CIE4061-09

Multidisciplinary Project

MDP 291

Port of Rotterdam Intertidal wetland

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Abstract

The Port of Rotterdam has many old harbours located close to the Rotterdam city center that are no longer suitable to be used for industrial purposes. Meanwhile due to expansion and population growth of the city, more recreational spaces are needed. The idea is to use the abundant dredged material from the Port of Rotterdam to fill in and construct intertidal wetland parks in some of these old harbours. They will serve as natural habitats for different types of flora and fauna such as migratory birds. These intertidal parks are also ideal recreational spaces for residents. This multidisciplinary project aims to provide a conceptual design of a tidal wetland in the Maashaven harbour. In this report, a general design is presented, and special attention is paid to technical issues that may occur in the construction process.

Content

1. Introduction

The Port of Rotterdam must dredge over 12 million m3 of sediment each year to keep Europe's largest port in operation. Normally, the dredged material would be dumped at sea, at a place called Loswallen, 12km away from the estuary of River Nieuwe Maas. Both the dredging and the disposal cost a significant amount of money. Unfortunately, 30-40% of the sediments will eventually come back to the harbour in a return current. Additionally, the amount of dredged sediment has been rapidly increasing each year. As a result, the Port of Rotterdam is looking for new and innovative ways to deal with this dredged material. (Veelen, 2019){Haberl, 1995 #3}

Since the construction of Maasvlakte 1 and 2 at the mouth of the Nieuwe Maas, many of the inner-city harbour's of Rotterdam are no longer functioning or suitable for industrial purposes. This creates a great opportunity for urban renewal. The City of Rotterdam plans to develop some intertidal parks or wetlands inside these harbours. This creates green spaces in the city and provides rich habitats for many migratory birds, fish, plants and other animals. The sediment from the dredging activity of the Port of Rotterdam become an ideal resource of hydraulic fill for the wetland. This is clearly a more beneficial use of sediment than simply dumping it at sea and is a very sustainable innovative solution.

The objective of this multidisciplinary project is to tackle the technological problem in using dredged material to develop a wetland and provide a concept design developing wetland in one specific harbour, Maashaven. (de Bruijn, 2018) The reason why we choose Maashaven is that the City of Rotterdam shows great interest in Maashaven.

1.1 Current site situation

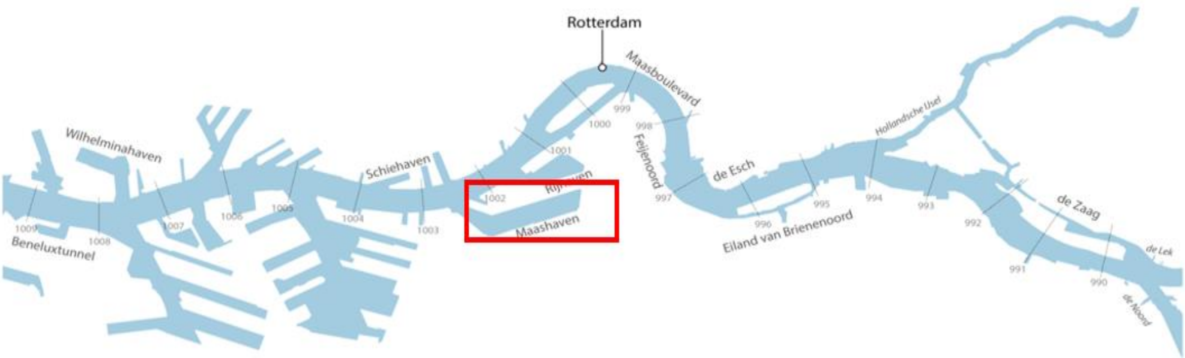

Figure 1 Layout of the Port of Rotterdam and location of Maashaven

Maashaven is a large, open water basin harbour of the Port of Rotterdam. It is located very close to the city centre of Rotterdam, which is an important reason why it is no longer suitable for industrial use anymore. {Veelen, 2019 #14}

The opening of Maashaven is relatively narrow and parallel to the river, after which the harbour bend parallel to the river. This location ensures that water does not easily enter the harbour. The Maashaven is at the middle of a slight outer bend. Almost all of the quays are vertical and there are many old industrial sites on the harbour. Sediment in the deeper layers of the soil could be contaminated. (Anne Zaat, 2019)

As mentioned above, water does not come in Maashaven easily. So the flow rate inside the harbour is quite low. The average tidal range is about 1.65m, which indicates high tidal energy.

The geometry of Maashaven is shown in the figure below. The total area of Maashaven is approximately 650000m2. The water depth inside Maashaven varies from 3 to 10 m. The topography of the harbour will be shown later in the report.{Veelen, 2019 #14}

Figure 2 Dimension of Maashaven harbour

1.2 General design

This design is based on the principle of a Contained Dike Disposal with modifications to make a wetland habitat.

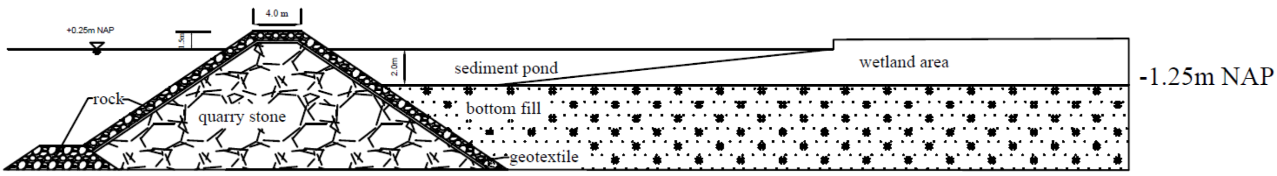

Figure 3 Cross-section of intertidal wetland

A dike is built near the harbor entrance as a barrier to prevent sediment loss from the wetland zone to the river. A weir at the crest of the dike controls the water level inside the confined area. At the wetland side of the dike, the bottom is elevated by 8 meters with dredging sediment. Since the original 10 m water depth is too deep for wetland construction. A 2m deep sediment pond is designed next to the dike at the wetland side. The purpose of the sediment pond is to catch the sediment flowing out from the wetland during consolidation and allow the fine grainers settling time. {Einsele, 2000 #13}

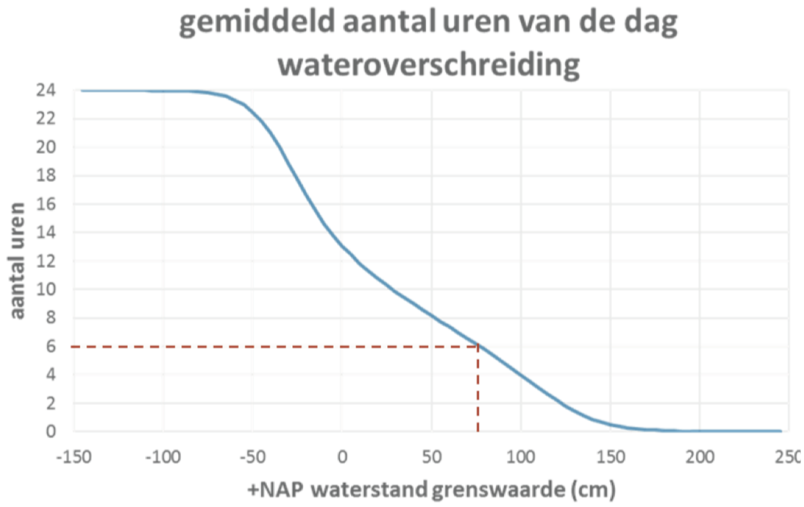

Figure 4 Relation between water level and time for intertidal area

Then, a gradual slope will be developed as the wetland toe. According to Figure 4, the land will be permanently underwater at the level of -1m NAP. Above +0.75m NAP land can obtain at least 6 hours above water daily, which makes +0.75m NAP the critical design level in this project because the wetland needs at least 6 hours above water in one day to maximize the growth of plants. Therefore, the vegetation area should be above +0.75m NAP. Permanently emerged ground should locate above +1.5m NAP. {Anne Zaat, 2019 #15}

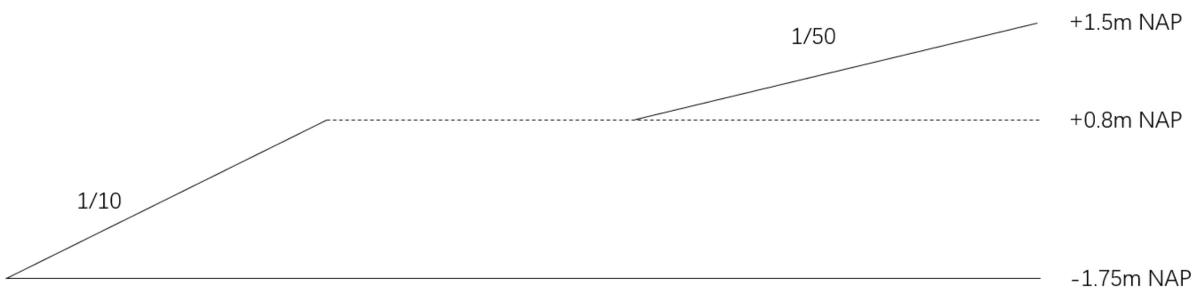

Figure 5 Cross-section slope design for wetland

The bottom level of the wetland area is -1.75m NAP. From -1.75m NAP to +0.8m NAP, the slope will be 1:10. At +0.8m NAP the land will extend horizontally for a certain distance, resulting in a large intertidal platform, providing spaces for vegetation to grow. The distance of extension should be determined based on the landscape designed by architects. At the end of the platform, the land will be elevated to +1.5m NAP with a slope of 1:50 to host some plants that cannot grow in an inundated environment. {DuPold, 2005 #19}

The wetland will have a curved sediment pond into the main area to allow the maximum amount of slope for marsh growth this is done in order to maximize the variation in topography. See the plan below. {Einsele, 2000 #13}

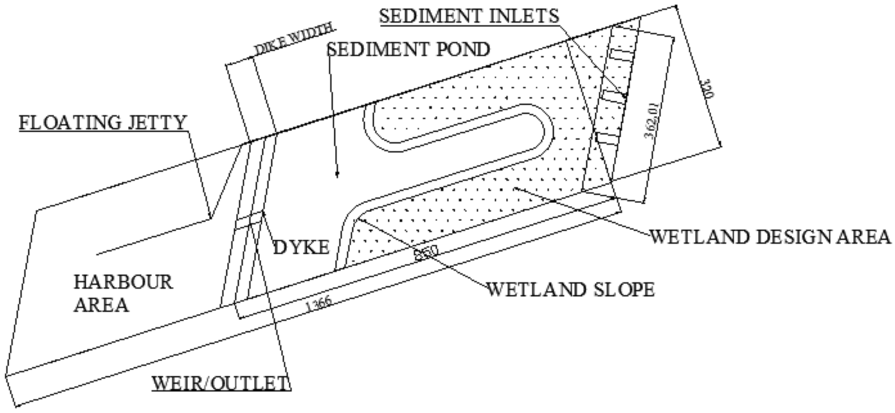

Figure 6 Maashaven harbour intertidal wetland design layout

2. Wetland Design

2.1 Introduction

Filling the harbour basin is a similar process to land reclamation or hydraulic fill. Dredged sediments from maintenance dredging harbour will be the fill material used including sand silts and clays. Delivery will take place by regular Trailer Section Hopper Dredgers that are operated directed by The Port of Rotterdam and also Rijkswaterstaat. To understand the filling process, it is first important to understanding the dredging process which will act as the source material for the wetland construction. {de Bruijn, 2018 #12}

Dredging in the port of Rotterdam

Maintenance dredging is constantly required in the Port of Rotterdam to ensure that the waterways have the required depth to remain navigable. As Europe's largest port this is critical to its operation and the business that rely on it. The Dredging is contracted to Van Der Kamp using 2 main Trailing Suction Hopper Dredgers: Hein and EcoDelta with a capacity of 3656m3 and 5901m3 respectively. They work 24 hours a day dredging the harbour and disposing of the sediments continuously. Port of Rotterdam divides the harbours into many different sections each given a 3 letter code example (AAP) which are individually managed see figure [17] below.(de Bruijn, 2018; Kamp, 2019)

The Port of Rotterdam uses 2 survey vessels to measure the depths in the harbour and directs the hopper dredgers to each section to ensure the water remain navigable. Nieuwe Maas main channel itself is managed independently by RWS Rijkswaterstaat. Typically, sediments are dumped in the North Sea in a place called Loswallen roughly 12km from the Port however there is a return current that brings roughly 30% of the sediment back to harbour which is a major concern of the Port. Periodic testing is done in each section to ensure sediments contained are not contaminated. If the dredging sections are found to be contaminated, they are taken to be dumped at The Slufter facility, a confined dike disposal site located in Maasvlakte 2. However this is considerably more expensive with a limited capacity, so it is only used when necessary. (de Bruijn, 2018)

Figure 7 Abbreviation for Rotterdam ports

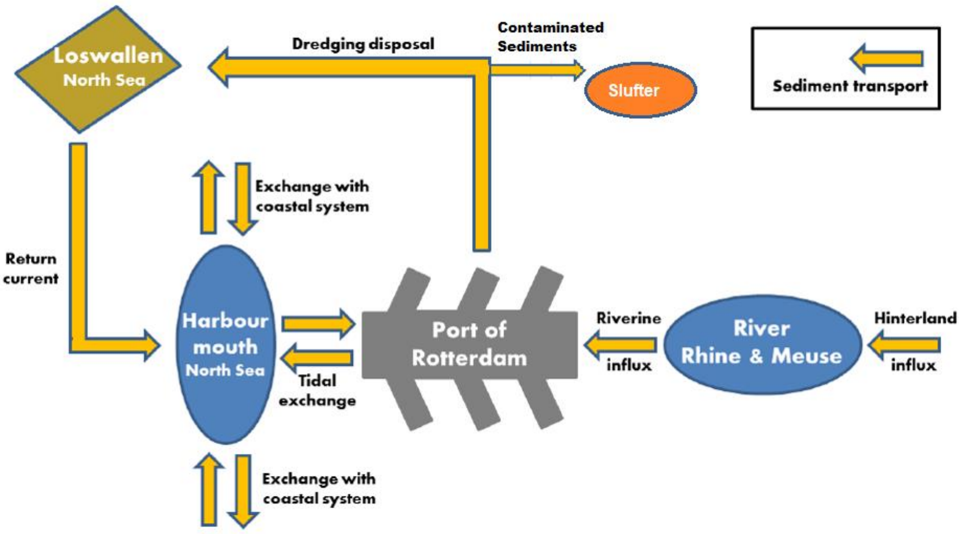

Figure 8 Dredged sediment flow chart

Maintenance dredging quantities have been increasing from 2012 with over 11.3 million cubic meters in 2016. This has led the Port of Rotterdam to consider different method of disposing of the sediments. Considering the capacity of our site is approx. 2.5 million cubic meters the site could be filled within several months however given that it must have no contaminated soil and it should be mainly sand it will most likely take longer in order to give the right soil profile. It certainly seems possible that port could be completely filled in time for the planting season in Spring 2020 this will however depend on the completion of the dike, the consolidation of soil and the stability of the dike. (de Bruijn, 2018)

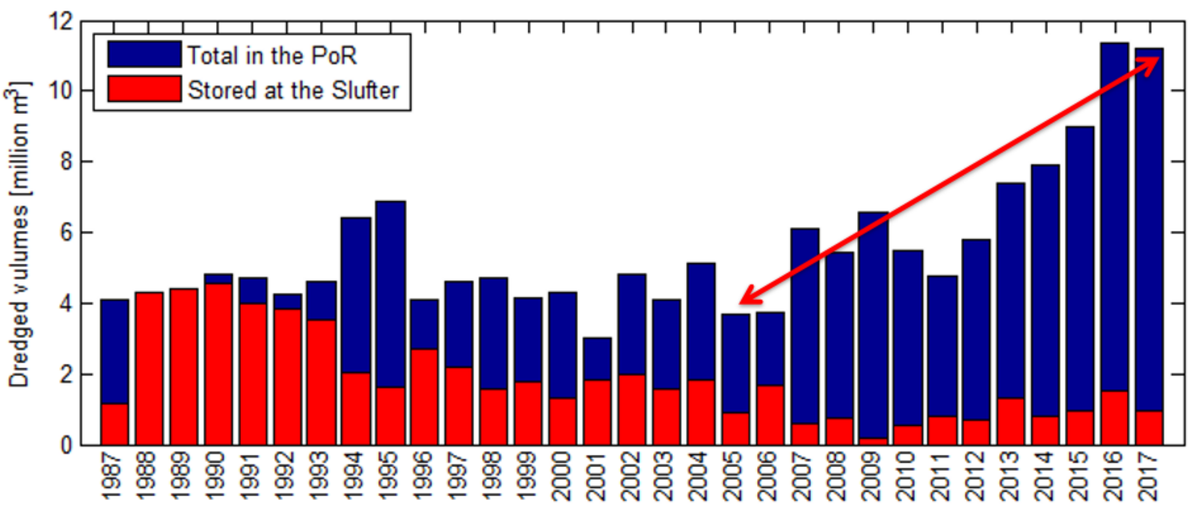

Figure 9 Maintenance dredging volume

Dredging vessels in the Port

Ecodelta

Length l.l.:
120,08 metres

Length o.a.:
134,10 metres

Breadth:
21,40 metres

Draught:
7,77 metres

Hopper capacity:
5.901 m^3

Suction pipe:
Ø 1000 mm. Dredge pump, drive unit 1500 kW

Maximum dredging depth:
37 metres

Discharge system by means of:
a. bottom slides
b. shore delivery unit, maximum 3.000 kW

Propulsion:
2 thrusters
2300 kW each

Speed:
empty 13 miles and loaded 11 miles/hour

Accomodation:
18 persons

Figure 10 Ecodelta vessel data

Figure 11 Ecodelta vessel figure

Hein

Lenght 1.1.:
98,81 metres

Length o.a.:
107,00 metres

Breadth:
15,47 metres

Draught:
6,00 metres

Hopper capacity:
3656 m^3

Suction pipe:
Ø 700 mm. electrically driven submerged pump, drive unit 1100 kW
maximum dredging depth
35 metres

Discharge system:
a. bottom slides
b. shore delivery units, maximum 3192 kW

Barge unloading system:
maximum 3192 kW

Propulsion:
2 thrusters,
1118 kW each

Speed:
empty 11 miles and loaded 10 miles/hour

Accommodation:
13 persons

Figure 12 Hein vessel data

Figure 13 Hein vessel figure

Hopper dredging time

The following table features an estimate of the dredging time required to fill the harbour. This is the governing dredging cycle of the Hopper Dredgers which given by the loading time, return travelling time loaded, unloading time and unloaded traveling time. The following diagrams describe the process. An estimate of the total time is given in the table below. Full details of the calculations used can be found in the appendix. There are a large number of estimates in the calculations but given the fact the dredging boats operate 24 hours a day it should be possible to fill the harbour completely in around 120-150 days however it was not possible to give an accurate standard deviation for the these calculations so it should be assumed to be much longer for certainty.(Schrieck & Tu Delft, 2006)

Table 1 Hopper dredging time

Cycle	Calculation method	estimate
Time to load	Function of the capacity and the discharge velocity	mean 32.5min
Time to travel to dredging area full	Function Average travel distance between the dredging site and harbor/ speed at full load	mean 90min
Time to unload at Maashaven	Pumping take the longest of all the 3 dredging disposal methods methods	Pumping ashore: 60 – 180: mean 120min
Time to travel to dredging section	Function average travel distance between the dredging site and harbor/ speed unloaded	mean 80 min
Total	Average time per load	320min
Average load	Average load of both boats	3656+5901 = 4780m^3
Rate of fill per minute	Average load/average time	4780/320min= 14.9m^3/min

Rate of fill per day	Rate of fill per minute converted to days	21500m^3 per day
Volume of fill	Determine by cross section of fill	2500 0000m^3
Min filling time	Volume of fill/ fill rate days	116 days

Nautical depth of Rotterdam

The nature of the sediments in the port is very specific to the port and has very large implication in designing the wetland. Properties of the sediments will determine the compressibility and strength of the wetland material. The sediment dredged from the harbor is referred to as 'fluid mud' which is defined as a cohesive mix of clay, silt and fine salts. The properties of the sediments dredged are defined by their position in the seabed, The Port of Rotterdam uses surveying ships to monitor the seabed to determine the suitable navigation depth. This is defined as fluid mud density 1.2kg/l If a ship is to pass the basin with a draught below this limit it must be dredged beforehand. This is done by surveying ships that use low frequency echo sound with different frequencies. The change in density causes a reflect of the sound waves at junction between layers. The layer above the nautical depth of 1.2kg/l is a semi fluid mud between 1.1kg/l and 1.2kg/l. Below this is preconsolidated mud with a density below 1.3kg/l. This mud has begun to show solid like properties as the mud settle and the water is removed. Below the preconsolidated level is the consolidated layer here the mud has mostly solid properties. Trailer section hopper remove sediments mostly within the preconsolidated layer and consolidated removing and transporting the sediments to Loswallen. [15]. Kirichek A Chassagne . Note that the definition of consolidated in this process is slightly different that the of geo-engineering as it refers to the point where the mud begins to behave like a solid not long term removal of water in soil consolidation which can take several years.

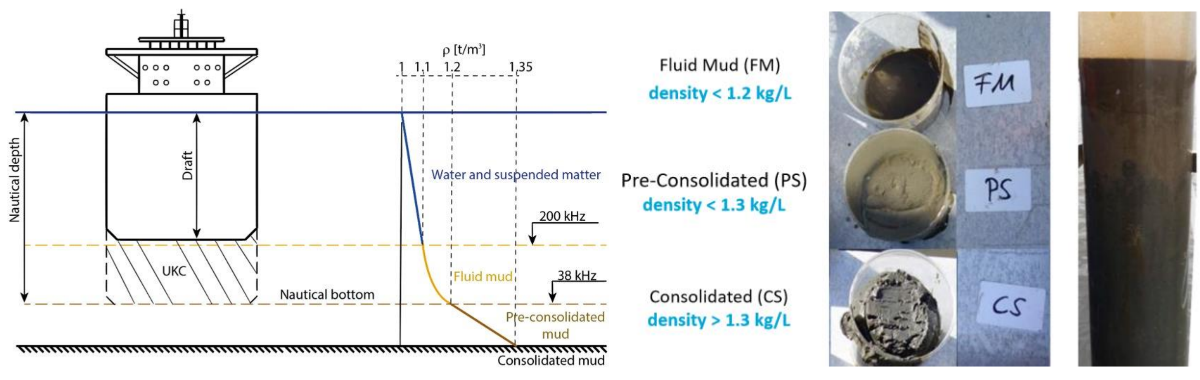

Figure 14 Sediment survey and sediments

Sediment properties

Collecting actual sediments from these locations and analysis them within the lab we can deduce certain properties. These samples were taken at places of high sedimentation so they would be suitable fill for the Maashaven site.

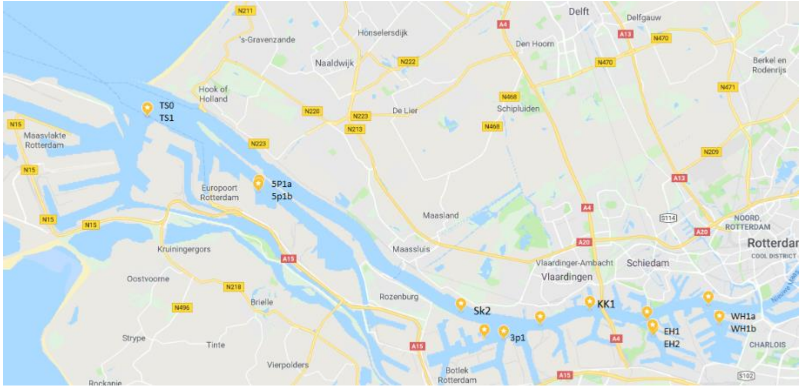

Figure 15 Sediment collect points

Sediment properties

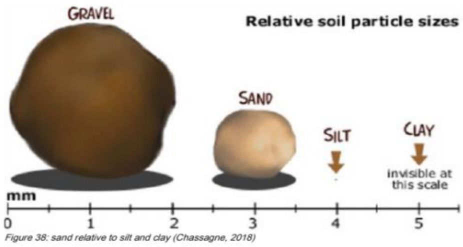

Figure 16 sediment grain size

- Mostly silt with fine sands and with very small quantities of clay 3% mud similar sizes through the harbour
- D50 between 10 um-40um
- Salinity between 3.5g/L to 19.1g/L Brackish-Brine
- Density between 1.1/kg/l to 1,3g/L Fluid and preconsolidated mud
- Zeta Potential between -2 to -14.8
- Fluidic yields stress between 0.5pa to 26,7pa

Example sediments Waalhaven Particle size distribution

D(.1) = 4.645um d(0.5)=18um d(0,9)=95um

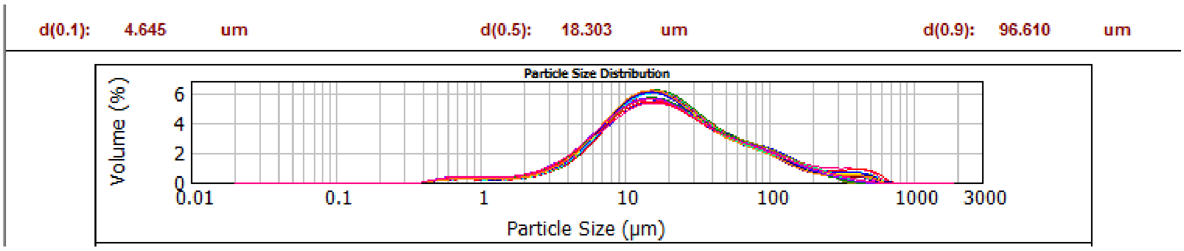

Figure 17 Waalhaven Particle size distribution

Being mostly silt with some clay the soil behaves like cohesive soil. This allows flocs to form and decreasing settling time. It also gives the soil additional strength which is helpful in the settling process. Being mostly silt is also highly compressible as it will form a thick mud mix when placed on site with density of about 1.3kg/L

Hydraulic Fill and pumping

The process of placing sediments in the harbour for wetland is very similar to a hydraulic fill or land reclamation problem. There is a long history of these sorts of projects in the Netherlands with Maaklavate 2 being one of the most recent ones. There are a number of different constraints to such a problem including placement sedimentation, slope stability, sediment capture, soil compaction, settlement and soil improved.

Figure 18.5 Maasvlakte 2 reclaimed land project 2013

Sediment placement.

Trailer Section Hopper Dredgers have 3 types of placement methods bottom dumping, rainbowing and pumping through a spreader of diffuser. Bottom dumping is where the TSHD open their bottom doors and unload sediment it is extremely quick it usually takes about 5 min and is by far the cheapest method of placing sediment however could only be used in the initial

Figure 19 Hopper dredgers rainbowing

stages of the construction. Since the plan is to build a dike to stabilize the sediment capture and prevent it returning to the harbour it would need to be done before this process. Since there is no dike to protect the sediment a lot of care would need to be taken to ensure that it does not drift back into the harbour. Additionally, as the draft of the boats are quite deep 7.7m and 6m for Ecodelta and Hein respectively with a harbour only 10m it is unlikely it could be done a long time as there must be some space for the TSHD to maneuver as well. (Kamp, 2019)

The next form of sediment placement is known as rainbowing where dredged sediments are forced out the TSHD bow of a dredging boat at high speeds in an arch. This is done at high speeds and can reach up to 150m and is quite quick the process takes about 45 min. This method is often used for beach nourishment due to its flexibility and ability to place material with some accuracy however it would be unsuitable for an inner-city project as fine grained sediments and nauseas salts may disperse amongst the city building create a hazard for residents.(Hoff, 2011)

The final method of sediment placement is the via pipeline. This is the most controlled but also the most time consuming and expensive, however given the constraints of the project it is suitable for most if not all of the sediment placement. There are few things to consider when using a hydraulic pipeline. The pipeline should be as short as possible and as straight as possible as bends in the pipe will increase the hydraulic resistance and increase the need for a more powerful pump or booster stations such as the Auger pump. Therefore, the front of the harbour in front of the dike should be kept open to allow access to the dredging boats. To maintain flow velocity the pipeline should be kept above the water level by means of pontoons. A diffuser is often used to spread-out the sediments this also reduces the kinetic energy and prevents damage to already settled soils. Alternatively, the pipe could be attached to a spreader pontoon or a nozzle in order to spray or 'rainbow' the fill. (Hoff, 2011)

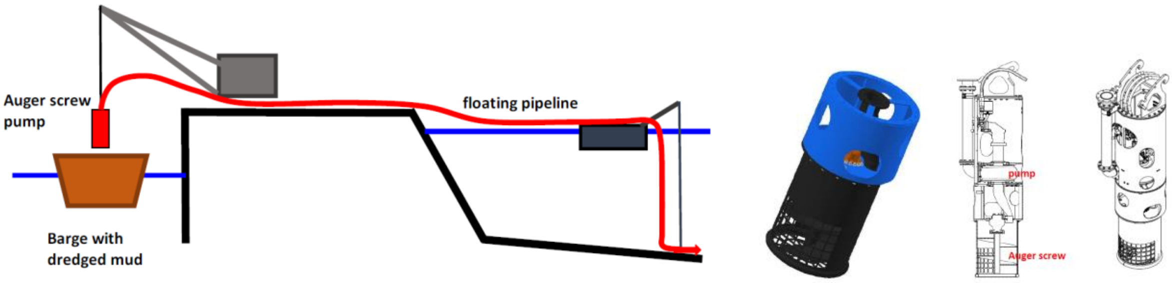

Figure 20 Cross section of pumping process with pump

Soil enrichment

Figure 21 Truck dumping and profiling with excavators

Once the soil reaches above water it should be placed at least 1 – 2m above ground to ensure a good spread of the settlement. In these above ground cases bulldozers should be employed to level the sand however care should be taken to prevent liquefaction as fine grained non-cohesive soils are sensitive to vibration. Materials also tend to separate, and this can affect the quality of the fill care should be taken to remove fine grains soil where they present a stability problem or allow them to settle in the sediment pond. (Hoff, 2011)

Other Important aspects to consider in hydraulic fill include:

- The pumping capacity of the dredgers whether it is sufficient to reach the required destination sometimes this may necessitate a booster pumping station
- The total length of pipeline available\the type of fill supply (continuous/discontinuous)
- The nature of the fill (particle size distribution, granular/cohesive, particle density, shape of particles, etc. and the homogeneity of these properties during filling
- As the geometry of the park is circular the pipes may need to be moveable.

- the existing infrastructure in the vicinity of the reclamation this is especially important in a busy harbour Maashaven as it can cause disruption to resident or industry.
- limitations imposed by permits, permissions
- the discharge should generally as be as far from the weir box as possible to allow the fine grains to settle without flowing out.
- As the development of shear stress in fine grains can be difficult to predict and can take a very long time it may be advisable to perform soil improvement methods to improve the liquid limit by 1.5 to 2 times. (Hoff, 2011)

Slope angles

A big factor in placing the fill is determined by the grain size and the velocity of the pumps. This will play a huge role in the establishment of the slopes essential to the development of stability and vegetation.

The slope below water is governed by the following equation (Schrieck & Tu Delft, 2006):

$$i_{UW} = 3200ds^{-0.4}$$

Where i_{UW} is the natural under water slope

d is the grain diameter

s is specific sand mass flow discharges ($s = \rho_k qc$

c is the volume concentration of solids

And the slope above (Schrieck & Tu Delft, 2006):

$$i_{AW} = 0.006(\frac{d}{d_0} - 1)(\frac{q}{q_0})^{-0.45}$$

Where i_{AW} is the natural slope above water

d is grain diameter

d_0 = 65

q is the specific mixture flow per meter width

q_0 = 1

This gives a range based on grain diameter (Hoff, 2011)of:

Table 2 Indicative natural slopes of granular material hydraulically placed by pipeline discharge from above water (from Athmer & Pycroft 1986)

Grain size [mm]	Indicative range of slopes		
		Below water	Below water
	Above water	Calm seas	Rough seas
0.060–0.200	1:50–1:100	1:6–1:8	1:15–1:30
0.200–0.600	1:25–1:50	1:5–1:8	1:10–1:15
0.600–2.000	1:10–1:25	1:3–1:4	1:4–1:10
> 2.000	1:5–1:10	1:2	1:3–1:6

As there is a lot of silt with low grain size it may be beneficial to use sand above water where a higher slope is desired however it is clear that careful placement play a role via the ‘s’ the specific sand discharge for below water and ‘q’ the specific flow per meter width.

2.2 Soil consolidation

Consolidation is a hugely important consideration in the design of the wetland because of the extreme sensitivity to tidal height of each wetland plant. The desired wetland height of 0.75m NAP guides most of the design since settlement can lead a lower height of the soil over time it needs to be considered. This need to be carefully considered as additional material may need to be placed in order to reach the desired height as the soil consolidates. Consolidation occurs both in the existing subsoil and in the hydraulic fill that is placed. There are 2 types of consolidation primary and secondary. Primary consolidation involves the removing of water in the pore’s under load. Secondary consolidation involves rearranging of particles over time. Addition

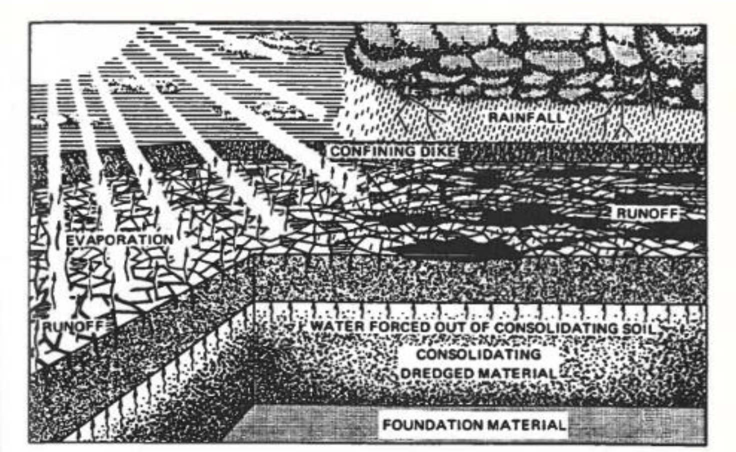

Figure 22 Conceptual diagram of dredged material consolidation and dewatering processes

settling of the subsoil will need to be considered as an additional load is place on the soil. This relationship can be expressed by these 2 graphs to receive the desired NAP (Herbich, 2000)

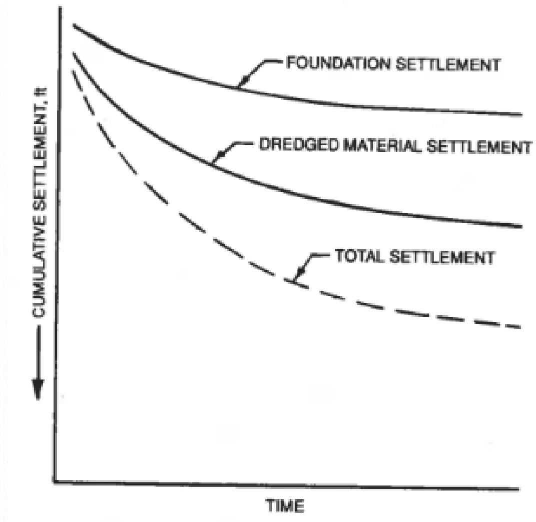

Figure 24 Cumulative settlement based on time scale

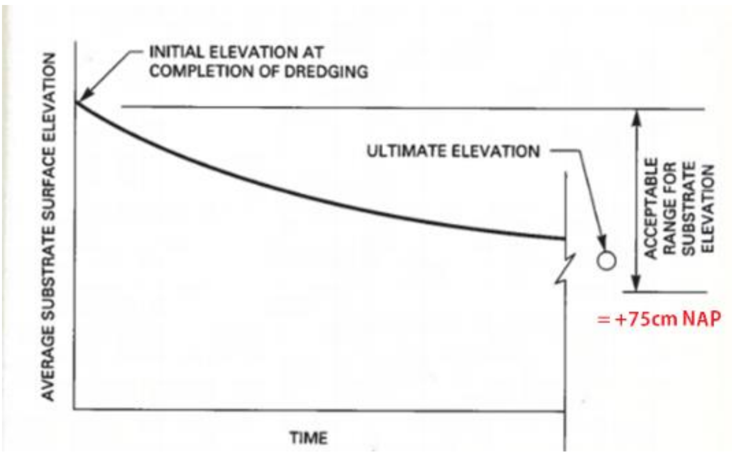

Figure 23 Average substrate surface elevation

Analysis method

D-settlement a software program is used to calculate consolidation because of the high accuracy of the model as opposed to numerical methods such as Terzaghi spreadsheets which don’t include creep and can be difficult to calculate with multiple layers. Deltares is a Netherlands company and the model is very suitable for analysing soil and has been collaborated to managing in the Rotterdam delta area. The. Settlement analysis is conducted on the dike profile in 2d dimensions and in 1d for the main fill area. model selected Tergazhi model one dimensional strain consolidation with a NEN-Bjerrum Isotache strain model linear strain which supports the common linear strain parameters Cr, Cc and C.(Deltares, 2016)

The model are governed by the following equations:(Deltares, 2016)

Terzaghi – General consolidation theory

The degree of consolidation U:

$$\mathrm{U(t)} = 1 - \frac{8}{\pi^2}\sum_{i=1}^{\infty}\frac{1}{(2i-1)^2}\exp[-(2i-1)^2\frac{\pi^2}{4}\frac{c_v t}{d^2}]$$

Where c_v is the coefficient of consolidation

d is the drainage depth

t is the time

If the vertical effective stress after loading is smaller than the preconsolidation pressure σ_p σ_p, the primary settlement contribution according to the idealized behavior can be calculated from:

$$\frac{\Delta h_{prim}}{h_0} = RR\log\frac{\sigma'}{\sigma_0} \quad \sigma_0 < \sigma' < \sigma_p$$

If the vertical effective stress after loading is larger than the preconsolidation pressure σ_p σ_p, the primary settlement contribution according to the idealized behavior can be calculated from:

$$\frac{\Delta h_{prim}}{h_0} = RR\log\frac{\sigma_p}{\sigma_0} + CR\log\frac{\sigma'}{\sigma_p} \quad \sigma_p < \sigma'$$

If the vertical effective stress after loading is larger than the preconsolidation pressure σ_p σ_p, the secondary settlement contribution according to the idealized behavior can be calculated from:

$$\frac{\Delta h_{prim}}{h_0} = C_\alpha\log\frac{\mathrm{t}}{\tau_0} \quad \sigma_p < \sigma'$$

With $\mathrm{RR} = \frac{C_e}{1+e_0}$ $\quad$ $\mathrm{CR} = \frac{C_\tau}{1+e_0}$

Where C_τ is the reloading/swelling index below preconsolidation pressure

C_e is the compression index above preconsolidation pressure

C_α is the coefficient of secondary compression above preconsolidation pressure

Δh_{prim} is the primary settlement contribution of a layer

h_0 is the initial layer thickness

e_0 is the initial void ratio

SubSoil locations

Information for the subsoil profile was taken from the Dinoloket database in the vicinity. All boreholes were analyzed with the Boreholes B37H0658 and B37H0658 were considered to be the most important for analyzing the dike and sediment consolidation respectively.

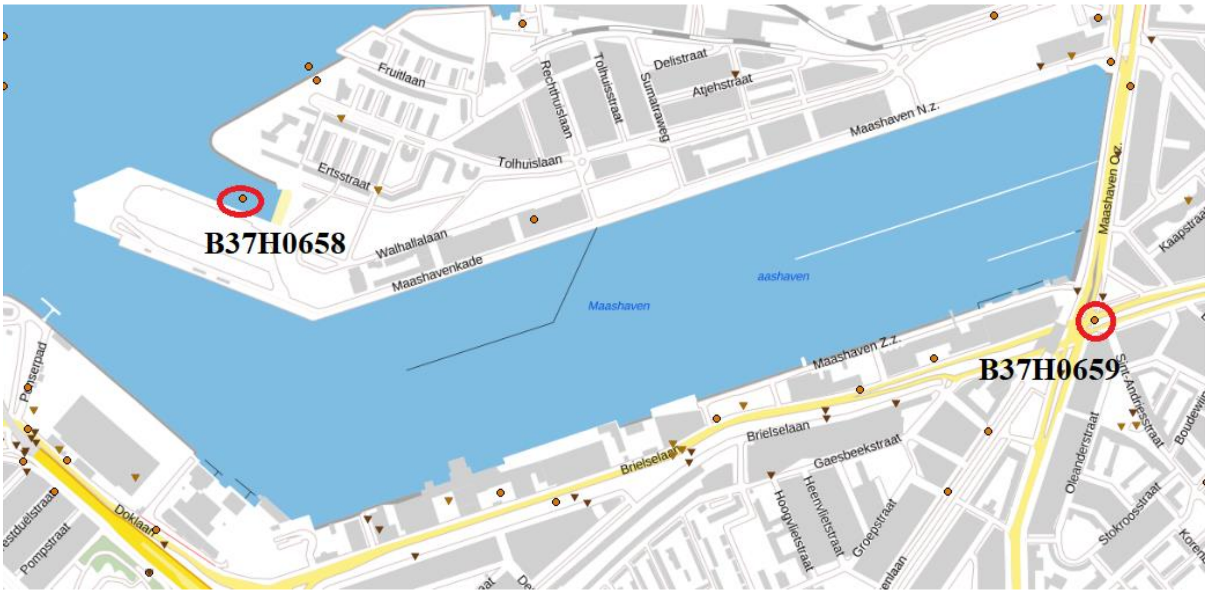

Figure 23 Boreholes from the Dinoloket database

As we are unable to obtain any direct information from the Port of Rotterdam about the subsoil properties directly in Maashaven. We have used Tutorial 2 from the D-settlement handbook as a basis for the consolidation properties. Tutorial 2 takes it data from the crossing of the Dutch A2 highway and at a viaduct crossing at the N201 road nearby Vinkeveen. This is seen as a suitable comparative site as the information is complete, it is in the Netherlands and located close to water. (Deltares, 2016)

Figure 25 Location of soil data

A summary of all the properties can be found below:

Table 3 Soil properties

Layer number	Material name	Unit weight Unsaturated [kN/m³]	Unit weight Saturated [kN/m³]	Vert. consolid. coefficient Cv [m²/s]	Layer number	Reloading/ swelling ratio RR [-]	Compression ratio CR [-]	Coeff. of sec. compression Ca [-]
4	Medium Clay	13,94	13,94	2,47E-08	4	0,1320000	0,2370000	0,0262000
3	Sand	17,00	20,00	1,00E-08	3	0,0001000	0,0023000	0,0000000
2	Medium Clay	13,94	13,94	2,47E-08	2	0,1320000	0,2370000	0,0262000
1	Sand	17,00	20,00	1,00E-08	1	0,0001000	0,0023000	0,0000000

Dike settlement analysis

To obtain information about the subsoil we have used the Dinolocket database. The borehole B37H0658 from figure 23 was used to estimate settlement as it located underwater in Katendrecht haven as small harbour near dike and extends to 22m. (Dinolocket, 2019). This was considered preferable to many of the surrounding above ground boreholes as being in an underwater location it would subjected to similar erosion and sedimentation processes giving a similar soil profile as opposed to the boreholes from ground sections in the which may be largely undisturbed. To estimate the soil profile the top 10m from +0 NAP was removed as it is assumed to be the depth of the water level. With the profile below this is mostly a combination of clay and fine sand.

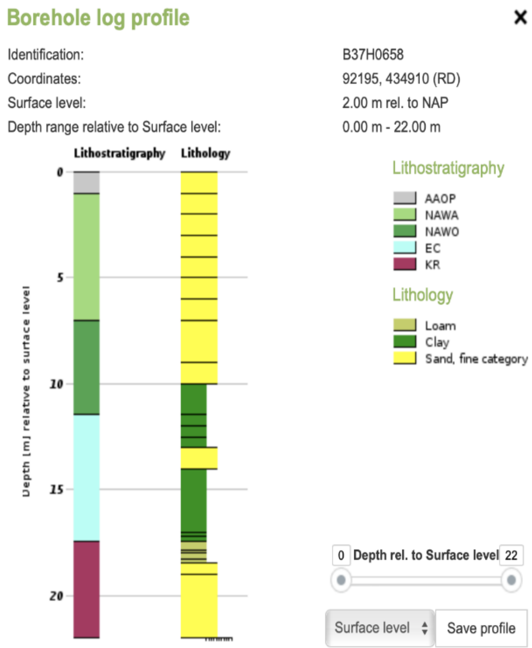

Figure 26 Borehole subsoil profile for dike design

Table 4 Soil profile used for calculations

From [m +MWL]	To [m +MWL]	Soil type
1.5	-10	Dike load 24/kN/m
-10	-13	Medium Clay
-13	-14	Fine Sand
-14	-18	Medium Clay
-18	-29	Medium Sand and Coarse Sand

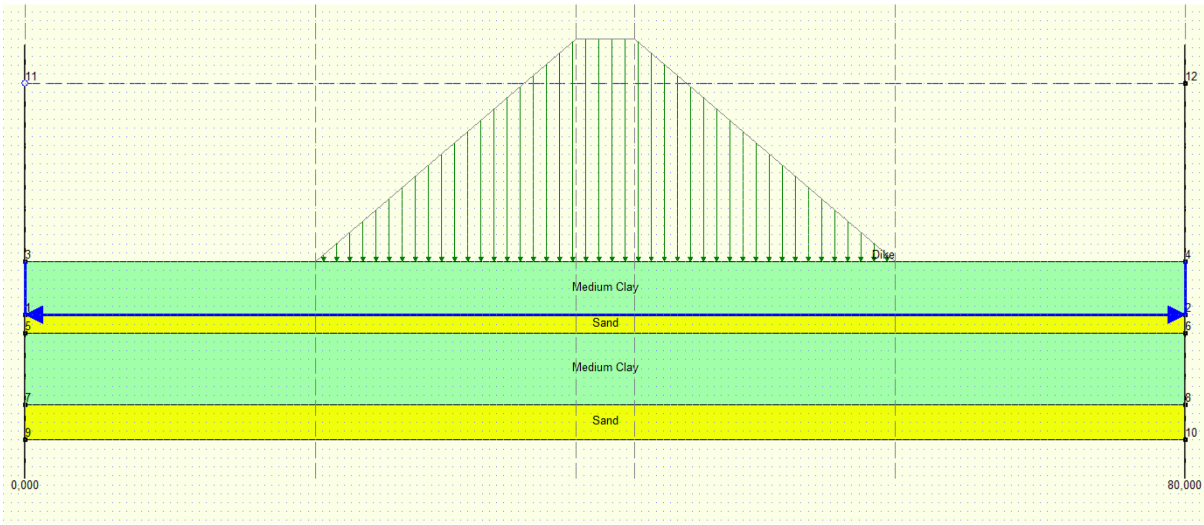

Figure 27 Model figure

Inputting these coordinates and the load given by the dike with a unit weight of 24kN/m3 for stone dike with a height of 11.5m or 1.5 NAP. We were able to obtain the following Settlement at 10950 days (30 years). Verticals 3 and 4 are the critical loads considered as they have maximum load on the subsoil. The following consolidation by time was obtained

Table 5 Settlements

Time [days]	Settlement [m]	Part of final settlement [%]	Residual settlements [m]
1	0,061	3,728	1,573
7	0,151	9,254	1,483
31	0,316	19,368	1,318
186	0,786	48,087	0,848
365	1,086	66,461	0,548
730	1,391	85,147	0,243
1825	1,577	96,508	0,057
3650	1,604	98,174	0,030
7300	1,623	99,328	0,011
10950	1,634	100,000	0,000

Consolidation was also done with the alternative dike design using the sand dike with a unit weight of 20kM/3 and a total height of 12.5m or +2.5NAP this consolidation was slightly less at 1.55m within 30 years.

Examining the progress of consolidation by date. We can see that there is substantial consolidation throughout the lifetime of the dike reaching approx. 1.6m over 30 years this settlement has been allowed for in the construction of the dike. There will also need to be a long-term maintenance plan put in place for the dike to make sure it is above the required water level. {Herbich, 2000 #8}

One Dimensional analysis of fill area

For the one-Dimensional analysis which is for the main fill-wetland area we have used borehole B37H0659 near the bottom right side of the harbour wall. This is considered the worst-case scenario as it has the greatest amount of compressible subsoils below 10m such as clay and peat compared to the other boreholes in the vicinity. However, this borehole is not located in an aquatic area so its accuracy for a subsoil profile could be disputed (Dinolocket, 2019).

Table 6 Soil profiles

Layer number	Material name	Unit weight		Reloading/ swelling ratio RR [-]	Compression ratio CR [-]	Coeff. of sec. compression Ca [-]	Vert. consolid. coefficient Cv [m²/s]
		Unsaturated [kN/m³]	Saturated [kN/m³]				
5	Sand	17,00	20,00	0,0001000	0,0023000	0,0000000	1,00E-08
4	Silt	13,	13,	0,1320000	1.5 0000	0,0262000	1,00E-07
3	Peat	10,15	10,15	0,1860000	0,4090000	0,0312000	3,05E-07
2	Medium Clay	13,94	13,94	0,1320000	0,2370000	0,0262000	2,47E-08
1	Sand	17,00	20,00	0,0001000	0,0023000	0,0000000	1,00E-08

Input

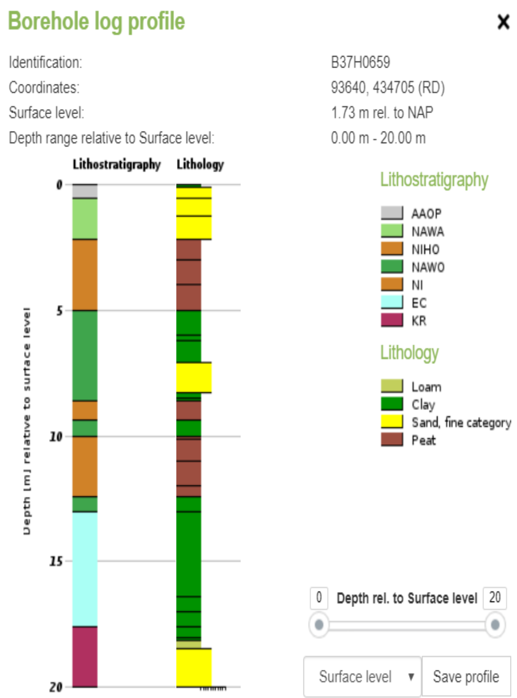

Figure 28 Input borehole profile

Estimating the properties of the hydraulic fill

As we were not able to perform necessary tests to estimate consolidation such an odometer test it is necessary to estimate the consolidation parameters of the dredged sediment. Silt is assumed to be dredged from between the preconsolidated layer so a bulk density of 1.3kg/l was assumed given some allowable settling time. When hydraulically placed the silt is considered to be unconsolidated OCR=1 with a relatively high cv of 10*10^-7.{Schrieck, 2006 #9} as the layer is very porous with a high water content it is take to have a consolidation Coeffiction Cc of 1.5 from literature. [17]. Develioglu I. Pulat H. (2019) Cr was assumed to be similar to the clay layer at 0.132.

Model approximation in D-settlement

Table 7 Model approximation input

From [m +MWL]	To [m +MWL]	Soil type
1.5	0	Sand
0	-10	Silt/ Dredged material
-10	-12.5	Peat
-12.5	-18	Medium Clay
-18	-20	Sand

For this section we have used the profile given by the borehole for the subsoil up until the -10 NAP depth of the harbour. The area above from -10m is to be filled by the dredged material from the port. The top layer is sand from 0m to 1.5m NAP to give a desirable sand beach layer and allow for a vertical load to enhance consolidation. This also allows for a buffer layer to reach the desired 0.75NAP. For the Medium clay, sand and peat the same soil properties were used for consistency with the dike layer however a new silt layer was added to simulate the dredged material. Computing these results in the D-settlement gave the following results:

Silt results

4.1 Settlement

Surface level [m]	Settlement [m]
1,50	4,578

4.2 Residual Times

Time [days]	Settlement [m]	Part of final settlement [%]	Residual settlements [m]
1	0,033	0,716	4,545
7	0,092	2,006	4,486
14	0,133	2,907	4,445
31	0,204	4,450	4,374
90	0,360	7,870	4,218
186	0,531	11,598	4,047
365	0,761	16,616	3,817
730	1,100	24,036	3,478
1825	1,791	39,127	2,787
3650	2,588	56,535	1,990
7300	3,731	81,490	0,847
10950	4,578	100,000	0,000

Figure 29 silt results

Following the settlement results for the silt and subsoil we can see a settlement after a year of 0.75m this would bring the wetland height to the desired wetland height of 0.75 NAP within a year so planting could begin. However the settlement continues after this point the combined settlement of the silt and subsoil to a total settlement of 4.5m in 30 years which is very significant. This indicates that incremental filling would need to take place to maintain the wetland height.

Silt settlement with drains

Vertical drains were added to the same profile to see the impact of additional drainage would have we can see that the additional of drains increases the speed and amount of consolidation. The removal of water would increase the overall strength of the soil increasing the bearing capacity and allowing heavier machinery to begin shaping the recreational park.

4.1 Settlement

Surface level [m]	Settlement [m]
1,50	5,807

4.2 Residual Times

Time [days]	Settlement [m]	Part of final settlement [%]	Residual settlements [m]
1	0,033	0,570	5,774
7	0,094	1,622	5,713
14	0,138	2,374	5,669
31	0,214	3,694	5,593
90	0,392	6,748	5,415
186	0,596	10,259	5,211
365	0,885	15,244	4,922
730	1,338	23,044	4,469
1825	2,313	39,838	3,494
3650	3,446	59,344	2,361
7300	4,914	84,613	0,894
10950	5,807	100,000	0,000

Figure 30 silt reults with drains

We can see the settlement is considerably faster reaching the desired wetland height of +0.75NAP somewhere between 6 months and a year. It also reaches a higher overall settlement of 5.8m over 30 years. This indicates that more water is removed increasing the overall strength and bearing capacity of the soil.

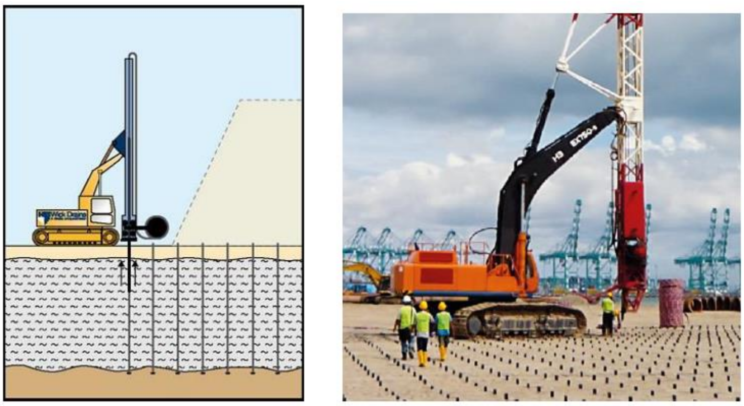

Figure 31 vertical consolidation drains

Alternate design Sand Drainage layers

An Alternate design was considered by adding drainage layers of 30cm between every 2m of silt this was chosen to increase the drainage rate and reduce consolidation time. It also increases the overall strength of the soil as it takes less time to drain and can support the above weight. Similar methods have been used with dredged materials before such as in the Francop disposal site in the Port of Hamburg.

Table 8 Soil distribution

From [m +MWL]	To [m +MWL]	Soil type
1,500	-0,800	Sand
-0,800	-2.8	Silt
-3	-3.1	Sand
-3	-5.1	Silt
-5	-5.4	Sand
-5	-7.4	Silt
-7	-0.77	Sand
-1	-9.7	Silt
-10	-10	Sand
-10	-12.5	Peat
-12.5	-18	Medium clay
-18	20	Sand

4.1 Settlement

Surface level [m]	Settlement [m]
1,50	3,993

4.2 Residual Times

Time [days]	Settlement [m]	Part of final settlement [%]	Residual settlements [m]
1	0,021	0,516	3,972
7	0,059	1,484	3,933
14	0,087	2,183	3,905
31	0,136	3,413	3,856
90	0,250	6,269	3,742
186	0,382	9,559	3,611
365	0,568	14,235	3,424
730	0,861	21,577	3,131
1825	1,500	37,559	2,493
3650	2,260	56,610	1,732
7300	3,300	82,659	0,692
10950	3,993	100,000	0,000

Figure 32 results

This method shows a similar consolidation rate to the drainage pipes however the total settlement overall is lower this indicates that site would gain strength quicker.

Conclusion

We can see that there is considerable consolidation both from the subsoil and the silt fill which is very porous. However, as the port of Rotterdam has considerably more silt to dispose of and sand has a potential value as a construction material it is preferable to use silt entirely as the fill. Using sand layers would require considerable expense to source and the added complexity would increase the cost of placing the sediment substantially. Therefore it's not a deemed a good alternative despite some increase in settlement rate. Given that the wetland is designed to be ready to plant within 1 year the 1.5m sand is a good solution to be place above the fill to obtain the desired +0.75m NAP. However due to the large ongoing consolidation After this period further material may need to be added incrementally as part of a long-term management plan. {Herbich, 2000 #8} Note there is a large amount of assumptions on these calculations due to lack of direct information in the literature on the compressible qualities of the soil accuracy could greatly be increased by performing an odometer test on dredged sediments from different harbours which would yield more accurate consolidation parameters. Additionally, boreholes could also be taken at sites within the construction zone to directly determine subsoil properties with these more accurate results could then be found and the filling elevation and constructions design could be adjusted accordingly.

2.3 wetland design

Sediment pond

Sediment pond is essential to allow the finer grains to settle when hydraulically placed. While large grains and sandy materials settle quit quickly clay and fine grains sediments can remain suspended in the fluid for some time. If there is significant current out of the wetland area these sediments could flow out of the location therefore a settlement pond is placed close to the dike allow the very fine grains settlement to settle. These is also typically a geotextile placed within the dike to prevent fine particles seeping through the dike. Additionally, at times of high deposition a geotextile curtain can be placed at the weir to prevent sediments flowing out.

Figure 33.4 example sediment pond

Settling velocity in very fine particles is often related to surface charge of the particles rather than gravity as in large grains. If the sediments are very fine, they often remain suspended as the negative charged particles are attracted to the positive charges of the water molecules. However eventually the particles collect positive ions and begin to attach to other particles to form aggregates this process is called flocculation. As these colloidal particles begin to grow in size and gravity begins to dominate and the settling velocity increases. This process increases in saline solutions with the addition of ions this governed by conductivity and a measure of the particles charge known as Zeta Potential ζ. This process can take several days and sometimes weeks for the flocculation process to complete. Hence a settling pond is required to retain the suspension in the wetland area. [16]. Chassagne C (2018).

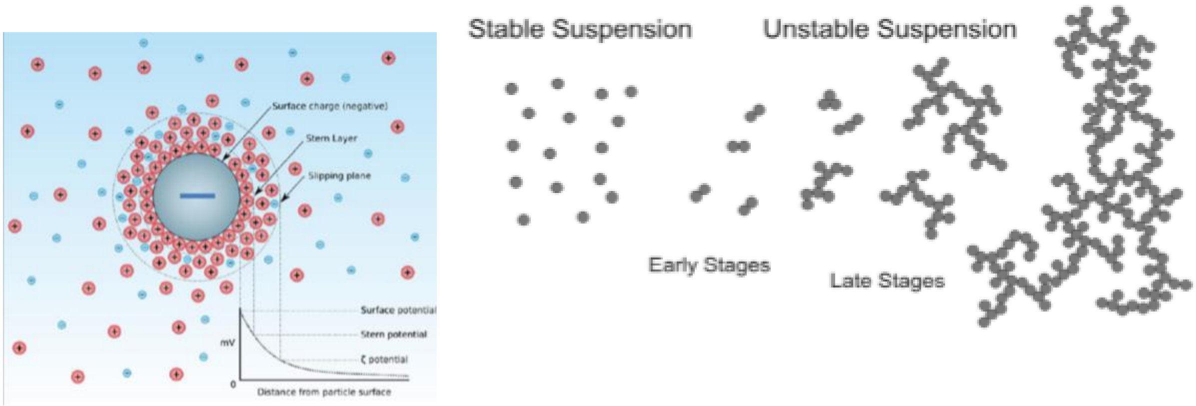

Figure 34.5 Zeta potential left and Flocculation of particles right

Sediment pond design

Since our sediment pond is permanently under water, therefore it should be designed as a wet sediment pond. Upper part of the sediment pond is the sediment settling zone. The minimum height of the sediment settling zone should be 0.6m. Due to the fact that the Maashaven is a large basin and large amount of sediment that would be used in the project, the height of the sediment settling zone is set to be 1.2m. (Einsele, 2000)

Table 9 Sediment pond slope

Slope (H:V)	Bank/soil description
2:1	Good, erosion-resistant clay or clay-loam soils
3:1	Sandy-loam soil
4:1	Sandy soils
5:1	Unfenced Sediment Basins accessible to the public
6:1	Mowable, grassed banks.

Considering the fact that the sediment pond is designed as part of the tidal park, a grassed bank will be in accordance with the surrounding environment and achieve a better landscape. Therefore, a slope equals or smaller than 6:1 should be used.

When the settled sediment has elevated the pond bottom by 0.8m, the pond needs to be cleaned out.

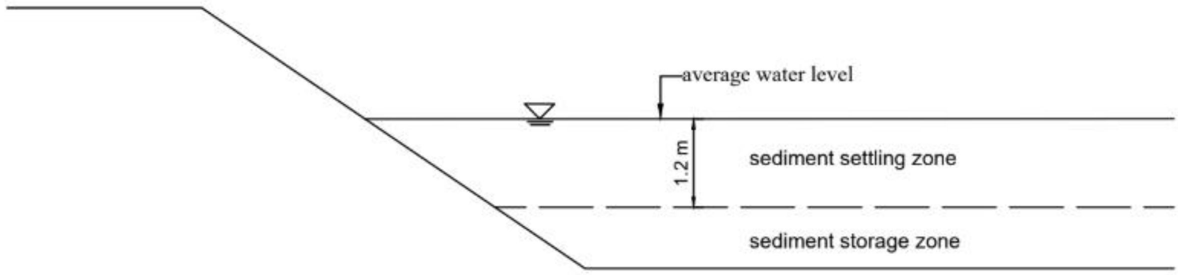

Figure 35 sediment pond cross-section

3. Hydraulic Structures

3.1 Boundary Condition

3.1.1 Hydraulic Boundary Condition

Probabilistic design method

In the probabilistic approach, the failure probability of a system can be analyzed by considering the distribution of load and resistance. Two types of probabilistic approach are using globally, the full probabilistic approach and semi-probabilistic approach. According to the Flood Defence Manual, the semi-probabilistic approach is commonly used for regular safety engineering assessments and design purposes of regular hydraulic defences. While the full probabilistic approach will be appreciated if a complex situation or nationwide risk assessment project is approached. Therefore, a semi-probabilistic approach is used as a probabilistic design method.

Design water levels

Water levels are stimulated mainly by tides in Maashaven. Since there is no data on the exact location of Maashaven harbour, the average data provided by Hydro Meteo Bundle NO.4, 2012 Port of Rotterdam is used. The average water levels in Rotterdam are shown in figure 31 with a unit in cm relative to NAP. The extreme high is 3.42 m +NAP for a return period of 1000 years and the lowest extreme is -1.2 m +NAP for a return period of 10 years. The NAP (or Normaal Amsterdams Peil) is a vertical datum in use in large parts of Western Europe. The average water level in the Netherlands is 0.25m +NAP and this value is used in this design.

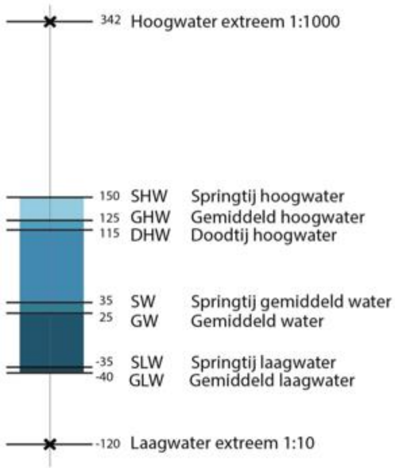

Figure 36 Tidal level chart

Water Depth

According to Figure32 by the Port of Rotterdam, the average depth of Maashaven harbor is assumed to be 10 meters. (Anne Zaat, 2019)

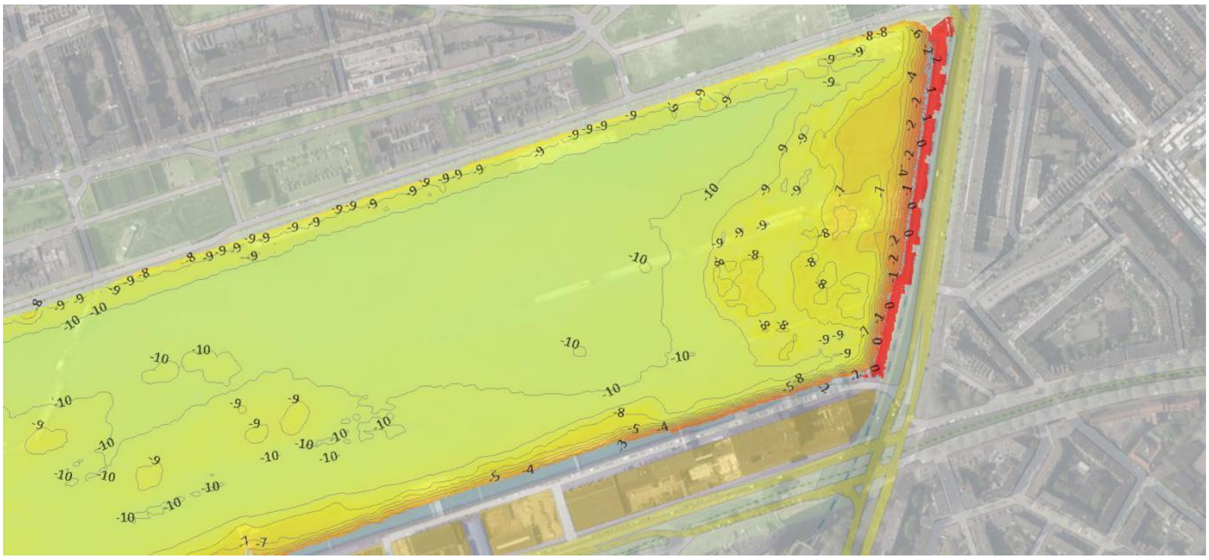

Figure 37 Harbour depth

Sea level rise

According to Hydraulic Structures Manual, the relative sea level rise is caused by several factors, mainly by the subsidence of the sea bottom and increase of the average temperature.

The observed trend of 19 cm relative sea level rise per 100 years is considered in the design. In the design, 0.19m is considered according to the 100 years design period.

Wind set-up

According to Hydraulic Structures Manual, wind set-up is the heading up of the water in closed off areas such as shallow seas, deltas, lakes and etc. Maashaven harbor has three closed-sides and an open entrance that connects with the river. Therefore, the wind set-up is considered and a value of 0.005m based on calculation is considered.

Seiches

Seiches are common sea phenomenon but also happen on lakes or other three-side surrounded water bodies. They define as standing waves that need an at least partially bounded water area to develop. Maashaven harbor is typically influenced under seiches. The period is calculated to be 403.85 seconds.

Wave condition

The wave run-up depends on the water level, the dike orientation, wave height and slope of the dike. The water level is determined above. The wind-induced wave height is influenced by water depth, effective fetch and wind velocity at an altitude of 10 m. No direct wind data is applied at Maashaven harbor. Wind data at Lekhaven is used, located 4000 meters away on the west side of Maashaven. (Anne Zaat, 2019)

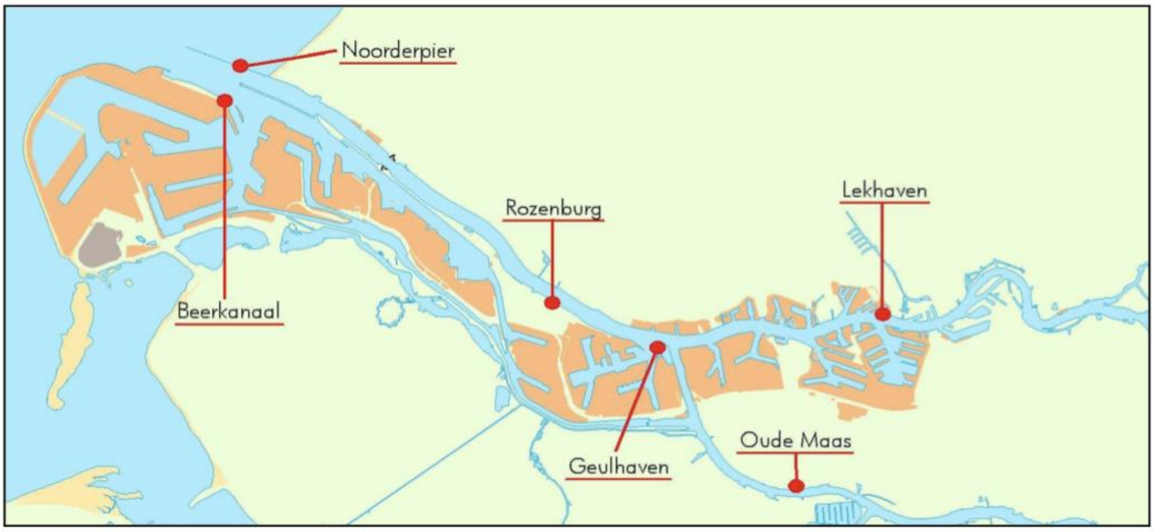

Figure 38 Location of Lekhaven used for wind-induced wave calculation

Wind analysis is given by Hydro Meteo Bundle no.4, 2012 Port of Rotterdam in figure 34:

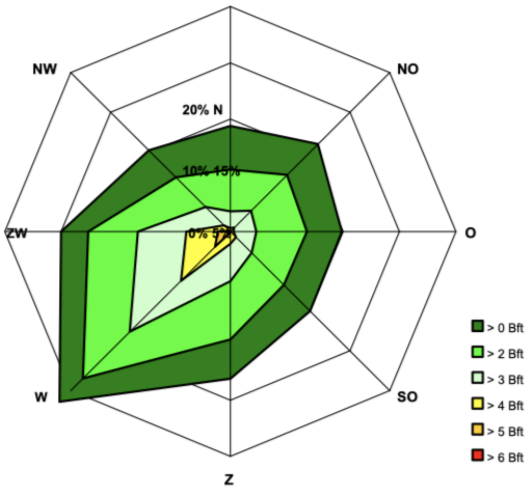

Figure 39 Wind data at Lekhaven

The strongest wind is the southwest wind larger than 6 Bft. According to online research, the maximum sustained wind speed has reached 93 km/hr, equivalent to 25.83 m/s in January. Therefore, in the report, the extreme value of 25.70 m/s for wind velocity 10 m above the surface is used in the direction of the southwest.

Based on the geometry of Maashaven harbor, the fetch is 1529 meter in the direction of the southwest, which is used in the wave calculation. (Anne Zaat, 2019)

The simulated significant wave height (Hm0) is 0.80 m and spectral wave period (Tm-1) is 2.54 s according to Young and Verhagen(1996).

3.1.2 Geotechnical boundary conditions

Soil structure

The subsoil data according to Borehole is shown in the figure below. Since there is no data from Maashaven harbour directly, two data points shown in the figure are analyzed in the dike design. To the depth from 10 meters to 15 meters, data from point 1 is used because it is underwater and has high similarity compared to other landfill data. Data from point 2 is used because of the lack of data from point 1 beneath 20 meters. Therefore, the aquifer thickness is used from point 2 data.

Figure 40 locations from Borehole

However, this analysis is not accurate and contains a high level of risk when construct. According to point 2, there are peat layers as subsoil, which will cause significantly consolidation when heavy structure constructs on top. In the dike design, the subsoil data is mainly based on point 1 which may cause failure. (Dinolocket, 2019)

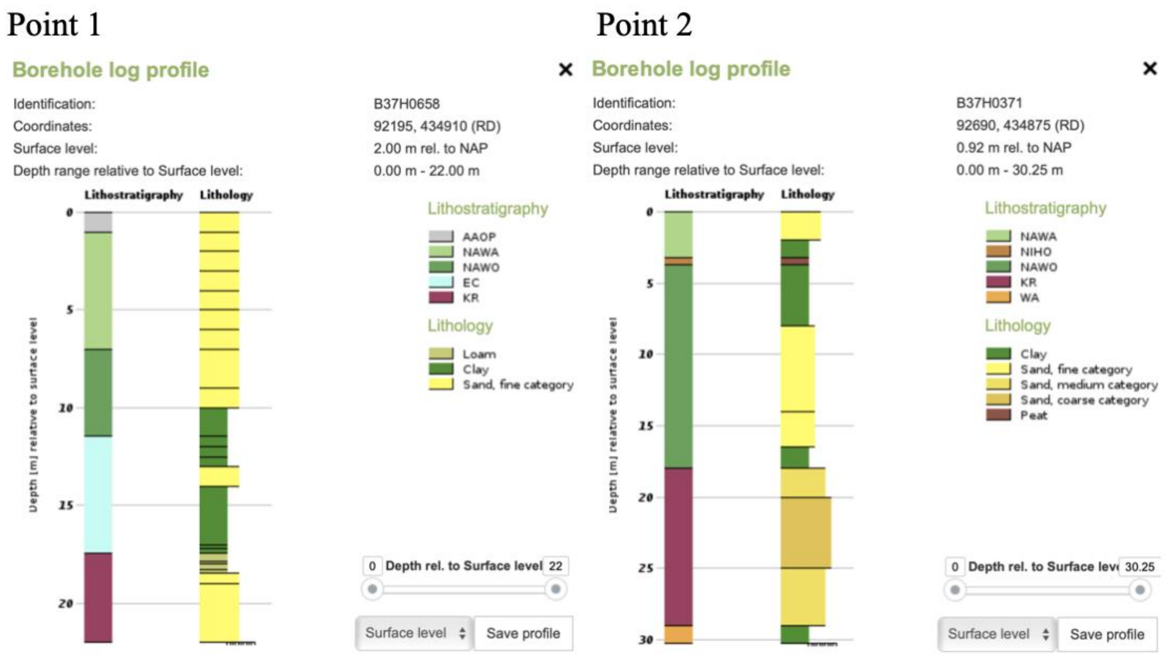

Figure 41 Borehole data for two locations

Table 10 Subsoil profile used for calculation

From [m +MWL]	To [m +MWL]	Soil type
-10	-13	Medium Clay
-13	-14	Fine Sand
-14	-18	Medium Clay
-18	-29	Medium Sand and Coarse Sand

3.2 Structure

The main hydraulic structure in our design is a dike. The dike will be constructed near the harbor entrance. On top of the dike, there is a weir to control the discharge into the wetland zone.

3.2.1 Dike

Design of adequate containment structures are important to act as sheltering protections from wave forces and sediment loss. Dike is one of the key structures that are able to control water level for wetland and dredging material running off to the river. The design for dike has a life period of 100 years.

Two types of dikes are designed in this report, a stone dike and an sand dike. The main differences are core material. The overall cross-section designs are shown below.

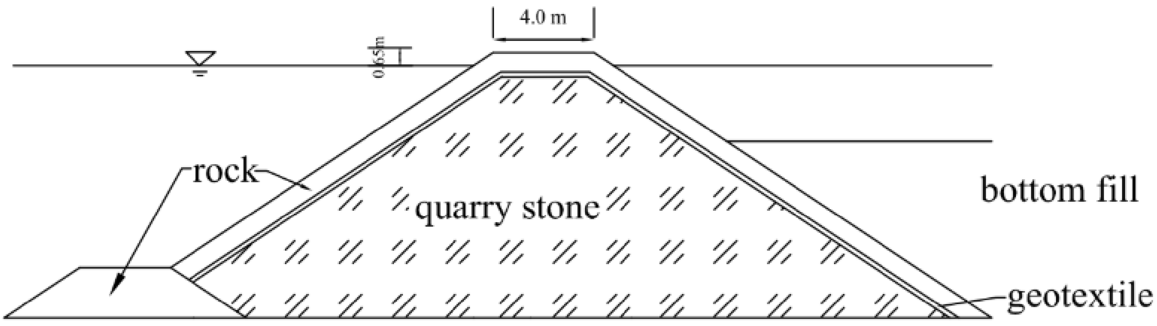

Figure 42 Stone dike cross-section

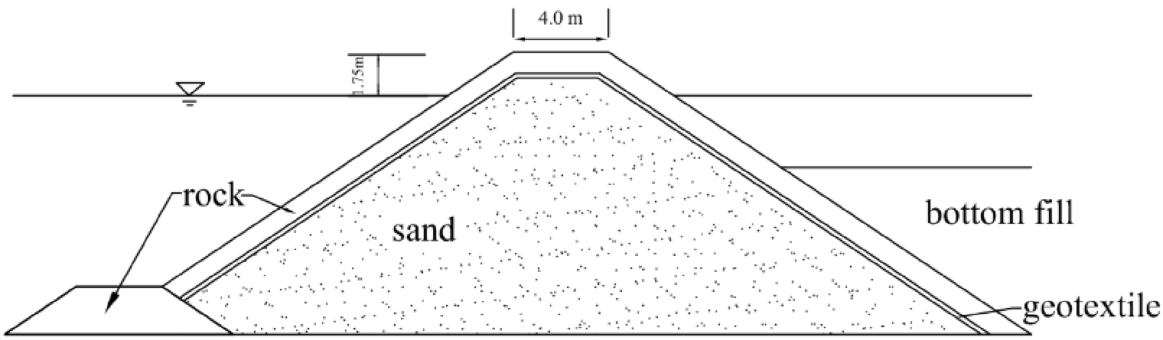

Figure 43 earthen dike cross-section

<u>Slope</u>

In the design, the wave condition is relatively small because of Masshaven harbor’s locations, compared to shoreline situation or coastal dike. It is mostly related to slope stability. The slope is designed to be steeper in this case and a toe is designed to protect the dike as well from erosion. A slope of 1V:1.5H is designed for the dike. Also, it is a cost-effective assumption for armor layer dike design (usually between 1V:1.5H to 1V:3H).

<u>Core</u>

There are two options for core design material, quarry stone or sandy material. The selection considerations include several characteristics, the protected dredged material (dredged clay in this design), the foundation compressible characteristics, the protection from hydrodynamic forces (waves, tides as analyzed before), availability of construction material, economics and so on. Of these, quarry stone dike and earthen dike are most common mainly due to availability and economics.

For stone dike, quarry run (10-60 kg) is selected as designed core material. Since stone dike is permeable, no other inflow system is needed in the design.

For earthen dike, sandy dredged material is selected. In this case, an under layer is added to the design between armor layer and core material. If more specific data about Rotterdam dredging material is applied, it is highly recommended to use dredging material not only cost-effective but also sufficient. Since earthen dike is impervious core made up of uniform cohesion-less material, a weir system is needed on the dike to control the inflow and outflow between open water and wetland zone.

<u>Crest</u>

Crest width: According to Hydraulic Structure Manual, the dike may have a crest width at least 4 m, depending on the available material. Width of 4 m is cost-efficient for this design. It also provides enough space for vehicles to be able to ride if either maintenance or construction for wetland is needed in the future (except the weir design part).

If flood situation is considered in this case, the crest height depends on several parameters such as design water level, wave run-up and effects of seiches. The combination of the above factors in a probabilistic approach defines the crest level and freeboard. The inputs are shown in Table 14.

Table 11 Crest calculation input

Input	Symbol	Data
Significant wave height [m]	H_{m0}	0.80
Wave direction [deg]	β	35
Spectral wave period [s]	T_{m-1}	2.54
Wave length [m]	L	10.10
Slope	tanx	0.667

In the Netherlands, critical overtopping discharges requirement is also essential to dike design. Wave overtopping on dike will cause erosion and soften of the foundation on the wetland side. Therefore, a reasonable critical run-up is needed. An assumption of overtopping discharge is 1 l/m/s and this situation gives no erosion damage to rubble mound structure.

By Van der Meer formula, the calculated crest level is 1.0664 meter. Therefore, the total crest level can be calculated shown in table 16.

Table 12 Reduction factor for Van der Meer formula

Input	Symbol	Data
Iribarren Parameter		2.526
Reduction due to friction	γ_f	0.6
Reduction due to oblique waves	γ_β	0.923
Reduction due to a berm	γ_b	1
Reduction due to a wall	γ_v	1
Overtopping height [m]	R_c	1.4452 m
Overtopping height limitation [m]	$R_{c,max}$	1.0664 m

According to Manual Hydraulic Structures, the sea level is expected to rise 0.40 m due to climate change.

Table 13 Freeboard

Input	Symbol	Data
Water level [m]	SWL	0.25
Sea level rise [m]	SLR	0.40
Local wind set-up [m]	nw	0.005
Designed freeboard [m]	d_{fb}	0.655

Since the main purpose of dike is to hold sediments running off from the reclaim land to the river, the dike height is related to site capacity, freeboard depth and dike settlement allowance. The designed crest height is calculated to be:

Table 14 earthen dike crest height

Input	Symbol	Data
Site Capacity [m3]	C	
Site Area [m2]	A	
	C/A	10
Settlement Allowance [m]	d_s	2.6
Freeboard Depth [m]	d_{fb}	0.655
Overtopping	R_c	1.06

Freeboard Total [m]	d_{fb}	1.75
Minimum Dike Height [m]	d_h	14.315
Designed Dike Height [m]	d_h	14.5

In this case, after construction, the earthen dike height will be 14.5 meter with 2.6 meters ground settlement. In the end, the free-board will be 1.75 meter. It is larger than the tidal range therefore inflow and outflow of wetland water will only be controlled by weir system.

In the case of stone dike, it is not necessary to consider overtopping failure since stone dike is permeable. The dike height is calculated to be

Table 15 stone dike crest height

Input	Symbol	Data
Site Capacity [m3]	C	
Site Area [m2]	A	
	C/A	10
Settlement Allowance [m]	d_s	2.6
Freeboard Depth [m]	d_{fb}	0.655
Minimum Dike Height [m]	d_h	13.255
Designed Dike Height [m]	d_h	13.5

In this case, after construction, the stone dike height will be 13.5 meter with 2.6 meters ground settlement. In the end, the free-board will be 0.9 meter. It is smaller than the tidal range of 1.65 m therefore water will flow in during high tide and flow out during low tide.

<u>Toe</u>

Toe design is one of the most important parts of the dike design because it is essential to the stability of the whole structure. In most cases, the rock size for toe has a smaller dimension than the armor layer to be cost-effective. Normally the toe width is 3 to 4.5 times the wave height. In the design, a 3.6-meter toe width is assumed.

Nod is the damage level used in the design. It is assumed to be 0.5 in order to apply a safe figure for design.

According to Van der Meer et al (1995), the parameters for toe design are shown in table 17.

Table 16 Toe design

Input	Symbol	Data
Damage number	Nod	0.5
Toe height [m]	h_t	4
Stability of toe protection	h_t/h	0.8
	$H_s/\Delta D_{n50}$	2.2457
Rock size [m]	D_{n50}	0.2199
Toe width [m]	W_t	3.6

<u>Rock layer</u>

Rock layer is decided to apply to the dike design to increase the stability of the front slope, reduce wave forces acting on the dike and the overtopping flow charge. Compared with concrete armor units and berm type protection, the rock layer is cost-effective and easy for construction. The armor layer stability on the dike is dependent upon the hydraulic conditions and structural parameters. At Maashaven, it is a deep water condition, the basic approach to evaluate the stability of rock-armored slope is Van der Meer formulae for deep water conditions.

Inputs are shown in table 18. By Van der Meer formulae, for plunging waves condition, Dn50 is designed to be 0.4m and the weight of unit armor stone is 131.2 kg.

The thickness of the armor layer is designed to be $2D_{n50}$ thickness, in this case 0.8 m, which defined by Rock Manual.

Table 17 Rock layer

Input	Symbol	Data
Slope angle	$\tan\alpha$	0.667
Number of waves	N	7500
Surf similarity parameter	ξ_m	2.3668
Critical value of surf similarity parameter	ξ_{cr}	4.4224
Relative buoyant density of armour stone	Δ	1.62
Relative water depth at toe	h/H_s-toe	12.5
Notional permeability parameter	P	0.4
Diameter of unit armour stone	D_{n50}	0.4197
Damage-storm duration ratio	$S_d/\sqrt{N}$	0.0231
Stability number	$H_s/(\Delta D_{n50})$	1.1766

Geotextile

Filter layer can protect erosion in the situation with large gradients on the interface of sediment and water. In this design, the geotextile is considered as a filter to protect the environment of the river from sediment running off from the wetland. The two main advantages of geotextile are cost-effective and limited thickness.

The two main parameters that classify geotextile are stability and permeability. The permeability of geotextile should be able to hold wetland sediment particles on one side. Since there is a lack of data for the dredging sediment in wetland, it is risky to make an assumption with no reference data. Also， this topic is beyond the knowledge. The method of filter rules is referred to Rock Manual [2007] or to SCHIERECK [2001].

<u>Overall</u>

The total design for dike is concludes in the table below:

Table 18 Overall stone dike design

Section	Material Type	Volume/meter [m3/m]	Length [m]	Total Weight [tones]
Core	Quarry Run (10-60 kg)	166.4	325	131390
Toe	Rock (300-1000 kg)	13	325	10266
Filter	Geotextile	7	325	5528
Armour	Rock (300-1000 kg)	28.2	325	22263
Total				169448

Table 19 Overall stone dike design

Section	Material Type	Volume/meter [m3/m]	Length [m]	Total Weight [tones]
Core	Dredged material	206.5	325	117446
Toe	Rock (300-1000 kg)	13	325	10296
Filter	Geotextile	7.6	325	6019

Armour	Rock (300-1000 kg)	36.8	325	29145
Total				162907

3.2.2 Weir

A weir is designed to allow water to flow to the sediment pond and the wetland from the river. This weir will be a contracted broad-crested rectangular weir. Since sediment is not expected to flow out through the weir, it is not necessary to design a v-notch. The construction level for the bottom of the weir is designed to be at +0.3m NAP. A weir at this level would block more than half of the tidal cycle. However, as time goes by the dike would gradually sink due to consolidation of the foundation, the weir level would also decrease. According to our calculation of consolidation, the dike would be lowered by 1m in one year and 1.3m in two years. After two years the weir bottom level would be at -1.0m NAP, which means the weir would stay underwater for most of the time. The tidal cycle is preserved.

Following equation is used to calculate the length of the weir:

$$q = \frac{2}{3} \times C_d \times L \times \sqrt{2g} \times h^{\frac{2}{3}}$$

where q = discharge (m3/s)

C_d = discharge coefficient

g = gravitational acceleration = 9.81 m2/s

h = head above the weir (m)

In this case, the weir length is set to be 20m, thus the maximum possible discharge is 83 m3/s. This value is assumed based on the tidal range and the design layout; C_d should be carefully determined after analysing the hydraulic characteristic in the harbour, but a typical value 1.71 could be used as a starting point if data is insufficient; the design value of h is set to be 0.75m, further adjustment may be needed. To account for extreme high tide and possible storm water, in construction at least extra 2m should be added.

The weir part of the dike is similar to a submerged breakwater. Thus, submerged breakwater equations are used to calculate the stability of the armor rocks.

According to Van der Meer et al (1990), the required armor stone diameter for a submerged breakwater could be calculated using the following equation:

$$\frac{h_c'}{h} = (2.1 + 0.1S)\exp(-0.14N_S^*)$$

where h_c and h is the water crest level before and after wave attack

S is the damage level

S=2 indicates start of damage, S=5 indicates moderate damage

$$N_S^* = \frac{H_s^{\frac{2}{3}} L_p^{\frac{1}{2}}}{\Delta D_{n50}}$$

L_p is the Airy wave length calculated using T_p and water depth at the toe of the structure

If we use S=1, the calculated Dn50 is 0.19, smaller than the required design size of the dike.

Right after the weir is constructed, during low water the bottom of the weir would be above water. According to Van der Meer et al (1990), the required armor stone diameter for a low crested breakwater could be calculated using the following equation:

$$D_{n50} = \frac{1}{1.25 - 4.8R_P^*}$$

$$R_P^* = \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}}$$

$$s_{op} = \frac{2\pi H_s}{gT_p^2}$$

where R_c is the freeboard

H_s is the significant wave height

T_p is the peak period

$D_{n50} = 0.94$

To make sure that the weir is stable, the weir part of the dike must be built by larger rocks than other parts of the dike.

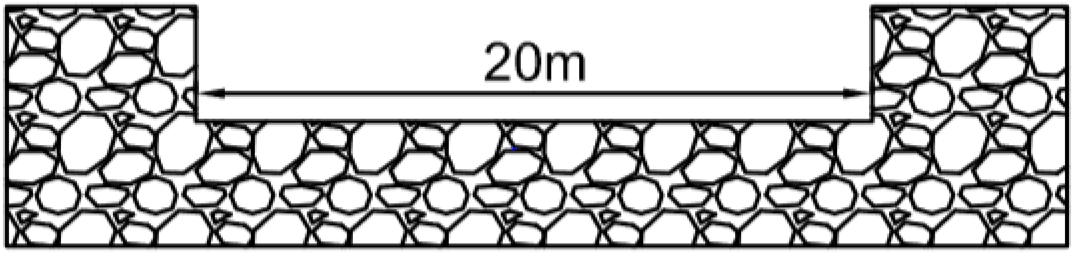

Figure 44 weir

4. Vegetation

With the design we have implemented, the wetland area would be approximately 192000m2. This area has been designed to be at the optimal tide height of +0.75mNAP which should allow for at least 6 hours of water coverage. This space should be landscaped properly by landscape architects with the assistance of ecologist to obtain the maximum aesthetic and ecological value.{Anne Zaat, 2019 #15}

An effective wetland should contain 20% mudflats 30% vegetation cover 50% open shallow water. This usually is the most productive from an ecological standpoint to maximize overall fish and wildlife.(Herbich, 2000)

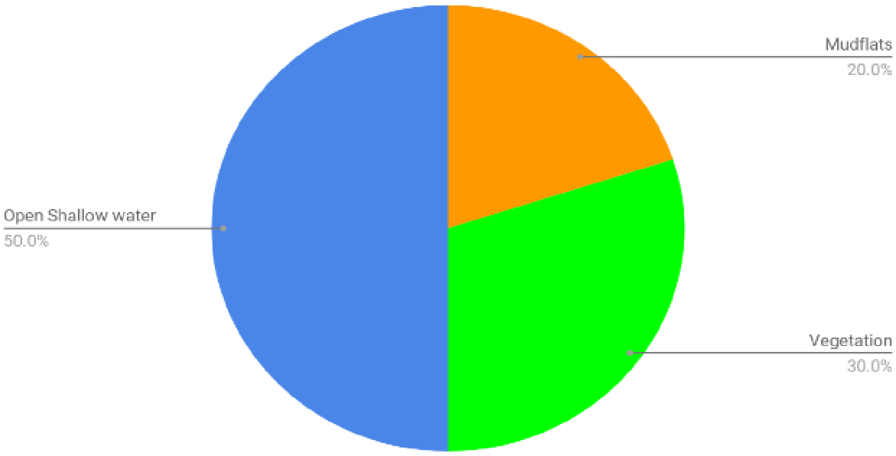

Figure 45 Wetland area design

Wetlands should typically be planted in the spring season after the daytime temperature reaches above 20°C (68 Fahrenheit). Variety of species is preferred. Vegetation species diversity leads to wildlife diversity so the more plants selected the better. (Herbich, 2000)

Table 20 Common wetland vegetation in Europe

Vegetation zone	Scientific Name
Aquatic vegetation	Potamogeton spp Nuphar lutea
Low marsh	Scirpus spp, Pharagmites australis Caltha palustris
High Marsh	Valeriana officienalis Lythrum salicaria Epilobuim spp

	Bidens spp Symphytum officinale Solanum dulcamare

While choosing vegetation, invasive species should always be avoided as they could be catastrophic to the whole wetland system. Before introducing non-native species, careful analysis and an emergency plan are necessary.

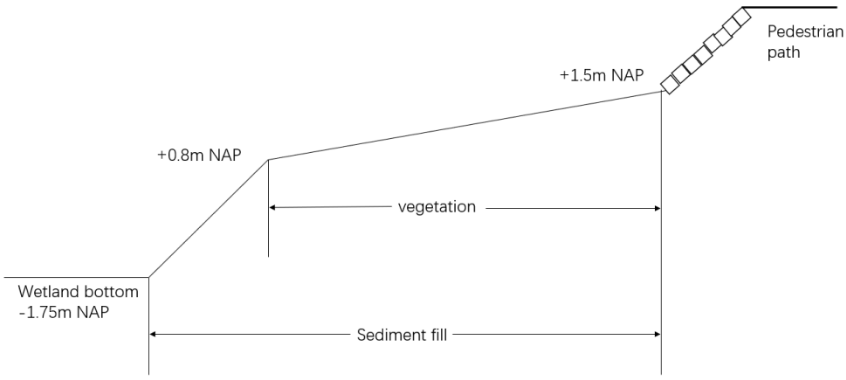

Figure 46 wetland cross-section layout

Reference wetlands are often the best sources of information on vegetation selection. A wetland close in Rotterdam with similar characteristics should be selected as reference wetland and imitated this could also be the sources of seeds and cutting in order to prepare for the planting season. Two examples of references wetlands are Nationaal Park De Biesbosch and Beningerslikken, another constructed wetland. The choice of vegetation is also highly depends on the local water and soil characteristics. As salinity, temperature, PH, water depth of water and the oxygen content of soil would greatly influence the growth of plants. For a newly constructed wetland, the first batch plants should be able to grow fast and spread quickly. In this case, several species recommended: Holcus lanatus, Pulicaria dysenterica, Iris pseudacorus, Mentha aquatica. All are fast growers and native in Netherlands.

For a constructed wetland, transplanting should be used at the first stage since it has greater successful rate than seeding. If transplants are used, parallel spacing in rows of between 0.3m and 1m are recommended to achieve uniform cover by the second season. A good example of this process in action is in Nassauhaven where planting has just began in April 2019. (Anne Zaat, 2019){DuPold, 2005 #19}

Figure 47 Wetland concept figures

5. Construction Plan

The Maashaven wetland construction consists of dredging sediment consolidation, dike system and wetland design. The order of construction is important. The dredging sediment needs to be kept in the harbour area and prevented from running off to the river. The dike is constructed before dumping dredging material. After construction of dike, dredging material is dumped to rise to the level of sediment pond. Wetland is constructed in the end. (Construction Industry & Information, 2013)

Table 21 Dike consolidation

Time (days)	Percentage of consolidation for earthen dike (%)			Percentage of consolidation for stone dike (%)		
	First stage	Second stage	Third stage	First stage	Second stage	Third stage
1	3.396	3.580	3.679	3.493	3.627	3.728
7	8.741	9.030	9.185	8.895	9.101	9.254
31	18.624	19.048	19.273	18.852	19.150	19.368
186	46.962	47.610	47.952	47.312	47.762	48.087
365	65.215	65.934	66.313	65.606	66.102	66.461

730	83.921	84.630	85.004	84.306	84.795	85.147
1825	95.615	96.132	96.405	95.896	96.252	96.508
3650	97.624	97.943	98.111	97.798	98.017	98.174
7300	99.125	99.242	99.304	99.188	99.269	99.328
10950	100.000	100.000	100.000	100.000	100.000	100.000

<u>Dike Construction</u>

Before list the construction steps, it is important to point out the main issue of dike construction is ground settlement. By analysis from previous, a 1.6 meter of ground settlement must be compensated for in the initial design. Since we assume it is uniform settlement for both dike and containment area, the net loss of dike height is considered during the construction as well. This problem can be compensated by overbuilding the dike or by stage construction.

Overbuilding dike is often appears to be easiest and cheapest solution. But it is not practical in many cases and it often causes shear failure or additional settlement.

Stage construction is somewhat more troublesome and expensive than overbuilding. It also time-consuming. But most of the case it is the only practical solution and has often been successful in the past. In this design, stage construction is selected.

Construction Steps:

1. Site Clearance:

   Design dike bottom area is cleaned and any hollow or surface irregularity is flattened. If any peat layer by site investigation, removing of peat layer by clay layer should be applied. If any sludge or clay material appears, it will be removed by sand. The optimize site base is to be sand layer.

2. Material transport

   Stone Dike: A quarry is located on the southeastern side of Masshavne where quarry stone can be transported by ship or truck. Shipping: The common European barge measures nearly 900 m2 goods and can carry up to about 2,450 tonnes. The average time for one round (including transportation 1 hour and up/unload 0.5 hours) is 1.5 hours. Shipping is recommended because larger carryings compared to land transportation and travel map shows below. To be specific, a split-hopper barge is considered because it is a vessel with a large open hold, used to load and transport dredged material. The capacity is 3.700 m3. By research and roughly analysis, split-hopper can travel between Maashaven and quarry field.

Earthen Dike: Quarry stone or rock transportation route is same as stone dike. For core material, shipping is also recommended from Rotterdam port dredging site to construction site.

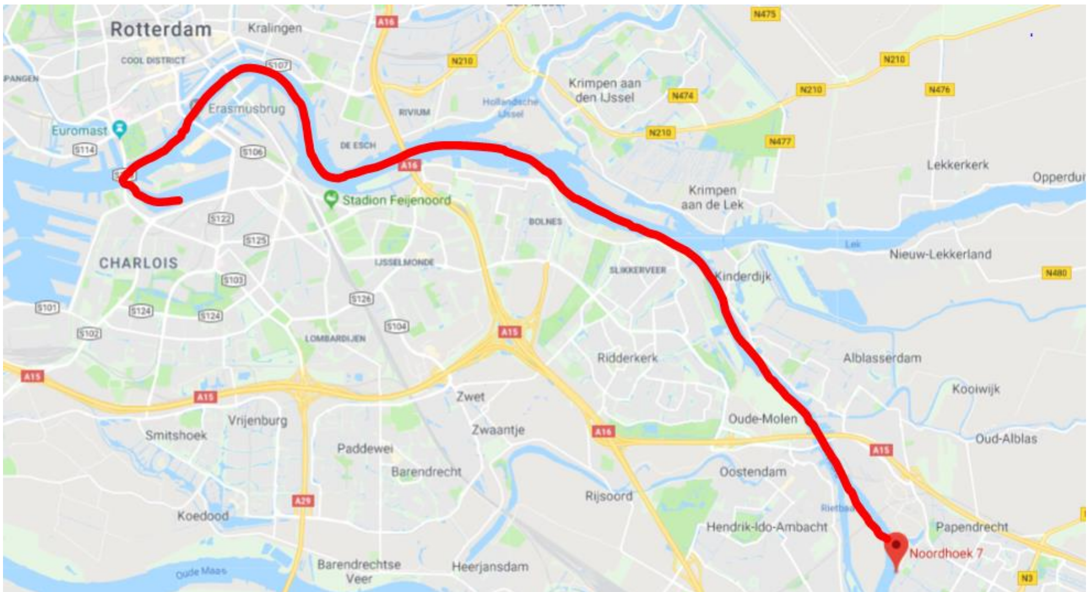

Figure 48 Stone transport route by shipping method

3. Core:

Core of the dike is required to be placed ahead of the various of slope stones and filter. The dike core is constructed in three stages because of construction scheduling and ground settlement. The first construction is dumping core material to 40% of dike height and 60% of full width. The second construction is dumping core material to 80% of dike height and 60% of full width. It takes two years for ground to settle for stage 1 and another two years for stage 2. Then the last construction is dumping the core material to full height and full width.

4. Slope construction:

For earthen dike, slope construction can be done by a hydraulic crane operated on the dike crest. The production capacity of hydraulic excavator mainly depends on the volume of lifting, working radius, rotation and lifting speed.

For rock dike, according to rock manual, the waterborne excavator (for example flat-top barges with excavator) which mass for the handling of 20 t is used for rock grading of 1-3 t in the design.

5. Toe:

   Toe is constructed by hydraulic excavator on the dike crest once the core design finished.

6. Geotextile:

   Since the geotextile construction is underwater, with a water depth of 10 meters. The normal way to install geotextile layer is not efficient. The construction method for sea dike is considered in this case. The construction needs to take place in low tide period. Cut-off grove is placed with geotextile pinned. The geotextile spreads from the toe up to the slope in the submerged condition. Careful operation is needed to prevent water and wave causing uplifting.

7. Rock layer:

   The amour layer is constructed at last by hydraulic excavator.

8. Weir:

   In the weir part of the dike the crest level would be lowered to +0.25m NAP, using larger stone as armour. The construction would also be carried out by a hydraulic excavator.

<u>Wetland construction</u>

1. Dredging material will be piped in from both northern and southern side.

2. Wetland construction:

   a. Excavation is performed to remove any weak or unsuitable materials from the dredging sediment top layer.

   b. Sediment pond and dredging material will be shaped as design.

   c. Top-soiling and vegetation involves the deposition and spreading of a layer of suitable soil which provide the vegetation growing.

Table 22 Construction plan

Project		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Site Clearance																		
Dike	Transportation																	
	Core																	
	Toe																	
	Geotextile																	
	Amour																	
	Weir																	
Consolidation																		
Wetland	Layout																	
	Vegetation																	

Note: The consolidation time would be considerably longer compared with other steps

Uncertainty Analysis

Table 23 Risk Assessment

Risks	Description	Probability	Effects	Mitigation
1	Wind chill and rain	High	Significantly influence the working conditions, adding to flooding of the sites and the damage to concrete design.	Construction needs to be operated in suitable weather. Early schedule planning is needed.
2	Weir design	High	Since the weir design is theoretical, there might be distinction and issues when constructing in the field.	Field investigation takes place before construction and further study is needed. Regular maintenance is needed to keep the weir functional.
3	Sediment runoff	Medium	Under unideal consolidation of dredging material, sediment pond storage limit can be reached in the short term.	Construct properly and dredge the sediment pond regularly if needed.
4	Failure of the Harbour wall	Medium	the construction of the harbour wall is largely unknown. Lateral loads introduced by the fill could cause a failure.	Obtain blueprints of the harbour from the city and calculate loads. Fill gradually to and monitor to prevent failure

5	Residents unhappy with design	Medium	Residents could object to permits and protest construction	Consult and Residents and create an open dialogue on design and constructions
6	Unexpected settlement in the dike lowers in below the water level	medium	Reference boreholes are located near but not near the exact area of the dike as the subsoil is not exactly known unexpected settlement could take place if the soil are found to be more compressible	Conduct boreholes near critical sites in the design. Redesign if necessary. Alternatively undertake repairs to increase the height
7	Not enough fill material can be obtained by one season to begin planting	low	If enough fill materials cannot be obtained to reach the wetland height. This means that the stabilising plants cannot be planted, and the sediments may erode.	If it is unlikely that enough sediment can be obtained a cellular fill method should be used so an area of the dike can reach the desired wetland height and other areas filled later.

6. Conclusion

Given our analysis of the harbor, river and dredging processes and within the port that there is a good design brief and a strong business case for developing an Intertidal wetland in Maashaven. Several different technical elements from different disciplines such as hydraulic, geotechnical, dredging engineering, ecology urban planning is necessary for successfully making it a true sustainable multidisciplinary project. Upon constructing such a project there will be many benefits to the Rotterdam community. The port of Rotterdam has a non-harmful place to dispose of dredged sediments, The city has a chance to generate urban renewal in some of the most disused parts of the city, residents will have more recreational spaces to enjoy, there will be more nature and habitats for birds and other animals. What's more the specific design for Maashaven is highly flexible and can easily be adapted to other harbours in Rotterdam where required.

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Behavior of Organic Dredged Soils

8. Appendix

Wind set-up

$$\eta_w = 1/2 * \rho_{air}/\rho_{water} * C_d * U_{10}{}^2/(gh) * F = 0.0053m$$

where

ρ_{air}=1.21kg/m^3 for air density

ρ_{water}=1025kg/m^3 for water density

C_d=3×10^{-3}，range (0.8~3.0) × 10^{-3}

U10=13.8m/s for largest condition in winter Bft = 6

h=10m for water depth

F=1529 value for closed water domain of length (m)

g=9.81;

Wave condition

Wind generated waves depend on wind direction, wind velocity at 10 meter above the ground, fetch length and water depth.

The wave condition is calculated based on the estimation of wave height and period because no measurements are available.

Young and Verhagen (1996) equations are applied as below:

$$\tilde{H} = \tilde{H}_\infty\{\tanh(0.343\tilde{d}^{1.14}) \times \tanh[\frac{4.41 \times 10^{-4}\tilde{F}^{0.79}}{\tanh\left(0.343\tilde{d}^{1.14}\right)}]\}^{0.572}$$

$$\tilde{T} = \tilde{T}_\infty\{\tanh(0.10\tilde{d}^{2.01}) \times \tanh[\frac{2.77 \times 10^{-7}\tilde{F}^{0.79}}{\tanh\left(0.10\tilde{d}^{2.01}\right)}]\}^{0.187}$$

$$\tilde{T} = \frac{gT_p}{U_{10}} \qquad \tilde{H} = \frac{gH_{m0}}{U_{10}{}^2} \qquad \tilde{F} = \frac{gF}{U_{10}{}^2} \qquad \tilde{d} = \frac{gd}{U_{10}{}^2}$$

where

F	[m]	=	fetch
d	[m]	=	water depth

U_{10} [m/s] = wind velocity at an altitude of 10 m

The calculation is based on excel sheet accordign to table []. All wave conditions are considered and the strongest wind happens in the direction 225 degree from north.

Table 24 Wind calculation input for different directions

Direction [deg]	Effective Fetch [m]	U_{10} [m/s]	$\tilde{H}$ [m]	$\tilde{T}$ [sec]
0	1007	5.4	0.0402	2.2167
45	1690	5.4	0.0507	2.5505
90	1428	5.4	0.0470	2.4365
135	1083	5.4	0.0415	2.2606
180	1014	7.9	0.0286	1.8065
225	1529	25.7	0.0118	1.0651
270	1600	10.7	0.0267	1.7343
315	1027	5.4	0.0405	2.2282

Wind-induced wave angle of incidence is considered due to the normal direction of dike shown in below:

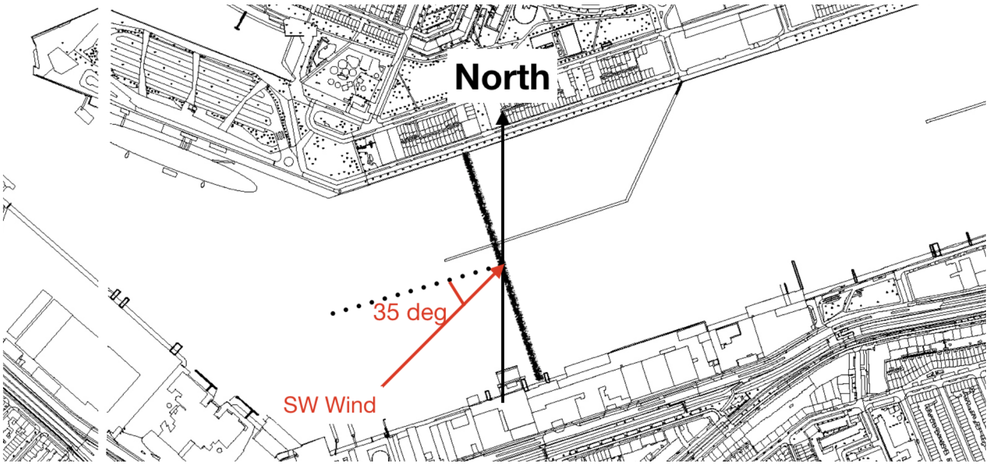

Figure 49 Angle of wind-induced wave to the dike

Table 25 Wind-induced wave reduction

Direction [deg]	Angle of incidence [deg]	H_{m0}	$T_{m-1,0}$
0	100	-	-
45	145	-	-
90	170	-	-
135	125	-	-
180	80	0.1817	1.3225
225	**35**	**0.7951**	**2.5367**
270	10	0.3113	1.7197
315	55	0.1204	1.1150

According to the result, the significant wave height is 0.80 meter and the significant wave period is 2.54 seconds in this design.

Wave run-up

Reduction factors for roughness, berms and oblique wave attack are used when calculating both wave run-up and overtopping.

Roughness is one of the parameters to decrease run-up and overtopping. According to Eurotop manual, factor of roughness for single layer of riprap is 0.7.

Oblique wave approach influences the run-up and overtopping because normally the wave attack is not perpendicular to the slope. The attack angle is 35 degree from previous analysis. The factor is calculated based on formula below:

For run-up: $\gamma_\beta = 1 - 0.0022|\beta| = 0.9230$

For overtopping: $\gamma_\beta = 1 - 0.0033|\beta| = 0.8845$

Berm reduction is neglect in the design because no berm is designed.

Reduction for walls is neglect in the design because no vertical wall is designed.

Nowadays, overtopping is used to determine the crest level instead of 2% wave run-up. In order to calculate required crest level by repdiction mean overtopping rates, wave condition needs to be identif. By calculation, the wind-induced wave condition in Maashaven harbor is deep water wave condition. Therefore, Van der Meer and Bruce 2014 formulas are used.

By Van der Meer and Bruce formula, the calculated crest level is 1.0664 meter.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.023}{\sqrt{tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot exp\{-\left(2.7 \cdot \frac{R_c}{\gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v \cdot \xi_{m-1,0} \cdot H_{m0}}\right)^{1.3}\}$$

with maximum of $\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.9\xi_{m-1,0} \cdot exp\{-\left(1.5 \cdot \frac{R_c}{\gamma_f \cdot \gamma_\beta \cdot H_{m0}}\right)^{1.3}\}$

For semi-probability calculation, the equation are:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot exp\{-\left(4.2 \cdot \frac{R_c}{\gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v \cdot \xi_{m-1,0} \cdot H_{m0}}\right)^{1.3}\}$$

with maximum of $\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot exp\{-\left(2.3 \cdot \frac{R_c}{\gamma_f \cdot \gamma_\beta \cdot H_{m0}}\right)^{1.3}\}$

Table 26 Overtopping results

Iribarren Parameter	2.526
Rc	1.4452 m
Rc,max	1.0664m

Therefore, the critical crest level due to overtopping iis 1.0664 meters.

According to the figure below, the freeboard depends on wave overtopping height, local wind setup, water level rise, mean water level and ground subsidence.

Construction level depends on freeboard and ground settlement or compaction.

$$Minimum\ construction\ level = 10 + 1.0664 + 0.005 + 0.25 = 11.3214m$$

The construction level is designed to be 11.5 m for safety consideration.

Toe

According to the Rock Manual, relative toe depth of ht/h is around 0.5 to 0.8. In the design, relative toe depth is 0.8 and therefore the stability of toe protection is 6.5 according to the figure below:

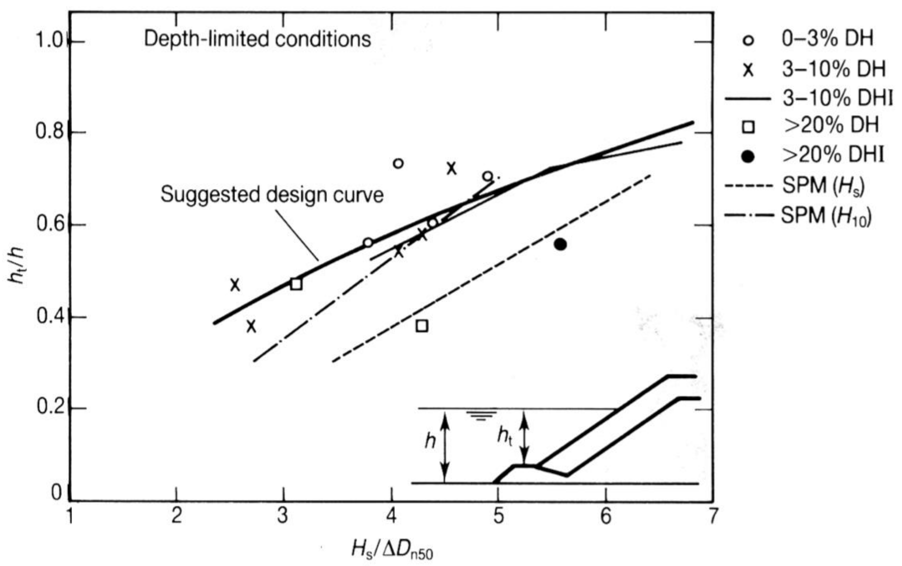

Figure 50 Toe stability as a function of relative toe depth h_t/h

In the toe design, a more generic approach by Van der Meer et al (1995) is used as well for more accurate value. The damage level N_{od} is designed to be 0.5 which means start of damage.

$$H_s = bN_{od}{}^{0.15}$$

$$\frac{H_s}{\Delta D_{n50}} = (2 + 6.2(\frac{h_t}{h})^{2.7})N_{od}{}^{0.15}$$

Armour stone

Armour stone grain size is calculated based on Van der Meer formulae for deep water wave condition shown as below:

$\frac{H_s}{\Delta D_{n50}} = 6.2P^{0.18}(S/\sqrt{N})^{0.2}\xi_m{}^{-0.5}$for plunging waves

$\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13}(S/\sqrt{N})^{0.2}\sqrt{cotx}\xi_m{}^{P}$ for surging waves

where $\xi_{mc} = [6.2P^{0.31}\sqrt{tanx}]^{1/(P+0.5)}$

Hopper dredging process

Hoppers are usually full 100% capacity due to the loading process. Hoppers begin loading soil-water mixture and load to overflow. The hoppers continue to fill in the overflow stage to maximise the sand concentration as some of the water and finer grains go out the overflow valve.

Minimum dredging time is equal to Hopper Volume / Discharge into hopper.

Discharge into hopper = number of suction pipes * Velocity in suction pipe * pipe cross section area.

Calculations dredging time

Velocity piper approx 5m/s

Ecodelta

Assuming 1 suction pipes and Velocity in suction pipe 5m/s

Ecodelta discharge into hopper = 1 $\times$ 5m/s $\times$ 0.25 π $\times$ d^2 m^2 = 3.93 m^3/2

Min dredging time = $5901m^3 \div 3.93m^3/s$ = 1501 sec = roughly 25 min

Approximately 5 min overflow time

Total time 30 min approx.

Hein

Assuming 1 suction pipes and Velocity in suction pipe 5m/s

Discharge into hopper = $1\times 5m/s \times 0.7^2m^2 = 1.924m^3/s$

Min dredging time = $3656\ m^3 \div 1.924m^3/s$= 1900s

Roughly 30 min

Approximate 5 min overflow time

Total time 35min

Average travel time

We will attempt to make a very rough approxiamation of the travel time using the distance of the dredging sections and the speed. Since Maasklate 2 is the maximum distance of the dredging sections from Maashaven is located approximately 50km away via the river channel. Assuming a roughly linear probability this gives and average 25km to travel from each dredging section.

Average travel time vessels

Ecodelta loaded = $11mph\times 1.61$= 17.7km/h

$25/17.7\times 60$=84.7 min unloaded

Unloaded = $13mph\times 1.61$= 20.9km/h

$25/20.8\times 60$= 72min

Hein

Loaded = $10mph\times 1.61$=16.1km/h

$25/16.1\times 60$ = 93 min

Unloaded = 11mph$\times$16.1= 17.7 km/

25/17.1/60 = 84.7 km/h

Boreholes from Dinoloket

Near the dike

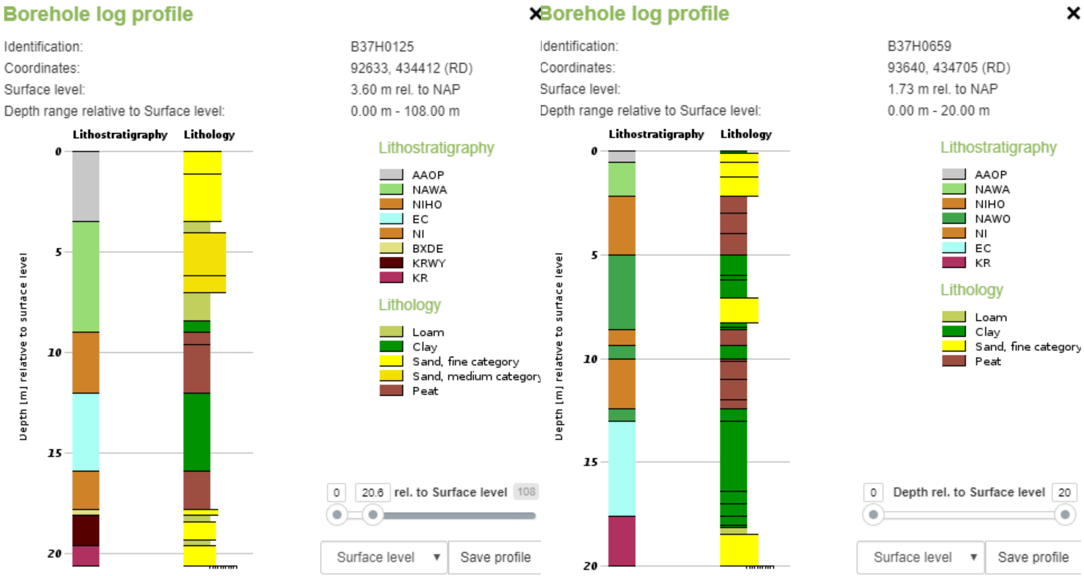

Figure 51 Near the dike soil data from Borehole

Near the harbor wall

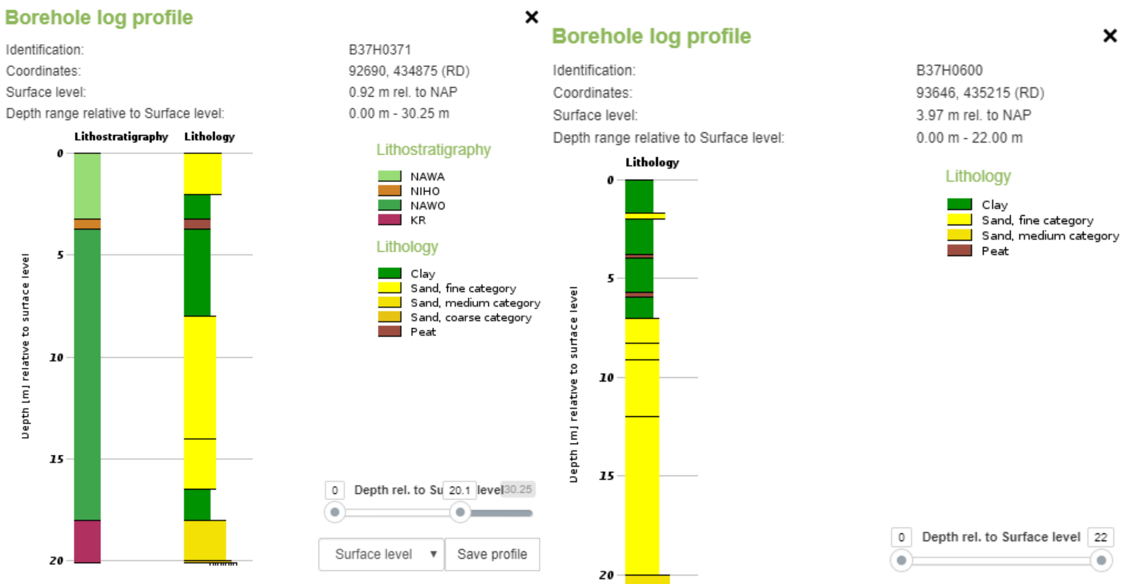

Figure 52 Near the harbor wall soil data from Borehole (Dinolocket, 2019)

Table 27 Typical values of coefficient of consolidation

Soil	c_v (cm^2/sec) x 10^{-4}
Mexico City Clay (MH) (Leonards & Girault, 1961)	0.9 - 1.5
Soft blue clay (CL - CH) (Wallace & Otto, 1964)	1.6 - 26
Organic Silt (OH) (Lowe, Zaccheo & Feldman, 1964)	5 - 170
Chicago Silty Clay (CL) (Terzaghi & Peck, 1967)	8 - 11
Sandy silty clay (ML - CL) dredge spoil (Van Tol et al, 1985)	5 - 20
Organic Silts and Clays (OH) (Sivakugan, 1990)	1 - 10