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Protecting canal banks and pipeline crossings against scour in the Hartelkanaal, Rotterdam

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In november 1997, the Beerdam, at the seaward end of the Hartelkanaal in Rotterdam, was opened to create a free route for inland navigation. This paper gives an overview of the design and realisation of protection works before opening the Beerdam and shows in the severe scour and erosion which occurred in the western part of the canal.

I. Introduction

The Hartelkanaal in the harbor area of Rotterdam is 20 km long canal for inland navigation. Until 1997 the Hartelkanaal was dead ended by a dam, the Beerdam, in the western, seaward end of the canal. At the eastern, landward end, the canal has an open connection with a tidal river, the Oude Maas. The tidal water volume of the Hartelkanaal and the connected harbor basins was entering and leaving the canal from the Oude Maas.

In November 1997 an opening was excavated in the Beerdam, which resulted in a drastic change of hydraulic conditions. The aim of the so called “Open Beerdam” was to provide an open connection for inland navigation to the Maasvlakte, were increasing activities in bulk and container transport take place.

From this moment, the tidal water volume was entering and leaving the canal mainly from the seaward end of the canal. This resulted in an increase of flow velocities and caused heavy erosion in some parts of the canal.

This paper focuses on the most western part of the Hartelkanaal, between the Beerdam and the Dintelhaven.

This part of the canal is relatively narrow, while the passing tidal volume here became relatively large. Before opening the Beerdam, extensive studies were worked out to predict the increase of flow velocity and the erosion rate. Because heavy erosion was expected here, extensive bottom protection works were realized.

The southern bank of the Hartelkanaal is part of the foreshore of the Brielse Maasdijk, a primary water retaining structure. In order to guaranty the safety level of this structure, an extensive program was started for monitoring the bottom level of the canal and scour of the underwater slopes of the southern bank.

II. Situation before opening the Beerdam

The western part of the Hartelkanaal, between the Beerdam and the Dintelhaven, has a relatively small cross section profile compared to other stretches of the Hartelkanaal. The canal is at NAP-level approximately 140 m wide in this area. The bed level varies between 6 and 8 m below NAP (Dutch Ordnance Datum, which is approximately equal to the local mean water level), with an average depth of NAP -7,25 m. The guaranteed depth for the navigation is NAP -6,10 m, based on the required depth for a push barge combination at extreme low water conditions. The discharging surface of a cross section is about 763 m^2 below NAP. The discharge in the most western part of the Hartelkanaal, lies around 100 m^3/s for both ebb and flood period.

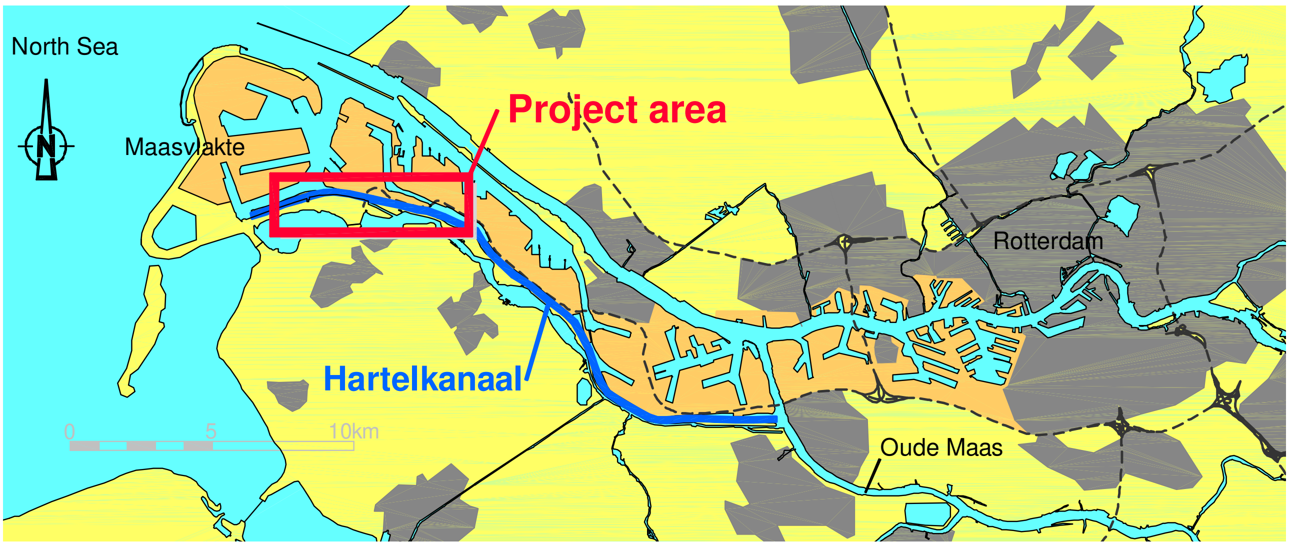


Figure 1: Map of project area

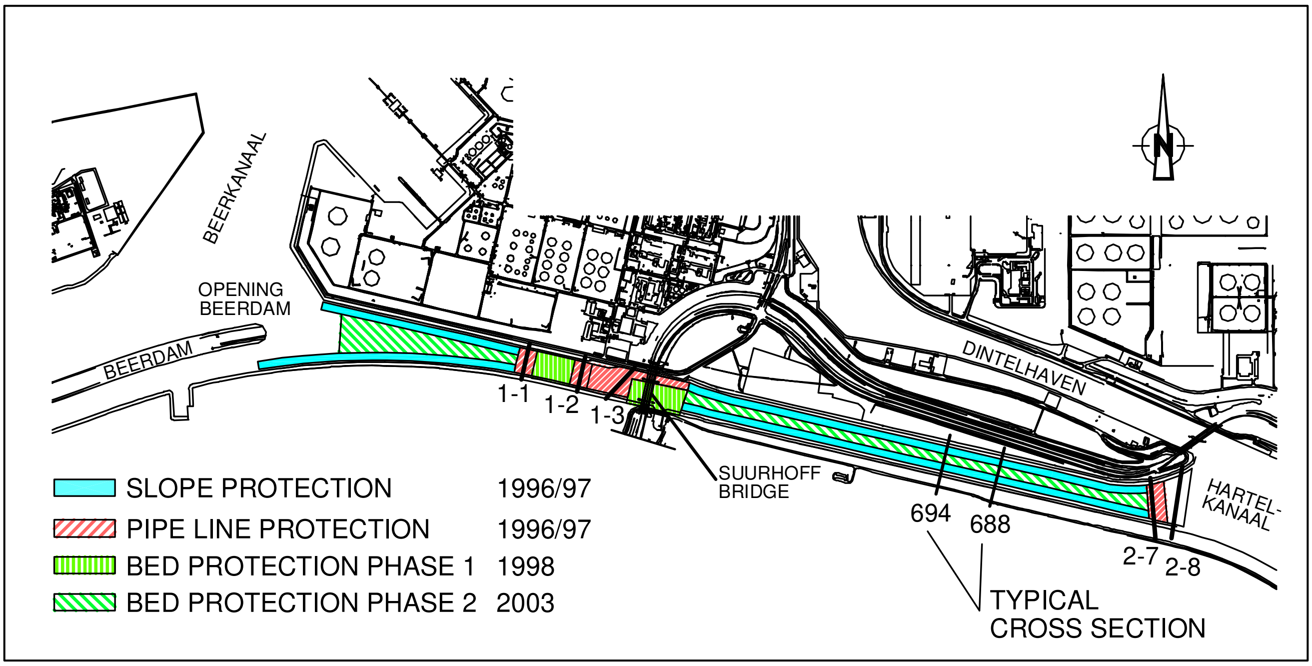


Figure 2: Project area

The banks are protection with rubble and hand placed stone from crest until NAP -4 m. The under water slope gradient is 1:4. Figure 3 gives a typical cross section for this part of the Hartelkanaal.

The Hartelkanaal is situated in the former delta of the river Rhine. Due to this fact some former flow channels are found here. From ground level till NAP -20 m the underground consist of sand which is strongly stratified with clay layers, dating from the Holocene period (Duinkerken, Calais). These Holocene layers are characterized by an irregular pattern of sand and thin, sandy clay layers of varying width.

III. PREDICTED HYDRAULIC AND MORFOLOGIC CHANGES

A. *Hydraulic conditions*

Until 1997 the tidal water volume of the Hartelkanaal and the connected harbor basins was entering and leaving the channel from the Oude Maas. As a result of opening the Beerdam, the main tidal water volume is entering and leaving the channel mainly from the seaward end of the channel. Also part of the river discharge from the Oude Maas is transported towards the sea through the Hartelkanaal. This means that, after excavating the Beerdam, the Hartelkanaal changed from a “dead ended channel” into a water discharging river “branch” with a relatively strong tidal flow. The maximum tidal discharge is about 7 times larger than the river discharge, and the river discharge volume is about 30% of the total tidal volume through the opening in the Beerdam. The tidal volume changes relatively strong along the Hartelkanaal: at the opening in the Beerdam the tidal volume is 2 times larger than the tidal volume at the debouchments at the Oude Maas.

The tidal discharge and currents in the western part of the Hartelkanaal depend on the local bed level. A lower bed level due to erosion will result in smaller tidal current velocities and meanwhile in an increase of tidal discharges, due to a decrease of hydraulic resistance.

The tidal discharges and the current velocities (averaged over a cross section) are calculated by the 1-dimensional model ZWENL for different bed levels. The result of these calculations for the original bed level (NAP-7,25 m) en for a strongly eroded canal bed (NAP-15 m) are presented in table I.

B. *Morphological modelling*

In general, river branches are characterized by a typical cross section which is related to a morphological equilibrium state. After changes in the hydraulic conditions, a river bed will reach a new equilibrium profile by erosion or sedimentation. In a tidal river, the equilibrium profile is strongly related to the total volume of water passing the river during a tidal period, i.e. the sum of water volume during ebb and flood period. Excavating the Beerdam will result in morphological changes of the bed of the Hartelkanaal in order to reach the equilibrium profile. In the most western part of the Hartelkanaal, between the Beerdam and the Dintelhaven, the tidal volume and therefore the equilibrium profile is the largest, while the cross section is relatively narrow. This means that a this part of the channel, strong erosion can be expected.

The western part of the Hartelkanaal lies in between two channel sections which a (much) larger cross profile. Westwards of opening in the Beerdam there is the Beerkanaal, which is 25 m deep and a 400 m wide entrance to several harbor basins for large seagoing vessels. The Beerkanaal has – contrary to the Hartelkanaal – a silty bed with very small current velocities near the bed, thus no sediment will be transported from the Beerkanaal into the Hartelkanaal. Eastward of the Dintelhaven there is a 2 km long relatively wide section of the Hartelkanaal. The width of this section is 500 m at NAP. Current velocities are smaller here than in the western section, so the amount of sediment which entering the western part of the canal during ebb is much smaller than the sediment transport capacity in the western section. This, combined with the fact that no sediment enters the canal from the Beerkanaal – means that the western section will erode at least until the cross section of the western part is as large as the cross section of the Hartelkanaal eastward of the Dintelhaven. This cross section is about 3.200 m.

The most conservative prediction of the equilibrium profile is that the canal bed will erode until the current velocities are reduced below the threshold for "start of movement" for the sand particles. With the predicted discharges from the ZWENDL model, this results in a equilibrium profile of 3.000 m^2.

Table 1 gives the actual surface of a cross section, using the original width of the channel at still water level and bank slopes of 1:4.

TABLE I.
MAXIMUM FLOW VELOCITIES AND DISCHARGES BEFOR E AND AFTER OPENING THE BEERDAM

	Before 1997 bed level NAP -7,25 m	Directly after opening bed level NAP -7,25 m	End situation bed level NAP -15 m
U_{max} [m/s]	0,2	1,25	1,0
Q_{max} [m^3/s]	136	1.004	1.120
Q_{min} [m^3/s]	-96	-1.200	-1.416
Surface of cross section [m^2]	763	763	1.106

It is very clear that in the end situation, with a bed level of NAP -15 m, the actual surface of a cross section is much smaller than the equilibrium profile. The equilibrium profile cannot be reached in between the original banks of canal. Increasing the width of the river was considered as a non-realistic option, taken into account the cost of dredging, replacing bank protection and "space" in the main port area. This implied that, in the future, the canal bed at a certain level, around NAP -15 m, should be protected for further erosion.

For a more detailed prediction of the morphological changes, a numerical model was build. Results of this model showed that directly west of the eastern pipeline protection, the minimum bottom level of NAP -15 m would be reached between 0,8 and 7,7 years. Directly east of the western pipeline protection (close to the Suurhoffbrug) the minimum bed level would be reached within 2,9 and more than 10 years.

C. Geotechnical stability

Before the opening of the Beerdam, an extended geotechnical survey was done, in order to check the failure rate of the Brielse Maasdijk, the primary water retaining structure at the southern bank of the Hartelkanaal According to Dutch legislation, the failure mechanisms of instability of the outer and inner slope, piping and overflow of the crest were taken into account.

The stability of the crest was calculated using the Bishop method. This was done for the original situation (bottom level NAP -7,25 m) and for the situation with a improved dike (crest height enlarged from NAP +5,70 m to NAP +6,20 m), related to the increased water level in the Hartelkanaal. Because severe scour of the bottom was expected due to the increase in flow velocity, also a situation with an under water slope of 1:4 and bottom level NAP -15 m, was investigated.

For the characteristic cross section 692 (see Figure 2) the stability factor is 1,40 for both the original cross section and the improved dike [x]. In the situation with the eroded river bed the stability factor is 1,37, which shows that erosion of the under water slope has an marginal effect on the stability. The stability factor must be at least 1,12, according to the safety level of the Brielse Maasdijk.

TABLE II.
STABILITY FACTORS IN CROSS SECTION 692

Situation	**Stability factor (Bishop)**
original situation	1.40
new situation: increased water level and improved dike	1.40
new situation: increased water level and improved dike eroded banks and bed level NAP -15 m	1.37
critical value	1.12

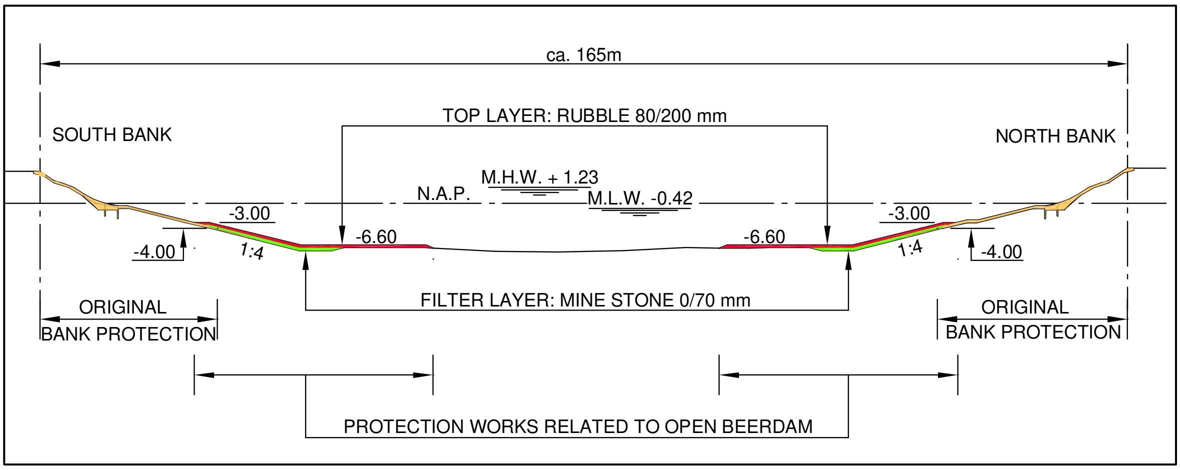


Figure 3: Typical cross section of the western part of the Hartelkanaal

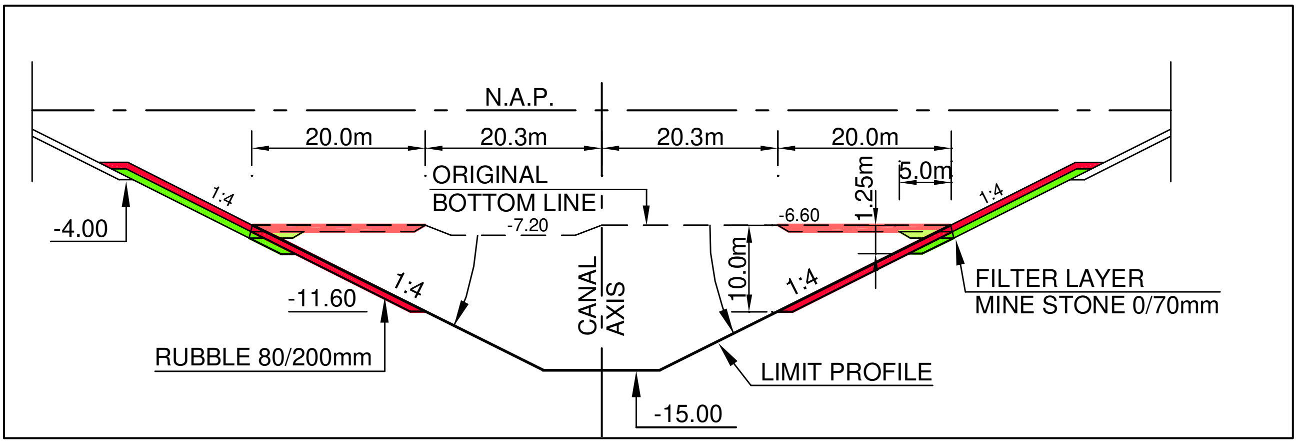


Figure 4: Slopeand toe protection with predicted settlement of toe

IV. Canal banks and toe protection

To guarantee the stability of the canal banks, the under water slope from the toe of the original protection at NAP -4 m to the canal bed and the adjacent 20 m of the flat canal bed were protected with a rubble layer.

A. Design method for the top layer

The design of the top layer of the protection works is rather special, because it is not based on a stability formulae with one maximum design load. The design is based on a quasi-probabilistic calculation of movement of stones using the Shields parameter and the transport relation of Paintal. The expected maintenance works, related to the movement of stones, are estimated.

The hydraulic loads which were taken into account are a combination of maximum ebb current, a seiche current and a return current caused by a full loaded push barge combination sailing in upstream direction, eccentric from the canal axis. The return current *under* the bow of the barge is taken into account, which is about two times larger than the mean return current *around* the ship. The stability of the stones is calculated in a quasi-probabilistic way, using the Shields-parameter, in which the bottom shear stress is caused by a combination of ebb, seiche and return current. Also variations in the current velocities are taken into account. Transport of the stones is calculated with the relations of Paintal. In a situation without ships, a rubble layer of standard sort 80-200 mm would be stable, without moving stones. In combination with a return current, at the slope and also at the 20 m width toe protection, the movement of stones is almost none. Only in more negative scenario where the current velocities are higher than the expected value, substantial movement of stones occurs. Under some circumstances in these scenarios one passing ship can move almost all stones in the upper layer. Because the stones mainly remain in the protected area, the damage is limited, although frequent monitoring is required. In the years after opening the Beerdam the flow velocities will decrease which reduces the movement of stones in the protection layer.

The rubble top layer consist of broken stone standard sort 80-200 mm. On the bank and the adjacent 5 m of the flat bed the 0,40 m thick top layer is placed at a filter layer, which consists of mine stone 0-70 mm with a layer thickness of 0,50 m. For the remaining 15 m toe protection the top layer was put directly on the canal bed.

This part of the bed protection was aimed to settle along with the erosion of the unprotected bed, until a stable slope gradient was reached.

B. Imperfect filter and falling apron mechanisms

For the settlement of the toe protection, two settlement mechanisms were taken into account: the "imperfect filter" and the "falling apron" mechanism.

In case of an imperfect filter, sand can wash out through the stones of a protective rubble layer because of the absence of a filter layer. Theoretically the permeable rubble layer should settle with the erosion of the unprotected bed uniform over the full stretch of the protection. Due to the settlement of the bed level protection the flow velocities reduce to a rate were no sand will be washed out and no further settlement occurs. There were no reliable relations available to predict the hydraulic loads at which a rubble layer of standard sort 80-200 mm is non sand permeable. Extrapolation of data for geometrically open filter layers of relatively fine stone, indicate that already in the situation with a non eroded bed and thus high velocities, the top layer will be almost sand tight. As this extrapolation is not verified, it is quite unreliable. Also the impact of water movements caused by ships is hard to quantify.

In case of a falling apron the settlement is due to local instabilities at the edge of a protection layer, caused by small scale slides or movement of stones. The settlement of the protection layers involves sideward movement of stones, which reduces the layer thickness and thereby effects the sand permeability and thus intensifies the imperfect filter mechanism. Laboratory test indicated a resulting slope of 1:3, for a situation of a rubble layer, 1 – 2 times D_{50}, placed on loosely packed sand.

Whereas in the Hartelkanaal the bed material is not loosely packed but contains a certain amount of clay, and the top layer is 2,7 stones thick, it is uncertain what the slope in the end situation will become.

It was expected that, with the relatively large layer thickness used for the slopes, the falling apron mechanism would dominate the imperfect filter. When also the falling apron mechanism; would not react as predicted, a steep slope could develop directly adjacent to the toe protection, which caused the risk of large scale slope instabilities (slides). To reduce this risk, the layer thickness of the toe protection was reduced, compared with the slope protections, to 0,30 m ((2 * D_{50}).

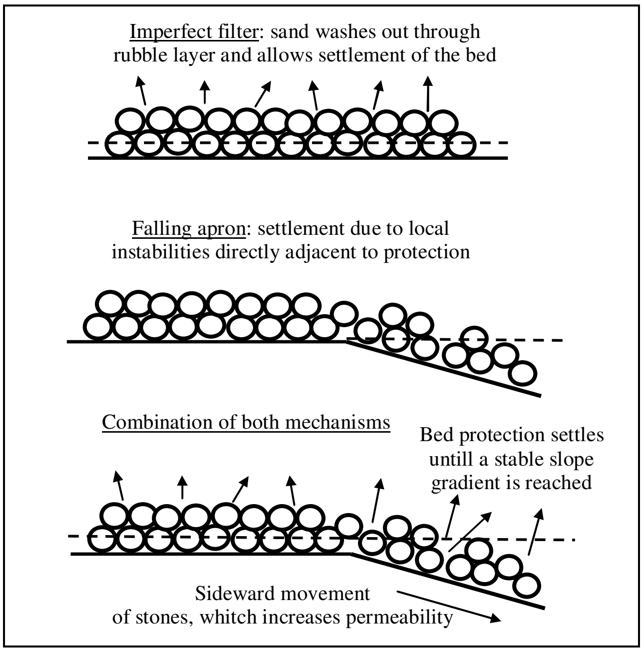


Figure 5:
Theory of "imperfect filter" and "falling apron"

The geotechnical stability of a steep slope directly adjacent to the toe protection was calculated using the Bishop method. In case of a slope gradient of 1:3, the stability factor is 1,39; for slope gradients steeper than 1:3 the stability factor reduces to 1,20 for a gradient of 1:2. The critical stability factor is 1,12.

When the bed in the middle of the canal would reach a level below NAP -15 m, it would be protected over the full width of the canal in order to prevent further erosion and instability of the slopes.

C. Pipeline crossings

In the western part of the Hartelkanaal at 4 locations, bundles of pipelines and cables cross the Hartelkanaal in one or more pipelines. The locations are indicated on the map in Figure 2 and are numbered 1-1, 1-2, 1-3 and 2-7/2-8. The expected bed level after the opening of the Beerdam is a few meters below the level of the pipelines. Protective rubble layers at these locations should prevent erosion of the canal bed. The minimum soil cover on the pipelines depends on the burial depth of the anchors of push barges and the tolerance in construction depth of the pipe lines. Laboratory tests performed in the 80s showed that the burial depth for a Danforth anchor can be as much as 1,40 m in a sand bed. With a 0,3 m tolerance in construction height for the pipeline itself, the minimum ground cover on a pipeline must be 1,70 m. This height includes the rubble protection layer.

During the process of the design of the protection works, the choice is made to protect the canal bed in between the pipeline protections *after* opening the Beerdam, when the canal bed is eroded to a certain level. The main reason for this choice is financial: due to the reduction in flow velocities caused by a lower level of the canal bed, the required dimensions of the rubble layer are less and thus the protection is cheaper. Some years of delay of investment cost, depending on the erosion rate of the canal bed, is financially attractive. On the other hand, it requires intensive monitoring to reduce the risk of instability of pipeline protections and slopes. Another negative consequence is the fact that the bed level of the pipeline protections becomes relatively high compared to the eroded, unprotected bed. This causes local changes in flow velocities, which effects the inland navigation. Also hydraulic loads on the pipeline protection itself increase.

In the first phase, three pipeline protections were realized: one protection for pipeline1-1, a protection for pipelines 1-2 and 1-3 and a protection for 2-7/2-8 (the so-called eastern protection). The protection for 1-3 is combined with the protection of the pier of the Suurhoffbrug. A few years after opening the Beerdam, in 1998, the area in between pipe lines 1-1 and 1-2 is covered with mine stone 10-125 mm. The slopes are covered with a variable layer thickness, in a way that the slopes are 1:3,5 until the level of the eroded canal bed. The flat bed is covered with a 0,30 m thick layer. Together with pipe lines 1-1 and 1-2 this is called the western pipeline protection. The area under the Suurhoffbrug is also protected with the same type of stone in this phase.

For the eastern pipeline protection, the choice is made to protect only pipeline2-7 with a new rubble layer. At the location of pipeline2-8, an existing rubble protection (sort 10-60 kg) shall give enough protection for a first phase. Dredging away this existing protection layer will give a high risk of damaging the pipelines, while the ground cover above the pipeline and cables has a critical thickness. Directly after the opening of the Beerdam, sedimentation is expected at this location, because of the supply of sediment from the narrow western part of the Hartelkanaal, where heavy erosion is expected. Monitoring the development of the bed level should indicate at which time erosion at the location of pipeline 2-8 brings the bed level at a critical level of NAP -6,75 m.

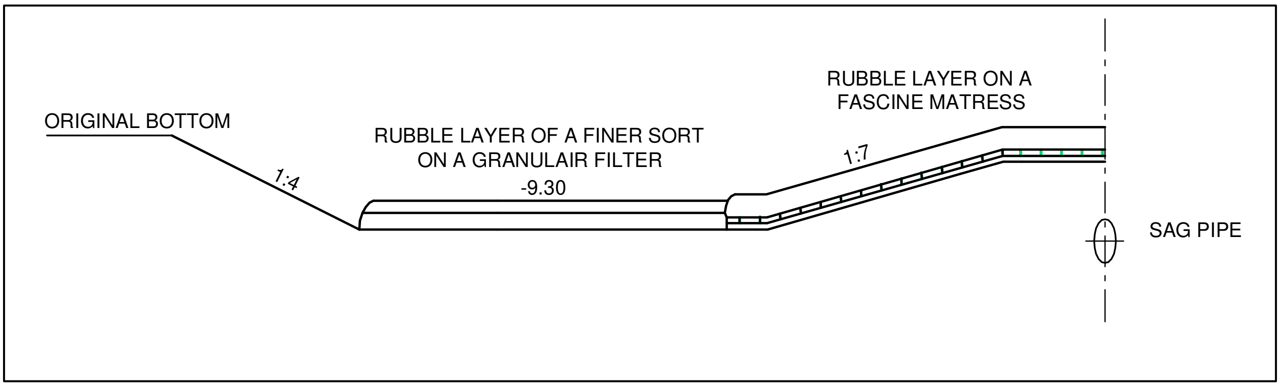


Figure 6: General design of pipeline protection

Figure 6 gives a general design of a pipeline protection. Directly above the pipeline, it consist of a rubble layer on a fascine mattress with a geotextile. The area in between the pipeline protections is covered with a rubble layer of a finer sort, on a granular filter layer. The transition area from the pipeline protection to the unprotected canal bed has a gentle slope gradient of 1:7, to prevent the separation of flow at the slope. Adjacent to the slope there is a horizontal bed protection ,10 – 20 m wide, at a level of NAP -9,30 m.

Comparable to the design of the bank protection works, the hydraulic loads are a combination of maximum ebb current, a seiche current and a return current caused by a full loaded push barge. The design is made for the morphological “end situation”, in which the bed of the Hartelkanaal is eroded until a level of NAP -15 m. In this situation the largest loads occur on the edges of the pipeline protection, caused by the difference between the bed level at the pipeline protections and the level of the unprotected canal bed. This extra load is calculated by multiplying the ebb and seiche current with factor 1,2.

The dimensions of the top layer are designed by calculating the movement of the stone for different scenario’s, and the expected maintenance, i.e. the volume of stones which is needed every year to prevent severe erosion of the top layer. The inaccuracies in both hydraulic conditions and the applied calculating methods, are calculated by a safety factor in the expected maintenance.

The western protection has a top layer consisting of broken stones of standard sort 10-60 kg (60 cm layer thickness) above pipeline1-1, and a top layer of standard sort 40-200 (70 cm layer thick ness) above pipeline1-2 and 1-3. The area between 1-2 and 1-3 is covered with standard sort 10-60 kg and 80-200 mm (30 cm layer thickness). Due to a wider cross section, at the eastern protection occur smaller flow velocities. At this pipeline protection, the top layer consists of broken stone 80-200 mm (layer thickness 30 to 40 cm), on a granular filter consisting of mine stone 0/70 mm. The expected amount of maintenance for the western protection varies between 10 – 1000 ton/year, for the eastern protection it is 0-80 ton/year.

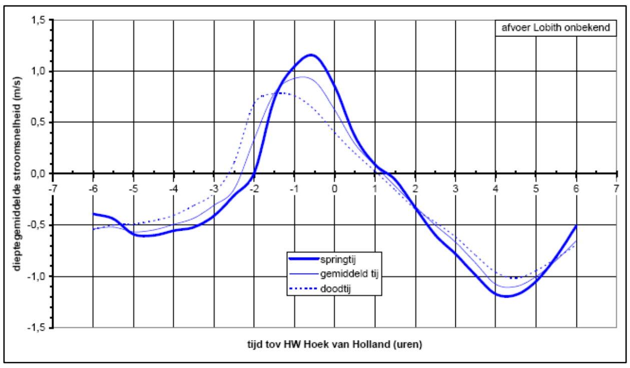


Figure 7: Average measured flow velocities at Suurhoffbrug

V. Scour and Erosion after Opening the Beerdam

A. *Measured hydraulic conditions*

On 8 November 1997, the Beerdam was excavated. At that moment, an extensive program of monitoring started, to be aware of possible erosion and possible risks for slope instability.

The velocities in the western part of the Hartelkanaal are measured at the Suurhoffbrug. The average velocities during one tidal motion, for spring, mean and neap tide are given in figure 1. The figure shows that the maximum velocity for both ebb and flood current is 1,2 m/s, slightly lower that was predicted.

B. *Erosion of the unprotected canal bed*

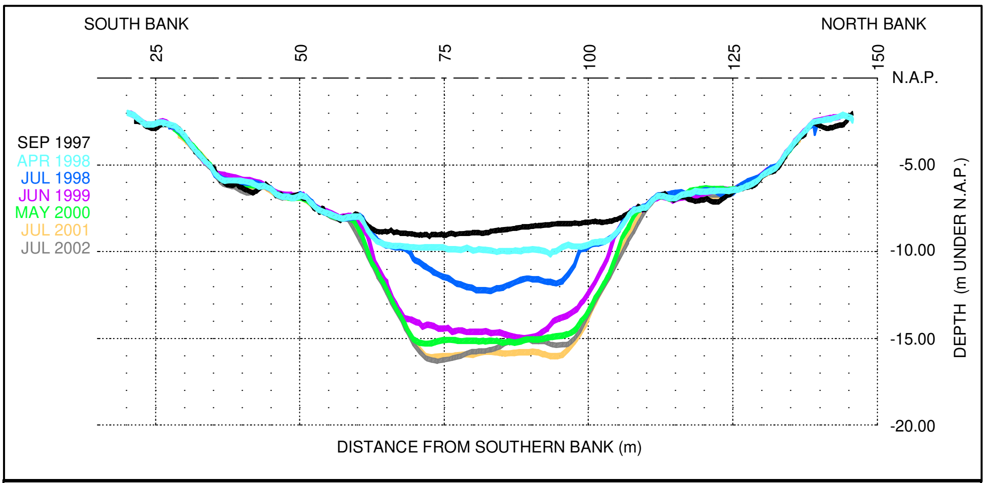
Figure 8 and 9 show the erosion process of the unprotected bed in cross sections 688 and 694 (Figure 2 gives a map for the location of this cross sections).

The most striking aspect in these figures is the very steep slope gradient which develop directly adjacent to the toe protection (20 m from the transition of the slope to the flat canal bed). Figure 10 and 11 give a closer look to the slopes for cross section 688. Within a period of a few months, very steep slopes develop, which move in the direction of the banks for only a few meters during the first two years, and by then remain completely stable. Directly from the start of the scour process, the slope gradient is very steep, between 1:1,5 and 1:2. It seems to be not at all related to the bed level, which continues to go down. This typical pattern can be seen in almost all cross sections in the western part of the Hartelkanaal.

The toe protection did not settle in the way it was expected. The top of the eroded slope moves back for a few meter towards the canal banks; this length could be a small part of the toe protection which does settle. The reason why the toe protection does not settle at all, or only a small part settles with the erosion of the bed, is not clear.

The flat bed between the steep slopes eroded very quick. In cross section 688 in the first half year, the canal bed eroded from NAP -9,0 m to NAP -14,5 m, with a maximum of 1,5 m/month, an average rate of almost 1 m/month. In the next years, the erosion continued at a lower rate until a level of NAP -16 m in 2003. The erosion is quite symmetric along the canal axis. The cross section 694 shows somewhat lower erosion rates. Here, the bed eroded from NAP -8,5 m to NAP -11,5 m in half a year. In 2003 the bed level was also NAP -16,5 m. The pattern of erosion looks slightly different especially at the southern bank, where an eroded “trench” seems to move sideward to the southern slope. The differences in both cross section are not examined but might be caused by differences in the cohesion of the bed material, due to local clay layers. In 2003, the unprotected bed was fixated with LD-slag, to prevent it from further erosion.

The geotechnical stability of the eroded slopes is investigated by the same way as was done before the opening of the Beerdam, using the Bishop method. In cross section 720 the stability factor in the original situation was 1,18. Using the measured depths of 1998, the stability factor is still 1,18, which is higher than the minimum of 1,12. The stability factor is not influenced by the bed level. Only in case of a slope with a steeper the gradient than 1,2, the stability factor reduces under the



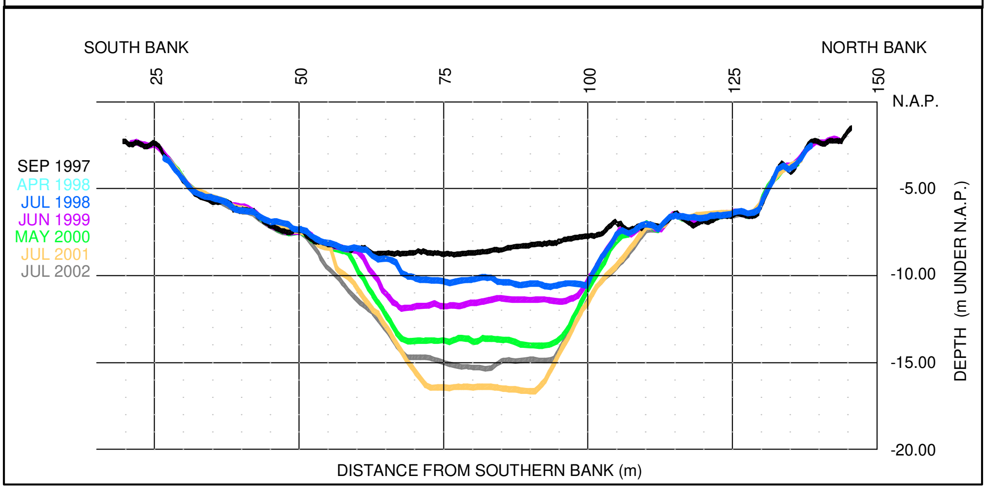
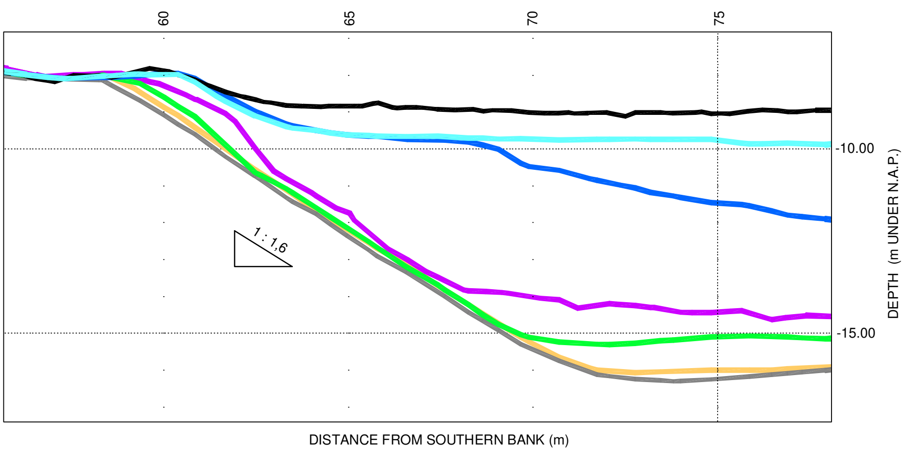


Figure 9: Eroded profile at cross section 688 (upper) and 694 (lower)



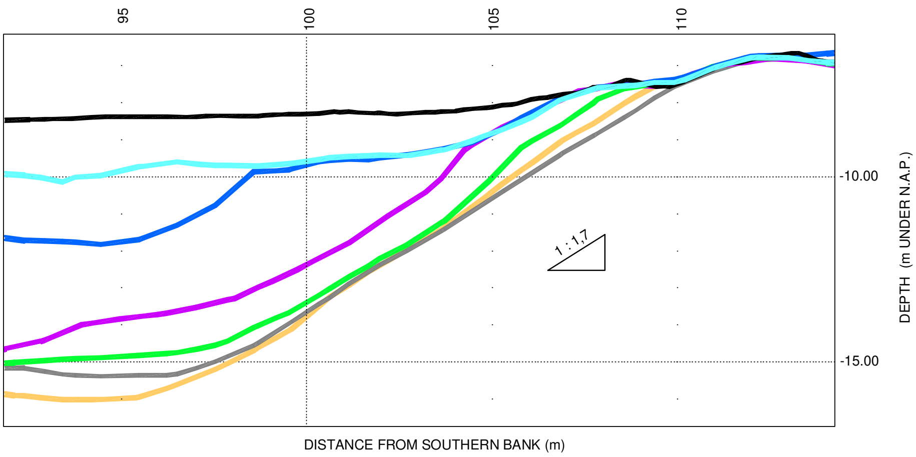


Figure 10 and 11: Eroded slopes at cross section 688 critical value.. The echo soundings shown in the cross sections, do never show a sign of slope instability or slides, despite the continuing erosion of the bed.

Figure 12 (not in draft version) gives a longitudinal profile for this part of the Hartelkanaal, and show the erosion process along the whole section. The erosion process does not start with a higher rate at the eastern or western end, but has a is quite constant along the length of this part of the canal. There are some sections which show less erosion. These differences might be caused by local clay layers, which are more resistant and cohesive. Also the "ridges" which develop but in later echo soundings are suddenly disappeared, must be due to the presence of clay.

C. *Scour adjacent to pipeline protections*

The longitudinal profile in figure 12 shows very clear the location and geometry of the pipeline protections. It shows that the protections itself and the protected slopes towards the unprotected bottom, are not eroded at all. The unprotected bed in between pipeline1.1 and 1-2 is eroded until a level of around NAP –13 m. In 1998, this part is protected, together with the unprotected bed around the Suurhoffbrug.

Directly east of the pipe line protection at pipeline2-7, within a few months a very deep scour hole developed. A detailed look at the echo soundings is given in figure 13. Within 3 months, the bed level lowered from NAP -7,2 m to NAP -13,4 m, and within a year to NAP – 16 m. The protection at pipeline2-8 remained stable, but the scour hole was close to it and was filled up for some meters with slag to prevent instability of the pipe lines.

The area directly east of pipeline2-8 is a transition area from erosion to sedimentation, because the width of the canal increases in landward direction. The flood current will not follow the strong curve in the Hartelkanaal, but go straightforward and decrease in velocity more eastward. On the other hand, the ebb current will accelerate when entering the western canal section due to the narrowing of the canal profile. During both ebb and flood current, there will occur erosion at this location. Due to the large differences in depth and salinity, at some moment during a tide cycle, waves occur at the surface.

VI. CONCLUSIONS

During the erosion process in the western part of the Hartelkanaal, when there is not a morphological equilibrium state, a typical profile develops in all cross sections. The canal bed in the centre of the canal is almost horizontal and sinks with a high rate as a result of erosion. At the same time at both sides of the canal, directly adjacent to the toe protections, very steep slopes develop, with slope gradients between 1:1,5 and 1:2,5. The slopes are in an almost stable position, independent of the depth of the flat canal bed.

Directly eastward of the pipeline protections works in the western section, a very deep scour hole (> 16 m) developed with a period of a year. The scour hole has a width in east-west direction of almost 70 m. At the western side it has a stable slope of around 1:2; at the eastern side the slope is much more gentle. The slope seems to be independent of the depth of the scour hole.

The toe protection of rubble 80/200 mm (D_{50} = 150 mm) with a of layer thickness of 0,30 m, directly placed on a sandy bottom shows no settlement in a situation with average tidal currents of 1,2 m/s in combination with return velocities below push barges of 1,5 to 2,1 m/s.

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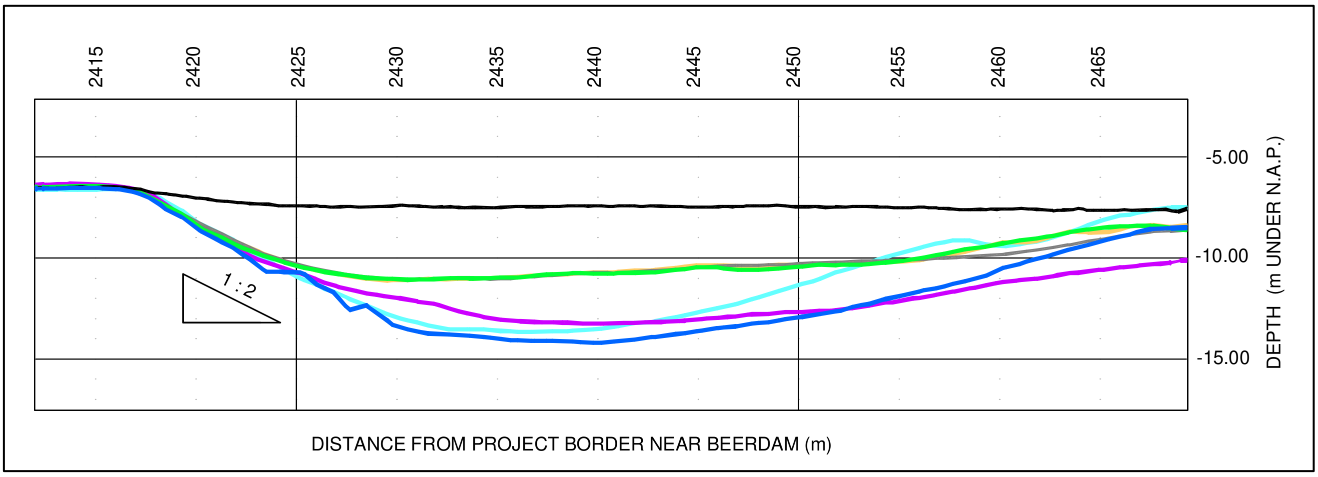

Figure 13: Scour hole at Dintelhaven, directly east of pipeline protection