

# BED PROTECTION FOR EXTREME STORM SURGE INFLOW THROUGH FAILING SLUICEGATES

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At the northern part of The Netherlands the closure dam Afsluitdijk is situated. It is the primary flood defense that closes off Lake IJssel from the Wadden Sea to protect the northern provinces against storm surges. This structure was built in the 1930's and needed to be improved to withstand the foreseen increase in storm surge conditions and sea level rise. It consists of a 32 km long dam with five discharge sluices for regulating the water level at Lake IJssel. Three of these sluices are located in the west end of the dam and two in the east end. Due to climate change larger discharges are foreseen. Therefore two new sluices were built in between the three old sluices at Den Oever in the west end of the dam to increase the capacity. A new bed protection was designed at Lake IJssel to ensure the stability of the old and new sluices. This bed protection needs to withstand the inflow during the worst case scenario that one sluice does not close during a 45 hour storm surge. This event has a probability of occurrence of 1/1,000,000 per year. During this extreme load case, water flows from the Wadden Sea through the open sluice gates into Lake IJssel. The water level difference is up to 6.6 m. This causes discharges up to 3600 m<sup>3</sup>/s and currents that are supercritical with speeds up to 9 m/s. Erosion of the bed cannot be prevented entirely with these high velocities. The length of the bed protection is designed large enough to keep the scour hole away from the sluices. The bed protection consists of a structural and a natural part. The natural part was introduced to minimize the length of the structural protection, reducing the construction costs and CO<sub>2</sub> footprint. For this design approach the erosion resistant boulder clay layers in the subsoil were taken into account as natural bed protection, 'Building with Nature'. To achieve the custom made design was a challenge. To predict the flow behavior in combination with the erosion process was extremely complex. This could not be predicted with the conventional formulas and methods. Therefore multiple approaches were used and combined. This resulted in the design being based on the combination of computer simulations, 2D and 3D model tests and calculations with conventional formulas. The structural part of the bed protection has a length of 24 m. The first 19 m is a concrete slab with a sheet pile at the end. The last 5 m consists of grouted rock. The thickness of the concrete slab and grouted rock is 1-1.5 m. This large thickness is required to prevent uplift due to the supercritical flow. Further downstream of this structural bed protection a scour hole will develop. At this location the boulder clay forms the natural part of the bed protection. The expected development of the flow field and geometry of the scour hole during the 45 hour storm surge was determined. The scour resistance of the boulder clay was based on laboratory tests with samples taken on site. This led to the conclusion that the boulder clay reduces the development of the scour hole enough to ensure the stability of the structural part of the bed protection and the stability of the sluices.

*Keywords: climate change; storm surge; flood defense; sluice; bed protection; erosion; scour hole; boulder clay; Building with Nature*

## INTRODUCTION

At the northern part of The Netherlands the closure dam Afsluitdijk is situated. It is the primary flood defense that closes off Lake IJssel from the Wadden Sea to protect the northern provinces against storm surges, see Figure 1. This structure was built in the 1930's and needed to be improved to withstand the foreseen increase in storm surge conditions and sea level rise. It consists of a 32 km long dam with five discharge sluices for regulating the water level at Lake IJssel. Three of these sluices are located in the west at Den Oever and two in the east at Kornwerderzand. Due to climate change larger discharges are foreseen. Two new sluices were built in between the three old sluices at Den Oever to increase the capacity. Each of the old sluices has 5 gates with a width of 12 m each. The two new sluices have 4 gates with a width of 12.6 m each and a lower floor. These gates of both old and new sluices can be closed by 2 steel doors to regulate the outflow from Lake IJssel into the Wadden Sea.

A new bed protection needed to be designed at Lake IJssel to ensure the stability of the old and new sluices at Den Oever. The total width of this bed protection is 460 m, covering all five sluices. In the worst case scenario one sluice does not close during a 45 hour storm surge at the Wadden Sea. This event has a probability of occurrence of 1/1,000,000 per year. During this extreme load case water flows from the Wadden Sea through the open sluice gates into Lake IJssel. This paper covers the hydraulic engineering applied in practice for the design to survive this extreme load case.

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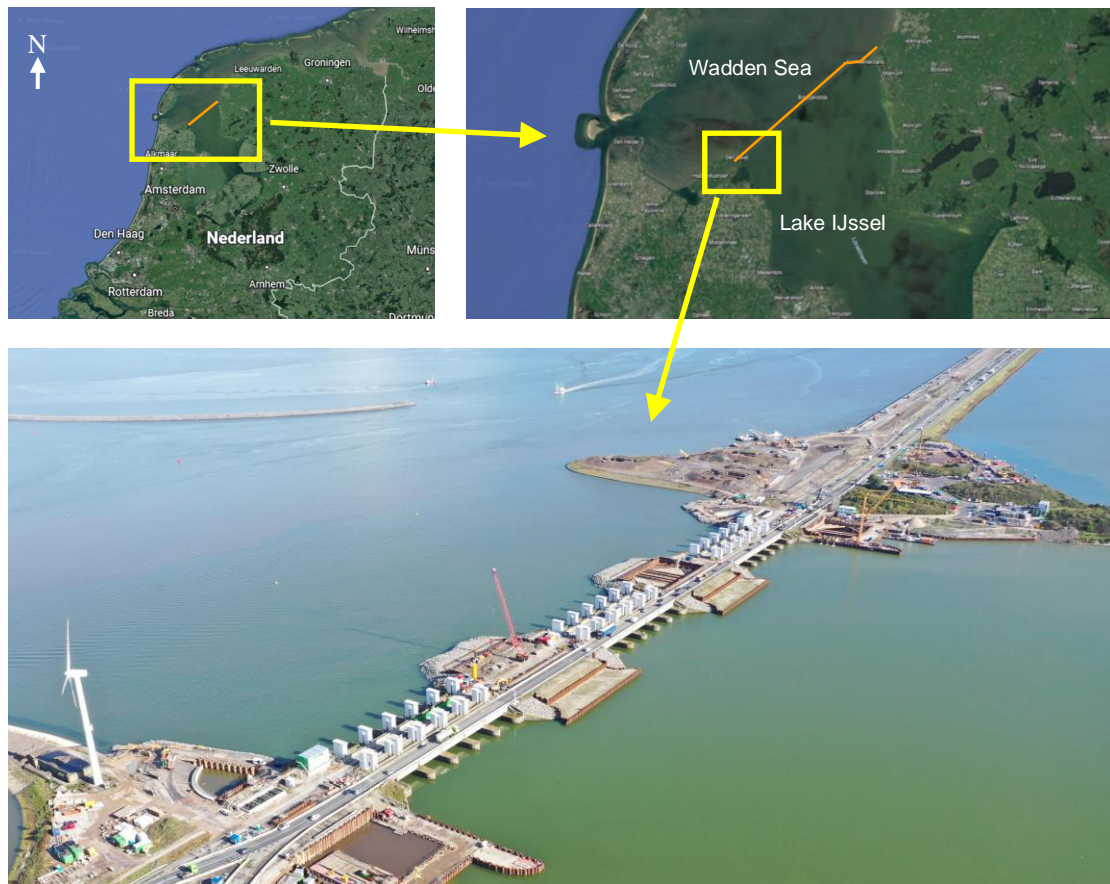


Figure 1. Location Afsluitdijk in the west at Den Oever (Lake IJssel right) showing the three old sluices with in between the construction of the two new discharge sluices.

## CONDITIONS

### Hydraulic Conditions

In the worst case scenario one sluice does not close during a 45 hour storm surge at the Wadden Sea. This event has a probability of occurrence of  $1/1,000,000$  per year. During this extreme load case water flows from the Wadden Sea through the open sluice gates into Lake IJssel. Several cases are taken into account with failed closure of 1, 2 or all gates at one discharge sluice, see Table 1 and 2. The design lifetime and therefore the design water levels differ between the old and new sluices. The old sluices are to be replaced after the year 2050 hence the design lifetime of the bed protection is not till the year 2120 as for the new sluices (design lifetime 100 years). However the water level differences are almost the same for both old and new sluices despite the difference in lifetime. This is because both the water levels at the Wadden Sea and Lake IJssel are higher in 2120. At the Wadden Sea the water levels are higher due to sea level rise. At Lake IJssel the target water level is anticipated to be increased in the future to cope with higher discharges of the rivers connected to the lake.

Depending on the amount of gates open, the water level return period varies up to  $1/10,000$  per year in case 1 gate is open. All combinations of opened gates with water level return period lead to the combined  $1/1,000,000$  return period. In Figure 2 the different water levels and water level difference are given for the case with 1 gate open in one of the old sluices. The water level difference is up to 6.5 m as shown. During the storm the water level at the Wadden Sea builds up to the peak of NAP +5.3 m with a constant water level at Lake IJssel of NAP -1.2 m (NAP: Amsterdam Ordnance Datum). For all cases the bed protection needs to protect the sluices from failing due to bed erosion.

**Table 1. Design water levels and water level differences for the old sluices (design lifetime till 2050).**

Return period per year	1/1,000	1/5,000	1/10,000
Number of gates open [-]	5	2	1
Water level Wadden Sea [m NAP]	4.76	5.12	5.28
Water level Lake IJssel [m NAP]	-1.09	-1.17	-1.21
Water level difference [m]	5.85	6.29	6.49

**Table 2. Design water levels and water level differences for the new sluices (design lifetime till 2120).**

Return period per year	1/1,000	1/5,000	1/10,000
Number of gates open [-]	4	2	1
Water level Wadden Sea [m NAP]	5.46	5.82	5.98
Water level Lake IJssel [m NAP]	-0.49	-0.57	-0.61
Water level difference [m]	5.95	6.39	6.59

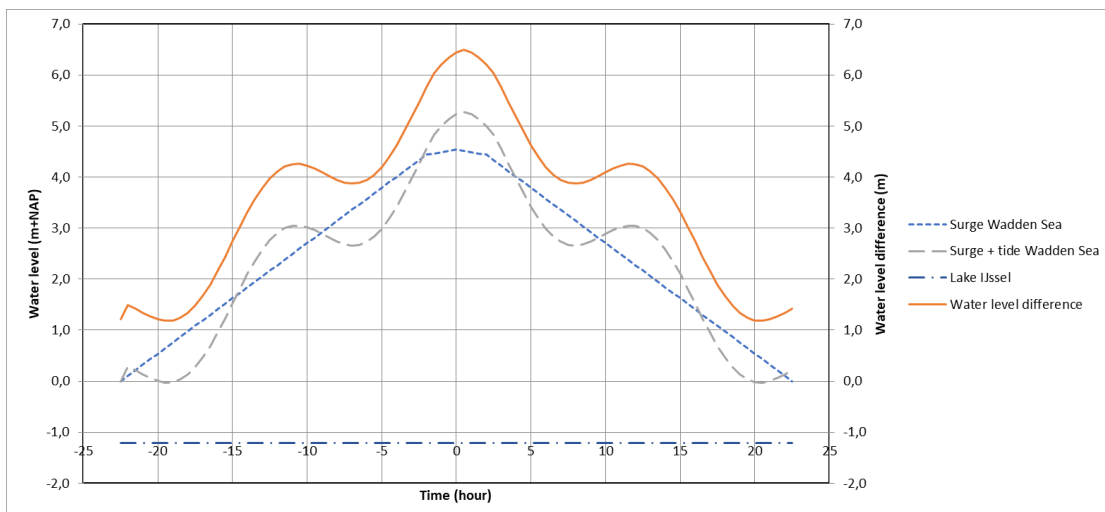


Figure 2. Design water levels and water level difference during the 45 hour storm surge at the Wadden Sea for the case of 1 gate open in one of the old sluices .

**Present Structures**

During the construction of the old sluices in the 1930’s bed protection was already placed at both Wadden Sea and Lake IJssel side. This to prevent scour during discharges through the sluices towards the Wadden Sea. In the years after, these bed protections were extended as erosion had occurred.



Figure 3. Construction of the sluices and bed protection at Den Oever in the 1930’s.

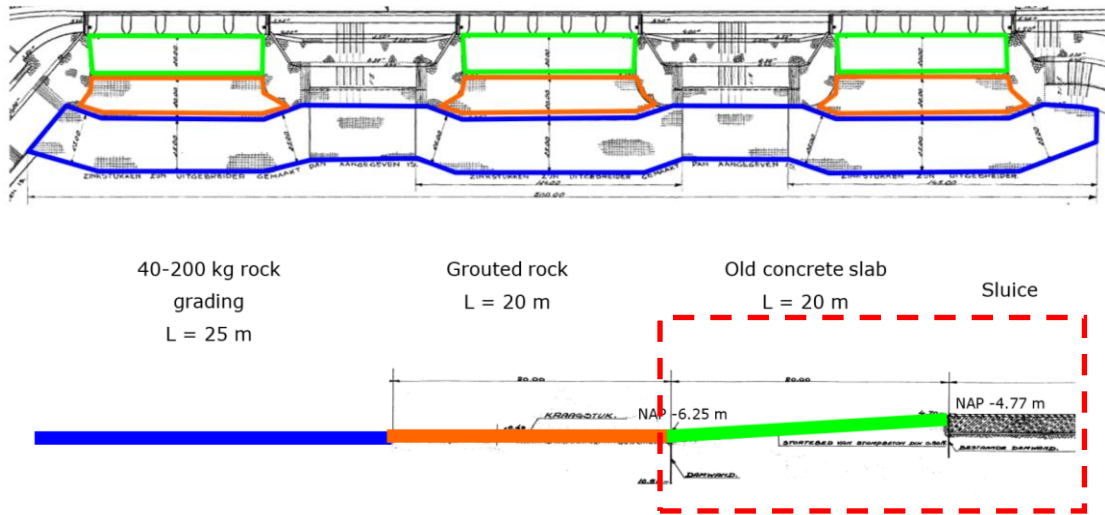


Figure 4. Lay-out and cross section of the old bed protection south of the Den Oever sluices at Lake IJssel.

After this extension the bed protection at Lake IJssel side consisted of a 20 m long concrete slab followed by a sheet pile, 20 m of grouted rock and 25 m of loose 40-200 kg rock, see Figure 4. For the renovation project the old concrete slab was replaced by a 19 m long new slab with a varying thickness of 1-1.35 m and a sheet pile at the south end (on the left side in Figure 5). The sluice floor is at NAP -4.77 m somewhat higher than the concrete slab which is situated at NAP -4.95 m. For the new sluices the same principle was used, with the only difference that there was no old concrete slab to be replaced. The slab at the new sluice has a length of 16 m, a constant thickness of 1 m and also a sheet pile at the south end. The floor of the new sluices is at NAP -6.55 m. This is lower compared to the old sluices to increase the discharge capacity. The concrete slab is situated somewhat lower at NAP -6.63 m. The new concrete slabs are part of the bed protection and their stability will be described briefly in the following sections. The main focus of this paper will be on the design of the new bed protection required south of these concrete slabs and sheet piles.

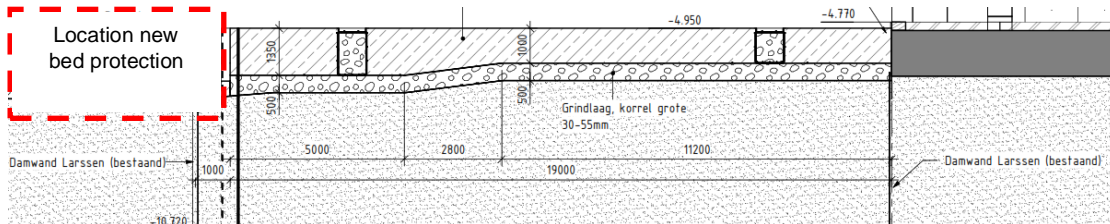


Figure 5. Cross section of the new concrete slab south of the Den Oever sluices at Lake IJssel.

### Subsoil Conditions

In the subsoil several thick layers of erosion resistant boulder clay are present. In Table 3 a representative profile of the different soil layers is given. It shows that the top boulder clay layer has a thickness of around 5.5 m and starts at NAP -6 m. The deeper layer has a thickness of around 4.5 m and starts at NAP -17 m. In between a moderately dense packed sand layer is present with thin lenses of (boulder) clay.

Soil type	Top of layer [m NAP]
Sand - moderately packed	-2.0
Boulder clay	-6.0
Sand pleistocene - moderately packed	-11.5
Boulder clay	-17.0
Sand pleistocene - moderately packed	-21.5
Sand pleistocene - well packed	-22.5

### DESIGN APPROACH

The minimum bottom profile that is required south of the concrete slab and sheet pile during the 45 hour storm was calculated. This profile is required for the geotechnical stability of the sheet pile ensuring the stability of the sluice. In Figure 6 the minimum required bottom profile is shown for the old sluices. The required profile for the new sluices is more or less the same with the difference that it is located deeper as the sluices floor and concrete slab are also constructed deeper. For the remaining of this paper the situation of the old sluices will be described as base case.

During the extreme event that 1 or more gates fail to close in the 45 hour storm surge, water will flow through the conduit(s) of the sluice into Lake IJssel. The inflow rate and current speeds will be so extreme that erosion cannot be prevented entirely and will occur south of the sheet pile. The new bed protection needs to be long enough to keep the scour hole away from the sheet pile to ensure the minimum required bottom profile.

The bed protection consists of a structural and a natural part. The natural part was introduced to minimize the length of the structural protection, reducing the construction costs and CO<sub>2</sub> footprint. For this design approach the erosion resistant boulder clay layers in the subsoil were taken into account as natural bed protection, based on the 'Building with Nature' principle.

In the following sections the stability and design of the structural and natural bed protection is described.

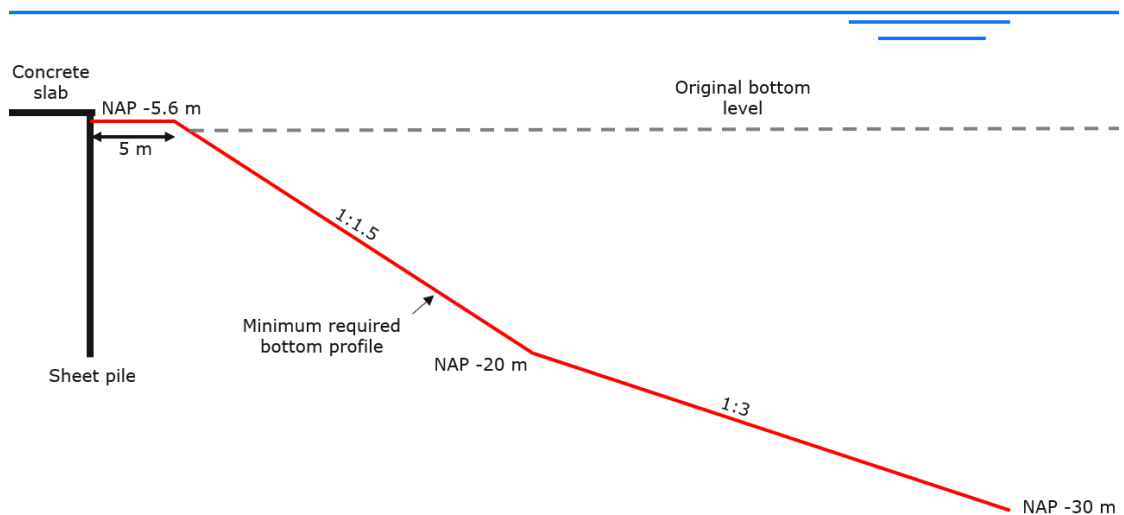


Figure 6. Minimum required bottom profile south of the old sluices after the extreme event.

### PHYSICAL MODEL TESTS

First 2D and 3D physical model tests were performed. The 2D model tests were carried out in the BAM-DMC wave flume that was adjusted to generate both waves and flow. For the 3D model tests a new hydraulic laboratory had been designed and built in-house. This facility was dedicated for the testing and design of the bed protections of all sluices (west and east). These model tests gave a good insight in the flow behavior occurring during the extreme event. However the observed geometry of the scour hole was too conservative as the erosion resistant boulder clay layers at Den Oever could not be modeled / scaled properly.

Based on the model tests it was concluded that the old bed protection will not be stable during the extreme event. The old bed protection was not taken into account for the design of the new protection. This is a conservative approach, because prior to becoming unstable the old protection will have some resistance at the beginning of the storm and will slow down the erosion process.

The new concrete slabs were tested stable with a layer thickness of 0.54 m and a density of 2,300 kg/m<sup>3</sup>. The designed thickness of 1-1.35 m is conservative for the required hydraulic stability.



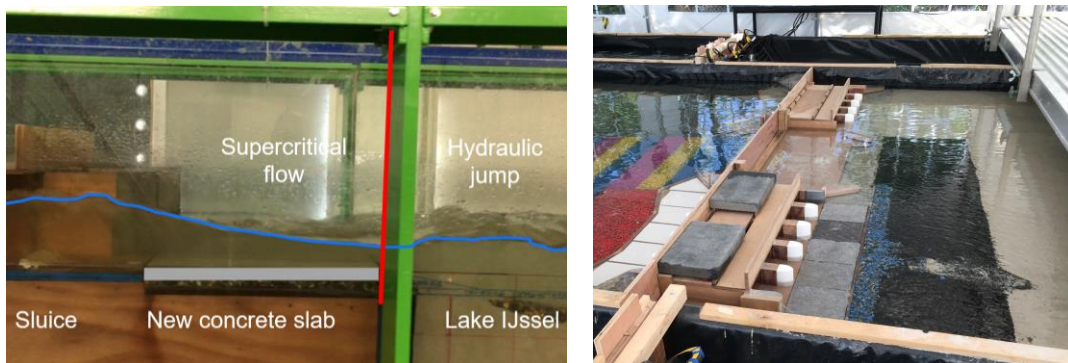


Figure 7. Picture left, 2D model test; Picture right, 3D model test with scour in the old bed protection.

### DESK STUDY

The design of the new bed protection, both the structural and natural part, was continued with a desk study. The new bed protection was not tested and the erosion of the boulder clay could not be modeled in the physical model. In this desk study the following steps have been carried out:

1. The expected upstream slope of the scour hole was determined based on field measurements.
2. The minimum required geometry of the bed protection, to keep the scour hole away from the sheet pile, was calculated.
3. The expected development of the flow field downstream of the bed protection was determined.
4. Based on this flow field the expected development in time of the scour hole was determined.
5. The erosion resistance of the boulder clay was tested in a laboratory.
6. The final geometry of the scour hole developed after the 45 hour storm surge was calculated.

These design steps are described in the following sections.

### Slope Scour Hole

In 1932 the sluices at Den Oever were being used for the first time. Immediately erosion at the side of the Wadden Sea occurred due to the discharge of the sluices. The development of this erosion was surveyed at that time, see Figure 8. A scour hole with an upstream slope of 1:10 formed in the subsoil with boulder clay layers. The sluices were taken out of service and a bed protection was constructed with loose rock covering the 1:10 slope of the scour hole. After completion of this bed protection the sluices were being used again. However further erosion occurred downstream of the new bed protection. This erosion led to an upstream slope of the scour hole of 1:2.

The subsoil at Lake IJssel side and the Wadden Sea side consist of the same profile of soil layers. Both have the boulder clay layers, one in the top and one deeper in the subsoil. Therefore the development of the same scour hole slope is expected at the Lake IJssel side when extreme inflow occurs due to failed closure of a sluice during the 45 hour storm surge. The steepest slope of 1:2 was used for the design of the new bed protection. This slope will develop during the storm downstream of the structural part of the bed protection.

### Geometry Bed Protection

The geotechnical stability of the sluices requires a minimum bottom profile (Figure 6) directly downstream of the sheet pile with a berm width of  $\geq 5$  m. The length of the structural part of the bed protection is  $\geq 5$  m to ensure the required berm width. The structural part of the protection is constructed with grouted rock as loose rock will not be stable in these extreme conditions. Downstream the scour slope of 1:2 is expected to develop. This slope is not as steep as the 1:1.5 slope of the minimum required bottom profile. The expected profile of the scour hole does not intersect the required geotechnical profile. It is also expected that the boulder clay layers will not fully erode during the storm leading to a buffer between the two profiles. The development and final geometry of the scour hole will be described in the following sections.

With the length of the bed protection known, the required thickness was calculated. To do so the current speeds above the bed protection were first determined. During the storm surge free weir flow is occurring. With the following formula (Nortier and De Koning 1993) the inflow rates for all cases (1, 2 and all gates open) were calculated and are given in Table 4 and 5. Based on these inflow rates and the geometry of the sluice conduits the current speeds above the bed protection were determined. At the peak of the storm the extreme inflow gives discharges up to 3600 m<sup>3</sup>/s through the open sluice gates into Lake IJssel. Leading to currents that are supercritical with speeds up to 9 m/s.

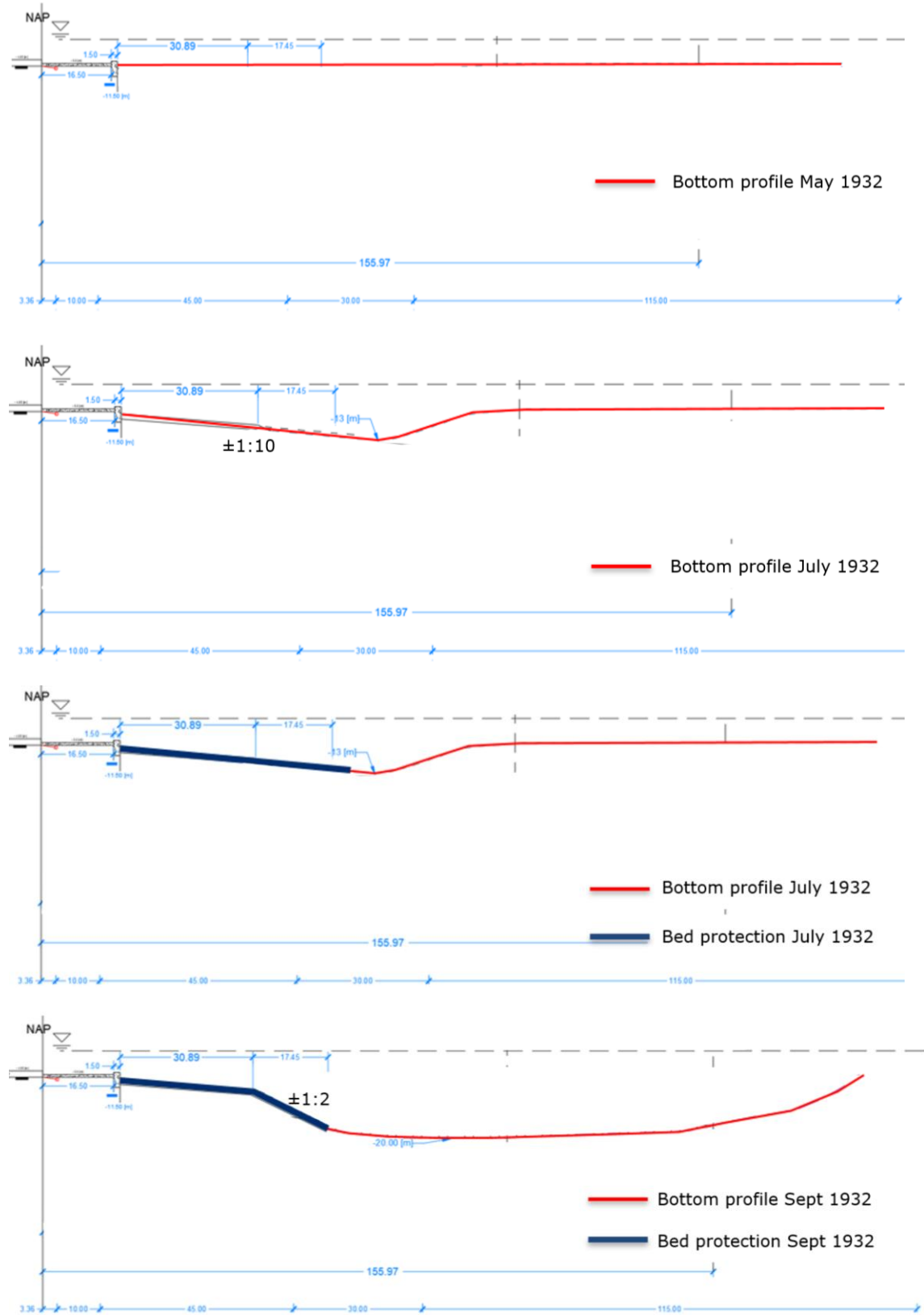


Figure 8. Development of the scour hole at Den Oever, Wadden Sea side in 1932 (Bazlen 1967).

The theoretical maximum discharge coefficient for a long free weir flow is:

$$c_{vl} = \frac{2}{3} \cdot \sqrt{\frac{2}{3}} \cdot \sqrt{g} = 1.7 \text{ m}^{1/2}/\text{s} \quad (1)$$

The discharge is calculated with:

$$Q_v = c_{vl} \cdot b \cdot H^{\frac{3}{2}}$$

For the free weir flow the following holds:

$$h_3 \leq \frac{2}{3}H$$

In which:

$Q_v$  = inflow rate [ $\text{m}^3/\text{s}$ ]

$c_{vl}$  = discharge coefficient for free weir flow [ $\text{m}^{1/2}/\text{s}$ ]

$b$  = width of the sluice conduit [m]

$h_3$  = downstream water depth above the sluice conduit floor [m]

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

$H$  = energy level of the flow upstream of the sluice relative to the sluice conduit floor [m]

Return period per year	1/1,000	1/5,000	1/10,000
Number of gates open [-]	5	2	1
Water level Wadden Sea [m NAP]	4.76	5.12	5.28
Water level Lake IJssel [m NAP]	-1.09	-1.17	-1.21
Water level difference [m]	5.85	6.29	6.49
$Q_v$ , inflow rate [ $\text{m}^3/\text{s}$ ]	3,000	1,268	650
$U$ , current speed [m/s]	7.9	8	8.1

Return period per year	1/1,000	1/5,000	1/10,000
Number of gates open [-]	4	2	1
Water level Wadden Sea [m NAP]	5.46	5.82	5.98
Water level Lake IJssel [m NAP]	-0.49	-0.57	-0.61
Water level difference [m]	5.95	6.39	6.59
$Q_v$ , inflow rate [ $\text{m}^3/\text{s}$ ]	3,572	1,866	951
$U$ , current speed [m/s]	8.9	9	9

As a contractual requirement the layer thickness of the bed protection needed to be determined with the following formula (MarCom Working Group 180 2015) unless the stability was verified with model tests.

$$D \geq \frac{C_l}{2\Delta g} v_{bottom}^2 \quad (2)$$

In which:

$D$  = average layer thickness of the bed protection [m]

$\Delta$  = relative density of the bed protection  $1.44 (\rho_{gr} - \rho_w)/\rho_w$  [-]

$\rho_w$  = density water in the Wadden Sea  $1,025 \text{ [kg/m}^3\text{]}$

$\rho_{gr}$  = density grouted rock  $2,500 \text{ [kg/m}^3\text{]}$

$v_{bottom}$  = current speed at the bottom [m/s]

$C_l$  = lift factor 0.5 [-]

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

With the grouted rock having a density of  $2,500 \text{ kg/m}^3$  the minimum required thickness is calculated to be 1.16 m and 1.44 m for respectively the old and new sluices. This is conservative compared to the



concrete slabs that were tested stable with a thickness of 0.54 m and a density of 2,300 kg/m<sup>3</sup>. The new bed protection is located directly downstream of the concrete slab and only 5 m long. Similar hydraulic loads can therefore be expected on both the concrete slab and the grouted rock. The significantly larger calculated thicknesses can be explained by the fact that the given formula is meant for the design of bed protections that need to withstand propellor jets caused by ships. These jets have a higher turbulence than the expected flow during the extreme event. Nonetheless, as the exact hydraulic stability was not tested, the calculated thicknesses are used for the design, see Figure 9 to 11. As the new sluices are placed deeper than the old sluices, dredging of the existing bottom was required. The excavation slopes are shown on the right side in Figure 10. Subsequent natural lowering of the bottom profile is expected.

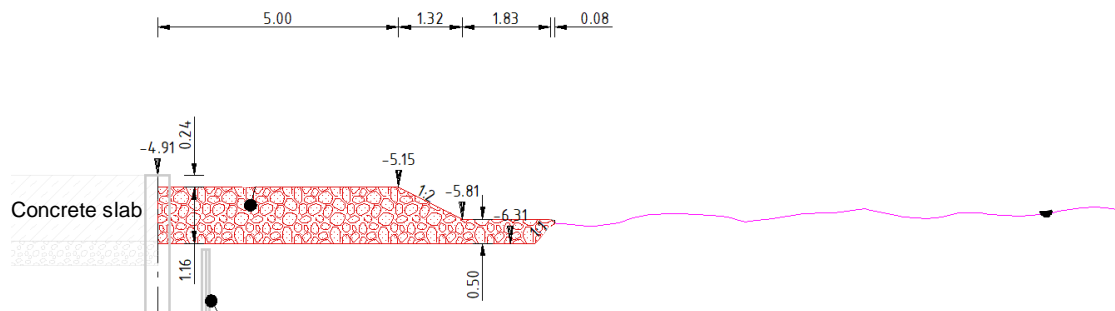


Figure 9. Geometry of the bed protection at the old sluices with on the left side (sluice side) the concrete slab and sheet pile and on the right side (Lake IJssel side) the existing bed level.

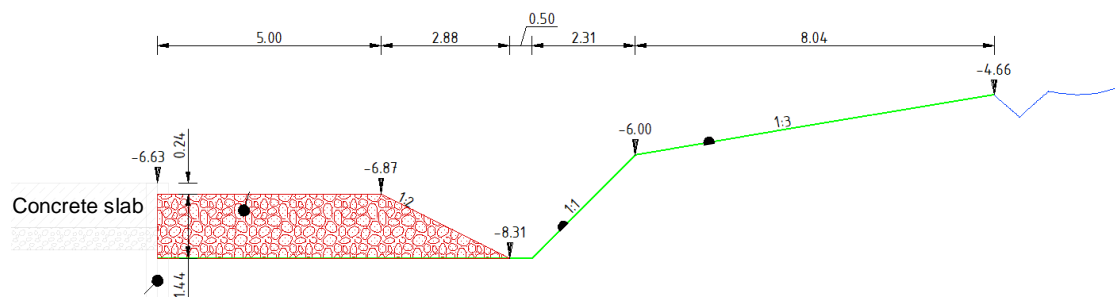


Figure 10. Geometry of the bed protection at the new sluices with on the left side (sluice side) the concrete slab and sheet pile and on the right side (Lake IJssel side) the excavation of the existing bed level.

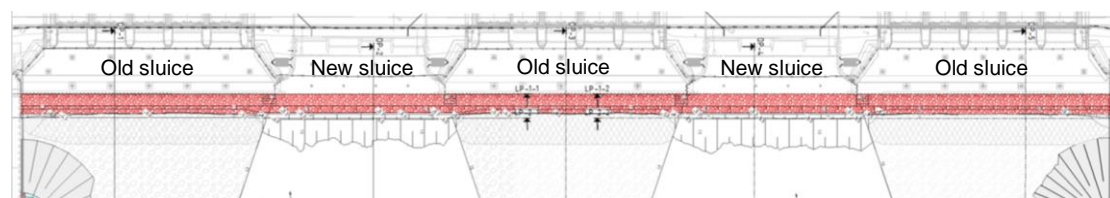


Figure 11. Lay-out of the 460 m wide bed protection of grouted rock along the three old and two new sluices.

This geometry of the structural part of the bed protection gives a significant reduction in construction costs and CO<sub>2</sub> footprint. For example at the sluices of Kornwerderzand in the east also a bed protection was designed for similar hydraulic conditions. However at that location no erosion resistant boulder clay layer is present in the top part of the subsoil. There the bed protection has a total length of 31 m of which 25 m is grouted rock and 6 m is 10-60 kg loose rock. The layer thickness is similar. This leads to the conclusion that, using the characteristics of the existing soil layers as natural part of the structure, is beneficial for both the project and climate. The design of this natural part is described in the following sections.

**Flow Behavior Downstream**

As described free weir flow is occurring. The physical model tests showed that the flow is supercritical above the concrete slab, see Figure 7. The same flow will occur above the new bed protection. Further downstream a hydraulic jump occurs that is initiated by the scour hole.

The development of the flow field in time in combination with the development of the scour hole cannot be calculated with the presented formula for free weir flow. Instead a numerical model was made. The subsoil in the model consisted of sand, as boulder clay cannot be modeled properly. In time a recirculation zone developed just downstream of the bed protection in the scour hole as shown in Figure 12 (Hoffmans and Verheij 1997). This zone developed when the depth of the scour hole was  $\geq 3$  m. This depth was measured from the top of the bed protection up to the bottom of the scour hole. In reality boulder clay is present slowing down the erosion. However it is expected that the same recirculation zone will develop, but only later in time. The current speed at the bottom below this zone is 2 m/s. To take into account model effects the current speed was increased to 3 m/s. This leads to a robust design. The same process of recirculation zone development in an initial scour hole, was found in the 2D physical model tests.

As in the recirculation the current speed is significantly reduced, the top boulder clay layer directly downstream of the bed protection will not fully erode. The water jet coming from the sluices will mix above the recirculation zone with the receiving water. This leads to the jet spreading downward with 1:6 towards the bottom of the scour hole. Downstream of the jet reattachment point, the current speed is significantly higher again. Beyond this point the top boulder clay layer will fully erode and the scour hole will develop further into the soil.

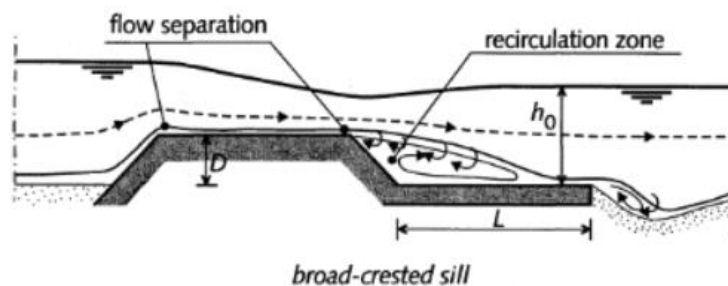


Figure 12. Development recirculation zone and location reattachment point further downstream (Hoffmans and Verheij 1997).

### Development Scour Hole

In Figure 13 the expected development of the scour in time is shown in four steps and can be described as follows:

1. The flow above the bed protection is supercritical. Further downstream this supercritical flow is followed by a hydraulic jump. This jump is initiated by the drop in bed level. At this time the top boulder clay layer is still intact.
2. The top layer of the boulder clay will start to erode as shown by the vertical arrows in Step 2. A recirculation zone starts to develop directly downstream of the bed protection.
3. Below this recirculation zone the current speed is significantly reduced leading to a slower erosion rate of the boulder clay layer. This area is marked in Step 3 by the vertical arrows below this zone, area A1. Due to this slower vertical erosion rate a plateau of boulder clay will remain in this area after the storm.

The jet coming from the sluices will spread downward with a 1:6 (V:H) slope towards the bottom of the developing scour hole. Further downstream of this reattachment point the top boulder clay layer will fully erode as the current speed in the jet is significantly higher. The scour hole will develop deeper into the subsoil of sand and thin (boulder) clay layers. The deeper thick layer of boulder clay is expected to form the bottom of the scour hole. The jet will then have spread over the full water depth leading to a significant reduction of the current speed. This deeper boulder clay layer will therefore not fully erode.

A second recirculation zone will develop downstream of the reattachment point. Leading to lower current speeds. The top boulder clay layer in this area will erode at a lower rate compared to the area in the direct path of the jet. In this area receding erosion will take place as shown by the hatched area downstream of the reattachment point and arrows pointing left downward, area A2. The underlying sand layer that will erode in the recirculation zone causes this receding erosion. This will undermine the remaining top boulder clay layer at its downstream edge. The boulder clay will break into chunks at the undermined edge and fall into the scour hole. This leads to exposure

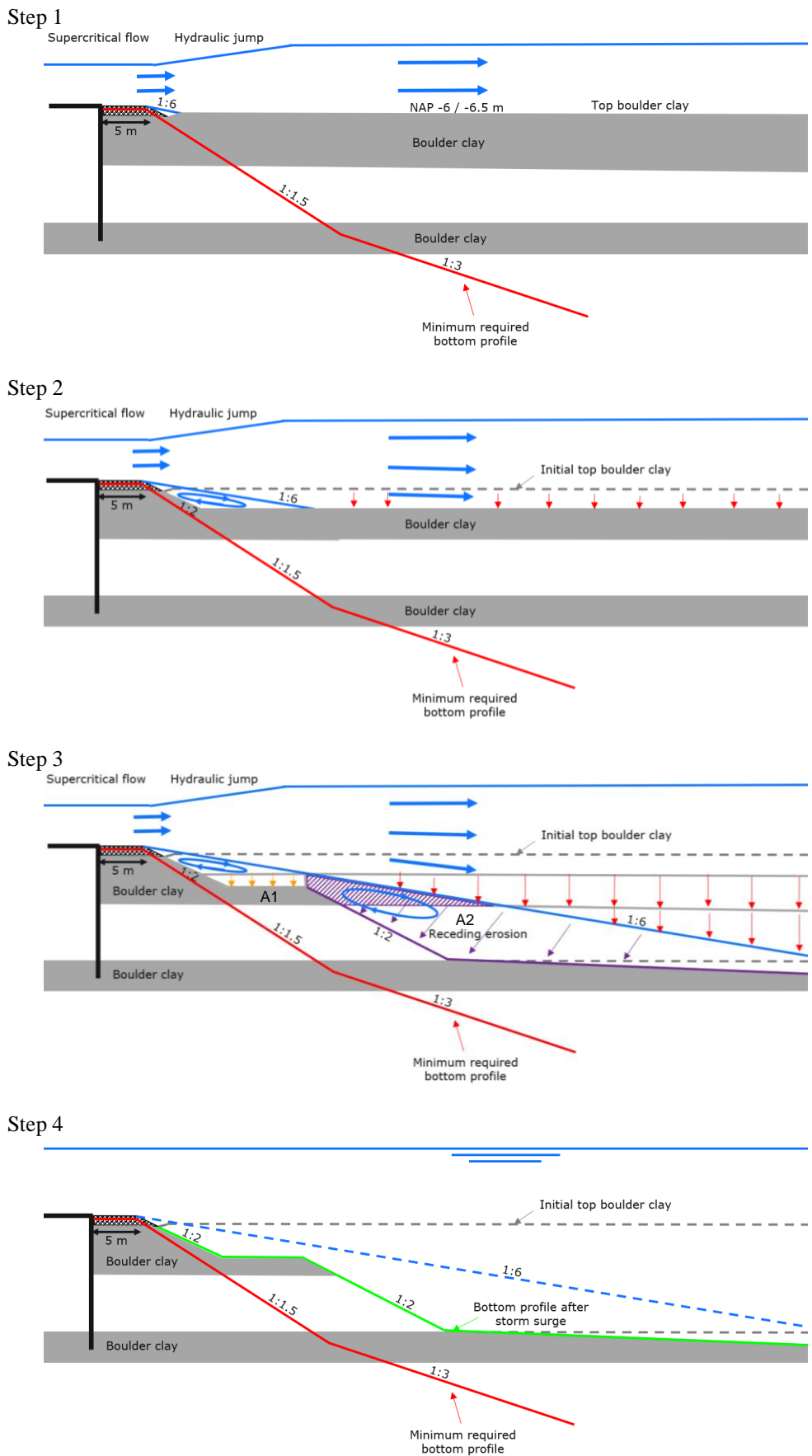


Figure 13. Expected development of the scour hole in time.

of a new part of the sand layer further upstream, which will erode and undermine the boulder clay layer further. This is a continuous process until the deeper boulder clay layer is reached. This deeper layer will form the lower boundary of the scour hole. The chunks of the undermined top boulder clay will start covering the exposed sand layer on the slope of the scour hole, slowing down the erosion process. For the robustness of the design it was taken into account that this entire area downstream of the reattachment point will erode.

4. After the storm a plateau of the top boulder clay layer with a reduced thickness remains, followed by the 1:2 slope towards the deeper boulder clay layer. With this profile of the scour hole a significant buffer is still present above the minimum required bottom profile.

In the following sections the expected geometry of the scour hole will be determined in more detail based on the results of laboratory erosion tests on the samples of the actual boulder clay.

#### Erosion Resistance Boulder Clay

To determine the erosion resistance of the boulder clay five samples were taken on site out of the boulder clay layer. These samples were sent to the Zachry Department of Civil Engineering of the Texas A&M University in the USA. The samples were tested using the Erosion Function Apparatus (Briaud et al. 2001), see Figure 14 and 15. With this method a soil sample is placed in a steel casing. The sample is pushed up 1 mm, by a piston in the casing, into a horizontal square tube. Through this tube water is pumped, which leads to erosion of the 1 mm soil protruding in the tube. The current speed is increased from 0.5 m/s up to 6.5 m/s in several steps. For each step the soil sample is pushed up with 1 mm into the tube. This way the erosion rate of the soil can be determined for several current speeds.

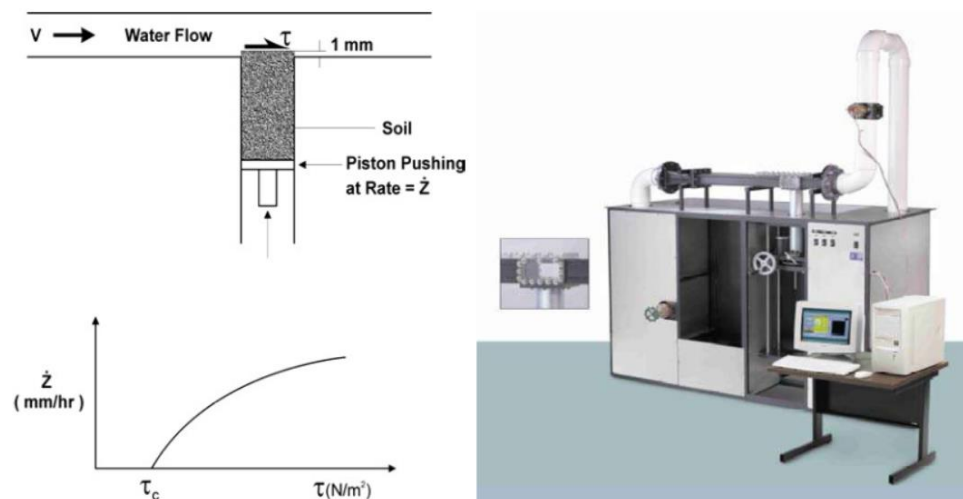


Figure 14. Erosion Function Apparatus (Briaud et al. 2001).



Figure 15. Testing of a sample taken from the boulder clay layer at Den Oever.

In Figure 16 the results are given of the performed tests. These graphs show that the test results are uniform up to current speeds of 5 m/s. For higher current speeds the erosion speed increases significantly and the results show more scatter. For the 3 m/s current speed in the recirculation zone the erosion rate is 30 mm/hour. This rate is applied for the entire 45 hours of the storm duration. In reality this current speed does not occur during the entire storm as it builds up and goes down again. However

the turbulence in the laboratory tests is not exactly the same as in reality but somewhat lower. To get a robust design the 30 mm/hour erosion rate is applied for the entire 45 hour storm duration. In total a 1.35 m layer thickness of boulder clay will erode in the plateau.

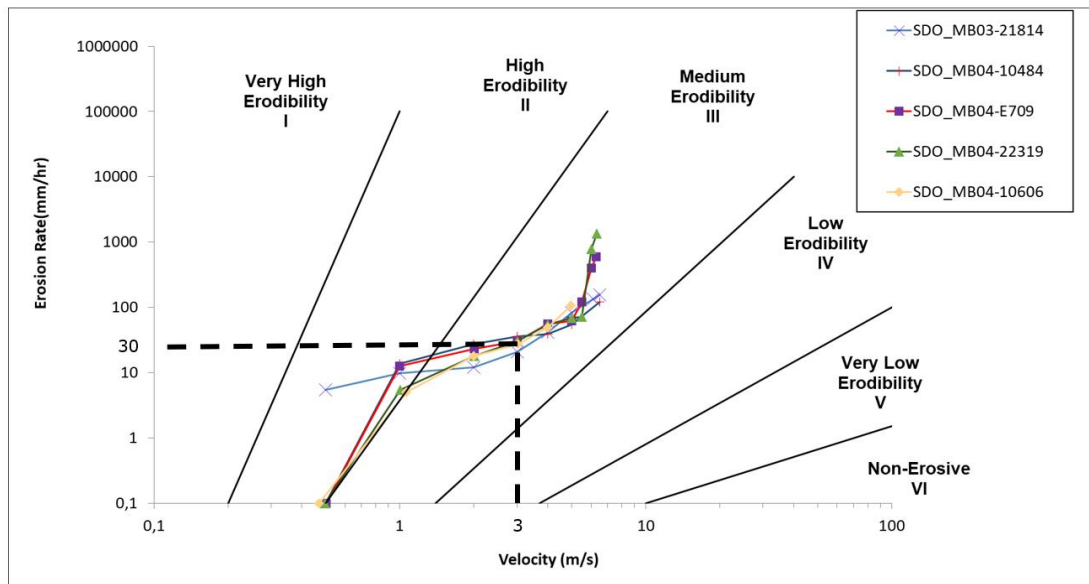
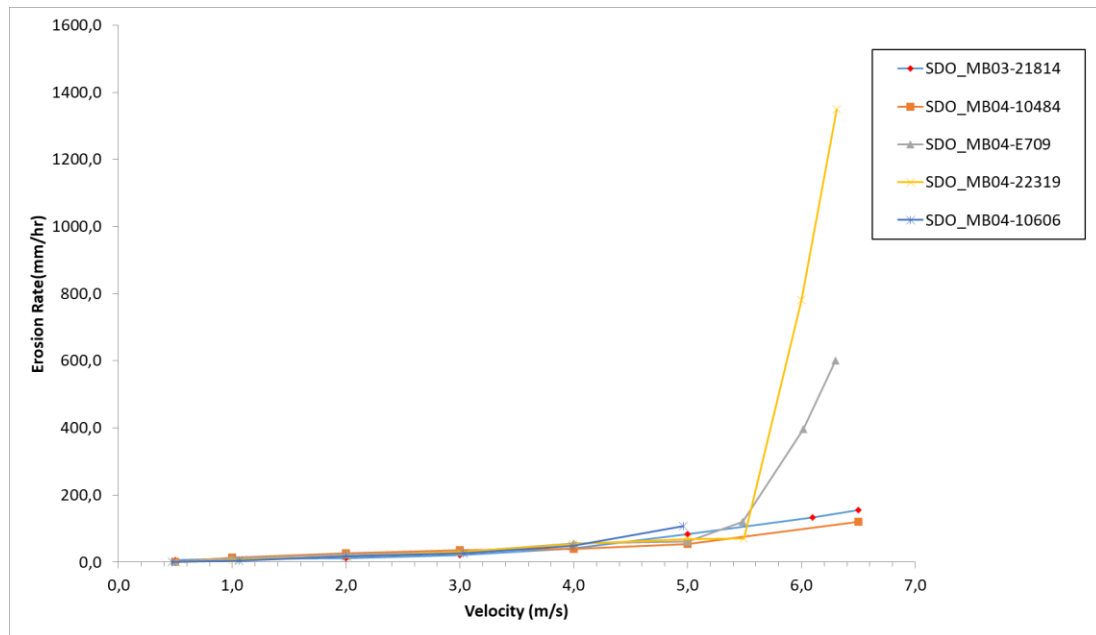


Figure 16. Results erosion resistance of tested boulder clay samples.

### Geometry Scour Hole

Based on the expected development of the scour hole and the tested erosion rate, the remaining boulder clay layer was determined and the geometry of the scour hole. The top boulder clay layer has a thickness of 5.5 m. Its top is situated at NAP -6 m. To develop the recirculation zone the bed needs to be  $\geq 3$  m lower than the bed protection. The bed protection is situated at NAP -5.2 m. The bottom needs to erode down to NAP -8.2 m for the recirculation zone to develop. This means that the boulder clay layer needs to erode for 2.2 m first. The time to develop this initial scour hole is not taken into account to get a robust design. During the 45 hour storm duration the top boulder clay layer will erode vertically with 1.35 m. A plateau with a thickness of around 2 m will remain after the storm, see Figure 17. For the robustness of the design it was taken into account that the entire area downstream of the reattachment point will erode by the receding erosion. The eroded boulder clay layer in this area is



hatched in Figure 17 and has a length of 8-16 m (top and bottom of the layer). This leads to a remaining plateau with a length of around 9 m followed by a 1:2 slope towards the deeper boulder clay layer. With this profile of the scour hole, a significant buffer is still present above the minimum required bottom profile which ensures the geotechnical stability of the sluice.

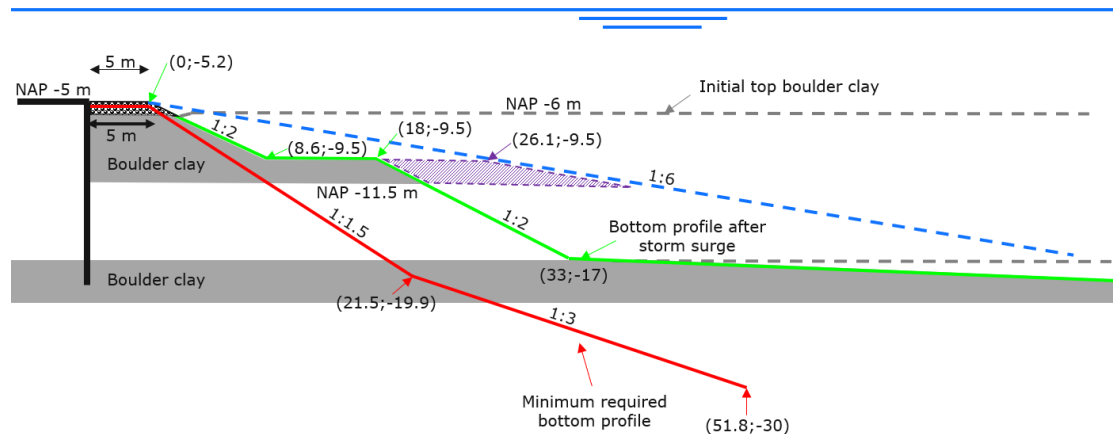


Figure 17. Expected profile of the scour hole after the 45 hour storm surge with respect to the minimum required bottom profile for the stability of the sluice.

### LESSONS LEARNED

Based on the studies performed for the design of the described bed protection the following lessons have been learned:

- The geometry of the structural part of a bed protection can be significantly reduced by using the characteristics of the existing bottom layers as a natural part of the structure, based on the 'Building with Nature' principle. This can lead to a significant reduction of the construction costs and CO<sub>2</sub> footprint which is beneficial for both the project and the climate.
- For this design approach detailed information on the subsoil, numerical and physical flow models, conventional formulas, laboratory erosion tests on soil samples and expert judgement need to be combined to determine the development of the scour hole.
- The stability of a hydraulic structure in case of gate failure can be analyzed by comparing the minimum bottom profile required for stability with the expected erosion profile after an extreme storm event.
- Combining conventional formulas with expert judgement gives results that are (at the time being) as good as computer simulations and physical scale models.
- The formula used to calculate the required layer thickness of the structural bed protection is conservative for supercritical flow.

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