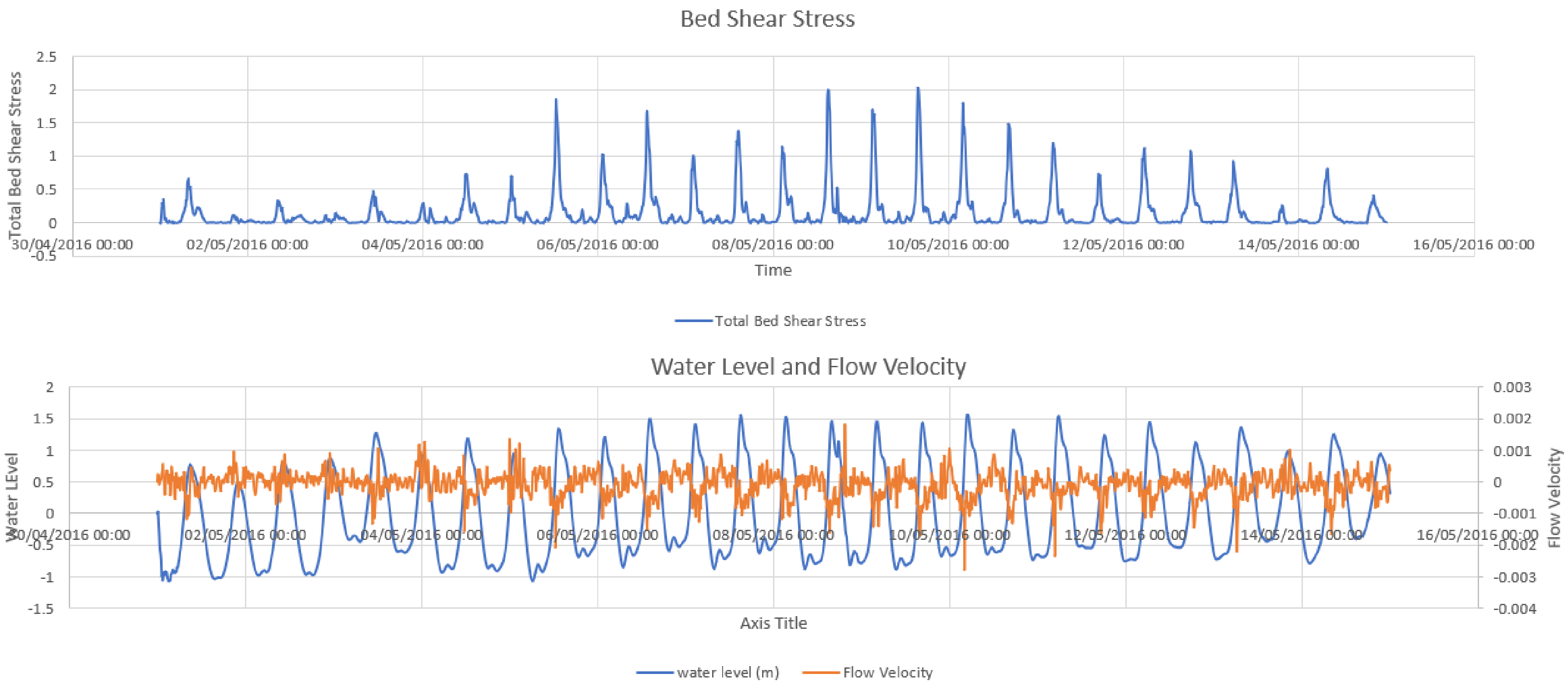


Figure B. 2 Bed shear stress, water level, and flow velocity at the T-Shirt (Location 1)

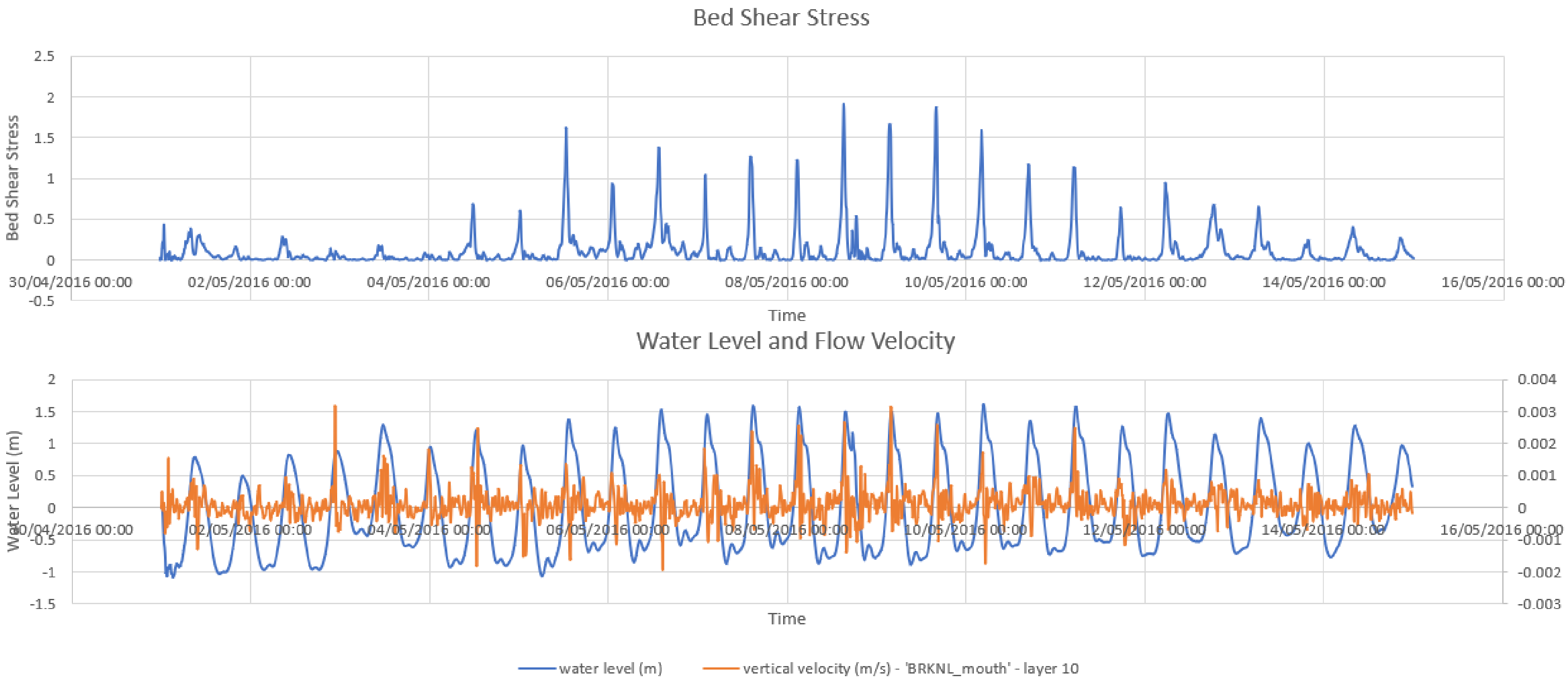


Figure B. 3 Bed shear stress, water level, and flow velocity at Beerkanal Mouth (Location 2)

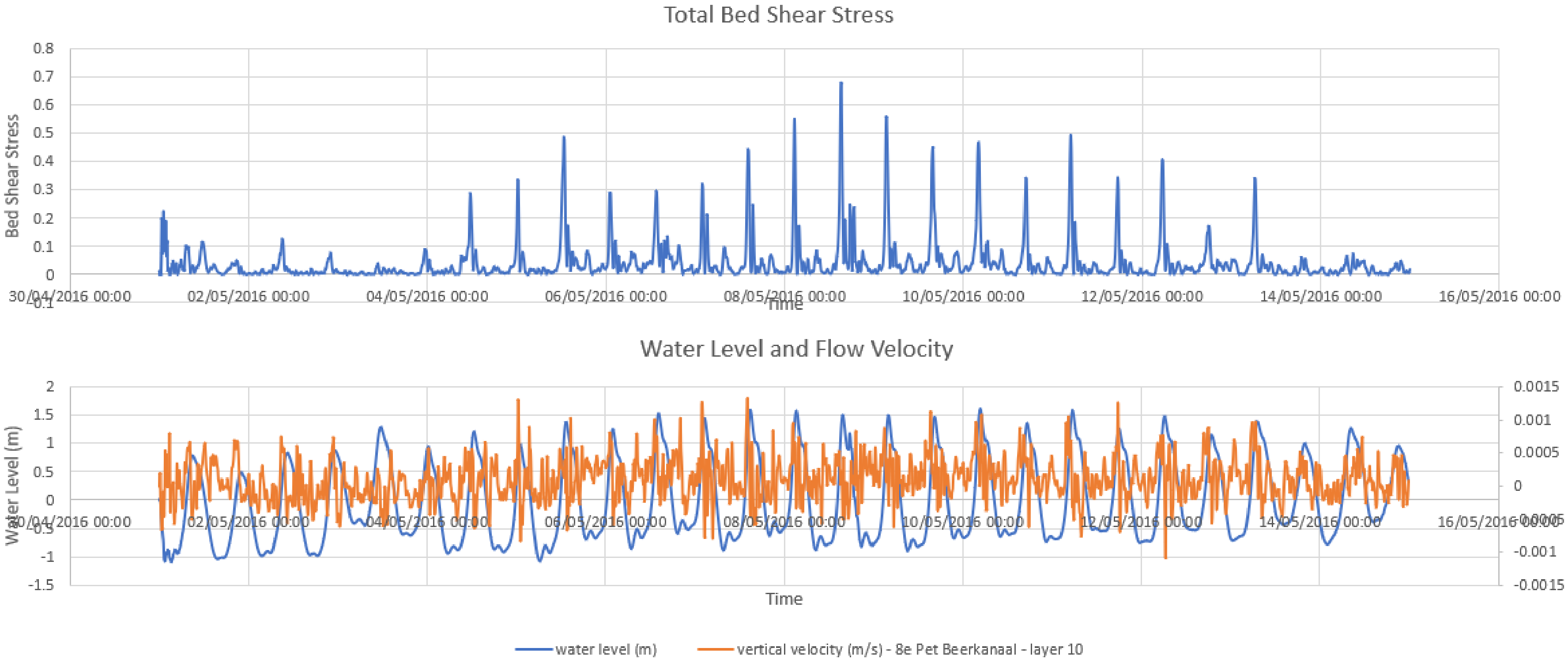


Figure B. 4 Bed shear stress, water level, and flow velocity at Maasvlakte Entrance (Location 3)

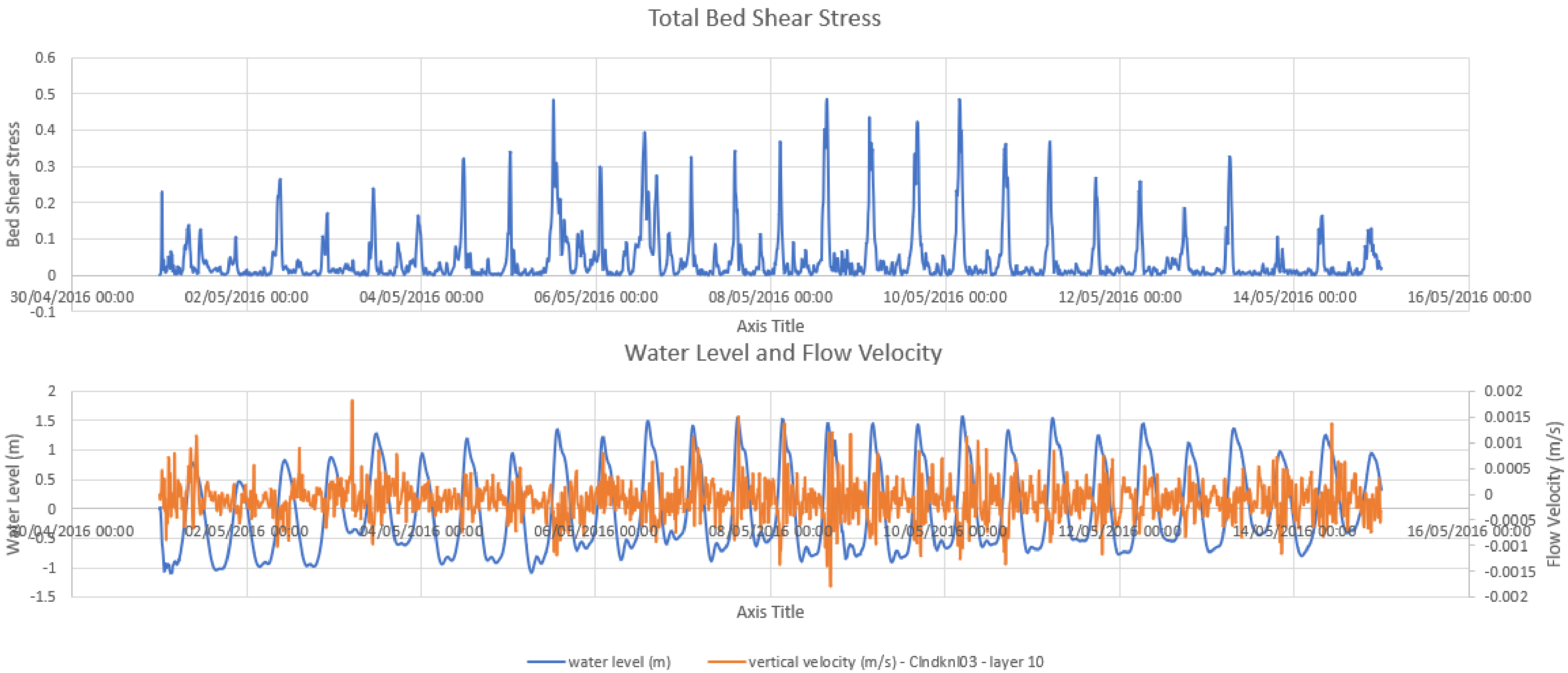


Figure B. 5 Bed shear stress, water level, and flow velocity at Calandkanaal (Location 4)

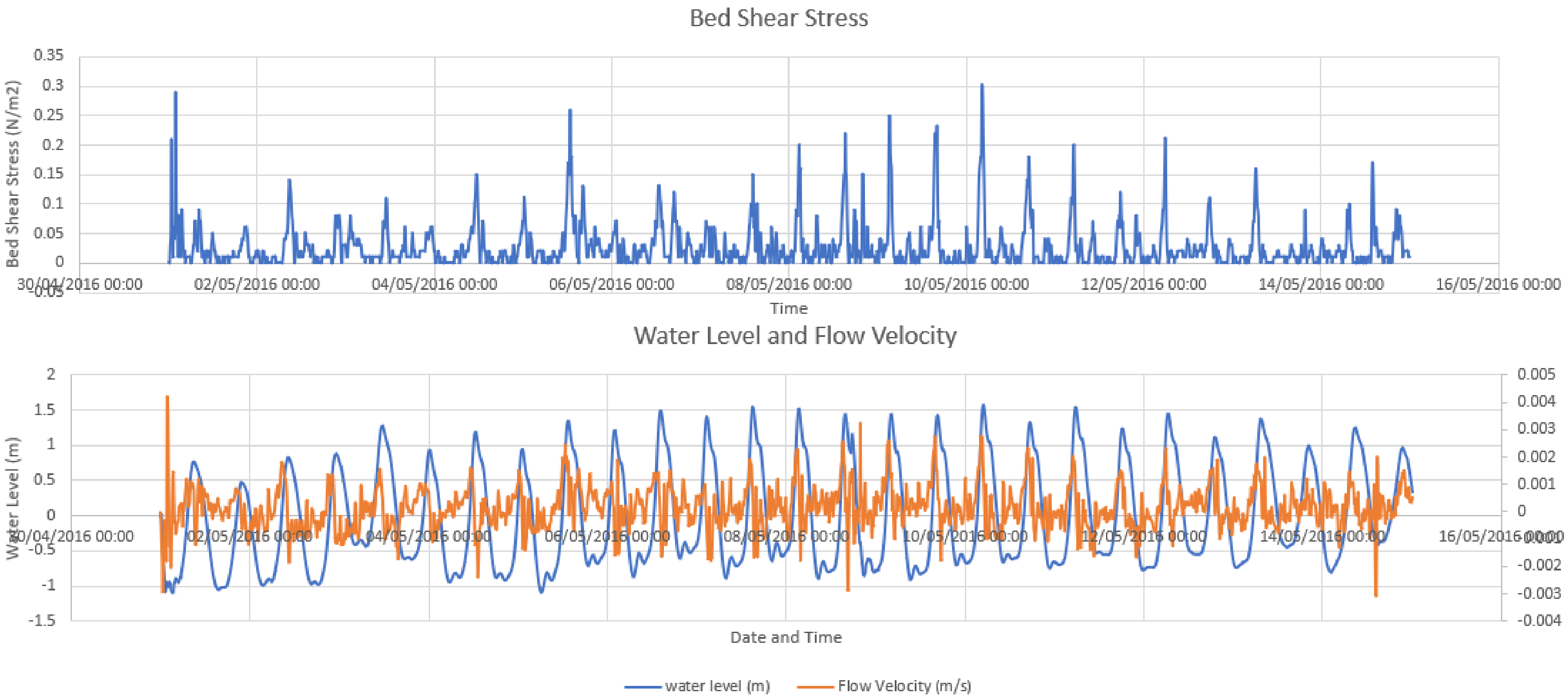


Figure B. 6 Bed shear stress, water level, and flow velocity at Breediep (Location 5)

Appendix C:

Return Sediment Volume

In this section volume of return sediment that passes through cross-sections is shown. First, a scenario without any sediment discharge is simulated, which means only natural sediments are involved. This scenario is called base scenario. After that, three scenarios with different discharging locations are modelled. Sediment transport volume is represented in positive or negative value depends on the direction. Figure C. 1 shows the direction of positive and negative value for each cross-sections. It can be concluded that positive value means sediment transport to the east.

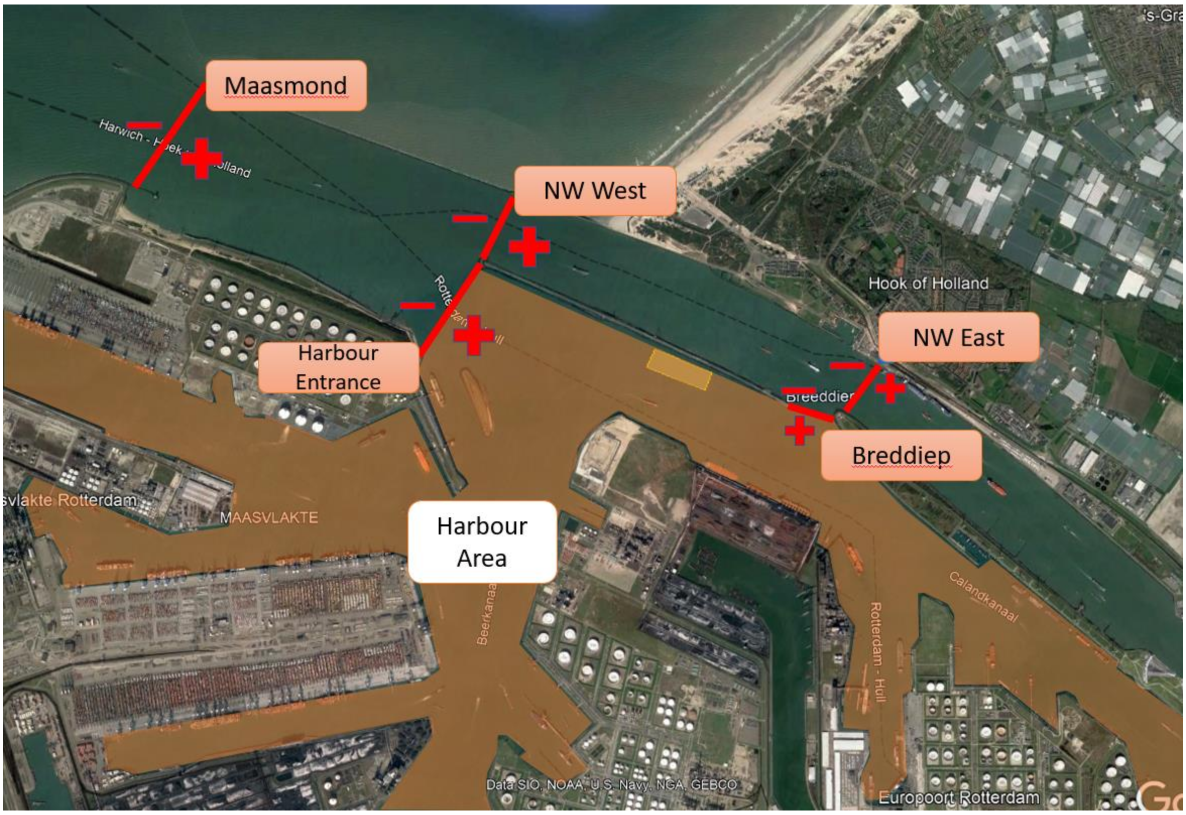


Figure C. 1 The direction of positive and negative value for sediment transport for each cross-section

Scenario 1: Discharge at the Inside of Nieuwe Waterweg

Main focus of this scenario is whether after discharged, the sediment will be deposited near the discharge (Nieuwe Waterweg between NW West and NW East), transported to the upstream (positive value of NW East), or transported towards the sea (NW West negative value). Sediment that has passed through NW West can transported to the harbour area via harbour entrance cross-section (positive value) and counted as sediment return.

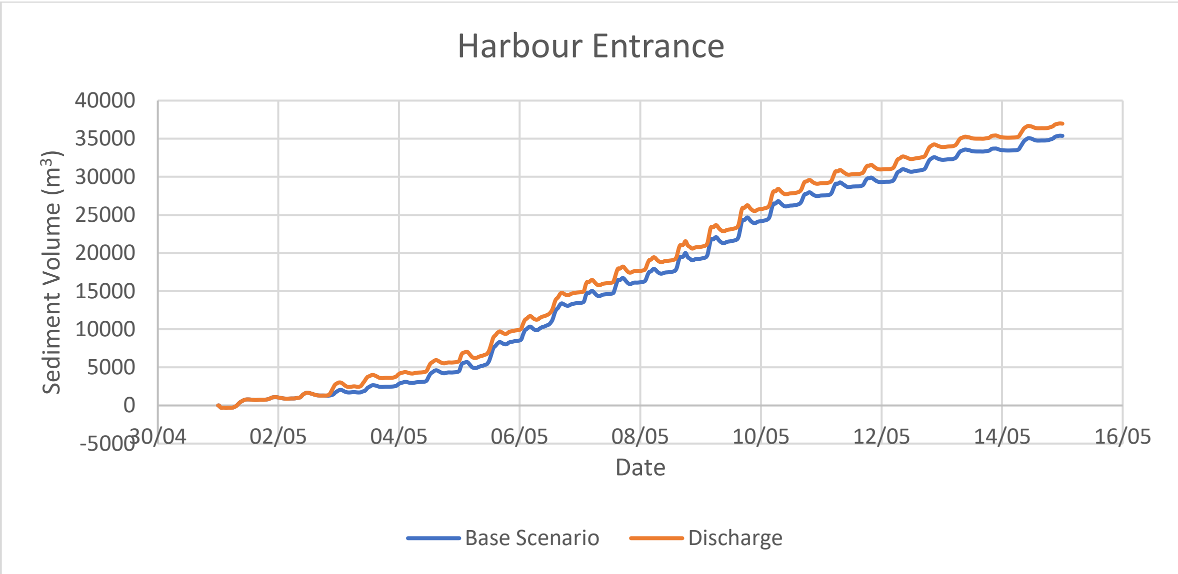


Figure C. 2 Sediment Transport Volume through Harbour Entrance Cross-section after being discharged at Nieuwe Waterweg

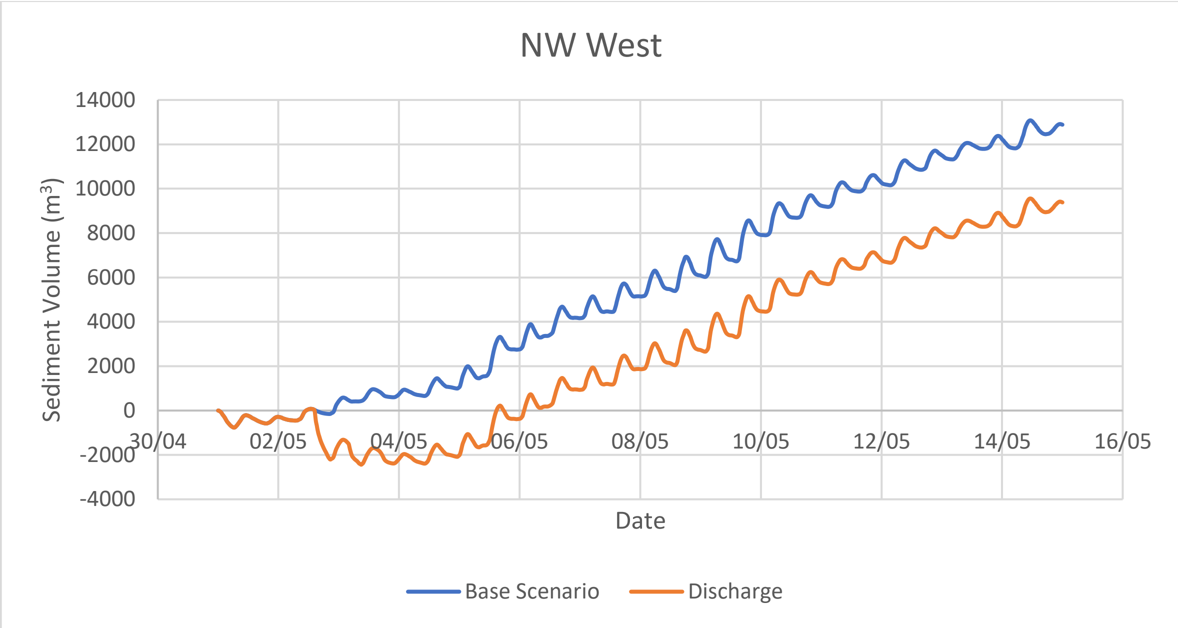


Figure C. 3 Sediment Transport Volume through NW West Cross-section after being discharged at Nieuwe Waterweg

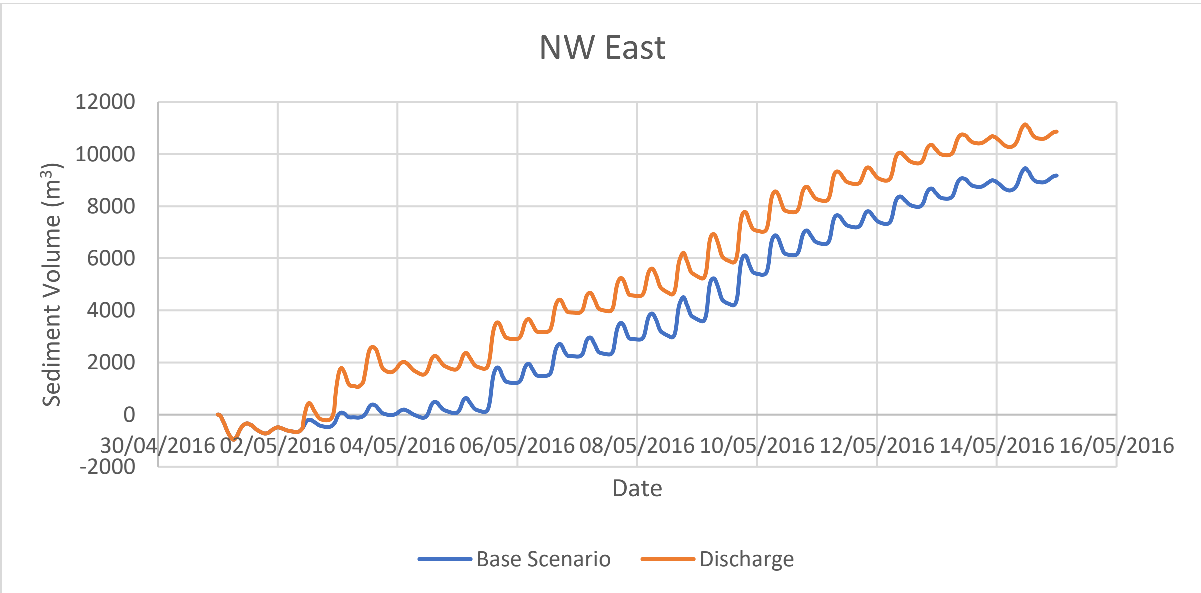


Figure C. 4 Sediment Transport Volume through NW East Cross-section after being discharged at Nieuwe Waterweg

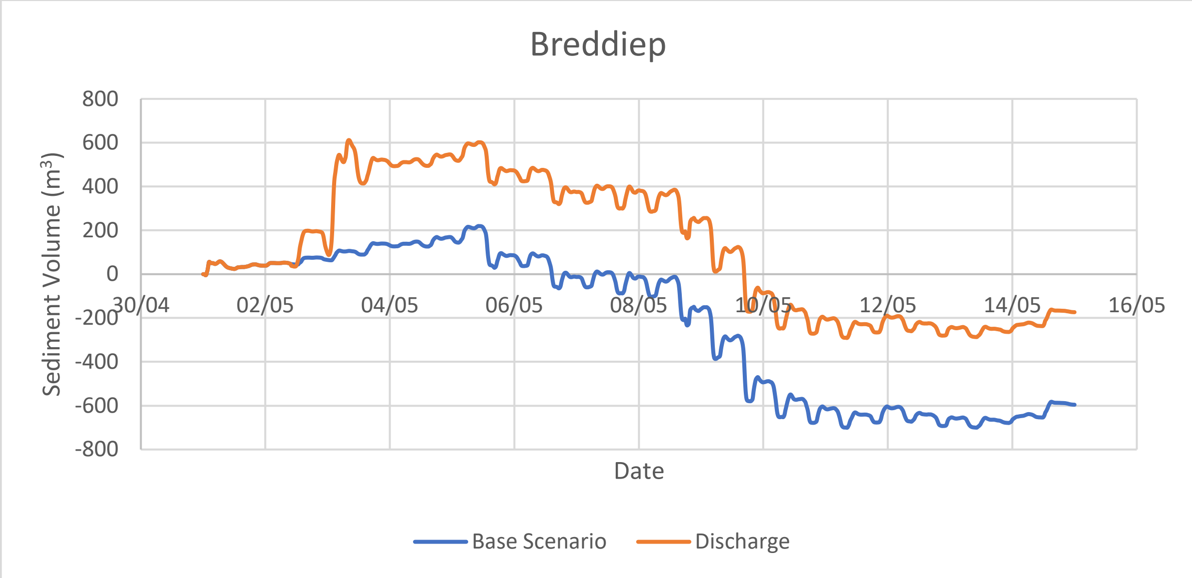


Figure C. 5 Sediment Transport Volume through Breeddiep Cross-section after being discharged at Nieuwe Waterweg

Table C. 1 Sediment transport volume (m^3) that passed through each cross-sections after being discharged at Nieuwe Waterweg

Cross-section	Base	Discharge	Difference	Percentage
NW West	12884	9382	3502	52%
NW East	9177	10864	1687	25%
Harbour Entrance	35372	36972	1600	24%
Breeddiep	-596	-173	422	6%

Scenario 2: Discharge at Verdiepte Loswal

Main focus of this scenario is to see whether the sediment enters the harbour area as sediment return. Sediment transport volume through NW West and NW East are not shown as Nieuwe Waterweg is not the area of interest in term of sediment return percentage. The graphs of sediment transport volume at Breeddiep is not shown as the difference between base scenario and discharge scenario is very small.

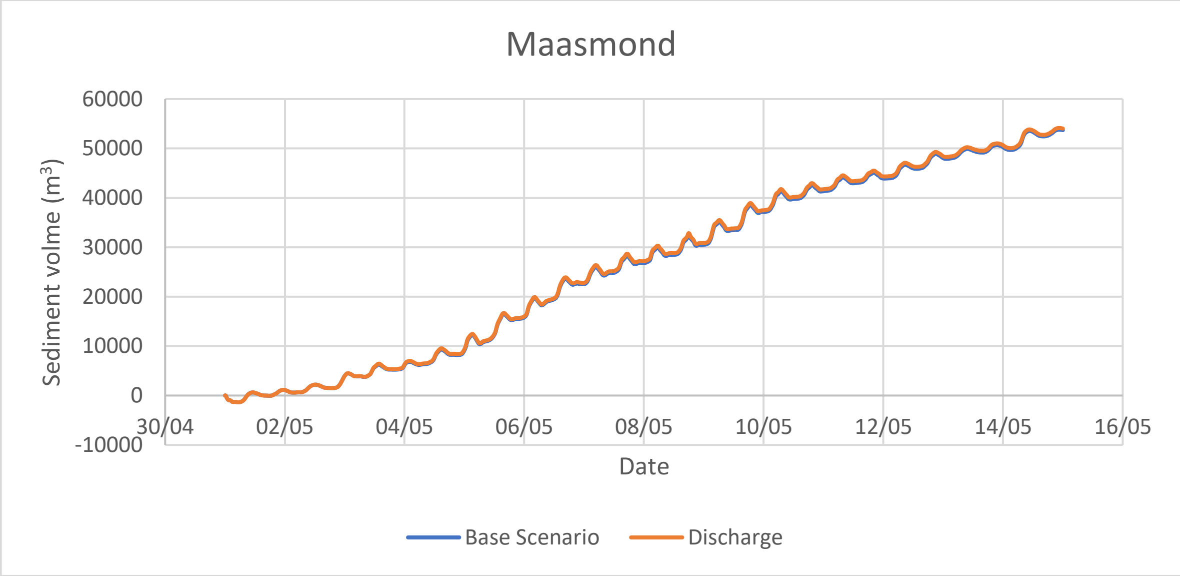


Figure C. 6 Sediment Transport Volume through Harbour Entrance Cross-section after being discharged at Verdiepte Loswal

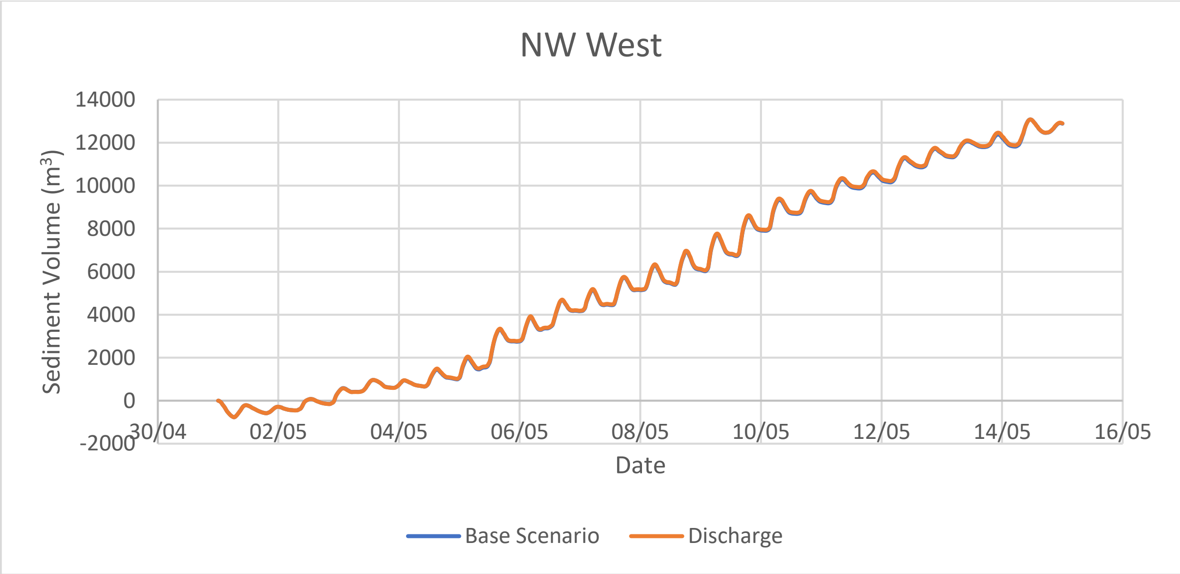


Figure C. 7 Sediment Transport Volume through NW West Cross-section after being discharged at Verdiepte Loswal

Table C. 2 Sediment transport volume (m^3) that passed through each cross-sections after being discharged at Verdiepte Loswal

Cross-section	Base	Discharge	Difference	Percentage
Maasmond	53703	54037	334	5.0%
NW West	12884	12897	13	0.2%
NW East	9177	9205	27	0.4%
Harbour Entrance	35372	35645	273	4.1%
Breeddiep	-596	-593	2	0.03%

Scenario 3: Discharge at the Outside of Nieuwe Waterweg

Outside of Niuewe Waterweg is chosen as an alternative between Nieuwe Waterweg which is very close to the sediment trap and Verdiepte Loswal which is far offshore. Similar with Verdiepte Loswal scenario, graph of sediment transport volume at NW East, NW West, and Breeddiep are not shown.

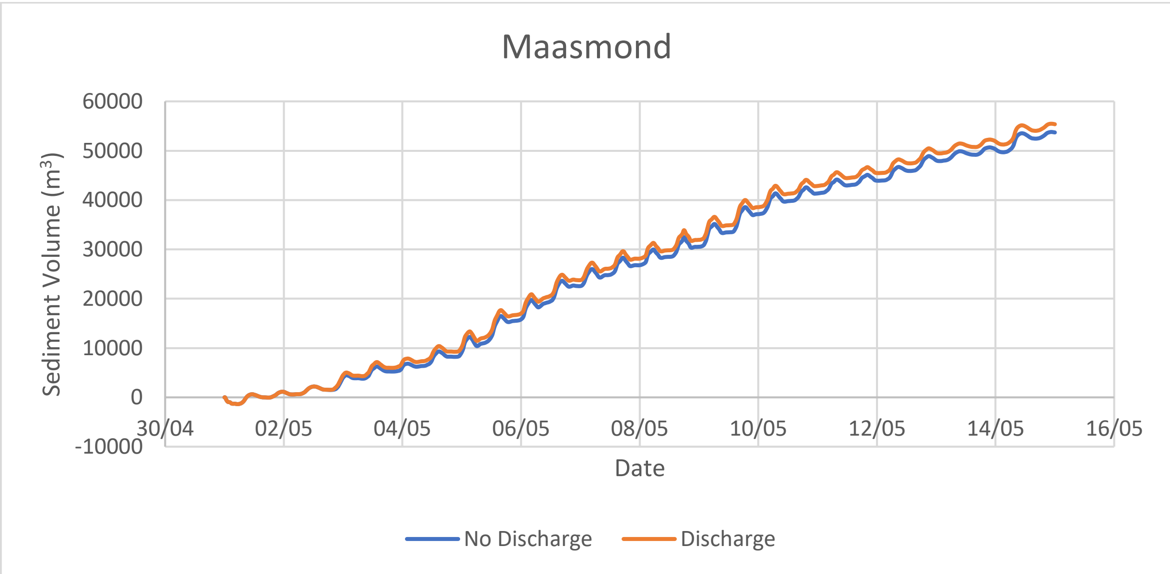


Figure C. 8 Sediment Transport Volume through Maasmond Section after being discharged at the Outside of Niuewe Waterweg

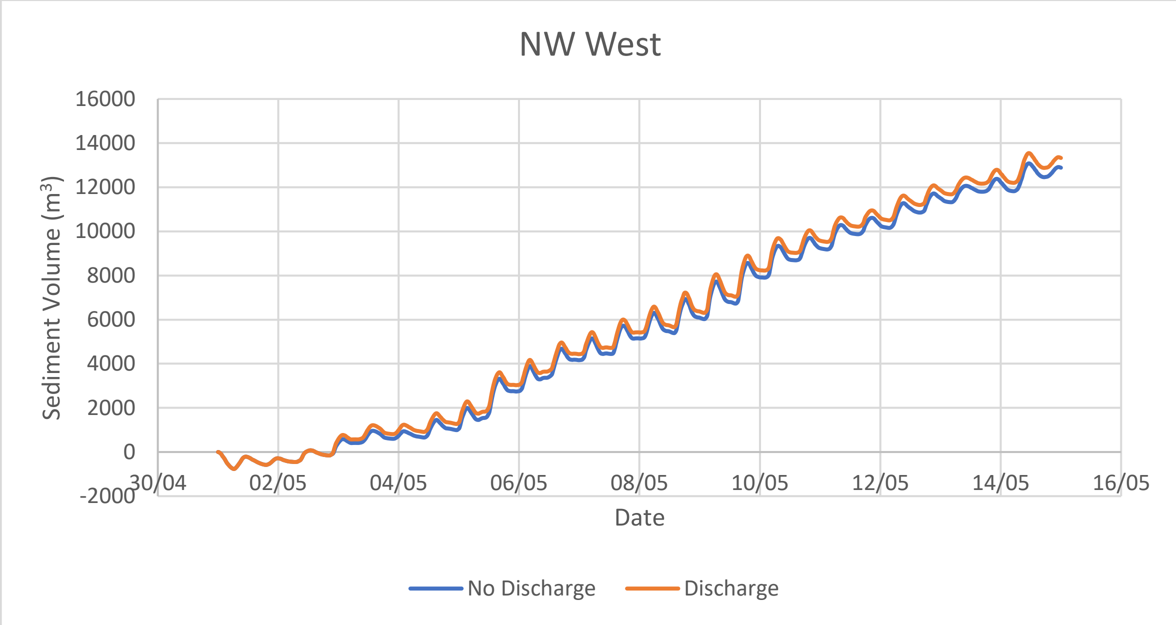


Figure C. 9 Sediment Transport Volume through NW West Cross-section after being discharged at the Outside of Niuewe Waterweg

Table C. 3 Sediment transport volume (m^3) that passed through each cross-sections after being discharged at the Outside of Niuewe Waterweg

Cross-section	Base	Discharge	Difference	Percentage
Maasmond	53703	55366	1663	25.0%
NW West	12885	13326	442	6.6%
NW East	9177	9504	327	4.9%
Harbour Entrance	35373	36368	995	15.0%
Breeddiep	-596	-606	10	0.2%

Appendix D:

Pump Power Calculation

In this appendix, the calculation of pump power is shown. All the calculations are done by using a python code. Only one example calculation, which is for the discharge at Nieuwe Waterweg is shown .

1. Laminar Flow Conditions

In this research, the flow in the pipeline of suction system is designed to be a laminar flow. The density of the fluid mud is assumed to be 1150 kg/m^3 and the yield stress is 10 pa. Transitional velocity, which is the maximum velocity that can be reached by laminar flow before it changes into a turbulence flow can be calculated with equation (A.1) by Slatter (Matousek, 2004).

$$V_T \approx 26 \sqrt{\frac{\tau_y}{\rho_m}} \tag{A.1}$$

$$V_T = 26 \sqrt{\frac{10}{1150}} = 2.42\ m/s$$

Where:

- V_T : Transitional velocity [m/s]
- τ_y : Yield stress [Pa]
- ρ_m : Mixture density [kg/m^3]

The maximum discharge for laminar flow can be determined by multiplying the turbulence velocity by the area of the pipeline, as shown at equation (A.2).

$$Q_{max} = V_T \frac{\pi}{4} D^2 \qquad (A.2)$$

Where:

- Q_{max} : Maximum discharge [m^3/s]
- D : Pipe diameter [m]

From the equation above, it is known that the maximum discharge for 1 m diameter pipe is 1.9 m^3/s and for 1.5 m diameter pipe is 4.28 m^3/s. Production of the pumping system is determined by multiplying the pumping discharge with the concentration of the sediment. Assuming the dry density of the sediment, ρ_s, is 2600 kg/m^3 and the water density, ρ_w, is 1025 kg/m^3, sediment concentration, C, is estimated and shown by equation (A.3).

$$\rho_m = \rho_s C + \rho_w (1 - C)$$

$$1150 = 2600\, C + 1025(1 - C) \qquad (A.3)$$

$$C = 0.079$$

Where:

- ρ_w : Fluid density [kg/m^3]
- C : Sediment volume fraction [-]

After sediment concentration is known, then the production rate can be estimated by determining the discharging rate. The suction system must be capable of pumping all of the sediments (1.8 million tds or 5.45 million m^3) in the period less than 1 year or 365 days. Table D. 1 shows the analysis for choosing a discharge rate. The pumping duration is 7636 hours per year with a discharging rate of 2.5 m^3/hour.

Table D. 1 Production analysis based on duration. Discharge of 2.5 m3/hour is chosen due to the pumping duration that is less than 365 days.

Diameter	Discharge	Production (m^3/s)	Duration (hour)	Duration (day)
1	1	0.08	19091	795
	1.3	0.10	14685	612
1.5	1	0.08	19091	795
	1.5	0.12	12727	530
	2	0.16	9545	398
	2.5	0.20	7636	318
	3	0.24	6364	265

2. Major Loss

Major loss is defined as energy loss due to friction between the fluid mud and the pipe wall. For laminar flow, an equation for friction coefficient λ_f is calculated by the function of Reynolds number. For a non-Newtonian flow inside a pipeline, the Reynolds number equation by Bingham is used (Matousek, 2004). The calculation of Bingham Reynolds number and the friction coefficient is shown by equation (A.4).

$$Re_B = \frac{\rho_m V_m D}{\eta_B\left(1 + \frac{\tau_y D}{6\eta_B V_m}\right)} = 1353.4$$

$$\lambda_f = \frac{64}{Re_B} = 0.047 \qquad (A.4)$$

Where:

- Re_B : Bingham Reynolds number [-]
- ρ_m : Mixture density [kg/m^3]
- V_m : mean mixture velocity in a pipe [m/s]
- D : Pipe diameter [m]
- η_B : tangential viscosity of Bingham plastic mixture [Pa.s[
- τ_y : yield stress [Pa]

- λ_f : Friction coefficient

Frictional head loss, I_m, energy loss per one length unit of pipe is calculated by using the Darcy-Weibach approach and then multiplied by the pipe length to determine the pressure drop due to major loss, as shown at equation (A.5) and (A.6).

$$I_m = \frac{\lambda_f v_m^2}{D^2 g} = 0.0032 \quad (A.5)$$

$$H_{major} = I_m L = \ 1.6\, m \quad (A.6)$$

Where:

- g : gravitational acceleration [m/s^2]
- I_m : head loss gradient per unit length [-]
- H_{major} : major pressure loss [m]
- L : pipe length [m]

3. Minor Loss

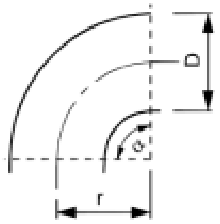
Minor loss is caused by fittings as bends, joint balls, expansions and contractions of a discharge area, valves and measuring instruments act as obstructions to the flow, as shown at Figure D. 1. The minor loss is represented by a minor loss coefficient. As the exact configuration of the pipeline is not determined yet, minor loss coefficient on every scenario is assumed to be 2. The calculation of minor head loss is shown at equation (A.7).

$$H_{minor} = \xi \frac{V_f^2}{2g} \frac{\rho_m}{\rho_f} = 0.23\, m \quad (A.7)$$

Where:

- H_{minor} : Minor head loss[m]
- ξ : Coefficient of minor losses [-]
- V_m : Flow velocity [m/s]
- g : Gravity acceleration [m/s^2]

- ρ_m : Mixture density [kg/m^3]
- ρ_f : Fluid density [kg/m^3]



	ξ-waarden: bocht α in °					
r/D	15°	22,5°	30°	45°	60°	90°
1,5	0,03	0,050	0,085	0,13	0,17	0,20
2	0,03	0,045	0,060	0,09	0,12	0,13
3	0,03	0,045	0,055	0,08	0,10	0,13
5	0,03	0,045	0,050	0,07	0,08	0,11
10	0,03	0,045	0,050	0,07	0,07	0,11

			Overeenkomstigeweerstandslengte in meters		
		ξ	D=0,30 m	D=0,60 m	D=0.90 m
Bocht 45°	r/D= 1,5	0,13	6,0	11	17
	r/D= 2,0	0,09	3,7	7,5	11
Bocht 90°	r/D= 1,5	0,20	7,5	15	22
	r/D= 2,0	0,13	5,0	10	15
Knik 30°		0,15	3,7	7,5	11
45°		0,3	7,5	15	22
Kogel		0,2-0.3	5,0-7,5	10-15	15-22
Zak, afsluiter		0.1	2,5	5	7
Wisselmet α	= 30°	0.6	15	30	45
		0,3	7,5	15	22

Figure D. 1 Minor loss coefficient for a flow in a pipeline. The minor loss coefficient in a pipeline system is the sums of all fitting in the system. (Matousek, 2004)

4. Head Loss from a Vertical Pipeline

For the pseudo-homogenous flow, a Darcy-Weibach approach is used.

$$H_{vert} = \lambda_f \frac{\Delta h}{D}\frac{v_m^2}{2g} = 0.16m \qquad (A.8)$$

Where:

- H_{vert} : Head loss from a vertical pipeline [-]
- λ_f : Friction factor [-]
- Δh : Elevation difference [m]

5. Total Pressure Drop Loss

Total frictional head loss is a sum of head losses. Total pressure drop over a pipeline can be calculated with equation (A.9).

$$\Delta \text{p} = \rho_m g\left(H_{major} + H_{minor} + H_{Vert}\right) = 23.64\ kPa \qquad (A.9)$$

Where:

- Δp = Total pressure loss [Pa]

6. Pump Power

Pump power, P, is the electricity needed to be supplied by the pump for transporting the mixture flow inside the pipeline. In this calculation, pump efficiency is assumed to be 0.6. The necessary pump power is calculated by equation (A.10).

$$P = \frac{\text{Q}\ \Delta P}{\eta} = 91.8\ kW \qquad (A.10)$$

Where:

- P : Necessary pump power [watt]
- Q : Discharge rate [m^3/s]

- η : Pump efficiency [-]

7. Energy Consumption

Energy consumption of the pump is estimated by multiplying the pump power with the pumping duration The pumping duration is determined by dividing the total dredging volume with the production rate of the suction system. It takes 7636 hours per year to reallocate the sediment. The total energy consumption is calculated by equation (A.11).

$$E = 92 \times 7636\ hour = \ 701\ kWh \tag{A.11}$$

Where:

- E : Total energy consumption [kWh]
- t : Operating time [hour]

Appendix E:

Price Breakdown

This appendix shows how the price of every item on financial analysis is estimated.

1. <u>Central Suction-WID: Investment Cost</u>

The investment costs of central suction WID system consist of the pump price, pipe material, and pipe installation cost. Unit price per item can be seen at Table E. 1. The unit price is just a rough estimation and has been confirmed by a cost engineer at Port of Rotterdam. The exact price for the real project has to be assessed further.

Table E. 1 Investment cost breakdown for central suction-WID system

Item	qty	Unit	Unit Price	Price
Pipeline material (d=1.5 m)	1	m	€ 650.00	€ 650.00
Suction pump	1	kW	€ 1,500.00	€ 1,500.00
Laying and assembly cost for sinker	1	m	€ 200.00	€ 200.00
Pipeline burying cost	1	m	€ 100.00	€ 100.00

2. Central Suction – WID: Operational Cost

Operational cost of central suction-WID system consists of pump electricity cost and WID rental cost. Since central suction-WID system produces less CO_2 than TSHD, the CO_2 reduction is monetized and counted as a profit. The electricity cost per kWh is €0.31, which is the price for businesses purposes (more than 1 million kWh annual consumption). Electricity costs for each scenario is presented at Table E. 2.

Table E. 2 Operational cost breakdown for central suction-WID system

Discharge Location	Energy (kWh)	Unit Price	Electricity Cost
Inside of Nieuwe Waterweg	701,171	€ 0.31	€ 217,362
Verdiepte Loswal	2,432,716	€ 0.31	€ 754,142
Outside of Niuewe Waterweg	28,932,235	€ 0.31	€ 8,968,993

The WID rental cost is €1250 per hour. The duration of WID operation is estimated by dividing the total dredging volume (1.8 million tds) with the production rate (2000 m^3/hour).

$$WID\ duration = \frac{\frac{1,800,000}{0.33}}{2000} = 2727 hour$$

The WID duration is then multiplied by the hourly rate of the rental cost.

$$WID\ Cost = 2727.27 \times €\ 1250 = \ €\ 3,409,090$$

The CO_2 reduction profit is counted as €100 per ton CO_2. For all alternatives of central suction-WID system, the CO_2 reduction volume is 2662.47 ton per year.

$$CO_2\ profit = 2662.47 \times €\ 100 = \ €\ 266,247$$

3. TSHD: Operational Cost

Operational cost of TSHD is only the TSHD dredging cost which depends on the dredging volume. The rent cost of a TSHD is € 0.5 per cubic meter. Based on this rate, the operational cost of TSHD is estimated.

$$TSHD\ Cost = \frac{1.800.000}{0.33} \times €0.5 = €\ 2,727,272$$

4. Cost Analysis for 25 Years Results

The detailed financial analysis for 25 years is shown at Table E. 3

Table E. 3 The annual financial analysis for every different alternatives of central suction-WID and TSHD. The initial costs (IC) is 0 for TSHD as there is no any installation needed.

Year	WID NW Inside	WID NW Outside	WID VL	TSHD
IC	€ 984,930	€ 3,866,655	€ 48,043,113	€ 0
1	€ 3,360,207	€ 3,896,986	€ 10,555,353	€ 2,817,273
2	€ 3,471,094	€ 4,025,587	€ 10,903,679	€ 2,910,243
3	€ 3,585,640	€ 4,158,431	€ 11,263,501	€ 3,006,281
4	€ 3,703,966	€ 4,295,659	€ 11,635,196	€ 3,105,488
5	€ 3,826,197	€ 4,437,416	€ 12,019,158	€ 3,207,969
6	€ 3,952,462	€ 4,583,851	€ 12,415,790	€ 3,313,832
7	€ 4,082,893	€ 4,735,118	€ 12,825,511	€ 3,423,189
8	€ 4,217,628	€ 4,891,377	€ 13,248,753	€ 3,536,154
9	€ 4,356,810	€ 5,052,792	€ 13,685,962	€ 3,652,847
10	€ 4,500,585	€ 5,219,534	€ 14,137,599	€ 3,773,391
11	€ 4,649,104	€ 5,391,779	€ 14,604,139	€ 3,897,913
12	€ 4,802,525	€ 5,569,708	€ 15,086,076	€ 4,026,544
13	€ 4,961,008	€ 5,753,508	€ 15,583,917	€ 4,159,420
14	€ 5,124,721	€ 5,943,374	€ 16,098,186	€ 4,296,681
15	€ 5,293,837	€ 6,139,505	€ 16,629,426	€ 4,438,471
16	€ 5,468,534	€ 6,342,109	€ 17,178,197	€ 4,584,941
17	€ 5,648,995	€ 6,551,399	€ 17,745,077	€ 4,736,244
18	€ 5,835,412	€ 6,767,595	€ 18,330,665	€ 4,892,540
19	€ 6,027,981	€ 6,990,925	€ 18,935,577	€ 5,053,993
20	€ 6,226,904	€ 7,221,626	€ 19,560,451	€ 5,220,775
21	€ 6,432,392	€ 7,459,940	€ 20,205,946	€ 5,393,061
22	€ 6,644,661	€ 7,706,118	€ 20,872,742	€ 5,571,032
23	€ 6,863,934	€ 7,960,419	€ 21,561,543	€ 5,754,876
24	€ 7,090,444	€ 8,223,113	€ 22,273,074	€ 5,944,787
25	€ 7,324,429	€ 8,494,476	€ 23,008,085	€ 6,140,965
TOTAL	**€ 128,437,292**	**€ 151,679,001**	**€ 448,406,715**	**€ 106,858,906**

Appendix F:

Combination of Central Suction-WID System and Bed Leveller

The information from the previous chapters are implemented to make an operational plan. To be noted that this plan is unique for the condition to the case study based on the condition at Port of Rotterdam area. The specification of the sediment trap is referred to Kirichek & Rutgers (2019). The dimensions of the trap are 600 m over 120 m, with the depth varies from 1 m to 1.3 m. The WID dredges continuously until the sediment trap is fully filled, which affects the dredging volume. After the sediment trap is fully filled, the WID stops dredging at the investigated location and move to the other location while the suction system is still working for emptying the sediment trap. Bed leveller is then used to transport sediment to the sediment trap once it is emptied. Since the sediment at the investigated area is softer post WID, it can be easily moved by a bed leveller. The advantage of a bed leveller is its lower price and fuel consumption.

All of the fluid mud produced by the WID is assumed to flow to the sediment trap (see Figure 3.6) driven by the density differences (see section 2.3). The density of the fluid mud produced by WID fluctuates depending on the timing after the operation. The density of the fluid mud found in the sediment trap was measured to be 1150 kg/m^3 after approximately one day after the WID operation (Kirichek & Rutgers, 2019). Continuing settling and consolidation cause and increase up to around 1200 kg/m^3 (Kirichek & Rutgers, 2019).

The production of the WID is assumed to be constant 2000 m^3/s (A. van Hassent, personal communication, 2022) . Smaller fluid mud density means the WID uses more water to reach the production rate and leads to higher volume of the fluid mud. In this analysis, it is assumed that the WID operates continuously without stopping until the sediment trap is full. The illustration of sediment trap filling is shown at Figure F. 1.

Table F. 1 shows the time needed for filling and emptying the sediment trap based on the fluid mud density. There are three different filling or emptying time considered:

- Time until full when discharging, the time needed for sediment trap to be filled until it is full while the suction system is working.
- Emptying time, the time needed for the suction system to pumping out the fluid mud inside the sediment trap after it being fully filled.
- Filling time without discharging, the time needed by the sediment trap to be fully filled by the fluid mud from WID without the suction system working.

It is found that the WID produces fluid mud in a higher rate than the pumping capability of the suction system. This condition leads to a much faster filling time compared to the emptying time. When the sediment trap is full, WID is not recommended to be operated as the fluid mud created can flow to other locations instead of the sediment trap.

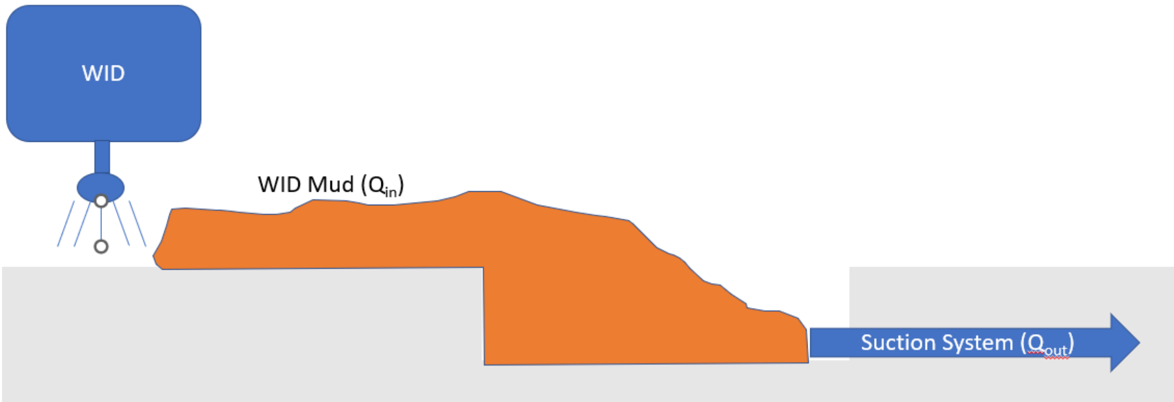


Figure F. 1 Illustration of sediment trap filling. The filling rate (Q_{in}) depends on the volume of fluid mud comes from WID while the pumping rate (Q_{out}) is determined by the discharging rate of the suction system.

Table F. 1 Filling and emptying time of the sediment trap depends on the density

Time (hour)	Density (kg/m3)	
	1150	1200
Time until full while discharging	5.3	9.6
Emptying time from full to empty	9.6	9.6
Filling time without discharging	3.4	4.8

Based on this analysis, WID is planned to work for 5 hours per day based on the time until full while discharging with the assumption of the fluid mud needs time to flow to the sediment trap. When the fluid mud is flowing to the sediment trap, settling and consolidation happens which increases the density of the mud until around 1150 kg/m^3. The WID then stop working while the suction system is still transporting the sediment out of the sediment trap for 10 hours. After the sediment trap is emptied, bed leveller can be used for moving the sediment that is also already softened by the WID but not flowing to the sediment trap. The density of the mud dredged by a bed leveller is 1200 kg/m^3 (M. Hoek, 2022. Personal communication). With the production rate of a bed leveller is around 1200 m^3/hour sediment (M. Hoek, 2022. Personal communication), using a bed leveller for 7.5 hours while the suction system is working and continued by emptying the sediment trap again for 1.5 hours complete the full cycle of 24 hours with an empty sediment trap at the end of the cycle. Figure F. 2 shows the daily operational plan.

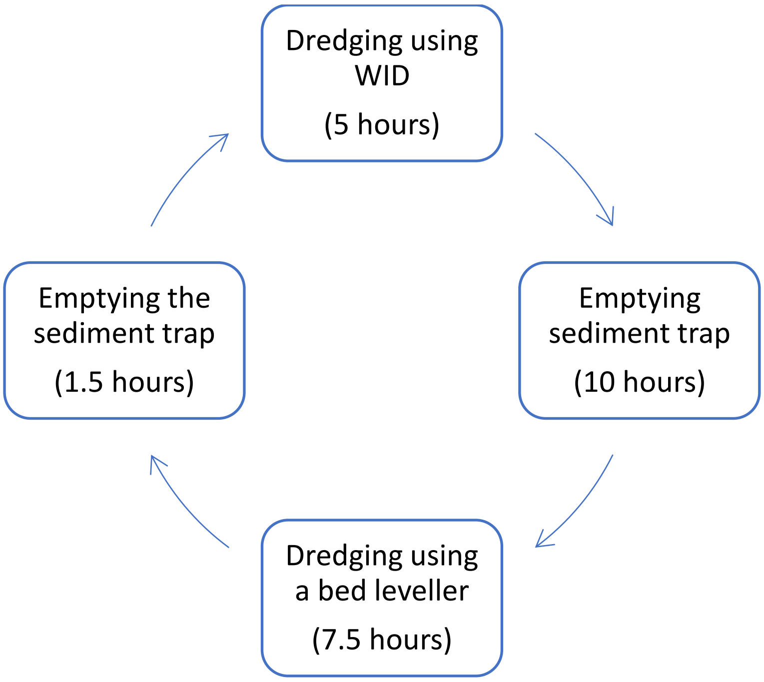


Figure F. 2 The daily cycle for doing a combination of central suction-WID system and a bed leveler. The total time for this plan is 24 hours, which means it can be repeated in a daily basis.

The WID and bed leveller combined has a productivity of 19000 m^3/day. For the dredging volume of 1.8 million tds sediment, it takes 287 days per year to reallocate all of the sediment. If this plan is operated for the full year, it can dredge 2.3 million tds sediment.

Combination of a central suction-WID system and a bed leveller can be a good alternative to be implemented. It is cheaper than using either only TSHD or WID. It also produces less greenhouse gases compared to the other alternatives. Bed leveller is very cheap to operate and also consume much less fuel than the other dredgers which leads to a cheaper cost and less greenhouse gases production.

The operation plan depends on the capacity of the sediment trap due to a high production rate caused by a WID. The density of the fluid mud also affects the operational, especially the filling time. Mud with lower density has a lower sediment content and leads to a faster sediment trap filling time.