Innovative concept overtopping dike: Crest Drainage Dike

Theoretical Study
Innovative concept overtopping dike: Crest Drainage Dike

Theoretical Study
Innovative concept overtopping dike: Crest Drainage Dike

Theoretical Study
CONTENTS

1 BACKGROUND OF THE CREST DRAINAGE DIKE
   1.1 ComCoast project
   1.2 Innovative concept for an overtopping dike: Crest Drainage Dike
   1.3 Approach for the theoretical study
   1.4 Requirements

2 WAVE OVERTOPPING
   2.1 Introduction
   2.2 Wave overtopping in theory
      2.2.1 Wave overtopping zones
      2.2.2 Overtopping flow into crest construction
      2.2.3 Short-crestedness wind-generated waves
   2.3 Wave overtopping calculations
      2.3.1 Approach and assumptions
      2.3.2 Results overtopping quantities
      2.3.3 Results overtopping velocities and layer thickness
      2.3.4 Conclusions

3 GLOBAL DIMENSIONS CREST DRAINAGE DIKE
   3.1 Introduction
   3.2 Discharge structure
      3.2.1 Introduction
      3.2.2 Calculations discharge pipe capacity
      3.2.3 Proposed discharge alternatives
   3.3 Crest construction
      3.3.1 Introduction
      3.3.2 Buffer capacity
      3.3.3 Width and shape of the crest construction
      3.3.4 Proposed crest construction alternatives
   3.4 Storage capacity hinterland

4 STABILITY OF THE DIKE
   4.1 Failure mechanisms
   4.2 Approach and assumptions
   4.3 Piping and heave
   4.4 Vertical and horizontal stability crest construction
   4.5 Macro-stability

5 CONSTRUCTIONAL ASPECTS CREST DRAINAGE DIKE
   5.1 Introduction
   5.2 Detailing of concrete crest construction
      5.2.1 Alternatives
      5.2.2 Detailing concrete construction
   5.3 Joints and transitions
5.4  Maintenance aspects  

6  EVALUATION CREST DRAINAGE DIKE  
6.1  Introduction  
6.2  Spatial planning issues  
6.3  Assessment Crest Drainage Dike  

7  PRELIMINARY DESIGN  
7.1  Introduction  
7.2  Preliminary design drawings concrete crest construction and landward pipes  
7.3  Preliminary design drawings concrete crest construction and seaward pipes  

8  COST ESTIMATION  
8.1  Introduction  
8.2  Estimation of building costs  
8.3  Maintenance costs  

9  PERMITS, EXEMPTIONS AND PROCEDURES  
9.1  Introduction  
9.2  Overview of permits, exemptions and procedures  

10  CONCLUSIONS AND RECOMMENDATIONS  
10.1  Feasibility and opportunities Crest Drainage Dike  
10.2  Further research  
10.3  Detailed design  

11  REFERENCES  

12  COLOPHON  

APPENDIX 1: Description of Crest Drainage Dike concept  
APPENDIX 2: Equations for wave overtopping calculations  
APPENDIX 3: Geo-hydrological calculation results  
APPENDIX 4: Cost estimation  
APPENDIX 5: ComCoast summary table estimated costs  
APPENDIX 6: Comments by WP3 users group and EU team + DHV reaction
1 BACKGROUND OF THE CREST DRAINAGE DIKE

1.1 ComCoast project

Based on several separately initiated studies on possible advantages and opportunities for overtopping dikes, a European project called ComCoast (Combined Functions in the Coastal Zone) was set up in 2004. The ComCoast project is carried out in the framework of the Interreg IIIb- North Sea programme. The objective of the ComCoast project is to study the possibilities of a wider coastal defence zone. Instead of raising and strengthening the dike, the coastal defence zone is widened. The wide coastal defence zone gives opportunities for new spatial developments and different types of functions within the zone.

The ComCoast concept involves measures in seaward and landward direction. One of the options in landward direction is an overtopping dike. In this concept the crest and/or inner slope of the dike is strengthened so that more overtopping of the dike can be allowed. Advantage is that heightening of the dike is not necessary and furthermore this can be combined very well with a wet and brackish zone behind the dike. This zone provides opportunities for nature development, recreation and possible concepts like ‘living in the water’ (houses on poles).

Figure 1-1: Ellewoutsdijk, ComCoast pilot area

In a State of the Art Study [CUR/RWS, 2005] an inventory was made of all available and at present often applied types of inner slope protections. It was found that the designs of the inner slope protections are usually based on methods and philosophies developed for the outer slope. Hence optimisation of the design methods and philosophies and a study on innovative types of inner slope protections for overtopping dikes are set as targets in the ComCoast project.
1.2 Innovative concept for an overtopping dike: Crest Drainage Dike

In April 2005 CUR issued a request to several parties to develop possible innovative concepts for overtopping dikes. DHV answered this request with the concept of the Crest Drainage Dike. The CUR selected the Crest Drainage Dike, together with two other concepts, to be worked out in more detail in a theoretical study. This theoretical study is presented to you here. In our presentation of the concept Crest Drainage Dike to the client (see [DHV, 2005]), we included an approach for the theoretical study. Based on the comments on this plan from a broad group of experts as collected by CUR, the activities foreseen in that approach were slightly adjusted (mainly shift in accents). The description of the concept of the Crest Drainage Dike as presented in [DHV, 2005] is included in Appendix 1. The conceptual sketch of the Crest Drainage Dike is shown in Figure 1-2. More detailed study in this phase might lead to other dimensions of the Crest Drainage Dike and other alternatives within the concept, but the main principles of the Crest Drainage Dike will of course be maintained.

![Figure 1-2: Conceptual sketch Crest Drainage Dike (not scaled!), source [DHV, 2005]](image)

1.3 Approach for the theoretical study

This theoretical study consists of the following activities:
1. Further study and detailing several aspects of the Crest Drainage Dike;
2. Preliminary Design Crest Drainage Dike;
3. Description of required licenses and permits;
4. Participation in meeting discussing the different concepts.

The results of the activities 1 to 3 are described in this draft report. The results of activity 4 will be taken into account when finalizing this report. The main aspects considered in activity 1 are:
- Wave overtopping (average and per wave), analysis available theory and execution of calculations;
- The required capacity and global dimensions Crest Drainage Dike;
- Impact of the Crest Drainage Dike on dike stability;
• Constructional aspects of the Crest Drainage Dike, including detailing of the construction and maintenance aspects;
• Evaluation of the Crest Drainage Dike with assessment criteria, including description of spatial planning issues;
• Cost estimation different alternatives for Crest Drainage Dike.

In the first introduction of the Crest Drainage Dike, see [DHV, 2005], these subjects were all described very roughly. In this theoretical study the subjects are studied in more detail.

The subjects are described in the following chapters in the presented order. This results in a preliminary design, as will be presented in chapter 7. Furthermore, one chapter is added with an overview of possibly required licences and permits for realisation of the Crest Drainage Dike. As pilot location for the design and calculations, the Hondsbossche Zeewering (see Figure 1-3) was used in this study.

![Aerial picture of pilot location, Hondsbossche Zeewering](image)

**Figure 1-3: Aerial picture of pilot location, Hondsbossche Zeewering (source?)**

### 1.4 Requirements

The minimum requirements for the Crest Drainage Dike are:
• The dike must be able to resist an average overtopping of at least 15 l/s/m² (with a maximum of 2300 l/m²). The more overtopping resistance, the better.
• Storm duration taken into account is three hours.
• The concept must be applicable for an inner slope gradient of 1:3 to 1:4.
• Heightening or widening of the dike is not allowed, the concept must fit within the present cross-sectional profile.
• Effects on nature, landscape and cultural values must be minimized.
• Maintenance efforts must be minimized during the first fifty years. Used materials must have a minimum life expectancy of fifty years.
• The costs for construction and maintenance must be acceptable compared to the costs for traditional heightening of the dikes.
• The concept must meet the current legal standards, such as the ‘Bouwstoffenbesluit’.
2 WAVE OVERTOPPING

2.1 Introduction

In this chapter, the wave overtopping rates in relation to the capacity and required dimensions of the Crest Drainage Dike are studied in more detail. Wave overtopping parameters can be expressed in average overtopping rates and in instantaneous overtopping rates (per wave). In Box 2-1 the difference between the parameters is explained and illustrated. For the Crest Drainage Dike, both parameters are of importance:

- The *average overtopping rate* determines the required average discharge capacity of the discharge pipes.
- The *instantaneous overtopping rate* determines the required buffer capacity of the crest construction to hold instantaneous overtopping water until it can be discharged. Some overtopping is allowed during extreme storm events in which the buffer capacity is not sufficient. Therefore, the erosion resistance of the grass inner slope is checked for expected maximum instantaneous overtopping rates and duration of this overtopping.
- Furthermore, the maximum *instantaneous overtopping rates* (expressed as the maximum overtopping flow velocities and flow layer thickness during such an event) determine the design load on the concrete crest construction.

**Box 2-1: Difference average-instantaneous overtopping parameters, source [Schuttrumpf and van Gent, 2003]**

The Dutch Guidelines [TAW, 1986] recommend critical overtopping rates on the basis of average overtopping rates for the design of sea dikes which depend on the quality of the soil and the quality of the turf. High instantaneous loads are not covered by average overtopping rates, as shown in the figure below. Therefore, the description of the instantaneous overtopping parameters is required separately.

To provide some more insight into the average and instantaneous overtopping rates, first the theory of overtopping and relevant aspects for the Crest Drainage Dike are described in short (paragraph 2.2) and some overtopping calculations were executed (paragraph 2.3), for average and instantaneous overtopping rates and for overtopping flow velocities and...
flow layer thickness. The knowledge obtained in this chapter regarding overtopping can be used when determining the required global dimensions in chapter 3.

2.2 Wave overtopping in theory

2.2.1 Wave overtopping zones

The overtopping rates depend mainly on the geometry of the seadike (berm, roughness, crest height, seaward slope) and the incoming waves (H_s and T_p). Wave overtopping can be reduced for example by heightening the dike, increasing the roughness of the outer slope, applying a berm in the outer slope or decreasing the slope gradient of the outer slope. Different zones/processes of wave overtopping are distinguished: incoming waves at the toe of the dike, breaking of waves on the seaward slope, wave run-up on the seaward slope, wave overtopping on the dike crest and wave overtopping on the landward slope. This is illustrated in Figure 2-1.

**Figure 2-1: Illustration wave overtopping zones at a seadike, source [Schuttrumpf and van Gent, 2003]**

2.2.2 Overtopping flow into crest construction

It is assumed that the crest level of an overtopping dike will not be located in the wave-breaking zone, as overtopping rates would become unrealistically high in that situation. This leads to the assumption that overtopping on the crest takes the form of a sheet flow with a certain flow velocity and flow layer thickness (see also Figure 2-1). Flow turbulence due to wave breaking is not taken into account in this zone. Taking into account the above, we can assume that the overtopping water on the crest will easily flow into the crest construction of the Crest Drainage Dike. The overtopping water will not “fly” over the crest like with low
crested breakwater structures where the crest is located in the more turbulent wave-breaking zone or with vertical structures.

**Outflow over sidewall of crest construction?**

An important aspect of the overtopping flow into the crest construction is whether the overtopping water might (partly) flow over the landward sidewall of the construction and on to the inner slope (see illustration of this process in Figure 2-2). Especially because of the small width of the construction (caused by the requirement that the concept must be applicable on a 3 m wide crest, see paragraph 3.1), this is a realistic risk for this concept. In a wide construction, most energy will be lost in the crest construction and the water will not be able to pass the vertical sidewall at the end. Whether this process will take place, first of all depends on flow velocities and flow layer thickness. These parameters will be calculated for the critical situation in paragraph 2.3, based on that information this subject is regarded further in paragraph 3.3, the determination of the global dimensions for the concrete crest construction.

**Figure 2-2: Illustration of possible (undesired) outflow over sidewall of crest construction**

![Diagram of outflow](image)

*When the crest construction is (partly) filled*  
When the crest construction is filled to a certain extent, overtopping water will start to flow over the present water layer in the crest construction (see Figure 2-3). At this point, the maximum buffer capacity to hold overtopping water in the crest construction is reached. The flow passes the crest construction without friction (water flowing over water) and reaches the inner slope despite of the crest construction. Some overtopping and thus hydraulic load on the inner slope is allowed within the Crest Drainage Dike concept. After all, the grass inner slope is maintained and, according to Dutch guidelines [TAW, 1987], a grass slope can normally resist average overtopping rates of 0.1 to 1.0 l/s/m². Because the average overtopping rates with longer duration will be trapped by the Crest Drainage Dike,
more study into the erosion resistance of the grass inner slope for instantaneous extreme overtopping events with relatively short durations (when the crest's construction buffer capacity is insufficient) is required for the Crest Drainage Dike. The remaining load on the inner slope is taken into account in paragraph 3.3, the determination of the required dimensions of the crest construction.

The moment when the overtopping water will start to flow over the water layer in the crest construction is reached before the crest construction is completely filled. Therefore, a correction factor should be applied to the crest’s volume when determining the buffer capacity of the crest construction. Because of a lack of theoretical knowledge or test experience on this very specific subject, based on our expert judgment we made an assumption for this correction factor to be 0.8. This implies that we assume that once the crest construction is 80 % filled, the overtopping water will start flowing over the water layer in the crest. This assumption is adopted in paragraph 3.3, the determination of the required dimensions of the crest construction. In a later stage however, this assumption should be verified and studied further.

2.2.3 Short-crestedness wind-generated waves

When regarding the overtopping rates, these are usually expressed in units/m². However, for the Crest Drainage Dike the longitudinal effect of overtopping over the dike length should be taken into account. The crest construction can spread the overtopping water in extreme conditions over the entire length of the dike. Important aspect in this is one of the characteristic properties of wind-generated waves; their short-crestedness (see Box 2-2).

Box 2-2: Long-crested and short-crested waves, source [v.d. Meer, 1995]

Long-crested means that the length of the wave crest is in principle assumed to be infinite. In investigations with long-crested waves the wave crest is as long as the wave board and the wave crests propagate parallel to one another. In nature, waves are short-crested. This implies that the wave crests have a certain length and the waves a certain main direction. The individual waves have a direction around this main direction. The extent to which they vary around the main direction (directional spreading) can be described by a spreading value. In nature, only long swell, for example coming from the ocean, has such long crests that it may virtually be called long-crested. A wind-generated wave field under storm conditions is short-crested.

Because of short-crestedness, the maximum instantaneous overtopping rates do not occur simultaneously along the entire length of the crest construction. When a wave overtops the dike at one point, a little but further the overtopping rate might be zero. This gives the crest
construction a chance to spread extreme overtopping rates in longitudinal direction of the dike. The influence of short-crestedness and directional spreading on the average overtopping rates has been studied before [v.d. Meer, 1995], but the instantaneous longitudinal overtopping effect has not been a subject of much research.

In [Goda, 2000] an illustration is given of short-crestedness of wind-generated waves. The output of a computer simulation using the directional wave spectrum gives a contour map of the surface elevation (relative to $H_{1/3}$). This was presented for wind waves (short crested) and swell (“long-crested”), see respectively in Figure 2-4a and Figure 2-4b.

Figure 2-4: Computer simulation of surface elevation contours of random waves, source [Goda, 2000]

![Wave figures](image)

The figures above indicate that the wave crests consist of short segments; an illustration of their short-crestedness. It is shown that the swell waves have longer crests, though not infinite. For wind-generated waves, the crest length is typically equal to 1-2 wavelengths ($L_{1/3}$). However, it cannot be assumed that over the entire length of the dike (assuming it has a length of several kilometers), no wave overtopping occurs simultaneously at all. Therefore it cannot be assumed that the overtopping volume of one wave can be spread over the entire dike length. Based on Figure 2-4 and expert judgment, when regarding the buffer capacity of the crest construction, in this study it is assumed that the length spreading reduction factor is 2. This means that the instantaneous overtopping rates in relation to the crest’s buffer capacity are divided by 2 because of the longitudinal spreading effect. This assumption is adopted in paragraph 3.3, in determining the global dimensions of the crest construction. In a later stage however, this assumption should be verified and studied further.

2.3 Wave overtopping calculations

2.3.1 Approach and assumptions

In this paragraph, the results of executed overtopping calculations and computations are described and analyzed. Two types of calculations were executed: 1) overtopping quantities
and 2) overtopping velocities and layer thickness. The approach and assumptions for both types of calculations are described below. In the following subparagraphs, the results are described separately for both types of calculations. In the end, the main conclusions in relation to the determination of the required global dimensions of the Crest Drainage Dike are given.

Overtopping quantities
For the determination of the wave overtopping quantities (average overtopping rates and instantaneous overtopping rates) a model called PC-overslag is available, see [TAW, 2002]. In PC-overslag the regularly used equations used for wave overtopping calculations are adopted. These equations and an explanation of the equations are given as background information in Appendix 2. For the overtopping calculations, the pilot location Hondsbossche Zeewering is used. Due to the generic character of this study, we do not focus on local aspects and results with high accuracy though.

The following assumptions have been made for the calculations:
- The exact cross profile (see Figure 2-5 and Table 2-1) has been adopted from [DWW, 2002b] and meets the minimum requirements as stated in paragraph 1.4.
- The results of the DWW study concerning wave overtopping quantities at this location serve as a reference for the calculations [DWW, 2002a];
- The calculations are first executed for a basic scenario, this concerns the scenario as is presented in [DWW, 2002a]. This scenario is more stringent than the current obligatory hydraulic conditions [RWS, 2001] because the future expected increase in wave period has already been adopted.
- From the basic scenario, first the influence of a mean sea level rise (because of climate change) is calculated by raising the water level with constant steps. The results are presented in relation to the crest margin.
- Furthermore, the influence of increasing the wave height and wave period compared to the basic scenario is calculated.

Figure 2-5: Cross profile of the Hondsbossche Zeewering, source [DWW, 2002b]

![Cross profile of the Hondsbossche Zeewering](image)
Table 2-1: Description of the applied cross profile and revetments

<table>
<thead>
<tr>
<th>Name</th>
<th>Hondsbosch Zeewering (dp 2.4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>x y coordinates</td>
<td>105036, 529278</td>
</tr>
<tr>
<td>Revetments</td>
<td>Basalt-pillars for section to 1.2 m +NAP</td>
</tr>
<tr>
<td></td>
<td>Asphalt for section 1.2 – 7 m +NAP</td>
</tr>
<tr>
<td></td>
<td>Grass for section 7 – 12.02 m +NAP</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Profile</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>13.87</td>
<td>-0.43</td>
</tr>
<tr>
<td></td>
<td>19.84</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>40.55</td>
<td>3.82</td>
</tr>
<tr>
<td></td>
<td>52.26</td>
<td>4.93</td>
</tr>
<tr>
<td></td>
<td>66.28</td>
<td>5.72</td>
</tr>
<tr>
<td></td>
<td>86.5</td>
<td>12.02</td>
</tr>
</tbody>
</table>

Overtopping velocities and layer thickness
For the determination of the global dimensions and required strength of the construction, more information is required about the overtopping velocities and layer thickness (water tongue on top of the crest). Most of the equations and methods applied in this section have been derived from model tests, see [van Gent, 2002] and [Schüttrumpf and Van Gent, 2003]. This topic is still subject of ongoing research, the results presented in this section should therefore be considered as an estimate.

The following assumptions have been made for the calculations:

- The exact cross profile (see Figure 2-5 and Table 2-1) has been adopted from [DWW, 2002b] and meets the minimum requirements as stated in paragraph 1.4.
- The overtopping velocities and layer thickness on the crest are determined at the outer crest line. In practice the velocities and layer thickness will decrease while passing the crest; therefore this is a conservative assumption. This assumption is similar to the prevailing method in the VTV [RWS, 2004].
- Besides the overtopping velocity at the outer crest line, the velocities on the inner slope are calculated, for this calculation it is also assumed that the overtopping flow velocity and flow layer thickness do not change along the crest.
- The wave run-up velocity and layer thickness exceeded by 2% of the incoming waves is calculated. This gives a good estimate of extreme flow velocities, as is required for determining the design load on the crest construction and inner slope.

2.3.2 Results overtopping quantities

In Table 2-2 the input and results of the PC-overslag calculations are shown for the basic scenario at Hondsbosch Zeewering. The parameter q [l/s/m] represents the average overtopping rate, while the parameters V_{max} and V_{10%} give information on the instantaneous overtopping rate per wave. V_{10%} gives the overtopping rate per wave that is exceeded by 10% of the waves.
Table 2-2: Overtopping quantities for the Hondsbosche Zeewering for the present situation

<table>
<thead>
<tr>
<th>$H_s$ [m]</th>
<th>$T_{m,1.0}$ [s]</th>
<th>$\beta$ [°]</th>
<th>Swl [m]</th>
<th>$q$ [l/s/m]</th>
<th>Crest level</th>
<th>$Z_{2%}$ + swl [m]</th>
<th>$V_{\text{max}}$ [l/wave/m]</th>
<th>$V_{10%}$ [l/wave/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>4.72</td>
<td>1.64</td>
<td>12.02</td>
<td>11.75</td>
<td>3793</td>
<td>3102</td>
</tr>
</tbody>
</table>

Table 2-3 shows the influence of a decrease in free crest height on the overtopping rates. The free crest height $Z_a$ is defined as the crest level – still water level. It can be seen that the overtopping rates in the beginning do not increase very rapidly with an increasing water level. The minimum requirement the Crest Drainage Dike must meet, is to be able to resist an average overtopping rate of 15 l/s/m². According to Table 2-3, this situation will not occur before a water level rise of about 1.70 m has taken place (assuming the other parameters stay the same). After that point however, the overtopping rates increase significantly more rapidly, this occurs because the still water level has become higher than the berm level. The berm then looses its effectivity.

Table 2-3: Influence of decrease in free crest height

<table>
<thead>
<tr>
<th>$Z_a$ [m]</th>
<th>$H_s$ [m]</th>
<th>$T_{m,1.0}$ [s]</th>
<th>$\beta$ [°]</th>
<th>$q$ [l/s/m]</th>
<th>$Z_{2%}$ [m]</th>
<th>$V_{\text{max}}$ [l/wave/m]</th>
<th>$V_{10%}$ [l/wave/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.30</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>1.6</td>
<td>7.03</td>
<td>3793</td>
<td>3102</td>
</tr>
<tr>
<td>7.10</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>2.0</td>
<td>7.04</td>
<td>4030</td>
<td>2945</td>
</tr>
<tr>
<td>6.90</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>2.4</td>
<td>7.06</td>
<td>4272</td>
<td>2807</td>
</tr>
<tr>
<td>6.70</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>2.9</td>
<td>7.09</td>
<td>4536</td>
<td>2699</td>
</tr>
<tr>
<td>6.50</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>3.8</td>
<td>7.18</td>
<td>4963</td>
<td>2653</td>
</tr>
<tr>
<td>6.30</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>5.1</td>
<td>7.28</td>
<td>5500</td>
<td>2665</td>
</tr>
<tr>
<td>6.10</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>6.8</td>
<td>7.40</td>
<td>6159</td>
<td>2733</td>
</tr>
<tr>
<td>5.90</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>9.3</td>
<td>7.54</td>
<td>6982</td>
<td>2863</td>
</tr>
<tr>
<td>5.70</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>12.7</td>
<td>7.68</td>
<td>8022</td>
<td>3065</td>
</tr>
<tr>
<td>5.50</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>17.5</td>
<td>7.84</td>
<td>9340</td>
<td>3350</td>
</tr>
<tr>
<td>5.30</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>24.1</td>
<td>8.02</td>
<td>11006</td>
<td>3731</td>
</tr>
<tr>
<td>5.10</td>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>33.3</td>
<td>8.20</td>
<td>13104</td>
<td>4225</td>
</tr>
</tbody>
</table>

Table 2-4 shows the influence of an increase in wave height on the overtopping rates. The applied values for the wave heights are for waves in shallow water near the construction. The wave height at the dike toe (as shown in Table 2-4) is depth-limited; the combinations of wave height and water level as presented here might therefore not be realistic. However, it does give some idea of the influence of the wave height on overtopping rates. It shows that the average overtopping rate triples when increasing the wave height by 20 %. As was expected, the wave height has a great influence on overtopping rates.

Table 2-4: Influence of increase in wave height

<table>
<thead>
<tr>
<th>$H_s$ [m]</th>
<th>$Z_a$ [m]</th>
<th>$T_{m,1.0}$ [s]</th>
<th>$\beta$ [°]</th>
<th>$q$ [l/s/m]</th>
<th>$Z_{2%}$ [m]</th>
<th>$V_{\text{max}}$ [l/wave/m]</th>
<th>$V_{10%}$ [l/wave/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.90 (basic)</td>
<td>7.30</td>
<td>12.60</td>
<td>11</td>
<td>1.6</td>
<td>7.03</td>
<td>3793</td>
<td>3102</td>
</tr>
<tr>
<td>3.50 (+20%)</td>
<td>7.30</td>
<td>12.60</td>
<td>11</td>
<td>5.1</td>
<td>8.15</td>
<td>6296</td>
<td>3279</td>
</tr>
<tr>
<td>3.90 (+35%)</td>
<td>7.30</td>
<td>12.60</td>
<td>11</td>
<td>8.6</td>
<td>8.72</td>
<td>8183</td>
<td>3732</td>
</tr>
<tr>
<td>4.50 (+55%)</td>
<td>7.30</td>
<td>12.60</td>
<td>11</td>
<td>16.5</td>
<td>9.50</td>
<td>11635</td>
<td>4648</td>
</tr>
</tbody>
</table>
Table 2-5 shows the influence of an increase in wave period on overtopping rates. The volumes of overtopping for each wave depend on the number of incoming waves and the mean wave period ($T_m$). The following relation has been applied to determine the mean wave periods for the increasing peak periods [TAW, 2002]:

$$\frac{T_{m-1.0}}{T_m} = 1.15$$

The results show that, as was expected, the wave period also has great influence on the average and maximum instantaneous overtopping rates. The volume per wave $V_{10\%}$ increases much less significantly (this is also true for an increase in wave height and increase in still water level when the berm is passed, see Table 2-3 and Table 2-4).

<table>
<thead>
<tr>
<th>$T_{m-1.0}$ [s]</th>
<th>$Z_a$ [m]</th>
<th>$H_s$ [m]</th>
<th>$\beta$ [°]</th>
<th>$q$ [l/s/m]</th>
<th>$Z_{2%}$ [m]</th>
<th>$V_{\text{max}}$ [l/wave/m]</th>
<th>$V_{10%}$ [l/wave/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.60</td>
<td>7.30</td>
<td>2.90</td>
<td>11</td>
<td>1.6</td>
<td>7.03</td>
<td>3793</td>
<td>3102</td>
</tr>
<tr>
<td>13.60 (+8%)</td>
<td>7.30</td>
<td>2.90</td>
<td>11</td>
<td>3.1</td>
<td>7.65</td>
<td>5298</td>
<td>3337</td>
</tr>
<tr>
<td>14.60 (+15%)</td>
<td>7.30</td>
<td>2.90</td>
<td>11</td>
<td>4.9</td>
<td>8.19</td>
<td>6533</td>
<td>3552</td>
</tr>
<tr>
<td>15.60 (+25%)</td>
<td>7.30</td>
<td>2.90</td>
<td>11</td>
<td>5.7</td>
<td>8.51</td>
<td>6880</td>
<td>3525</td>
</tr>
</tbody>
</table>

Above, all influences were regarded separately. In practice this will never be the case. An increase in still water level (decrease in free crest height) is the most likely and realistic development in the near future because of mean sea level rise. Predictions about an increase in storminess are also available, but these figures are under a lot of discussion. The expected increase in wave period for the obligatory hydraulic conditions coming years was already taken into account in the basic scenario. Table 2-6 gives a more realistic combination of parameters that leads to the average overtopping rate of 15 l/s/m (minimum average overtopping Crest Drainage Dike must be able to withstand), the exact correlation between decrease of free crest height and increase of wave height has not been studied further here, this goes beyond the scope of this study.

<table>
<thead>
<tr>
<th>$T_{m-1.0}$ [s]</th>
<th>$Z_a$ [m]</th>
<th>$H_s$ [m]</th>
<th>$\beta$ [°]</th>
<th>$q$ [l/s/m]</th>
<th>$Z_{2%}$ [m]</th>
<th>$V_{\text{max}}$ [l/wave/m]</th>
<th>$V_{10%}$ [l/wave/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.60</td>
<td>7.30</td>
<td>2.90</td>
<td>11</td>
<td>1.6</td>
<td>7.03</td>
<td>3793</td>
<td>3102</td>
</tr>
<tr>
<td>14.60</td>
<td>6.90</td>
<td>3.30</td>
<td>11</td>
<td>15</td>
<td>9.03</td>
<td>11317</td>
<td>4684</td>
</tr>
</tbody>
</table>

**Probability frequency instantaneous overtopping rates**

Different parameter combinations that lead to the minimum average overtopping rate (15 l/s/m) have been identified in the calculation results. For the analysis of the required buffer capacity of the Crest Drainage Dike, it is of interest to obtain some more insight in the probability of occurrence of instantaneous overtopping rates belonging with an average overtopping rate of 15 l/s/m. In Table 2-7, this is shown for three hydraulic combinations:

1. Increase in still water level with 1.70 m;
2. Increase in wave height to 4.40 m (at present 2.90 m);
3. Combination of increase in still water level with 0.40 m, increase in wave height to 3.30 m (at present 2.90 m) and increase in wave period to 14.60 s (at present 12.60 s).

The results show that the combination of parameters changes gives a significantly higher increase in instantaneous overtopping rates than changing only one of the hydraulic parameters (for equal average overtopping rates). This is taken into account in paragraph 3.3, with determining the required buffer capacity and remaining load on inner slope in extreme conditions.

**Table 2-7: Overtopping volumes per wave [l/wave/m] when average overtopping rate is 15 l/s/m**

<table>
<thead>
<tr>
<th>$T_{m,1.0}$ [s]</th>
<th>$Z_{0}$ [m]</th>
<th>$H_{s}$ [m]</th>
<th>$q$ [l/s/m]</th>
<th>$V_{\text{max}}$</th>
<th>$V_{2%}$</th>
<th>$V_{5%}$</th>
<th>$V_{10%}$</th>
<th>$V_{33%}$</th>
<th>$V_{50%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.60</td>
<td>5.60</td>
<td>2.90</td>
<td>15</td>
<td>8642</td>
<td>6840</td>
<td>4540</td>
<td>3196</td>
<td>1192</td>
<td>645</td>
</tr>
<tr>
<td>12.60</td>
<td>7.30</td>
<td>4.40</td>
<td>15</td>
<td>11003</td>
<td>9078</td>
<td>6360</td>
<td>4478</td>
<td>1669</td>
<td>903</td>
</tr>
<tr>
<td>14.60</td>
<td>6.90</td>
<td>3.30</td>
<td>15</td>
<td>11317</td>
<td>9495</td>
<td>6652</td>
<td>4684</td>
<td>1746</td>
<td>945</td>
</tr>
</tbody>
</table>

**Traditional heightening of the dike**

Additional a calculation was made to compare the Crest Drainage Dike to traditional heightening of the dike. If an increase in the hydraulic conditions (increase still water level, wave height, wave period) leading to an average overtopping rate of 15 l/s/m, should be handled in a traditional way (heightening of the dike to meet the requirement of maximum average overtopping rate 1 l/s/m) it was calculated that the dike must be heightened with approximately 3.5 to 4.0 meters (depending on the exact combination of changed parameters).

### 2.3.3 Results overtopping velocities and layer thickness

The calculated overtopping velocities and layer thickness (with the use of equations 2-11 and 2-12 in Appendix 2), exceeded by 2% of the incoming waves, are shown in Table 2-8. Calculations have been made for different combinations (see previous section) corresponding to an average wave overtopping of 15 l/s/m.

**Table 2-8: Calculated overtopping velocity and layer thickness at outer dike crest line**

<table>
<thead>
<tr>
<th>$H_{s}$ [m]</th>
<th>$T_{m,1.0}$ [s]</th>
<th>$\beta$ [$^\circ$]</th>
<th>$Swl$ [m]</th>
<th>$q$ [l/s/m]</th>
<th>Crest level</th>
<th>$Z_{0}$ [m]</th>
<th>$Z_{2%} + Swl$ [m]</th>
<th>$Z_{2%}$ [m]</th>
<th>$u_{\text{a,2%}}$ [m/s]</th>
<th>$h_{\text{a,2%}}$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.90</td>
<td>12.60</td>
<td>11</td>
<td>6.42</td>
<td>15</td>
<td>12.02</td>
<td>5.60</td>
<td>14.17</td>
<td>7.75</td>
<td>6.0</td>
<td>0.32</td>
</tr>
<tr>
<td>4.40</td>
<td>12.60</td>
<td>11</td>
<td>4.72</td>
<td>15</td>
<td>12.02</td>
<td>7.30</td>
<td>14.09</td>
<td>9.37</td>
<td>5.9</td>
<td>0.31</td>
</tr>
<tr>
<td>3.30</td>
<td>14.60</td>
<td>11</td>
<td>5.12</td>
<td>15</td>
<td>12.02</td>
<td>6.90</td>
<td>14.15</td>
<td>9.03</td>
<td>5.9</td>
<td>0.32</td>
</tr>
</tbody>
</table>

From the results, it can be concluded that the flow velocities and layer thickness are similar for different combinations leading to an average overtopping of 15 l/s/m. The results are taken into account as design load for the concrete crest construction (see paragraph 3.3 and chapter 5).

If the capacity of the Crest Drainage Dike is insufficient to trap all overtopping in extreme conditions, overtopping will reach the inner slope. Maximum flow velocities to be expected...
in that situation are required to determine if the erosion resistance of the inner grass slope is sufficient. With equations 2-13 and 2-14 in Appendix 2, the maximum occurring flow velocities at the inner slope were calculated. With \( f = 0.03 \) (Chézy coefficient 50 \( m^{1/2}/s \)), this results in a maximum value for \( u_{0.2} \) of 7.2 m/s at the inner slope.

The calculated flow velocities on the crest and inner slope are of the same order as can be found in literature and other studies.

2.3.4 Conclusions

The main conclusions that are drawn from the executed calculations in relation to the determination of the required dimensions of the Crest Drainage Dike are:

- The required average discharge capacity of the Crest Drainage Dike is 15 l/s/m.
- Obtained insight on probability of exceedance of instantaneous overtopping rates (see Table 2-7) can be used when determining the required buffer capacity of the crest construction and the remaining hydraulic load on the inner grass slope.
- Design overtopping flow speed taken into account for the crest construction is 6.0 m/s, overtopping flow layer thickness 0.3 m.
- Design overtopping flow speed for the inner slope is 7.2 m/s, period of occurrence is dependent on the buffer capacity of the crest construction in relation to the instantaneous overtopping rates.
3 GLOBAL DIMENSIONS CREST DRAINAGE DIKE

3.1 Introduction

Based on the obtained knowledge on overtopping in the previous chapter and the requirements the Crest Drainage Dike must meet (see paragraph 1.4), the global principles and dimensions of possible alternatives for the discharge construction can be determined in paragraph 3.2. After that, the global principles and dimensions of the concrete crest construction in relation to the required buffer capacity are determined in paragraph 3.3. In that paragraph, the required and available buffer capacity of the crest construction is also related to the erosion resistance of the grass inner slope. Furthermore, the possible connection of the discharge from the Crest Drainage Dike to the hinterland and the storage capacity of the hinterland is described in paragraph 3.4. The conclusions in this chapter form the basis for the following chapters concerning dike stability, structural aspects, cost estimation and spatial planning issues.

3.2 Discharge structure

3.2.1 Introduction

In the first introduction of the Crest Drainage Dike, see [DHV, 2005], discharge pipes in landward direction with a diameter of 250 mm every 30 meters were proposed. This was based on a simplified Darcy-Weissbach calculation, which takes into account friction losses in the pipes, but neglects entrance and exit losses. The hydraulic head was assumed to be the main driving force over the pipes. In this study, these assumptions were studied in more detail.

The pipes in landward direction, buried under the inner slope, have a length of 40 to 50 meters. The hydraulic head is about 13 meters. From calculations in this study it turned out that not the hydraulic head is critical for the discharge capacity of these pipes, it is the water pressure above the pipe that is needed to force the water into the pipes (because of entrance losses) that determines the discharge capacity of the pipes. Following this, the flow velocities in the discharge pipe should be determined with the following equation:

\[ v = \sqrt{\frac{2gh}{}} \]

where:
- \( v \) = flow velocity in the pipe [in m/s]
- \( g \) = gravitational acceleration [9.8 m/s²]
- \( h \) = water pressure height above pipe entrance [m]

For a constant discharge, it is required to maintain a certain pressure height above the pipe entrance at all times. Therefore, the construction of a deeper lying pit is needed for the discharge pipes (see Figure 3-1). The water flows from the crest construction into the pit, and then into the discharge pipes. In the crest construction a gutter is realized along the sides to discharge possible rainwater towards the pit.
The obtained pressure height above the pipe entrance is equal to the depth of the pit plus the water height in the crest construction. The depth of the pit depends on the diameter of the discharge pipe, it is defined as the pipe diameter + 0.10 m (wall thickness and margin).

Friction losses and exit losses are not taken into account. The pipe in landward direction is realized under a 1:3 slope; the water will flow through it very fast and the pipe will not be completely filled with water, which makes the friction losses negligible. The exit of water is assumed to be under water (in a drainage canal), a certain pressure height is required to avoid flow deceleration at the exit. Because of the steep slope of the pipe (1:3), it can be assumed that this pressure height is easily available above the pipe exit.

**Alternative location discharge pipe**

Because it turned out that not the hydraulic head, but the water pressure height above the pipe entrance is critical for the discharge capacity of the pipes, the discharge pipe can just as well be located towards the seaward side of the dike. The discharge pipe doesn’t need a slope to obtain a certain hydraulic head. It can be situated directly outward, with a slope of minimum 1:10 to discharge small water quantities (for example rainwater). The outer slope is assumed to be sufficiently protected to withstand the hydraulic load of water flowing out of the discharge pipe (this should be checked in the local situation though). Furthermore, the discharge pipe should exit the dike above the wave-breaking zone to prevent a constant water pressure from the seaward side. In this way free outflow of the water can be assured, therefore exit losses can be ignored. Friction losses can also be ignored because of the small length of the pipe (about 10 meters) compared to the pipe diameter.

Point of attention for this alternative is the possible entrance of water into the discharge pipes during wave run-up on the outer slope. As long as the water pressure inside the crest construction is sufficient (which will be the case during the peak of a storm event, when the crest construction is expected to be filled with some water most of the time), the amount of inflow can probably be ignored. Furthermore, extra measures at the exit of the discharge pipe can be taken to prevent water from flowing into the pipe. For example the discharge pipe can be lengthened so it sticks out of the slope a bit. The outflow of the discharge pipe should be worked out in the detailed design.
3.2.2 Calculations discharge pipe capacity

The executed calculations are applicable for both the discharge pipes towards the landward side and the seaward side. For both alternatives, the water pressure height above the discharge pipe is critical for the discharge capacity. Exit and friction losses can be ignored (see previous section). With the equation as introduced in paragraph 3.2.1, the flow velocities in the discharge pipe can be calculated for different pipe diameters. This also leads to the discharge capacity for the pipe and the maximum distance between the pipes to meet the minimum discharge condition (15 l/s/m) for the Crest Drainage Dike. The results are shown in Table 3-1.

<table>
<thead>
<tr>
<th>Pipe diameter [mm]</th>
<th>Put depth [m]</th>
<th>water height in the crest construction</th>
<th>0,1 m</th>
<th>0,4 m</th>
<th>0,7 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>[x m] [v m/s] [Q m^3/s]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>0.35</td>
<td>10 2.97 0.15</td>
<td>13 3.83 0.19</td>
<td>15 4.54 0.22</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>0.40</td>
<td>15 3.13 0.22</td>
<td>19 3.96 0.28</td>
<td>22 4.64 0.33</td>
<td></td>
</tr>
<tr>
<td>400</td>
<td>0.50</td>
<td>29 3.43 0.43</td>
<td>35 4.20 0.53</td>
<td>41 4.85 0.61</td>
<td></td>
</tr>
<tr>
<td>500</td>
<td>0.60</td>
<td>49 3.70 0.73</td>
<td>58 4.43 0.87</td>
<td>66 5.05 0.99</td>
<td></td>
</tr>
</tbody>
</table>

with:
- **x** required distance between pipes to meet the minimum discharge condition of 15 l/s/m
- **v** flow velocity in the pipes
- **Q** discharge capacity

The results are shown for different water heights in the crest construction, because of the deepened pits under the crest construction also at small water heights (0,1 m) a reasonable discharge capacity is obtained. The results show that for a pipe diameter of 250 mm, a pipe has to be placed every 10 to 15 m. In our opinion this distance is too small, the dike will be affected at small interval distances. Based on Table 3-1 a pipe diameter of 400 mm with a distance of 30 to 35 meters is chosen (exact distance is dependent on detailing and realization method). This is worked out in a preliminary design for both the pipe towards the landward side and towards the seaward side (see chapter 7). The choice for pipe diameter and distance is very dependent on the local situation and should be looked into for every location. Especially for the pipe towards the seaward side, because of the small length, it might be interesting to regard the possibility of applying a 500 mm pipe every 50 to 60 meters.

3.2.3 Proposed discharge alternatives

For the discharge of overtopping water from the crest construction, two alternatives are worked out in the continuation of this study:
- Landward discharges pipes (synthetic PP-pipes), diameter 400 mm every 30 m, pipe length is 40 to 50 meters.
- Seaward discharges pipes (synthetic PP-pipes), diameter 400 mm every 30 m, pipe length is 10 to 15 meters.
Some variations on the alternatives above can be considered, for different reasons these
are not included in the continuation of this study. However, depending on the local situation
and the requirements/ priorities, it might be useful to consider one of the variations below:
- For the landward discharge pipes, in the alternative above the pipes are buried into the
inner slope (under the present clay layer, which will first be excavated and then put back
on top of the discharge pipe). It would be easier to lay the discharge pipes on the inner
slope, however this creates a barrier on the inner slope, which is very undesirable from
a landscape point of view but also creates a barrier for maintenance activities and
grazing sheep on the inner slope.
- For the seaward discharge pipes, instead of a round discharge pipe, also the realization
of a square concrete construction (sort of culvert) can be considered. It is expected that
this is a more expensive solution though.
- Instead of discharge pipes with a diameter of 400 mm every 30 meters, another
possibility is to choose for bigger pipes with longer interval distances. A possibility is a
diameter of 500 mm every 50 to 60 meters. This might be preferable in a situation
where it is desirable to concentrate the impact on the dike on as few locations as
possible.
- Instead of seaward discharge pipes, it is also possible to create gaps in the crest
construction through which the water can easily be discharged seaward after an
overlapping wave. The gaps need a minimum width of about 1 m to ensure very rapid
discharge and negligible entrance and friction losses. A possible risk is the entrance of
more overlapping water into the crest construction through the gap during wave run-up.
On the other hand, the discharge capacity is much higher for this option than for
discharge pipes so the extra overlapping water can probably easily be discharged again.
The gap can be dimensioned in such as way that entrance of water is more difficult than
discharge of water (narrow at the seaside and wide at the side of the crest
construction). Because the crest construction is packed in a ground package (the
existing dike), this requires a prefabricated concrete construction to guide the water from the
crest construction towards the seaward slope. Usually this upper part of the seaward
slope consists of a grass revetment, this will probably not have enough erosion
resistance to cope with the foreseen discharge from the gap, requiring extra protection
measures on the outer slope. If relatively large discharge capacities (average > 20
l/s/m) are required for the Crest Drainage Dike, this alternative could be considered. It
requires some further research though, especially regarding the extra amount of
entering water during overlapping and the exact shape and form of the concrete gap
construction.

3.3 Crest construction

3.3.1 Introduction

One of the conditions for the crest construction is that is must be applicable in a 3 m meter
wide crest. Next to the crest construction some space is required to ‘pack’ the construction
in the natural dike material (based on the principle that the construction should not be
seen to minimize the impact on the natural landscape). The chosen inner width of the crest
construction is 2.0 m, however if more space is available it is recommendable to increase
the width as much as possible (to increase the buffer capacity of the crest construction and to increase friction which decreases the flow velocities in the construction).

The height of the crest construction is determined by the possible joint use of the crest construction for recreational purpose. The crest construction can be used as a cycle path or footpath; this leads to some requirements for the Crest Drainage Dike. First of all the height of the crest construction should be small enough to ensure a free sight for strollers and cyclists to both the landward as the seaward side. For this purpose, we chose an inner height of the crest construction of 0.8 m. From an analysis of the required buffer capacity and the flow in the crest construction, it will turn out whether the chosen inner dimensions of the crest construction are feasible.

Furthermore, following from chapter 2, the following assumptions and information are used in this paragraph:
- The critical requirement for the average overtopping rate the Crest Drainage Dike must be able to resist is 15 l/s/m;
- Table 2-7 gives information on the probability frequency of instantaneous overtopping rates corresponding to the above-mentioned critical requirement;
- A certain quantity of wave overtopping is allowed to flow over the inner slope in extreme conditions, this must be looked into in relation to the erosion resistance of the grass slope;
- The length spreading factor for instantaneous wave overtopping rates is 2;
- The reduction factor for the buffer capacity of the crest construction is 0.8;
- The design overtopping flow speed for the crest construction is 6.0 m/s with a flow layer thickness of 0.3 m;
- The design overtopping flow speed for the inner slope is 7.2 m/s, period of occurrence is dependent on the buffer capacity and remaining period of overtopping towards the inner slope.

### 3.3.2 Buffer capacity

Besides the average discharge capacity of the Crest Drainage Dike, as was described in paragraph 3.2, the Crest Drainage Dike offers a buffer capacity to hold the water in the crest construction in extreme overtopping events. The capacity is determined by the inner dimensions of the crest construction:

\[
\text{Capacity} = 0.8 \text{ m (height) } * 2.0 \text{ m (width) } * 0.8 \text{ (reduction factor)} = 1.3 \text{ m}^3/\text{m}^4
\]

Because of the length spreading effect, the capacity to trap instantaneous overtopping is determined by multiplying the above-calculated capacity/m\(^4\) with a factor 2. This leads to a buffer capacity of 2.6 m\(^3\)/m\(^4\) (=2600 l/m\(^4\)). Based on Table 2-7, it can be estimated that this available buffer capacity is exceeded by the instantaneous overtopping rates of about 25 % of the overtopping waves. During the regarded peak of the storm period, not all waves will cause overtopping. However, in a conservative approach it was assumed that this is the case. That means that 25 % of the overtopping waves can be translated to 25 % of the time during the peak of the storm period (duration 3 hours).
The above implies that during about 25 % of a 3-hour period, the instantaneous overtopping volume is higher than the buffer capacity and thus will reach the inner slope. It is assumed that the overtopping velocity and layer thickness is not decreased by crest friction, this has led to the calculated design flow speed for the inner slope of 7.2 m/s, occurring during about 25 % of the regarded storm period.

Grass covers can resist flow velocities up 2.0 m/s without any problem. For higher flow velocities, it depends on the flow duration if the erosion resistance is sufficient. CIRIA [Hewlett, 1985] published a design graph for maximum permissible flow duration. The graph is based on the following equations:

Plain grass, good cover: \( u_{\text{max}} = 1.934 \times 10^4 / T + 2.33 \)
Plain grass, average cover: \( u_{\text{max}} = 1.589 \times 10^4 / T + 1.707 \)
Plain grass, poor cover: \( u_{\text{max}} = 1.062 \times 10^4 / T + 1.205 \)

In Table 3-2, \( u_{\text{max}} \) is shown for \( T=2700 \) s (representing the minimum condition 25 % of storm period) and for \( T=3600 \) s (1 hour) to give an indication of margins. The results show that for a poor grass cover, erosion resistance is not sufficient under these conditions. For an average cover, erosion resistance is sufficient in the minimum critical condition \( (T=2700 \) s), however little margin is available. Plain grass with a good cover offers sufficient erosion resistance, also when the overtopping duration is a bit longer \( (T=3600 \) s).

<table>
<thead>
<tr>
<th></th>
<th>( u_{\text{max}} ) for ( T=2700 ) s</th>
<th>( u_{\text{max}} ) for ( T=3600 ) s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain grass, good cover</td>
<td>9.5 m/s</td>
<td>7.7 m/s</td>
</tr>
<tr>
<td>Plain grass, average cover</td>
<td>7.6 m/s</td>
<td>5.0 m/s</td>
</tr>
<tr>
<td>Plain grass, poor cover</td>
<td>5.1 m/s</td>
<td>4.1 m/s</td>
</tr>
</tbody>
</table>

Concluding, for plain grass with an average to good cover, the inner slope is sufficiently erosion resistant for the remaining overtopping in extreme conditions as calculated for the Hondsbossche Zeewering Crest Drainage Dike.

### 3.3.3 Width and shape of the crest construction

As was mentioned before, an important aspect for the Crest Drainage Dike is whether the flow layer will not exit the crest construction when it hits the landward sideward of the crest construction for the relatively small width of 2.0 m. The design flow velocity is 6.0 m/s with a layer thickness of 0.3 m. The entering flow layer causes a load on the crest construction, this aspect is worked out in chapter 5. The layer thickness of 0.3 m is significantly lower than the construction height of 0.8 m, therefore it is not probable that the flow layer will completely overtop the sideward at the end. However, some overtopping over the sideward might occur. Little is known at present about overtopping (for a flow layer, comparable to wave run-up) over a vertical structure. Therefore, it is recommended to execute physical model tests to check the overtopping rate over the sideward in a crest construction like this. The flow into the crest construction at the other sideward can be included in the model tests. In this study, it’s been assumed that the overtopping water will fully flow into the crest.
construction (sheet flow) and will stay in the construction unless this is already filled with water. This assumption should be checked with the recommended model tests.

If it turns out from the model tests that the inflow and/or outflow of the crest construction do not agree with the assumptions as made, measures can be taken to ensure the overtopping flow will be trapped in the crest construction anyway. The shape of the crest construction can be adjusted to guide the flow in the right direction, see Figure 3-2 for an example. When adjusting the shape, the multifunctional use of the crest construction should be kept in mind. Shapes as presented in the figure below are made in prefab elements; costs might be a bit higher than for the regular crest construction. However, for relatively long dike lengths different shapes can be cost-effective.

**Figure 3-2: Illustration of possible different shape concrete crest construction to prevent overtopping landward sidewall**

![Diagram of possible shapes for concrete crest construction](image)

Regarding the width of the crest construction, it is recommended to chose a width as large as possible in relation to the available space. The width of 2 m as chosen in this study is based on the crest width of 3 m; if the crest width is larger it is recommendable to also increase the width of the crest construction. This increases friction and thus decreases possible outflow by overtopping of the landward sidewall of the crest construction, it increases the buffer capacity to trap overtopping water in extreme conditions and it creates more space for recreational purposes.

### 3.3.4 Proposed crest construction alternatives

For the crest construction, three alternatives are worked out in the continuation of this study:
- Prefab concrete crest construction with prefab pits connecting to the discharge pipes;
- In situ concrete crest construction with prefab pits connecting to the discharge pipes;
- Steel sheet piles with an in situ concrete floor.

The building costs for placing sheet piles in an existing dike are expected to be significantly higher than for the other alternatives. Therefore this alternative is worked out only roughly in dimensions and a cost estimation.

### 3.4 Storage capacity hinterland

The hinterland of the Crest Drainage Dike must have a certain storage capacity to handle the overtopping water, at least if it is discharged in the landward direction. The direct hinterland can offer the required storage capacity, or the discharge pipes of the dike can
end in a discharge channel that leads the water to another place where it can be stored (or maybe back to the sea).

Storage in direct hinterland
If the direct hinterland stores the overtopping water, a secondary dike or higher grounds is needed to keep the rest of the hinterland dry. Furthermore, a certain surface area in between the primary sea dike and the secondary dike is required to offer enough storage area. If it is assumed that the average overtopping rate has the duration of the entire storm period, the total quantity of overtopping water is equal to 15 l/s/m * 3 (hours) * 3600 s = 162,000 l/m = 162 m³/m². This means a zone width of for example 200 meters will lead to a water layer of 0.80 m in the hinterland. The storage possibilities in the hinterland are dependent of the local situation.

Discharge channel
If the discharge pipes end in a discharge channel, the total discharge capacity of the channel must at least be:

\[ Q_{\text{total}} = \frac{L_{\text{dyke}}}{\text{dyke}} \times q_{\text{overtopping, average}} \]

For example, if \( L_{\text{dyke}} = 2 \text{ km} \) and \( q_{\text{overtopping}} = 15 \text{ l/s/m} \) then \( Q_{\text{total}} = 30,000 \text{ l/s} = 30 \text{ m}^3/\text{s} \). With the Chézy equation for open water flow, see below, it was calculated that with a width of 15 m for the channel, \( i = 0.001 \) and \( C = 50 \text{ m}^{1/2}/\text{s} \), the equilibrium water depth is still 2.5 m. The discharge channel needs minimum dimensions of 15 m (width) * 2.5 m (depth), this comes down to a relatively big channel.

If the discharge channel goes in two directions towards two different storage areas or towards two different points where the water can be brought back to the sea, \( Q_{\text{total}} \) can be halved. It was calculated that with a width of 10 m for the channel, \( i = 0.001 \) and \( C = 50 \text{ m}^{1/2}/\text{s} \), the equilibrium water depth is 2.1 m. The discharge channel then needs minimum dimensions of 10 m (width) * 2.1 m (depth).

The applied Chézy equation for open water flow is:

\[ h_e = \frac{1}{3}(Q^2 / (C^2 \cdot i)) \]

where:
\( h_e \) = equilibrium depth [m]
\( Q \) = discharge capacity [m²/s]
\( C \) = Chézy coefficient [m¹/²/s]
\( i \) = hydrostatic fall over the channel [-]
4 STABILITY OF THE DIKE

4.1 Failure mechanisms

Due to wave overtopping, different types of failure mechanisms can be of importance for the inner slope. First of all, the waves overtopping the dike cause a hydraulic load on the inner slope, which can cause erosion of the top layer. This aspect was already described in paragraph 3.3 in relation to the crest’s buffer capacity. Furthermore the overtopping water will infiltrate into the dike, this can lead to instability of the inner slope in different ways.

The Crest Drainage Dike will consist of a combination of a basic soil construction with a concrete reservoir in the dike crest. In order to examine the stability of the concrete element under normative conditions, geohydrological and geotechnical calculations have been performed. The following stability aspects are looked into:
- Piping and heave, geohydrological calculations;
- Vertical and horizontal stability crest construction, geotechnical calculations;
- Macro-stability.
These aspects have been regarded for sand dikes as well as clay dikes.

4.2 Approach and assumptions

The dike dimensions are as follows; a crest width of 3.0 m and slopes of 1v:3h. The inside dimensions of the concrete reservoir in the dike crest are assumed to be 0.8 m by 2.0 m (height to width). With a concrete thickness of 0.25 m (first estimate), the outside dimensions are 0.95 by 2.5 m. Since the construction can be considered as infinite, a standardized length of 1.0 m is taken into account in the calculations. In Figure 4-1 the construction is schematised. The applied soil parameters are based on normalised Dutch soils (NEN 6740) and are listed in Table 4-1.

**Figure 4-1: Global dimensions taken into account in study of stability aspects Crest Drainage Dike**

**Table 4-1: Soil parameters for geohydrological and geotechnical calculations**

<table>
<thead>
<tr>
<th>Material</th>
<th>( \gamma_{sat}/\gamma_{dry} ) [kN/m³]</th>
<th>( \phi' ) [°]</th>
<th>( c' ) [kPa]</th>
<th>( f_{wcr} ) [kN/m²]</th>
<th>( k ) [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dike sand (medium fine grained)</td>
<td>21/19</td>
<td>30</td>
<td>0</td>
<td>n/a</td>
<td>( 1.0 \times 10^{6} )</td>
</tr>
<tr>
<td>Dike clay</td>
<td>19/17</td>
<td>20</td>
<td>5</td>
<td>5</td>
<td>( 8.0 \times 10^{5} )</td>
</tr>
</tbody>
</table>
### 4.3 Piping and heave

It is not expected that piping and heave will cause an extra threat to the stability of the dike because of the placement of the crest construction and the discharge pipes in the dike. This expectation must be verified though. Because it is difficult to predict the exact saturation height in the outer slope when wave overtopping occurs (because it occurs only during a short time period, it does not represent a permanent situation), the calculations are first executed for the very conservative assumption that the seaward slope will be completely saturated by the overtopping water layer. If the results of these calculations show that no piping is expected for this situation, the stability regarding piping and heave is proven.

The four situations that have been considered in the geohydrological calculations are listed in Table 4-2. In Figure 4-2 and Figure 4-3 the situations are schematised.

#### Table 4-2: Geohydrological schematisations

<table>
<thead>
<tr>
<th>Situation</th>
<th>Dike material</th>
<th>Subsoil material</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. permeable sand dike on less permeable sandy subsoil</td>
<td>dike sand</td>
<td>very fine grained or silty sand</td>
</tr>
<tr>
<td>2. permeable sand dike on more permeable sandy subsoil</td>
<td>dike sand</td>
<td>very coarse grained, non silty sand</td>
</tr>
<tr>
<td>3. non-permeable clay dike on permeable sandy subsoil</td>
<td>dike clay</td>
<td>very fine grained or silty sand</td>
</tr>
<tr>
<td>4. non-permeable clay dike on high permeable sandy subsoil</td>
<td>dike clay</td>
<td>very coarse grained, non silty sand</td>
</tr>
</tbody>
</table>

**Figure 4-2: Sandy dike on sandy subsoil**

![Sandy dike on sandy subsoil](image)

**Figure 4-3: Clay dike on sandy subsoil**

![Clay dike on sandy subsoil](image)
Landward discharge pipes
First, the alternative with landward discharge pipes was studied. Figure 4-4 gives an illustration of possible water pressures along the discharge pipes. Because the water can flow through the dikes past the discharge pipes, a hydrostatically fall and thus piping can develop over the dike.

Figure 4-4: Water pressures, situation with landward discharge pipes

Conclusion: Piping is possible

In Appendix 3, figures A3-1 to A3-4, the calculation results are graphically presented. The results from these graphics are described below:

1. Permeable sand dike on less permeable sandy subsoil:
   - water pressure gradient over concrete construction $i < 0.5$;
   - micro instability of inner slope may occur.

2. Permeable sand dike on more permeable sandy subsoil:
   - water pressure gradient over concrete construction $i > 0.5$;
   - piping will occur.

3. Non-permeable clay dike on permeable sandy subsoil:
   - clay layer will form a seal round the concrete construction;
   - water pressure gradient over dike body $i > 0.5$;
-piping will not occur.

4. Non-permeable clay dike on high permeable sandy subsoil:
- clay layer will form a seal round the concrete construction;
- water pressure gradient over dike body \( i > 0.5 \);
- piping will not occur.

From the geohydrological calculations it is demonstrated that the maximum hydrostatic fall over the concrete construction exceeds the allowable gradient of water pressure in all considered situations, except when the permeability of the dike is higher than the permeability of the subsoil. In this case the phreatic line will be high, resulting in a small water pressure gradient over the construction. However, micro instability of the inner slope of the dike might occur in that situation. In the other situations piping might occur around the pipelines.

Whether or not piping will occur, is dependant on the hydrostatic fall between in- and outside of the dike (\( \Delta H \)). As was mentioned, the geohydrological calculations as described above were based on the conservative assumption that the seaward side of the dike is completely saturated because of the overtopping water. Based on expert judgement, it is still not expected that piping and heave will cause a danger to the stability of the Crest Drainage Dike, despite of the results of the calculations. However, because the requirements regarding piping were not met under the adopted conservative assumption, a verification of possible piping danger must be executed locally. This requires an analysis of the exact local water pressure development in the outer slope caused by wave overtopping.

Furthermore, attention point for this alternative is that the discharge pipe must be realized under the present clay layer. At the locations where the clay layer is excavated to place the discharge pipe, special attention must be paid to the settlement and condition of the clay layer after replacement of the clay layer to guarantee the required erosion resistance of the grass slope.

*Seaward discharge pipes*
Secondly, the alternative with landward discharge pipes is examined. In this case the potential hydrostatic fall over the construction is zero (see Figure 4-5). Instead of piping or heave, saturation of de dike body might occur, resulting in micro instability of the inner slope of the dike. Installing of a seepage screen is therefore not required. It is advised to prevent the water to enter the dike by realizing a watertight connection between the outer slope revetment and the discharge pipe (see also paragraph 5.3, joints and transitions).

*Figure 4-5: Water pressures, situation with seaward discharge pipes*
Conclusion: Piping is not possible, dike saturation and a high phreatic line will occur, resulting in possibility of micro instability for inner slope.

4.4 Vertical and horizontal stability crest construction

Foundation calculations are performed in order to specify the bearing capacity of the subsoil and the horizontal stability of the concrete construction. Both founding of the concrete construction on a sand dike (drained) as well as on a 1.0 m thick clay seal (undrained) have been considered. All calculations relate to a standardized construction length of 1.0 m$^2$.

Sand dike (drained)

In the calculation of the bearing capacity of the dike body, the original pore pressure ($\sigma_{	ext{vozd}}$) at the foundation level is assumed to be zero. This is a conservative assumption because, depending on the amount of condensation of the layer, $\sigma_{	ext{vozd}}$ is always > 0 kPa. However, the quantity of this parameter is difficult to estimate. The weight load of a completely filled water reservoir exerted on the foundation is 37,4 kN/m$^2$. The calculation parameters used are listed in Table 4-3.

<table>
<thead>
<tr>
<th>Table 4-3: Calculation parameters drained condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{\text{vozd}}$</td>
</tr>
<tr>
<td>$B_{\text{st}}$</td>
</tr>
<tr>
<td>$L_{\text{st}}$</td>
</tr>
<tr>
<td>$A_{\text{st}}$</td>
</tr>
<tr>
<td>$F_{\text{sch,dr}}$</td>
</tr>
<tr>
<td>$F_{\text{sc,dr}}$</td>
</tr>
<tr>
<td>$s_{q}$</td>
</tr>
<tr>
<td>$s_{a}$</td>
</tr>
<tr>
<td>$s_{c}$</td>
</tr>
<tr>
<td>$N_{q}$</td>
</tr>
<tr>
<td>$N_{a}$</td>
</tr>
<tr>
<td>$N_{c}$</td>
</tr>
<tr>
<td>$n$</td>
</tr>
<tr>
<td>$\lambda_{q}$</td>
</tr>
<tr>
<td>$\lambda_{c}$</td>
</tr>
<tr>
<td>$\alpha$</td>
</tr>
<tr>
<td>$F_{\text{vozd}}$</td>
</tr>
<tr>
<td>$F_{\text{vozd}}$</td>
</tr>
</tbody>
</table>

Since the calculated strength of the soil ($F_{\text{v,dr}}$) of 50.0 kN > the weight load ($F_{\text{sc,dr}}$) of 37.4 kN, the vertical stability of the construction is guaranteed.

In critical conditions the outward soil consists of saturated material and the inward soil of unsaturated. This will result in a horizontal component of the calculated load of 8.54 kN.
Since the calculated soil resistance of 13.61 kN exceeds this load, the horizontal stability of the construction is considered to be sufficient (under the condition that no erosion of the adjacent soil bodies will occur).

**Clay dike (undrained)**

In the calculation of the bearing capacity of the dike body, the original pore pressure ($q_{v,odd}$) at the foundation level is assumed to be zero. This is a conservative assumption because, depending on the amount of condensation of the layer, $q_{v,odd}$ is always > 0 kPa. However, the quantity of this parameter is difficult to estimate. The weight load of a completely filled water reservoir exerted on the foundation is 37.4 kN/m². The calculation parameters used are listed in Table 4-4.

<table>
<thead>
<tr>
<th>$q_{v,odd}$</th>
<th>0 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_{ef}$</td>
<td>2.27 m²</td>
</tr>
<tr>
<td>$L_{ef}$</td>
<td>1 m²</td>
</tr>
<tr>
<td>$A_{ref}$</td>
<td>2.27 m²</td>
</tr>
<tr>
<td>$F_{v,odd}$</td>
<td>8.54 kN</td>
</tr>
<tr>
<td>$F_{v,odd}$</td>
<td>23.4 kN</td>
</tr>
<tr>
<td>$B_c$</td>
<td>1.45</td>
</tr>
<tr>
<td>$B_s$</td>
<td>1.1</td>
</tr>
<tr>
<td>$N_s$</td>
<td>0.57</td>
</tr>
<tr>
<td>$N_c$</td>
<td>7.50</td>
</tr>
<tr>
<td>$A_s$</td>
<td>0.37</td>
</tr>
<tr>
<td>$A_c$</td>
<td>0.50</td>
</tr>
<tr>
<td>$k_c$</td>
<td>0.98</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>0.01</td>
</tr>
<tr>
<td>$\beta$</td>
<td>0.30 rad</td>
</tr>
<tr>
<td>$\sigma_{v,odd}$</td>
<td>21.0 kPa</td>
</tr>
<tr>
<td>$F_{v,odd}$</td>
<td>47.0 kN</td>
</tr>
</tbody>
</table>

Since the calculated strength of the soil ($F_{v,odd}$) of 47.0 kN > the weight load ($F_{v,odd}$) of 37.4 kN, the vertical stability of the construction is guaranteed.

In critical conditions the outward soil consists of saturated material and the inward soil of unsaturated. This will result in a horizontal component of the calculated load of 8.54 kN. Since the calculated soil resistance of 11.35 kN exceeds this force, the horizontal stability of the construction is considered to be sufficient (under the condition that no erosion of the adjacent soil bodies will occur).

### 4.5 Macro-stability

The macro-stability of the Crest Drainage Dike is guaranteed in all situations, because the weight load of the (water filled) concrete construction is less than the weight of the soil package that it replaces.
5 CONSTRUCTIONAL ASPECTS CREST DRAINAGE DIKE

5.1 Introduction

In this chapter the constructional aspects of the Crest Drainage Dike are worked out. Based on the global dimensions as determined in chapter 3, first of all the design of the concrete crest construction is worked out in more detail for different alternatives in paragraph 5.2. In paragraph 5.3 the attention points regarding the joints and transitions that should be worked out in a detailed design are described. Finally, in paragraph 5.4 the maintenance aspects of the Crest Drainage Dike are described.

5.2 Detailing of concrete crest construction

5.2.1 Alternatives

The construction of the alternatives for the crest construction, as described shortly below, are worked out in more detail in this paragraph.

1. Prefab concrete crest elements
Due to crane capacity the length of the elements is limited to about 4 to 6 meters. The elements are placed without couplers next to each other. Pre-stressing is an option if a slender construction is wanted. The elements are easily interchangeable and replaceable, which makes maintenance easy. Every 30 meters, a prefab pit construction is placed in between the elements. The voids in between the prefab pit and crest elements are poured with in situ concrete. The pit is connected to the PP discharge pipes.

2. In situ concrete crest elements
Sections of up to 30 m length can be made in situ. Temperature effects are normative for the maximum length. This construction method is probably the cheapest, though at longer dike lengths the prefab elements can become competitive (see chapter 0). The elements are not interchangeable or easily replaceable. Every 30 meters (thus in between every "normal" element), a prefab pit construction is placed. The voids in between the prefab pit and in situ crest elements are poured with in situ concrete. The pit is connected to the PP discharge pipes.

3. Sheet piles with concrete floor
Between two sheet pilings, a floor is made of in situ concrete. Section of 30 m length can be made. A disadvantage is the pile frame on top of the dike during erection. Furthermore, the costs of this alternative are expected to be relatively high.

5.2.2 Detailing concrete construction

Alternatives 1 and 2: concrete crest elements
For the calculations of the concrete construction (same for alternatives 1a and 1b) the following assumptions were adopted:
• Design overtopping flow speed on crest construction is 6.0 m/s with a flow layer thickness of 0.3 m;
• Global inner dimensions of the crest construction are 0.8 m (height) * 2.0 m (width);

To create more friction and stability for the crest construction, the shape of the concrete elements is adjusted: the floor is extended a bit outside the crest walls as can be seen in Figure 5-1. For the presented cross-profile calculations of the horizontal load on the crest construction are executed. Because the static force in the crest construction is low (maximum hydraulic head 0.8 m), the wall and floor do not need to be very thick. For practical reasons a thickness of 0.20 m was chosen for both walls and floors. Reinforcement on in- and outside of □ 10 – 150 meets the requirements for strength and durability.

**Figure 5-1: Illustration of concrete crest construction shape**

The dynamic load on the concrete crest construction can be calculated with the following equation:

\[ F = \frac{\gamma_w Q}{g} (\Delta \nu) \]

where:
- \( F \) = dynamic load [kN/m²]
- \( \gamma_w \) = relative weight water [kN/m²]
- \( Q \) = flow into crest construction (flow velocity * layer thickness) [m²/s]
- \( g \) = gravitational acceleration [m/s²]
- \( \Delta \nu \) = flow velocity difference [m/s]

**Figure 5-2: Illustration of dynamic load and counterforces**

With \( \gamma_w = 10 \text{ kN/m}^3 \), \( Q = 6.0 \text{ m/s} * 0.3 \text{ m} = 1.8 \text{ m}^2/\text{s} \), \( g = 9.8 \text{ m/s}^2 \) and \( \Delta \nu = 6.0 \text{ m/s} \) it can be calculated that the load \( F = 11.0 \text{ kN/m}^2 \). Two mechanisms can withstand this horizontal load (see Figure 5-2):

- Friction forces of the floor with sandy underground
The friction forces of the floor are determined by the weight of the crest construction. The crest construction’s volume is 0.88 m³/m. With a relative weight of 25 kN/m³, a weight of 22 kN/m² is calculated. This gives a horizontal force of 0.25 (friction coefficient) * 22 kN/m² = 5.5 kN/m².

- Friction of the ground package next to the crest construction
The friction forces of the sand package next to the crest construction are also determined by the weight of the sand package. The sand package’s volume is 2.0 m³/m. With a relative weight of 16 kN/m³, a weight of 32 kN/m² is calculated. This gives a horizontal force of 0.25 (friction coefficient) * 32 kN/m² = 8 kN/m².

Together, the horizontal friction forces are 5.5 kN/m² + 8 kN/m² = 13.5 kN/m² > 11.0 kN/m² (dynamic load). It can be concluded that the horizontal stability of the crest construction under the dynamic load of the overtopping water is secured for the proposed construction dimensions.

Alternative 3: sheet piles with concrete floor
With M-sheet calculations, the required length of the sheet piles were determined. The minimum length is 4.0 m. The horizontal load caused by overtopping water can easily be handled by this construction because of the large sand package that is activated by the sheet piles. The sheet piles can be made of steel or wood. For the concrete in situ floor a thickness of 0.2 m was chosen for practical reasons. The floor has to be connected to the sheet piles, with bolts (steel sheet piles) or screws (wooden sheet piles). Durability of the sheet pile materials requires attention.

The impact of alternative 1 (a. prefab or b. in situ) on the existing dike is much less than for the sheet pile alternative (during realization and after that). Minimizing of the impact on the existing dike is preferable. Furthermore, it is expected that the costs for alternative 2 will be higher than for alternative 1. This will be worked out in chapter 8, the cost estimation.

Concrete durability
The durability of the concrete is not considered to be critical for the working life of this construction, concrete in the Netherlands is usually prepared for a working life of around 80 years, or more if wanted. For the concrete it should be kept in mind that special demands should be set because of the salty environment and water in the construction. The discharge pipes will be made of durable synthetic material.

5.3 Joints and transitions

A construction is only as strong as the weakest section. Experience shows that erosion or damage often starts at joints and transitions. Therefore, important aspects of revetment constructions, which require special attention, are the joints and the transitions; joints onto the same material and onto other revetment materials, and transitions onto other structures or revetment parts. The joints or transitions may influence loads in terms of forces due to differences in stiffness or settlement, migration of subsoil from one part to another (erosion), or strong pressure gradients due to a concentrated groundwater flow. Proper design and execution are essential in order to obtain satisfactory joints and transitions.
As a general principle one can state that the transition should be of a strength equal to or greater than the adjoining systems. Very often it needs reinforcement, for example in one of the following ways:

- Increase the thickness of the cover layer at the transition,
- Grout riprap or block cover layers with bitumen, and
- Use concrete edge strips or boards to prevent damage progressing along the structure.

Top edge and flank protection are needed to limit the vulnerability of the revetment to erosion continuing around its ends. Care should be taken that the discontinuity between the protected and unprotected areas is as small as possible (use a transition roughness) so as to prevent undermining. For transition from a hard protection into a grass mat, open cell blocks or open block mats (possibly vegetated) can be used. The flank protection between the protected and unprotected areas usually needs a thickened or grouted cover layer, or a concrete edge strip with some flexible transition i.e. riprap.

For the Crest Drainage Dike, the following transitions and joints require further attention and elaboration in a detailed design:

**Transition zone between upper revetment outer slope and concrete crest construction**
Vulnerable zone for erosion, this zone should be made non accessible for recreational users of the concrete crest construction. Fences and/or signs should be placed to prevent people from entering this zone, because this can cause significant damage to the slope protection and thereby endanger the stability of the zone. This is specifically true for grass slopes in this zone. Furthermore, if necessary, this transition zone can be included in the guidance of flow into the crest construction. The shape of the transition zone can be adjusted so the overtopping water will flow into the crest construction more easily.

**Transition zone between concrete crest construction and the inner slope protection**
Just like the transition zone between the outer slope protection and the concrete crest construction, also the transition between the crest construction and the inner slope protection is a zone vulnerable for erosion. Here also, the zone should be made non accessible for recreational users of the concrete crest construction by the use of signs and/or fences. Furthermore, on the inner slope of sea dikes often sheep are grazing in the Netherlands. Measures should be taken to prevent the sheep from falling into the crest construction; fences could be used for this purpose.

Erosion of the transition zone between crest construction and inner slope can be very dangerous for the horizontal stability of the crest construction. The weight of the ground package in this zone is required as a counterforce to handle the dynamic load on the crest construction when the overtopping water enters the concrete construction.

**The joints between the prefab pit construction and the PP-discharge pipes**
In the prefab pit construction a hole is present in which the PP-discharge pipe can be connected. A watertight and flexible connection must be realized between the PP-pipes and the concrete pit, this can be obtained for example with a rubber ring. Leakage at this point can be a great danger for the dike’s stability.
Joints between the concrete elements (prefab or in situ)
A flexible chain of elements should be realized, so that small settlements can be coped with without serious damage to the construction. Furthermore, one should be aware of leakage in these joints because this could form a threat to the stability of the dike.

Transition zone between the discharge pipe and outflow construction in the hinterland (in case of landward discharge pipes)
The landward-directed discharge pipes can be connected to a discharge channel or to some kind of outflow construction, from where the water enters the hinterland. In both cases, a protected outflow construction is needed to cope with high flow velocities coming from the pipes. It might be necessary to realize an energy dissipation construction such as a spillway to slow down the water flowing from the discharge pipes.

Joints between the discharge pipe and the outer slope protection (in case of seaward discharge pipes)
Between the seaward-directed discharge pipes and the outer slope protection, at the point where the pipes leave the dike body, a flexible and watertight connection must be realized. It is assumed that the pipes leave the dike above still water level and above the wave attack zone. In this area the outer slope protection might consist of grass or a hard protection. In case of a grass revetment, the erosion resistance of the outer slope under the load of exiting water should be looked into. In case of a hard revetment, a hole must be made in the revetment so the pipes can exit.

5.4 Maintenance aspects

Maintenance activities in normal conditions
The maintenance activities for the Crest Drainage Dike consist of keeping the concrete construction and the pipes clean to prevent obstruction of the flow. Especially the foreseen metal grid, pit construction and the discharge pipes are attention points that should be cleaned before the storm season and monitored during storm season. The concrete construction can be cleaned with normal road cleaning vehicles, provided these can enter the crest construction at some location. The discharge pipes should be flushed at least once before every storm season, special flushing vehicles are available to easily flush the pipes (also provided these can enter the crest construction). The metal grid is to be inspected during the storm season to make sure it is not clogged by trash or other things.

Besides cleaning, possible damage to the concrete construction and the inner slope should be monitored and repaired if necessary. Small damage can be repaired, if the damage to an element is of a larger scale, the element can also be replaced entirely. This is easier for the smaller prefab elements than for the in situ elements.

For easy cleaning, monitoring and repairing damage accessibility to the construction is of the highest importance. The accessibility is well preserved in this concept, the concrete U-profile functions as a low-lying walking and cycling path in the dike crest. The construction can easily be monitored visually and measures can be taken if necessary. For good accessibility, entrance points into the crest construction from the hinterland must be
created. These entrance points are also required for possible recreational users of the crest construction. The entrance points can consist of a slope or some steps into the construction. How many of these entrance points are required, the approach over the inner slope and the best shape and dimensions of the entrance are aspects that are very dependent of the local situation. These aspects should be worked out in a detailed design, depending on the local criteria and ambitions.

Maintenance activities in storm conditions
In normal conditions, the crest construction can be used as footpath and/or cycle path. However, in storm conditions entrance of the crest construction must be made impossible. Possible measures are the placement of signs to warn people for the danger of entering the crest construction in storm conditions and/or physically closing off the entrances when extreme storm conditions are expected. Furthermore, after an extreme storm condition when overtopping and discharge towards the hinterland has taken place it might be necessary to pump dry the hinterland storage area.

According to Dutch standards [VTV, 2004], the dike crest is not accessible for inspection or repair activities during a storm period when the average overtopping rate is higher than 10 l/s/m. Therefore, for the Crest Drainage Dike it must be shown that not being able to access the dike during storm does not endanger the safety and stability of the dike. The metal grids at the discharge pipes might get clogged by trash or other objects. However, we expect that the turbulent overtopping water entering the crest construction will actually flush the metal grids clean during every wave-overlapping event, minimizing the risk of clogging. This should be worked out further however.

Maintenance inner grass slope
As was calculated before, a reasonable to good grass cover is required on the inner slope to cope with high flow velocities for short time periods. To provide a good grass cover, good maintenance of the inner slope is important; e.g. grazing by sheep and prevention of bare spots by cattle tracks, near obstructions, and so forth. Due to meteorological variations, the condition of the grass cover will be varying as well.

Vandalism
The concrete construction and buried pipes are not vulnerable for vandalism. The grids over the deeper lying pits are vulnerable for vandalism though and these should be secured firmly, furthermore the presence of the grids must be checked visually before every storm season.
6 EVALUATION CREST DRAINAGE DIKE

6.1 Introduction

In the SOTA study ComCoast [CUR/RWS, 2005], assessment criteria were set up for the evaluation of different alternatives to strengthen the inner slope. These criteria can be adopted for the evaluation of the Crest Drainage Dike also, though for the innovative concepts some extra criteria could be added. The evaluation of the Crest Drainage Dike is based on different criteria, such as costs, technical feasibility, realization and maintenance aspects but also spatial planning issues such as recreation and LNC-values (landscape, nature and cultural history). The first mentioned aspects were already worked out in different ways in the previous chapters. Possible spatial planning issues are described in this chapter, see paragraph 6.2 After that, in paragraph 6.3 the assessment criteria as described in SOTA, complemented with some extra criteria for innovative concepts, are scored for the Crest Drainage Dike.

6.2 Spatial planning issues

For the Crest Drainage Dike, two types of spatial planning issues are of main interest:

1. Damage to present LNC-values or other functions of the dike;
2. Enhancement of recreational function of the dike.

Damage to present LNC-values or other functions of the dike
By applying concrete prefab elements for the crest construction, inconvenience during the construction of this concept can be minimized. The concrete construction does not allow for any nature development at the crest. However, the effects on LNC-values are limited to the crest because at the inner slope a grass revetment can be maintained which provides good opportunities for nature development. Only locally during the digging of trenches and burial of the pipes, the present nature and landscape will be disturbed at the inner slope.

Regarding the aspect of landscaping and scenic quality, the concrete construction is ‘packed’ in the natural crest; the general view of the landscape therefore does not change. The principle of the Crest Drainage Dike is based on a minimization of impact on the present dike and landscape- and nature values on the inner slope.

Enhancement of recreational function of the dike
Because of the possible use of the crest construction as a footpath or cycle path, the Crest Drainage Dike gives good opportunities for the development of recreational values on the dike. To make this possible, entrances towards the dike’s crest must be realized. These entrances are also required for maintenance activities (see paragraph 5.4). The entrance points can consist of a slope or some steps into the construction. How many of these entrance points are required, the approach over the inner slope and the best shape and dimensions of the entrance are aspects that are very dependent of the local situation. These aspects should be worked out in a detailed design, depending on the local criteria and ambitions.
6.3 Assessment Crest Drainage Dike

Table 6-1 gives the assessment criteria for an overtopping dike, as taken from the state-of-the-art study ComCoast [CUR/RWS, 2005]. The weight of nature potentials, maintenance and costs are considered to be the most important. The environmental impact is considered to be a minor item (this impact is hardly identifiable), as well as proven technology (new systems can easily be tested) and vandalism (to be counteracted by monitoring).

A separate item, that should be considered, is the adaptability for coping with further increasing impacts, indicated as ‘adaptation’. For instance, systems that can cope with extreme loads, have a high adaptation, in case of a future increase of loads by ongoing sea level rise. However, when this increase is anticipated to thus small that the overtopping remains limited, a more ‘modest’ reinforcement may suffice. Hence this property is not given a weight factor, but is should be taken into account separately. The adaptation to increased loads is setting a selection criterion on beforehand (‘boundary condition’) for a reinforcement system.

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Weight factor criterium</th>
<th>Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nature potentials</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Environmental impact</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Maintainability</td>
<td>3</td>
<td>0</td>
</tr>
<tr>
<td>Proven technology</td>
<td>1</td>
<td>-1</td>
</tr>
<tr>
<td>Recreational potential</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Sensitivity to vandalism</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Sustainability (lifetime)</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>Scenic quality</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Animal accessibility</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Costs</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Adaptation ref. increased loads</td>
<td>n.a.</td>
<td>-</td>
</tr>
</tbody>
</table>

Compared to the results for the state-of-the-art alternatives (see [CUR/RWS, 2005]), the Crest Drainage Dike obtains a relatively high score because of the good nature potentials on the inner slope, the low impact on the landscape and scenic quality, good opportunities for development of recreational values and relatively low costs. Disadvantages for the Crest Drainage Dike are the lack of knowledge on some issues, e.g. in- and outflow patterns crest construction, erosion of grass inner slope and the length spreading effect of overtopping waves, and the adaptation possibilities for increased loads; once the construction is realized. Once the Crest Drainage Dike has been constructed, it might be difficult to increase the buffer capacity of the crest construction. A possible option is to place two (or one) vertical sidewalls along side the crest construction, in this way the overtopping rate will decrease and the buffer capacity will increase. However, in that case the ffree sight in landward and/or seaward direction is lost. Adding more discharge pipes can increase the average discharge capacity; this is expected to be a relatively simple measure. The described possible adjustment measures for increasing hydraulic conditions in the future is an important point of attention in the design process of the Crest Drainage Dike. The design
conditions should be carefully chosen based on the possibilities to adjust the construction to changing hydraulic conditions.
7 PRELIMINARY DESIGN

7.1 Introduction

Based on the information as gathered in the previous chapters, a preliminary design was set up for two alternatives:

- A concrete crest construction (prefab or in situ) with drainage pipes towards the landward side of the dike;
- A concrete crest construction (prefab or in situ) with drainage pipes towards the seaward side of the dike.

Because of the relative high costs (see chapter 8), the disadvantages during realization (pile frame on top of the dike) and the greater impact on the dike, the alternative with sheet piles as described in chapter 5 was not worked out in a preliminary design.

The preliminary design consists of views from above, cross-profiles of the entire dike and more detailed cross profiles of the crest construction.
7.2 Preliminary design drawings concrete crest construction and landward pipes

Cross-profile entire dike, including pit and landward discharge pipe

View from above, including pit with connection to landward discharge pipe
Detail of concrete crest construction

Detail of concrete crest construction, including pit and connection to landward discharge pipe
7.3 Preliminary design drawings concrete crest construction and seaward pipes

Cross-profile entire dike, including pit and seaward discharge pipe

View from above, including pit with connection to seaward discharge pipe
Detail of concrete crest construction

Detail of concrete crest construction, including pit and connection to seaward discharge pipe
8 COST ESTIMATION

8.1 Introduction

The cost estimation was worked out for the preliminary design as presented in chapter 7. Besides the presented alternatives (concrete crest construction with landward or seaward discharge), the alternative with steel sheet piles and an in situ concrete floor was also included in the cost estimation. Furthermore, the difference in costs between a prefab concrete crest construction and an in situ concrete crest construction is shown in the cost estimation.

This leads to six different alternatives, for which a cost estimation was set up:
1. Prefab concrete crest construction, landward discharge;
2. Prefab concrete crest construction, seaward discharge;
3. In situ concrete crest construction, landward discharge;
4. In situ concrete crest construction, seaward discharge;
5. Steel sheet piles with in situ concrete floor, landward discharge;

The cost estimation was set up for a dike length of 1000 m, especially for the prefab alternative this is of importance because of the fixed costs for making the prefab moulds.

8.2 Estimation of building costs

In Table 8-1 to Table 8-6, the cost estimation is shown for the six alternatives as presented in the introduction. The building costs, divided in direct and indirect costs, and other costs such as engineering, supervision, permits and other are included. In Appendix 4, the detailed foundation of these cost estimates is shown for every alternative (in Dutch).

**Table 8-1: Cost estimation alternative 1, prefab concrete and landward discharge**

<table>
<thead>
<tr>
<th>Description</th>
<th>Costs/ m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleaning activities</td>
<td>€ 7.5</td>
</tr>
<tr>
<td>Groundwork activities</td>
<td>€ 31.2</td>
</tr>
<tr>
<td>Building costs prefab concrete construction</td>
<td>€ 428.0</td>
</tr>
<tr>
<td>Building costs prefab pit construction, tailpiece and landward discharge pipes</td>
<td>€ 206.4</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>€ 15.0</td>
</tr>
<tr>
<td><strong>Subtotal direct costs</strong></td>
<td><strong>€ 888.1</strong></td>
</tr>
<tr>
<td>To be detailed (15%)</td>
<td>€ 103.2</td>
</tr>
<tr>
<td>Indirect costs</td>
<td>€ 162.7</td>
</tr>
<tr>
<td>Incidental expenditures object (10%)</td>
<td>€ 95.4</td>
</tr>
<tr>
<td><strong>Subtotal building costs per m²</strong></td>
<td><strong>€ 1049.4</strong></td>
</tr>
<tr>
<td>Engineering, preparation, administration and supervision (15%)</td>
<td>€ 173.2</td>
</tr>
<tr>
<td>Other costs (permits and insurances) (2%)</td>
<td>€ 23.1</td>
</tr>
<tr>
<td>Incidental expenditures project (10%)</td>
<td>€ 124.6</td>
</tr>
<tr>
<td><strong>Total (excl. VAT)</strong></td>
<td><strong>€ 1370.2</strong></td>
</tr>
</tbody>
</table>
## Table 8-2: Cost estimation alternative 2, prefab concrete and seaward discharge

<table>
<thead>
<tr>
<th>Costs/ m²</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleaning activities</td>
<td>€ 7.5</td>
</tr>
<tr>
<td>Groundwork activities</td>
<td>€ 31.2</td>
</tr>
<tr>
<td>Building costs prefab concrete construction</td>
<td>€ 428.0</td>
</tr>
<tr>
<td>Building costs prefab pit construction, tailpiece and seaward discharge pipes</td>
<td>€ 133.6</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>€ 15.0</td>
</tr>
<tr>
<td><strong>Subtotal direct costs</strong></td>
<td><strong>€ 615.3</strong></td>
</tr>
<tr>
<td>To be detailed (15%)</td>
<td>€ 92.3</td>
</tr>
<tr>
<td>Indirect costs</td>
<td>€ 145.5</td>
</tr>
<tr>
<td>Incidental expenditures object (10%)</td>
<td>€ 85.3</td>
</tr>
<tr>
<td><strong>Subtotal building costs per m²</strong></td>
<td><strong>€ 938.4</strong></td>
</tr>
<tr>
<td>Engineering, preparation, administration and supervision (15%)</td>
<td>€ 154.8</td>
</tr>
<tr>
<td>Other costs (permits and insurances) (2%)</td>
<td>€ 20.6</td>
</tr>
<tr>
<td>Incidental expenditures project (10%)</td>
<td>€ 111.4</td>
</tr>
<tr>
<td><strong>Total (excl. VAT)</strong></td>
<td><strong>€ 1225.2</strong></td>
</tr>
<tr>
<td><strong>Total (incl. VAT 19%)</strong></td>
<td><strong>€ 1458.0</strong></td>
</tr>
</tbody>
</table>

## Table 8-3: Cost estimation alternative 3, in situ concrete and landward discharge

<table>
<thead>
<tr>
<th>Costs/ m²</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleaning activities</td>
<td>€ 7.5</td>
</tr>
<tr>
<td>Groundwork activities</td>
<td>€ 31.2</td>
</tr>
<tr>
<td>Building costs prefab concrete construction</td>
<td>€ 552.5</td>
</tr>
<tr>
<td>Building costs prefab pit construction, tailpiece and landward discharge pipes</td>
<td>€ 206.4</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>€ 15.0</td>
</tr>
<tr>
<td><strong>Subtotal direct costs</strong></td>
<td><strong>€ 812.6</strong></td>
</tr>
<tr>
<td>To be detailed (15%)</td>
<td>€ 121.9</td>
</tr>
<tr>
<td>Indirect costs</td>
<td>€ 192.2</td>
</tr>
<tr>
<td>Incidental expenditures object (10%)</td>
<td>€ 112.7</td>
</tr>
<tr>
<td><strong>Subtotal building costs per m²</strong></td>
<td><strong>€ 1239.3</strong></td>
</tr>
<tr>
<td>Engineering, preparation, administration and supervision (15%)</td>
<td>€ 204.5</td>
</tr>
<tr>
<td>Other costs (permits and insurances) (2%)</td>
<td>€ 27.3</td>
</tr>
<tr>
<td>Incidental expenditures project (10%)</td>
<td>€ 147.1</td>
</tr>
<tr>
<td><strong>Total (excl. VAT)</strong></td>
<td><strong>€ 1618.1</strong></td>
</tr>
<tr>
<td><strong>Total (incl. VAT 19%)</strong></td>
<td><strong>€ 1925.6</strong></td>
</tr>
</tbody>
</table>
### Table 8-4: Cost estimation alternative 4, in situ concrete and seaward discharge

<table>
<thead>
<tr>
<th>Description</th>
<th>Costs/ m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleaning activities</td>
<td>€ 7.5</td>
</tr>
<tr>
<td>Groundwork activities</td>
<td>€ 31.2</td>
</tr>
<tr>
<td>Building costs prefab concrete construction</td>
<td>€ 552.5</td>
</tr>
<tr>
<td>Building costs prefab pit construction, tailpiece and seaward discharge</td>
<td>€ 133.6</td>
</tr>
<tr>
<td>pipes</td>
<td></td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>€ 15.0</td>
</tr>
<tr>
<td><strong>Subtotal direct costs</strong></td>
<td>€ 739.8</td>
</tr>
<tr>
<td>To be detailed (15%)</td>
<td>€ 111.0</td>
</tr>
<tr>
<td>Indirect costs</td>
<td>€ 174.9</td>
</tr>
<tr>
<td>Incidental expenditures object (10%)</td>
<td>€ 102.6</td>
</tr>
<tr>
<td><strong>Subtotal building costs per m²</strong></td>
<td>€ 1128.2</td>
</tr>
<tr>
<td>Engineering, preparation, administration and supervision (15%)</td>
<td>€ 186.2</td>
</tr>
<tr>
<td>Other costs (permits and insurances) (2%)</td>
<td>€ 24.8</td>
</tr>
<tr>
<td>Incidental expenditures project (10%)</td>
<td>€ 133.9</td>
</tr>
<tr>
<td><strong>Total (excl. VAT)</strong></td>
<td>€ 1473.1</td>
</tr>
<tr>
<td><strong>Total (incl. VAT 19%)</strong></td>
<td>€ 1753.0</td>
</tr>
</tbody>
</table>

### Table 8-5: Cost estimation alternative 5, steel sheet piles and landward discharge

<table>
<thead>
<tr>
<th>Description</th>
<th>Costs/ m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleaning activities</td>
<td>€ 7.5</td>
</tr>
<tr>
<td>Groundwork activities</td>
<td>€ 28.4</td>
</tr>
<tr>
<td>Building costs sheet pile construction and in situ concrete floor</td>
<td>€ 669.3</td>
</tr>
<tr>
<td>Building costs prefab pit construction, tailpiece and landward discharge</td>
<td>€ 220.5</td>
</tr>
<tr>
<td>pipes</td>
<td></td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>€ 15.0</td>
</tr>
<tr>
<td><strong>Subtotal direct costs</strong></td>
<td>€ 940.8</td>
</tr>
<tr>
<td>To be detailed (15%)</td>
<td>€ 141.1</td>
</tr>
<tr>
<td>Indirect costs</td>
<td>€ 222.5</td>
</tr>
<tr>
<td>Incidental expenditures object (10%)</td>
<td>€ 130.4</td>
</tr>
<tr>
<td><strong>Subtotal building costs per m²</strong></td>
<td>€ 1434.8</td>
</tr>
<tr>
<td>Engineering, preparation, administration and supervision (15%)</td>
<td>€ 236.7</td>
</tr>
<tr>
<td>Other costs (permits and insurances) (2%)</td>
<td>€ 31.6</td>
</tr>
<tr>
<td>Incidental expenditures project (10%)</td>
<td>€ 170.3</td>
</tr>
<tr>
<td><strong>Total (excl. VAT)</strong></td>
<td>€ 1873.5</td>
</tr>
<tr>
<td><strong>Total (incl. VAT 19%)</strong></td>
<td>€ 2229.4</td>
</tr>
</tbody>
</table>
Table 8-6: Cost estimation alternative 6, steel sheet piles and seaward discharge

<table>
<thead>
<tr>
<th>Description</th>
<th>Costs / m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cleaning activities</td>
<td>€ 7.5</td>
</tr>
<tr>
<td>Groundwork activities</td>
<td>€ 28.4</td>
</tr>
<tr>
<td>Building costs sheet pile construction and in situ concrete floor</td>
<td>€ 669.3</td>
</tr>
<tr>
<td>Building costs prefab pit construction, tailpiece and landward discharge pipes</td>
<td>€ 147.7</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>€ 15.0</td>
</tr>
<tr>
<td><strong>Subtotal direct costs</strong></td>
<td><strong>€ 868.0</strong></td>
</tr>
<tr>
<td>To be detailed (15%)</td>
<td>€ 130.2</td>
</tr>
<tr>
<td>Indirect costs</td>
<td>€ 205.3</td>
</tr>
<tr>
<td>Incidental expenditures object (10%)</td>
<td>€ 120.3</td>
</tr>
<tr>
<td><strong>Subtotal building costs per m²</strong></td>
<td><strong>€ 1323.8</strong></td>
</tr>
<tr>
<td>Engineering, preparation, administration and supervision (15%)</td>
<td>€ 218.4</td>
</tr>
<tr>
<td>Other costs (permits and insurances) (2%)</td>
<td>€ 29.1</td>
</tr>
<tr>
<td>Incidental expenditures project (10%)</td>
<td>€ 157.1</td>
</tr>
<tr>
<td><strong>Total (excl. VAT)</strong></td>
<td><strong>€ 1728.5</strong></td>
</tr>
<tr>
<td><strong>Total (incl. VAT 19%)</strong></td>
<td><strong>€ 2056.9</strong></td>
</tr>
</tbody>
</table>

In Table 8-7, a summary is given of the estimations for building costs per alternative. This summary shows that for a dike length of 1000 m, the prefab alternative is about 20 % cheaper than the in situ alternative. For smaller dike lengths though, at some point the in situ alternative will become cheaper. Furthermore, as was expected, the landward discharge alternative is more expensive than the seaward discharge alternative. On the total price, the difference in costs is about 10 %. Another conclusion that can be drawn is that the alternative with steel sheet piles is significantly more expensive than the concrete crest construction. In combination with the larger impact on the dike, and the difficulties in realization in an already existing dike (erected pile frame on top of the dike crest where little space is available), it is concluded that the sheet pile alternative is not interesting for further study.

Table 8-7: Summary of cost estimates for different alternatives

<table>
<thead>
<tr>
<th>Crest construction</th>
<th>Discharge pipes 400 mm / 30 m</th>
<th>Building costs per m²</th>
<th>Other costs</th>
<th>Total (excl. VAT)</th>
<th>Total (incl. VAT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prefab concrete</td>
<td></td>
<td>Landward</td>
<td>€ 1049.4</td>
<td>€ 320.8</td>
<td>€ 1370.2</td>
</tr>
<tr>
<td>Prefab concrete</td>
<td></td>
<td>Seaward</td>
<td>€ 938.4</td>
<td>€ 286.9</td>
<td>€ 1226.2</td>
</tr>
<tr>
<td>In situ concrete</td>
<td></td>
<td>Landward</td>
<td>€ 1239.3</td>
<td>€ 378.8</td>
<td>€ 1618.1</td>
</tr>
<tr>
<td>In situ concrete</td>
<td></td>
<td>Seaward</td>
<td>€ 1128.2</td>
<td>€ 344.9</td>
<td>€ 1473.1</td>
</tr>
<tr>
<td>Steel sheet piles, in situ concrete floor</td>
<td></td>
<td>Landward</td>
<td>€ 1434.8</td>
<td>€ 438.6</td>
<td>€ 1873.5</td>
</tr>
<tr>
<td>Steel sheet piles, in situ concrete floor</td>
<td></td>
<td>Seaward</td>
<td>€ 1323.8</td>
<td>€ 404.7</td>
<td>€ 1728.5</td>
</tr>
</tbody>
</table>

*for example engineering, permits, supervision, incidental expenditures project

*Risks in cost estimation*
The cost estimation is based on general assumptions for the existing dike. Local circumstances might increase the costs significantly. For example, if some type of construction (road, house or other) is present on the dike crest, this needs to be removed and this can increase the costs enormously. Furthermore, for private buildings (houses or companies), compensation costs need to be taken into account.

Furthermore, the costs for possible measures against piping were not included in the cost estimation because the necessity for these measures cannot be established in general. Only for the landward alternative, this might be necessary. More insight in the actual water pressure build up during wave overtopping needs to be obtained for the local situation, to determine whether a seepage screen is necessary. Because the seepage screen has to be placed in an existing dike, this measure will increase the building costs significantly. An alternative is to place a clay layer around the crest construction and discharge pipes, the costs for this measure depends on the availability of clay.

8.3 Maintenance costs

For the maintenance of the concrete construction, a cleaning vehicle can be used like normally used on roads. This should be done a few times a year (at least once before storm season!). Furthermore the connections between the concrete construction and the discharge pipes must be checked and cleaned manually with certain regularity. The visual inspection should include a check if the flow is not blocked anywhere and a check of the condition of the concrete construction. Costs for cleaning and visual inspection are relatively low, a very rough estimate of the maintenance costs is about € 250 per 100 m dike length per year.

Furthermore, the discharge pipes have to be flushed at least once a year before storm season. For this purpose, special vehicles are easily available and these can do their work from the crest construction. Costs for flushing of the discharge pipes are roughly estimated to be about € 450 per 100 m dike length per year for the landward alternative (length discharge pipes about 50 meters) and about € 150 per 100 m dike length per year for the seaward alternative (length discharge pipes about 10 to 15 meters).

Together, the maintenance costs are estimated at € 700 per 100 m dike length per year for the landward alternative, and € 400 per 100 m dike length per year for the seaward alternative.
9 PERMITS, EXEMPTIONS AND PROCEDURES

9.1 Introduction

From different acts and regulations in the Netherlands, required permits and licenses follow that might be required for constructing the Crest Drainage Dike. This can concern acts with relation to flood management (e.g. ‘Wet op de Waterkering’), environment (e.g. ‘Flora and Faunawet’), acts concerning burial and disposal of polluted sediments, acts concerning materials used (e.g. ‘Bouwstoffenbesluit’) and acts to protect other functions and values of the sea dike (cultural monuments and objects, landscape value and functions such as housing, industry and agriculture). The acts and regulations might have a national status, or a regional status (province, municipality or water board). In the following paragraph, all possibly required permits and licenses for construction of the Crest Drainage Dike in an existing sea dike are summarized. The responsible authority, possible risks and the normal duration to obtain this permit of license are included.
9.2 Overview of permits, exemptions and procedures

<table>
<thead>
<tr>
<th>Acts and regulations</th>
<th>Procedures</th>
<th>Relevant activities</th>
<th>Rough terms</th>
<th>Competent authorities</th>
<th>Risks</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Environmental Management Act</td>
<td>Environmental Impact Assessment</td>
<td>Change or extension of a sea dike, delta dike or river dike of 5 km or more.</td>
<td>1 year</td>
<td>Related to consent by Provincial Executive of the plan (as referred to in article 7 section 1 from the ‘Wet op de Waterkering’)</td>
<td>The term necessary to formulate an environmental impact assessment can take extra time. If for this purpose required information isn’t available (in time), extra research must be done.</td>
<td>NB: Check the activities on the European Directives, The Decree on Environmental Impact Assessments 1994, the in there mentioned threshold values, the decree the environmental impact assessment is applied to and the competent authority.</td>
</tr>
<tr>
<td>Flora- and Fauna Act</td>
<td>Exemption</td>
<td>Activities that can generate a serious threat for plants and wild life in the immediate vicinity can generate a deterioration of the circumstances that are necessary for the survival of the species.</td>
<td>8 weeks (in real terms ca. 6 months)</td>
<td>Minister of Agriculture, Nature Management and Fisheries</td>
<td>The possibility of objection and appeal can cause a substantial delay.</td>
<td></td>
</tr>
<tr>
<td>Spatial Planning Act</td>
<td>Revision of the zoning plan</td>
<td>Every modification that is not established in the zoning plan.</td>
<td>1.5 year</td>
<td>Local Council</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td>NB: Check the necessity of an environmental impact assessment.</td>
</tr>
<tr>
<td>Land Clauses Act, Title III (about the alienation for the benefit of construction, repair, strengthening or maintenance of dikes)</td>
<td>Legal procedure</td>
<td>Non-amicably obtaining of property or rights on behalf of construction, repair, strengthening or maintenance of dikes.</td>
<td>1.5 year</td>
<td>The Crown, by Royal Decree</td>
<td>Risks are included in the rough term.</td>
<td>The preparations for the legal procedure run analogue with the zoning plan procedure. The legal procedure may only be commenced after the zoning plan has become irreversible.</td>
</tr>
<tr>
<td>Agricultural Holdings Act</td>
<td>Approval from the Land Tenure</td>
<td>Amicable rescission of the lease (Land Tenure), Judicial rescission of</td>
<td>Lease Tenure 1 tot 1.5 year</td>
<td>Land Tenure respectively Lease Tenure</td>
<td>Risks are included in the rough term.</td>
<td>This takes place at the cantonal court if parties can not come to terms.</td>
</tr>
</tbody>
</table>
### DHV Environment and Transportation

<table>
<thead>
<tr>
<th>Acts and regulations</th>
<th>Procedures</th>
<th>Relevant activities</th>
<th>Rough terms</th>
<th>Competent authorities</th>
<th>Risks</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Environmental Management Act / Public administrative act</td>
<td>Application for a permit</td>
<td>Temporary storage of (polluted) sediment.</td>
<td>6 months</td>
<td>The bench of Mayor and Aldermen or Provincial Executive</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td></td>
</tr>
<tr>
<td>Pollution of Surface Waters Act</td>
<td>Application for a permit</td>
<td>Pollution of the surface water by waste products, harmful products or poisonous products (for example polluted ground).</td>
<td>6 months</td>
<td>Ministry of Transport and Public Works</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td></td>
</tr>
<tr>
<td>Soil Protection Act</td>
<td>Report a pollution</td>
<td>Relocation of polluted sediment. The use of polluted sediment in the construction of structural works. Construction of a depot for fourth class polluted sediment.</td>
<td>13 weeks</td>
<td>Ministry of Transport and Public Works or Provincial Executive</td>
<td>A newly established pollution takes extra time and money due to research and soil sanitation.</td>
<td></td>
</tr>
<tr>
<td>Building Materials Decree</td>
<td>Report</td>
<td>The use of polluted sediment in the construction of structural works.</td>
<td>2 days (first category), 1 month (other allowed categories)</td>
<td>The bench of Mayor and Aldermen or Provincial Executive or the ministry for Housing, Regional Development ad the Environment or the district water board</td>
<td>An unsatisfactory or not representative batch sample, the batch must be checked again. A batch from which the concentration of pollution is too high must be removed.</td>
<td></td>
</tr>
<tr>
<td>‘Wet beheer Rijkswaterstaat-werken’</td>
<td>Application for a permit</td>
<td>Relocation of polluted sediment. Temporary storage of sediment from the dikes in the summer bed or winter bed. Changes in the winter</td>
<td>8 weeks</td>
<td>Ministry of Transport and Public Works</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td></td>
</tr>
<tr>
<td>Acts and regulations</td>
<td>Procedures</td>
<td>Relevant activities</td>
<td>Rough terms</td>
<td>Competent authorities</td>
<td>Risks</td>
<td>Comments</td>
</tr>
<tr>
<td>----------------------</td>
<td>------------</td>
<td>---------------------</td>
<td>-------------</td>
<td>----------------------</td>
<td>-------</td>
<td>----------</td>
</tr>
<tr>
<td>‘Ontgrondingenwet’</td>
<td>Application for a permit</td>
<td>Relocation of ground in the dike area. Relocation of polluted sediment. Relocation of cables or wires.</td>
<td>6 months</td>
<td>Ministry of Transport and Public Works or Provincial Executive</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td>The application for an permit is only necessary if this is stated expressly in the zoning plan</td>
</tr>
<tr>
<td>Spatial Planning Act</td>
<td>Application for a permit</td>
<td>Construction of structural works.</td>
<td>4 weeks</td>
<td>The bench of Mayor and Aldermen</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td></td>
</tr>
<tr>
<td>Housing Act</td>
<td>Application for a permit</td>
<td>Construction of structural works.</td>
<td>13 - 26 weeks</td>
<td>The bench of Mayor and Aldermen</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td></td>
</tr>
<tr>
<td>Local regulation and local act to fell trees</td>
<td>Exemption of the local regulation and a permit to fell trees</td>
<td>Sound pollution caused by felling of trees.</td>
<td>8 weeks</td>
<td>The bench of Mayor and Aldermen</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td></td>
</tr>
<tr>
<td>‘Waterschapskeur’</td>
<td>Application for an exemption</td>
<td>Relocation of ground in the dike area. Temporary storage of sediment. Relocation of cables or wires. Felling of trees.</td>
<td>8 weeks</td>
<td>The district water board</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td></td>
</tr>
<tr>
<td>Road Traffic Act</td>
<td>Application for an exemption</td>
<td>Measures regarding roads on the dikes, relocation of traffic signs.</td>
<td>8 weeks</td>
<td>The bench of Mayor and Aldermen</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td></td>
</tr>
<tr>
<td>Telecommunications facilities Act</td>
<td>Application concerning the duty to tolerate the relocation of cables or wires</td>
<td>Relocation of cables or wires.</td>
<td>2 months</td>
<td>Ministry of Transport and Public Works</td>
<td>The risk of extra not mentioned cables or wires</td>
<td></td>
</tr>
<tr>
<td>Historic Buildings and Ancient</td>
<td>Application for a permit</td>
<td>Demolition or changes of a monument, among which</td>
<td>6 months</td>
<td>Minister van OCW, Gedeputeerde Staten of</td>
<td>The possibility of objection and appeal can cause a substantial delay</td>
<td>One should work out if there are high or middle high values of archaeological</td>
</tr>
</tbody>
</table>

CUR/Innovative concept overtopping dike: Crest Drainage Dike

WG-SE20051409

10 November, version 1

- 55 -
## DHV Environment and Transportation

<table>
<thead>
<tr>
<th>Acts and regulations</th>
<th>Procedures</th>
<th>Relevant activities</th>
<th>Rough terms</th>
<th>Competent authorities</th>
<th>Risks</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monuments Act</td>
<td>archaeological sites.</td>
<td>College van B&amp;W</td>
<td>delay</td>
<td>sites in advance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Impediments Act</td>
<td>Enforce the duty to tolerate use of property</td>
<td>In case individuals (who can claim private law protection) must tolerate activities concerning construction or modification due to projects</td>
<td>no terms are mentioned; in general circa. 1 year</td>
<td>Ministry of Transport and Public Works</td>
<td>Risks are included in the rough term</td>
<td></td>
</tr>
</tbody>
</table>
10 CONCLUSIONS AND RECOMMENDATIONS

10.1 Feasibility and opportunities Crest Drainage Dike

From this theoretical study it can be concluded that the concept of the Crest Drainage Dike is technically and financially feasible and offers good opportunities for recreational and environmental development. The concept does require some further research however to determine its effectivity with more certainty.

Financial feasibility
First of all, the Crest Drainage Dike is competitive in costs with possible alternatives. The costs are significantly lower than the traditional heightening of the dike and the costs are relatively low compared to the measures to strengthen the inner slope as presented in the state-of-the-art study for ComCoast [CUR/RWS, 2005].

Technical feasibility
Based on the assumptions that were adopted in this study, it can be concluded that the crest construction offers sufficient discharge capacity and sufficient buffer capacity in extreme conditions to ensure the stability of the dike for an average overtopping rate of 15 l/s/m (minimum requirement). To be able to cope with higher average overtopping rates, the average discharge capacity can be increased (by increasing the number of or size of the discharge pipes) and the buffer capacity can be increased by enlarging the concrete crest construction.

An important aspect in the adopted assumptions is the amount of overtopping water that is trapped by the crest construction in relation to the remaining overtopping flow (and thus hydraulic load) on the inner grass slope. This is mainly determined by the inflow and possible outflow pattern of the crest construction (see Figure 2-2 for an illustration of undesired outflow patterns) in extreme conditions. This aspect requires further research (see paragraph 10.2), which might lead to an optimization in the shape of the crest construction to ensure a smooth inflow and to prevent an outflow at the landward sidewall of the construction (see Figure 3-2 for possible other shape). However, the concept and technical feasibility of the Crest Drainage Dike will be maintained. For the prefab alternative, other shapes of the crest construction will not influence the costs significantly.

Other important technical aspects are the stability of the dike and the crest construction. From this study it can be concluded that the realization of the Crest Drainage Dike has minor influence on the overall stability of the sea dike. The safety against piping could not be proven because of uncertainty about the saturation height in the outer slope under wave overtopping. Based on expert judgment however it is not expected that piping will have a significant influence on the stability of the Crest Drainage Dike. A local verification of the possible threat of piping is required for the profiles where the landward discharge pipes are placed; this verification is based on an analysis of the exact local water pressure development in the outer slope under wave overtopping.
The dynamic horizontal forces exerted on the Crest Drainage Dike, caused by inflow of overtopping water, can be coped with by counteracting friction forces caused by the weight of the crest construction and the sand package next to the crest construction. Vertical stability is assured by sufficient bearing capacity of the subsoil.

Opportunities
The Crest Drainage Dike offers good opportunities for the development of recreational values on the dike. The crest construction can be used as a footpath and/or cycling path, offering an excellent view in both the seaward and the landward direction. The seaward direction can offer good views of beach/ dunes and sea, while in landward direction a view of nature areas or villages might be interesting. The alternative of landward discharge of the overtopping water, offers extra opportunities in combination with the development of salty and wet nature in the hinterland. Furthermore, the present natural landscape of the sea dike (grass inner slope, with sheep grazing) will not affected by the Crest Drainage Dike.

10.2 Further research

The most important technical aspect of the Crest Drainage Dike that requires further research is the amount of overtopping water that is trapped by the crest construction in relation to the remaining overtopping flow (and thus hydraulic load) on the inner grass slope.

Physical model tests
Regarding the first aspect, the amount of overtopping water that is trapped by the crest construction, further research into the inflow and possible undesired outflow patterns is required (as was mentioned in the paragraph above). Physical model tests are recommended to obtain more insight into the flow patterns in the crest construction, and if necessary measures to counter undesired flow patterns. Different assumptions regarding the in- and outflow of the crest construction should be checked:

- Check of the overtopping rate over the landward sidewall of the crest construction, under the expected overtopping flow conditions (flow velocity and flow layer thickness). If it turns out that overtopping rate is significant, the effects of alternative shapes for the landward sidewall on the overtopping rate should be tested.
- Check whether the overtopping flow will easily flow into the crest construction (as a sheet flow) under the expected flow conditions. If it turns out that this is not the case, the shape of the dike zone in front of the crest construction and the shape of the seaward sidewall of the crest construction can be adjusted; this can be included in the model tests.
- Check when the overtopping water will start flowing over the present water layer in the crest construction. In this study this was assumed at the point when 80 % of the construction is filled with water. If it turns out to be less, the shapes of the construction might be adjusted or the depth of the crest construction can be increased to increase the buffer capacity of the crest construction (depth increase is limited though, because of required free sight from the crest construction for recreational purposes).

Further theoretical research
Other aspects of importance for the Crest Drainage Dike, which have not yet been subject of sufficient research in the past, are:

- The short-crestedness of wind-generated waves in relation to instantaneous overtopping rates over the entire length of the dike. A thorough theoretical analysis of the wave spectrum, directional spreading and short-crestedness in relation to the dike length can provide more information. Check of assumed length spreading factor 2 for the Crest Drainage Dike (see paragraph 2.2.3).

- The erosion resistance of the inner grass slope in conditions with high flow velocities during a short time period (when the crest construction’s capacity is insufficient in extreme conditions). Further development of theoretical relations for the erosion resistance of a grass revetment in relation to the overtopping flow velocities and period of occurrence of these flow velocities. Check of calculations erosion resistance grass slope as executed in paragraph 3.3.2.

The subjects above require further research in general; physical model tests might be included in this research.

*Points of attention in further development*

Besides the theoretical and physical research that is required for further development of the Crest Drainage Dike concept, several attention points should be taken into account in further development. These points need to be worked out further and taken into account in the design of the Crest Drainage Dike:

- In the design of the Crest Drainage Dike, the options to adapt the construction to possibly increasing hydraulic loads after the design period of 50 years need to be taken into account carefully. Possible measures to adapt the construction are described in section 6.3 (page 39). This aspect needs to be worked out further in relation to the life cycle costs of the Crest Drainage Dike.

- Maintenance aspects are of importance for the Crest Drainage Dike, especially during storm conditions. Since the crest is not accessible during these conditions, it must be shown that this not endanger the safety of the dike. The metal grids at the discharge pipes might get clogged by trash or other objects. However, we expect that the turbulent overtopping water entering the crest construction will actually flush the metal grids clean during every wave-overtopping event, minimizing the risk of clogging. This should be worked out further however.

- Another maintenance aspect is flushing of the discharge pipes. This can be executed with simple equipment, however it must be kept in mind in the design that sharp curves in the pipes will create sediment traps. This will make it more difficult to flush the pipes.

- Depending on the available space in the dike crest, it is advised to make the crest construction as wide as possible. Increasing the width increases the buffer capacity and increases the friction for the water that enters the crest construction, minimizing the risk of overtopping of the landward sidewall. In general, it can be said that the Crest Drainage Dike is probably best applicable on relatively wide dike crests.

- In the design, extra attention must be given to measures to prevent people from walking on the vulnerable grass slope transition zones alongside the crest construction. Also measures are required to prevent sheep from falling into the crest construction.
10.3 Detailed design

Some important aspects and attention points to be worked out in the detailed design are given below:
• the shape of the concrete crest construction;
• the shape of the zone seaward of the crest construction;
• the outflow construction at the outflow of the discharge pipes in the discharge channel of hinterland (for the landward discharge alternative);
• the storage capacity and/or capacity discharge channel in the hinterland (for the landward discharge alternative);
• the slope protection at the outflow of the seaward discharge pipe (for the seaward discharge alternative);
• detailed design of joints and transitions;
• whether measures against piping are necessary (seapage screen or clay layer around construction);
• the entrance points into the crest construction.
REFERENCES


[Battjes, 1982] Effect of short-crestedness on wave loads on long structures, Jurjen A. Battjes, Department of Civil Engineering, Delft University of Technology, Delft, the Netherlands, Applied Ocean Research, 1982, Vol. 4, No. 3


[DHV, 2005] Innovative concept for an overtopping dike, proposal, DHV Environment and Transportation, WG-SE20050625, April 2005


COLOPHON

Client: Fout! Verwijzingsbron niet gevonden.
Project: Innovative concept overtopping dike: Crest Drainage Dike
File: X1498
Length of report: 63 pages
Author: O.E. Nieuwenhuis
Project Manager: H. van Hemert
Project Director: T. Louters
Date: 10 November 2005
Name/Initials:  

APPENDIX 1: DESCRIPTION OF CREST DRAINAGE DIKE CONCEPT

Background of the concept
In the State of the Art study [Haskoning, 2005] all sorts of different types of revetments are described. The hard type of revetments, such as pitched revetments, concrete, gabions and asphalt, on the one hand have a high erosion resistance but on the other hand the costs for these alternatives are relatively high and the effects on landscape and nature can be very negative.

In our opinion, instead of increasing the revetment strength of the entire inner slope, as is the case in all the State of the Art alternatives, it is also possible to decrease the overtopping loads at the inner slope without alteration of the cross section of the dike. We see a challenge in trying to find a way to decrease the loads on the inner slope by taking more local measures at the crest of the dike. Because of the smaller physical scale of measures at the crest, this will have much less negative effects on LNC-values and the costs will be lower because at the inner slope the grass revetment can be maintained.

In short, we propose an alternative where the wave-overtopping load on the inner slope is reduced to an acceptable level by measures taken at the crest of the dike.

Crest drainage dike
This concept consists of a concrete construction in the crest of the dike (in the form of a wide U-profile), see Figure 12-1 for a conceptual sketch. Most of the overtopping water will be caught in this construction and will be discharged through drains either to the inner side of the dike or if necessary also (partly) to the outer side of the dike. For the ComCoast concept it should be studied further in a following phase first of all how much salt water input is desired and can be stored at the landward side of the dike, and second if the option of also discharging water towards the seaward side of the dike could be preferable in extreme situations. If it is, the seaward discharge should be adjustable, for example by installing a valve.

Figure 12-1: Conceptual sketch Crest Drainage Dike (not scaled!)
Because the crest construction will trap a significant part of the overtopping water, this water will not run off the inner slope, resulting in a reduction of the loads on the inner slope to a normal level. In extreme situations, the capacity of the drains might not be sufficient, causing more water to run off the inner slope. However, also in this situation the crest construction will cause a load reduction for the inner slope. The overtopping waves will not pass the crest in a sheet flow (as is the case for smooth dike crests) but will loose its energy in the crest construction and finally flow over the edge towards the inner slope if the construction’s capacity is exceeded. The water that eventually reaches the inner slope will have less energy because of the crest construction, even if the occurring wave overtopping exceeds the construction’s capacity.

The discharge takes place under natural drop through for instance a simple synthetic PP pipe (Poly Propylene). The pipe will be buried superficially to take it out of sight. The concrete U-profile’s bottom should have a small gradient towards the discharge pipes, which are located at the inner side of the crest. The concrete construction can function as a promenade for walking and cycling with a nature area on one side and the sea on the other side.

A variation to be worked out in a later stage is to replace the concrete U-profile with for instance two lighter and easier to manufacture L-profiles on a body of gravel, if necessary partly penetrated with an open structure of colloidal concrete or bitumen in order to resist the loads by the water mass on the crest. This alternative is shown in Figure 12-2.

Figure 12-2: Conceptual sketch variation Crest Drainage Dike (not scaled!)

A variation that was considered for the discharge method was to apply concrete channels instead of synthetic pipes to discharge the water from the crest construction. However it is expected that the costs for this option are much higher and the effects on nature, landscape and cultural values are expected to be higher. Furthermore when wave overtopping exceeds the design value, overflowing of the channel might cause local scour of the grass slope next to it.
APPENDIX 2: EQUATIONS FOR WAVE OVERTOPPING CALCULATIONS

Wave run up
The wave run-up height is given by \( z_{2\%} \). This is the wave run-up level, measured vertically from the still water line, which is exceeded by 2% of the number of incoming waves. The number of waves exceeding this level is hereby related to the number of incoming waves and not to the number that run-up. The general equation that can be applied for wave run-up on dikes is given by:

\[
\frac{Z_{2\%}}{H_{m0}} = 1.75 \times \gamma_b \times \gamma_f \times \gamma_\beta \times \zeta_0
\]

\[\text{equation 2-1}\]

with a maximum for larger \( \Delta_0 \) of:

\[
\frac{Z_{2\%}}{H_{m0}} = \gamma_f \times \gamma_\beta \times \left( \frac{4.3 - 1.6}{\sqrt{\Delta_0}} \right)
\]

\[\text{equation 2-2}\]

where:

- \( Z_{2\%} \): wave run-up level above still water line (m)
- \( H_{m0} \): significant wave height at toe of dike (m)
- \( \Delta_0 \): breaker parameter (-)
- \( \gamma_b \): influence factor for a berm (-)
- \( \gamma_f \): influence factor for roughness elements on slope (-)
- \( \gamma_\beta \): influence factor for angled wave attack (-)

The equation is valid in the area \( 0.5 < \gamma_b \Delta_0 < 8 \) to 10. The relative wave run-up \( \frac{Z_{2\%}}{H_{m0}} \) depends on the breaker parameter \( \Delta_0 \) and three influence factors: for a berm (applied to the breaker parameter), roughness elements on the slope, and angled wave attack. The significant wave height is \( H_{m0} \), the average wave period is \( T_m \), and the spectral period is \( T_m_{0.1} \).

Average overtopping rates
The crest height is lower than the wave run-up levels of the highest waves in case of wave overtopping. The parameter that must be used is the free crest height \( R_c \), the difference in height between the still water line and the crest height.

Average wave overtopping is the average discharge per metre construction width; \( q \) in l/s per m. Wave overtopping is calculated in relation to the height of the outer crest line. Average wave overtopping equations are exponential functions with the general form:

\[
q = a \exp b \times R_c
\]

\[\text{equation 2-3}\]
The coefficients a and b are functions of the wave height, slope angle, breaker parameter and other influence factors. The complete equation is as follows:

$$\frac{q}{\sqrt{g \cdot H_{m0}} \cdot \tan \alpha} = \frac{0.067}{\sqrt{\tan \alpha}} \cdot \frac{1}{\xi_0} \cdot \frac{1}{\exp \left( -4.3 \cdot \frac{h_k}{H_{m0}} \cdot \frac{1}{\xi_0} \cdot \frac{1}{\gamma_f} \cdot \frac{1}{\gamma_b} \right)}$$

equation 2-

With a maximum of:

$$\frac{q}{\sqrt{g \cdot H_{m0}^2}} = 0.2 \cdot \exp \left( -2.3 \cdot \frac{h_k}{H_{m0}} \cdot \frac{1}{\gamma_f} \cdot \frac{1}{\gamma_b} \right)$$

equation 2-

where:

- $q$ = average wave overtopping rate (l/s/m)
- $g$ = acceleration due to gravity (m/s²)
- $H_{m0}$ = significant wave height at toe of dike (m)
- $\xi_0$ = breaker parameter (-)
- $s_0$ = wave steepness = $2 \cdot H_{m0}/(g \cdot T_{m,1.0})$ (-)
- $T_{m,1.0}$ = spectral wave period at toe of dike (s)
- $\tan \theta$ = slope (-)
- $R_c$ = free crest height above still water line (m)
- $\theta$ = influence factors for influence of berm, roughness elements, angle of wave attack (-)

**Instantaneous overtopping rates (per wave)**

Using the average overtopping rates, the probability distribution function for the overtopping rates per wave can be calculated. This probability distribution function is a Weibull distribution with a shape factor of 0.75 and a scale factor $a$, which depends on the average overtopping rates and the probability of occurrence for overtopping per wave. The probability distribution function is given by:

$$P_V = P(V \leq V) = 1 - \exp \left( -\left( \frac{V}{a} \right)^{0.75} \right)$$

equation 2-

with:

$$a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}}$$

equation 2-

where:

- $P_V$ = probability that wave overtopping volume per wave $V$ is greater than or same as $V$ (-)
- $V$ = wave overtopping volume per wave (l/m)
- $T_m$ = average wave period ($NT_m$ is duration of storm or examined time period) (s)
- $q$ = average wave overtopping discharge (l/m per s)
- $P_{ov} = N_{ov}/N$ = probability of overtopping per wave (-)
- $N_{ov}$ = number of overtopping waves (-)
\( N = \text{number of incoming waves during period of storm (} \cdot \text{)} \)

The probability of overtopping per wave can be calculated as follows:

\[
P_{0v} = \exp \left( - \ln 0.02 \frac{h_k}{Z_{2\%}} \right)^2 \quad \text{equation 2.8}
\]

Besides the probability of overtopping per wave, the actual overtopping rate per wave can be calculated with:

\[
V = a \ast ( - \ln(1 - P_v) )^{5/3} \quad \text{equation 2.9}
\]

A first estimate of the value for the maximum volume of one wave that can be expected in a certain period can be obtained by taking into account the number of overtopping waves \( N_{0v} \):

\[
V_{\text{max}} = a \ast ( \ln( N_{0v} ) )^{4/3} \quad \text{equation 2.10}
\]

**Overtopping velocity and layer thickness, seaward slope**

The evolution of the wave-run-up tongue on the seaward slope is required to determine the layer thickness and the overtopping velocity at the beginning of the dike crest. The layer thickness and the overtopping velocity on the seaward slope depend on the incoming wave parameters. The wave run-up velocities at the outer crest line can be determined by [Schüttrumpf and Van Gent, 2003]:

\[
\frac{u_{A,2\%}}{\sqrt{gH_s}} = \frac{\text{c}^*_{A,u}}{H_s} \sqrt{\frac{Z_{2\%} - z_s}{H_s}} \quad \text{equation 2.11}
\]

with: 
- \( u_{A,2\%} \) = wave run-up velocity exceeded by 2% of the incoming waves 
- \( z_s \) = position on the seaward slope with respect to SWL 
- \( \text{c}^*_{A,u} \) = empirical coefficient 
- \( Z_{2\%} \) = wave run-up height exceeded by 2% of the incoming waves

The layer thickness on the seaward slope can be determined assuming a linear decrease of the layer thickness from SWL to \( Z_{2\%} \) [Schüttrumpf and Van Gent, 2003]:

\[
\frac{h_{A,2\%}}{H_s} = \frac{\text{c}^*_{A,h}}{H_s} \left( \frac{Z_{2\%} - z_s}{H_s} \right) \quad \text{equation 2.12}
\]

with: 
- \( h_{A,2\%} \) = wave run-up velocity exceeded by 2% of the incoming waves
\[ c^{*}_{A,n} = \text{empirical coefficient} \]

The empirical coefficients were determined in model tests in [Van Gent, 2002]:
\[ c^{*}_{A,u} = 1.30 \]
\[ c^{*}_{A,n} = 0.15 \]

**Overtopping velocity, landward slope**

The maximum flow velocity at the landward slope can be calculated with the following simple equation, as was derived by van Gent [van Gent, 2002]:

\[
 u_{B,2\%} = \sqrt{\frac{2gh_{B}u_{B} \sin \beta}{f}} \quad \text{equation 2-13}
\]

where:
- \( u_{B,2\%} \) = maximum velocity on inner slope that is exceeded by 2\% of the waves [m/s]
- \( g \) = gravitational acceleration [m/s\(^2\)]
- \( h_{B} \) = layer thickness at inner crest line [m]
- \( u_{B} \) = flow velocity at inner crest line [m/s]
- \( \beta \) = slope angle inner slope
- \( f \) = friction coefficient

with \( f = \frac{8g}{C^{2}} \quad \text{equation 2-14} \)

where:
- \( C \) = Chézy coefficient [m\(^{1/2}\)/s]
APPENDIX 3: GEOHYDROLOGICAL CALCULATION RESULTS

Permeable sand dike on less permeable sandy subsoil

Permeable sand dike on more permeable sandy subsoil
Non-permeable clay dike on permeable sandy subsoil

Permeable sand dike on high permeable sandy subsoil
APPENDIX 4: COST ESTIMATION

DHV Environment and Transportation

Orientation:
- prefab concrete and landward discharge

Alternative 1: Prefab concrete and landward discharge

<table>
<thead>
<tr>
<th>Operation</th>
<th>Quantity</th>
<th>Unit</th>
<th>Cost</th>
<th>Unit Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erection</td>
<td>ft²</td>
<td>$</td>
<td>$</td>
<td></td>
</tr>
<tr>
<td>Work</td>
<td>hour</td>
<td>$</td>
<td>$</td>
<td></td>
</tr>
</tbody>
</table>

Landward discharge
- Prefab concrete and landward discharge

Cost estimation

Total cost

Calculations

Additional costs:
- prefab concrete and landward discharge

Conclusions:

CUR: Innovative concept overtopping dike: Crest Drainage Dike

10 November, version 1

WG-SE20051409

- 72 -
### Alternative 2: Prefab concrete and seaward discharge

**Groundarea per m2**

<table>
<thead>
<tr>
<th>Description</th>
<th>m²</th>
<th>15.00</th>
<th>0.00</th>
<th>7.5</th>
</tr>
</thead>
</table>

**Amort (Grootschalige)**

<table>
<thead>
<tr>
<th>Description</th>
<th>€</th>
<th>1.4</th>
</tr>
</thead>
</table>

**Groundwater**

- oogstgrond gravel/loam
  - m³ | € | 520.0 | 0.0 | 140.0
- subsoil gravel/loam
  - m³ | € | 24.0 | 0.0 | 6.0

**Amort Groundwater**

| | € | 15.0 |

**Prefab building components**

- element prefabricated
  - m² | € | 3.0 | 0.6 | 15.0
- prefabricated prefabricated
  - m³ | € | 1.0 | 0.2 | 340.0
- prefabricated prefabricated
  - m³ | € | 1.0 | 0.2 | 340.0

**Amort Prefab building components**

| | € | 42.0 |

**Aveersevoort construction directors in prefabrication**

<table>
<thead>
<tr>
<th>h</th>
<th>m</th>
<th>€</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.0</td>
<td>500.0</td>
</tr>
</tbody>
</table>

**Aveersevoort construction directors in prefabrication**

| | € | 15.0 |

**Amort Diverse**

| | € | 0.0 |

**Optimalisatie**

| | € | 0.0 |

**Natuurlijke kosten**

| | € | 0.0 |

**Suboptimale kosten**

| | € | 0.0 |

**AANBROUDESTAAT**

| | € | 20.0 |

**AANBROUDESTAAT**

| | € | 0.0 |

**Groothoek per m²**

| | € | 20.0 |

**IVESTERING**

| | € | 0.0 |

| | € | 14.0 |

**Voorzieningen**

| | € | 0.0 |

**Totaal Vastgoed**

| | € | 0.0 |

**OVERIGE KORRIGERENDE Kosten**

| | € | 20.0 |

**Project overzicht**

| | € | 0.0 |

**Totaal Project overzicht**

| | € | 0.0 |

**CUR/Innovative concept overtopping dike: Crest Drainage Dike**

10 November, version 1

WG-SE20051409

- 73 -
### Alternative 3: In situ concrete and landward discharge

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>alternative concrete and landward</td>
<td></td>
<td></td>
</tr>
<tr>
<td>discharge (concrete and landward)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>total cost</td>
<td></td>
<td>€ 232.5</td>
</tr>
</tbody>
</table>

**Note:**
- The value represents the total cost associated with the alternative of in situ concrete and landward discharge.
### Alternative 4: In situ concrete and seaward discharge

<table>
<thead>
<tr>
<th>Activity</th>
<th>Description</th>
<th>Unit</th>
<th>Cost (€)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>In situ concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>seaward discharge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>Total cost</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Alternative 5: Steel sheet piles and concrete in-situ floor, landward discharge

<table>
<thead>
<tr>
<th>Cost Item</th>
<th>Unit Cost</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel sheet piles</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete in-situ floor</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Landward discharge</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Detailed Breakdown

- **Steel Sheet Piles**
  - **Cold Formed Steel Sheet Piles**: Using Cold Formed Steel Sheet Piles for sheeting, with a thickness of 8 mm.
  - **Concrete in-Situ Floor**: Constructing the in-situ concrete floor for stability and load-bearing capacity.
  - **Landward Discharge**: Ensuring effective drainage and sediment control.

### Additional Considerations
- **Environmental Impact**: Assessing the environmental impact of using steel and concrete, including lifecycle analysis.
- **Construction Timing**: Coordinating the construction schedule to minimize disruptions.
- **Maintenance Plan**: Establishing a maintenance plan for the new infrastructure.

---

**DHV Environment and Transportation**

**Innovative concept overtopping dike: Crest Drainage Dike**

10 November, version 1

WG-SE20051409

- 76 -
Alternative 6: Steel sheet piles and concrete in situ floor, seaward discharge

DHV Environment and Transportation

### Kosten en parametrisering

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Waarde</th>
<th>Unit</th>
<th>Opmerkingen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume</td>
<td>7.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Biederservice

<table>
<thead>
<tr>
<th>Biederservice</th>
<th>Waarde</th>
<th>Unit</th>
<th>Opmerkingen</th>
</tr>
</thead>
</table>

### Eerste analyses

**Bijlagen:**

- [Bijlage A: Teknische specifieken](#)
- [Bijlage B: Economische analyses](#)
- [Bijlage C: Technologische besluitvorming](#)

---

CUR/Innovative concept overtopping dike: Crest Drainage Dike

10 November, version 1

WG-SE20051409

- 77 -
### Costs Crest Drainage Dike, prefab concrete and landward discharge

Dimensions are based on the Hondsbossche Zeewering 4300 m

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost per m (€)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purchase of land</td>
<td>20</td>
</tr>
<tr>
<td>Required m² of land needed behind the existing dike</td>
<td>0</td>
</tr>
<tr>
<td>Cleaning activities</td>
<td>7.5</td>
</tr>
<tr>
<td>Groundwork activities</td>
<td>31.2</td>
</tr>
<tr>
<td>Building costs prefab concrete construction</td>
<td>428</td>
</tr>
<tr>
<td>Building costs prefab pit construction, tailpiece and seaward discharge pipes</td>
<td>206.4</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>15</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>688.1</strong></td>
</tr>
</tbody>
</table>

Mobilisation cost per project: 15000 €

Total construction costs: 2973830 €

Maintenance

- Yearly maintenance cost/ 100 m: 700 €/ 100 m dike
- Yearly maintenance cost total: 30100 €/project
- Expected repair cost: 5 €/m
- Total yearly maintenance: 21500 €
- Lifetime of the structure: 50 years
- Interest rate: 5%
- Capitalised maintenance: 942006 €

Total cost of structure: 3.9 million €

To be filled in by ComCoast
To be filled in by Consultant
## Costs Crest Drainage Dike, prefab concrete and seaward discharge

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost (€/m)</th>
<th>Total (€)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensions are based on the Hondsbossche Zeewering</td>
<td>length</td>
<td></td>
</tr>
<tr>
<td>Purchase of land</td>
<td>4300 m</td>
<td></td>
</tr>
<tr>
<td>Required m2 of land needed behind the existing dike</td>
<td>20 €/m²</td>
<td></td>
</tr>
<tr>
<td>Cleaning activities</td>
<td>7.5 €/m</td>
<td></td>
</tr>
<tr>
<td>Groundwork activities</td>
<td>31.2 €/m</td>
<td></td>
</tr>
<tr>
<td>Building costs prefab concrete construction</td>
<td>428</td>
<td></td>
</tr>
<tr>
<td>Building costs prefab pit construction, tailpiece and landward discharge</td>
<td>133.6 €/m</td>
<td></td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>15 €/m</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>615.3 €/m</td>
<td>2645790 €</td>
</tr>
<tr>
<td>Mobilisation cost per project</td>
<td>15000 €</td>
<td></td>
</tr>
<tr>
<td><strong>Total construction costs</strong></td>
<td></td>
<td>2660790 €</td>
</tr>
<tr>
<td>Maintenance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yearly maintenance cost/100 m dike</td>
<td>400 €/100 m</td>
<td></td>
</tr>
<tr>
<td>Yearly maintenance cost total</td>
<td>17200 €/project</td>
<td></td>
</tr>
<tr>
<td>Expected repair cost</td>
<td>5 €/m</td>
<td>21500 €</td>
</tr>
<tr>
<td><strong>Total yearly maintenance</strong></td>
<td></td>
<td>38700</td>
</tr>
<tr>
<td>Lifetime of the structure</td>
<td>50 years</td>
<td></td>
</tr>
<tr>
<td>Interest rate</td>
<td>5%</td>
<td></td>
</tr>
<tr>
<td>Capitalised maintenance</td>
<td></td>
<td>706504 €</td>
</tr>
<tr>
<td><strong>Total cost of structure</strong></td>
<td></td>
<td>3.4 million €</td>
</tr>
</tbody>
</table>

*To be filled in by ComCoast*

*To be filled in by Consultant*
APPENDIX 6: COMMENTS BY WP3 USERS GROUP AND EU TEAM + DHV REACTION

In this appendix, the comments as given by the WP3 users group and EU team on the first draft report about the Crest Drainage Dike by DHV are given (black text). These comments were discussed on 20 October during a meeting in Deft, the Netherlands. Underneath the comments, in blue and italic text, the reaction of DHV on these comments and the way that the comments were integrated in the final version of this report are described.

Comments notes meeting 20 oktober:

Based on the comments in the notes about the meeting of 20 oktober 2005, some more text regarding the possibilities to adapt the Crest Drainage Dike to changing hydraulic conditions was added in the report in section 6.3 ‘Assessment Crest Drainage Dike’ (page 39), described also in relation to costs of these measures. Based on the comments, this aspect was also added as a point of attention for further development of the concept in the final chapter Conclusions and Recommendations. Furthermore, the ComCoast cost table was set up for the Crest Drainage Dike (added in Appendix 5).

Written comments users group:

1. Once installed this solution is not flexible (can not be altered, so should be designed very carefully) Mentioned in chapter Evaluation Crest Drainage Dike (page 39). Furthermore, added as point of attention in final chapter Conclusions and Recommendations.

Maintenance (grid may not get covered with plastics, ...) is important factor Extra remarks were added on page 36, where this aspect is mentioned. Furthermore, added as point of attention in final chapter Conclusions and Recommendations.

The assumption that the water will flow over the dike after a filling of 80% is essential for the further study. Should be proven. Mentioned in final chapter Conclusions and Recommendations in section ‘further research’ (page 55).

Additional information is necessary about the length spreading reduction factor. Important for knowing the buffer capacity (as well as the assumption above) Mentioned in final chapter Conclusions and Recommendations in section ‘further research’ (page 56).

When calculating the horizontal forces (p. 34) no safety factor is included. Belgian standards do include this (e.g. 1,2) Belgian standards were not considered in our study. Furthermore, if the friction forces are divided by the load, the ‘safety’ factor is 13.5 kN/m : 11 kN/m = 1.23 > safety factor 1.2. No problems are expected.

The use as a footpath does not seem that attractive when walking in a concrete construction of approx. 1m height. Here the natural element seems lost From our point of view, the adopted internal height of 80 cm offers a free sight for recreational users of the concrete construction in both landward as seaward direction. The natural element comprises
the view of the dike (because the crest construction is hidden in the dike) and the inner slope (because this can remain a grass slope in stead of a strengthened slope), not the crest construction itself.

2.

Uitvoerige studie en omvangrijk rapport!
Maar toch heb ik mijn twijfels. Vooral omtrent buffer en afvoercapaciteit vande kruijn bak.

(Blz 7) "some overtopping is allowed ........ Therefore, the erosion resistance of the grass inner slope is checked for maximum instantaneous overtopping rates ......" (blz 7). Dit doen ze vervolgens echter maar in zeer beperkte mate. M.i. te beperkt omdat de (overslag)belasting op het binnentalud aanzienlijk is. Op blz 16 en 22 wordt daaromtrent gemeld dat u = 7,2 m/s en "...... during 25% of the time ...... , the instantaneous overtopping volume is higher than the buffer capacity and thus will reach the inner slope (blz 22)". Onvoldoende buffer bij 25% van de (overslapping) waves betekent dat de hoeveelheid overtopping water misschien wel 80%*(see our reaction in third paragraph below) van het totale hoeveelheid overslag bedraagt (dus aanzienlijk meer dan 25%). Ofwel gemiddeld 0,80 maal 15 = 12 l/s/m. En tevens dat de maximale stroomsnelheden langs het binnentalud nauwelijks worden gereduceerd. Ik betwijfel of de bouw van een "crest drainage structure" hiermee wel gerechtvaardigd is (of ik moet me vergissen).

The objective of the Crest Drainage Dike is to decrease the duration of the maximum hydraulic load on the inner slope. The maximum occurring flow speed on the inner slope is indeed not reduced by the Crest Drainage Dike, but the duration of this hydraulic load is reduced. Instability of the inner slope will occur after a certain (longer) period of overtopping flow, the concept of the Crest Drainage Dike is based on the idea that the inner slope can handle extreme flow velocities for short durations. Therefore, it is not realistic to translate the total amount of overtopping water (as mentioned 80% of the total overtopping water, however see our reaction below* on this aspect) back into an average overtopping rate to show that this concept is not effective. The calculated average overtopping rate in this way cannot be related to the effectiveness of the Crest Drainage Dike. What should be checked for the Crest Drainage Dike (and was checked in our study) is whether the inner slope can handle the extreme flow speeds for the calculated decreased duration of this maximum flow speed.

Currently, little knowledge is available on the erosion resistance of a grass inner slope under short but extreme hydraulic loads; this requires further research as is indicated in the final chapter Conclusions and Recommendations. Furthermore, a thorough statistical analysis of instantaneous wave overtopping rates in a storm wave field in relation to the buffer capacity of the crest construction and the remaining hydraulic load on the inner slope should be executed. Based on the given comments, this recommendation for further development of the concept was added in the final chapter Conclusions and Recommendations.

Another remark concerning the given comments is that some erosion of the inner grass slope is not expected to cause any problems. Most important is maintaining the design crest height. The concrete crest construction will not erode and therefore, as long as this construction is stable, the crest height is maintained. Only if erosion of the inner slope is of such extent that the stability of the concrete construction is endangered, will failure of the dike occur.

*An insufficient buffer capacity for the highest 25 % of the waves does not mean that 80 % of the overtopping water is not trapped in the crest construction. Not all of the overtopping water can be trapped during these highest waves, but for most of these waves the biggest part of the overtopping water is trapped in the crest construction.

Ik vermoed dat er op blz 22 een foutje wordt gemaakt ! Er wordt gesproken over “25% of the time in the storm period ...... the calculated design flow speed for the inner slope of 7,2 m/s, occurring
during 25% of the storm period.” Het moet volgens mij zijn “25% of the overtopping waves” in plaats van “25% of the waves”. Dit betekent een veel kortere totale belastingstijd.

Indeed a misinterpretation of the results of the overtopping calculations has occurred in the study. As mentioned above, it is not true that 25 % of all waves exceed the buffer capacity of the crest construction. It should be that 25 % of all overtopping waves exceed the buffer capacity of the crest construction. This has been adjusted in the report. Because the first (wrong) assumption was conservative in relation to the new one, this will not influence the technical feasibility in a negative way. The duration of the load decreases which only makes the design safer.

Afvoer- en buffercapaciteit crest structure voldoende voor 15 l/s/m (blz 54):
-relatieve Capaciteit bak is 2,6 m³/m.
-debit bij maximale golf 11,0 m³/m en bij 5% golf 6,4 m³/m. Omdat grootste golf meestal niet alleen komt, zal er heel wat water overheen spoelen.
Dit bevestigt dat heel veel water over de bak zal heenspoelen.
See described objectives and effectiveness of Crest Drainage Dike above.

Piping/onderloopseheid – als mogelijkheid wordt kruin afdekken met klei genoemd (blz 49). Maar ik denk niet dat daar klei alleen volstaat. Niet in de eerste plaat vanwege onderloopseheid maar vooral vanwege erosie (golfaanval op kruin bij 15 l/s/m).

Quote page 49: ‘An alternative is to place a clay layer around the crest construction and discharge pipes, the costs for this measure depends on the availability of clay.’ This does not mean a clay layer is placed as top layer on the crest, it is placed under and around the concrete crest construction and discharge pipes with the sole purpose to prevent piping. Erosion resistance of the top layer at the outer slope run up zone and transition zone to crest construction is a different issue. This last issue was added as attention point in the final chapter Conclusions and Recommendations.

Kosten orde k€ 2000 per km zonder dat huizen/gebouwen of infrastructuur moeten worden afgebroken en/of (financieel) gecompenseerd. Omtrent deze kosten wordt opgemerkt (blz 54):
costs significantly lower than traditional heightening
relatively low compared to other measures for strengthening the inner slope
Mede gezien het bovenstaande (mbt buffer- en afvoercapaciteit) ben ik nog niet overtuigd !
See our reaction above, according to our calculations the inner slope can handle the remaining short lasting hydraulic load under the adopted conservative assumptions. We agree it requires further research such as a thorough statistical analysis of instantaneous wave overtopping rates in relation to the buffer capacity of the crest construction. This recommendation was added in the final chapter Conclusions and Recommendations.

Op blz 39 wordt als negatief punt de “adaptation possibilities for increased loads” genoemd. Dit is met name van belang als de effectiviteit niet erg groot is (mbt buffer- en afvoercapaciteit).

See reaction above, the effectivity is sufficient to deal with 15 l/s/m at least and more depending on the possible dimensions of the crest construction (related to available space).

Er word een experimentele fase aanbevolen als een volgende stap. In dit geval modelproeven. Ik denk dat dit terecht is voor zover het de hydraulische aspecten van de crest drainage structure en de belasting op het binnentalud betreft. Niet de erosieweerstand van het binnentalud.

Mentioned as such in the final chapter Conclusions and Recommendations in section 10.2 ‘further research’ (page 55). For the hydraulic aspects model tests are recommended, for the erosion resistance of the grass inner slope further research in general was recommended.

3.
- Monitoring and maintenance (cleaning) of the system are crucial given possible risk of drainage pipe blockage.
Extra remarks were added in section 5.4 ‘maintenance aspects’ (page 36), where this aspect is mentioned. Furthermore, added as point of attention in final chapter Conclusions and Recommendations.

- Consider possible risk of overflow at one specific location at the inner crest (due to blockage pipe, damage structure, settlement of crest element). This will lead to high local velocities at inner slope (exceeding the used 7.2 m/s).

  The calculated 7.2 m/s is the maximum flow speed that will occur according to the used relations at the inner slope without the presence of the crest construction! Therefore, this value will not be exceeded when the construction is ineffective one specific location. The duration of this load might increase. However, because of the length spreading effect over the dike length, the failure of one of the discharge pipes locally is not expected to cause instability of the inner slope. Furthermore, it is expected that an overtopping wave will enter the crest construction with such power and turbulence that the metal grids are naturally flushed and cleaned. Clogging is therefore not expected to occur. However, based on this comment this issue was added as attention point in further research in the final chapter Conclusions and Recommendations.

- Access to crest structure should be prohibited and prevented during storms. Alternative access road required for dike monitoring during storms?

  Maintenance possibilities during storm conditions have been added in section 5.4 ‘maintenance aspects’ (page 37) and added as important point of attention in further development of the Crest Drainage Dike in the final chapter Conclusions and Recommendations. VTV demands were checked and adopted in these additional texts.

- Table 2-3: suggests that it shows the influence of sea level rise. However, given depth limited conditions at the toe, an increase in water level will also imply higher waves and hence larger overtopping volumes (not included in Table 2-3)

  The text formulation with this table was adapted to minimize possible confusion.

4.

General comment: The study/analysis is very practical! The design of the crest drainage dike is based on simple design relations and expert judgment. The most important failure mechanism are considered such as wave overtopping, macro stability, piping and heave. Moreover, constructional aspects are included in the design and a preliminary design has been discussed. My compliments for this.

However, at present I still do not know whether the design is safe enough and whether it is the most economical attractive solution.

In chapter 2 the amount of wave overtopping has been discussed. The mean discharge is about 15 l/s/m. The volume of an excessive wave could be 300 times larger than that averaged wave, for example V10% is about 4500 cubic meter per wave (not correct: 4,500 l/m per wave= 4.5 m³/m² per wave). In such situations you have to deal with extreme load conditions. I wonder what will happen if two extreme waves will occur in a row. The capacity of the storage reservoir is about 10 cubic meter (?? buffer capacity of 2.6 m³/m² was calculated in our study). The volume of two super waves is 10,000 cubic meters (not correct: 10,000 l/m per wave= 10 m³/m² per wave). If two super waves will overtop, the reservoir is of no use. What do you think?

As was indicated, the numbers presented above in our opinion are not correctly interpreted from our study; the unit of cubic meters was mixed up with liters. The reservoir is always of use, and the main objective is to change the overtopping and therefore hydraulic load on the inner slope from a long-lasting condition to a short but extreme condition. Based on this comment and other comments however, the additional recommendation is added to execute a thorough statistical analysis of a storm period and instantaneous overtopping rates in relation to the buffer capacity of the crest construction.
The CIRIA curve: The CIRIA curve has been discussed on page 23. What is the definition of these curve. I mean what is the damage factor if such curve will be used for design purposes? Will there any damage at all or will the damage be restricted to for example a few centimeters erosion?

This curve does not say anything about a damage factor, it indicates which flow speed and duration the inner grass slope can handle without causing instability. This implies that some damage will occur in the extreme design condition, but the stability of the inner slope is maintained.

5.

Beside the problem of entrance points towards the crest construction it should be considered that the “traffic safety” will be a problem. It will remain very attractive to walk or ride with a bike on top of the dike, now very close to the 0,8m deep concrete construction. And finally, the grassing sheep must be guarded against falling into the drainage channel.

Measures are required to prevent these things, as is described in section 5.3 ‘joints and transitions’ for the transition zones along side the crest construction. Furthermore, it was added as point of attention in the final chapter Conclusions and Recommendations.

Alternative drainage to the seaside: Normally, in Germany the revetments are not built up to a level that a 1:10 drainage would dewater onto the revetment. An additional erosion protection jointed to the drainage pipe on the outer slope will be necessary to protect the outer slope.

In that case, this alternative option of the Crest Drainage Dike is probably not preferable since extra protection is required.

The maintenance of the drainage pipe system (landward side alternative) will result in higher effort and amount than stated in the report. We think that an inspection of the pipe (and a regular flushing) will be necessary. To do so, the construction of inspection manholes should be taken into consideration.

The metal grid and pit construction is open for visual inspection from the crest construction and flushing of the discharge pipes can be executed with simple equipment (see section 5.4 “maintenance aspects”). Inspection manholes are not required in our opinion. In the design however, it must be taken into account that sharp curves in the pipes can create sediment traps, which can make flushing of the pipes more difficult and expensive.

6.

It is assumed that the layer thickness and velocity on the inner slope equals the values on the outer slope. This will depend on the resistance of the bottom layer. In this case the bottom layer is water and is very smooth. This may cause the velocity to increase and the layer thickness to decrease. A friction factor decrease by factor 3 causes 1.4 times higher velocities. The grass cover on the inner slope cannot handle this. This makes the solution not feasible.

It was assumed that the velocity does not decrease or increase over the crest. In our opinion this is a conservative assumption because 1) there is no driving force to accelerate the flow over the crest, so an increase of the flow velocity will not occur and 2) probably the flow layer will experience some friction passing the crest because of the irregular shape of the crest and turbulence when the water enters the crest construction, this might cause a decrease in the flow velocity which is not taken into account. Therefore, in our opinion this does not affect the feasibility of the solution.

In tables 2-3 till 2-7 the used hydraulic conditions are unrealistic. For realistic combinations a relation like Hs=Hs_ref+0,5xDelta_H should be used where Delta_H = increase in water level and Hs_ref= basic boundary wave condition. This will give different design combinations in table 2-7. Also it is not clear what is the probability of the design combinations in table 2-7.

The study on overtopping rates in the report has the objective to 1) gain more insight into the instantaneous overtopping rates and the probability of occurrence of these overtopping rates and 2) gain some insight into the sensitivity of these overtopping rates when changing certain parameters.
It was never the intention to determine the exact and accurate design conditions and the probability of the computed combinations.

The design was roughly determined on the approximate range of possible instantaneous overtopping rates; the adopted level of detail in the design conditions in this study fit with the design method. In a further stage, a more thorough statistical analysis of design conditions and overtopping rates in relation to the design of the crest construction should be executed. Based on this comment and other comments, this recommendation was added in the final chapter Conclusions and Recommendations.

The seaward alternative pipe location will have a much higher resistance because of the design wind speed entering the pipe (>30m/s). This may cause a serious feasibility problem but is not taken into account. Also the pipe will exit at a level around 9.3m+NAP. This is in the serious run up zone and needs attention.

In our opinion, the wind speed will not cause a higher resistance for the water discharge; the wind is not probable to enter the pipe because it has sufficient options/ directions to go offering less resistance. The wind will therefore diverge in another direction than into the pipe. Furthermore, the water force is expected to be greater than the wind pressure on the pipe.

The design of the pipe exit in the wave run up zone is mentioned in section 5.3 ‘joints and transitions’ (page 36) and is included as a point of attention in further detailing of design (as mentioned in section 10.3 ‘detailed design’ on page 56).

The transition from the CDD to the inner grass slope is a potential weak point. The design of this transition is unclear.

The crest construction is packed in the dike body so the transition zone consists of a grass slope with clay just like the inner slope. The main risk for this zone is people walking or biking on the grass slope, affecting the state and quality of the grass slope. Measures are needed to prevent this, as is described in section 5.3 ‘joints and transitions’ for the transition zones along side the crest construction. Furthermore, based on the comments, this was added as point of attention in the final chapter Conclusions and Recommendations.

Despite the measures at the crest still a very large drainage capacity is needed. Is this included in the cost analysis?

We do not understand this comment; the crest construction cannot be regarded without the drainage measures. Both have different functions within the system of the Crest Drainage Dike. Taking more measures at the crest does not the decrease the required drainage capacity. In the cost analysis, the entire solution is taken into account including the drainage measures.

7.

Bebouing op de dijk geeft met name bij zwaardere randvoorwaarden (al of niet t.g.v. zeespiegelrijzing en/of bodemdaling) problemen bij aanpassing van de waterkering. Hoewel bij de dimensionering rekening is gehouden met zeespiegelstijging en verzwaarde randvoorwaarden tgv toegenomen golfhoogte en –periode blijft er een risico bij onverwachte ontwikkelingen.

In de tekst van DHV is er al ingegaan op de overgangs- en aansluitconstructies. Bij dit ontwerp vraagt dit veel aandacht.

De Hondsbossche bevat dijkovergangen en trappen. Hoe wordt dit gedacht mee te nemen in het ontwerp?

Verder vraagt de constructie nogal wat inspectie/onderhoud.

These comments were all mentioned before and reacted on already. Based on the comments, these aspects have been added as point of attentions for further development of the Crest Drainage Dike in the final chapter Conclusions and Recommendations.