Reducing hydraulic loads on river dikes by construction of longitudinal mounds of local soil

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Cover:	High water level on the Waal at Beuningen in February 2020. Photo by Tom Hessels	

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Abstract

For the past 2000 years river training works have been performed on the Dutch rivers. These training works have shaped the river system of today. Another result of the river training works, especially over the last two centuries, is a decrease in biodiversity. To protect the biodiversity some floodplains in the Netherlands have been classified as Natura2000 areas. Construction works on these floodplains are not allowed, unless these measures are a last resort. Also, if this is the case, compensating measures have to be taken elsewhere.

Before 2050 about 1500 kilometres of dikes and 500 sluices and pumping stations need reinforcements. Dike reinforcements could be executed by only adding soil to the dike. Another option is to add structural elements to the dike. A soil-based approach is preferred because there is more experience and a higher level of security of the reliability for a soil-based structure.

One such soil-based approach is a longitudinal mound. A longitudinal mound is a body of soil which is parallel with the dike, with the goal to reduce the wave height at the dike itself. As a result of the wave height reduction the necessary dike crest level will be reduced as well. Therefore, a reinforcement of the dike itself is not needed. The crest of this longitudinal mound is lower than the crest height of the dike. The longitudinal mound will be submerged during design conditions and will act like a submerged wave breaker.

Costs, emissions and construction time could potentially be reduced by using local soil. This local soil can be obtained in two different ways. Firstly, it is possible to use the surplus of soil of another local project for the longitudinal mound. Secondly, the soil for the longitudinal mound could be taken from the floodplain itself.

However, only little is known about the hydrodynamic effects of a longitudinal mound on the floodplain. This thesis research is done to find possible locations for a longitudinal mound, the hydrodynamic effects and the differences between a simple and more complex model of the longitudinal mound. This is done with a multicriteria analysis for the location study and with a conceptual model and a 2D D-Flow FM model for the hydrodynamic effects.

In the multicriteria analysis the studied criteria are the size of the floodplain, structures on the floodplain and inside the dike, the availability of clay on the floodplain, the habitats on the floodplain and the wave height at the dike.

The multicriteria analysis has been performed from the point of view from multiple stakeholders. For all locations a compromise is necessary. Different locations for a longitudinal mound are preferred depending of the point of view of the stakeholders.

In the conceptual model three design parameters for the longitudinal mound are taken into account, the crest height, the crest width and the slope. For each combination of these three parameters the conceptual model calculates the new equilibrium water level and the transmitted wave height from the longitudinal mound towards the dike. The transmitted wave height is calculated with the best empirical fit on multiple datasets by Friebel and Harris in 2003.

With the Van der Meer overtopping formula the freeboard of the dike above the water level can be determined. This is done for the original situation without longitudinal mound and subsequently for

the situation with all combinations of the longitudinal mound. From these calculations it can be concluded that the necessary dike crest height decreases when a longitudinal mound is present. However, more soil is needed than for a traditional dike reinforcement.

Also the conceptual model does not include a backwater effect. The water level does not immediately jump to the new equilibrium water level, so the water level increase should be smaller than calculated in the conceptual model. On the other hand, in the conceptual model all waves are assumed to be perpendicular to the dike. If waves are not perpendicular the necessary freeboard is smaller. The absolute dike crest height reduction with a longitudinal mound is therefore smaller for non-perpendicular waves than for perpendicular waves.

The 2D D-Flow FM model has been supplied by Deltares. The grid consists of cells of 20 by 10 square metres on the main river channel and 20 by 20 square metres on the floodplain. To model the longitudinal mound with a higher accuracy the grid on the floodplain has been refined to 5 by 5 square metres. On this refined grid three different variants have been modelled. All variants have a crest height of about half a metre below design water level and their alignment is identical. For Variant 2 a connection of half the longitudinal mound height has been made with the dike. For Variant 3 the same volume of soil needed for the longitudinal mound has been removed from the floodplain by lowering it by 0.3 metres.

There are only small differences between the three variants. Compared to the original situation there was only a difference in the order of millimetres of water level at the main river channel. The main differences are found between the dike and the longitudinal mound. In this area the Bernoulli effect is found, at locations of increased flow velocity lower water levels are found and vice versa. The subsequent difference in water level is about 5 to 10 centimetres.

The flow velocity depends on the difference of flow area in longitudinal direction between the longitudinal mound and the dike, following the Bernoulli principle. So, the main contributor to the water level change on the floodplain is the alignment of the longitudinal mound. Therefore, the alignment of the longitudinal mound is an important design parameter and can be used to find a trade-off between increased water levels and increased flow velocity.

As this process is not incorporated in the current version of the conceptual model the results between the conceptual model and the 2D D-Flow FM model are different. Therefore, it is recommended that the water levels between the longitudinal mound and dike are calculated separately in the conceptual model. To do this the area between the dike and longitudinal mound can be split into multiple segments. With energy and momentum balances the water levels in these segments can be calculated.

It is also recommended that the 2D D-Flow FM model is used at a smaller floodplain as well to see if the effect on the main river channel is similarly small. Next, it could be helpful to try different alignments for the longitudinal mound to see how these influence the water levels and flow velocities.

Finally, in this research only the flow has been modelled in 2D. However, the wave reduction is also of importance. The next step is to add a wave model to the 2D model to as well. With this addition it would be possible to make the comparison between the wave height reduction in the conceptual model relative to a 2D model as well as for the water level.

Contents

1. Introduction

1.1. Context

For the past 2000 years people have been intervening in the Dutch rivers. These river training works have been executed for military purposes, safety against flooding, protection against erosion, land reclamation, freshwater supply, navigation and nature restoration. These interventions have had an influence on how the Dutch rivers are shaped today (Mosselman, 2022).

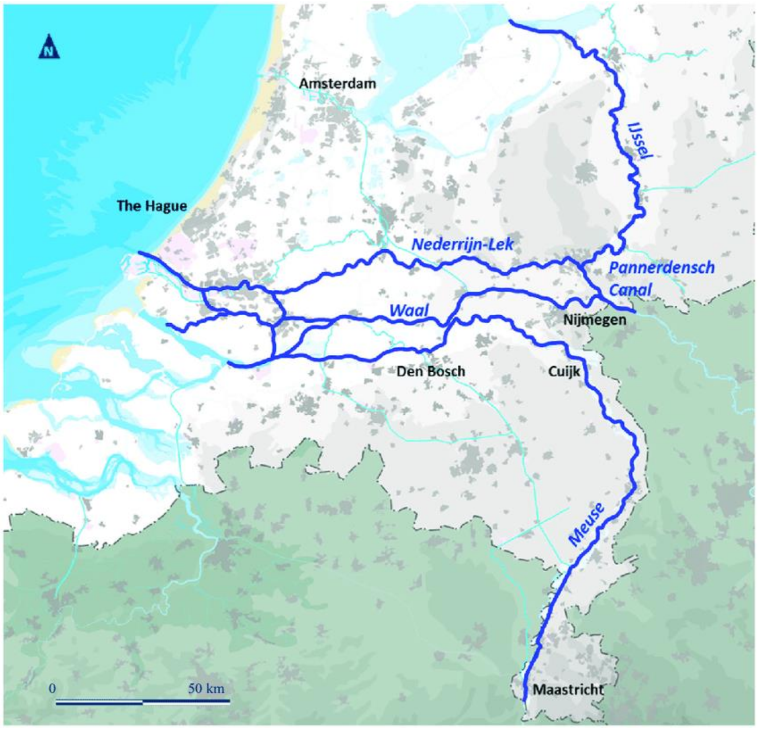

Figure 1 Current course of the rivers in the Netherlands (Klijn et al., 2018)

This process has set the current course of the rivers in the Netherlands, this is shown in Figure 1. Along most of these main rivers dikes are located. However, a significant portion of the dikes do not conform to the regulated safety levels. On many locations projects to increase the resilience of these dikes are planned. These projects are collected in the High Water Protection Program (HWBP, Dutch: Hoogwaterbeschermingsprogramma). Before 2050 about 1500 kilometres of dikes and 500 sluices and pumping stations need reinforcements (Rijksoverheid, 2023). Therefore, many projects are planned to increase the safety levels of the river dikes in the Netherlands.

In general there are two different approaches to these dike reinforcement projects. Firstly, a soil-based approach. With this approach the current dike will be heightened, widened or both. Generally, this extra volume of soil will be attached to the current dike. However, an extra body of soil is an option as well. Secondly, a construction based approach. With this approach a construction will be attached to the current dike. A construction based approach could be useful when there is little space available, for instance in a urban area, or for the protection of a special landmark. However, soil-based solutions are preferred. This preference is because there is more experience and there is a higher level of security of the reliability of a soil-based structure (Klijn & Bos, 2010).

Deltadike

The term deltadike is used nowadays to refer to dikes that won't breach when water and waves flow over its crest. Also, a deltadike should still be safe in 2100. These requirements result in dikes that are a hundred times less likely to fail relative to the current requirements (Klijn & Bos, 2010). An inventory of examples of deltadike designs has been conducted by Klijn & Bos. The following types have been distinguished:

1. Dike inward expansion
2. Dike outward expansion
3. A combination of dike inward and outward expansion
4. A wide flood defence zone
5. A camouflaged dike
6. A constructive solution

Solutions, 1, 2, 3 and 6 can be combined with a crest height increase of the original dike. A crest height increase also causes an increase of the footprint of the dike assuming the dike slope is not allowed to become steeper. A cross-section of each of the types is shown in Figure 2.

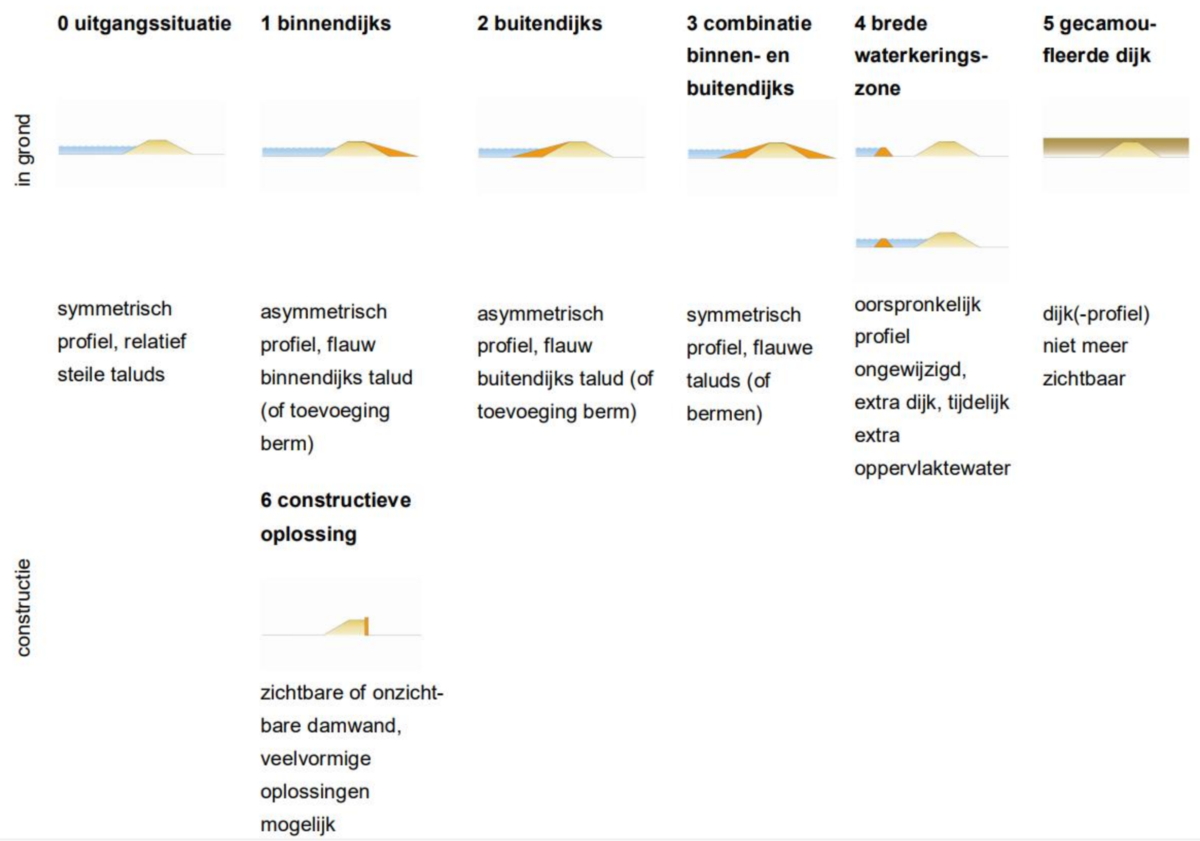

Figure 2 Cross sections of the different types of deltadikes. (In Dutch, Klijn & Bos, 2010)

If the crest height in the original situation is not sufficient this means an increase in both crest height and footprint, resulting in a wider dike. There are two options for this expansion, on the inner side or on the outer side of the dike. Or a combination of the two. For both an inward and an outward expansion there are negative effects.

When using the inner side of the dike the extra area that is needed for the dike is located on the dry side of the dike. If structures are present on the inner side of the dike this leads to a lack of space for the construction of this dike expansion. Therefore, it is especially difficult to expand to the inner side of the dike in urban areas. Although, along the Dutch river dikes there are often farms, or other structures, located at the inner side of the dike. So, also in rural areas these difficulties may arise.

When using the outer side of the dike the extra area that is needed for the dike is located on the floodplains. Therefore, it reduces the flow capacity of the river and during design conditions this will result in higher water levels relative to the situation before the dike reinforcement. So, outer dike expansion is not preferred for river dikes. Also, there may be legal limitations with respect to ecology and habitats.

The wide flood defence zone has some different applications. It can consists of two or more parallel dikes, one primary river dike with a lower dike in front or an artificial or natural (beach, salt marsh) wavebreaker in front of the dike. To be able to construct this type of protection space has to be available. Generally, waves on rivers tend to be smaller than on lakes and on the sea. Therefore, the effect on river dikes is expected to be smaller (Klijn & Bos, 2010). The effects and the subsequent feasibility of one application of a wide flood defence zone is treated in this thesis.

Longitudinal mound

In this thesis the option for a longitudinal mound is further examined. The longitudinal mound is a wide flood defence zone type of solution. The longitudinal mound will be constructed on the floodplain parallel to the river dike, with the goal to decrease wave heights and subsequent wave run-up and overtopping at the river dike.

The longitudinal mound will be constructed on the floodplain. Therefore, the flow area during design conditions will be decreased. Because of the restricted flow capacity as a result of the longitudinal mound the water level during design conditions will increase.

So, there are two main effects of the longitudinal mound that determine the design of the longitudinal mound. The first is an increase of water level as a result of restricting the flow during design conditions. This leads to an increase of the dike crest height. On the other hand a decrease of wave attack on the dike results in a decrease of wave overtopping over the dike. This will subsequently result in a decrease of necessary dike crest height.

At some locations dams are build perpendicular to river dikes. This is done as part of the Water Framework Directive (WFD, Dutch: Kaderrichtlijn Water, KRW). The WFD is a directive from the European Union which mandates that all ground and surface water in the EU is clean and should provide a healthy habitat for the flora and fauna that are present in these systems. Next to the water safety aspect a longitudinal mound could provide benefits towards the goals of the WFD.

Local soil

As this is a soil-based approach this means that soil is necessary for the construction of the longitudinal mound. Generally sand and clay are transported over relatively large distances before it is used in projects like this. A possibility to reduce costs, reduce emissions and decrease operating time arises when local soil is used (RHDHV, 2020). This local soil can be obtained with two different methods.

The first option is that at another project soil is removed which subsequently will be available to use in this project. However, this means a match is necessary with another project where this soil is removed. Therefore, there is only a small window for the timing. If there is no project with a surplus of soil available at the same time a depot could be used to temporarily store this soil. However this also causes extra transport. All in all, this costs money, time and causes extra emissions.

The second option is that soil is taken from the floodplain itself. This means this window is not present and reduces those risks. An extra advantage is that using soil from the floodplain itself means that the water level increase should be countered, at least partly, as there is no net volume difference between the before and after situation. However, because of friction and by changing the topography of the floodplain a difference in water level is still expected. For this research local soil will be described as soil from the same floodplain as the construction.

Natura2000

The floodplains along the Dutch rivers are a dynamic biodiverse area. However, due to the river training works this biodiversity has decreased significantly over the last two centuries (Uehlinger et al., 2009). As this biodiversity is key in keeping the ecosystem in place, some floodplains are designated as Natura2000 areas.

Natura2000 is a coordinated network of protected areas with Europe's most valuable and threatened species and habitats (European Commission, 2023). As these species and habitats are important a Natura2000 area has a legal status which mandates the habitat surface area and quality for all valuable and threatened species in the Natura2000 area.

Construction work on the floodplains and river dikes influences these habitats, both in surface area and in quality. Therefore, it is not allowed to do construction work in a Natura2000 area unless there are no alternatives, there are compelling reasons in benefit of the public interest or if compensating measures are taken to guarantee the overall cohesion of Natura2000 (BIJ12, 2019).

These compensating measures should be seen as a last resort for if other measures are not feasible (BIJ12, 2019). On the other side, with the construction of a longitudinal mound there is also the possibility to create a new ecological area. Increasing the biodiversity the longitudinal mound could be constructed to retain water for a longer period of time on the floodplain. This water retention could result in a new habitat for bird, fish and plants. This is mainly a possibility for floodplains that are not classified as a Natura2000 area. On the other side, if there is still water on the floodplain because of higher water levels which is subsequently followed by design conditions, the water level may exceed design conditions.

1.2. Problem statement

Currently not much is known about the feasibility of a longitudinal mound on a river floodplain instead of a traditional dike reinforcement as a measure to increase water safety.

When using a longitudinal mound there are two main effects that will have an influence on the crest height of a dike. Firstly, because the longitudinal mound is located on the floodplains the flow area during design conditions is decreased. This decrease will subsequently lead to an increase of water levels during design conditions. This might be countered by taking the soil for the longitudinal mound from the floodplains which results in no net changes to the flow area, however lowering the floodplains might give other unwanted effects.

Secondly, the longitudinal mound decreases the wave height at the dike. This is because the longitudinal mound acts like a (submerged) wave breaker during design conditions.

Both these effects are dependent on the design of the longitudinal mound. It is to be expected that by increasing the size of the longitudinal mound the effect on the water level increases as well. On the contrary this increase of size makes for a larger reduction of wave attack on the dike. Therefore an optimum design can be determined.

These effect can be studied with complex 2D models. The disadvantage of these complex models is that they take a lot of time to set-up and need a lot of computing power resulting in longer simulation times. Therefore, if a simpler model can accurately predict the effects of a longitudinal mound this could reduce the need of these complex 2D models in earlier stages of the design.

However, before a longitudinal mound can be constructed it is necessary to assess possible locations. There will be locations where waves play an insignificant roll, where only little space is available or where the floodplain is part of a Natura2000 area. This will result in less favourable conditions.

Ecology is a big part in decision making nowadays. Therefore, it is important to look at the ecological impact of the longitudinal mound. It is difficult to execute projects in current Natura2000 areas as they are protected by law because of the value these Natura2000 areas have. However, with a longitudinal mound there is also a possibility to create a wetland on the floodplain which could increase biodiversity. Over the past centuries this biodiversity has been lost along the banks of the Dutch rivers, mainly due to river training works (Uehlinger et al., 2009).

1.3. Objectives

The thesis objective is to answer the following main research question:

Where along the main rivers in the Netherlands could a longitudinal mound be constructed while working with a closed ground balance and to what extent will the longitudinal mound influence the hydrodynamical conditions along river dikes to possibly limit necessary dike reinforcements?

The thesis objective is twofold. Firstly, it is to find locations along the Dutch rivers that satisfy the conditions for the construction of a longitudinal mound. Secondly, it is to determine how different designs of a longitudinal mound will influence the hydrodynamical conditions. This could subsequently lead to a decrease in necessary dike reinforcements. Furthermore, it is of interest to see if these hydrodynamical changes can be reliably approximated with a conceptual model or more complex calculations are necessary.

Questions

To reach these goals several research questions have been set.

1. "Which locations along the main Dutch rivers are suitable for a longitudinal mound on the floodplain?".

   To determine which locations are suitable for the use of a longitudinal mound there are multiple conditions that are of importance. Firstly, the geometry and local structures at and around the floodplain have to be known. Also, as the goal is to use local soil it is important to know if the sand and especially clay is available locally. The location of current habitat and Natura2000 areas are of importance too. And finally, the expected wave height at the location where a longitudinal mound will be constructed is of importance.

2. "How to optimize the effects of decreased wave run-up and increased water levels when a longitudinal mound is used to reduce the crest level of the dike?"

   The optimization between the reduced wave run-up and increased water level will lead to an optimum design of the longitudinal mound. However, the volume of soil needed to reach this maximum dike crest height reduction might be much larger than a variant which reduces the dike crest height a bit less. Therefore, there is another optimum based on dike crest reduction per volume used.

3. "How accurate does the conceptual model calculate the water level change resulting from the longitudinal mound?"

   If the calculated water level changes in the conceptual model are similar to complex 2D calculations done by D-Flow FM, this means that in preliminary stages the simple calculations of the conceptual model are satisfactory. This will result in a reduction of necessary computation power and significantly speeds up the design process.

1.4. Approach

To find the answers to the research questions, which are needed to reach the research objective, a certain method is necessary. Therefore the method is presented per research question.

To answer the first subquestion data will be mapped in a GIS to find suitable locations for a longitudinal mound. Firstly, a selection of locations is needed as a base for the GIS analysis. For these locations multiple parameters will be investigated and by giving a score for all parameters the most suitable locations will be determined.

To answer the second question first a conceptual model will be made. In this conceptual model the expected design water levels and wave heights will be determined for different variants. To do this the expected wave damping for each variant is needed. As the longitudinal mound will function as a submerged wave breaker, methods to calculate the decrease in wave height from submerged wave breakers will be used.

After the conceptual model a more complex 2D D-Flow FM calculation will be done in which multiple designs will be calculated as well.

Finally, to answer the third question the results from the conceptual model and 2D D-Flow FM model will be compared. Therefore, the same designs on the same floodplain will be calculated both for the conceptual model and the 2D D-Flow FM model. It is important to do this for multiple designs to see if the results will differ.

1.5. Wrong parameters in the conceptual model

During the finalisation process it was found that a mistake was made in the conceptual model. In the Friebel and Harris wave height transmission formula the water level instead of the water depth had been used. Also for the height of the structure the crest height in m+NAP was used instead of the height from the floodplain level. Therefore, the calculated dike crest height reductions as a result of the height of the longitudinal mound are not entirely correct in Chapter 4. The results with the correct parameters in the Friebel and Harris formula give a lower dike crest height reduction for low crested longitudinal mounds. The difference for high crested longitudinal mounds is very small. More information can be found in Chapter 5

1.6. Outline

In chapter 2 the background that is needed to perform the location study, the conceptual model and the 2D D-Flow FM model is discussed. This is background on both the data that is used and for the empirical formulas and the physics behind those formulas.

In chapter 3 the methodology for the location study, the conceptual model and the 2D D-Flow FM model is described. In chapter 4 the results of all these parts are treated.

In chapter 5 a discussion is provided in which the consequences of the assumptions that are made and other limitations that were encountered during the thesis are mentioned. Next, in chapters 6 and 7 the conclusions and recommendations are given.

2. Background

For a detailed description of HydraNL and D-Flow FM it is recommended to consult the respective user manuals (Duits, 2020 and Deltares, 2020a, 2020b, 2020c, 2020d, 2020e).

2.1. Data – GeoTOP

To analyse the suitability and effectiveness of a longitudinal mound, data about the different possible locations is necessary. This data should cover local properties like floodplain size, soil composition, habitat and wave heights.

Satellite imagery

With satellite imagery lots of physical properties of each location can already be measured. This is done with publicly accessible imagery by Google and PDOK. (PDOK does not provide satellite imagery, but high-quality maps, including terrain maps as AHN3, the Dutch elevation map.) With Google's imagery distances can be measured by their Google Maps and Google Earth programs. The PDOK data has been added to QGIS in which more functions are available. QGIS is further used to visualize the results of the locations study, see Section 3.1 and Section 4.1. So, the length, width and average width of the floodplains can be measured. With this data and tools it is also possible to determine the percentage of structures on the inside of the dike and vegetation on the outside of the dike. This data is used in the location study.

GeoTOP

If local soil is used to construct a longitudinal mound it is of importance to know what the local soil consists of. As the locations along the rivers are spread over a large area of land, taking into account all local borehole results is very time consuming. Therefore, the GeoTOP database is used to determine what kind of soil is present at all locations.

GeoTOP is a voxelmodel that provides a three-dimensional view of the subsurface of the Netherlands. The model is divided into cells of 100 by 100 metres with a depth of 50 centimetres. Subsequently, all cells contain information about lithological classes. For all lithological classes a probability of occurrence is given. The probability of occurrence is based on the available digital borehole logs of the DINO database. This database consists of approximately 540 000 borehole logs. Along the Rhine-Meuse area also borehole logs from the Department of Physical Geography of the University of Utrecht have been used (DINOloket, 2023).

Although there are a lot of borehole logs available, if the boreholes are spaced evenly, about 10 per cent of the ground level cells is penetrated by a borehole. This value decreases further over depth. The estimation of the content of each cell is done based on nearby borehole logs, with stochastic interpolation techniques (Stafleu and Dubelaar, 2016).

The lithological classes used in GeoTOP are anthropogene, organic material, clay, clayey sand and sandy clay, fine sand, moderately coarse sand, coarse sand, gravel and shells. In this research the focus is on clay. This is because the clay is used as toplayer on the longitudinal mound and would have to satisfy relatively strict requirements before the clay can be approved for use.

"GeoTOP is not appropriate for use at a local scale, e.g. a building site, individual houses, apartment blocks and water defences (Stafleu and Dubelaar, 2016)." However, as the floodplains are a magnitude larger than the before mentioned cells, the lithological class with the highest probability of occurrence at each cell is assumed to be present at that cell. So, in the volume calculations of each lithological class the most probable class is used for each cell.

Another argument in favour of this assumption is that there are more borehole logs present in the Rhine-Meuse area and only the upper 5 metres is used. Unfortunately, not the entire area of the Netherlands has GeoTOP coverage, see Figure 3. Therefore, the IJssel-river is excluded in the location study.

Figure 3 Area covered by GeoTOP (Stafleu and Dubelaar, 2016)

HydraNL

The effectiveness of the longitudinal mound is based on the design wave height for the river dike at each of the locations. The wave heights are calculated with HydraNL. HydraNL makes use of Bretschneider's wave growth formulas to calculate the wave height and wave period.

2.2. Wave growth formula – Bretschneider

HydraNL uses the Bretschneider wave growth formulas to calculate the significant wave height and significant wave period for each point along the river dike. Bretschneider's wave growth formulas are as follows:

Deep water

$$H_s = \frac{0.283\, u^2}{g} tanh\left(0.0125\left(\frac{g\, F}{u^2}\right)^{0.42}\right)$$

$$T_s = \frac{2.4\, \pi\, u}{g} tanh\left(0.077\left(\frac{g\, F}{u^2}\right)^{0.25}\right)$$

Shallow water

$$H_s = \frac{0.283\, u^2}{g} tanh\left(0.530\left(\frac{g\, d}{u^2}\right)^{0.75}\right)$$

$$T_s = \frac{2.4\, \pi\, u}{g} tanh\left(0.833\left(\frac{g\, d}{u^2}\right)^{0.25}\right)$$

With:

H_s = the significant wave height [m]
T_s = the significant wave period [s]
u = the windspeed at 10 metres height [m/s]
g = the gravitational constant, 9.81 [m/s^2]
F = the effective fetch [m]
d = the water depth [m]
(TAW, 1989)

Intermediate water

The wave growth formulas as shown above apply to the case where either the fetch (deep water) or the water depth (shallow water) is fully responsible for the wave growth. However, generally the water depth and the fetch both influence the expected wave height (TAW, 1989). Therefore, a new formula for intermediate water is needed. This is done by adding variables v_1 and v_2 to the original formulas for deep water:

$$H_s = \frac{0.283\, u^2\, v_1}{g} tanh\left(\frac{0.0125}{v_1}\left(\frac{g\, F}{u^2}\right)^{0.42}\right)$$

$$T_s = \frac{2.4\, \pi\, u\, v_2}{g} tanh\left(\frac{0.077}{v_2}\left(\frac{g\, F}{u^2}\right)^{0.25}\right)$$

With:

$$v_1 = tanh\left(0.530\left(\frac{gd}{u^2}\right)^{0.75}\right)$$

$$v_2 = tanh\left(0.833\left(\frac{gd}{u^2}\right)^{0.375}\right)$$

(Duits, 2020)

With these formulas it is possible to determine the wave height and wave period. These wave heights and wave periods have been visualized by a graph for different (average) water depths, in which the fetch and windspeed are variable. An example is given in Figure 4.

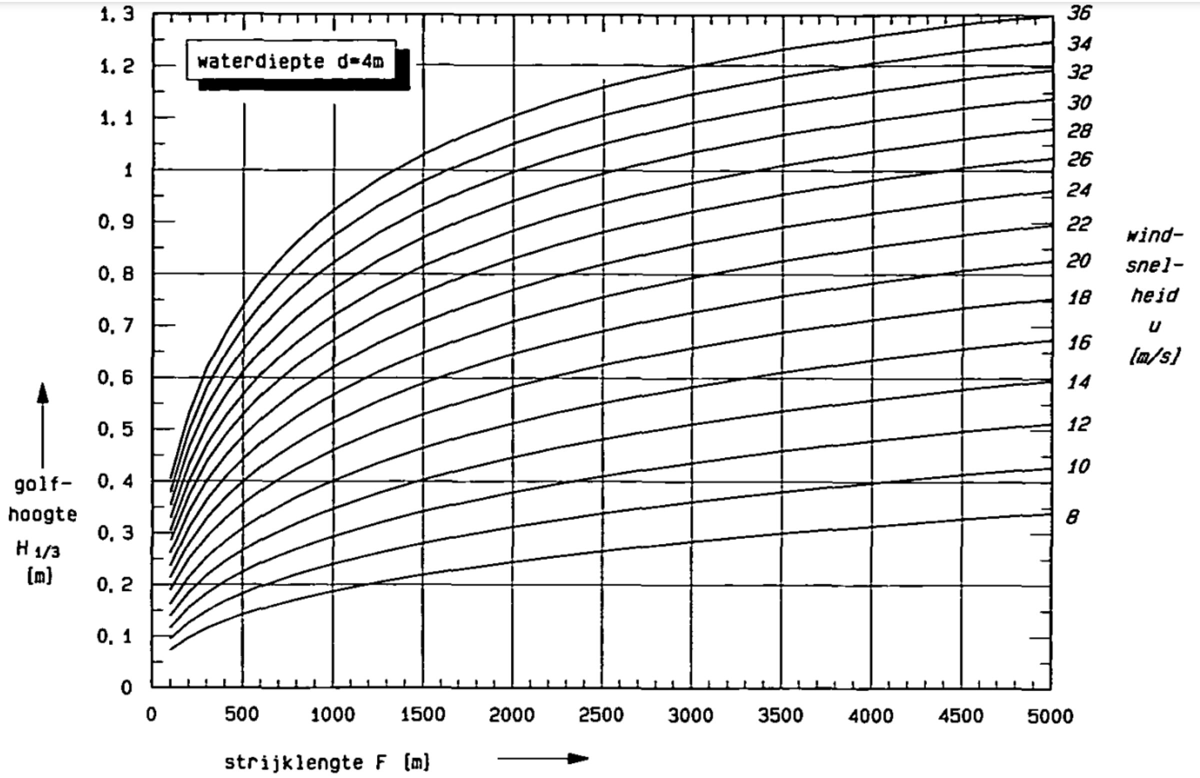

Figure 4 Example of Bretschneider wave growth curve. Water depth of 4 metres. (TAW, 1989)

Windspeed

For different wave growth formulas different definitions for the wind speed have been used. Bretschneider uses the 10-metre-above-water-level wind speed in the wave growth calculations. This average surface wind speed is determined by taking into account the effect of curvature and air-sea temperature differences on the geostrophic wind speed (Bretschneider, 1964).

Water depth

To calculate the wave height at the toe of the river dike HydraNL uses the average water depth over the fetch. The water depth is not split in multiple sections along the fetch line (Duits, 2020). It is possible to manually split the water depth parameter if there are significant differences of the water depth over the fetch. However, an increase of water depth over a short distance does not warrant the water depth to be split in multiple sections (TAW, 1989). Especially at locations where the floodplains are wide this is the case during design conditions.

Effective fetch

The fetch used in the Bretschneider wave growth formula is the effective fetch. The principle of the effective fetch is based on that the wind transfers energy to the water not only in the wind direction, but also within a certain angle (θ) in both directions from the wind direction (Camarena Calderon et al., 2016). In HydraNL θ_{max} is set to 42° with an interval of 6°. Therefore, 15 fetch rays are considered for all wind directions. The fetch rays are the distance from the output location to the edge of the water line. The effective fetch is calculated as follows:

$$F_e = \frac{\sum R_i(\theta_i) \cdot \cos^2(\theta_i)}{\Sigma \cos(\theta_i)}$$

With:
F_e = the effective fetch
R_i = the length of fetch ray i
θ_i = the angle between fetch ray i and the wind direction
(Duits, 2020)

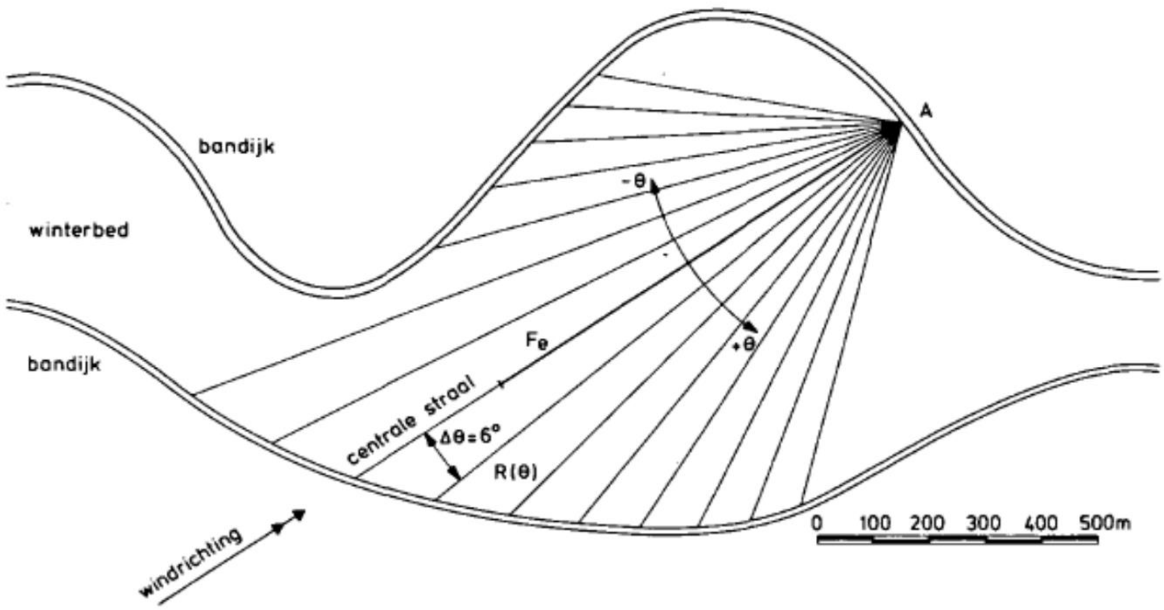

Figure 5 Visual example of the calculation of the effective fetch. (Duits, 2020)

In Figure 5 above a visual example of the calculation of the effective fetch is shown. In HydraNL the effective fetch is calculated for 16 wind directions split by 22.5°. Subsequently, the design wave heights in combination with design water levels are calculated. The combination of wave height and water level is different for each direction. The resulting hydraulic load is equal for each direction. The probability for each direction is given to assess which combination is most likely to occur.

2.3. Wave height transmission over a submerged breakwater – Friebel and Harris

With the goal of the longitudinal mound to reduce the wave attack on the river dike it is important to know how the waves change over the longitudinal mound. The longitudinal mound acts as a submerged breakwater during design conditions. This results in a reduction of wave height as the wave passes the longitudinal mound. In Figure 6 below an example of a rubble-mound submerged breakwater from Seabrook (1997) is shown.

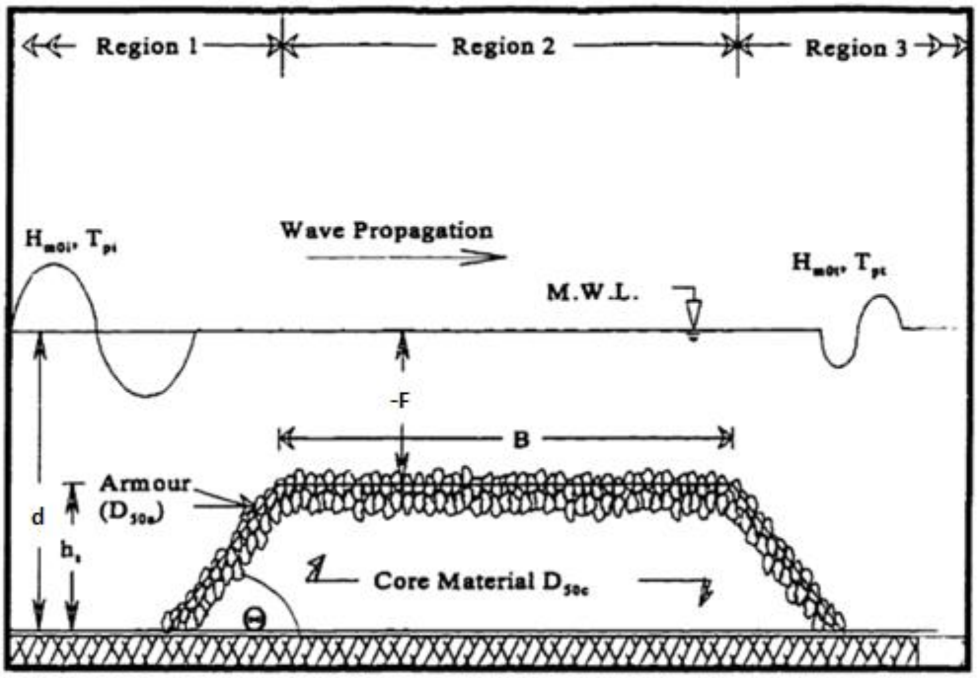

Figure 6 Example of a rubble mound submerged breakwater from Seabrook (1997).

In the past multiple researches have been done to find the influence of a submerged breakwater on waves. Most of this research has been performed on (coastal) rubble-mound submerged breakwaters, for instance in Seelig (1980) and Daemen (1991). These rubble-mound breakwaters are permeable which is not the case for the longitudinal mound, which has a clay cover. However, for a submerged breakwater the permeability has only a small influence on the wave transmission coefficient. The slope angle of a submerged breakwater also has a small influence. So, these parameters are not taken into account (Daemen, 1991).

Friebel and Harris (2003) took five datasets from different researches (Seelig 1980, Daemrich and Kahle 1985, Van der Meer 1988, Daemen 1991 and Seabrook 1997) to construct an empirical best-fit model for the wave transmission over submerged breakwaters. This best fit is based on the five data sets that have been used (Friebel and Harris, 2003). The empirical formula reads as follows:

$$k_t = -0.4969\, exp\left(\frac{F}{H_i}\right) - 0.0292\left(\frac{B}{d}\right) - 0.4257\left(\frac{h}{d}\right) - 0.0696\, ln\left(\frac{B}{L}\right) + 0.1359\left(\frac{F}{B}\right) + 1.0905$$

With:

k_t	= the wave transmission coefficient [-]
F	= the freeboard [m] determined as: $h - d$ (For a submerged breakwater this means F is negative.)
H_i	= the incident wave height [m]
B	= crest width of the structure [m]
d	= water depth at the toe of the structure [m]
h	= height of the structure from the bed [m]
L	= wavelength at local depth [m]

The wave transmission coefficient depends on five dimensionless variables. In order of decreasing significance the five dimensionless variables are the relative freeboard, which is the ratio of freeboard to incident wave height $\left(\frac{F}{H_i}\right)$, structure crest width to water depth $\left(\frac{B}{d}\right)$, height of the structure from the bed to water depth $\left(\frac{h}{d}\right)$, crest width to wavelength $\left(\frac{B}{L}\right)$ and freeboard to crest width $\left(\frac{F}{B}\right)$. Thereby, it is important to note that these five variables should be in the range given in Table 1 to assure the validity of the model (Friebel and Harris, 2003).

Table 1 Limits related to the five variables used in the Friebel and Harris formula (Friebel and Harris, 2003).

Lower limit	variable	Upper limit
-8.696	$\le \left(\frac{F}{H_i}\right) \le$	0.000
0.286	$\le \left(\frac{B}{d}\right) \le$	8.750
0.440	$\le \left(\frac{h}{d}\right) \le$	1.000
0.024	$\le \left(\frac{B}{L}\right) \le$	1.890
-1.050	$\le \left(\frac{F}{B}\right) \le$	0.000

All in all, this means that the wave transmission depends on two design parameters of the submerged breakwater and on three local condition parameters. The design parameters of the submerged breakwater are its crest height and crest width. The slope angle only has a small influence and is therefore neglected. For the longitudinal mound a shallower slope results in less flow area during design conditions and still influences the water level.

The local condition parameters are the water depth, the wave height and the wave length. The values for these parameters that are used are the design values. These values can be determined by using HydraNL with the corresponding design return periods.

2.4. Wave period transmission over a submerged breakwater – Carevic

Not only the wave height changes when the wave travels over a submerged breakwater, also the wave period changes. In Figure 7 below it is shown that for longer waves (with a lower wave steepness) the number of transmitted waves can be up to 40% larger than the amount of incident waves. This percentual increase in number of waves decreases for an increased relative freeboard (the freeboard over the wave height) and for an increasing wave steepness. So, a lower wave steepness and higher wave height will result in an increase of number of transmitted waves (Carevic et al., 2013).

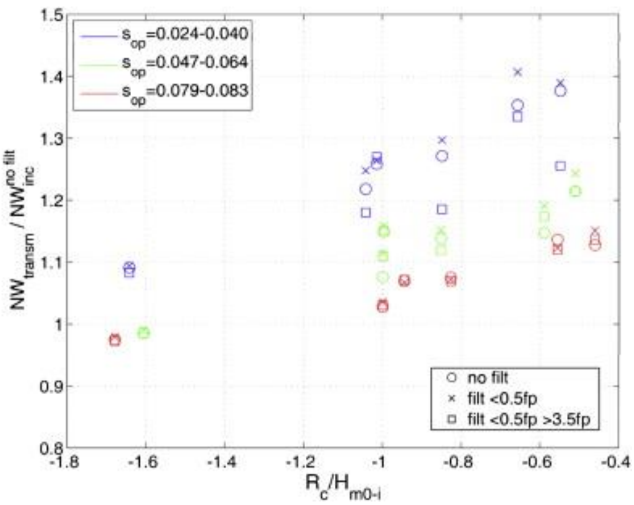

Figure 7 Increase in number of transmitted waves based on wave steepness and relative freeboard (Carevic et al. 2013)

As a result of the wave transmission over the submerged breakwater wave energy is transferred to higher frequencies, along an increase in the second spectral moment m_2. This causes a reduction of the mean spectral wave period. This reduction is shown in Figure 8 below. In this figure also the results of Van der Meer et al. (2000) for similar-shape emerged breakwaters are added. It can be seen that for the less steep waves both curves tend to 0.68 when the relative freeboard approaches 0 (Carevic et al.).

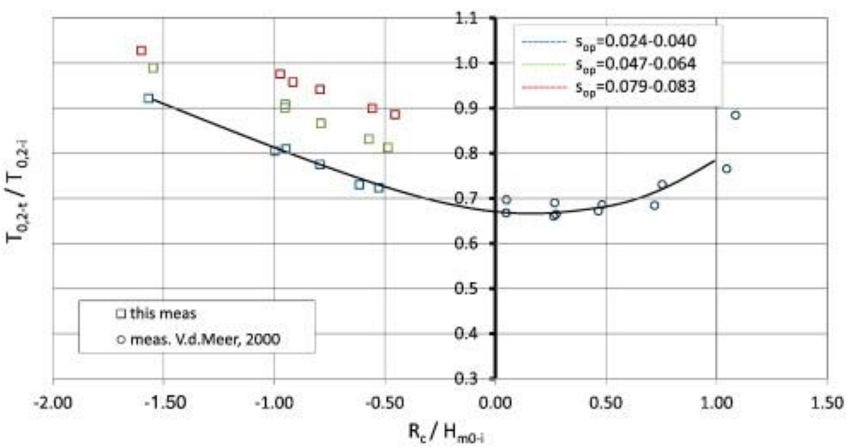

Figure 8 Reduction of the wave period based on the relative freeboard (Carevic et al. 2013)

2.5. Overtopping – Van der Meer equations

The crest height of a riverdike is determined by following the GEKB (Grasbekleding Erosie Kruin en Binnentalud, Eng: Erosion grass cover crest and inner slope) failure path as prescribed in Appendix iii of the *Regeling veiligheid primaire waterkeringen 2017* (Rijkswaterstaat, 2017). In the GEKB a maximum allowable overtopping is given based on the allowable erosion of grass on the inner side of the dike. This overtopping is calculated with the Van der Meer wave overtopping formula.

The wave overtopping discharge is given as an average discharge per metre of dike crest width, and is usually given as m^3/(s*m or l/(s*m). The allowable overtopping is dependent on the wave height. Furthermore, the allowable overtopping depends on the quality of the grass cover and potential hazards posed by overtopping. The relevant overtopping limits are shown below in Table 2 and Table 3 (EurOtop, 2018).

Table 2 Allowable overtopping discharges based on soil and grass cover (EurOtop, 2018).

Soil and grass cover	Allowable average overtopping discharge l/(s*m)
Grass covered crest and landward slope; maintained and closed grass cover; H_{m0} = 1 – 3 m	5
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; H_{m0} = 0.5 – 3 m	0.1
Grass covered crest and landward slope; H_{m0} < 1 m	5-10
Grass covered crest and landward slope; H_{m0} < 0.3 m	No limit

Table 3 Relevant allowable overtopping discharges based on hazard type (EurOtop, 2018), more can be found in EurOtop 2018.

Hazard type and reason	Allowable average overtopping discharge l/(s*m)
People at seawall / dike crest. Clear view of the sea.	
H_{m0} = 3 m	0.3
H_{m0} = 2 m	1
H_{m0} = 1 m	10-20
H_{m0} < 0.5 m	No limit
Cars on seawall / dike crest, or railway close behind crest	
H_{m0} = 3 m	<5
H_{m0} = 2 m	10-20
H_{m0} = 1 m	<75
Property behind defence	
Building structure elements; H_{m0} = 1-3 m	≤1
Damage to equipment set back 5-10m	≤1

Although the maximum volume per overtopping wave is of importance, it is left out in Tables 2 and 3. This is done as in the Van der Meer wave overtopping formula the average overtopping discharge is calculated.

At river dikes it is expected that design wave heights of up to about 1.5 metres will occur. From the tables above it is clear that there are different limits based on different scenarios. In the calculations further in this research a maximum of 0.1 l/(s*m) is used to accommodate for all possible hazards and dike qualities. Also Appendix III of the *Regeling veiligheid primaire waterkeringen 2017* mentions to use an average discharge of 0.1 l/(s*m) for waves smaller than 3 metres.

Another benefit of using the same value for the allowable wave overtopping discharge on multiple locations, is in the comparison of the results. It eliminates a variable and makes the effect of a longitudinal mound on the floodplain on the results easier to distinguish. In reality the local maximum overtopping discharges are probably larger than in the calculations. This might have an effect on the crest hight decrease.

The aforementioned Van der Meer wave overtopping formula is as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{tan\,a}} \cdot \gamma_b \cdot \xi_0 \cdot exp\left(-4.3\frac{Rc}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)$$

$$\xi_0 = \frac{\tan a}{\sqrt{s_0}}$$

$$s_0 = \frac{2\pi H_{m_0}}{gT_{m-1,0}^2}$$

with a maximum of (when breaker parameter is larger than ~1.8):

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \cdot exp\left(-2.3\frac{Rc}{H_{m0}} \cdot \frac{1}{\gamma_f \cdot \gamma_\beta}\right)$$

With:

q = average wave overtopping discharge [m³/s per m]
g = gravitational constant, 9.81 [m/s²]
H_{m0} = significant wave height at the toe of the dike [m]
$tan\ \alpha$ = dike slope [-]
R_c = free crest height above the water line [m]
ξ_0 = wave breaker parameter [-]
s_0 = wave steepness [-]
$T_{m-1,0}$ = spectral wave period at the toe of the dike [s]
γ_b = influence factor of the berm [-]
γ_f = influence factor of roughness elements [-]
γ_β = influence factor of the angle of wave attack [-]
γ_v = influence factor of a vertical wall on slope [-]

With this formula the necessary dike crest height can be determined. The free crest height above the waterline, R_c, is the unknown parameter. The rest of the formula consists of the wave parameters wave height and wave length and the dike slope. These parameters are also used in the calculation of the breaker parameter and four influence parameters.

The four influence parameters, γ, are the berm factor, roughness factor, wave angle factor and the vertical wall factor. The roughness factor is determined for different materials and artificial blocks, with a factor of 1 for grass. The other three parameters are calculated by the following formulas:

Berm:

$$\gamma_b = 1 - r_b(1 - r_{dh})$$
$$r_b = \frac{B}{L_{\text{berm}}}$$
$$r_{dh} = 0.5 - 0.5\cos\left(\pi \frac{d_h}{2H_{m0}}\right)$$

With:
$B =$ the length of the berm [m]
$L_{\text{berm}} =$ the length of the berm between 1 H_{m0} below and above the berm level [m]
$d_h =$ the water depth on the berm [m]
$H_{m0} =$ the significant wave height [m]

Wave angle:

$$\gamma_\beta = 1 - 0.0033 \cdot |\beta| \qquad \text{for } 0^\circ \leq |\beta| \leq 80^\circ$$
$$\gamma_\beta = 1 - 0.0033 \cdot 80 \qquad \text{for } |\beta| > 80^\circ$$

With:
β = incident wave angle [°]

Vertical wall:

$$\gamma_v = 1.35 - 0.0078 \cdot \alpha_{wall}$$

With:
α_{wall} = the angle of the steep slope in degrees (larger than 45°) [°]

In the Overtopping manual from 2018 a new Van der Meer formula is used. In the new version the numerical empirical parameters changed from 0.067, -4.3, 0.2 and -2.3 to 0.026, -2.5, 0.1035 and -1.35 respectively. Also an exponent of 1.3 is added to the e-exponent on the right hand sight. With these changes in the formula the application area has increased from $R_c/H_{m0} > 0.5$ to $R_c/H_{m0} > 0$. However, in HydraNL the old Van der Meer formula is used. Therefore, this formula has been used in this research as well.

2.6. D-Flow FM background

2.6.1. Refining grid and CFL criterion

To obtain a higher resolution in the area of interest the provided grid is to be refined locally. This is done by the algorithm CellsandFaces2 which is delivered within RGFGRID, which is Deltares' grid editor. The CellandFaces2 algorithm is fairly simple. It splits the original cell edges in two equal parts and creates 4 new cells. This can be done multiple times dividing the cell area by four for every iteration. Between the new cells and the original cells one band of intermediate cell are created. This intermediate band consists of three triangular cells that span one original cell. It is of importance to keep the non-orthogonality of the grid below a maximum of 0.5, as this is the maximum that D-Flow FM is able to work with.

The numerical method used in D-Flow FM is stable for a Courant number up to 0.7. The Courant number is specified as the flow velocity times the time step divided by length interval (cell size). D-Flow FM uses the maximum possible time step to remain stable. Because of the smaller cell size a smaller timestep is needed. This results in an increased runtime.

2.6.2. Hydrodynamics

D-Flow FM solves the shallow-water equations numerically. The shallow-water equations are derived from the Navier-Stokes equations. The most important assumption for the validity of the shallow-water equations is that the water depth is much smaller than the horizontal length scale. This has as a result the vertical momentum equation is reduced to the hydrostatic pressure relation. This leads to the following equations:

Continuity equation:

$$\frac{\partial h}{\partial t} + \frac{\partial hu}{\partial x} + \frac{\partial hv}{\partial y} = Q$$

Horizontal momentum equations:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} - fv = -\frac{1}{\rho_0}\frac{\partial P}{\partial x} + \frac{\partial}{\partial z}\left(v_V\frac{\partial u}{\partial z}\right) + F_x + M_x$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + fu = -\frac{1}{\rho_0}\frac{\partial P}{\partial y} + \frac{\partial}{\partial z}\left(v_V\frac{\partial u}{\partial z}\right) + F_y + M_y$$

with:

x	= x-coordinate [m]
y	= y-coordinate [m]
z	= z-coordinate [m]
t	= time [s]
h	= water depth [m]
u	= depth average velocity in x direction [m/s]
v	= depth average velocity in y direction [m/s]
Q	= added discharge or withdrawal per unit area [m/s]
f	= Coriolis parameter [-]
ρ_0	= water density [kg/m^3]
P	= water pressure [N/m^2]
v_V	= vertical eddy viscosity coefficient
$F_{x,y}$	= unbalance of horizontal Reynolds stresses
$M_{x,y}$	= external sources and sinks of momentum
g	= gravitation acceleration [m/s^2]

On the left-hand side are the local acceleration and advection terms and on the right hand side the driving forces are stated. These driving forces are the pressure gradients, the vertical eddy viscosity, the unbalance of horizontal Reynolds stresses and external sources and sinks of momentum of the system (Deltares, 2020d).

To solve the shallow-water equations every grid cell needs a bed level value. However, due to the grid refinement not all cells do have a bed level value. Within the D-Flow FM GUI this can be solved by Delaunay triangulation interpolation. This gives all cells without bed level value an interpolated value based on the original bed levels surrounding the new cell.

3. Methodology

3.1. Location analysis

The goal of the location analysis is to determine which areas in the Netherlands could profit from a longitudinal mound. To do this multiple criteria have to be met. Therefore, the following parameters have been taken into account: the floodplain size, local structures, the local soil, the current habitat and the expected wave height. The first four parameters together form the suitability of each location and the wave height determines the effectiveness. The process is shown in Figure 9.

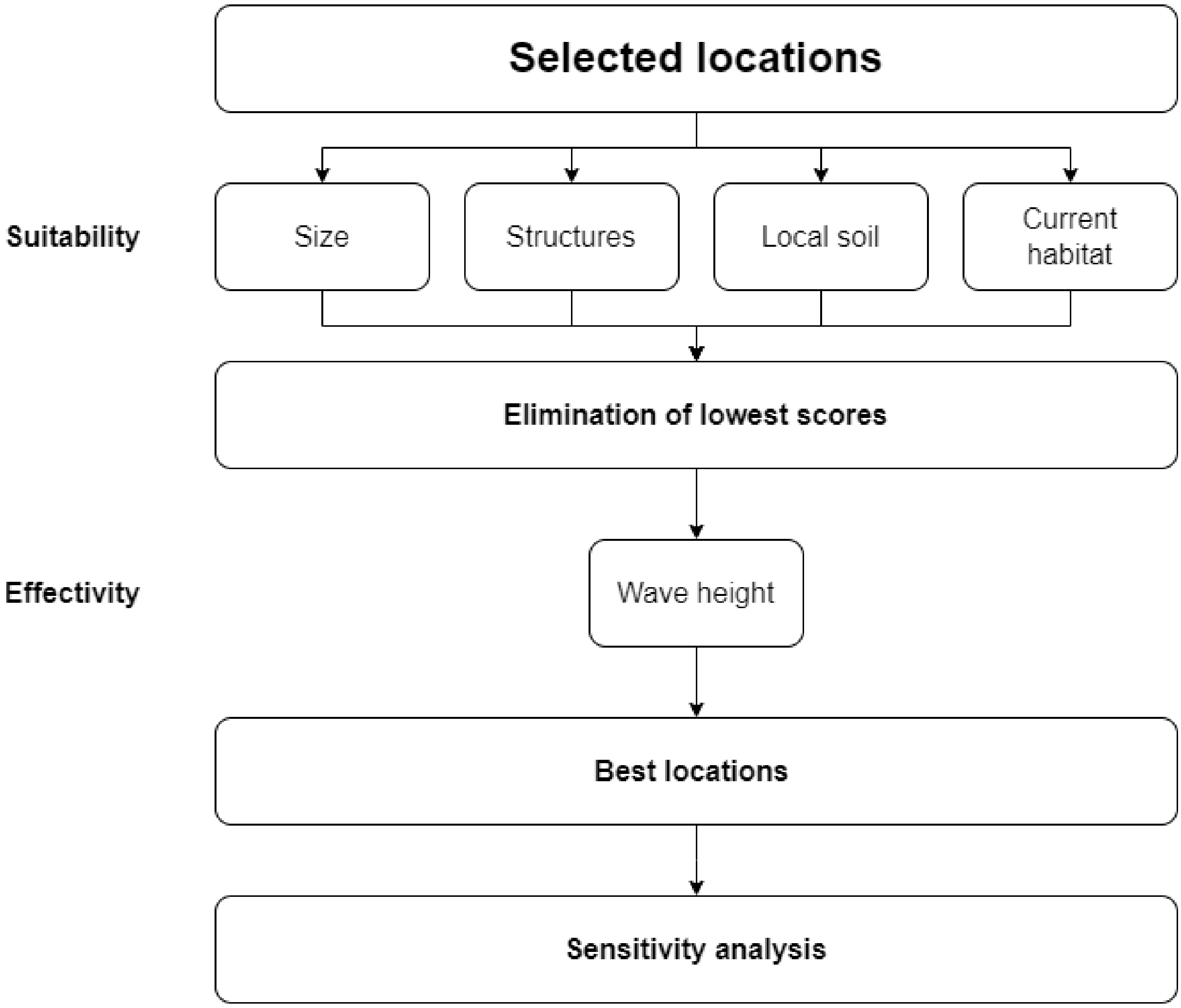

Figure 9 Flow chart of methodology for the location analysis.

3.1.1. Analysis for potential locations

Before the location study can be done a preselection is conducted. A couple of factors have already been taken into account in this preselection. The first of these is that the main wind direction in the Netherlands is primarily into an northeast direction, a wind rose from De Bilt is shown in Figure 10. Therefore, floodplains where the dike is expected to have wave impact from the main wind direction are preferred. However, there are some exceptions to this general rule as following this rule very strictly would reduce the sample size significantly. Therefore, more locations are taken into account in the study than only the locations where the dike is perpendicular to the wind direction. So, it is possible to compare the results of differently orientated floodplains.

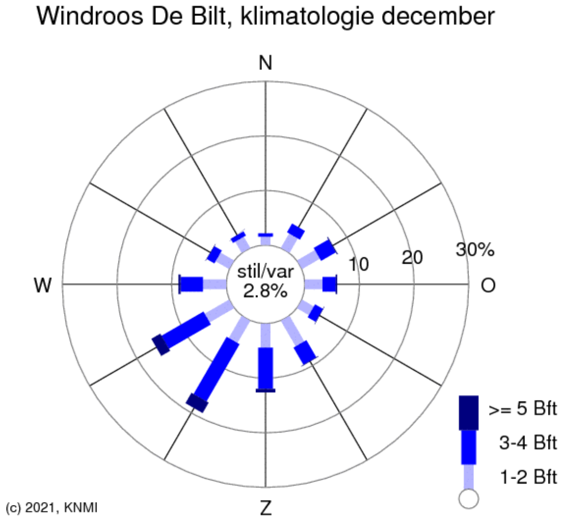

Figure 10 Wind rose of prevailing winds for De Bilt of December from 1991-2020. Source: KNMI (2021)

Also the sizes of the floodplains are taken into account. Wider floodplains are preferred. The reason for this is twofold. Firstly, this means that there generally is a larger fetch present before the dike, resulting in higher wind waves. Also space is needed for the longitudinal mound as it should not fill up the entire floodplain. With the following assumptions a minimum of 80 metres is necessary for the longitudinal mound itself.

Maximum crest height of mound above floodplain level:	~5 metres
Minimum slope of longitudinal mound:	~1:6
Maximum longitudinal mound crest width:	~20 metres

Space is also needed between the dike and mound and preferably the main channel and mound. Therefore the minimum distance is multiplied by a factor of 3. This results in a minimum floodplain width of about 250 metres.

Finally, there are some floodplains that accommodate a side channel or are inundated during normal conditions. These floodplains are not used in the analysis as constructing a longitudinal mound on an inundated floodplain requires a larger amount of soil and construction is more difficult. Also side channels are generally constructed to accommodate extra space during design conditions, which will be reduced when a longitudinal mound would be constructed there.

After taking these conditions into account along the Dutch branches of the Rhine (Bovenrijn, Pannerdensch Kanaal, Neder-Rijn, Lek, Waal) and the Meuse downstream of Gennep 43 floodplains have been selected for the analysis, as shown in Figure 11. The IJssel is not considered as no GeoTOP data was available for most of its flow area.

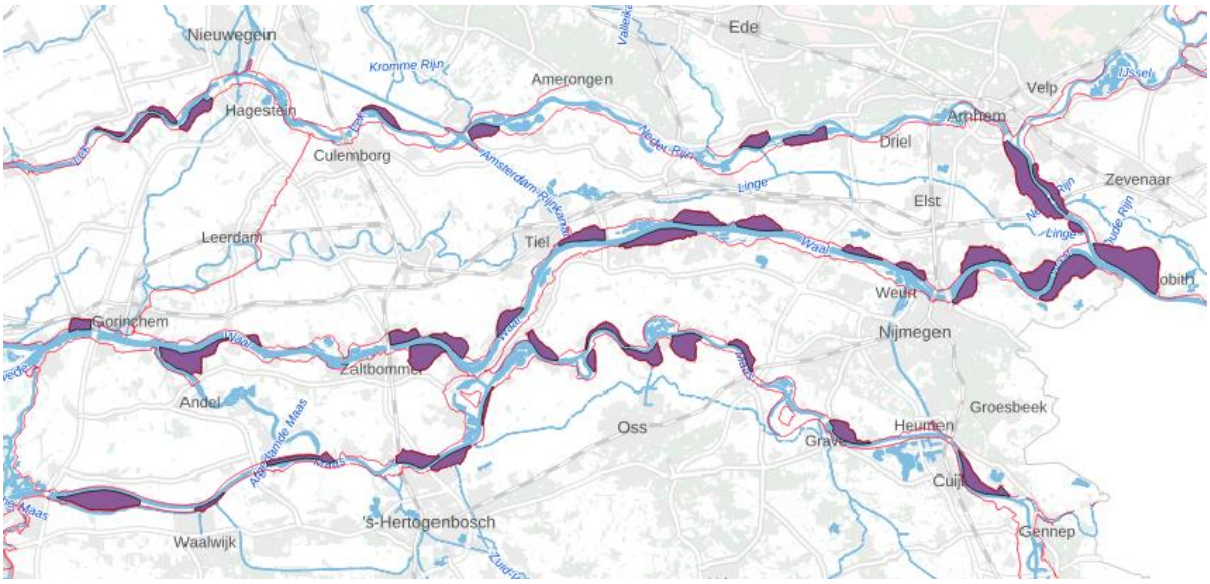

Figure 11 All selected locations that are analysed in this study

3.1.2. Suitability

To analyse potential locations where a longitudinal mound can be constructed multiple criteria are of importance. These criteria are split into two categories. The first category is the suitability of the location. This suitability is based on four parameters: the floodplain size, local structures around the floodplain, the availability of local soil on the floodplain and the current habitat. For all parameters a rating from 1 to 5 points is given to all locations. The exact determination of the points is explained in each subsection related to all the parameters.

The suitability depends on the following criteria. Firstly, the size of the floodplain is looked at. Not only the maximum width and the length of the floodplain are taken into account, but the average width is taken into account as well. The width of the floodplain is important as a wider floodplain means the longitudinal mound will be using relatively less area for water flow during design conditions then a smaller floodplain.

Secondly, structures around the floodplain are taken into account. This is done by examining the relative amount of structures like houses and farms along the dike and potential other features on the floodplain that influence flow on the floodplain like factories and harbours.

Thirdly, to accommodate the goal of using local soil, the potential of clay located in the upper layers of the floodplain is analysed. This analysis is done for the upper 2.5 metres and the upper 5 metres. Clay will be the soil type for which it is most difficult to find soil that is suitable for use, as clay will be used in the toplayer of the longitudinal mound. Therefore this analysis is done for clay and sandy clay.

Fourthly, the current habitat on the floodplain is taken into account. Some floodplains are classified as Natura2000 areas. Natura2000 areas are ecologically valuable areas. Therefore, these areas are protected to preserve the unique ecosystems within. So, to make changes in Natura2000 areas there is a need for a compensation in other areas to not lose ecological value (BIJ12, 2019). However, this would change these valuable areas, which might not be recreated elsewhere. Also, this could lead to higher costs due to the compensation of constructing a longitudinal mound in a Natura2000 area. Therefore, construction of a longitudinal mound in a Natura2000 area is not preferred.

3.1.2.1. Size of the floodplain

To determine the points for the size of the floodplains there are three different categories: maximum width, average width and length. The largest floodplains received 5 points and 1 point is given to the smallest floodplains. This is done for each of the categories. The points are distributed as follows.

Maximum width:

Width > 1 500 m:	5 points
1 100m < width ≤ 1 500 m:	4 points
800 m < width ≤ 1 100 m:	3 points
500 m < width ≤ 800 m:	2 points
500 m ≥ width:	1 point

Average width:

Width > 900 m:	5 points
700m < width ≤ 900 m:	4 points
500 m < width ≤ 700 m:	3 points
300 m < width ≤ 500 m:	2 points
300 m ≥ width:	1 point

Maximum length:

Length > 4 500 m:	5 points
3 500m < length ≤ 4 500 m:	4 points
2 500 m < length ≤ 3 500 m:	3 points
1 500 m < length ≤ 2 500 m:	2 points
1 500 m ≥ length:	1 point

3.1.2.2. Structures around the floodplain

Determining the points for the structures around the floodplain is done by taking the percentage of the length of the dike around the floodplain over which structures are located. These structures block inland extension of the dike footprint. When a harbour, factory or other structure is located immediately on the outside of the dike this distance is removed from the total dike length. A higher percentage means that there are more structures in front of the dike blocking inland dike expansion. When inland dike expansion is not possible, the construction of a longitudinal mound is a potential solution. This results in the following point distribution:

Length > 60%:	5 points
50% < length ≤ 60%:	4 points
40% < length ≤ 50%:	3 points
30% < length ≤ 40%:	2 points
30% ≥ length:	1 point

At first 60% might seem like a relatively low threshold to give 5 points. However, this percentage is based on the absolute distance where inward expansion is not possible. This means that at locations where two farms are located some distance from each other this distance is counted as possible to expand inwards. However, in practise this would lead to inefficient design for dike reinforcements as different measures would have to be provided for relatively small stretches of dike.

3.1.2.3. Availability of local soil

In this assessment the soil that is present on the floodplain directly adjacent to the dike that is to be protected with the longitudinal mound is deemed to be local soil. Clay is the material which is of most importance to the longitudinal mound, as the toplayer of the longitudinal mound will be a clay layer. Although the volume of clay on the toplayer is less than the volume of soil that is needed to fill the longitudinal mound, the toplayer has to satisfy more stringent requirements than the fill material. Therefore the main focus of this assessment is on clay.

Within the GeoTOP voxel model multiple lithological classes are determined. The anthropogene areas indicate human made structures like roads, buildings or other structures. These are only shown for the upper voxel. However, these have to be marked as anthropogene on the layers beneath the upper layer as well. That is because it is not possible to retrieve clay from lower layers if a structure is built on top of it. An example of the upper voxel at Waal_08 is shown in Figure 12, in which the thin red line indicates the dikes and the thick red line indicates location Waal_08.

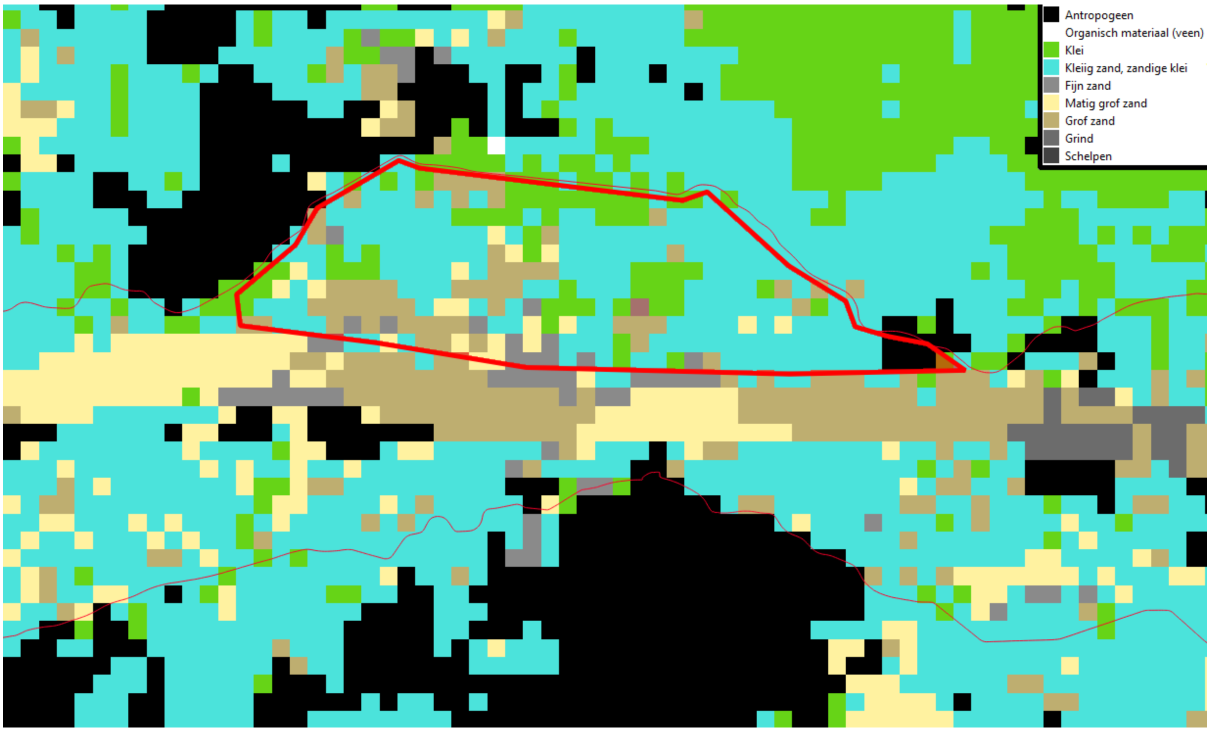

Figure 12 Example of voxel model at location Waal_08

Next, the availability of local clay is based on the percentage of clay (and sandy clay) in the upper 2.5 metre beneath ground level. The upper 2.5 metre is used as that is quite shallow. Therefore it is relatively easy to excavate. The points are distributed based on this percentage. The reason to do this based on the percentage is because it decreases the difference between smaller and larger floodplains. Therefore a comparison between the floodplains is possible.

The points are given based on the following clay percentages:

Clay > 25%:	5 points
17.5% < clay < 25%:	4 points
12.5% < clay < 17.5%:	3 points
7.5% < clay < 12.5%:	2 points
7.5% > clay:	1 point

Next to the 2.5 metre clay percentage, also percentages of the upper 5 metre of clay and sandy clay are determined. It is possible that in the marginally deeper layers more or less clay is available. The same distribution of points is used.

3.1.2.4. Current habitat

The assessment on the current habitat is mainly done by determining if the floodplain is classified as Natura2000 area. As mentioned before Natura2000 areas are ecologically valuable areas. The value of these Natura2000 areas grow as their surface area increases. Therefore one Natura2000 area with the same size as two smaller areas has a larger ecological value than these two areas combined (P.M.J. Herman, personal communication, 13-01-2023). Because current habitats may be disturbed or disappear when these areas are changed, it is more difficult to construct a longitudinal mound in a Natura2000 area.

Therefore the locations which are classified as Natura2000 receive less points. Locations that have a Natura2000 area received 1 or 2 points. The difference between 1 or 2 points is based on the amount of area that is classified as Natura2000 area within the locations. There is 1 point awarded when more than 50% of the floodplain area is Natura2000 and 2 points if this area is smaller than 50%.

To reflect the importance of the value of the Natura2000 areas 3 points are not awarded and locations that do not have a Natura2000 area receive 4 or 5 points. The difference between 4 or 5 points is based on the visible amount of trees and other flora on satellite and *Streetview* images. The construction of a longitudinal mound results in removal of vegetation if that is located at the same place the longitudinal mound will be. Therefore, if there is no vegetation to a maximum of a couple of single trees the location receives 5 points. If there is more vegetation present on the floodplain this location receives 4 points.

So, the points are given as follows:

Non to very little vegetation:	5 points
Small amount of vegetation:	4 points
0% < Natura2000 area ≤ 50%:	2 points
Natura2000 area > 50%:	1 point

3.1.3. Effectivity

After all the parameters for the suitability have been assessed, the lowest scoring locations are eliminated. Next, the effectivity of the longitudinal mound on the remaining locations is assessed. This analysis has been done on the expected significant wave height at the dike.

3.1.3.1. Expected significant wave height

The potential wave height decrease of the longitudinal mound is larger when the incoming waves are large. At locations where the fetch is larger, generally the waves are expected to be larger as well. With HydraNL the significant wave height along multiple points at every location are calculated.

In all calculations a return period of 1000 years is used. Although return periods used for the design of river dikes is generally larger than 1000 years, these return periods are not the same for all dike sections. With an equal return period the wave conditions at the different locations can be compared with each other.

After the calculations of the significant wave height for all points have been performed the significant wave heights at all data points for each location are used to determine the wave height used for this comparison. The points are given based on the following wave heights for the 1000 year return period:

Wave height > 1.05 m:	5 points
0.90 m < wave height ≤ 1.05 m:	4 points
0.75 m < wave height ≤ 0.90 m:	3 points
0.60 m < wave height ≤ 0.75 m:	2 points
0.60 m ≥ wave height:	1 point

At some locations there is a large difference between data points. Therefore the standard deviation of all wave heights is calculated for all location. A high standard deviation indicates a large difference between these different data points at one location. If the standard deviation is above 0.2 metres the determination of the score does not have to follow the rules above, but is determined for these locations separately.

3.1.4. Weights and sensitivity

Because not all mentioned parameters are of equal importance the given values have been multiplied with a weight factor. The wave height is the only parameter in the effectivity class. Therefore it has the highest weight of all parameters. Its group weight is twice as high as for the Structures, Soil and Habitat groups.

As the Soil group consists of the upper 2.5 metres and upper 5 metres clay content this group is split into two, in which the upper 2.5 metres is weighted twice as much as the upper 5 metres clay content. Then only the Size group remains. As this is of least importance its weight factor is one third of the weight factor of the wave height. The Size group is then split into three parts in which both length and width have an equal weight factor and the average width has a weight factor of two. This leads to following factors as shown in Table 4.

Table 4 Weight factors

Parameter		Individual weight	Group weight
Size	Length	1	4
	Width	1	
	Average width	2	
Structures	Structures	6	6
Soil	Upper 2.5 metres	4	6
	Upper 5 metres	2	
Habitat	Habitat	6	6
Wave height	Wave height	12	12

In the sensitivity check three new combinations of weight factors next to the original one, mentioned above, are examined. These combinations are determined from the point of view of different stakeholders. The first of the three combinations is based on the environmental and ecological view. In this combination the weight of the soil parameter is doubled and the weight of the habitat parameter is tripled. The next combination is based on the current occupants of the structures on the inside of the dike. Therefore, in this combination the weight of the structures parameter is tripled. The final combination is based on the water safety aspect. So, in this combination the weight of the wave height parameter is tripled.

Table 5 Weight factors for all combinations

Parameter		Original		Environmental		Structure		Water safety	
Size	Length	1	4	1	4	1	4	1	4
	Width	1		1		1		1	
	Average width	2		2		2		2	
Structures	Structures	6	6	6	6	18	18	6	6
Soil	Upper 2.5 metres	4	6	8	12	4	6	4	6
	Upper 5 metres	2		4		2		2	
Habitat	Habitat	6	6	18	18	6	6	6	6
Wave height	Wave height	12	12	12	12	12	12	36	36

3.2. Conceptual model

The goal of this conceptual model of the longitudinal mound is to analyse how different design and local parameters influence the hydraulic load on the river dike. This is then compared to the volume of soil needed to construct the longitudinal mound. The conceptual model is applied on five different locations. In Figure 13 this process is shown visually.

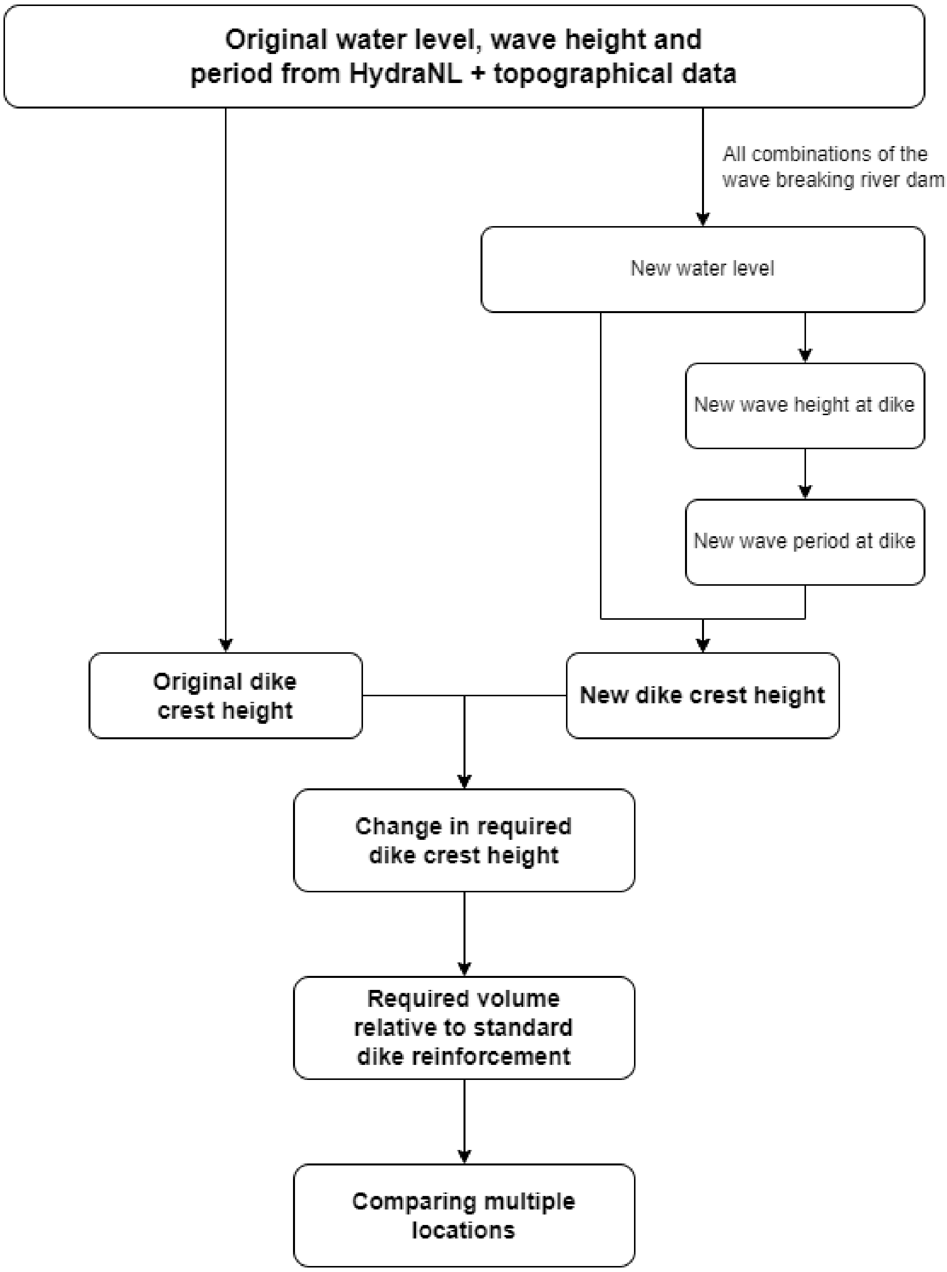

Figure 13 Flow chart for the conceptual model

3.2.1. Determining dike crest height without longitudinal mound

The required crest height of the river dike without longitudinal mound needs to be known. Then the dike crest height with the longitudinal mound has to be determined. When that has been done the effect of the longitudinal mound on the dike crest level can be determined. The determination of the required crest height can be found by following the GEKB (Grasbekleding Erosie Kruin en Binnentalud, Eng: Erosion grass cover crest and inner slope) failure path as prescribed in Appendix iii of the *Regeling veiligheid primaire waterkeringen 2017* (Rijkswaterstaat, 2017). In the GEKB a maximum allowable overtopping is given based on the allowable erosion of grass on the inner side of the dike. This overtopping is determined by the dike crest height.

First, the design return period has to be determined. The design return period depends on the return period per dike section and the *failure chance budget* (Dutch: faalkansbegroting). The dike sections in this analysis are classified with a return period of 1:3 000, 1: 10 000 and 1:30 000 years. These return periods are based on a combination of multiple failure mechanisms. The GEKB failure mechanism takes a *failure chance budget* of 0.24. Therefore the return periods used for this calculation is the return period of the dike section divided by the *failure chance budget*. That means the return periods for the GEKB failure path are 1:12 500, 1:41 667 and 1:125 000 respectively.

With these return periods HydraNL can calculate the required crest level based on the maximum allowed overtopping in litres per second per metre [l/s per m]. In the calculation for dike crest level the slope of the dike is necessary. In this research the effect of the longitudinal mound on the necessary dike crest height is examined. Therefore, to compare the dike crest heights with and without longitudinal mound the same dike slope is assumed in all calculations. A slope of 1:3 can be considered as a slope commonly used along the Dutch river dikes and has been used for the calculations of all crest levels.

3.2.2. Variables for the longitudinal mound concept

In the design for the longitudinal mound multiple design parameters are to be taken into account. These parameters determine the profile of the longitudinal mound on the floodplain and its roughness and are as follows:

- Crest height ‘*h*’
- Crest width ‘*w*’
- Slope ‘*S*’
- Roughness ‘*C*’

In Figure 14 a visual representation is given. The lateral slope of the floodplain is not taken into account. This is because the slope of the floodplain is assumed to be constant over its width, if this is the case the lateral slope of the floodplain does not influence the volume that is needed to construct the longitudinal mound, as long as the crest height of the longitudinal mound is taken from the middle of the longitudinal mound.

To determine the governing design water level and wave height in the original situation HydraNL is used. For this calculation a standard dike slope of 1:3 is assumed.

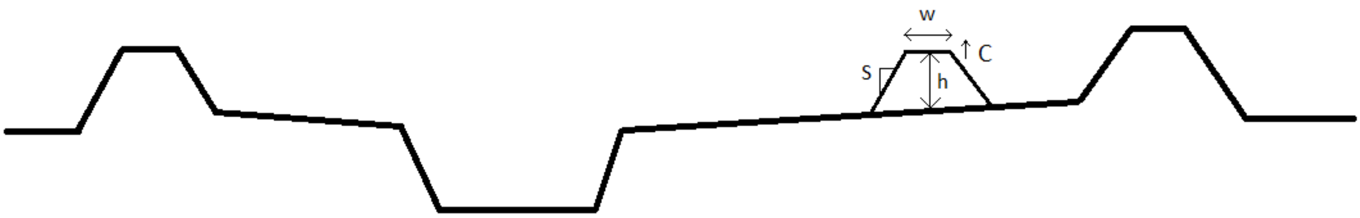

Figure 14 Design variables for the longitudinal mound

3.2.3. Water level increase

The next step is to calculate the required dike crest height when the longitudinal mound is in place. Therefore the water level with and without longitudinal mound needs to be calculated. In the calculation for the water level increase the assumption is made that the flow is steady and uniform. The water level without the longitudinal mound is calculated by HydraNL. In HydraNL the corresponding discharge is given as well.

The relation between water depth and discharge can be presented with a rewritten form of the equation for the equilibrium water depth for a steady river flow:

$$h_e = \left(\frac{q^2}{C^2 i}\right)^{1/3}$$

With h_e Equilibrium water depth [m]
q Specific discharge [m^2/s]
C Chézy coefficient [$m^{1/2}/s$]
i River bed slope [m/m]

In this case the flow has been split into three parts. Part 1 is the flow through the main channel, part 2 is the flow over the floodplains and part 3 is the flow over the longitudinal mound. However, in the original situation without longitudinal mound part 3 is non-existent and part 2 is the full width of the floodplain. With an average river bed slope of the Waal of $1.1*10^{-4}$ (Domhof et al. 2018) the Chézy coefficients are the only unknowns. Therefore, they are iteratively determined to ensure the relation between water level and discharge as determined in HydraNL is maintained. To do this the assumption is made that the Chézy coefficient on the floodplain is a factor 1.5 times smaller (more rough) than in the main channel. In Figure 15 the schematized river model used in the calculation is shown.

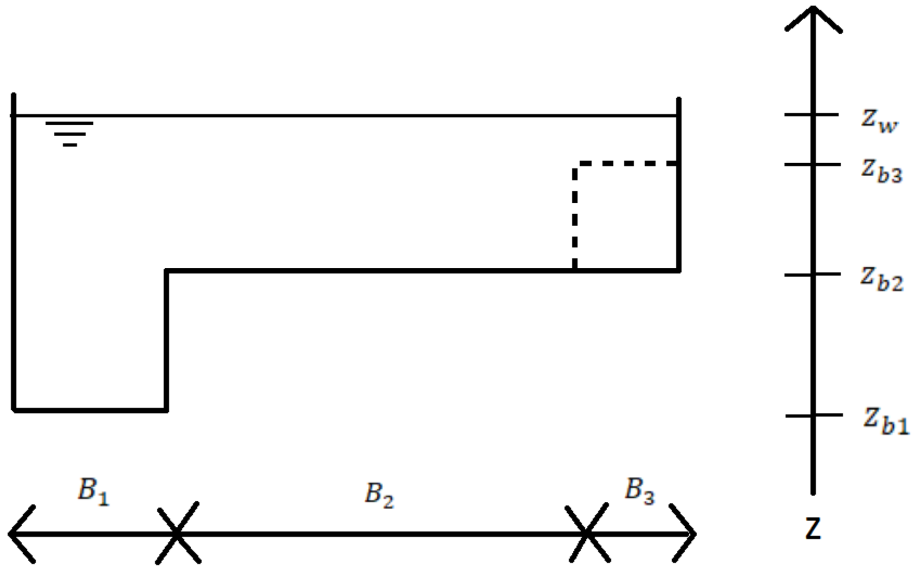

Figure 15 Schematized river model as used in the conceptual model

With the Chézy coefficients of the main river an floodplain known the third part can be added to the equation. The width and height of the third part depend on the three parameters that determine the shape of the longitudinal mound. This also reduces the size of the second part. The new water level after construction of the longitudinal mound is the only unknown in the equation. The equation can now be solved iteratively.

$$Q = B_1 C_1 \sqrt{i}(z_w - z_{b1})^{3/2} + B_2 C_2 \sqrt{i}(z_w - z_{b2})^{3/2} + B_3 C_3 \sqrt{i}(z_w - z_{b3})^{3/2}$$

3.2.4. Wave height reduction

Subsequently, the reduction of the wave height and the spectral wave period need to be determined. The reduction of the wave height over the longitudinal mound is determined by the empirical formula of Friebel and Harris (2003). This empirical formula is made as a best fit on 5 data sets from experiments performed in the 1980s and 1990s. For more information see Chapter 2. Most input parameters of this formula are determined by the design of the longitudinal mound, the HydraNL results used as input for the conceptual model or a combination of the two. Only the local wave length is not calculated by HydraNL. With linear wave theory it is possible to calculate the wavelength.

With all parameters known the formula of Friebel and Harris can be filled in. However there are five combinations of variables that determine if the formula is valid for the input parameters. Therefore a check for these five combinations of parameters has been performed as well.

3.2.5. Wave period

To determine the transmitted spectral wave period the maximum value of the transmitted wave period over the incoming wave period is assumed to be 1, and a linear relation is assumed between this ratio and the relative freeboard. However, there is also an influence from the wave steepness. In Figure 16 three band widths of wave steepness are given. To be on the conservative side of the reduction of the wave period, the linear relations from Table 6 are used.

Table 6 Conservative linear relations between the reduction of the spectral wave period and the relative freeboard.

Wave steepness S_0	Relative freeboard = 0	$T_{0,2t}/T_{0,2i} = 1$	$T_{0,2t}/T_{0,2i} = a * R_c/H_{m0i} + b$
$S_0 < 0.040$	$T_{0,2t}/T_{0,2i} = 0.70$	$R_c/H_{m0\text{-}I} = -2.0$	a = -0.150 b = 0.70
$0.040 < S_0 < 0.064$	$T_{0,2t}/T_{0,2i} = 0.80$	$R_c/H_{m0\text{-}I} = -1.5$	a = -0.133 b = 0.80
$0.064 < S_0$	$T_{0,2t}/T_{0,2i} = 0.85$	$R_c/H_{m0\text{-}I} = -1.0$	a = -0.150 b = 0.85

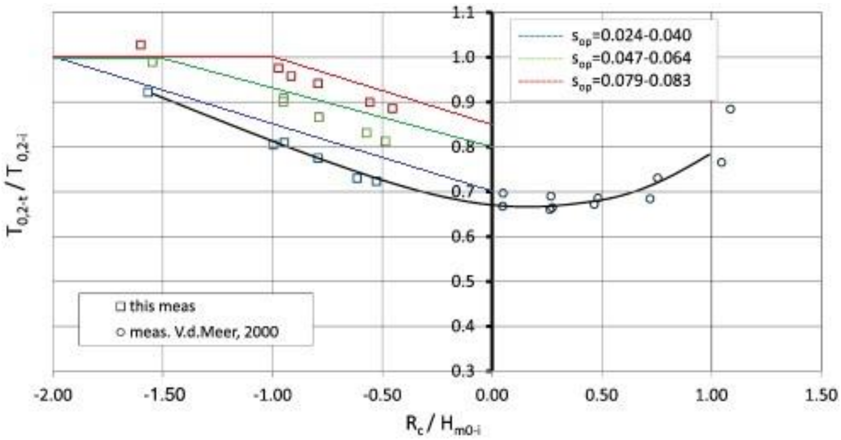

Figure 16 Reduction of spectral wave period based on the relative freeboard (Carevic et al. 2013). Includes conservative approximations based on the wave steepness.

3.2.6. Reduction of dike crest height with longitudinal mound

With the water level increase and wave height reduction over the longitudinal mound it is possible to determine the required dike crest height. To do this the Van der Meer wave overtopping formula is used (TAW, 2002). The Van der Meer wave overtopping was previously also used in HydraNL to calculate the original crest height. More about the Van der Meer wave overtopping formula is found in Chapter 2.

The reduction of the dike crest height depends on the new water level and the required freeboard as calculated by the Van der Meer formula. The inputs for the Van der Meer formula are the transmitted wave height and wave period from the longitudinal mound.

Calculating de required dike crest height for many combinations of possible geometries of the longitudinal mound gives an optimum design of the possible decrease of the required dike crest height. Although the reduction of the dike crest height is an important parameter, both monetary and ecological costs will increase when more soil is used. Therefore the volume of soil is taken into account as well. For a dike reinforcement soil is needed as well. For all possible geometries the volume of soil needed for the longitudinal mound is calculated. The amount of soil for a dike reinforcement of equal height as is reduced by the longitudinal mound is calculated as well. With these two volumes a ratio of volume needed for the longitudinal mound relative to an equal dike reinforcement can be calculated.

3.2.7. Multiple locations

To analyse the results the calculations have been performed for multiple locations. The locations that are selected all have different properties. The properties that are of importance in this stage are the width of the floodplain and the expected wave height. Also the selected locations are all from different river stretches. The design return period and the direction of the exposed dike face are given in Table 7.

Table 7 Selected locations for the conceptual model

Location	Floodplain width	Wave height	Direction of exposed dike face	Design return period
Waal_07	Medium	High	NW	1:30 000
Lek_05	Narrow	Medium	N	1:30 000
Waal_09	Medium	Small	S	1:10 000
Rijn_01	Wide	High	N + W	1:30 000
Maas_06	Narrow	Small	O + N + W	1:3 000

With these locations a wide range of different floodplains are taken into account. With the information provided by the different results it is possible to conclude in which cases the longitudinal mound performs better.

3.3. 2D D-Flow FM model

The 2D D-Flow FM model focusses on one location. This location is chosen based on the results of the location study and the conceptual model. Therefore location Waal_07 has been chosen, as it ended high on the ranking in the location study and is used as location for the conceptual model.

The goal of the 2D D-Flow FM model is to determine the effects on the water depth more accurately than is done in the conceptual model. Therefore a 2-dimensional hydrodynamic calculation was executed with D-Flow FM. In the conceptual model the water depth is based on a schematized cross section. In this calculation the equilibrium water depth is calculated. With D-Flow FM the water depth is calculated on a 2D grid with cell sizes of a maximum of 20 by 20 square metres. The steps used to perform this study are shown in Figure 17 below.

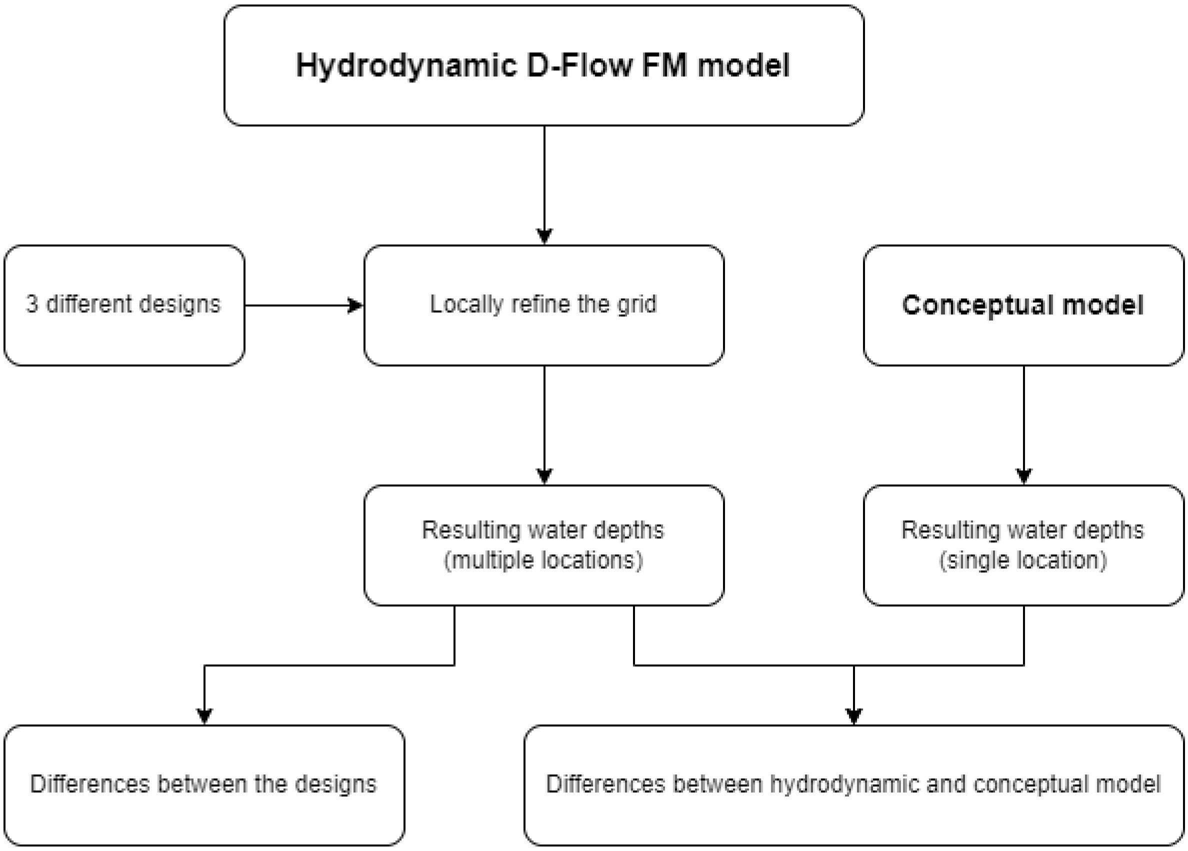

Figure 17 Flow chart for the D-Flow FM model

3.3.1. Setup of 2D model of the Waal

To find the more accurate effects on the water level the choice is made to use a 2D hydrodynamical model. With this model it is possible to determine water levels at multiple locations along the floodplain. Also at locations upstream of the longitudinal mound some effect on the water level is to be expected. This 2D model simulates the entire Waal river. As a result of this it is possible to monitor locations further away from the floodplain itself.

The computational grid and bed topography for the model are provided by Deltares/Rijkswaterstaat. At the upstream boundary a timeseries for the discharge is needed. In this case the timeseries consists of a constant discharge over the entire simulation period. For the downstream boundary a timeseries for the water level is needed. This water level is also constant over the entire simulation period and depends on the upstream discharge. The upstream discharge is determined by the return period. This discharge is equal to the discharge used for Waal_07 in the conceptual model. In the model the D-RTC module is active as well. The D-RTC module controls the hydraulic structures along the river.

3.3.2. Local grid refinement

To make the different variants the local grid and bed topography had to be adjusted. A resolution of 20x20 square metres is not accurate enough to accommodate the longitudinal mound in the model. Therefore, at the floodplain where the longitudinal mound is modelled the resolution of the grid is increased to 5x5 square metres. This is done with the CellsAndFaces2 tool within RGFGRID (Deltares, 2020b). With this resolution it is possible to model the new topography resulting from the longitudinal mound more accurately. However, due to limitations in the process of creating the new topography on the floodplain the slope of the longitudinal mound has been reduced from 1:3 to 1:4. Not only the topography is grid-dependent. Also the vegetation is grid-dependent, so those files have to be updated as well.

3.3.2.1. Bed topography

Changes to the topography can be done directly in the GUI of D-Flow FM. After refining the grid every old grid cell is changed into 16 new grid cells, while only one of those has a bed level value. The other points are determined by interpolation with the built in Delauney triangulation method (Deltares, 2020d).

The new topography is different for each variant. This is done with the built-in spatial operations in the GUI. First the *Set Value* tool is used to set the crest height. Then with the *Gradient* and *Smoothing* tools the slopes are set so that the slope is about 1:4 and the crest width is about 10 to 15 metres. See Figure 18 for an example cross-section of the longitudinal mound.

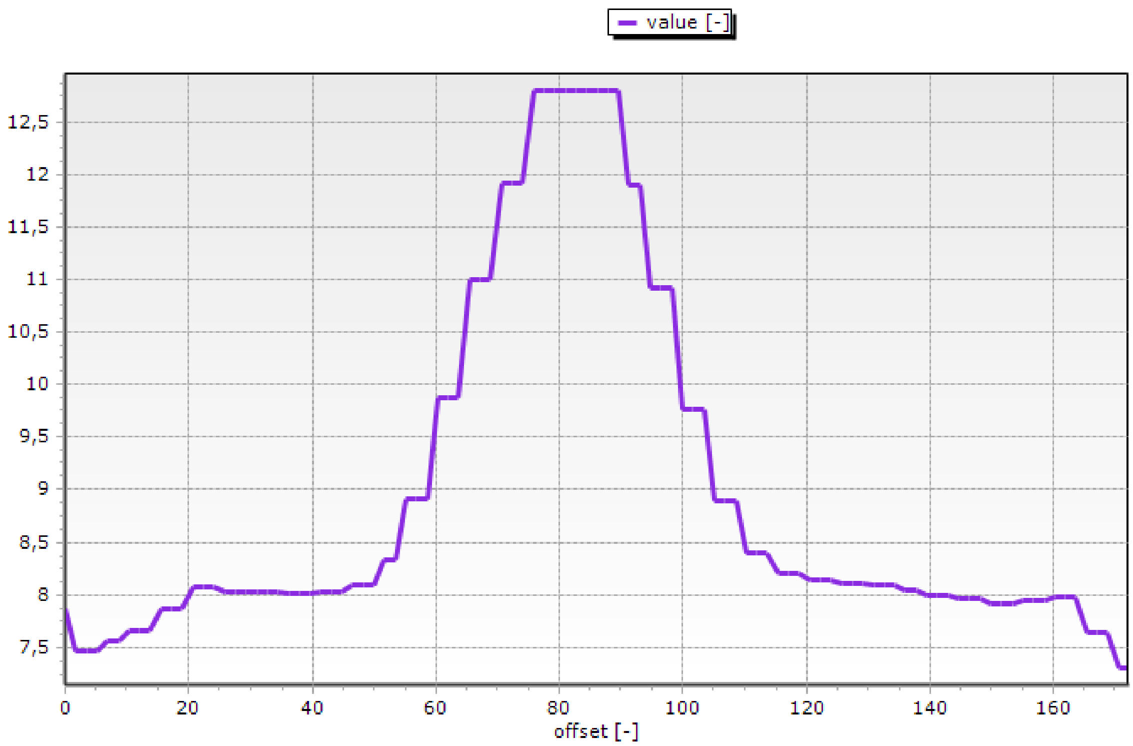

Figure 18 Cross-section of the longitudinal mound in the middle of floodplain Waal_07

3.3.2.2. Trachytopes (vegetation)

The vegetation is split over two files (.arl and .cll). These files cannot be changed directly in the GUI. Therefore, the values and coordinates for the 16 new cells per old cell have to be determined by using a script. The values used for the new cells are the same as for the single old cell. However, the new coordinates have to be linked with the old coordinate to assign the correct value. This is done with a nearest-neighbour algorithm. After this the values of the old cells can be assigned to all 16 new cells that are closest to the old cell. This script is shown in Appendix E.

3.3.3. Different variants

After setting up the model multiple variants of the longitudinal mound have been tested with the model. To be able to assess the results also a base version without any changes to the floodplain was run. In all variants the mound had a slope of 1:4. The main differences between the variants were the net soil difference and partly closing off the front and back of the area between the longitudinal mound. All different variants are described in the following sections.

In Figure 19 below the area around Waal_07 is shown. The outer purple line indicates the dike. It can be seen that on the inside of the dike also a bottom level is shown. This area is also shown in the figures with the variants. In the figures with the variants the dike is shown in black.

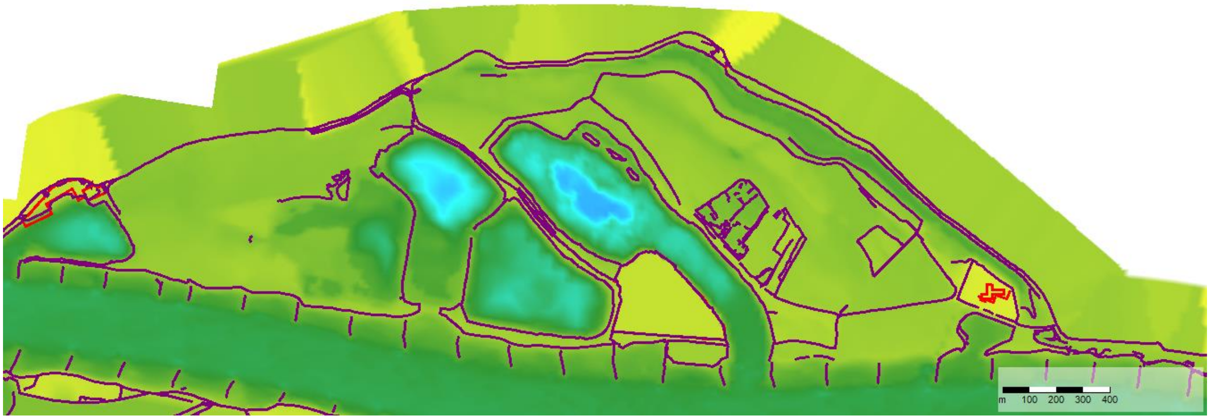

Figure 19 Area of Waal_07 with the bottom profile and with the D-Flow FM fixed weirs in purple

3.3.3.1. Variant 0 – Current situation

In the current situation the water levels without any intervention are calculated. Therefore the base model is used. This calculation is necessary to be able to quantify the results of the models where the effects of an intervention are calculated. In Figure 20 the topography of variant 0 is shown.

3.3.3.2. Variant 1 – Longitudinal mound with 5 metre crest height

From the conceptual model it was determined that the maximum decrease of necessary dike crest level is achieved for the highest longitudinal mound. However, it is chosen to round the crest height to the nearest metre. This means the crest height is about half a metre below the maximum water level. Therefore, this variant consists of a longitudinal mound with a crest height of 5 metres above the original level of the floodplain. The crest width of the longitudinal mound is kept between 10 and 15 metres over the entire length. In Figure 21 the topography of variant 1 is shown.

3.3.3.3. Variant 2 – Longitudinal mound with 5 metre crest height and a semi-closed-off area between the dike and the longitudinal mound

In this variant the base longitudinal mound as in variant 1 is used, in which the longitudinal mound is placed on top of the original floodplain. However, for this variant the dike and the longitudinal mound are connected to each other by a soil body with a crest height of 2.5 metres. This causes an area on the floodplain between the dike and the longitudinal mound which only floods with higher water levels than the crest height of this soil body. During design conditions the flow area is restricted more than in variant 1. On the other side when the water level is coming down after the flood, this closure results in a retention of water between the dike and the longitudinal mound. In Figure 22 the topography of variant 2 is shown.

3.3.3.4. Variant 3 – Longitudinal mound with 5 metre crest height and a net soil volume of 0

In the previous variants the longitudinal mound is placed directly on the existing floodplain. However, if local soil from the floodplain itself is available to construct the longitudinal mound, the cross-sectional area where flow occurs remains equal. The design parameters of the longitudinal mound are the same as in Variant 1, but the same volume of soil as necessary for the longitudinal mound is taken from the floodplain. This is done by lowering the area between the river channel and the longitudinal mound by 0.3 metres. This reduction of bed level about equals the volume needed for the construction of the longitudinal mound. In Figure 23 the topography of variant 3 is shown.

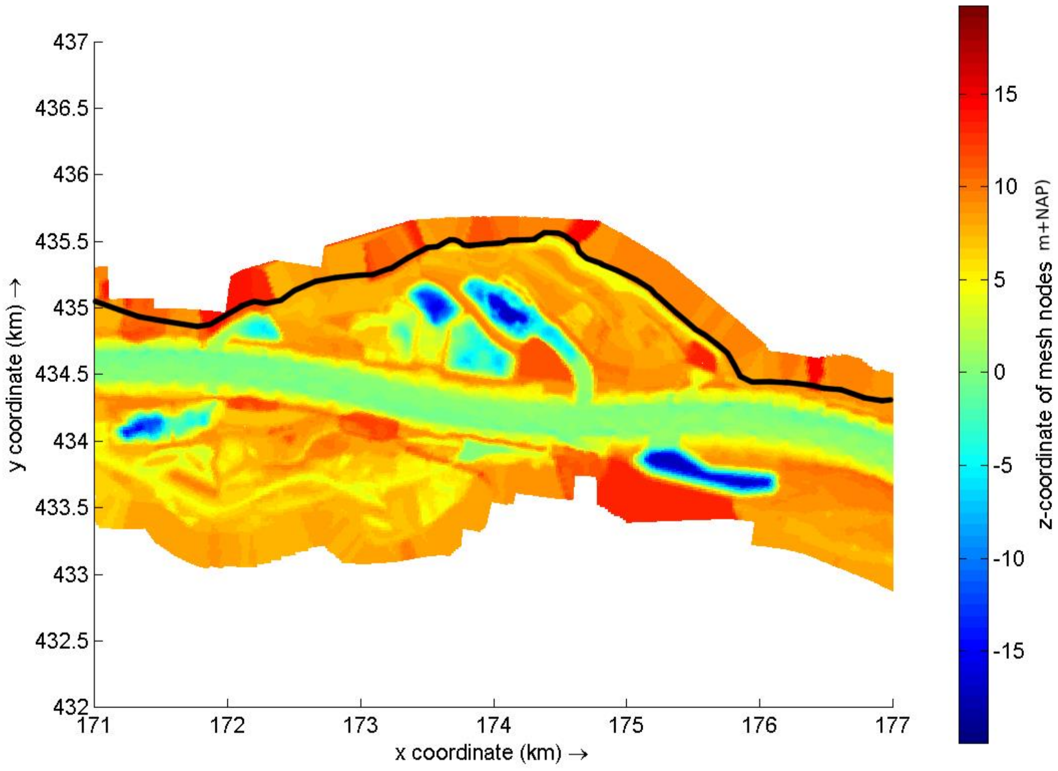

Figure 20 Topography of variant 0

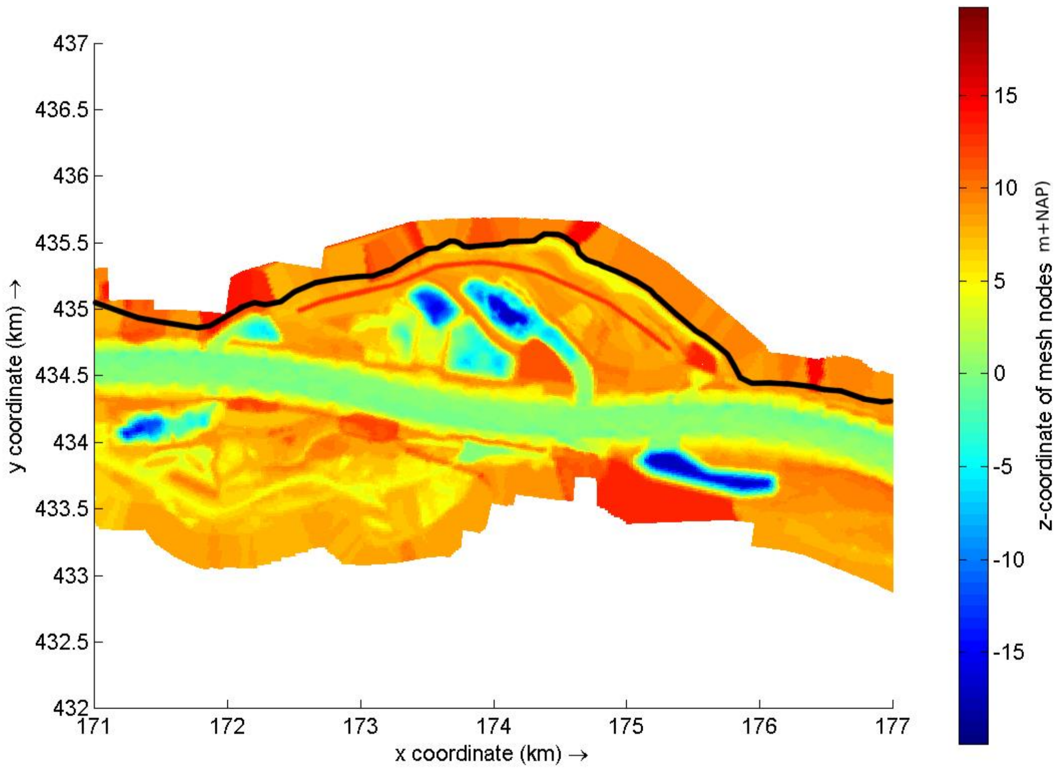

Figure 21 Topography of variant 1

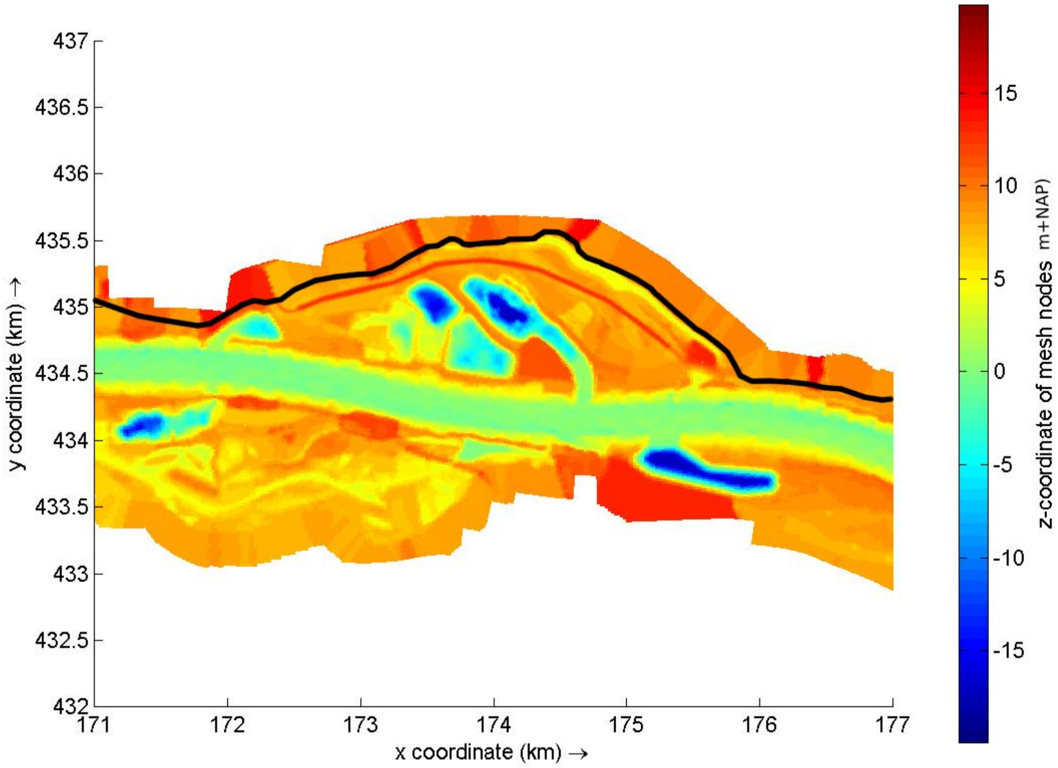

Figure 22 Topography variant 2

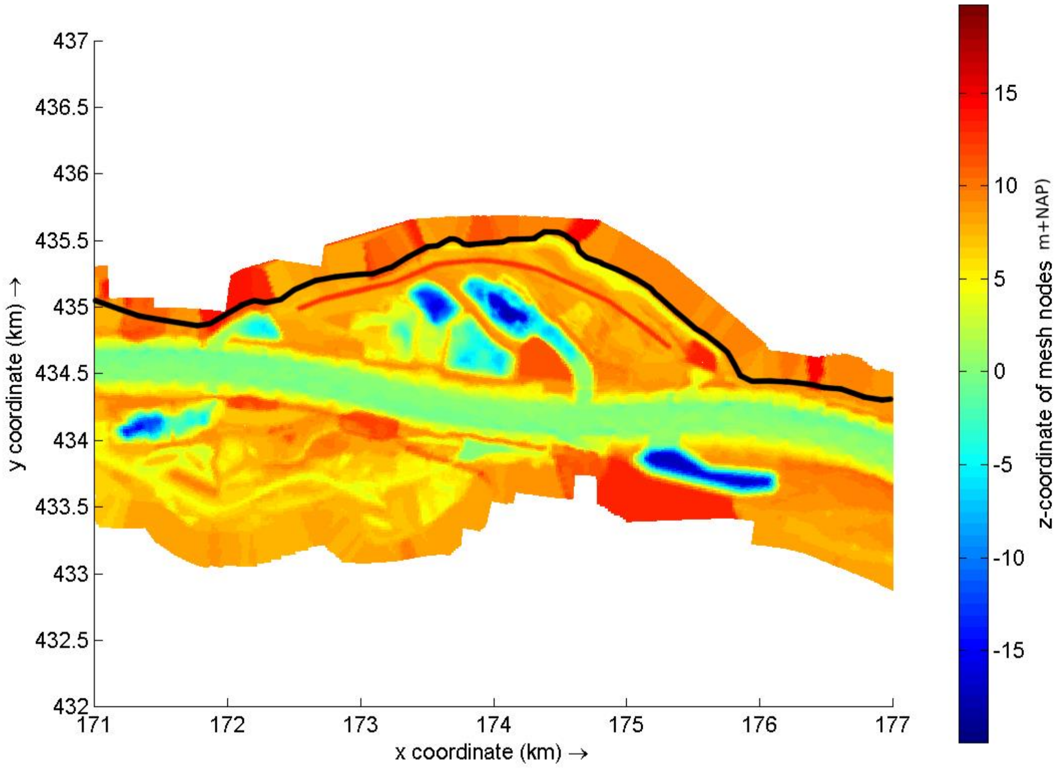

Figure 23 Topography variant 3

3.3.4. Influence on water levels

With the results of these three different designs of the longitudinal mound an insight can be obtained into how they influence the water levels locally and at a distance from the longitudinal mound itself. An increase of water level upstream of the longitudinal mound as a result of a backwater could result in a higher required dike crest height upstream of the longitudinal mound. This could nullify the potential benefit of the longitudinal mound.

3.3.5. Comparison to conceptual model

The results from the water level increase of the hydrodynamical model can be compared to the calculated water level increase in the conceptual model. However, the calculations done before with the conceptual model have some slight differences with respect to the situations used in the variants in D-Flow FM. Therefore, the conceptual model is run again with parameters that are more in line with the parameters used in the hydrodynamic model.

In the conceptual model only the local water level is calculated, however this value is also calculated in the hydrodynamical model. If these calculated water levels are comparable a case could be made that the calculation with the conceptual model would be sufficient to give a first approximation of the water level after the longitudinal mound has been constructed.

The conceptual model is a simple model compared to the more complex hydrodynamical model. If the conceptual model is able to calculate the water level accurately and the hydrodynamical model is not needed this has a couple of advantages. Firstly, there is less data needed. In the conceptual model only a few parameters are taken into account, which is then used one-dimensionally. In the hydrodynamical model a more detailed two-dimensional topography is taken into account with a resolution of 20 metres. This leads into the second advantage, the calculation time of the conceptual model is much shorter. Therefore, it is possible to do many more calculations to find optimal designs. This then can be further worked out with the help of the hydrodynamical model.

4. Results

4.1. Location analysis

In the following chapter the results of the parameters for the location analysis are discussed. The scores are shown on a map with the following colours for scores:

- Green 5 points
- Light green 4 points
- Yellow 3 points
- Orange 2 points
- Red 1 point

In Appendix B an overview of all scores is given in a set of tables, where more details per location can be found. Each location is named after the river the floodplain is located on, with ascending numbering from upstream to downstream.

4.1.1. Size of the floodplain

In Figure 24 the results of the floodplain length are shown. The longer floodplains are generally more prevalent upstream of the river stretches. The locations at the Pannerdensch Kanaal are the longest. Also it can be seen that the floodplains of the Waal tend to be slightly longer than the floodplains along the Lek and Meuse.

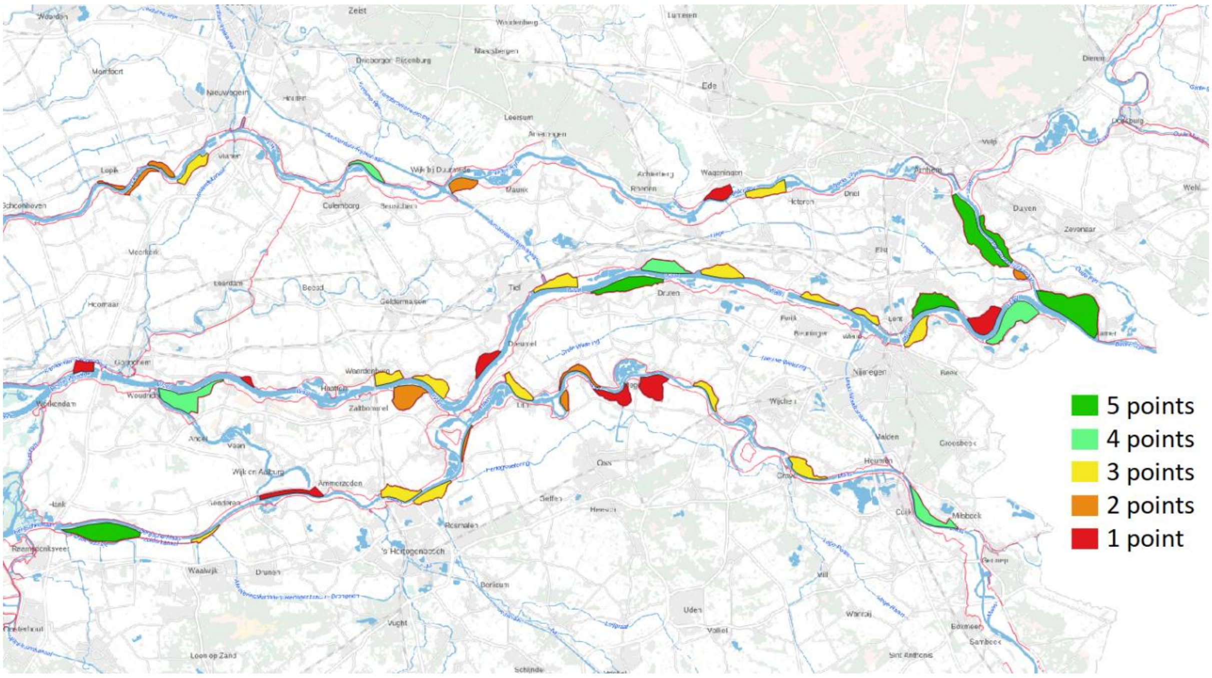

Figure 24 Results floodplain length

In Figure 25 the result of the maximum width of the floodplains is shown. Most noticeable are the relatively narrow floodplains at the Lek. Also for the maximum width the floodplains at the Waal tend to be the largest of the main three river stretches. In this case also the floodplain of the Boven-Rijn and the floodplains of the Waal just downstream of the Pannerdensche Kop are significantly wider than the other floodplains.

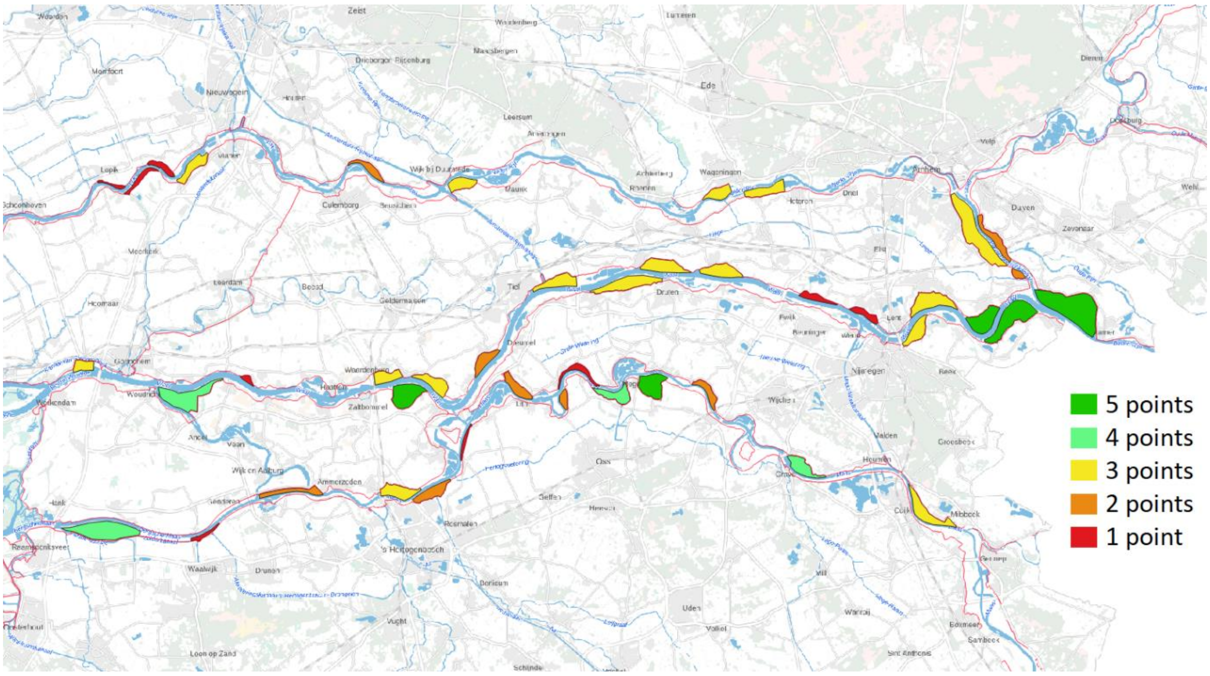

Figure 25 Results floodplain maximum width

In Figure 26 the results of the average width are shown. The results do not differ that much from the maximum width, as expected. However, looking at the average width of the floodplain does create some differences. The aforementioned three floodplains are still in the largest category, however more other floodplains are as well. Also it can be seen that some individual floodplains have moved up or down one point.

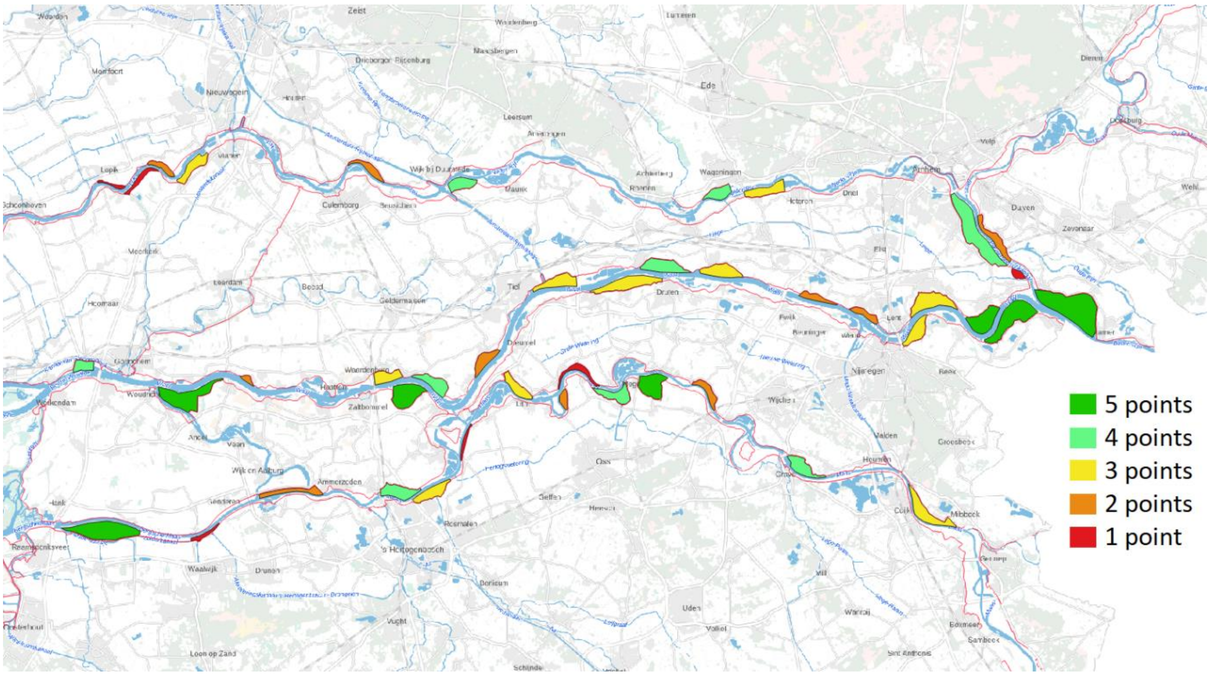

Figure 26 Results floodplain average width

4.1.2. Structures around the floodplain

The results of the structures are shown in Figure 27 below. The most noticeable result is that most locations score 4 or 5 points. This means that at most locations dike inwards expansion of the dike footprint is difficult. There are some areas where the lower scores are grouped together, mainly around 's-Hertogenbosch and Nijmegen. Although it is expected that in urban areas more structures would put a constraint on the possible inside dike expansion, just outside these urban areas there are less farms present along the dikes. Therefore there is less blockage for inward dike expansion.

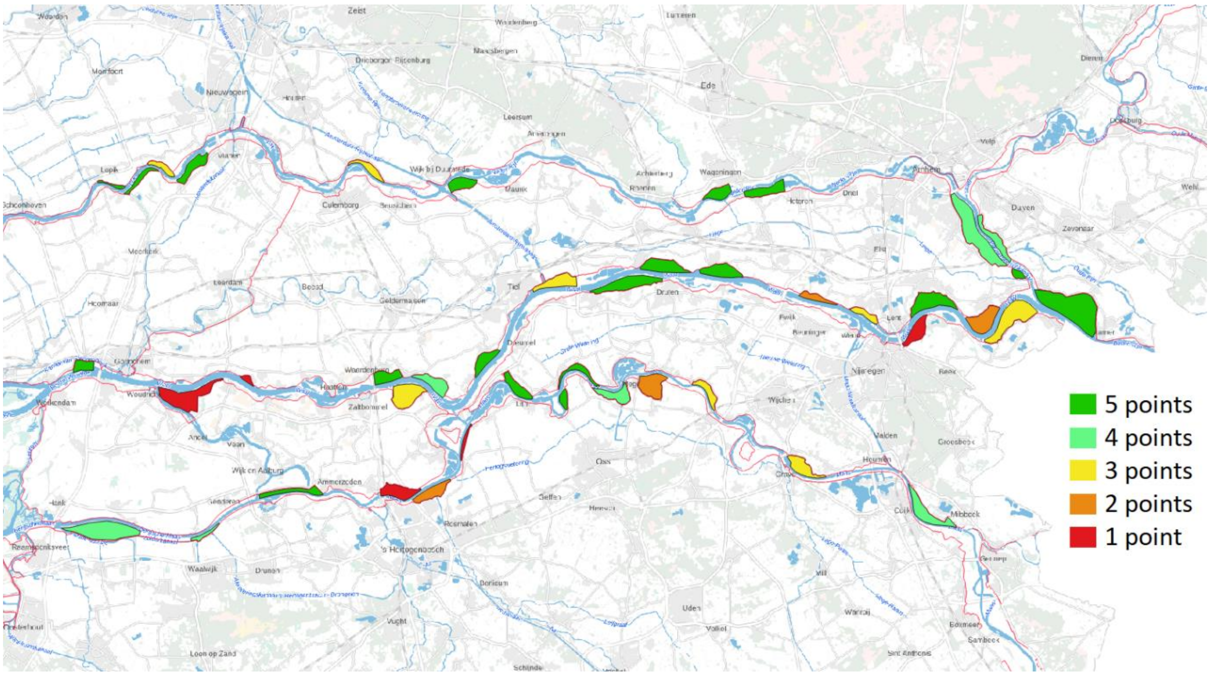

Figure 27 Results structures

4.1.3. Availability of local soil

The results are shown in Figure 28 and 29 below. As mentioned in Chapter 3 the percentage of clay in the upper 2.5 metre is of higher importance than the percentage of clay in the upper 5 metres. Looking at both the upper 2.5 metres and the upper 5 metres there are some differences. For most locations the percentage is in the same point bracket. However, the average points scored decreases from 3.10 to 2.95. At seven locations the score increased when going from a depth of 2.5 metres to a depth of 5 metres. At two of those locations it did with two points (Waal_08 and Waal_11). At fifteen locations the score decreased and only once it did with two points (Maas_01).

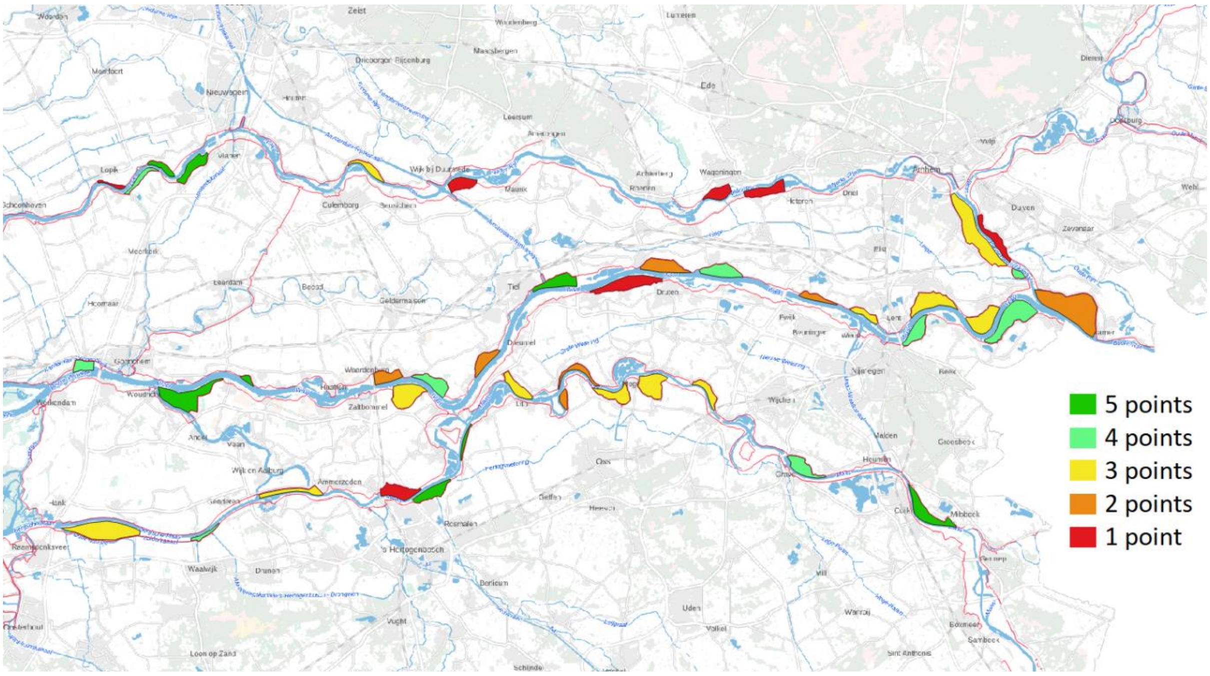

Figure 28 Results soil 2.5 metres

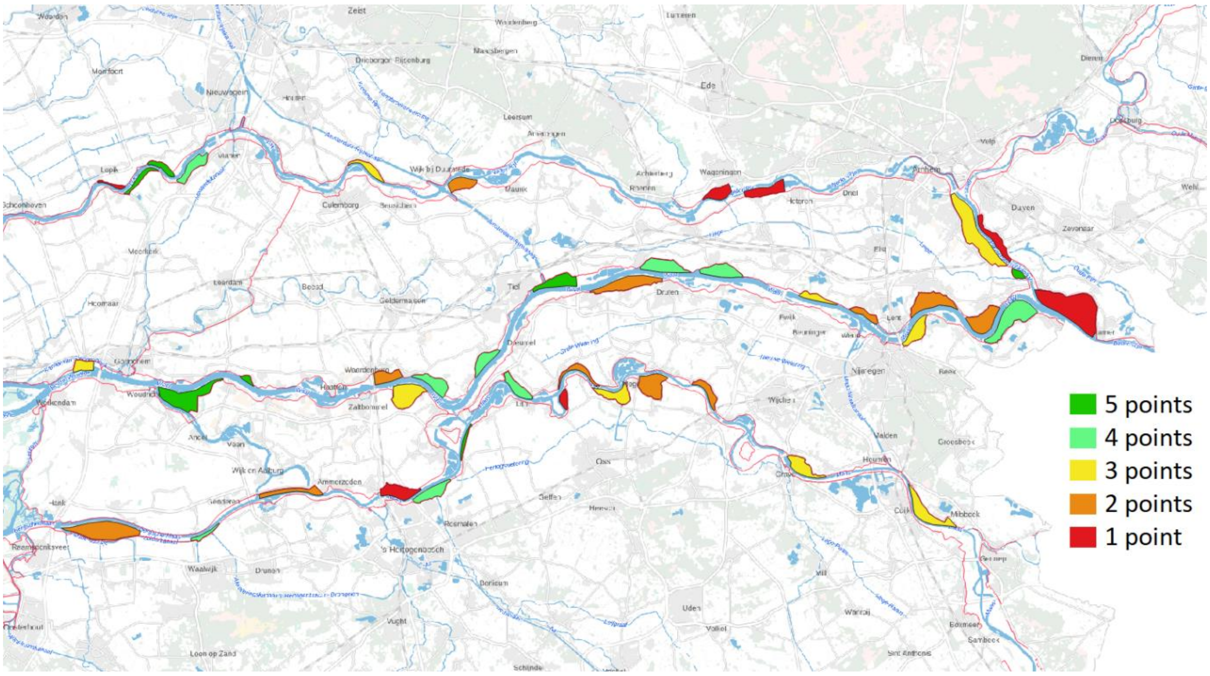

Figure 29 Results soil 5 metres

It can be seen that the more western locations on average tend to have slightly more clay than the locations that are further to the east, both in the upper 2.5 metres and the upper 5 metres. Although there are some exceptions like Waal_01 and Pannerdensch_Kanaal_01.

The locations where there is less clay in the upper 5 metres relative to the upper 2.5 metres tend to be more to the east. This indicates that the layer between 2.5 metres and 5 metres is less clayey in the east than in the west. The locations in the middle of the Waal are more likely to have more clay in the layer between 2.5 metres and 5 metres than in the upper 2.5 metres.

4.1.4. Current habitat

The results are shown in Figure 30 below. In Appendix B an extended overview of the Natura2000 areas and habitats on the floodplains is given.

Because the floodplains of the Meuse are not classified as Natura2000 the scores are higher than for the other rivers. However the floodplains of the Lek are mostly not Natura2000 areas as well and therefore also score relatively high.

For the locations where a Natura2000 area is present it could be seen that there seems to be a small bias for a relatively larger Natura2000 area in the eastern locations. However, this difference is relatively small. Therefore it is not likely that the location of the floodplains is of influence of this.

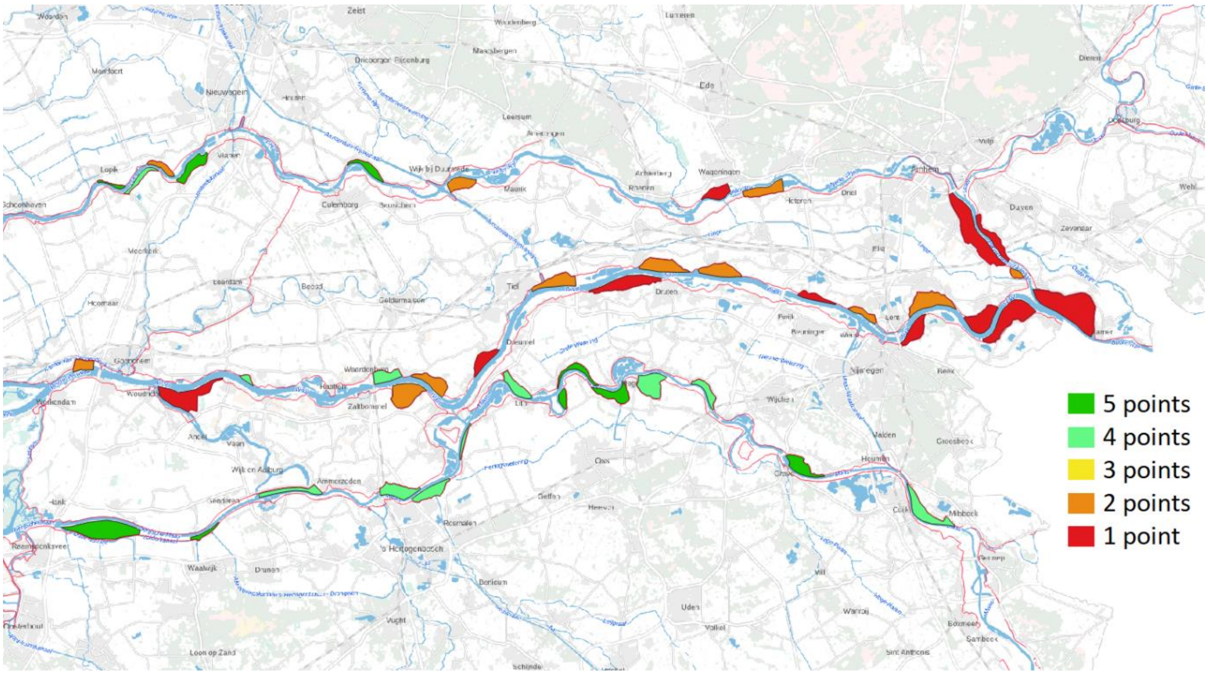

Figure 30 Results of the current habitat

4.1.5. Suitability

To determine the final score for all locations the points are multiplied for all parameters with their respective weight factors and are then added together. The final scores are shown in Table B-5 in Appendix B. All locations with a score of 65 points or higher are taken into account for the effectivity check. All locations with a score lower than 65 points are discarded.

This means that from the 43 initial locations 30 are selected for the next category and 13 of them are discarded. From these 30 locations relatively many are located along the Meuse. This is mostly due to the fact that the floodplains of the Meuse are not Natura2000 areas. Therefore the locations along the Meuse outscore the other river branches based on the habitat.

4.1.6. Expected wave height

After the calculation of the wave heights at every location at six locations the standard deviation between all data points was larger than 0.2 metres. These locations are shown in orange in Figure 31. That means these six locations have been awarded points based on further analysis. For the remaining locations there is a relatively even spread over the points. Although, 2 points are awarded more often and 1 point is awarded less often.

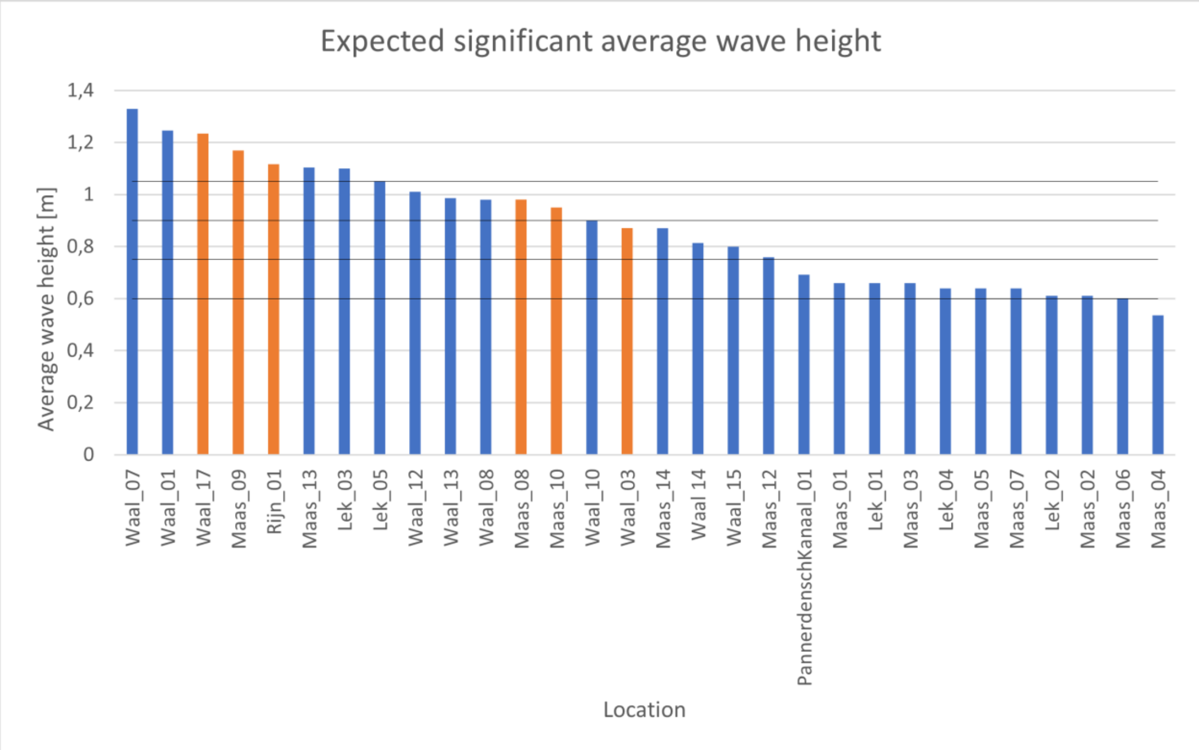

Figure 31 Expected significant average wave height with a return period of 1 in 1000 years

The locations with a high standard deviation have been investigated further. For Waal_03 at the eastern part there is an area with a significant wave height of 1.4 metres, but a larger area comprises of wave heights varying between 0.6 and 0.8 metres. The average wave height would be worth 3 points, but because of the higher wave height at the eastern part 4 points are awarded.

For Maas_10 there are relatively many data points between 0.95 and 1.15 metres. The lower extremes (0.65 m) are a bit more frequent than the higher extremes (1.45 m), however the lower peaks are located at the place where the dike is orientated parallel to the main wind direction. When omitting the high and low peaks the average is about 1.05 metres. Therefore it is difficult to determine the result. However, because of the original average still 4 points are awarded.

At Maas_08 in the middle of the dike the maximum wave height peaks at 1.6 metres. However, as this is only for a small number of data points the average would be about 0.9 metres when the biggest outliers are omitted. Therefore 4 points are awarded.

For Rijn_01 there are a couple of data points where the wave height is calculated to be in the order of 0.1 metres. Those data points have a big influence on the standard deviation. When these outliers are omitted the standard deviation decreases to below 0.2 metre. The remaining calculated wave heights satisfy the threshold for awarding 5 points. Therefore, Rijn_01 does get 5 points.

Maas_09 has a relatively high average, however there is not such a high peak as with the previously mentioned locations. Topography-wise this location should not have high wind waves based on the fetch. Therefore only 4 points are awarded even though the average would result in 5 points.

At Waal_17 there are larger waves at the eastern part, similarly as at Waal_03. In this case the length with larger waves at the eastern side is larger. Also the average was already high enough for 5 points. Therefore Waal_17 receives 5 points.

All in all it can be seen in Figure 32 that the wave heights on the upstream half of the Meuse are the lowest. At the downstream half of the Meuse the wave heights tend to be a bit higher, but not as consistently high as on the Waal. On the Lek the wave heights are quite inconsistent. However, this can be explained by the orientation of the floodplains. Higher waves are expected at the floodplains with a dike orientated northwest to southeast.

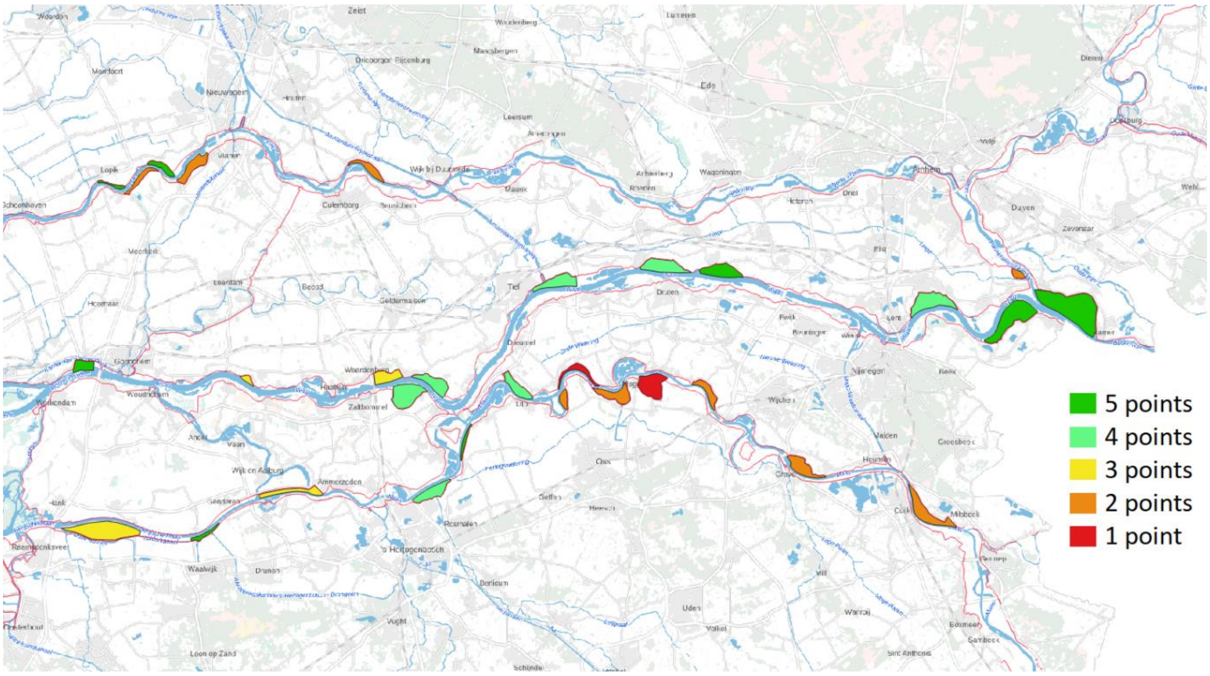

Figure 32 Results of the expected wave height

4.1.7. Final results

In Table B-6 in Appendix B a summary of all points is given for all locations that entered the effectivity round. In this table the total points from the suitability study, the points from the wave height and the total points are shown. Location Maas_13 has scored the most points in total.

In Figure 33 all locations, that are also shown in Figure 11, are given a rank. Dark green is the highest and given to the locations with more than 125 points. The locations with more than 115 and a maximum of 125 points are made light green, the locations with more than 105 and a maximum of 115 points are made yellow, the locations that have been taken into account after the suitability check and score less than 105 points are made orange and all locations that have been omitted after the suitability check are in red.

There is not a clear bias for certain rivers for the preferable locations, although there seems so be relatively more locations from the Waal that are green. On the flip side there are also more locations from the Waal that are red. This is caused by that most locations along the Waal do consist of Natura2000 areas. Even though relatively few locations from the Waal are used in the effectivity part of the study and there are relatively many green locations, this indicates that generally the wave height in the Waal is higher than in the other rivers. This corresponds to the results in Section 4.1.6.

The locations at the Lek also score high. Although these floodplains are relatively small they do score good on the other parameters, which are weighted higher. So, as there is not a location which scored high on all parameters there will always be trade-offs when choosing a location for a longitudinal mound. At different locations other factors could be more important than how the weights are given in this study. Therefore, in the sensitivity analysis other opinions on the weights are given based on the stakeholders.

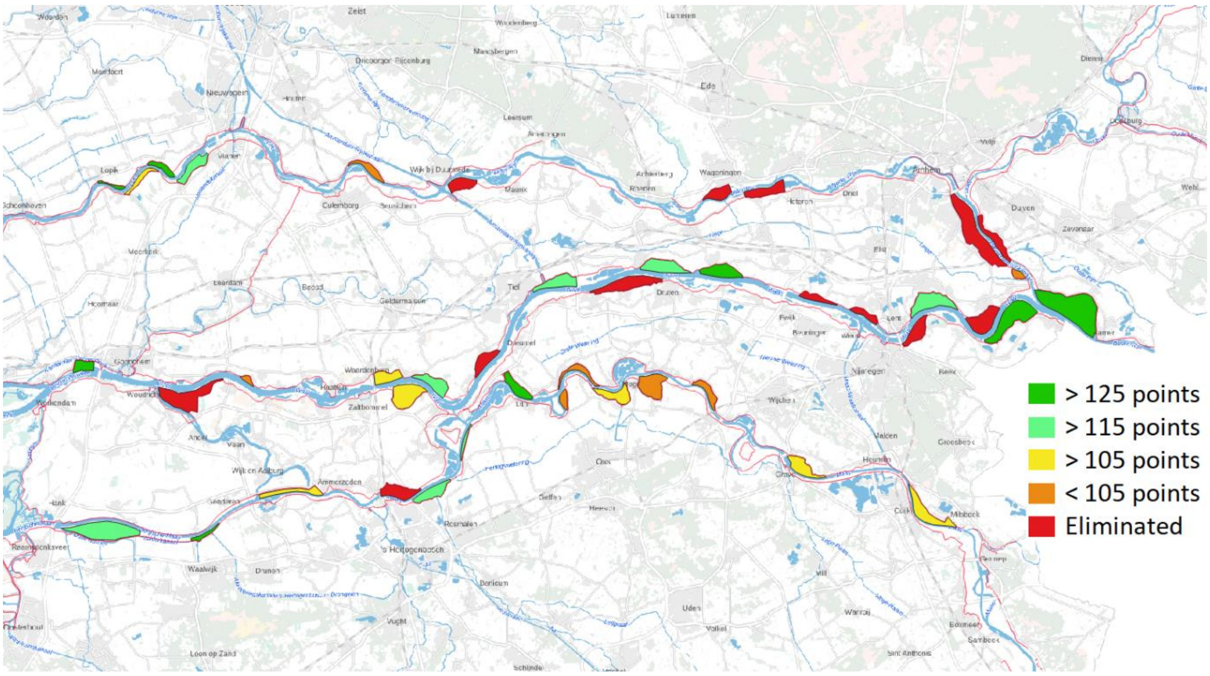

Figure 33 Final results of all locations

4.1.8. Sensitivity analysis

The sensitivity analysis has been performed with three new weightings from the point of view of different stakeholders in addition to the original Base distribution of weights. In Table 8 below the scored points are divided by the maximum possible points available. Therefore, the numbers in Table 8 represent the percentage of the maximum available points scored by each location. The locations in this table are sorted on the results of the Base distribution. In the following columns all other weight distributions are also sorted on the Base distribution. All columns are coloured based on their percentage of the maximum score of their respective distribution, in which the darkest green is the highest percentage and the darkest red the lowest percentage.

Table 8 Results of the sensitivity analysis

Location	Base	Ecological	Structures	Water safety
Maas_13	84.7	87.7	83.5	91.0
Waal_07	81.2	71.5	86.1	89.0
Waal_17	80.0	70.0	85.2	88.3
Maas_08	78.2	77.3	83.9	79.0
Lek_05	77.1	75.8	83.0	86.6
Lek_03	74.7	69.6	70.9	85.2
Waal_01	74.7	62.7	70.9	85.2
Rijn_01	74.1	56.9	80.9	84.8
Maas_09	73.5	78.1	59.6	84.5
Maas_14	73.5	77.3	75.2	67.9
Lek_02	72.9	81.5	80.0	59.3
Maas_10	72.4	76.5	63.9	75.5
Waal_12	71.8	65.4	73.9	75.2
Waal_08	71.2	61.9	78.7	74.8
Waal_03	70.6	61.5	78.3	74.5
Waal_10	70.6	66.9	67.8	74.5
Waal_14	67.1	66.9	75.7	64.1
Maas_12	66.5	68.1	75.2	63.8
Waal_13	66.5	59.6	64.8	72.1
Maas_01	65.3	71.2	69.1	54.8
Lek_04	64.1	70.4	73.5	54.1
Maas_02	64.1	73.5	63.0	54.1
Maas_05	64.1	71.9	68.3	54.1
Maas_07	60.0	66.2	70.4	51.7
Waal_15	60.0	69.2	49.6	60.0
Lek_01	58.8	68.5	59.1	51.0
Pannerdenschkanaal_01	57.6	56.9	68.7	50.3
Maas_03	53.5	59.6	55.2	47.9
Maas_06	52.4	61.9	64.8	39.0
Maas_04	47.1	55.4	45.2	35.9

This makes it possible to visibly assess if a location has increased or decreased its rank in the Ecological, Structures or Water safety distribution relative to the Base distribution. For example, in the column of the Ecological distribution Rijn_01 is red, while it is light green in the column for the Base distribution. This means that it scores relatively high on the Base distribution and fairly low on the Ecological distribution. Therefore, this location, Rijn_01, has a lower rank on the Ecological distribution than on the base distribution. For Maas_02 this is the other way around.

Results

In the Ecological distribution Maas_13 is still the location with the most points. Behind Maas_13 the locations along the Waal and Rijn_01 have decreased in their ranking. This is mainly due to the Natura2000 areas. This causes the locations on the Lek and Maas to increase their ranking. However, Maas_04 is still the location with the least points.

In the Structures distribution Waal_07 is the location with the most points. There are less changes relative to the Base distribution than there are for the Ecological distribution. The first 5 location are the same, only in a different order. However, there are a couple of locations, Lek_03, Waal_01, Maas_09 and Maas_10 that have decreased in rank. On the other side mainly location along the Waal have increased in rank.

The Water safety distribution has been mainly the same as the Base distribution. Although there are some locations that dropped a little bit, like Maas_08, Maas_14 and Lek_02. And on the other hand Waal_03, Waal_10 and Waal_13 have increased their rank slightly.

Because the Water safety distribution has most in common with the Base distribution, this could indicate that the Base distribution has a bias towards the water safety aspect. As the wave height parameter was weighted the highest of all parameters in the Base distribution this could be the case. On the other side, the effectivity only accounts for about 35% of the total score while the suitability accounts for 65%.

4.2. Conceptual model

The results of the conceptual model are based on the data of five locations with different characteristics. Location Waal_07 is discussed in detail and after that the results of the other locations are compared.

4.2.1. Crest height original situation

The original water level at location Waal_07 has been calculated with HydraNL and is 12.83 m+NAP. In combination with the design wave conditions and the Van der Meer overtopping formula a necessary dike crest height of 14.36 m+NAP is calculated.

4.2.2. Crest height with longitudinal mound

Calculating the necessary dike crest height with the longitudinal mound in place is done by determining the effect of the longitudinal mound on the water level, wave height and wave period.

4.2.2.1. New water level

For the calculation of the new water level the formula from Section 3.2.3. is used. The discharge is the same as it is in the original situation. For all combinations of the design parameters, longitudinal mound slope, crest height and crest width, there is a difference in B_3 and z_{b3} depending on the combination. With all other parameters known the new water level can be calculated. The resulting water levels are in the range of 12.831 to 13.134 m+NAP. With 12.83 m+NAP as the original water level, this is a maximum increase of about 0.30 metres.

4.2.2.2. New wave height

For the calculation of the new wave height the formula from Friebel and Harris is used. In this calculation the new water depth is used. The water depth is directly in the formula and also indirectly via the negative freeboard (water level over the crest of the longitudinal mound). For the incident wave height the original wave height is used. It is assumed the incident wave height does not change as a result of the increased water level.

The formula of Friebel and Harris is not valid for all combinations of the longitudinal mound, as mentioned in Chapter 2. Not all calculated variants are possible. Most notable is that the crest height of the longitudinal mound has to be a minimum of 0.44 times the water depth. *Because the water level was used in the conceptual model instead of the water depth the figures in this chapter show longitudinal mounds that do not conform to this limit. In Chapter 5 more information about this mistake can be found and the differences on the results are shown.*

The resulting wave heights range from 0.43 metre to 0.20 metres. With an original wave height of 0.57 metres, this is a reduction of 25% up to 65%. The 65% decrease is in the case of the largest longitudinal mound when the water level is just above the crest height of the longitudinal mound in combination with the maximum longitudinal mound crest length. Decreasing to 25% for the smallest longitudinal mound.

4.2.2.3. New wave period

Calculation of the wave period after passing the longitudinal mound is done by Carevic (2013). Based on the relative freeboard over the longitudinal mound and the wave steepness, the factor between the incident wave period and transmitted wave period can be determined. In most cases the factor is 1, which means no change in the wave period is assumed. For longitudinal mounds with a large crest height the relative freeboard is small enough to reduce the wave period.

4.2.2.4. New situation

With the new values of these three parameters known the van der Meer formula can be used again to determine the crest height of the dike with the longitudinal mound. The new crest heights are in the range of 13.46 to 14.10 m+NAP. With an original dike crest height level of 14.36 m+NAP this means that for Waal_07 a reduction of 0.26 to 0.90 metres is obtained with the longitudinal mound. The largest dike crest height reduction occurs for the largest longitudinal mounds.

4.2.3. Optimum design of the longitudinal mound

To determine the optimum design of the longitudinal mound the design water level is used. Therefore, the following results are for the maximum load situation. In Appendix D the figures for all 5 locations are shown.

Longitudinal mound slope

The effect of a shallower slope is a slight increase of dike crest height. This is because the flow becomes more constricted while the reduction in wave height is only effected indirectly by the increased water level.

Longitudinal mound crest width

An increased crest width of the longitudinal mound results in a small increase of the dike crest height reduction. In Figure 34 below, the dike crest height reduction of Waal_07 as a function of the longitudinal mound crest width is shown, in which the height of the longitudinal mound is set at the highest level for which the longitudinal mound is submerged, which for Waal_07 is 5.5 metres, and the slope is set at 1:3. It can be seen that the dike crest height reduction increases by about 10 centimetres between a longitudinal mound crest width of 4 metres and 20 metres. *This 10 centimetre difference is a result of the usage of the water level instead of the water depth. In Chapter 5 more information about this mistake can be found. The difference in dike crest height reduction should be around 25 centimetres between crest width of 4 metres and 20 metres.*

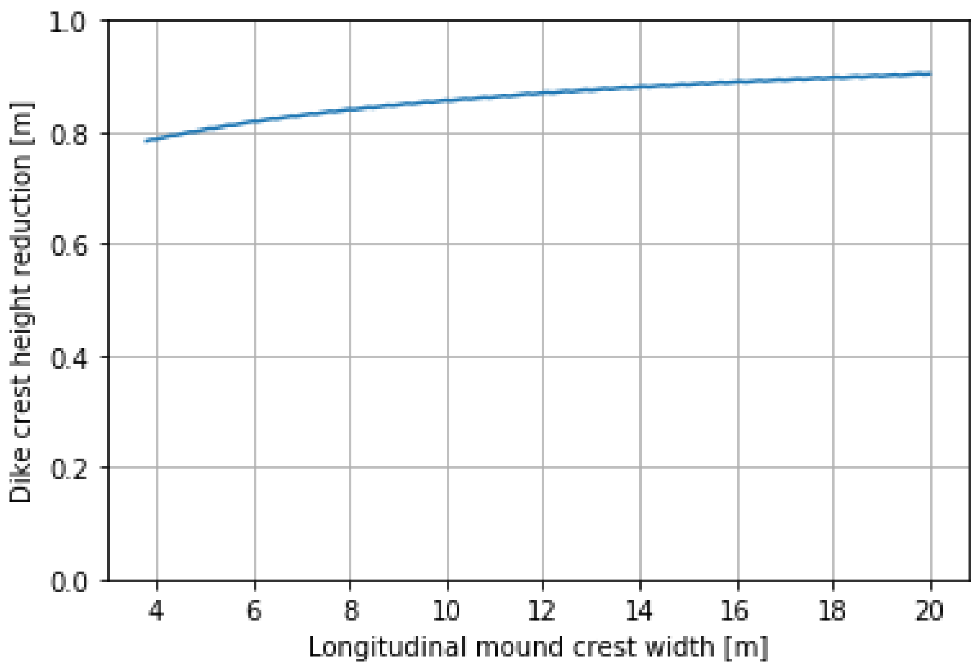

Figure 34 Dike crest height reduction based on crest width of longitudinal mound

Longitudinal mound crest height

A crest height of the longitudinal mound of 1 metre already gives a reduction of 0.3 metres in dike crest height. *This dike crest height decrease is a result of the usage of the water level instead of the water depth. In Chapter 5 more information about this mistake can be found and the differences on the results are shown.* When the negative freeboard above the longitudinal mound gets below 1.5 to 1 metre the dike crest height reduction starts increasing rapidly. This is shown in Figure 35 below, in which the crest width is set at 10 metres and the slope at 1:3.

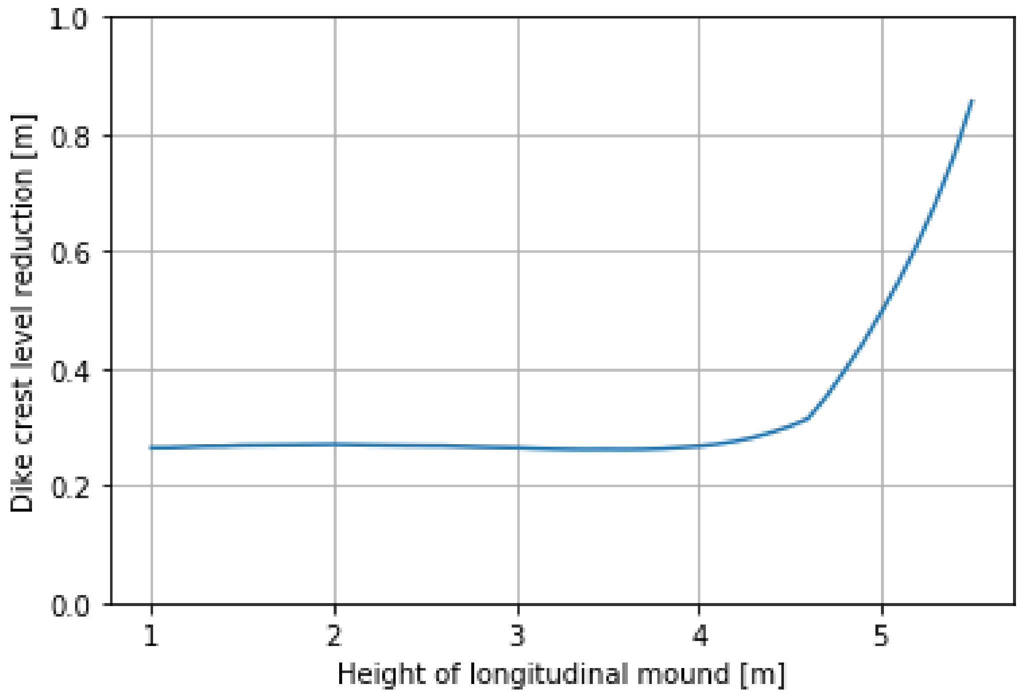

Figure 35 Dike crest height reduction based on crest height of longitudinal mound

The optimum design purely based on the reduction of dike crest height is therefore the highest mound with the largest crest width. In Figure 36 below a 2D plot is shown for every combination of crest height and length of the longitudinal mound and a slope of 1:3 which shows the decrease in dike crest level.

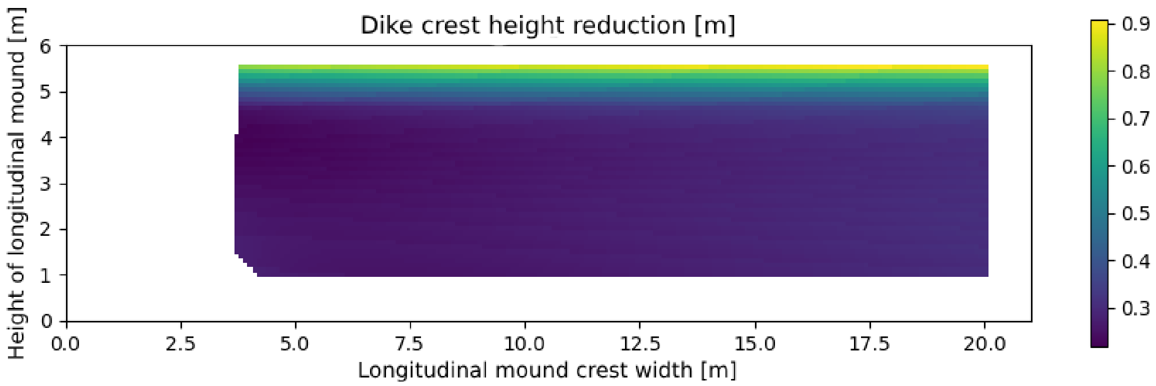

Figure 36 Dike crest height in metres for all combinations of width and length of the longitudinal mound

4.2.4. Optimum design with soil volume taken into account

The dike crest height reduction is not the only parameter on which an optimum design can be determined. Also the volume of soil used for the longitudinal mound can be taken into account. Volume is the third power of distance, therefore if the height or width of the longitudinal mound increases, the volume increases with a power of three. This results in a maximum dike crest height reduction per volume for the smallest longitudinal mounts. This is a direct contradiction to the optimum based on its dike crest reduction alone.

However, when the dike crest height is increased instead of a longitudinal mound is constructed, there is also the need of soil. Therefore, the volume of soil that is needed for a dike crest height increase is calculated. To compare the volumes of this dike crest height increase relative to the volume needed in the longitudinal mound, the dike crest height increase is assumed to be the same as the dike crest height reduction would be when the longitudinal mound is used.

Looking at the same designs as in Section 4.2.3. it can be seen that the volume ratio increases when the crest width of the longitudinal mound is increased, see Figure 37. The volume ratio when increasing the height of the longitudinal mound increases until it peaks at the height at which the crest height reduction starts increasing rapidly. Despite this decrease the minimum volume ratio is still at the lower end of the heights of the longitudinal mound, see Figure 38.

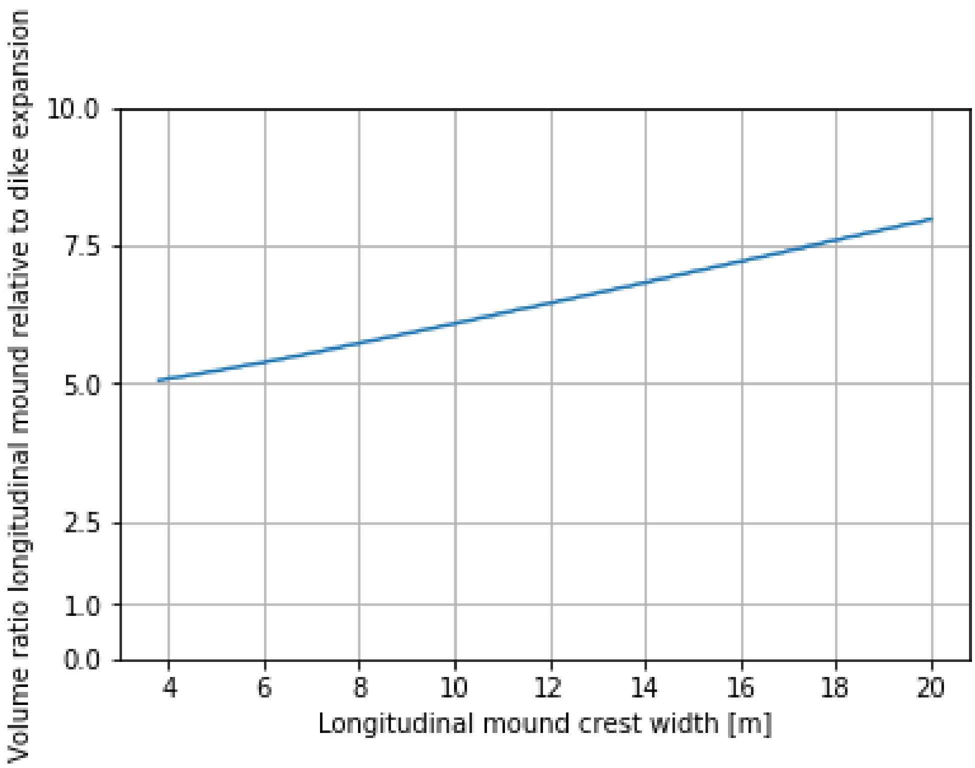

Figure 37 Volume ratio based on crest width of longitudinal mound

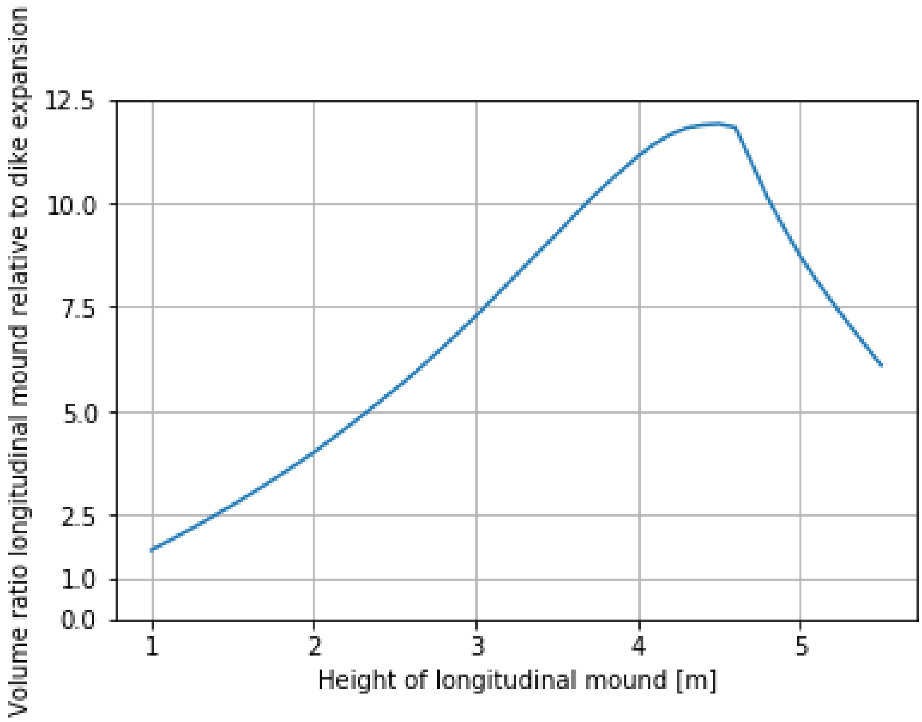

Figure 38 Volume ratio based on crest height of longitudinal mound

For almost all designs of the longitudinal mound this would increase the amount of soil used. In the case of Waal_07 this is for all designs. The ratio of soil volume used between the longitudinal mound and a traditional dike reinforcement has a maximum of 15 at location Waal_07. In Figure 39 below a 2D plot is shown for every combination of crest height and length of the longitudinal mound with a slope of 1:3 at location Waal_07.

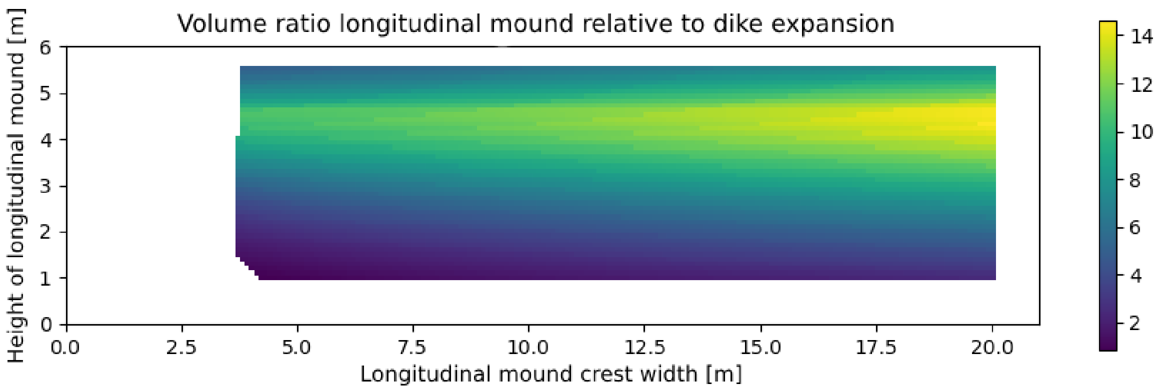

Figure 39 Volume ratio between longitudinal mound and dike expansion for all combinations of the longitudinal mound

4.2.5. Differences between locations

In the previous section only Waal_07 is taken into account. To see if the effects are the same in different conditions the same calculations are performed at four other locations as well. After calculating the wave heights with HydraNL for the design return period some wave heights are higher or lower than was expected in Chapter 3. This is a result of the coupling between the return period of water levels and wave heights in the HydraNL calculations. In Chapter 3 Waal_07 was expected to have high wave heights but ended up to be in the medium category. Lek_05 and Waal_09 were expected to have medium and low wave heights respectively but ended up to be in the high and medium category respectively. So in Table 9 below the revised table from Chapter 3 is shown, in which the floodplain widths and wave heights are given numerically.

Table 9 Selected locations with quantified floodplain width and wave height

Location	Floodplain width [m]	Wave height [m]	Direction of exposed dike face	Design return period
Waal_07	1000	0.58	NW	1:30 000
Lek_05	275	1.13	N	1:30 000
Waal_09	900	0.74	S	1:10 000
Rijn_01	1800	1.01	N + W	1:30 000
Maas_06	150	0.27	E + N + W	1:3 000

4.2.5.1 Maximum longitudinal crest heights

To compare the different locations with each other a standard design has been selected. This design has a slope of 1:3 and a crest width of 10 metres. The crest height of the longitudinal mound differs between the locations. The longitudinal mound crest height is set to the maximum crest height below the design water level measured from the floodplain level. The heights of the longitudinal mounds can be seen in Table 10 below. Also the water level increase, dike crest height decrease and the ratio of soil that is necessary for the longitudinal mound relative to a traditional dike expansion are shown in Table 10.

Table 10 Results for all 5 locations based on a longitudinal mound crest width of 10 metres and slope 1:3

Location	Crest height of longitudinal mound [m]	Water level increase [m]	Dike crest height decrease [m]	Amount of soil [x version dike]
Waal_07	5.5	0.23	0.86	6.1
Lek_05	3.1	0.11	2.12	1.3
Waal_09	4.6	0.16	1.23	3.4
Rijn_01	4.6	0.09	1.76	2.3
Maas_06	3.6	0.14	0.36	9.7

Water level increase

It is expected that the water level increase is higher in case of a smaller floodplain. This would follow from the fact that a larger percentage from the flow area is blocked. However, this influence is not seen when comparing the water level increase with the floodplain widths. This can, at least partly, be explained by the maximum height of the longitudinal mound. The maximum height of the longitudinal mound is based on the water level during the design return period and the floodplain level. Floodplains with a higher bed level have a lower flow capacity. Therefore, the water level increase on floodplains with a high bed level are smaller than was expected based on the width of the floodplain. So, the water level increase is a result of both the floodplain width and bed level height.

Crest height decrease and soil usage

In all cases a decrease in crest height and increase of soil volume relative to a traditional dike reinforcement is seen. However, there are clear differences between the locations. These differences coincide with the wave height at each of the locations. Higher wave heights result in a higher dike crest height decrease and to a lower ratio of soil relative to a traditional dike expansion.

4.2.5.2 Lower longitudinal crest heights

The same comparison between the locations is done for a longitudinal mound with a crest height that is lower than in Section 4.2.5.1. This is done to see if there are differences when the longitudinal mound crest height is in the flat range, see Figure 35. The differences between the locations are similar as for the maximum longitudinal mound crest heights. In Table 11 the results for the lower longitudinal mound crest heights are shown.

Table 11 Results for all 5 locations based on a longitudinal mound crest width of 10 metres and slope 1:3

Location	Height of longitudinal mound [m]	Water level increase [m]	Crest height decrease [m]	Amount of soil [x version dike]
Waal_07	5.0	0.19	0.49	8.7
Waal_07	2.5	0.06	0.26	7.3
Lek_05	3.0	0.10	2.12	1.3
Lek_05	1.5	0.03	0.73	1.2
Waal_09	4.0	0.12	0.75	4.3
Waal_09	2.0	0.03	0.36	3.1
Rijn_01	4.0	0.07	1.22	2.5
Rijn_01	2.0	0.02	0.57	1.8
Maas_06	3.5	0.13	0.27	12.2
Maas_06	1.8	0.03	0.12	10.1

The differences between the locations as seen in Table 11 are very similar to the differences that occurred for the maximum height of the longitudinal mound, as seen in Table 10. The differences between the high and low crested longitudinal mounds are more interesting. Halving the longitudinal mound crest height results in about a three times smaller water level increase. This is as expected as halving the crest height of the longitudinal mound results in a reduction of about two thirds of the soil needed. A low crested longitudinal mound results in a dike crest height decrease of a factor of 3 to 4 times lower than for the maximum longitudinal mound crest height. This is a result of the fact that the lower crested longitudinal mounds are located in the flat range.

4.2.6. Relative freeboard

The dike crest height reduction as a result of the crest height of the longitudinal mound for all five locations is shown in Figure 40. The wave heights are mentioned before in Table 9 and are as follows: Waal_07 0.58 metres, Lek_05 1.13 metres, Waal_09 0.74 metres, Rijn_01 1.01 metres and Maas_06 0.27 metres. For all five locations from Figure 40 and the wave heights it can be concluded that the increase in dike crest height reduction starts at a relative negative freeboard of 2 [-]. As mentioned in Chapter 2 the relative freeboard is the most influential parameter in the Friebel and Harris formula. It can be seen that from the longitudinal mound crest height where the relative negative freeboard is 2 [-] the wave starts to be affected by the longitudinal mound.

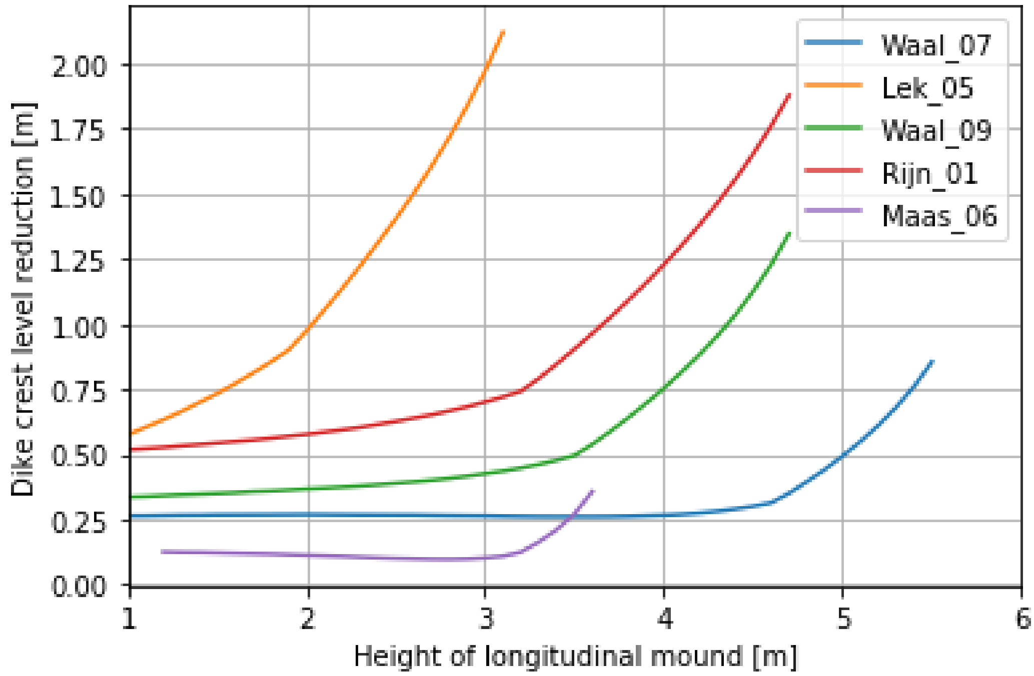

Figure 40 Dike crest height reduction based on height of longitudinal mound for all locations

Because location Lek_05 has a combination of high waves and a small difference between floodplain level and dike crest height, a smaller longitudinal mound is already effective at location Lek_05.

4.3. 2D D-Flow FM model

In this chapter the D-Flow FM results are presented. Therefore, many maps are shown with a colour bar to indicate water levels and flow velocities. The colour band goes from dark blue (lowest) to red (highest). However, the minimum and maximum values are not always equal between the figures that show the same parameter but for different variants. So, be wary of this difference and keep the colour band in mind when looking at the figures.

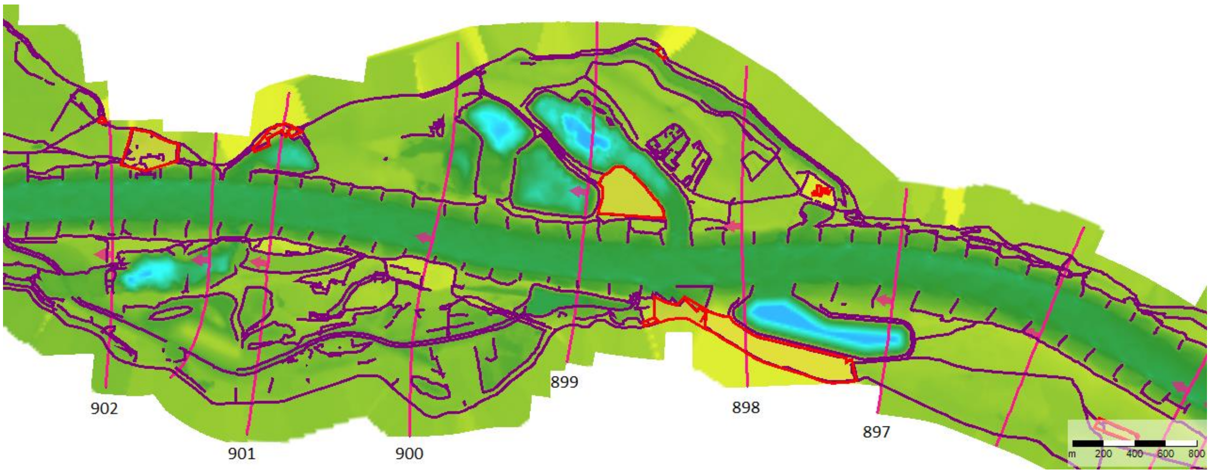

Figure 41 Original topography at location Waal_07 including the cross-section measurement locations.

In Figure 41 the original topography at location Waal_07 is shown including the cross-section measurement locations.

4.3.1. From original to refined grid

With the original grid having been validated by Deltares it is important to check if the results with the new locally refined grid yields the same results. Although a map image cannot be made because of the different grid size it could be seen that the changes are minimal, in the order of millimetres. Although a map image is not provided it is possible to look quantitatively at the measurement locations.

In Table 12 the results per river kilometre are given. This measurement is taken at the river axis. In Figure 42 and Figure 43 below the table the results over time for river kilometre 899 are given. It can be seen that the spin-up time is about 2.3 days for the new grid and 1.4 days for the original grid. Also the difference between the final water level and the maximum error during the spin-up time is quite a lot larger for the new grid. However, it can also be seen that in both cases the water level at river kilometre 899 converges to about 12.99 metres.

Table 12 Water levels after 5 days simulation time at the measurement locations.

River kilometre	Water level original grid [m +NAP]	Water level new grid [m +NAP]	Difference [mm]
897	13.1990	13.1955	3.5
898	13.0970	13.0954	1.6
899	12.9856	12.9870	-1.4
900	12.9228	12.9236	-0.8
901	12.8230	12.8256	-2.6
902	12.6742	12.6750	-0.8

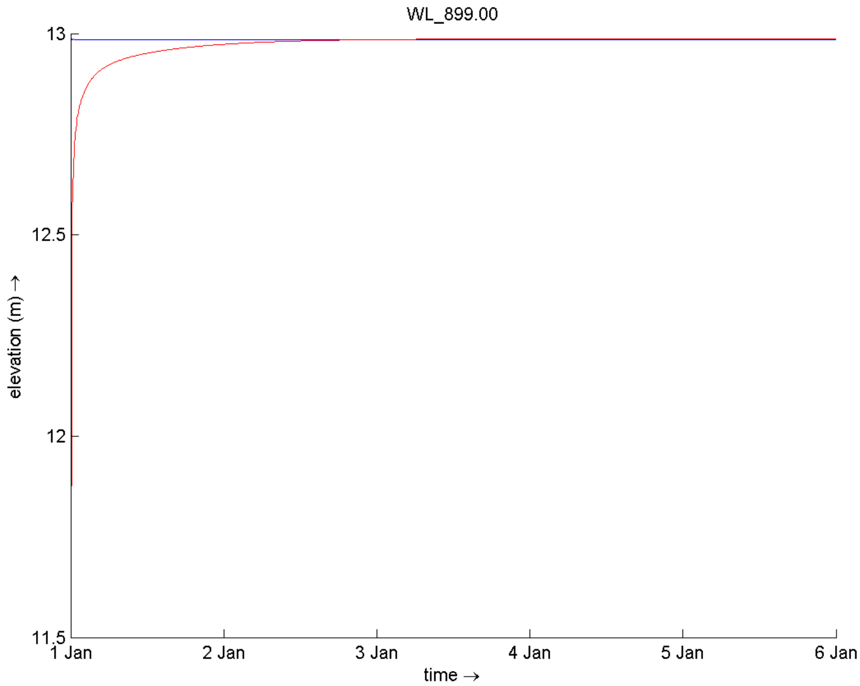

Figure 42 Water levels at WL_899 over the entire simulation period for both the original and new grid.

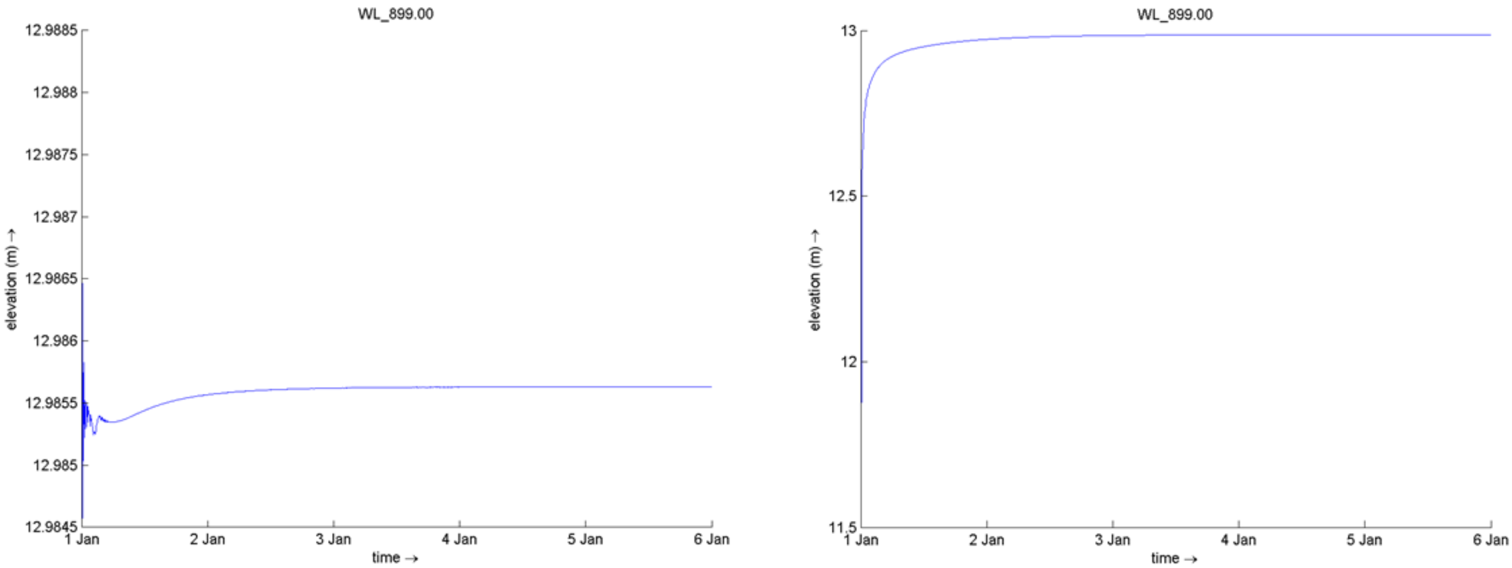

Figure 43 Water levels at WL_899 over the entire simulation period. Left side: original grid. Right side: new grid.

4.3.2. Local water level differences as result of the longitudinal mound

In Table 13 the water levels per river kilometre at the river axis for all variants are given. It can be seen that the differences in water level on the river axis are in the order of millimetres. It is expected that the difference occurring when changing the grid is also present in the simulations for the different variants. As this difference was also in the order of millimetres the effect on the water level at the river axis is still small. However, the exact difference depends on if the numerical error induced by the new grid is equal or not for all variants.

Table 13 Water level differences at the river axis for all variants.

River kilometre	Water level new grid (variant 0) [m +NAP]	Water level variant 1 [m +NAP]	Water level variant 2 [m +NAP]	Water level variant 3 [m +NAP]
897	13.1955	13.2073	13.2080	13.2025
898	13.0954	13.1023	13.1022	13.0994
899	12.9870	12.9844	12.9843	12.9862
900	12.9236	12.9209	12.9208	12.9232
901	12.8256	12.8213	12.8212	12.8225
902	12.6750	12.6739	12.6739	12.6743

4.3.2.1. Water level differences between the original situation and with longitudinal mound

In Figure 44 below the difference in water level between the original situation and variant 3 is shown. Only one variant is shown here as the general results are quite similar between the variants. Variant 3 is shown here as this image gives the most clear output. The results for the other two variants can be found in Appendix F. The differences between the variants are discussed in Section 4.3.2.2

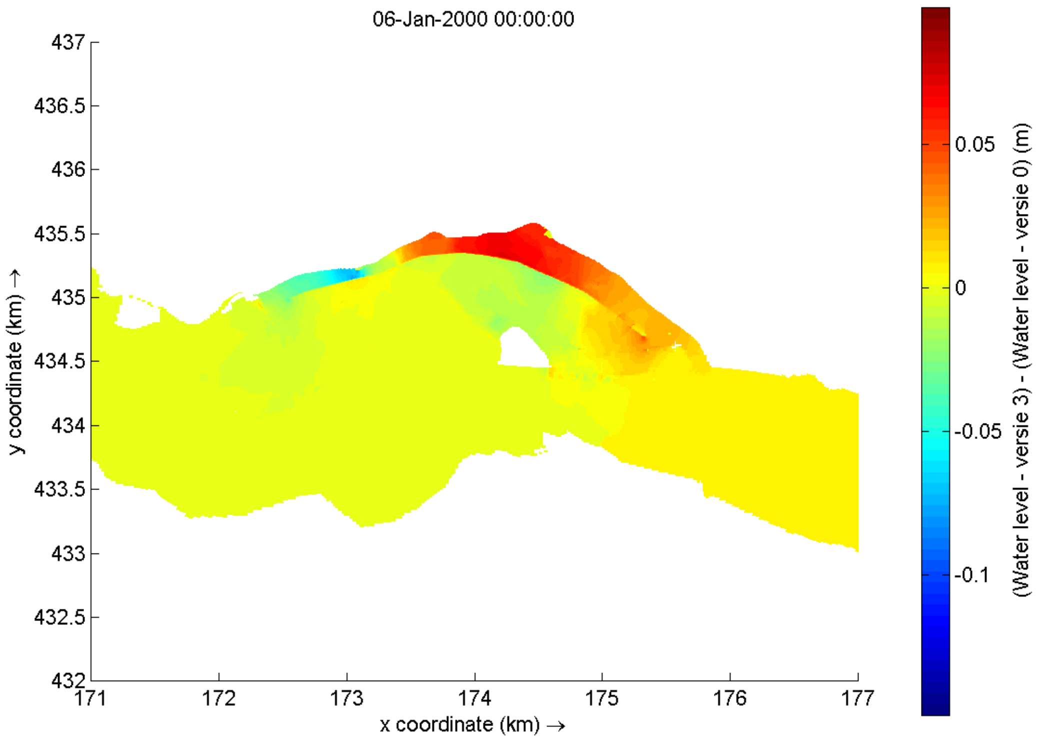

Figure 44 Water level increase from variant 0 to variant 3

It can be seen that the main changes in water level occur between the longitudinal mound and the dike. A rise in water level of about 0.05 to 0.10 metres can be seen between the longitudinal mound and the dike. This could be a result of a lower flow velocity into this part. The Bernoulli principle states that a reduction of flow velocity results in an increased water level, and vice versa. At the upstream part of the floodplain the defunct nuclear reactor of Dodewaard is located. This possibly decreases the flow into the area between the dike and longitudinal mound even further. This could be another factor in reducing flow velocities and therefore the increased water level.

Further downstream, where the area between the longitudinal mound and the dike decreases, a drop in water level of about 0.05 to 0.10 metres can be seen. This most likely occurs because of the contraction. A contracted flow has an increased flow velocity, which subsequently results in a decrease of water level. In Section 4.3.3. the flow velocity is looked at in more detail.

On the other side of the longitudinal mound a slight decrease in water level, between 0 and 0.03 metres is found.

4.3.2.2. Water level differences between the variants with a longitudinal mound

In Figure 45 below the difference in water level between the variant 1 and variant 2 is shown. The results for the other two combination of variants can be found in Appendix F. Some minor differences can be found when comparing variant 1 and variant 2. The main difference occur at the begin and end of the longitudinal mound. At these locations the 2.5 metres high soil bodies are placed in variant 2. In variant 2 an extra water level decreases is observed at these locations. This is a result of the flow over these 2.5 metre high soil bodies which increases flow velocity and decreases the water level. Also the increase and decrease of water level behind the longitudinal mound is slightly larger for variant 2.

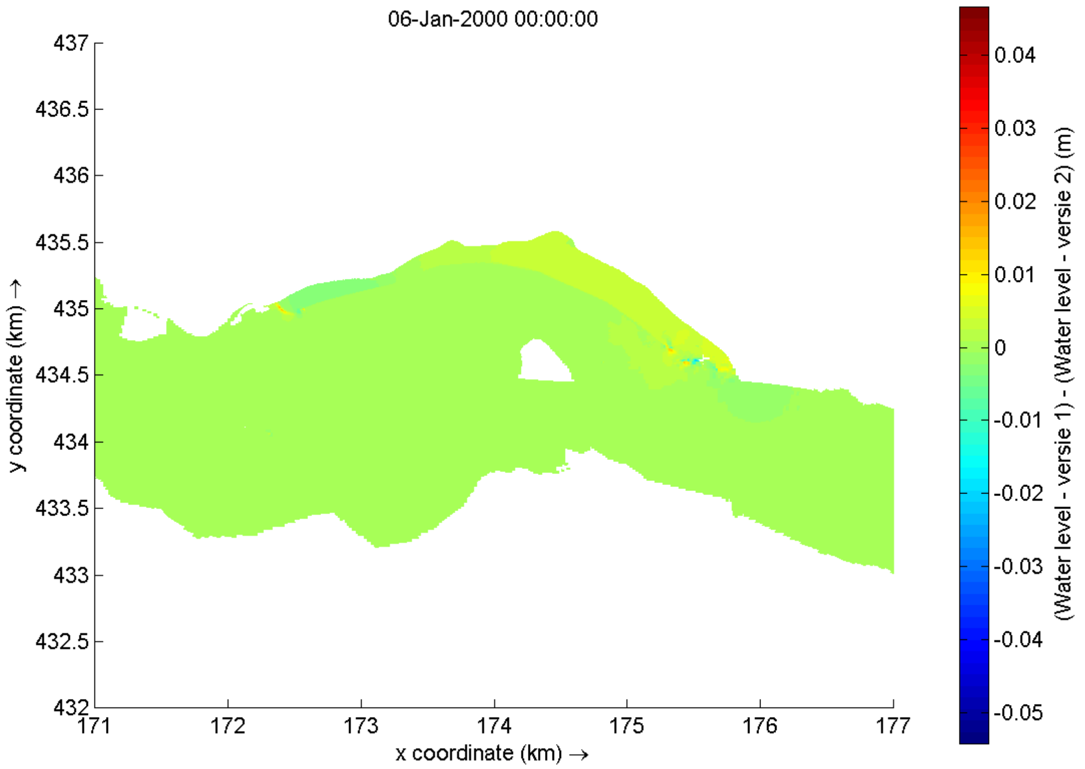

Figure 45 Water level increase from variant 1 to variant 2

4.3.3. Local flow velocity differences as result of the longitudinal mound

In Figure 46 the original flow velocities are shown. It can be seen that the flow through the river channel is higher than on the floodplain, as expected. And on the floodplain itself the flow velocities depend on the bed topography. For instance the middle light blue bar (higher velocity than its surroundings) is located between two deeper areas.

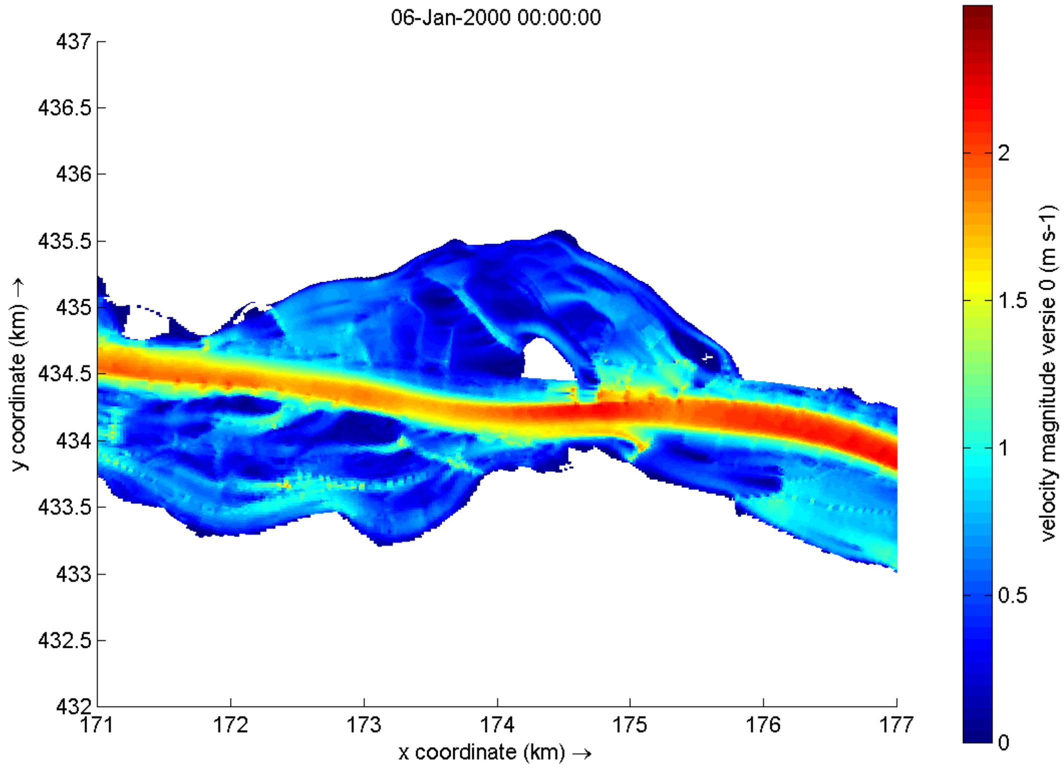

Figure 46 Flow velocities in the original situation (variant 0)

In Figure 47 it can be seen that the longitudinal mound has quite some influence on the flow velocity on the floodplain. At the entry the flow accelerates along the beginning of the longitudinal mound. After that the flow slows down in the same area where the water level increases. Then the flow accelerates going through the contraction and decelerates afterwards. The dark blue patches indicate that the flow on top of the longitudinal mound is going towards 0 m/s. So, there is little flow over the crest of the longitudinal mound. Directly on the inside of the longitudinal mound some increased flow velocities can be seen. These also correspond with the slight decreased water level at this location.

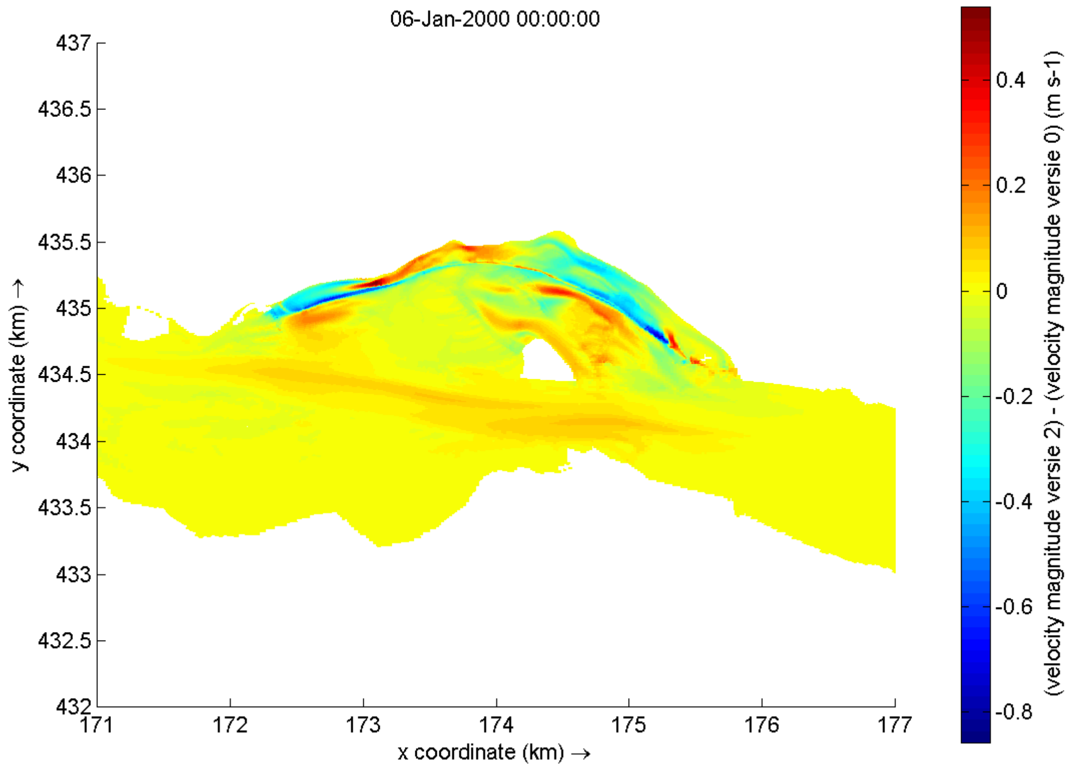

Figure 47 Flow velocity increase from variant 0 to variant 2

Just as is the case for the water level, the difference between variant 1 and 2 is present at the start and end of the longitudinal mound where the 2.5 metres high soil bodies are located. In variant 2 the flow velocities at the entry is 0.1 m/s higher. However, there is also a smaller flow velocity in the return zone. In all variants there is also a small increase in flow velocity in the river channel. This increase is smallest in variant 3. Figures can be found in Appendix F.

4.3.4. Effect upstream of the longitudinal mound

Most water level changes occur on the floodplain and not in the main channel. There is an difference of water level on the main river channel only in the order of millimetres. This is because the floodplain is not long enough to have the water level go to the new equilibrium water level with the longitudinal mound. But also due to the increased flow velocity on the main channel resulting in a water level reduction as follows from the Bernoulli principle.

This also has as a result that there is only a small backwater effect upstream as the new water level at the stream upwards boundary of the floodplain only deviates a little from the equilibrium water depth stream upwards of the floodplain. Although this effect only leads to a small water level changes, it is of importance to keep these in mind. The calculations for dike failure mechanisms on the upstream part of the river have to be done with the new water levels.

4.3.5. Comparing D-Flow FM to the conceptual model

After the conceptual model was adapted to be equal to the D-Flow FM model at river kilometre 899 the new equilibrium water level for Waal_07 has been calculated again with the conceptual model. In D-Flow FM a backwater curve occurs which does not fully develop towards the new equilibrium depth. Therefore there is a discrepancy in the calculated water levels. These water levels are shown in Table 14.

Table 14 Water levels for all variants at river kilometre 899 for D-Flow FM

	Conceptual model	D-Flow FM
Variant 0	12.987	12.987
Variant 1	13.176	12.9844
Variant 2	13.269	12.9843
Variant 3	12.965	12.9862

Also from the D-Flow FM model it can be concluded that the differences in water level are mainly local and induced by the topography of the (altered) floodplain. As the conceptual model only calculates the equilibrium depth for a simplified profile the new water level in the conceptual model does not represent the actual water level. As can be seen before the water level changes on the main river channel for the D-Flow FM simulation are small.

5. Discussion

5.1. HydraNL: combination of waves and water level

Design return period

With HydraNL the design wave height and water level depend on the selected return period of the hydraulic load. There are two variables that influence these parameters, namely the wind velocity (and direction) and the discharge. Both variables have their own return periods. Together they form a return period for the sum of these variables. However it is possible that for a higher combined return period a different combination of wind velocity and discharge is dominant. This means that it is possible that for a larger return period lower waves occur than for a smaller return period.

As a result the wave heights used in the calculations, for the conceptual model, depend on the combination of discharge and wind velocity. This mean there could be a shorter return period than the design return period that consists of a combination of a higher water level and smaller waves or vice versa. In the case of smaller waves the potential damping of the wave is smaller as well. In combination with a higher water level the necessary dike crest level could be higher than for the larger design return period.

During periods of normal flow the floodplains tend to be dry. In these periods the longitudinal mound has no influence on the water level and flow velocities. During rising water levels the water will eventually reach the toe of the longitudinal mound. In these conditions waves will not reach the dike, however the wave attack will be present onto the longitudinal mound.

Other failure mechanisms

In this research only the design return period is taken into account. However, most of the time the conditions are less severe. In these conditions the longitudinal mound still has an influence on the hydrodynamics and therefore the forces on the dike. There is a change in wave forces on the dike, not only during at the design return period but also in the case of lower water levels. Also the flow velocity along the dike will most likely change. So, a change in wave forces and flow velocity as a result of the longitudinal mound is expected. This has an influence on multiple dike failure mechanisms. Erosion of the outer slope reduces with lower wave impacts, however increased flow velocities induce more erosion. These hydrodynamical changes could also have an effect on soil instability and shear stresses inside the dike. For now these effects are unknown.

5.2. Conceptual model

From the conceptual model it can be concluded that there are two optimum designs. The first is the highest possible longitudinal mound as this results in the lowest necessary dike crest height. The second is a longitudinal mound that is as small as possible to use the least soil relative to a dike expansion. This is still the case when ignoring the limits for the validity of the Friebel and Harris formula (except the submerged crest height requirement).

As mentioned in Section 5.1 a longitudinal mound could potentially be beneficial in situations with lower water levels to reduce damage on the outside of the dike. So, a lower longitudinal mound crest height would be sufficient for this purpose. However, in the case of design return periods it has been shown that the lower design of the longitudinal mound is less effective in reducing the dike crest height and uses more soil than the optimal longitudinal mound design.

Some simplifications and assumptions have been made in the conceptual model. The most influential ones are explained below.

Wave heights and overtopping

In the conceptual model the calculations for wave overtopping have been done with the Van der Meer overtopping formula. With this formula only the overtopping discharge is taken into account. However, in recent years also the maximum overtopping volume V_{max} has been taken into account. This maximum overtopping volume is the maximum per wave. For smaller waves an equal V_{max} results in a higher allowable overtopping discharge (EurOtop, 2018). If there are no further restrictions on the inside of the dike, the result of a higher allowable overtopping discharge is that the necessary dike crest height decreases. As the effect of the longitudinal mound is to decrease wave heights this effect could potentially decrease the necessary dike crest height even further.

On the other hand in the conceptual model the angle of incidence is ignored and all waves are assumed to be perpendicular to the dike, and the longitudinal mound. As a result in some cases, where the wave attack has a significant angle of incidence, the wave height used in the calculation is too large. The effect of this larger wave height is that the reduction of the wave height over the longitudinal mound is overestimated. This subsequently means that the reduction of dike crest height is also overestimated. For location Waal_07 with the largest longitudinal mound this overestimation is about 10 to 15 centimetres of dike crest height reduction for waves under 45°.

Water level

In the conceptual model the water level is calculated based on the equilibrium water depth. The equilibrium depth is the water depth for uniform flow. However, as a result of the longitudinal mound there is no uniform flow. The equilibrium depth changes along the river. The actual water depth will change over the river length with a backwater curve. At the downstream part of the longitudinal mound the water depth is equal to the original equilibrium depth. The actual water depth will adjust slowly upstream towards the new equilibrium depth. However, this adjustment takes in the order of tens of kilometres to adapt to the new equilibrium depth. In the case of Waal_07 the length of the floodplain is only about 4 kilometres. This means that the new equilibrium depth is not reached yet at the floodplain. In Section 4.3 it can be seen that the water level difference on the river axis is only in the order of millimetres.

Upstream of the longitudinal mound a backwater occurs also. This backwater starts from the new equilibrium depth towards the original water depth. Therefore, the water level upstream of the longitudinal mound will increase relative to the original situation. Due to the small backwater effect of the longitudinal mound the water level difference upstream is only small as well. However, even the effects in the order of millimetres have to be documented.

Using water level instead of water depth in the Friebel and Harris formula

The wave transmission calculated with Friebel and Harris shows a decrease of wave height over a submerged breakwater. This effect is larger than any water level increase, therefore the dike crest level reduces. In Chapter 4 it can be seen in Figure 35 that this reduction is about equal for longitudinal mound heights of 1 to 4 metres. This occurred as a result of using the water level instead of water depth in the Friebel and Harris formula. When using the correct parameters the lowest longitudinal mounds are not within the limits for h/d (structure height over water depth) as mentioned in Chapter 2. The resulting dike crest height reductions are similar to the results shown in Chapter 4. However, there are some differences.

In Figure 48 a comparison between the conceptual model and the correct Friebel and Harris formula at location Waal_07 is shown. The first difference is that the minimum longitudinal mound crest height is at 2.4 metres. At this minimum longitudinal mound crest height the dike crest level reduction is

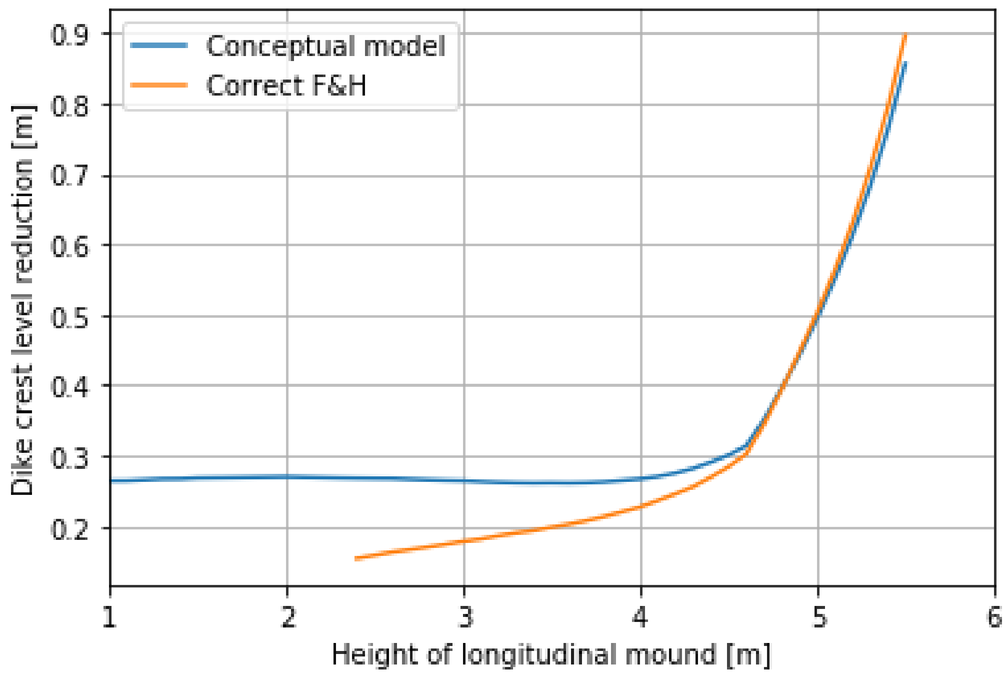

Figure 48 Difference between the conceptual model and the correct Friebel and Harris formula

about 10 centimetres smaller than it was when using the water level. For the maximum longitudinal mound crest height the dike crest level reduction is a couple of centimetres larger than it was before. All in all this results in that the flat range, as mentioned in Chapter 4, is shorter and less flat. The dike crest level reduction in the longitudinal mound height from 4.5 to 5.5 metres is very similar. Therefore, the same conclusions can still be drawn.

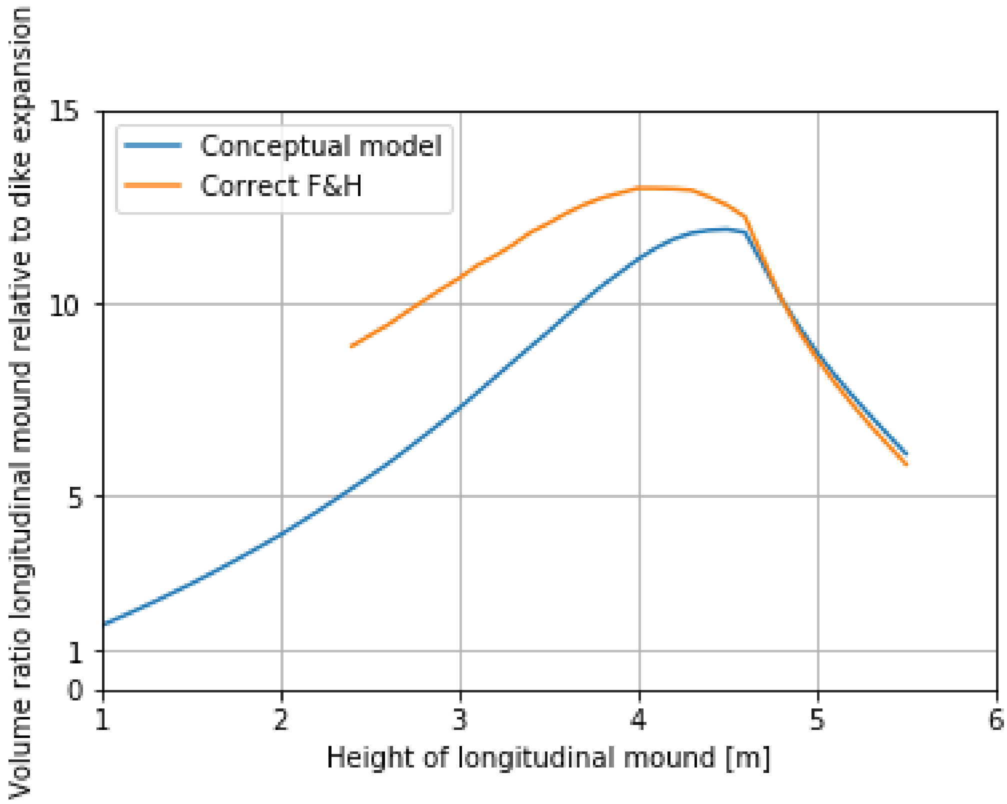

Figure 49 Difference between the conceptual model and the correct Friebel and Harris formula

This change in dike crest height reduction for lower longitudinal mounds also has an effect on the volume ratio of the longitudinal mound compared to a dike expansion. The lowest longitudinal mounds are not possible anymore as they are too low for the limits of the Friebel and Harris formula and the longitudinal mounds between 2.5 and 4.5 metres do have a smaller dike crest height reduction now. This means that the highest longitudinal mound now also has the lowest volume ratio. This is shown in Figure 49.

The influence on the results of the longitudinal mound crest width is smaller. This is shown in Appendix D.

5.3. D-Flow FM

Conversion from the original grid to the new refined grid

With the increase of resolution at the floodplain the water depth at the river axis showed a small discrepancy between the two grids, because the local grid refinement introduces numerical errors. The difference on the river axis is small, however this is also the case for the different variants, all of which were on the new grid. It is not possible to compare the water levels on the different grids directly via Quickplot as the gridsize is not the same. Before running it was assumed that the water level would change over the entire width of the river and floodplain, therefore no extra observation points were added at the floodplain. After running the simulation of the original grid and the refined grid with variant 0 with extra observation points it was found that there is a similar difference on the floodplain as on the river axis.

Roughness of the longitudinal mound

With the refinement of the grid, the roughness value of the old grid has been imposed on all new grid cells on top of the old grid cell. This means that there has not been a change in roughness. However, it is more likely than not that the roughness value of the longitudinal mound would change relative to the original roughness on the floodplain. In the conceptual model the roughness was also kept the same, however there is the possibility in the code to change the roughness.

Boundary conditions

During the simulation a constant downstream water level and upstream discharge boundary condition have been used. This means that in the simulation there was enough time to find the equilibrium water levels and flow velocities. However, in a real life situation the discharge will change over time. Simulating an increasing and subsequently decreasing discharge would give an extra insight into what would happen, for instance during a flood wave.

Initial conditions and spin-up time

The initial conditions for all variants have been the same as the initial conditions in the verified testruns as performed by Deltares. This is done as the discharge used in the highest discharge testrun was about equal to the discharge used in the conceptual model. Because the initial conditions were not changed the spin-up time for all variants was longer than for the original grid. As the runtime was 5 days and the spin-up time of around 2.5 days this should not lead to any differences in the final result.

Longer runtime of variant 0

Another point of interest is that the three variants all had a very similar runtime of ~41 hours. Initially variant 0 had a runtime of ~30 hours, this was due to an error that the entire part of the grid that was refined was not taken into account. After this was solved the runtime for variant 0 was ~50 hours. It was expected to be similar to the other three variants. The source of this increased runtime has not been discovered (yet).

6. Conclusions

6.1. Location study

From the 43 selected locations the following locations came out on top, however keep in mind that this is not a complete list of suitable locations.

- Maas_13 located at Waalwijk
- Waal_07 located at Dodewaard
- Waal_17 located at Gorinchem
- Maas_08 located between Heerewaarden and Lith
- Lek_05 located at Lopik

These five locations do not per se follow all three conditions mentioned above. Therefore, it should always be studied if a longitudinal mound is the best solution. With the construction of a longitudinal mound compromises are always necessary.

From the sensitivity analysis it can be concluded that from the perspective of different stakeholders different locations are suitable for a longitudinal mound. The biggest difference is between an ecological and water safety perspective. This is mainly due to Natura2000 areas along the floodplains of the Waal.

The ranking of the Water safety distribution is the most in common with the Base distribution. This could indicate that the Base distribution is biased towards the water safety aspect of the multicriteria analysis.

6.2. Conceptual model

As mentioned in Chapter 5 the conceptual model is far from perfect yet and should be further fine-tuned, but some conclusions can be made. From the conceptual model it is clear that, at least theoretically, with a longitudinal mound the hydraulic load on the dike during design conditions is reduced. For higher waves, the relative wave height reduction is larger than for smaller waves.

In the conceptual model a 1D approach is used. This means there are three design parameters: slope, crest width and crest height.

- A flatter slope reduces the flow area more than a steep slope and therefore a flatter slope results in a larger water level increase. The slope has no direct influence on the wave height reduction, however the larger water level increase of the shallower slope results in a little less wave height reduction as the negative freeboard is increased.
- A wider crest width of the longitudinal mound increases the dike crest height reduction. This effect gets smaller for every extra metre added. Therefore, there is a maximum crest width for which the dike crest height reduction increases. However, this point has been outside the limits for Friebel and Harris. At location Waal_07 a crest width of 4, 12 and 20 metres results in a dike crest height reduction of 0.81 metres, 0.92 metres and 1.01 metres respectively.
- The crest height of the longitudinal mound is most important. For the lowest longituindal mound as in within the limits of the Friebel and Harris formula the dike crest height decrease is small. Increasing the longitudinal mound height increases the dike crest height reduction. When the negative freeboard reaches twice as size of the wave height the reduction of the dike crest height becomes exponential. With a larger negative freeboard the effectivity of the longitudinal mound is low. At location Waal_07 a wave height of 0.58 metres is present. A longitudinal mound crest height of 2.5, 4.5, 5 and 5.5 metres results in a negative freeboard

over wave height ratio of 4.3, 1.8, 1.0 and 0.1 respectively, which subsequently results in a dike crest height reduction of 0.15, 0.29, 0.51 and 0.90 metres respectively.

Looking at the soil volume used for a longitudinal mound relative to the volume used for a dike crest height increase, that is equal to the dike crest height reduction as a result of the longitudinal mound, it can be seen that for a larger longitudinal mound this ratio increases. This is until the negative freeboard is twice the wave height. At that moment the ratio reaches its maximum and starts decreasing. However, also for the maximum wave height reduction more soil is needed for the longitudinal mound than for a dike reinforcement.

Although a longitudinal mound could reduce the necessary dike crest height, it does need more soil. Also, the longitudinal mound takes in quite a lot of space on the floodplain. So, other options to reduce the wave height might be favourable. However, before that can be concluded more research between methods for wave height reduction are necessary.

6.3. 2D D-Flow FM model

With the D-Flow FM model three variants have been tested in comparison to the original situation on location Waal_07 near Dodewaard. The water level in the main river channel is very similar for all variants. The maximum difference found is about 1 millimetre. However, between the longitudinal mound and the dike water level differences are found in the order of 10 centimetres. These water level differences are caused by a decrease and increase of flow velocity.

Therefore it can be concluded that the water level differences are a result of the flow area between the longitudinal mound and the dike, as flow velocities increase when the flow area gets restricted and decreases when there is a widening as by the Bernoulli principle. This means that the flow velocity and water levels between the longitudinal mound and the dike can be managed by the design of the alignment of the longitudinal mound. If necessary a water level decrease can be induced. However, this will also lead to an increase of flow velocity, which subsequently increases the possibility of erosion.

As the alignment of the longitudinal mound of the three variants was identical no large differences have been found between the variants. This further indicates that the alignment is important for the effect the longitudinal mound has.

6.4. Differences between the conceptual model and 2D D-Flow FM model

In the conceptual model a much larger water level change as a result of the longitudinal mound is found. Mainly this is due to the lack of the backwater effect in the conceptual model yet. So, the current version of the conceptual model is not able to replicate the results of the D-Flow FM model accurately enough to use in the first design steps, not even on the river axis.

The main water level changes occur on the floodplain as an effect of the 2D local situation. Therefore, it is more difficult to accommodate this in the conceptual model as the alignment of both the dike and the longitudinal mound determine the flow velocity and subsequent water level change. However, this process can be simplified as well by using a few cross-sections and by calculating the water level differences with the energy and momentum balances for contraction and widening of the flow respectively.

7. Recommendations

Before a longitudinal mound can be constructed further research is necessary. These recommendations are for improvements of the conceptual model and for further 2D research.

Other methods for wave height reduction

Reducing wave heights before they reach the dike has the potential to reduce wave run-up and overtopping and could therefore be used to reduce dike reinforcements. However, a longitudinal mound is not the only way to reduce the wave heights at the dike. Other methods could be planting more tall vegetation like trees on the floodplain. There are some negative aspects of a longitudinal mound, like the need of a lot of space and soil that are not present in other methods. Therefore, it is recommended to qualitatively and quantitively compare different methods of wave breaking on river floodplains. Before this comparison can be made some further improvements on the conceptual and D-Flow FM models of the longitudinal mound are needed. These are described below.

Improvement of the conceptual model

The conceptual model did not correspond to the D-Flow FM model very well. The two largest areas in which the conceptual model can be improved are the wave incidence and the backwater effect. It is recommended to add these effects to the conceptual model to improve the result of the water level at the river axis.

On top of those improvements it is also recommended to calculate the water levels between the longitudinal mound and dike by segmenting this area and applying the energy and momentum balance to calculate the water levels in these segments.

Extra D-Flow FM simulations

For the D-Flow FM model it is recommended to also test a longitudinal mound at a location where the river and floodplain width is smaller. Simulating a narrower area of the river shows if the water level on the river axis is influenced more, or that the adaptation length of the backwater is still dominant. Next, different alignments of the longitudinal mound should be investigated, to confirm that the water level depends on the local topography.

2D wave model

Finally, in the D-Flow FM model the flow is calculated on a 2D grid. However the waves have not yet been implemented in this model. So, the next step is to incorporate a D-Waves model into the D-Flow FM model. With this extra module the effect of the longitudinal mound on the waves can be calculated in 2D. The resulting change in waves and wave height can then be compared to the conceptual model as well.

Other applications and failure mechanisms

In this research the longitudinal mound has only been tested for overtopping discharge at design conditions. However, other failure mechanisms have not been taken into account. This means that it is not known what the effects of the longitudinal mound are on those other failure mechanisms. Also, only the design conditions are used. However, for other failure mechanisms there could be an effect at different conditions as well, which would result in other designs for the longitudinal mound. Therefore, it is also recommended that the effects for other failure mechanisms are researched.

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Appendix A – Larger figure depicting all studied locations

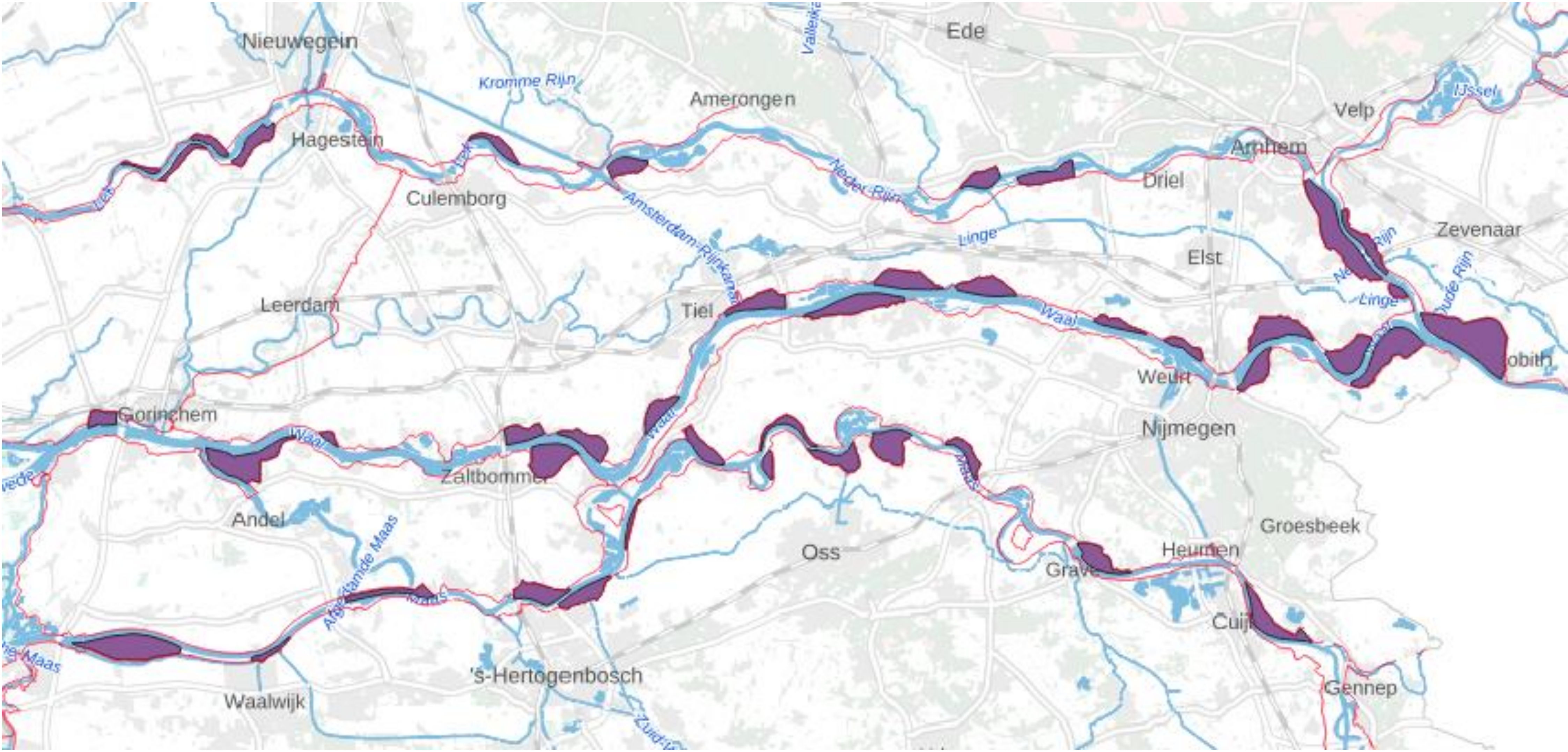

Figure A-1 All 43 studied locations

Appendix B – Tables location study

In this appendix all results of all locations are shown in tables. These tables contain the numerical values of which the scores are based on.

Size of the floodplain and structures around the floodplain

In the table below the maximum width, average width and maximum length of the floodplain are given in metres. Also, the percentage of dike length at which a structure in front of the dike is present.

Table B-1 Floodplain width, length and percentage of structures

Location	Maximum width [m]	Average width [m]	Maximum length [m]	percentage structure in front of the dike
Lek_01	600	400	3 700	41%
Lek_02	900	700	3 000	86%
Lek_03	450	400	2 200	46%
Lek_04	400	300	2 500	91%
Lek_05	300	200	2 200	87%
Maas_01	1 000	600	3 800	52%
Maas_02	1 250	750	3 000	44%
Maas_03	600	400	3 000	42%
Maas_04	2 000	1 600	4 100 (1500)	38%
Maas_05	1 200	800	4 500 (1300)	55%
Maas_06	500	300	2 500	71%
Maas_07	750		1 700	65%
Maas_08	800	600	2 800	71%
Maas_09	250	150	2 400	18%
Maas_10	750	600	3 300	34%
Maas_11	900	800	3 000	23%
Maas_12	700	500	1 400 (900)	89%
Maas_13	250	150	2 700	58%
Maas_14	1 500	1 000	5 500	54%
Neder-Rijn_02	900	700	3 500	67%
Neder-Rijn_03	1 000	800	1 000	100%
Neder-Rijn_04	1 000	800	2 500	70%
Pannerdenschkanaal_01	600	300	1 600	78%
Pannerdenschkanaal_02	1 100	800	7 500	51%
Pannerdenschkanaal_03	600	450	5 000	59%
Rijn_01	2 500	1 300	5 000	71%
Waal_01	1 700	1 000	4 500	45%
Waal_02	2 300	2 100	800	38%
Waal_03	1 100	600	5 000	62%
Waal_04	1 000	600	3 300	28%
Waal_05	400	350	2 800	43%
Waal_06	500	350	3 000	37%

Waal_07	1 100	600	3 500	67%
Waal_08	1 000	800	4 000	62%
Waal_09	1 000	600	5 500	70%
Waal_10	1 000	700	3 400	44%
Waal_11	800	500	3 000 (1200)	71%
Waal_12	1 100	800	3 000	52%
Waal_13	2 100	1 600	4 500 (2200)	44%
Waal_14	1 000	700	2 700	63%
Waal_15	500	350	700	25%
Waal_16	1 500		4 000	29%
Waal_17	900	750	2 200 (1500)	100%

Availability of local soil

Table B-2 Percentage of clay and clayey sand in the upper 2.5 and 5 metres of all floodplains

Location	Clay 2.5m [%]	Clayey sand 2.5m [%]	Total [%]	Clay 5m [%]	Clayey sand 5m [%]	Total [%]
Rijn_01	8.7	30.9	39.6	7.0	21.9	28.9
Pannerdenschkanaal_01	17.9	65.0	82.9	32.3	43.9	76.3
Pannerdenschkanaal_02	14.7	49.0	63.7	12.9	40.6	53.5
Pannerdenschkanaal_03	4.3	63.7	68.0	4.7	44.5	49.2
Neder-Rijn_02	4.9	9.4	14.3	3.9	6.8	10.7
Neder-Rijn_03	5.3	39.0	44.3	12.1	30.4	42.5
Neder-Rijn_04	16.8	41.5	58.3	16.7	32.5	49.2
Lek_01	35.6	25.7	61.3	24.1	17.3	41.4
Lek_02	27.1	38.9	66.0	29.0	33.4	62.5
Lek_03	17.9	46.3	64.2	30.2	29.9	60.1
Lek_04	5.6	48.7	54.4	3.3	34.4	37.7
Lek_05	8.7	30.9	39.6	7.0	21.9	28.9
Waal_01	23.5	38.7	62.2	20.7	25.5	46.2
Waal_02	14.2	44.9	59.1	11.2	34.8	45.9
Waal_03	16.5	43.9	60.4	12.0	31.0	43.0
Waal_04	22.0	34.8	56.8	14.4	29.2	43.6
Waal_05	15.1	47.7	62.8	9.8	29.7	39.5
Waal_06	7.8	62.8	70.6	12.8	48.8	61.6
Waal_07	23.1	47.1	70.2	20.8	40.0	60.8
Waal_08	12.4	43.5	56.0	18.8	33.8	52.6
Waal_09	6.5	44.9	51.5	11.3	42.6	53.9
Waal_10	29.0	35.0	64.0	26.6	28.9	55.5
Waal_11	10.9	65.3	76.2	19.7	55.8	75.5
Waal_12	20.8	39.5	60.4	18.3	48.1	66.4
Waal_13	16.1	39.0	55.0	16.0	29.1	45.0
Waal_14	10.6	57.4	68.0	8.9	54.5	63.4
Waal_15	38.5	43.9	82.4	42.0	36.8	78.8
Waal_16	29.0	47.0	75.9	30.6	35.0	65.6
Waal_17	24.2	63.2	87.4	16.1	46.2	62.2
Maas_01	26.4	53.6	80.0	16.6	46.3	62.9
Maas_02	19.1	42.4	61.5	14.5	23.3	37.7
Maas_03	16.3	57.8	74.1	8.8	50.7	59.4
Maas_04	16.0	69.1	85.0	11.0	45.9	56.9
Maas_05	16.8	63.5	80.3	13.0	49.1	62.1
Maas_06	9.8	50.5	60.3	12.0	48.5	60.5
Maas_07	8.9	56.5	65.4	5.6	38.3	43.8
Maas_08	15.3	40.1	55.4	17.8	31.8	49.6
Maas_09	26.2	54.2	80.4	32.1	33.5	65.6
Maas_10	30.0	21.8	51.8	22.3	16.8	39.1
Maas_11	5.6	34.3	39.9	4.5	25.3	29.8
Maas_12	14.7	40.1	54.8	10.1	34.6	44.7
Maas_13	18.2	33.4	51.6	23.2	53.2	76.4
Maas_14	7.8	32.8	40.6	14.9	58.6	73.5

For the availability of local soil not only a table is made, also a bar chart for both the upper 2.5 and upper 5 metres are made. The locations in these bar charts are sorted by the percentage of clay in the upper 2.5 metres. The percentage of clayey sand is stacked on top of the percentage of clay.

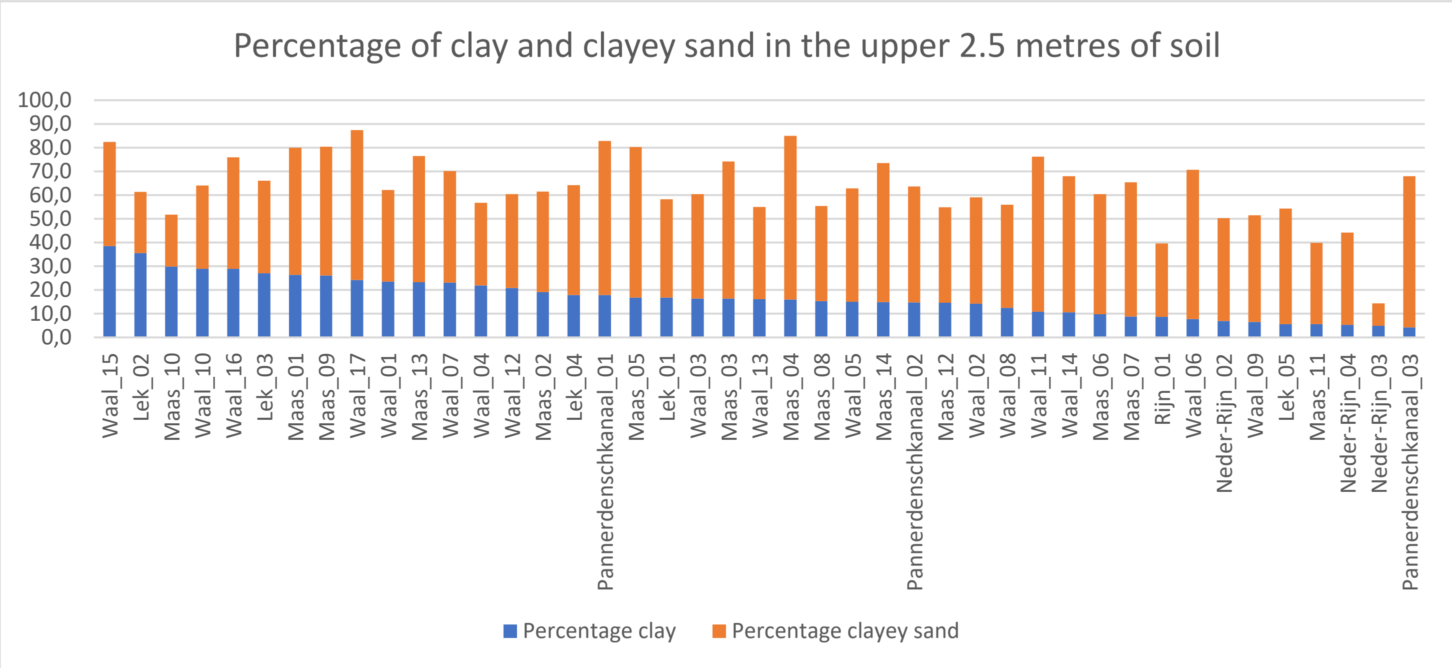

Figure B-1 Percentage of clay and clayey sand in the upper 2.5 metres of soil

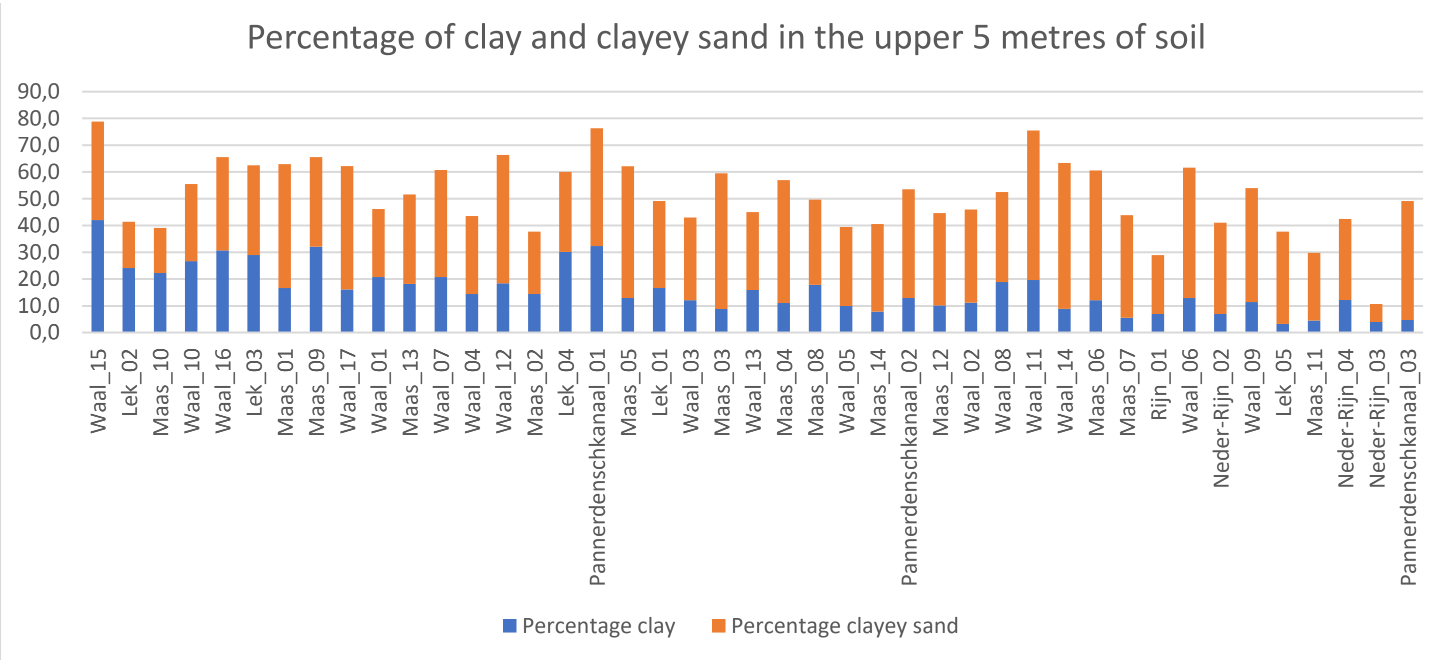

Figure B-2 Percentage of clay and clayey sand in the upper 5 metres of soil

Natura2000 and habitats

In the table below for all locations it is noted if (part of) the floodplain is classified as Natura2000. For the Natura2000 areas also the specific habitats that are located on the floodplain are mentioned, these habitats are mentioned in Dutch.

Table B-3 Results for the current habitat

Location	Natura2000	Specific mentioned habitats (Dutch)
Rijn_01	Yes	Geïsoleerde meander en petgat; Zachthoutooibos
Pannerdenschkanaal_01	Yes	Klein gebied zachthoutooibos
Pannerdenschkanaal_02	Yes	Kleine gebieden Kamgrasweide & bloemrijk weidevogelgrasland
Pannerdenschkanaal_03	Yes	Kleine gebieden Kamgrasweide & bloemrijk weidevogelgrasland en zachthoutooibos
Neder-Rijn_02	Yes	Kleine gebieden Kamgrasweide & bloemrijk weidevogelgrasland
Neder-Rijn_03	Yes	Kamgrasweide & bloemrijk weidevogelgrasland
Neder-Rijn_04	Yes	Heel klein gebied Kamgrasweide & bloemrijk weidevogelgrasland
Lek_01	No	
Lek_02	No	
Lek_03	Yes	Stroomdalgrasland naast de oever van de rivier; Zachthoutooibos in het midden van de uiterwaard
Lek_04	Yes	-
Lek_05	No	
Waal_01	Yes	Zachthoutooibos
Waal_02	Yes	Direct naast de dijk: Zachthoutooibos; Glanshaver en vossenstaarthooiland; Kamgrasweide &bloemrijk weidevogelgrasland; Geïsoleerd meander en petgat
Waal_03	Yes	Groot gebied geïsoleerd meander en petgat; significant gebied zachthoutooibos
Waal_04	Yes	Geïsoleerd meander en petgat; Glanshaver en vossenstaarthooiland
Waal_05	Yes	Klein gebied Kamgrasweide & bloemrijk weidevogelgrasland
Waal_06	Yes	Significant gebied Kamgrasweide & bloemrijk weidevogelgrasland
Waal_07	Yes	Kleine gebieden Kamgrasweide & bloemrijk weidevogelgrasland
Waal_08	Yes	Midden op de uiterwaard: Kamgrasweide & bloemrijk weidevogelgrasland
Waal_09	Yes	Kleine gebieden Kamgrasweide &bloemrijk weidevogelgrasland
Waal_10	Yes	Klein gebied glanshaver en vossenstaarthooiland en klein gebied kamgrasweide & bloemrijk weidevogelgrasland
Waal_11	Yes	Kamgrasweide & bloemrijk weidevogelgrasland en klein gebied zachthoutooibos
Waal_12	Yes	Klein gebied zachthoutooibos, geïsoleerd meander en petgat met krabbenscheer en fonteinkruiden

Waal_13	Yes	Geïsoleerd meander en petgat en zachthoutooibos
Waal_14	Yes	-
Waal_15	No	
Waal_16	Yes	-
Waal_17	No	
Maas_01	No	
Maas_02	No	
Maas_03	No	
Maas_04	No	
Maas_05	No	
Maas_06	No	
Maas_07	No	
Maas_08	No	
Maas_09	No	
Maas_10	No	
Maas_11	No	
Maas_12	No	
Maas_13	No	
Maas_14	No	

Wave heights

Initially only 20 locations were used in the effectivity study. However, after some changes in the suitability study some locations ended up higher than locations already used for the effectivity study. All locations higher than the lowest scores location already used in the effectivity study have been added. Resulting in a total of 30 locations. The average wave height over all measurement points and the standard deviation are collected in the table below. For locations with a standard deviation higher than 0.2m more details are given.

Table B-4 Results of the effectivity based on the expected significant wave heights

Location	Significant wave height average [m]	Significant wave height average std [m]	More detailed information for the location with a std higher than 0.2m
Lek_01	0.66	0.087	
Lek_02	0.61	0.035	
Lek_03	1.10	0.051	
Lek_04	0.64	0.045	
Lek_05	1.05	0.062	
Waal_01	1.25	0.142	
Waal_03	0.87	0.261	Partly 1.4 m (eastern) Partly 0.8 m Partly 0.6 m (western)
Waal_07	1.33	0.134	
Waal_08	0.98	0.174	
Waal_10	0.90	0.025	
Waal_12	1.01	0.114	
Waal_13	0.99	0.174	
Waal_14	0.81	0.068	
Waal_15	0.80	0.149	
Waal_17	1.23	0.275	Mostly 1.4 m (middle and western) Partly 0.9 (eastern
PannerdenschKanaal_01	0.69	0.176	
Rijn_01	1.12	0.367	Couple of locations only 0.1 m
Maas_01	0.66	0.060	
Maas_02	0.61	0.112	
Maas_03	0.66	0.109	
Maas_04	0.54	0.078	
Maas_05	0.64	0.074	
Maas_06	0.60	0.039	
Maas_07	0.64	0.042	
Maas_08	0.98	0.383	Middle peaks at 1.5/1.6 m edges are lower
Maas_09	1.17	0.221	Highest in the middle, which is unusual looking at the topography
Maas_10	0.95	0.244	Relatively much changes for the points next to each other
Maas_12	0.76	0.022	
Maas_13	1.10	0.136	
Maas_14	0.87	0.074	

Table B-5 Final results of the suitability study

Location	Length	Average width	Width	Structures	Soil 2.5 metres	Soil 5 metres	Habitat	Total
Rijn_01	5	5	5	5	2	1	1	66
Pannerdensch kanaal_01	2	1	2	5	4	5	2	74
Pannerdensch kanaal_02	5	4	3	4	3	3	1	64
Pannerdensch kanaal_03	5	2	2	4	1	1	1	47
Neder-Rijn_02	3	3	3	5	1	1	2	60
Neder-Rijn_03	1	4	3	5	1	1	1	54
Neder-Rijn_04	2	4	3	5	1	2	2	63
Lek_01	4	2	2	3	3	3	5	76
Lek_02	3	3	3	5	5	4	5	100
Lek_03	2	2	1	3	5	5	2	67
Lek_04	2	1	1	5	4	5	4	85
Lek_05	2	1	1	5	1	1	5	71
Waal_01	4	5	5	3	4	4	1	67
Waal_02	1	5	5	2	3	2	1	50
Waal_03	5	3	3	5	3	2	2	72
Waal_04	3	3	3	1	4	3	1	46
Waal_05	3	2	1	3	3	2	2	54
Waal_06	3	2	1	2	2	3	1	40
Waal_07	3	3	3	5	4	4	2	78
Waal_08	4	4	3	5	2	4	2	73
Waal_09	5	3	3	5	1	2	1	58
Waal_10	3	3	3	3	5	5	2	72
Waal_11	1	2	2	5	2	4	1	59
Waal_12	3	4	3	4	4	4	2	74
Waal_13	2	5	5	3	3	3	2	65
Waal_14	3	3	3	5	2	2	4	78
Waal_15	1	2	1	1	5	5	4	66
Waal_16	4	5	4	1	5	5	1	60
Waal_17	1	4	3	5	4	3	2	76
Maas_01	4	3	3	4	5	3	4	87
Maas_02	3	4	4	3	4	3	5	85
Maas_03	3	2	2	3	3	2	4	67
Maas_04	1	5	5	2	3	2	4	68
Maas_05	1	4	4	4	3	3	5	85
Maas_06	2	1	1	5	2	2	5	77
Maas_07	2	2	2	5	2	1	5	78
Maas_08	3	3	2	5	3	4	4	85
Maas_09	2	1	1	1	5	5	4	65
Maas_10	3	3	2	2	5	4	4	75
Maas_11	3	4	3	1	1	1	4	50
Maas_12	1	2	2	5	3	2	4	77
Maas_13	3	1	1	4	4	4	5	84
Maas_14	5	5	4	4	3	2	5	89

Table B-6 Final scores sorted from high to low

	Total suitability	Wave height	Total
Maas_13	84	60	144
Waal_07	78	60	138
Waal_17	76	60	136
Maas_08	85	48	133
Lek_05	71	60	131
Lek_03	67	60	127
Waal_01	67	60	127
Rijn_01	66	60	126
Maas_09	65	60	125
Maas_14	89	36	125
Lek_02	100	24	124
Maas_10	75	48	123
Waal_12	74	48	122
Waal_08	73	48	121
Waal_03	72	48	120
Waal_10	72	48	120
Waal_14	78	36	114
Maas_12	77	36	113
Waal_13	65	48	113
Maas_01	87	24	111
Lek_04	85	24	109
Maas_02	85	24	109
Maas_05	85	24	109
Maas_07	78	24	102
Waal_15	66	36	102
Lek_01	76	24	100
Pannerdenschkanaal_01	74	24	98
Maas_03	67	24	91
Maas_06	77	12	89
Maas_04	68	12	80

Appendix C – Python code conceptual model

Below the full Python code of the conceptual model is shown. The conceptual model has been made in Spyder 4.0.1. using Python 3.7.6. To make longer lines of code readable in the code below the lines of code that did not fit on one line have been altered by adding enters and tabs. So, if copying this code be aware of that.

```
# -*- coding: utf-8 -*-
"""
@author: Rokus de Bie
"""

#import
import numpy as np
import matplotlib.pyplot as plt

#%%
#Input parameters local. #### = from HydraNL
z_w = 12.83      ####Water level [m+NAP]
H_i = 0.58       ####Incoming wave height [m]
T_i = 2.67       ####Incoming wave period [s]
H_L = 14.02      ####Hydraulic load from HydraNL [m]
z = 7.5          #Local elevation of floodplain at middel of LM [m+NAP]

Q = 15931        ####Discharge [m^3/s]
B1 = 260         #Width channel [m]
B2 = 1000        #Width floodplain [m]
B3 = 0           #width LM [m] (current situation)
z_b1 = 0         #Bed level channel [m+NAP]
z_b2 = z         #Bed level floodplain [m+NAP]
z_b3 = z         #Bed level LM [m+NAP]
i_r = 0.00011    #River slope [m/m]

#Input parameters WBRD
#wave height
h = np.linspace(1,6,51)      #Height of LM [m]
B = np.linspace(0,20,201)    #Crest width of LM [m]\
S = [1,2,3,4,5,6]            #Slope of LM [1/x]
S = [3]
#Following input parameters have not been used
#B_t = np.linspace(0,200,11)     #Dry part between dike and LM (No flow)
#C3_new = [30,25,20,15,10]       #New roughness

#Standard value for fike slope and no dry flow area between dike and mound
S_dike = 3   #Slope of dike [1/x]
B_t = 0      #Dry part between dike and LM (Width no flow) [m]


# Calculate wavelength
g = 9.81                 #[m/s^2]
L = 10                   #Start value for L [m]
L_array = L*np.ones(100) #Start value for L [m]

for i in range(100-1):
    L_array[i+1] = ((g*(T_i**2))/(2*np.pi))*np.tanh((2*np.pi*z_w)/L_array[i])
    if abs(L_array[i+1]-L_array[i]) < 0.00001:
        L=L_array[i+1]
        break

Regime = z_w/L

#%%
#Calibration parameters C1, C2 and C3
C1_start = np.linspace(0,100,10001) #Roughness channel [m^0.5/s]
C2_start = (2/3) * C1_start         #Roughness floodplain [m^0.5/s]
Q_k = np.zeros([len(C1_start)])     #Discharge for calibration
```

```
#calibration
for i in range(len(C1_start)):
    Q_k[i] = B1*C1_start[i]*(i_r**0.5)*((z_w-z_b1)**(3/2))
       + (B2+B3)*C2_start[i]*(i_r**0.5)*((z_w-z_b2)**(3/2))

cal_Q = np.abs(Q-Q_k) #Compare calculated discharges with discharge from HydraNL
C1 = np.argmin(cal_Q) / (len(C1_start)-1) * (C1_start[-1])
C2 = (2/3) * C1
C3 = C2

#%%
#New water level
z_w_new = np.zeros([len(h),len(B),len(S)])

Q_test = B1*C1*(i_r**0.5)*((z_w-z_b1)**(3/2))
   + B2*C2*(i_r**0.5)*((z_w-z_b2)**(3/2))
       + B3*C3*(i_r**0.5)*((z_w-z_b3)**(3/2)) #Discharge check

for i in range(len(h)):
    print(i)
    for j in range(len(B)):
        for k in range(len(S)):

            B3_new = B[j] + (S[k]*(h[i]**2)) + B_t  #Width of LM, crest width
                                                     + halve the width of the slopes
            z_b3_new = z_b3 + h[i]                  #Crest height of LM
            B2_new = B2-B3_new                      #Reduced floodplain width

            #Water level new
            n = 30001 #
            z_w_array = np.linspace(0,30,n)         #Array for new waterlevel [m]
            Q_array = Q*np.ones(n)

            for l in range(n):   #check if height of dam is higher than the water
                                 level
                if z_w_array[l]-z_b3_new > 0:
                    Q_array[l] = B1*C1*(i_r**0.5)*((z_w_array[l]-z_b1)**(3/2))
                       + B2_new*C2*(i_r**0.5)*((z_w_array[l]-z_b2)**(3/2))
                          + B3_new*C3*(i_r**0.5)*((z_w_array[l]-z_b3_new)**(3/2))
                if z_w_array[l]-z_b3_new <= 0:
                    z_b3_new2 = z_w_array[l]
                    Q_array[l] = B1*C1*(i_r**0.5)*((z_w_array[l]-z_b1)**(3/2))
                       + B2_new*C2*(i_r**0.5)*((z_w_array[l]-z_b2)**(3/2))
                          + B3_new*C3*(i_r**0.5)*((z_w_array[l]-z_b3_new2)**(3/2))

            match = np.abs(Q_array - Q)       #Compare calculated discharge with
                                              discharge from HydraNL

            #output
            z_w_new[i,j,k] = np.nanargmin(match)/(len(z_w_array)-1)*(z_w_array[-1])

#%%
#Input transmission
H_t = np.zeros([len(h),len(B),len(S)])
T_k = np.zeros([len(h),len(B),len(S)])

#New wave height and wave period
s0_i = (2*np.pi*H_i) / (g*T_i**2)    #Wave steepness
for i in range(len(h)):
    for j in range(len(B)):
        for k in range(len(S)):

            z_b3 = z+h[i]              #crest height of LM
            F = z_w_new[i,j,k]-z_b3    #Freeboard [m]

            #Friebel & Harris
            K_t = -0.4969*np.exp(-F/H_i)-0.0292*B[j]/z_w_new[i,j,k]-
```

```
                0.4257*z_b3/z_w_new[i,j,k]-0.0696*np.log(B[j]/L)-0.1359*F/B[j]+1.0905

            #New wave period
            F_rel = F/H_i                   #Relative freeboard
            #Carevic linear approximation
            if s0_i < 0.04:
                if F_rel < 2 and F_rel > 0:
                    T_k[i,j,k] = T_i * (0.15*F_rel+0.7)
                else:
                    T_k[i,j,k] = T_i * 1

            if s0_i > 0.064:
                if F_rel < 1 and F_rel > 0:
                    T_k[i,j,k] = T_i * (0.15*F_rel+0.85)
                else:
                    T_k[i,j,k] = T_i * 1

            else:
                if F_rel < 1.5 and F_rel > 0:
                    T_k[i,j,k] = T_i * (0.13333*F_rel+0.8)
                else:
                    T_k[i,j,k] = T_i * 1

            #New wave height
            H_t[i,j,k] = H_i * K_t

            #Check if Friebel and Harris is valid
            if -F/H_i < -8.696 or -F/H_i > 0:
                H_t[i,j,k] = float("nan")
                print("Warning: Friebel & Harris not valid for",[i,j,k],".
                    Value =", -F/H_i,". Range is: -8.696 -- 0.000")
            if B[j]/z_w_new[i,j,k] < 0.286 or B[j]/z_w_new[i,j,k] > 8.75:
                H_t[i,j,k] = float("nan")
                print("Warning: Friebel & Harris not valid for",[i,j,k],".
                    Value =", B[j]/z_w_new[i,j,k],". Range is: 0.286 -- 8.750")
            if z_b3/z_w_new[i,j,k] < 0.44 or z_b3/z_w_new[i,j,k] > 1.0:
                H_t[i,j,k] = float("nan")
                print("Warning: Friebel & Harris not valid for",[i,j,k],".
                    Value =", z_b3/z_w_new[i,j,k],". Range is: 0.440 -- 1.000")
            if B[j]/L < 0.024 or B[j]/L > 1.89:
                H_t[i,j,k] = float("nan")
                print("Warning: Friebel & Harris not valid for",[i,j,k],".
                    Value =", B[j]/L,". Range is: 0.024 -- 1.890")
            if -F/B[j] < -1.050 or -F/B[j] > 0:
                H_t[i,j,k] = float("nan")
                print("Warning: Friebel & Harris not valid for",[i,j,k],".
                    Value =", -F/B[j],". Range is: -1.050 -- 0.000")

#%%
#Original dike crest height
q_l = 0.1       #Allowable overtopping [l/s/m]
q = q_l/1000    #Allowable overtopping [m^3/s/m]

s_0_original = (2*np.pi*H_i) / (g*T_i**2)           #Wave steepness
ksi_original = (1/S_dike)/np.sqrt(s_0_original)     #Breaker parameter
Fb_original = (H_i*ksi_original/-4.3) *
np.log((np.sqrt(1/S_dike)*q)/(ksi_original*0.067*np.sqrt(g*H_i**3))) #Freeboard in
the original situation

#New dike crest heights
Fb = np.zeros([len(h),len(B),len(S)])
s_0 = np.zeros([len(h),len(B),len(S)])
for i in range(len(h)):
    for j in range(len(B)):
        for k in range(len(S)):
            s_0[i,j,k] = (2*np.pi*H_t[i,j,k]) / (g*T_k[i,j,k]**2)
            ksi = (1/S_dike)/np.sqrt(s_0[i,j,k])
            Fb[i,j,k] = (H_t[i,j,k]*ksi/-4.3) *
```

```
                np.log((np.sqrt(1/S_dike)*q)/(ksi*0.067*np.sqrt(g*H_t[i,j,k]**3)))

#Crest heights
crest_original = z_w + Fb_original
crest = z_w_new + Fb

#%%
#Calculation of soil needed per meter
#soil volume dike crest height difference
delta_H = crest_original - crest    #Assumed dike crest height increase
B_crest = 10                        #Assumed dike crest width
#Volume needed for dike reinforcement
V_d = (delta_H * (B_crest - (S_dike * delta_H))) + S_dike *delta_H * ((crest-z_b2)
+ delta_H)

#Volume of soil in LM
V = np.zeros([len(h),len(B),len(S)])
for i in range(len(h)):
    for j in range(len(B)):
        for k in range(len(S)):
            V[i,j,k] = (h[i] * B[j]) + (S[k] * (h[i]**2))

delta_H = crest_original - crest #Crest height difference between original
                                 situation and with LM
Ratio_V = V/V_d                  #Ratio of volume between dike reinforcement and LM

#%%
np.savetxt("Filepath\Waal_7_z_w_new.csv", z_w_new[:,:,0], delimiter=",")
np.savetxt("Filepath\Waal_7_Ratio_V.csv", Ratio_V[:,:,0], delimiter=",")
np.savetxt("Filepath\Waal_7_delta_H.csv", delta_H[:,:,0], delimiter=",")
```

Appendix D – Selection of results of all locations used in the conceptual model

In Chapter 4 the resulting figures of Waal_07 are shown. In this appendix these figures for all 5 locations are shown in combination with a figure where all locations are shown.

The following three figures show the dike crest height reduction for the longitudinal mound crest width and crest height at location Waal_07.

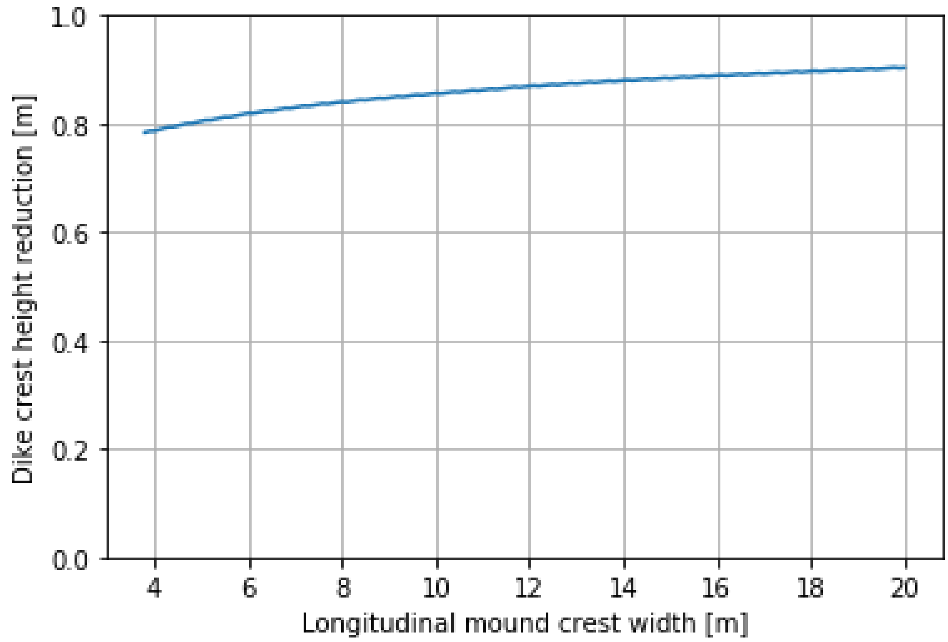

Figure D-1 Dike crest height reduction based on crest width of longitudinal mound for location Waal_07

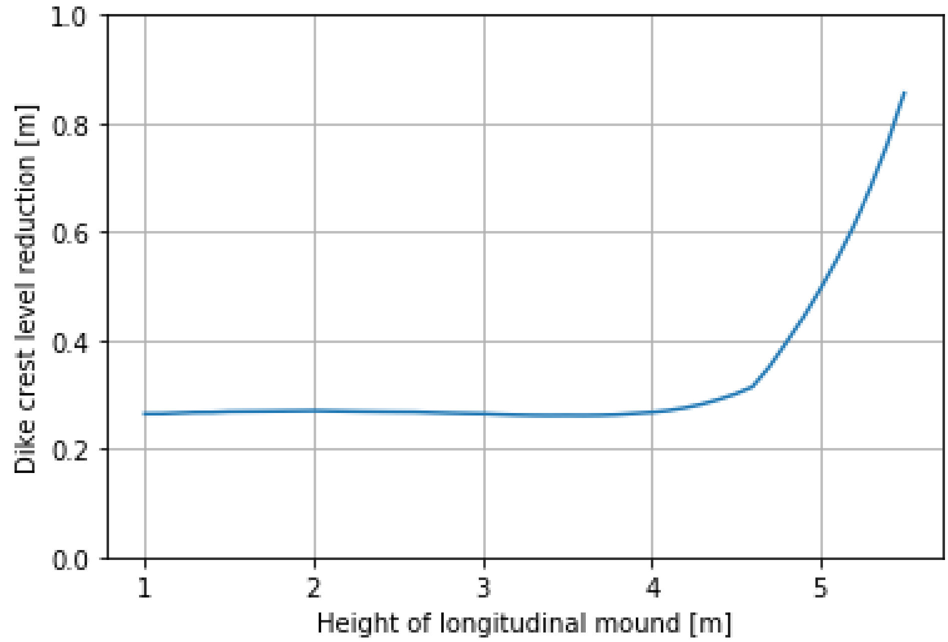

Figure D-2 Dike crest height reduction based on crest height of longitudinal mound for location Waal_07

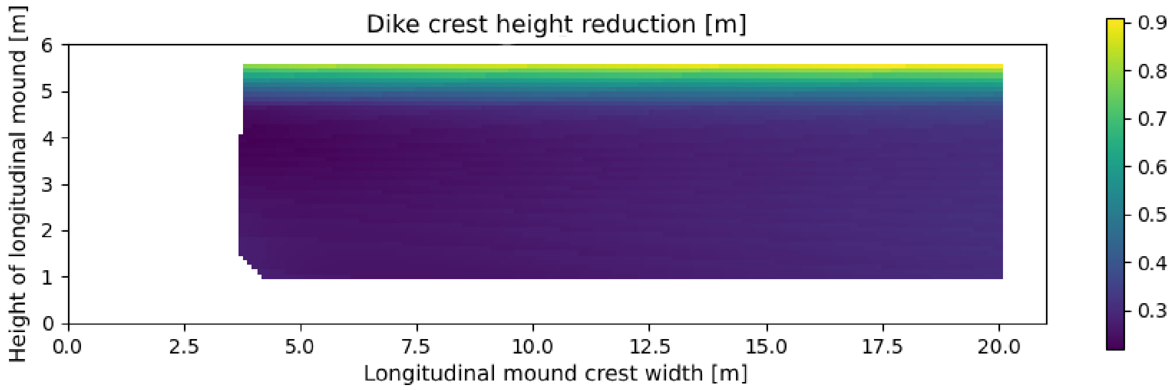

Figure D-3 Dike crest height in metres for all combination of width and length of the longitudinal mound for location Waal_07

The following three figures show the volume ratio of the longitudinal mound to dike expansion for the longitudinal mound crest width and crest height at location Waal_07.

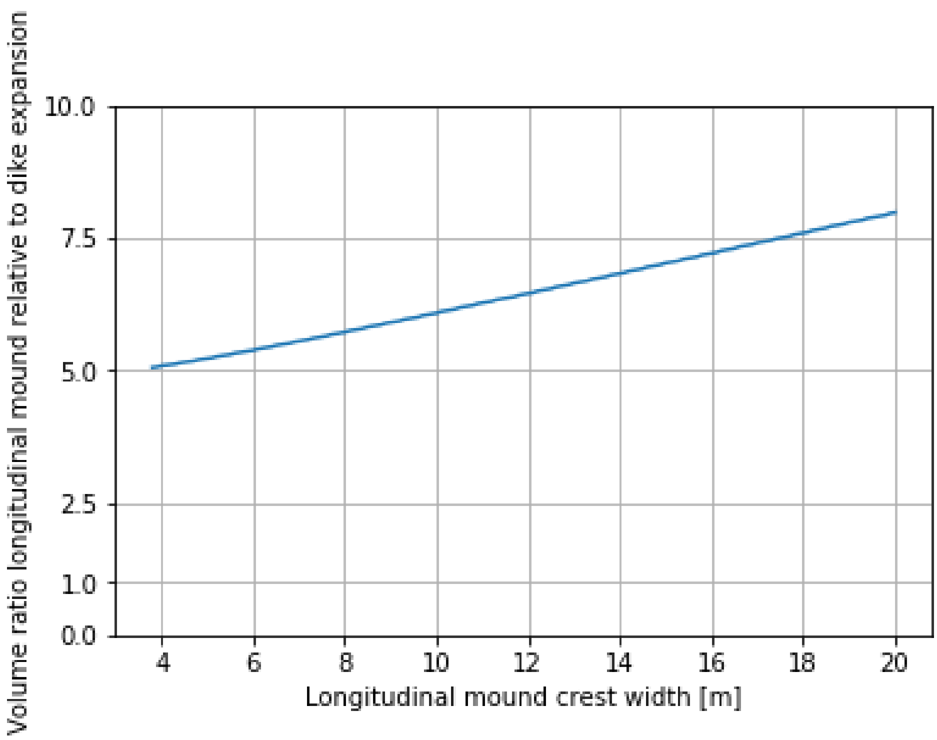

Figure D-4 Volume ratio based on crest width of longitudinal mound for location Waal_07

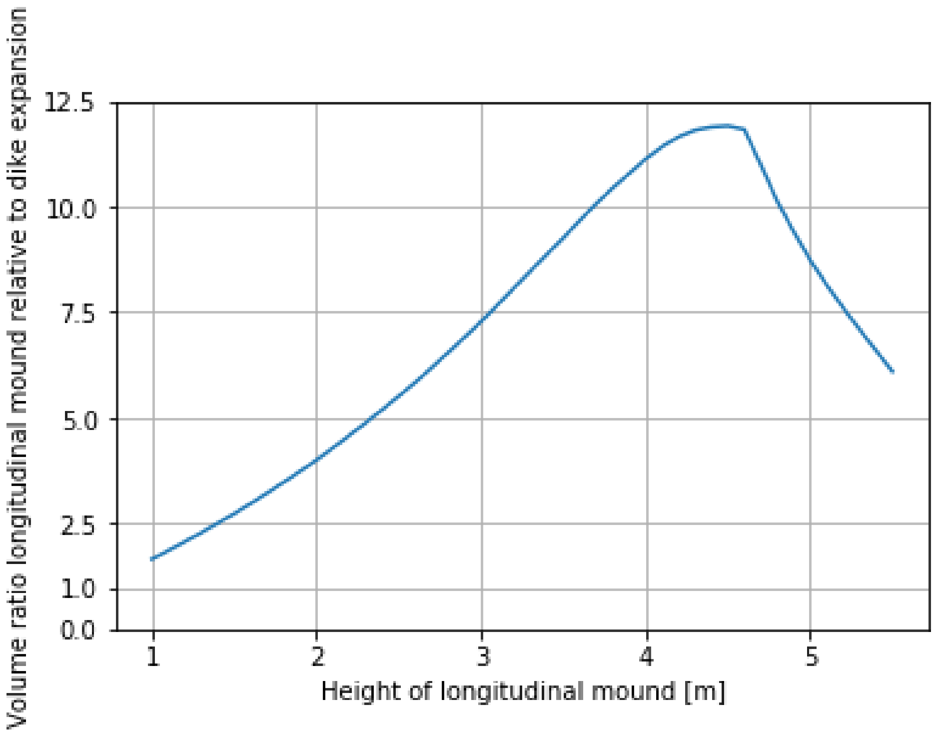

Figure D-5 Volume ratio based on crest height of longitudinal mound for location Waal_07

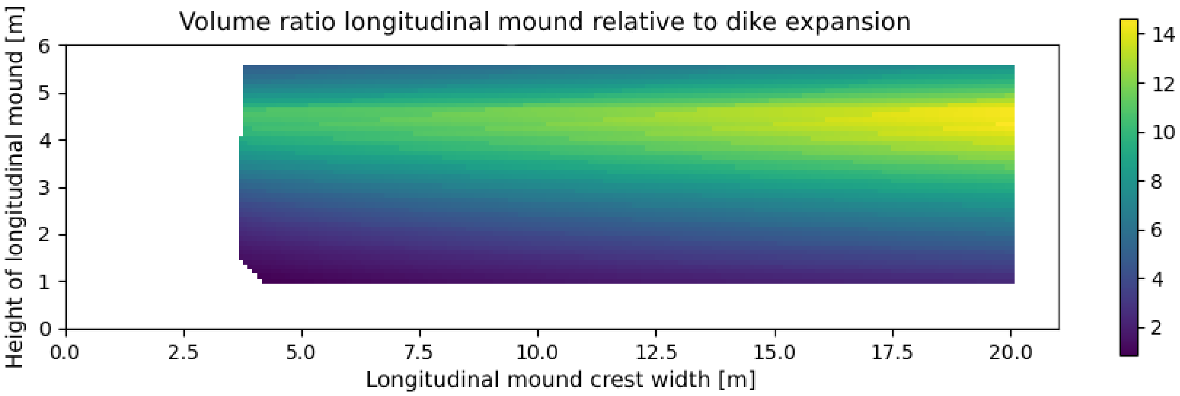

Figure D-6 Volume ratio between longitudinal mound and dike expansion for all combinations of the longitudinal mound for location Waal_07

The following three figures show the dike crest height reduction for the longitudinal mound crest width and crest height at location Lek_05.

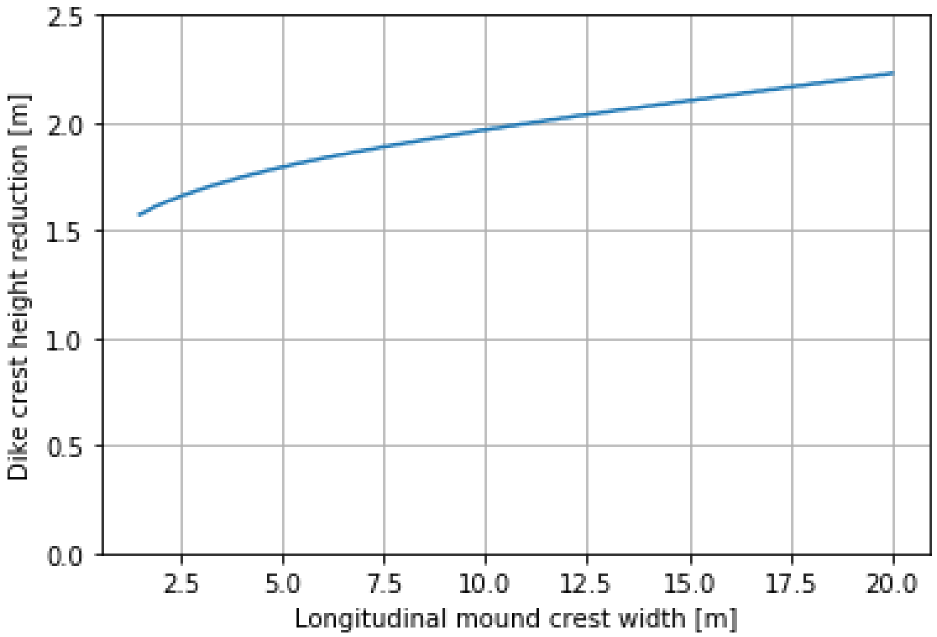

Figure D-7 Dike crest height reduction based on crest width of longitudinal mound for location Lek_05

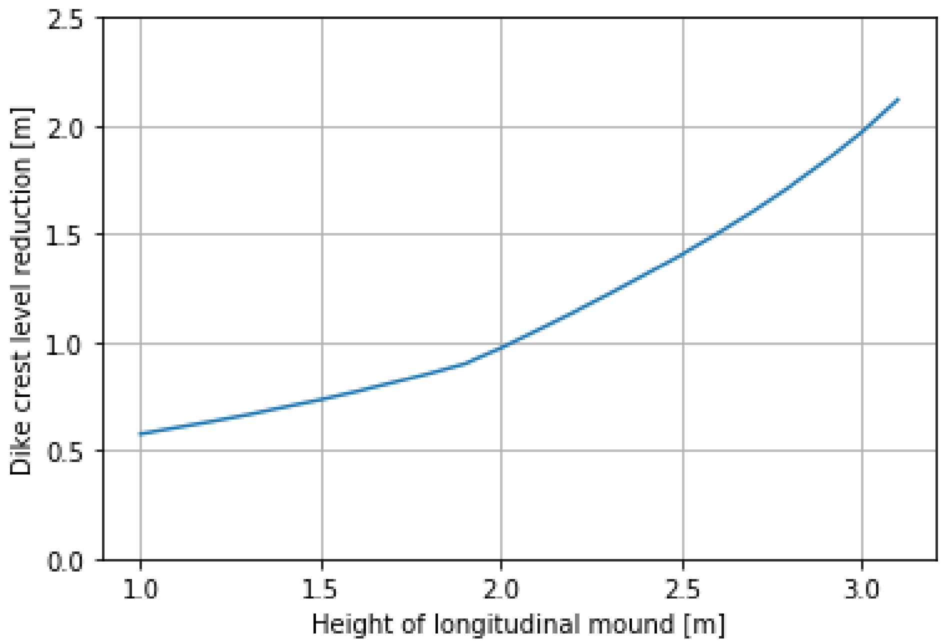

Figure D-8 Dike crest height reduction based on crest height of longitudinal mound for location Lek_05

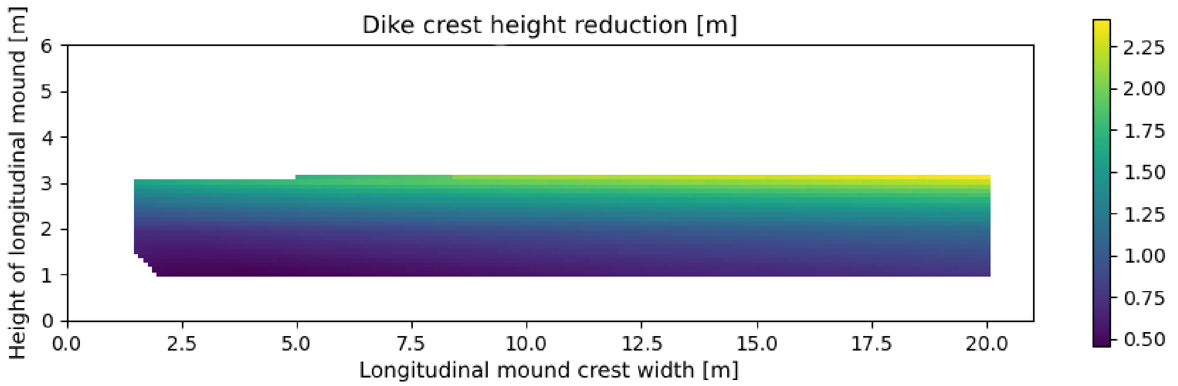

Figure D-9 Dike crest height in metres for all combination of width and length of the longitudinal mound for location Lek_05

The following three figures show the volume ratio of the longitudinal mound to dike expansion for the longitudinal mound crest width and crest height at location Lek_05.

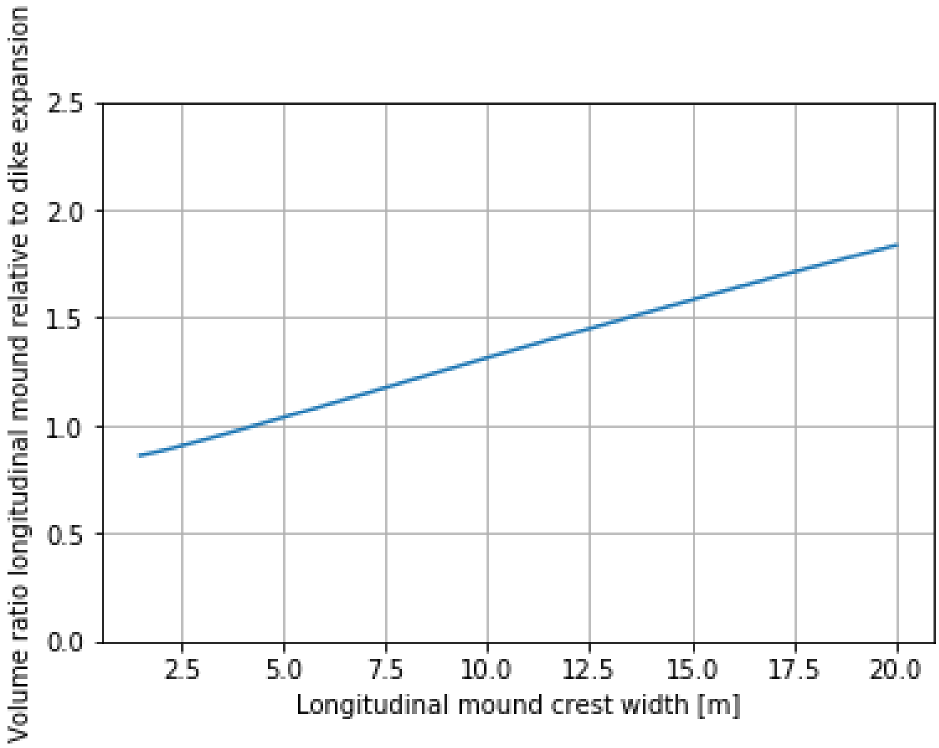

Figure D-10 Volume ratio based on crest width of longitudinal mound for location Lek_05

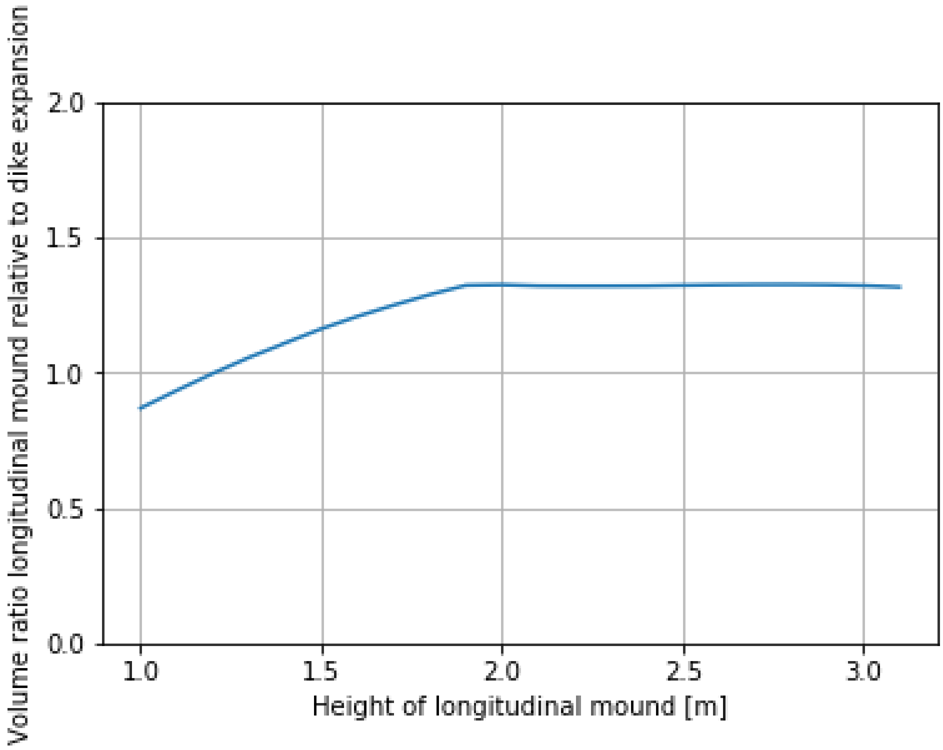

Figure D-11 Volume ratio based on crest height of longitudinal mound for location Lek_05

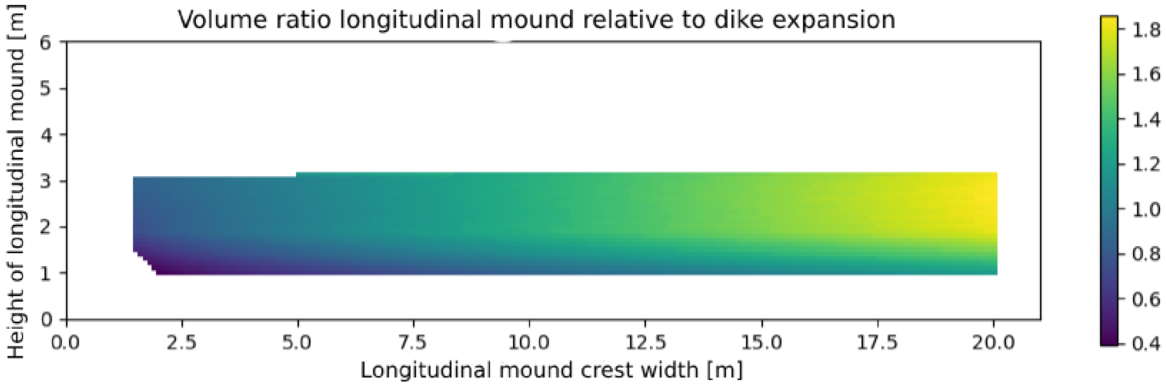

Figure D-12 Volume ratio between longitudinal mound and dike expansion for all combinations of the longitudinal mound for location Lek_05

The following three figures show the dike crest height reduction for the longitudinal mound crest width and crest height at location Rijn_01.

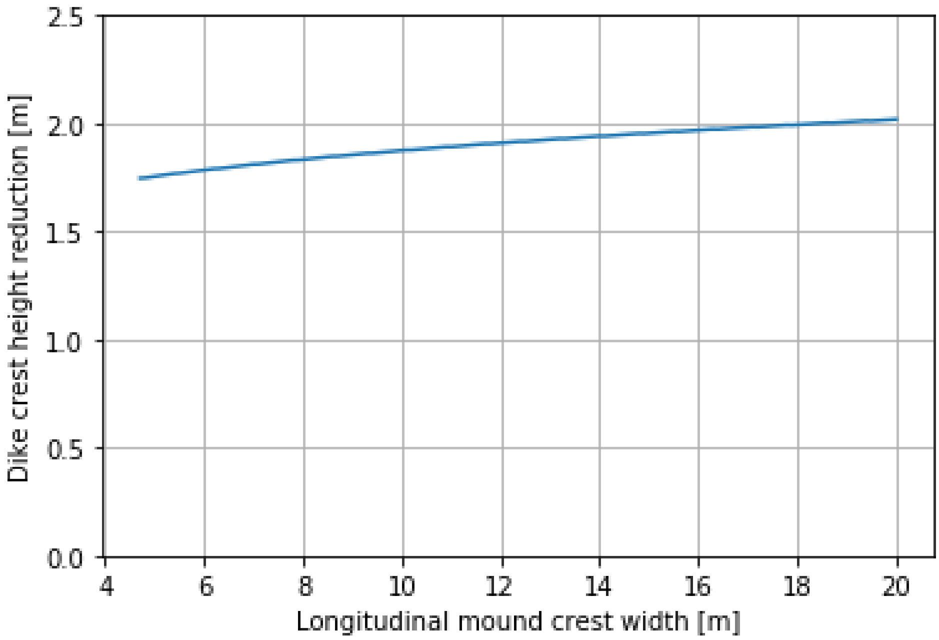

Figure D-13 Dike crest height reduction based on crest width of longitudinal mound for location Rijn_01

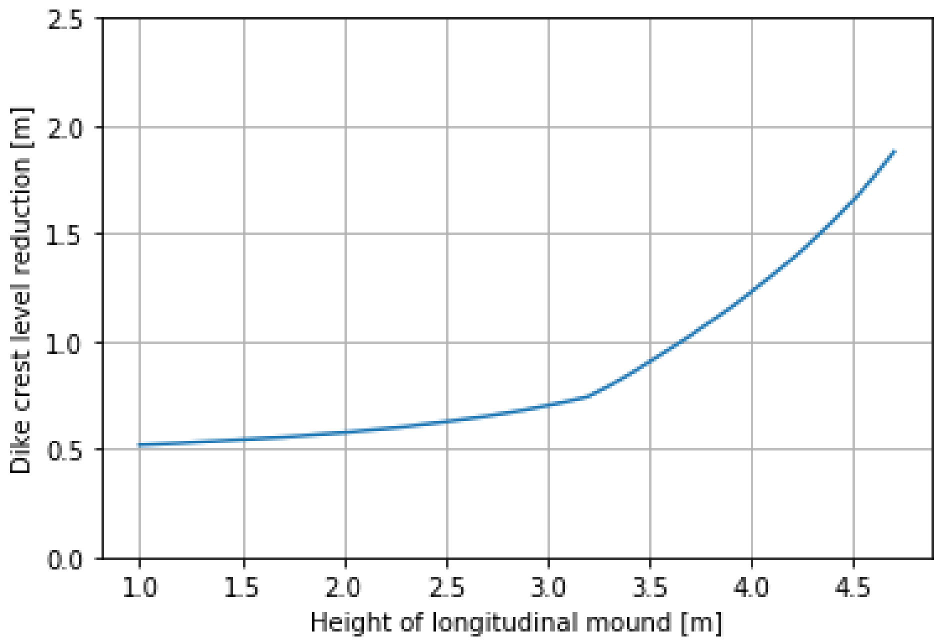

Figure D-14 Dike crest height reduction based on crest height of longitudinal mound for location Rijn_01

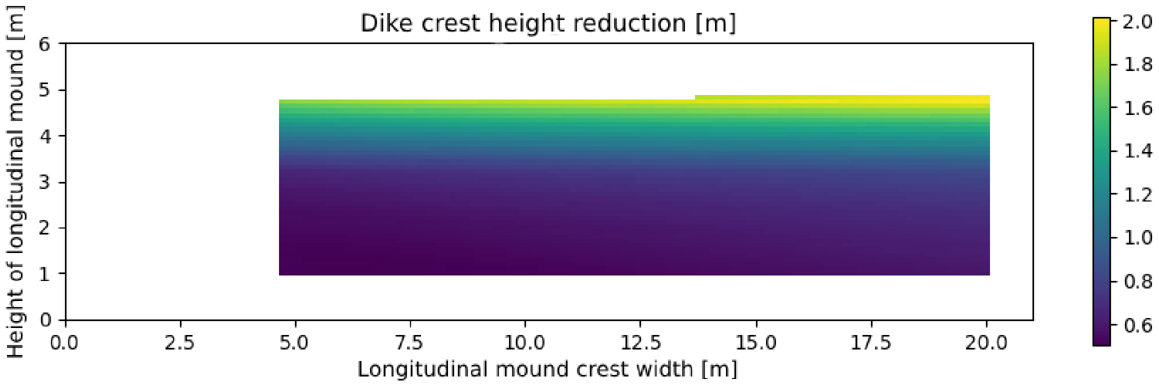

Figure D-15 Dike crest height in metres for all combination of width and length of the longitudinal mound for location Rijn_01

The following three figures show the volume ratio of the longitudinal mound to dike expansion for the longitudinal mound crest width and crest height at location Rijn_01.

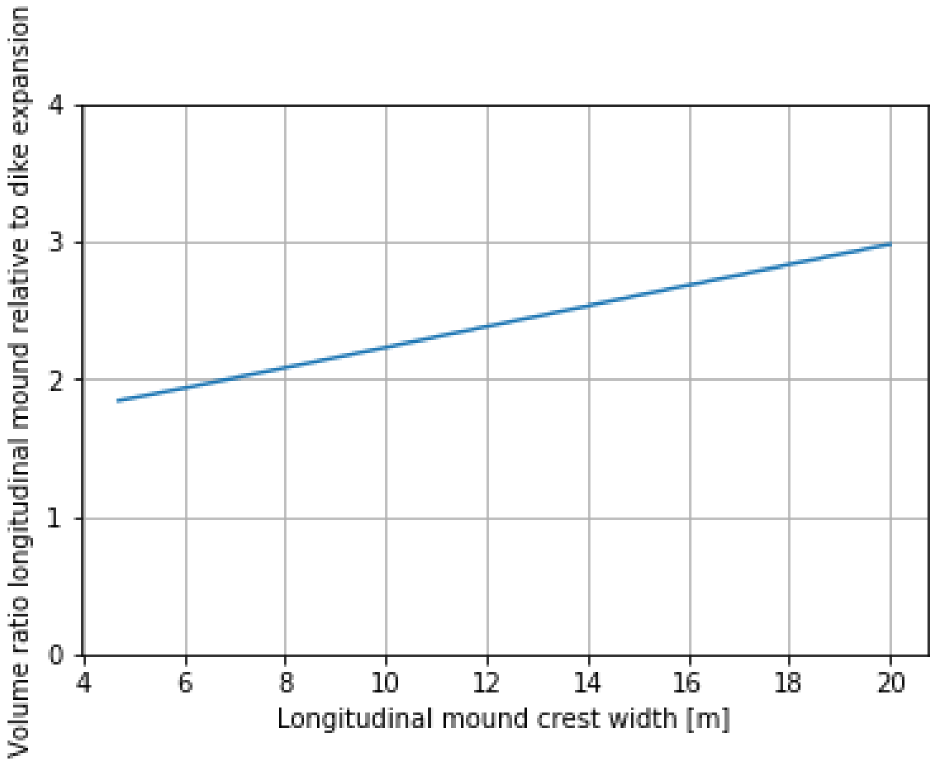

Figure D-16 Volume ratio based on crest width of longitudinal mound for location Rijn_01

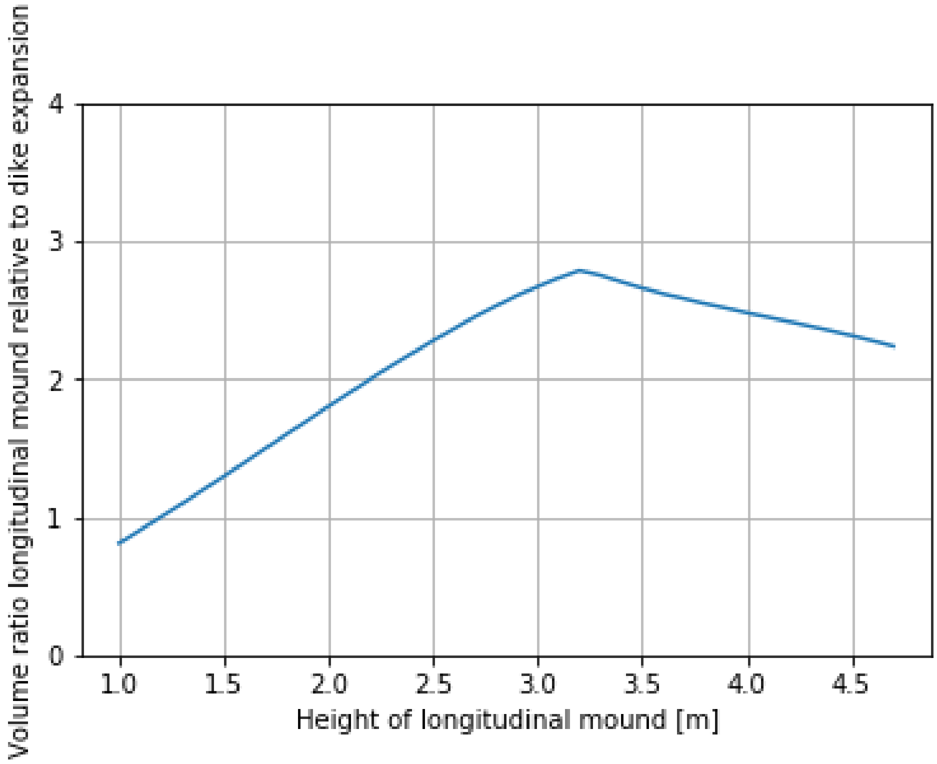

Figure D-17 Volume ratio based on crest height of longitudinal mound for location Rijn_01

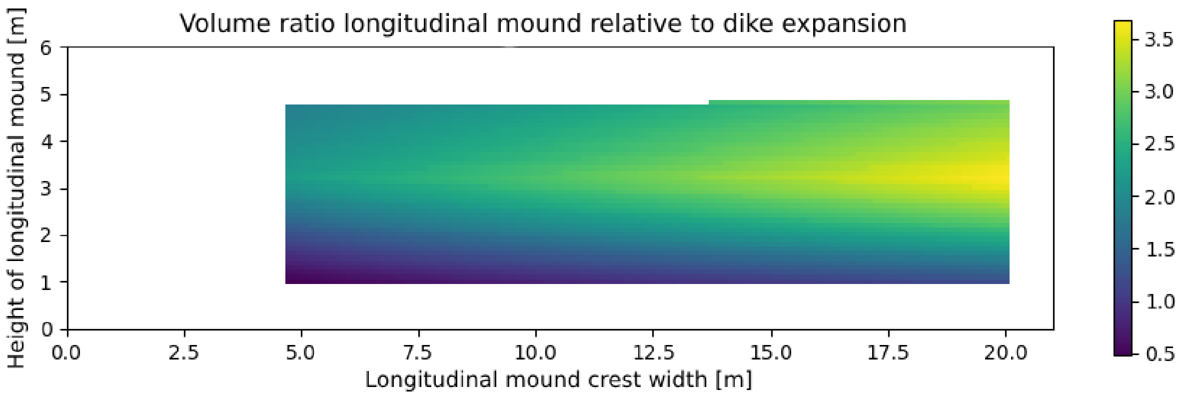

Figure D-18 Volume ratio between longitudinal mound and dike expansion for all combinations of the longitudinal mound for location Rijn_01

The following three figures show the dike crest height reduction for the longitudinal mound crest width and crest height at location Waal_09.

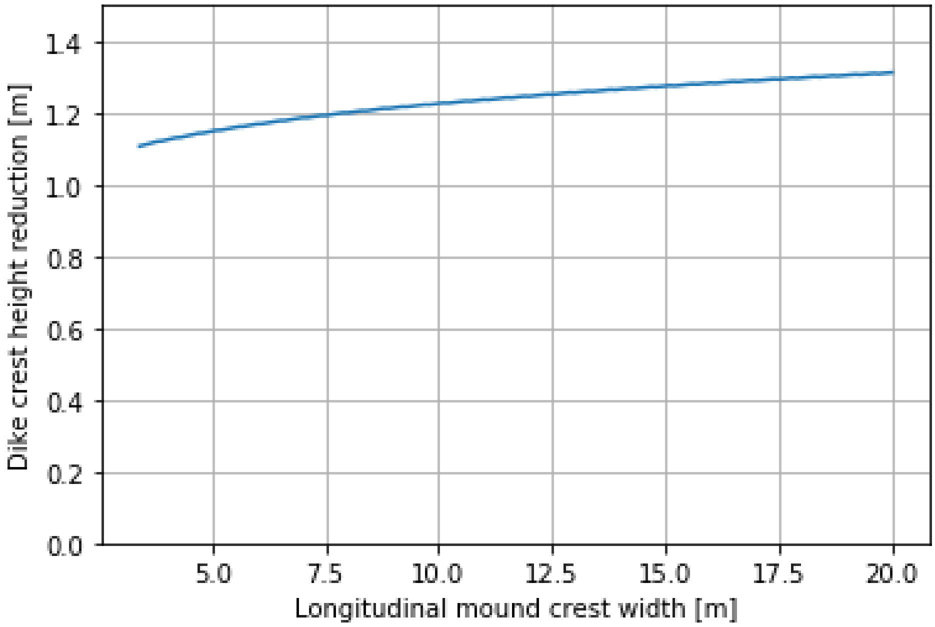

Figure D-19 Dike crest height reduction based on crest width of longitudinal mound for location Waal_09

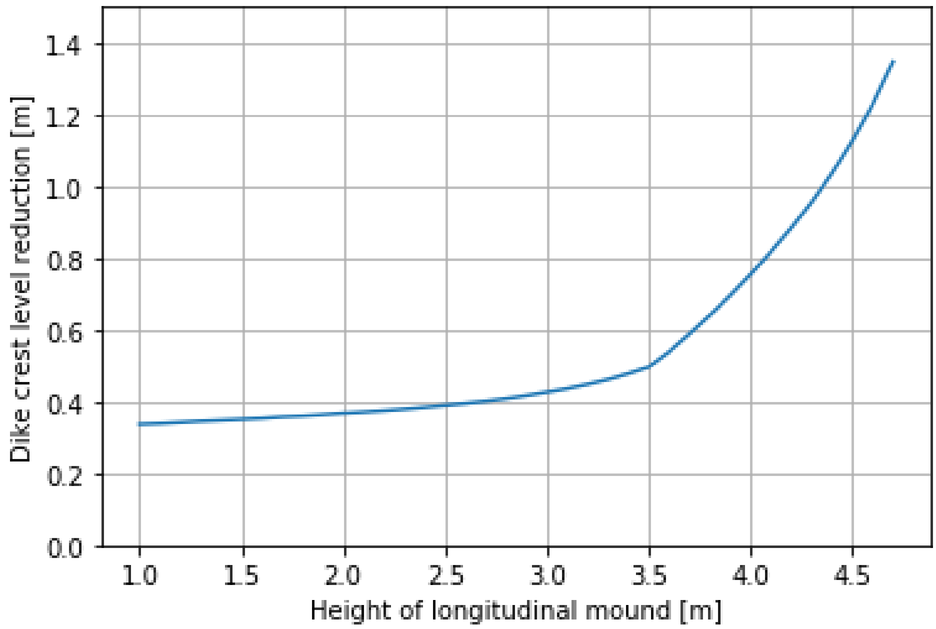

Figure D-20 Dike crest height reduction based on crest height of longitudinal mound for location Waal_09

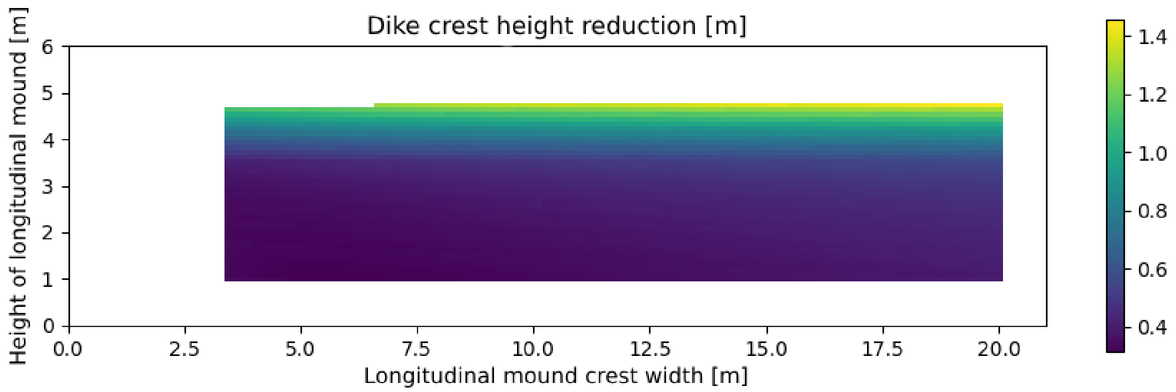

Figure D-21 Dike crest height in metres for all combination of width and length of the longitudinal mound for location Waal_09

The following three figures show the volume ratio of the longitudinal mound to dike expansion for the longitudinal mound crest width and crest height at location Waal_09.

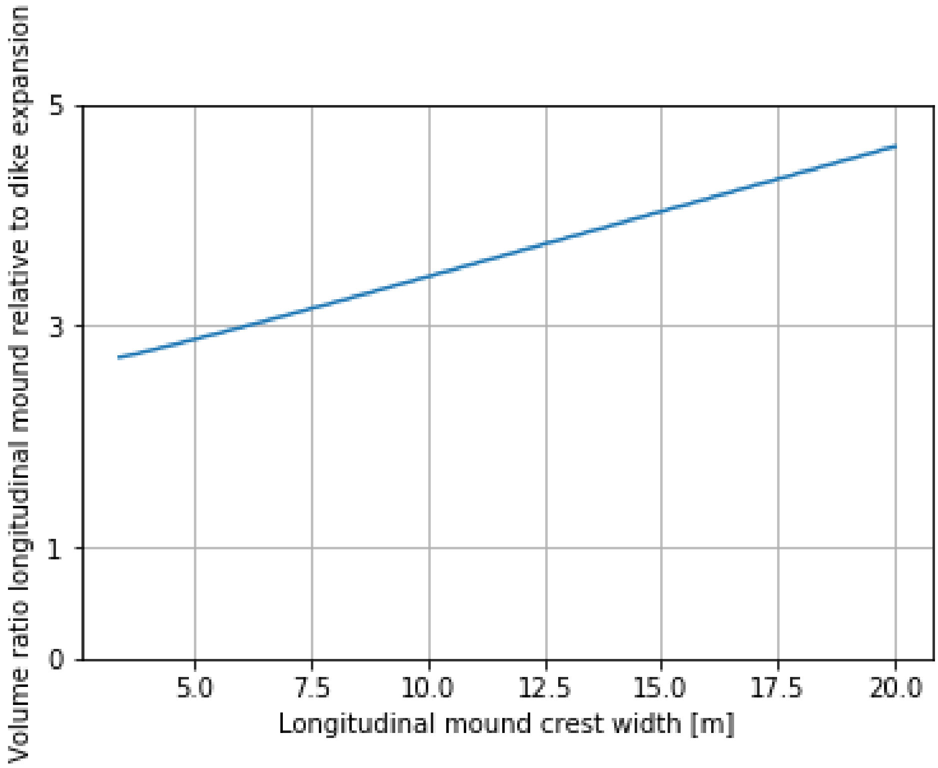

Figure D-22 Volume ratio based on crest width of longitudinal mound for location Waal_09

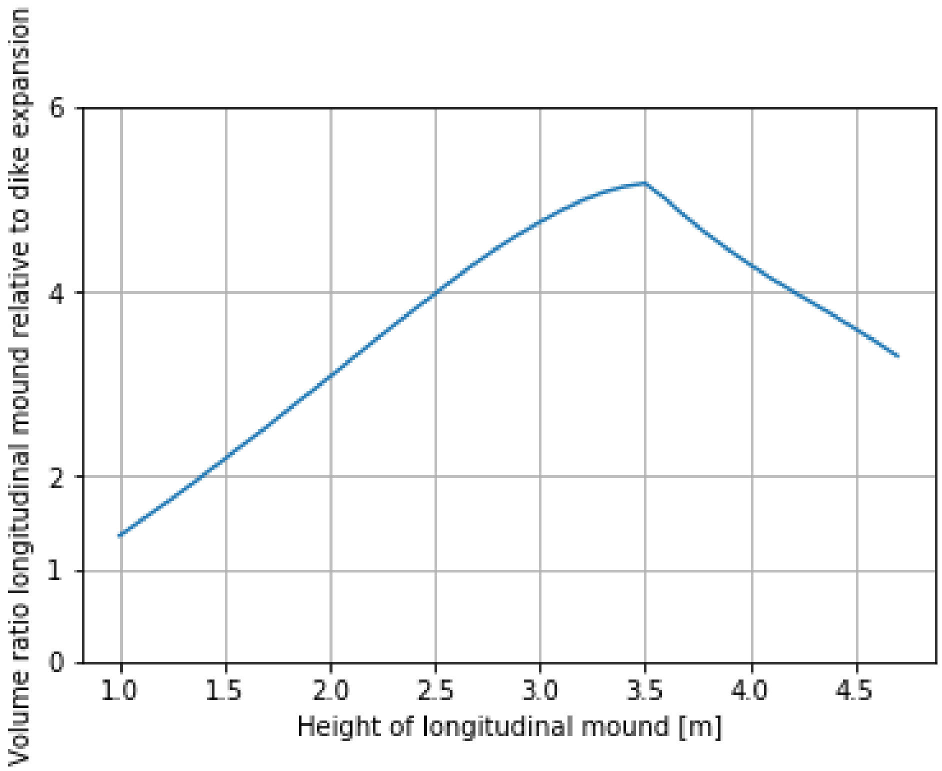

Figure D-23 Volume ratio based on crest height of longitudinal mound for location Waal_09

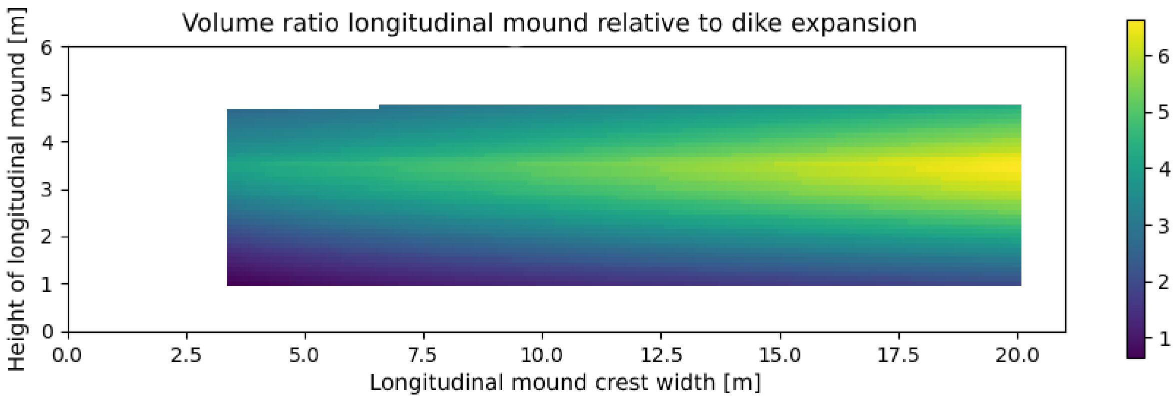

Figure D-24 Volume ratio between longitudinal mound and dike expansion for all combinations of the longitudinal mound for location Waal_09

The following three figures show the dike crest height reduction for the longitudinal mound crest width and crest height at location Maas_06.

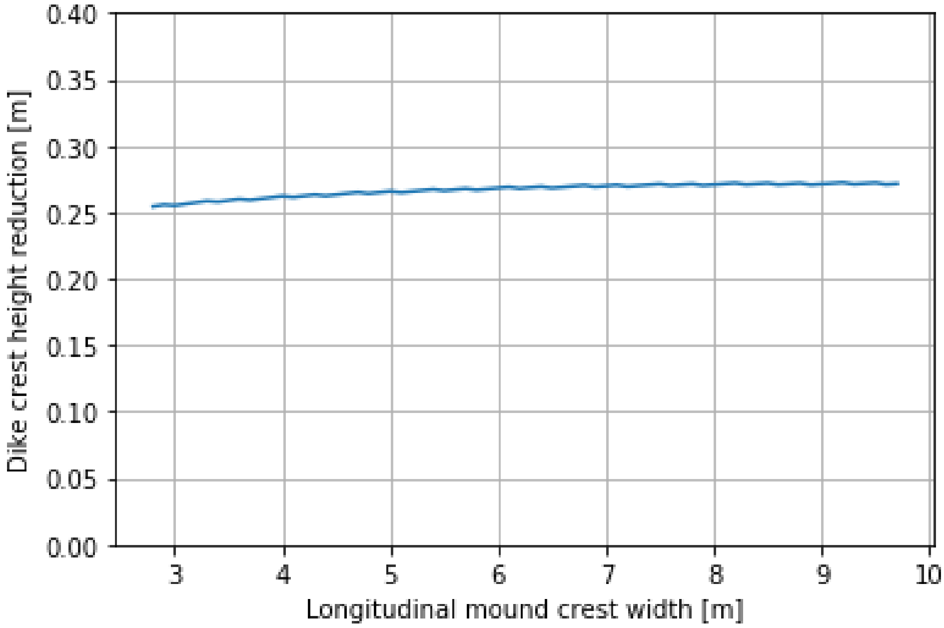

Figure D-25 Dike crest height reduction based on crest width of longitudinal mound for location Maas_06

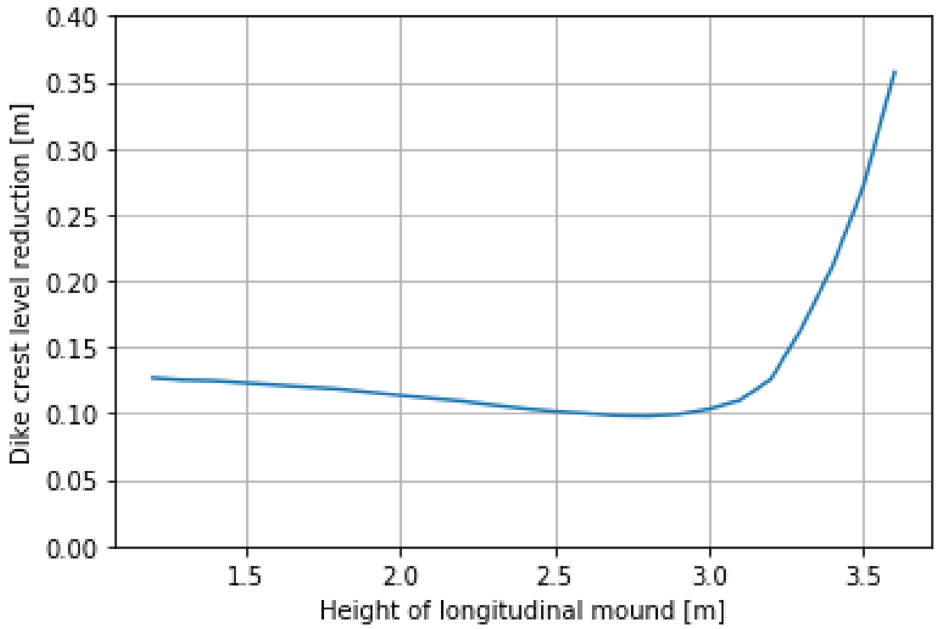

Figure D-26 Dike crest height reduction based on crest height of longitudinal mound for location Maas_06

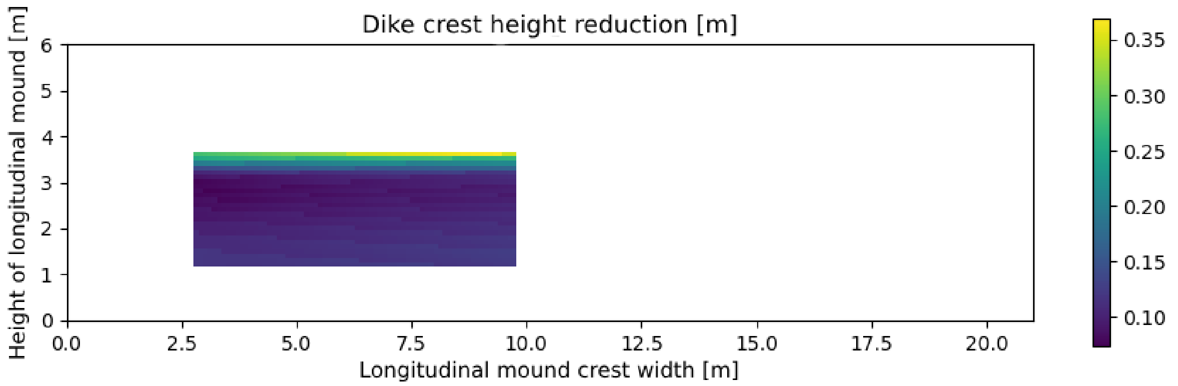

Figure D-27 Dike crest height in metres for all combination of width and length of the longitudinal mound for location Maas_06

The following three figures show the volume ratio of the longitudinal mound to dike expansion for the longitudinal mound crest width and crest height at location Maas_06.

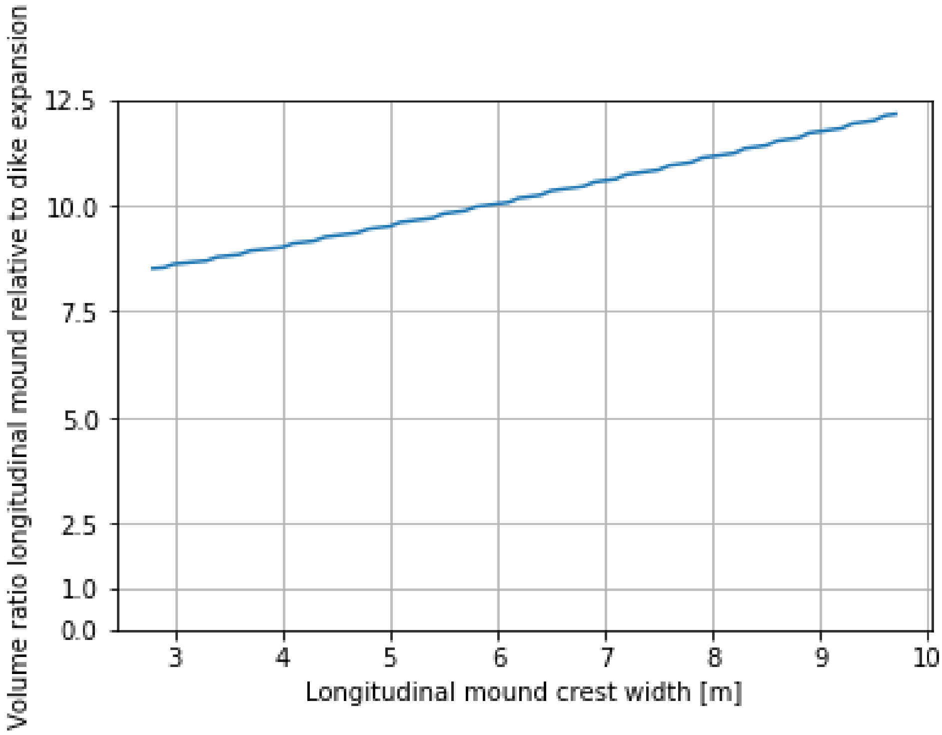

Figure D-28 Volume ratio based on crest width of longitudinal mound for location Maas_06

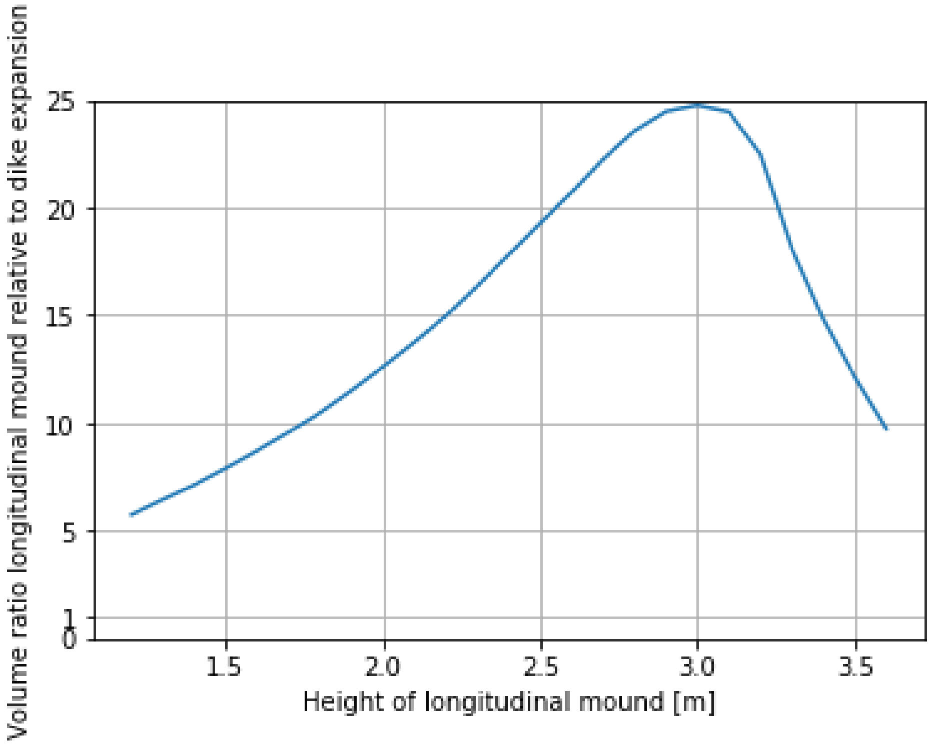

Figure D-29 Volume ratio based on crest height of longitudinal mound for location Maas_06

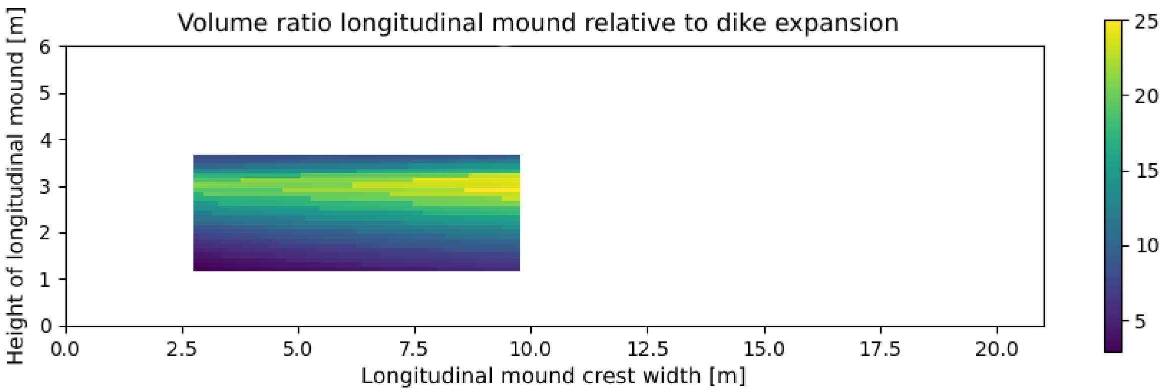

Figure D-30 Volume ratio between longitudinal mound and dike expansion for all combinations of the longitudinal mound for location Maas_06

The following four figures show the dike crest height reduction and the volume ratio of the longitudinal mound to dike expansion for the longitudinal mound crest width and height at all five locations.

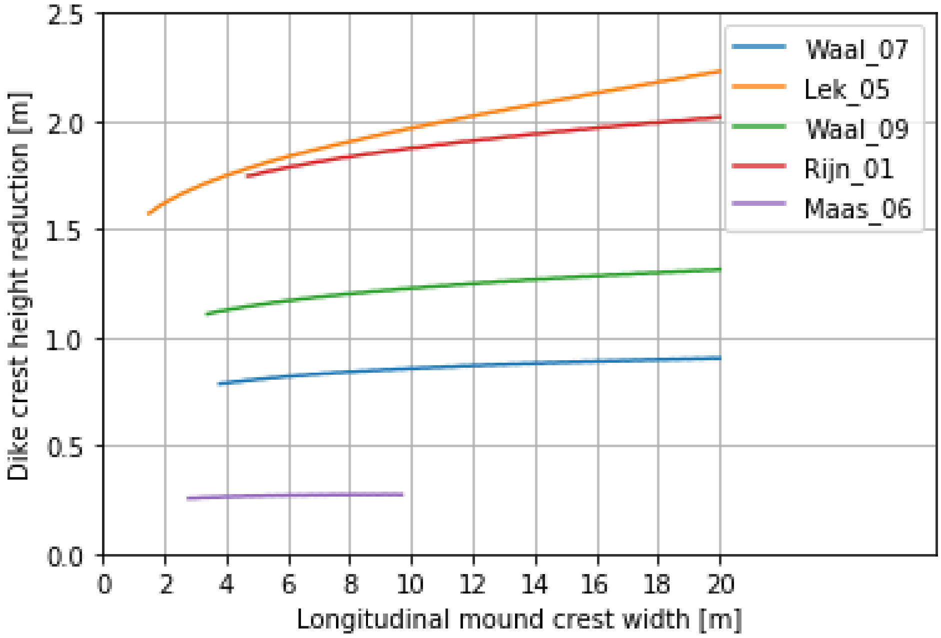

Figure D-31 Dike crest height reduction based on crest height of longitudinal mound for all locations

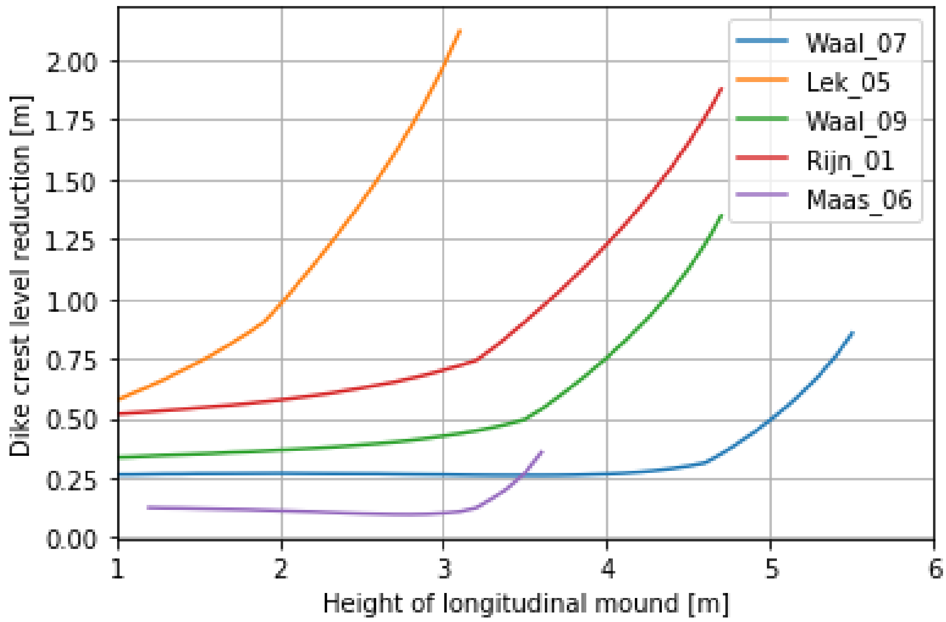

Figure D-32 Dike crest height reduction based on crest height of longitudinal mound for all locations

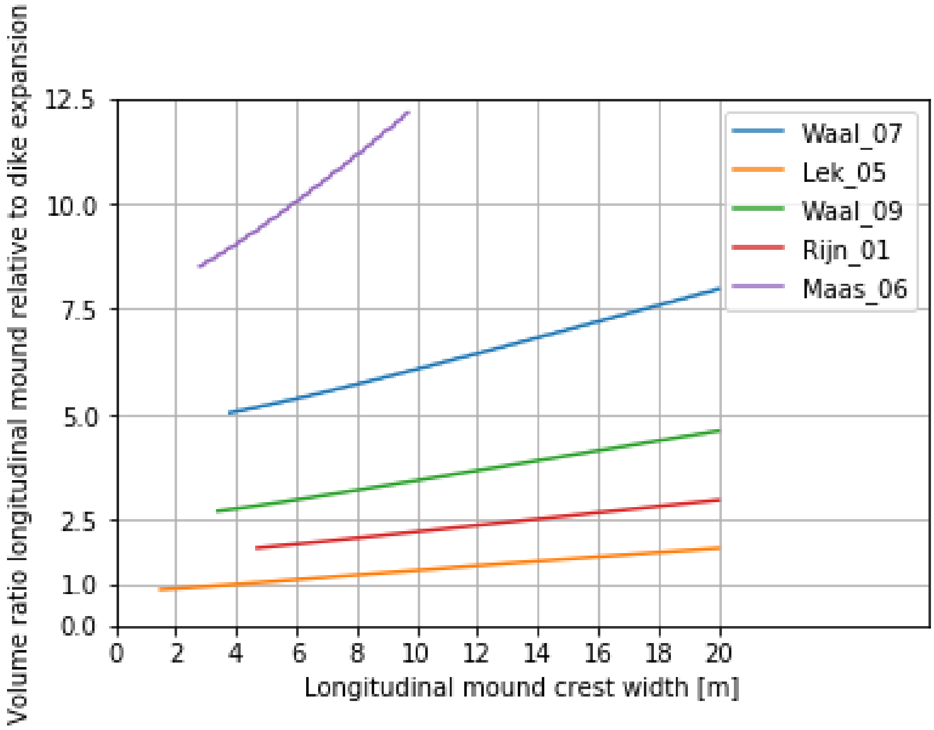

Figure D-33 Volume ratio based on crest width of longitudinal mound for all locations

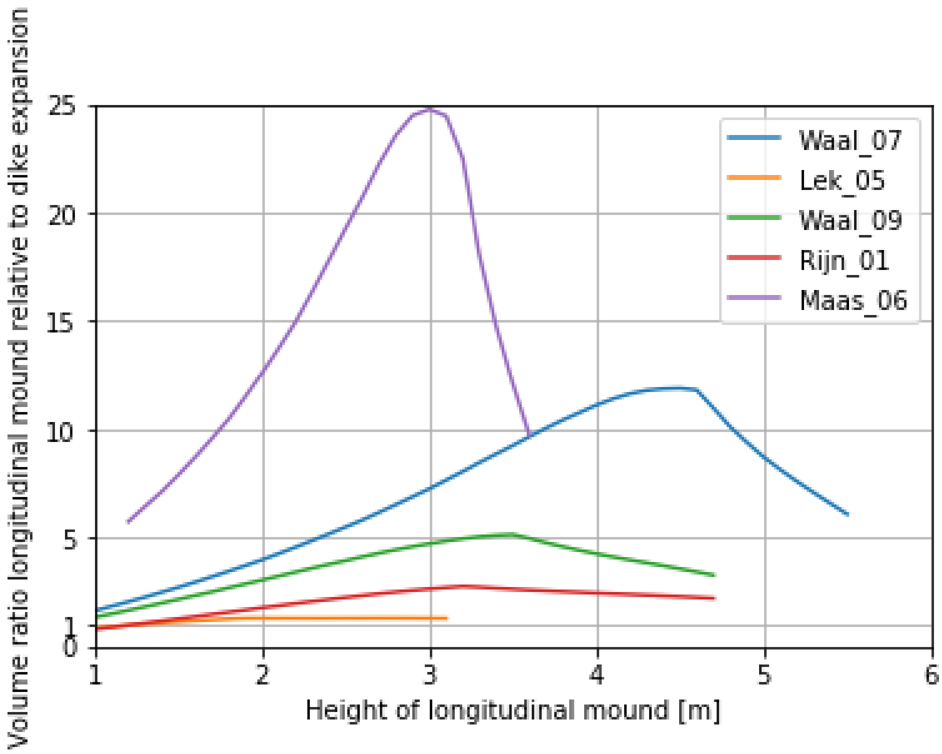

Figure D-34 Volume ratio based on crest height of longitudinal mound for all locations

In the following two figures the difference between the calculation with water level and water depth are shown on the dike crest height reduction and volume ratio based on the longitudinal mound crest width at location Waal_07. With the correct water depth the effect of the longitudinal mound crest width is larger. Which also results in a reduced volume ratio.

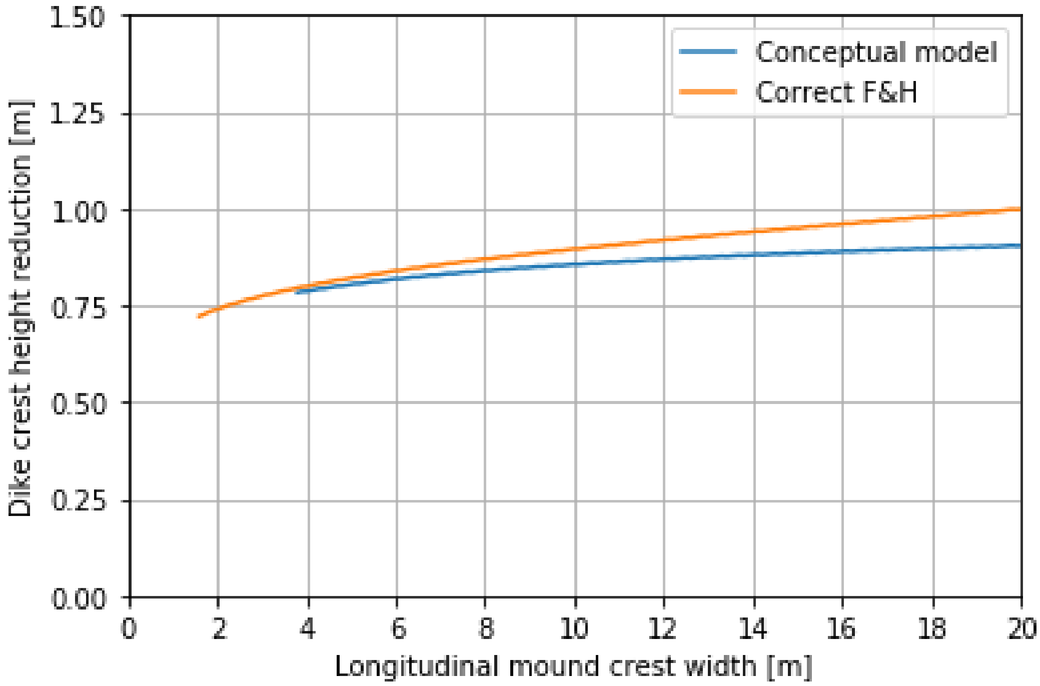

Figure D-35 Difference between the conceptual model and the correct Friebel and Harris formula

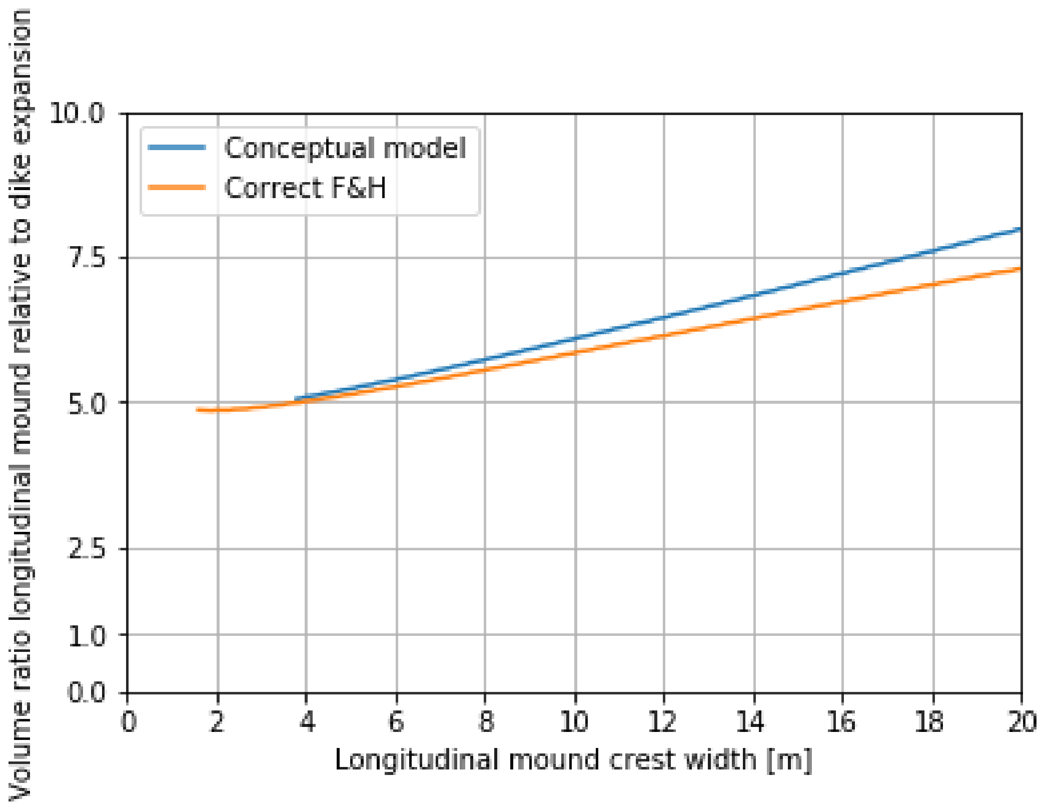

Figure D-36 Difference between the conceptual model and the correct Friebel and Harris formula

Appendix E – Python code making .cll and .arl files

Below the full Python code for making the .cll and .arl files correct for the new refined grid is shown. The conceptual model has been made in Spyder 4.0.1. using Python 3.7.6. To make longer lines of code readable in the code below the lines of code that did not fit on one line have been altered by adding enters and tabs. So, if copying this code be aware of that.

```
# -*- coding: utf-8 -*-
"""
@author: Rokus de Bie
"""

import numpy as np
import matplotlib.pyplot as plt
from sklearn.neighbors import KDTree
from collections import Counter

#%%
xyz = np.loadtxt("Filepath\xy_x_grid_5x5.xyz")      #xyz of refined grid
xyz20 = np.loadtxt("Filepath\xyx_grid_20x20.xyz")  #xyz of original grid
cll20 = np.loadtxt("Filepath\calibration.txt")     #.cll file of original grid
arl20 = np.loadtxt("Filepath\trachytopes.txt")     #.arl file of original grid
xy5 = xyz[:,:2]                                     #xy-coordinates of refined grid
xy20 = xyz20[:,:2]                                  #xy-coordinates of original grid

#%%
#Load smaller area and lookup table, made later in this script. Replaces blocks X,
Y and Z to reduce computing time when revisiting the script
xy5_local_extended = np.loadtxt("R:\\Rokus\\Documents\\Laptop Sweco\\30
April\\.spyder-py3\\aa_xy5_local_extended.txt")
cll_local  = np.loadtxt('Filepath\aa_cll_local.txt')
arl_local  = np.loadtxt('Filepath\aa_arl_local.txt')
xy20_local = np.loadtxt('Filepath\aa_xy20_local.txt')
xy5_local  = np.loadtxt('Filepath\aa_xy5_local.txt')

#%%
#area of grid refinement
xmin = 171975
xmax = 176130
ymin = 434580
ymax = 435694

#%% X1
#Reduce size of the four files to only the area of grid refinement
cll_local = np.zeros([1,5])
for i in range(len(cll20)):
    print('cll=',i)
    if cll20[i,0] > xmin and cll20[i,0] < xmax and cll20[i,1] > ymin and cll20[i,1]
       < ymax:
        cll_local = np.append(cll_local,[cll20[i,:]],axis=0)

arl_local = np.zeros([1,5])
for i in range(len(arl20)):
    print('arl=',i)
    if arl20[i,0] > xmin and arl20[i,0] < xmax and arl20[i,1] > ymin and arl20[i,1]
       < ymax:
        arl_local = np.append(arl_local,[arl20[i,:]],axis=0)

xy20_local = np.zeros([1,2])
for i in range(len(xy20)):
    print('xy20=',i)
    if xy20[i,0] > xmin and xy20[i,0] < xmax and xy20[i,1] > ymin and xy20[i,1]
       < ymax:
        xy20_local = np.append(xy20_local,[xy20[i,:]],axis=0)
```

```
xy5_local = np.zeros([1,2])
for i in range(len(xy5)):
    print('xy5=',i)
    if xy5[i,0] > xmin and xy5[i,0] < xmax and xy5[i,1] > ymin and xy5[i,1] < ymax:
        xy5_local = np.append(xy5_local,[xy5[i,:]],axis=0)

#%% X2
#remove first row
cll_local  = np.delete(cll_local,0,axis=0)
arl_local  = np.delete(arl_local,0,axis=0)
xy20_local = np.delete(xy20_local,0,axis=0)
xy5_local  = np.delete(xy5_local,0,axis=0)

#%% Z1
#np.savetxt('Filepath\aa_cll_local.txt',cll_local)
#np.savetxt('Filepath\aa_arl_local.txt',arl_local)
#np.savetxt('Filepath\aa_xy20_local.txt',xy20_local)
#np.savetxt('Filepath\aa_xy5_local.txt',xy5_local)

#%% Y1
#Nearest neighbours with KDTree
kdt = KDTree(xy20_local, leaf_size=40, metric='euclidean')
indices = kdt.query(xy5_local, k=1, return_distance=False) #returns indices of
                                                            nearest neighbour

#%% Y2
#Check how many times each index is a nearest neighbour
indices_list = list(map(tuple, indices))
count_indices = Counter(indices_list)
print(count_indices)

#%% Y3
#Couple xy-coordinates of oroginal grid with its index as array
xyI20_local = np.zeros([len(xy20_local),3])
xyI20_local[:,0:2] = xy20_local
xyI20_local[:,2] = np.arange(len(xy20_local))

#%% Y4
#Construct lookup table for all new grid coordinates with the original grid
coordinates
xy5_local_extended = np.zeros([len(xy5_local),6])
xy5_local_extended[:,0:2] = xy5_local                 #Add new grid coordinates
xy5_local_extended[:,2] = np.arange(len(xy5_local))  #Add indices of new grid
xy5_local_extended[:,3] = indices[:,0]                #Add nearest neighbour indices
                                                       of original grid
#Search for original indices and add coordinates
for i in range(len(xy5_local)):
    print(i)
    for j in range(len(xy20_local)):
        if xy5_local_extended[i,3] == xyI20_local[j,2]:
            xy5_local_extended[i,4:6] = xyI20_local[j,0:2]

#%% Z2
#Save lookup table
#np.savetxt('Filepath\aa_xy5_local_extended.txt',xy5_local_extended)

#%%
#Make new local .cll and .arl files
cll_local_final_2 = np.zeros([1,5])
arl_local_final_2 = np.zeros([1,5])

for i in range(len(xy5_local_extended)):
    print('i=',i)
    for j in range(len(cll_local)): #cll_local
        if xy5_local_extended[i,4] == cll_local[j,0]:
            cll_local_final_2 =
              np.append(cll_local_final_2,[cll_local_final_2[0,:]],axis=0)
            cll_local_final_2[-1,0:2] = xy5_local_extended[i,0:2]
```

```
                cll_local_final_2[-1,2:5] = cll_local[j,2:5]


        for k in range(len(arl_local)): #arl_local
            if xy5_local_extended[i,4] == arl_local[k,0]:
                arl_local_final_2 =
                  np.append(arl_local_final_2,[arl_local_final_2[0,:]],axis=0)
                arl_local_final_2[-1,0:2] = xy5_local_extended[i,0:2]
                arl_local_final_2[-1,2:5] = arl_local[k,2:5]

#%%
cll_local_final_3 = np.delete(cll_local_final_2,0,axis=0)
arl_local_final_3 = np.delete(arl_local_final_2,0,axis=0)
#%%
#save new local .cll and .arl files
np.savetxt('Filepath\aa_cll_local_final_3.txt',cll_local_final_3)
np.savetxt('Filepath\aa_arl_local_final_3.txt',arl_local_final_3)

#%%
#Remove local points from the original .cll and .arl files
CLL5 = cll20
#Remove locations inside of refinement area for new .cll file
CLL5 = np.delete(CLL5,np.where((cll20[:,0] > xmin) & (cll20[:,0] < xmax) &
       (cll20[:,1] > ymin) & (cll20[:,1] < ymax)),axis=0)
CLL5_file=CLL5
ARL5 = arl20
#Remove locations inside of refinement area for new .arl file
ARL5 = np.delete(ARL5,np.where((arl20[:,0] > xmin) & (arl20[:,0] < xmax) &
       (arl20[:,1] > ymin) & (arl20[:,1] < ymax)),axis=0)
ARL5_file=ARL5

#%%
#Add new local .cll file to the reduced original .cll file
CLL5_combined = CLL5_file
for i in range(len(cll_local_final_3)):
    CLL5_combined = np.append(CLL5_combined,[cll_local_final_3[i,:]],axis=0)

CLL5_combined_s = CLL5_combined[np.argsort(CLL5_combined[:,0])] #sort on x-
                                                                   coordinate
#%%
#Add new local .arl file to the reduced original .arl file
ARL5_combined = ARL5_file
for i in range(len(arl_local_final_3)):
    print(i)
    ARL5_combined = np.append(ARL5_combined,[arl_local_final_3[i,:]],axis=0)

ARL5_combined_s = ARL5_combined[np.argsort(ARL5_combined[:,0])] #sort on x-
                                                                   coordinate
#%% control
cll20_s = np.sort(cll20,axis=0)
arl20_s = np.sort(arl20,axis=0)


#%%Save sorted .cll and .arl files
np.savetxt('Filepath\aa_cll_combined_s.txt',CLL5_combined_s)
np.savetxt('Filepath\aa_arl_combined_s.txt',ARL5_combined_s)

#%%Save sorted .cll and .arl files with correct decimals and integers for D-Flow FM
np.savetxt('Filepath\rijn-beno19_6_local5x5m_waal-v2a_calibration.cll',
      CLL5_combined_s, fmt = "%6.6f %6.6f %1.1f %i %1.6f")
np.savetxt('Filepath\rijn-beno19_6_local5x5m_waal-v2a_trachytopes.arl',
      ARL5_combined_s, fmt = "%6.6f %6.6f %1.1f %i %1.6f")
```

Appendix F – All results D-Flow FM

The figures in this appendix have been made with Quickplot. The colour bars on the right hand side of the figures start at the lowest value and end at the highest value. These maxima are not necessarily on the area shown in the figures. Often they occur as a result of one cell where a larger difference is found. The most extreme cases have been filtered. The maxima are not equal in positive and negative direction and not equal in the different figures. Therefore, 0 is generally not in the middle of the colour bar and is different for the different figures. So, the colours are not on the same scale between the figures.

The following six figures show the water level differences between the three variants and the original variant, variant 0. Also, the differences in water level between the three variants itself are shown. A selection of these figures have been shown in Chapter 4.

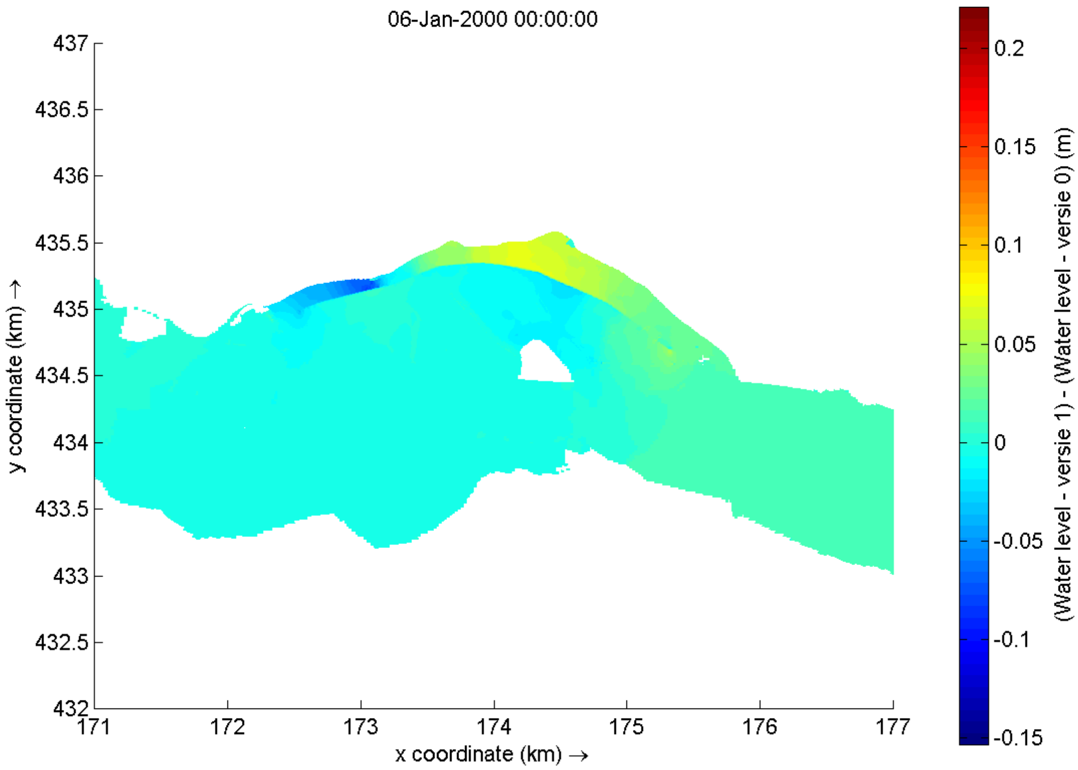

Figure F-1 Water level differences between the original situation and variant 1. Red indicates a water level increase and dark blue a water level decrease

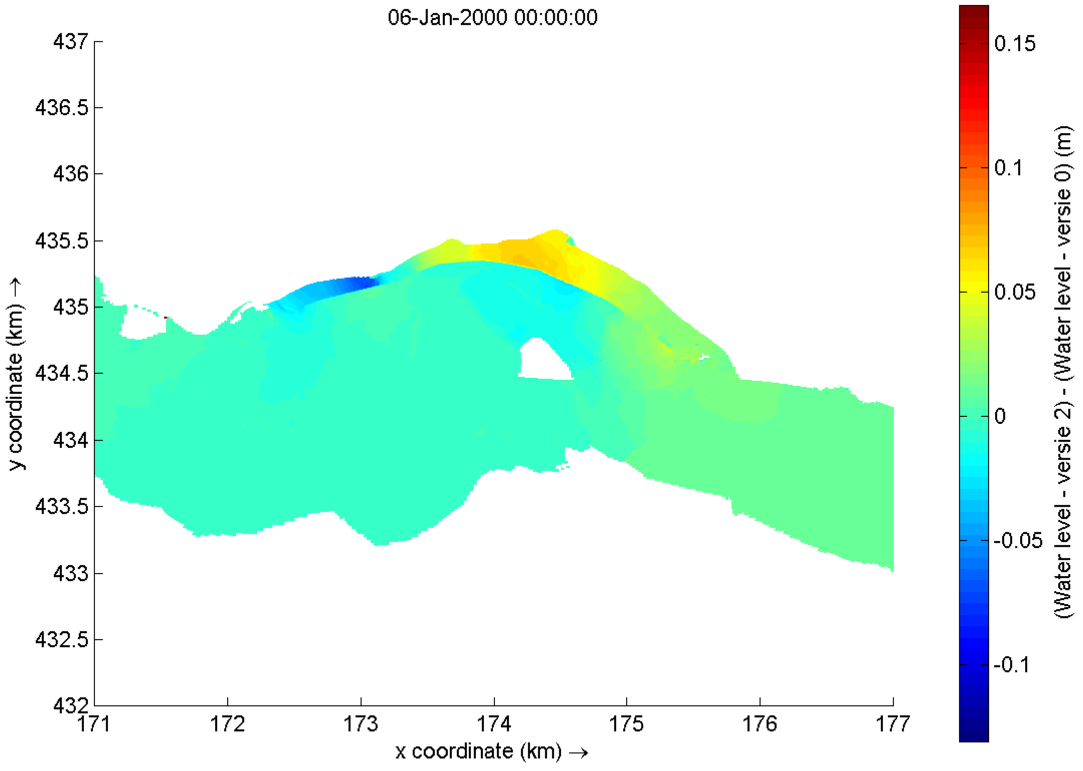

Figure F-2 Water level differences between the original situation and variant 2. Red indicates a water level increase and dark blue a water level decrease

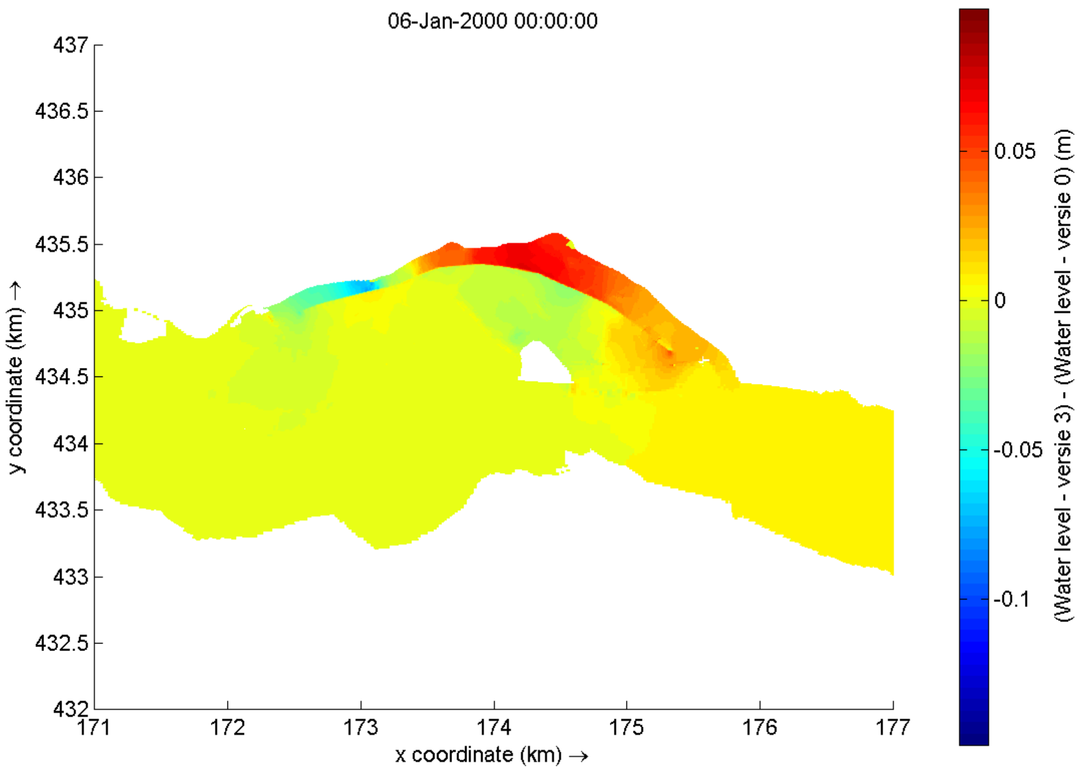

Figure F-3 Water level differences between the original situation and variant 3. Red indicates a water level increase and dark blue a water level decrease

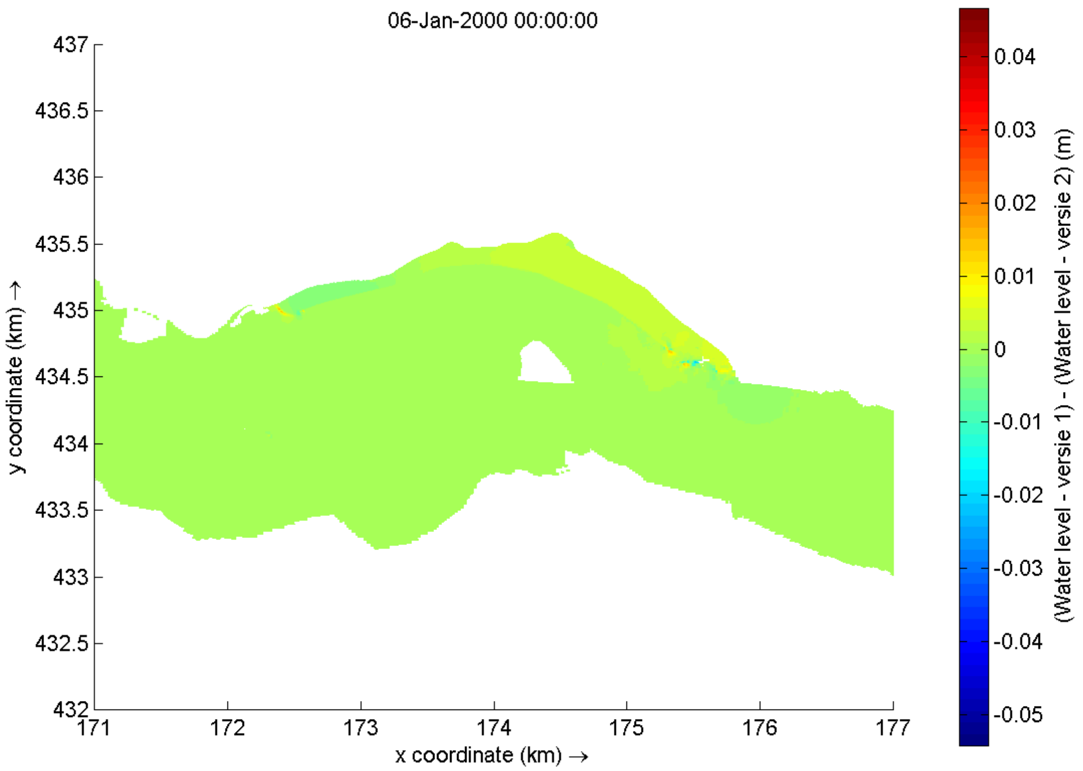

Figure F-4 Water level differences between variant 1 and variant 2. Red indicates a higher water level and dark blue a lower water level for variant 1 than variant 2

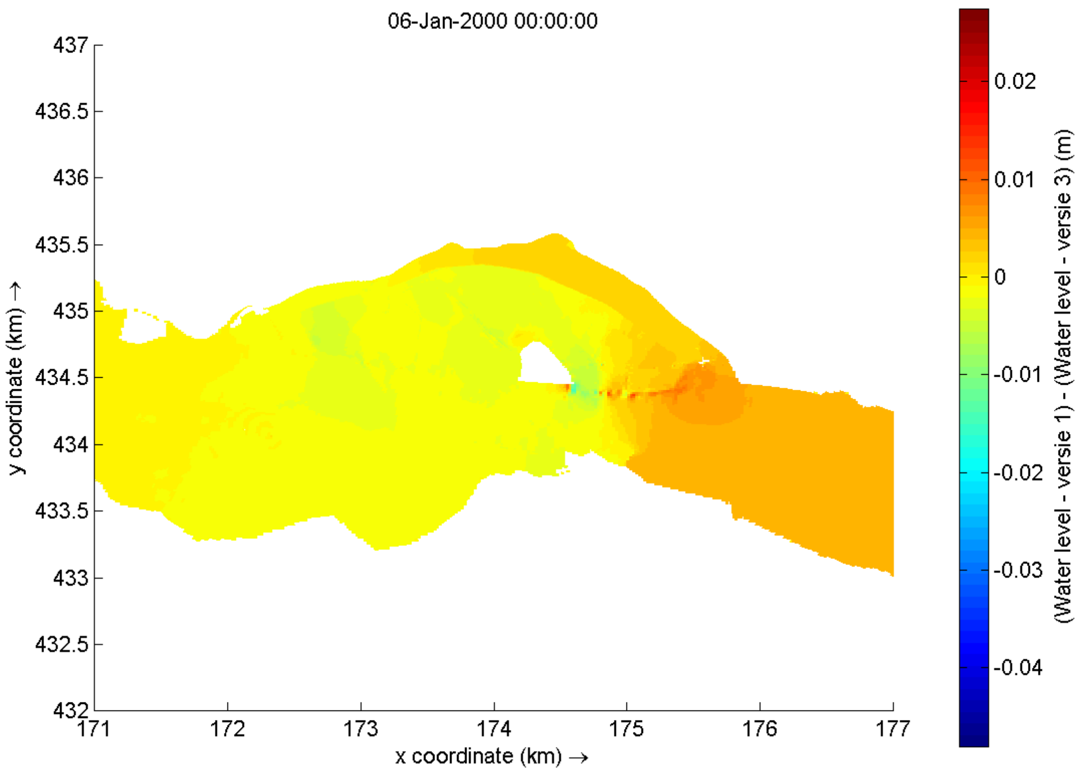

Figure F-5 Water level differences between variant 1 and variant 2. Red indicates a higher water level and dark blue a lower water level for variant 1 than variant 3

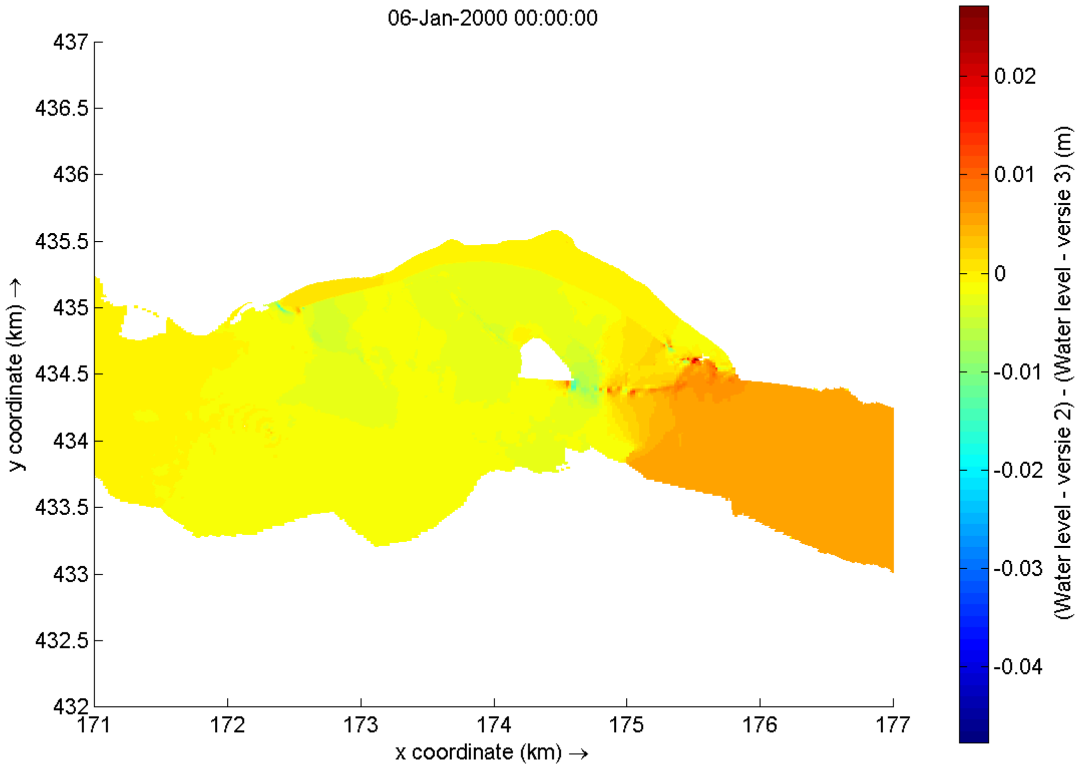

Figure F-6 Water level differences between variant 1 and variant 2. Red indicates a higher water level and dark blue a lower water level for variant 2 than variant 3

The following seven figures show the flow velocity in the original situation and the differences of flow velocity between the three variants and the original variant, variant 0. Also, the differences in flow velocity between the three variants itself are shown. A selection of these figures have been shown in Chapter 4.

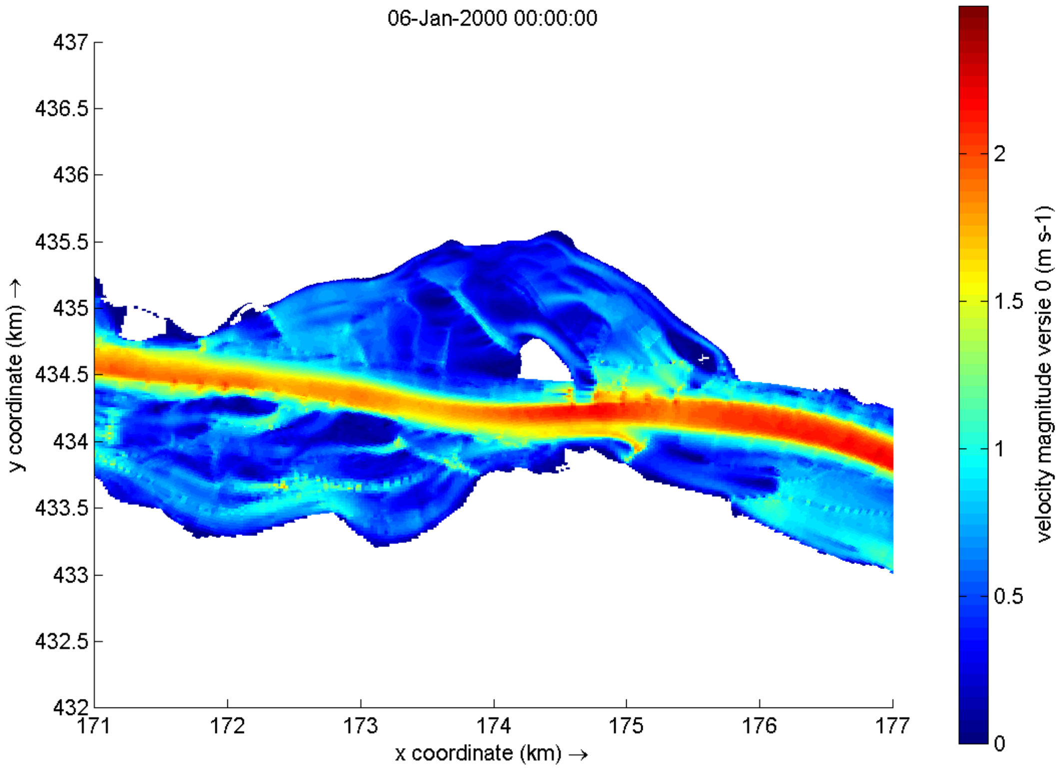

Figure F-7 Flow velocities in the channel and on the floodplain in the original situation

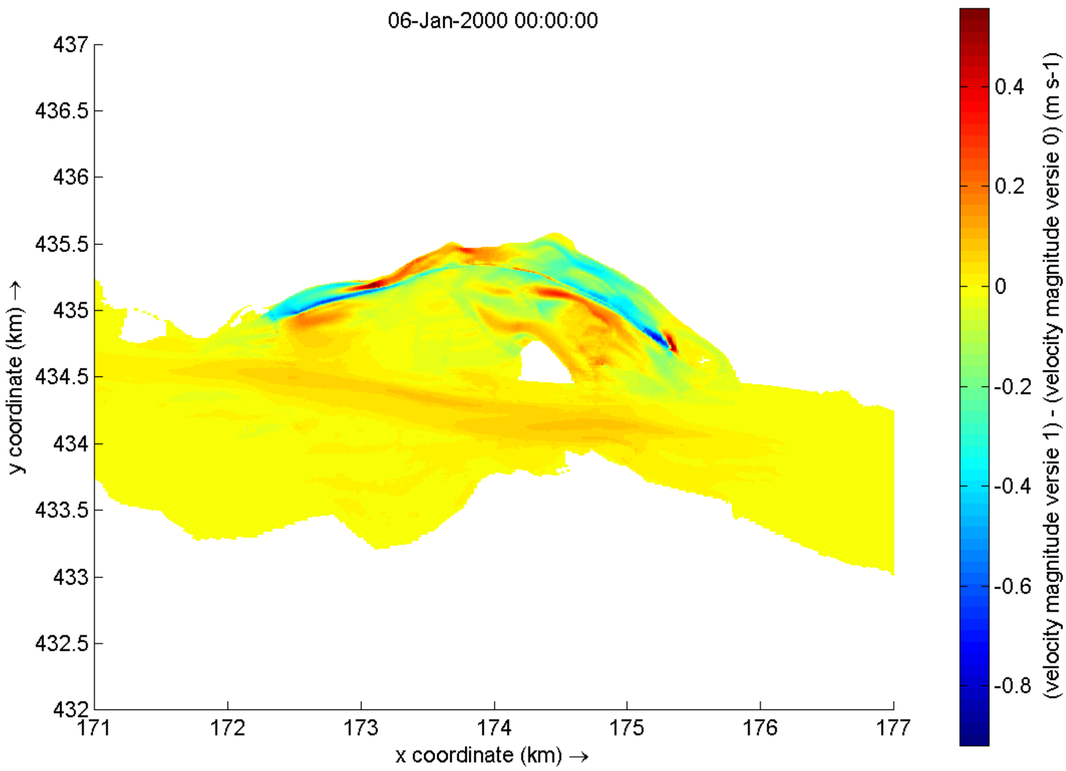

Figure F-8 Differences in flow velocity between the original situation and variant 1. Red indicates an increase in flow velocity and dark blue a decrease in flow velocity

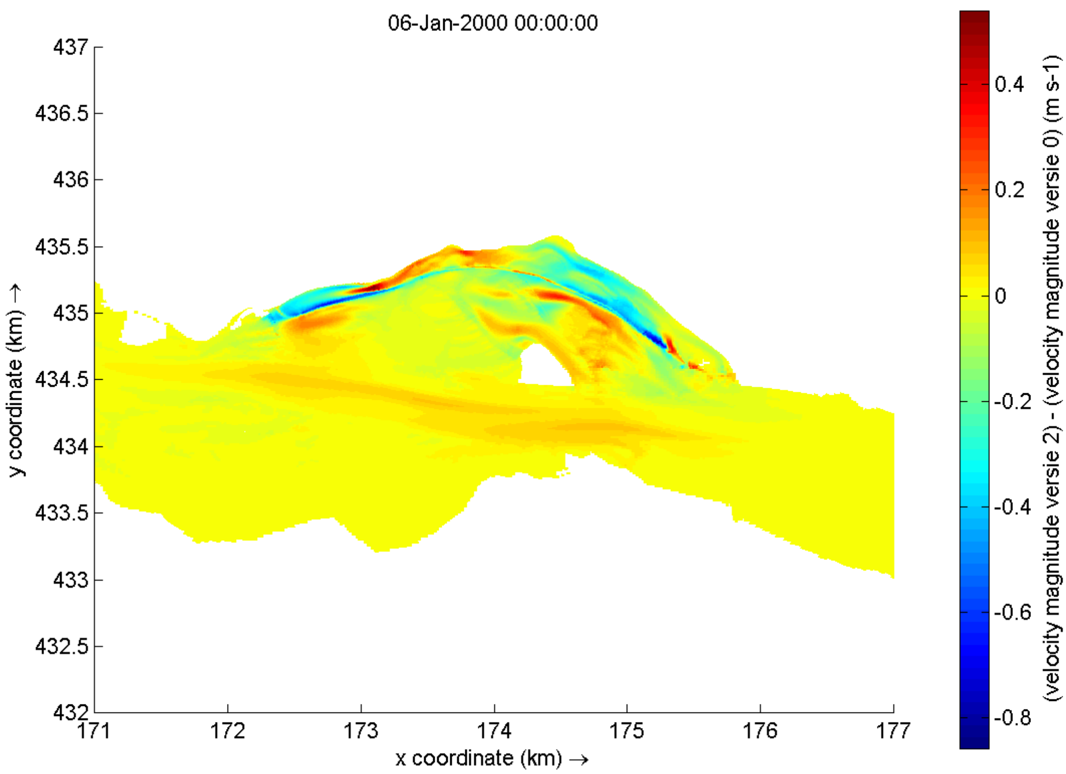

Figure F-9 Differences in flow velocity between the original situation and variant 2. Red indicates an increase in flow velocity and dark blue a decrease in flow velocity

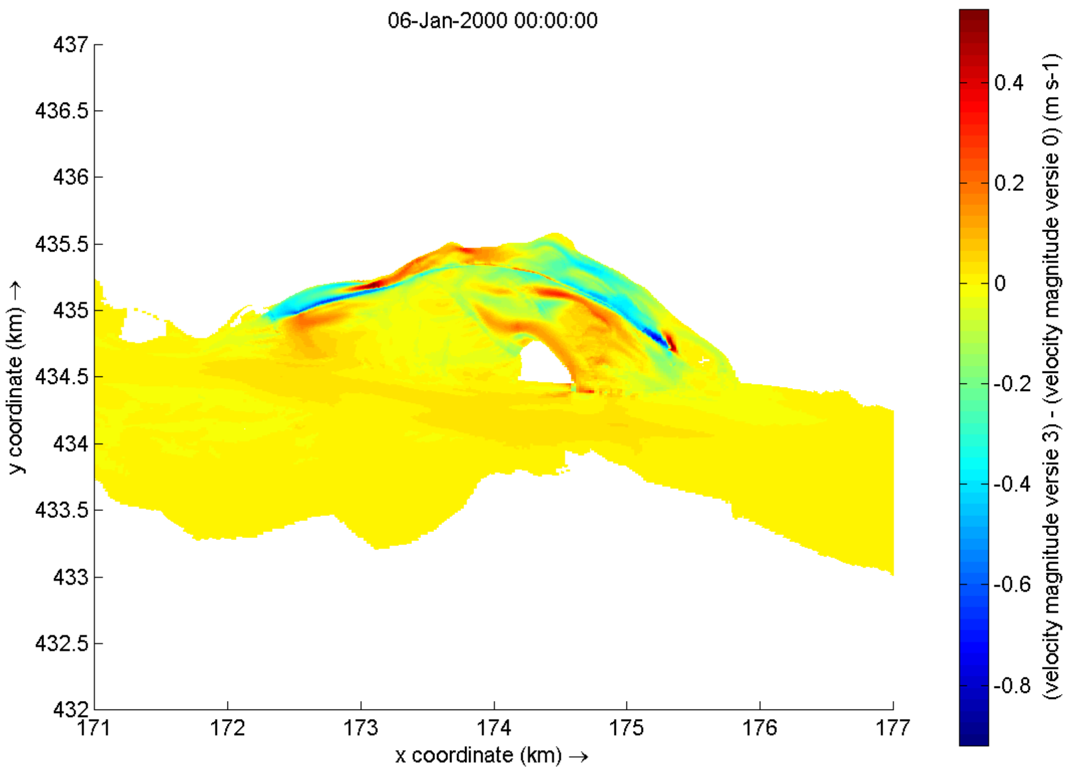

Figure F-10 Differences in flow velocity between the original situation and variant 1. Red indicates an increase in flow velocity and dark blue a decrease in flow velocity

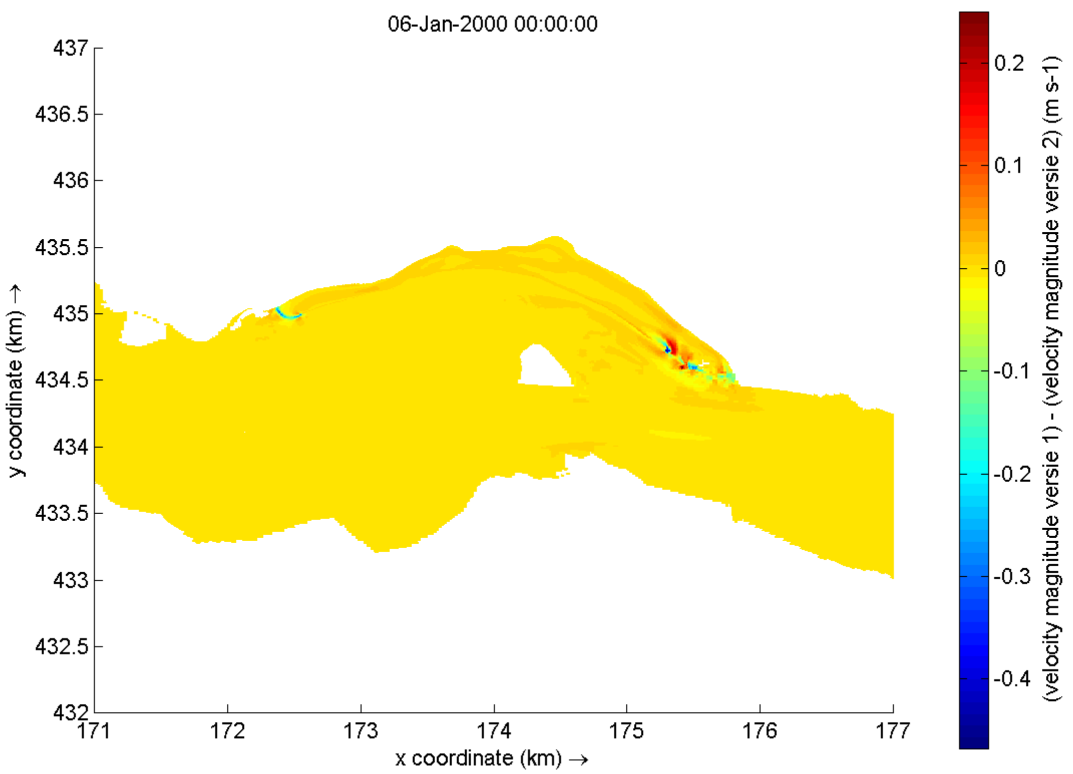

Figure F-11 Differences in flow velocity between the variant 1 and variant 2. Red indicates a higher flow velocity and dark blue a lower flow velocity in variant 1 than in variant 2

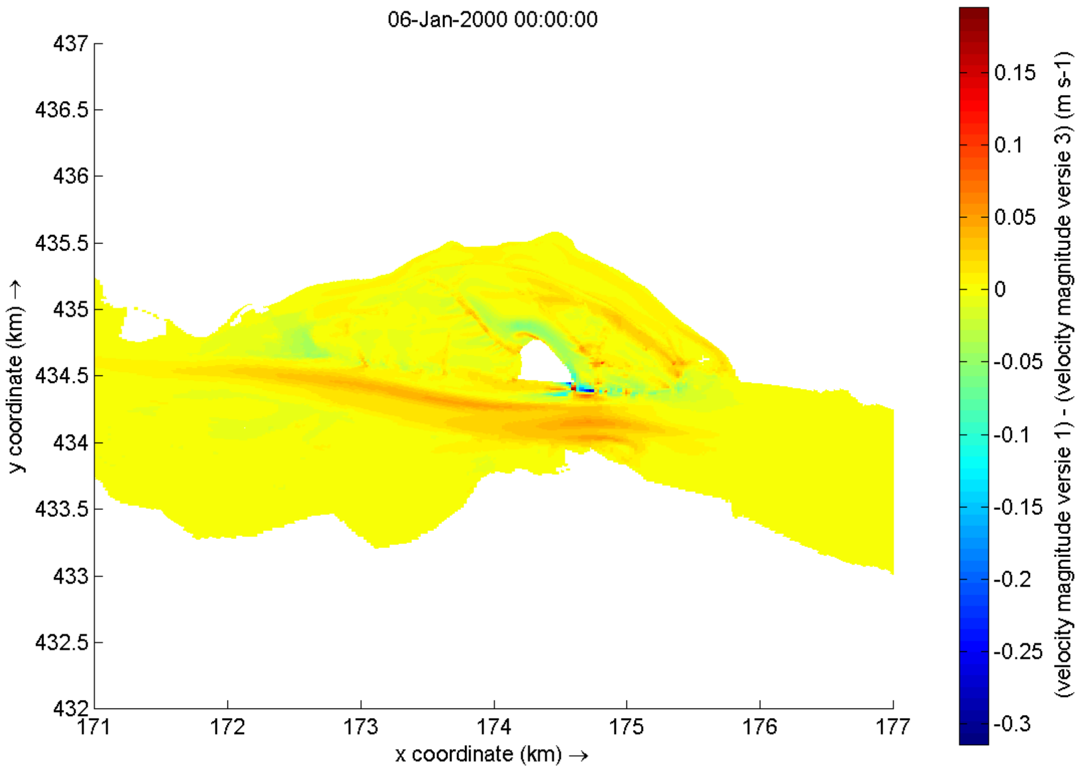

Figure F-12 Differences in flow velocity between the variant 1 and variant 2. Red indicates a higher flow velocity and dark blue a lower flow velocity in variant 1 than in variant 3

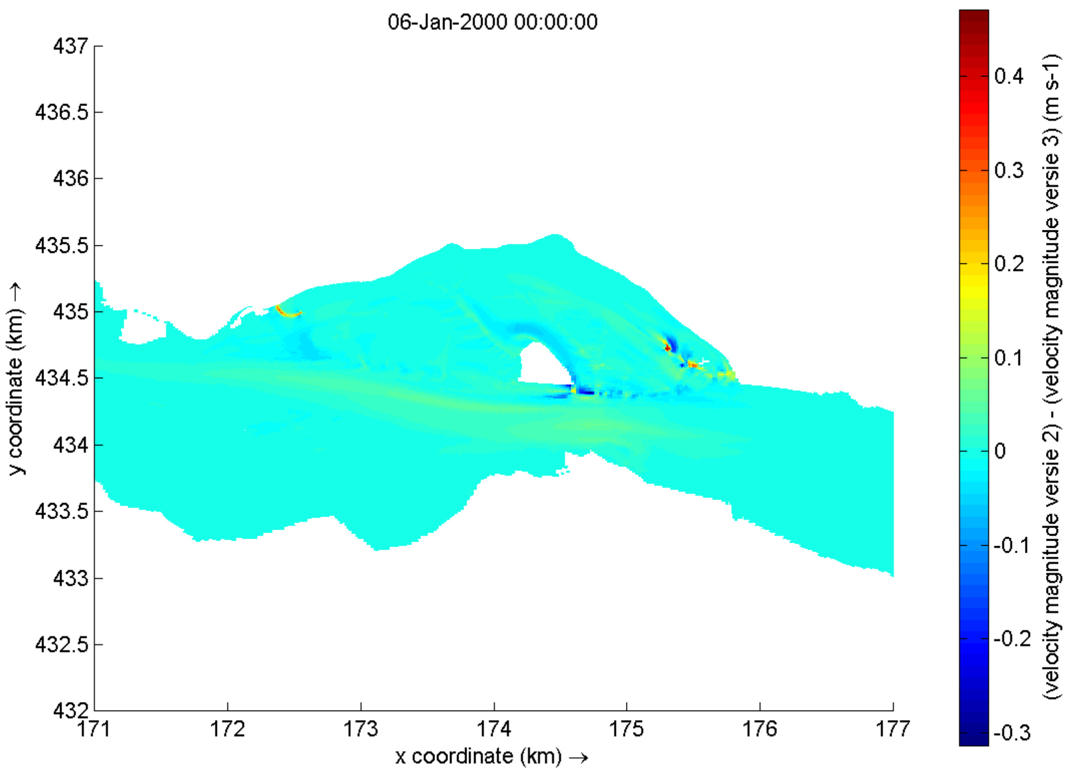

Figure F-13 Differences in flow velocity between the variant 1 and variant 2. Red indicates a higher flow velocity and dark blue a lower flow velocity in variant 2 than in variant 3