Integrated Flood Risk Analysis
and Management Methodologies

FLOODsite

# Preliminary reliability analysis of flood defences in the pilot site ‘Scheldt’

Date May 2006

Report Number T07-06-03
Revision Number 1_2_P12

Task Leader Delft University of Technology

FLOODsite is co-funded by the European Community
Sixth Framework Programme for European Research and Technological Development (2002-2006)
FLOODsite is an Integrated Project in the Global Change and Eco-systems Sub-Priority
Start date March 2004, duration 5 Years

Document Dissemination Level

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|---|---|---|
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Co-ordinator: HR Wallingford, UK
Project Contract No: GOCE-CT-2004-505420
Project website: www.floodsite.net

SIXTH FRAMEWORK PROGRAMME

## DOCUMENT INFORMATION

| | |
|---|---|
| **Title** | **Preliminary reliability analysis of flood defences in the pilot site Scheldt** |
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| **Distribution** | **Task 7 Partners, Theme 4 (Task 27)** |
| **Document Reference** | **T07-06-03** |

## DOCUMENT HISTORY

| Date | Revision | Prepared by | Organisation | Approved by | Notes |
|---|---|---|---|---|---|
| 29/05/06 | 1_0_P12 | PVG, SG | TUD | | 1st draft |
| 15/06/06 | 1_2_P12 | J Bushell | HRW | | Formatting, and change of name from 'Floodsite_Scheldt.doc' and change of Report Number from 'T07-06-02' |
| 10/06/09 | 1_2_P12 | Paul Samuels | HR Wallingford | | Formatting and file name. PU dissemination. Correction to summary (Schelde not German Bight) |
| | | | | | |
| | | | | | |

## ACKNOWLEDGEMENT

The work described in this publication was supported by the European Community's Sixth Framework Programme through the grant to the budget of the Integrated Project FLOODsite, Contract GOCE-CT-2004-505420.

## DISCLAIMER

## SUMMARY

This report describes the flood defence structures of the pilot site River Schelde Estuary and the overall probability of failure of all flood defences in the area. First, the flood prone area is briefly described as follows:

- the flood prone area
- the failures observed in the past
- an overview of all defence structures
- the flood defence structures in detail together with their potential failure modes

The report then continues to describe the algorithm how the flood defences are split into various sections. For each section the probability of failure is then calculated using level II and III methods. From this, the overall failure probability $P_f$ is calculated.

The overall purpose of this report is to provide a first idea on the failure probability calculations of the flood defences in the pilot site. This information will identify the gaps in knowledge (e.g. failure modes) and incomplete procedures to calculate $P_f$. Results will be fed into Theme 1 to improve understanding and knowledge where identified necessary.

Calculations have been made by DHV Consultancy and Engineering (Amersfoort, NL) and TU Delft with checks by VNK (Ministry of Water Management, Delft, NL) and assessments by WZE (Water board, Zeeland, NL).

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# 1. Introduction

## 1.1 Background

FLOOD*site* is aiming for Integrated Flood Risk Analysis and Management Methodologies. New research efforts in this field will be undertaken to fill gaps in knowledge and to achieve a better understanding of the underlying physics of flood related processes.

Any new knowledge developed in FLOOD*site* will be developed and tested at selected pilot sites in Europe which will help to identify missing elements in research. These pilot sites are

- River Elbe Basin
- River Tisza Basin
- Flash Flood Basins
    - the Cévennes-Vivarais Region (France);
    - the Adige River (Italy);
    - the Besos River and the Barcelona Area (Spain);
    - the Ardennes Area (Trans-national);
- River Thames Estuary
- River Scheldt Estuary
- River Ebro Delta Coast
- German Bight Coast

It can be seen that pilot sites are well distributed over the types of waters like rivers, estuaries and coasts as well as types of floods like plain and flash floods. For each of those sites at least two pilot areas with different properties have been selected to test as many newly developed tools as possible. The ‘Scheldt’ has been selected as a typical North Sea area which is protected against coastal flooding by means of different flood defence structures such as forelands, sea dikes, dunes and other constructions.

The methodologies developed under FLOOD*site* are partly based on a probability based risk analysis. This analysis will require a set of failure modes and related limit state equations for each of the flood defence structures under question. The aim of this report is to provide a first calculation of the overall failure probability of flood defence structures in the Scheldt area. The limit state equations which will be used within this report is based on available LSEs outside FLOOD*site*. These equations will be updated when more information is available from Task 4 of FLOOD*site*.

At the beginning of a reliability analysis of a flood defence system, a very limited physical knowledge will be available on failure modes, their interactions and the associated prediction models, including the uncertainties of the input data and models. Therefore, a detailed flood risk assessment based on a sound physical understanding of the failures and the possible flooding of the protected area will not be feasible at this stage. Therefore, initially, the reliability analysis focuses on providing support to feasibility level decisions.

In order to identify the relative importance of the gaps in the existing knowledge and to help to optimise research objectives, it is necessary to perform a very preliminary flood risk analysis using a holistic approach (feasibility level). For this purpose, three selected pilot sites in different countries and from different areas (coast, estuary, river) will be used (HRW, TUD, and LWI). The main outputs and benefits from this preliminary study will identify more precisely (i) the relative importance of the uncertainties and their possible contributions to the probability of flooding, (ii) the gaps related to prediction models and limit state equations by means of a detailed top-down analysis; (iii) the uncertainties which are worth reducing by the generation of new knowledge, (iv) the priorities with respect to the allocation of research efforts for the various topics to be addressed in the other sub-projects, (v) the areas of high, low and medium uncertainty.

There is potential for significant differences in the PRA approach between the 3 pilot studies. TUD/HR/LWI need to review before any work starts to ensure that, at minimum, there is a common understanding of each PRA approach, and at best, that a common approach is adopted for all three.

The preliminary analysis in this report will assess the probabilities of flooding and related uncertainties in the south-western province of the Netherlands. Dike ring area 32 will be examined to see how reliable the flood defences are and to identify any weak points. In particular attention will be paid to the special elements in the dike rings; hydraulic structures such as locks, weirs and pumping stations. To date, little is known about the safety of these elements.

Existing techniques (among others the PC Ring approach) will be applied in first instance. Refined techniques will be proposed in case the resulting failure probability from PC Ring is too inaccurate.

The Western Scheldt forms the entrance to the harbour of Antwerp (Belgium). Water levels are influenced by the wind surges on the North Sea, as well as the river discharges from the Scheldt. There are four surrounding dike ring areas along the Western Scheldt (no. 29 to 32).

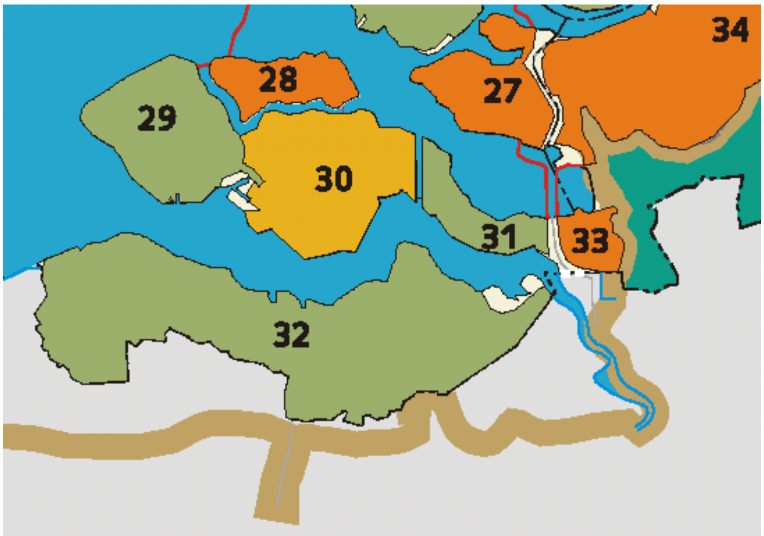



*Figure 1 Dike ring areas in the southern part of the province Zeeland, along the estuary Western Scheldt: no. 29 = Walcheren, no. 30 = Zuid Beveland West, no. 31 = Zuid Beveland Oost, no. 32 = Zeeuwsch Vlaanderen*

The water board Zeeuwse Eilanden (http://www.wze.nl) has provided the problem identification and data with respect to problematic dike sections along the western Scheldt. The study of VNK (Ministry of Water Management) will serve as a basis for further investigations of this test pilot site.

# 2. Pilot Site 'Scheldt'

This section provides a description of dike ring area 32, Zeeuws-Vlaanderen, and the schematizations of the various dike sections. The assessment of the water board is given in this section as well.

Section 2.1 provides general information concerning the location and the characteristics of the dike ring followed by an overview of the dikes and structures in section 2.2. Sections 2.3 to 2.8 take a closer look at the schematization of the dikes and dunes. Section 2.9 finally gives an overview of the assessment of the water board. Calculations have been made by DHV with checks by VNK and assessments by WZE.

## 2.1 Location and characteristics

Dike ring area 32 encompasses all of Zeeuw-Vlaanderen with primary embankments of category a, these are embankments that enclose the dike ring areas – either with or without high grounds- and directly retain outside water, along the North Sea and Westerschelde. The length of primary embankments in Zeeuws-Vlaanderen amounts to 85 kilometers, of which 8 kilometers of dune coast. The exceedance frequency for this area equals to 1/4000 years. The dike ring is border-crossing with Belgium. The embankments in Belgium are of category d. Its length is unknown. A system of regional (secondary) embankments is situated at a variable distance from the primary embankments along the whole North Sea coast and Westerschelde.

An overview of the dike ring area is given in figure 2-1.

The dike ring is enclosed by the following embankments:

- The dike along the Westerschelde
- The dike along the Schelde
- The high grounds in Belgium and Northern France
- The sea retaining dunes or dikes of Belgium, Northern France and the Netherlands

## 2.2 Dikes, dunes and structures

An overview of the embankments in dike ring 32 is given on the overview map primary and regional embankment of dike ring area 32. The following important water retaining structures can be distinguished:

- Dike with stone covering
- Dike with grass covering
- Dike with asphalt covering
- Dune
- Sea walls RWS (Public Works and Water Management)
- Engineering structure

The following division can be made:

- 0 - 0.8 km : dike with stone covering
- 0.8 - 4.3 km : dike with grass covering
- 4.3 - 20.1 km : dike with stone covering
- 20.1 - 22.0 km : sea wall RWS
- 22.0 - 40.2 km : dike with stone covering
- 40.2 - 44.7 km : sea wall RWS
- 44.7 - 67.0 km : dike with stone covering
- 76.0 - 68.2 km : dune

- 68.2 - 69.7 km : sea wall RWS
- 69.7 - 70.1 km : dike with stone covering
- 70.1 - 71.2 km : dune
- 71.2 - 76.3 km : dike with stone covering
- 76.3 - 77.3 km : dune
- 77.3 - 78.8 km : dike with grass covering
- 78.8 - 79.8 km : dike with stone covering
- 79.8 - 82.7 km : dune
- 82.7 - 82.9 km : dike with stone covering
- 82.9 - 84.3 km : dune
- 84.3 - 84.6 km : dike with stone covering
- 84.6 - 85.1 km : dune
- 85.1 - 85.7 km : grass

The division and selection of dike and dune section is looked further into in section 2.3.

14 Structures are present in dike ring area 32. An overview of these structures is given in table 2-1.

| | |
|---|---|
| 1 | Pumping station Cadzand |
| 2 | Pumping station Campen |
| 3 | Pumping station Nieuwe Sluis |
| 4 | Pumping station Nummer Een |
| 5 | Pumping station Othene |
| 6 | Pumping station Paal |
| 7 | Sluice station Terneuzen Oostsluis |
| 8 | Sluice station Terneuzen Middensluis (schutsluis) |
| 9 | Sluice station Terneuzen Middensluis (spuiriool) |
| 10 | Sluice station Terneuzen Westsluis |
| 11 | Sluice station Terneuzen Westsluis (spuiriool) |
| 12 | Discharge sluice station Braakman |
| 13 | Discharge sluice station Hertogin Hedwigepolder |
| 14 | Discharge sluice station Nol Zeven |

*Table 2.1: Structures in dike ring 32*

## 2.3 Division in 33 dike and 4 dune sections

The dike ring area “Zeeuws-Vlaanderen” was initially divided into 287 dike sections according to the VNK-schematization. These were mainly dikes, but encompassed a number of dunes and structures as well. Because calculating the probability of failure for this number of dike sections with PC-Ring is very elaborate, a selection has been made by DHV. This selection is based on the presently existing sections in PC-Ring. Thus no routes with representative dike sections have been selected.

The chosen 33 dike and 4 dune sections are dike ring covering and are deemed to be representative for the total dike ring.

The dike ring area is divided into parts for the selection, each with their own characteristic orientation. One or more dike sections are selected within these parts, where thought is given to the following aspects:
Length of the dike section
Height of the crown
Height of the toe
Orientation of the dike section
Presence of shoulder and/or bend (in other words type of dike section)

Dike covering

The results of the already calculated overflow/wave run-up and bursting/piping of PC-Ring are considered for the choice of dike sections. The dike sections with a significant higher probability of failure have been selected. It was decided to add two more weak links, in consultation with the District Water Board Zeeuws-Vlaanderen. These are dike sections 7009 and 7023. This brings the total number of sections that are taken into account in PC-Ring to 37, of which 33 dike and 4 dune sections. This number is without the water retaining structures (14 structures). The location of the selected dike sections is shown in figure 2-1 (in which dike section 2 represents dike section number 7002 etc). The selected dune sections are given in figure 2-2 (dune section 8 represents dune section number 7008 etc).

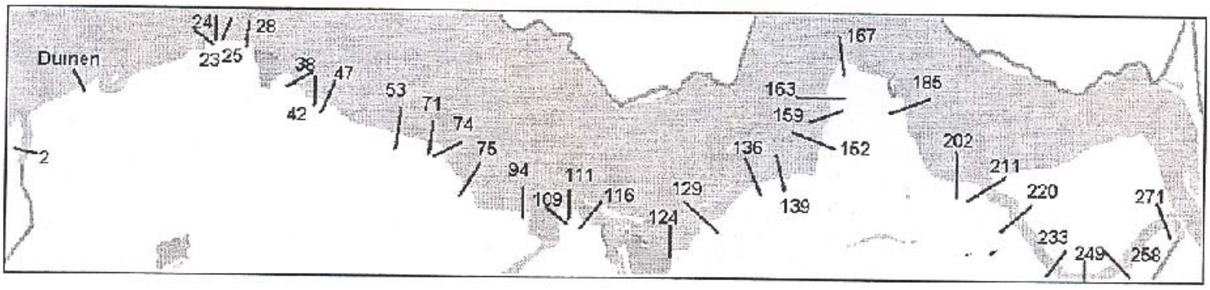



*Figure 2-1 Selected dike sections*

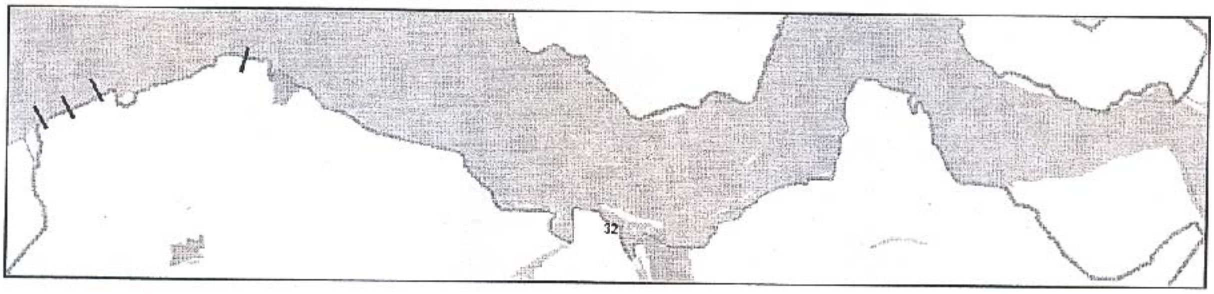



*Figure 2-2 Selected dune sections*

The 33 dike sections are numbered according to the following distances in kilometer :

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7002 | 7009 | 7023 | 7024 | 7025 | 7028 | 7038 | 7042 | 7047 | 7053 | 7071 | 7074 | 7075 | 7094 | 7109 |
| 85.2 | 82.4 | 71.7 | 71.6 | 71.2 | 70.1 | 65.1 | 64.1 | 63.6 | 61.9 | 57.6 | 56.9 | 55.7 | 51.7 | 47.4 |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7111 | 7116 | 7124 | 7129 | 7136 | 7139 | 7152 | 7159 | 7163 | 7167 | 7185 | 7202 | 7211 |
| 46.4 | 45.7 | 39 | 36.7 | 33.3 | 32 | 28.2 | 27.1 | 25.6 | 24.2 | 18.8 | 14.1 | 12.6 |

| | | | | |
|---|---|---|---|---|
| 7220 | 7233 | 7249 | 7258 | 7271 |
| 11.5 | 8.8 | 6.4 | 3.9 | 0.9 |

## 2.4 Adjustments of profiles

DHV has made several adjustments to the PC-Ring database during the calculations. Apart from the adjustment of the dike section selection, as discussed in the previous section, the dike profiles are adjusted to recently measured cross-sections of the water board . The adjustments of the profiles is further commented on in appendix A.

## 2.5 Schematization of coverings

Often more than one type of covering on a dike section is present in dike ring 32. PC-Ring is unable to perform calculations for more than type of covering for 1 dike section. In case more than one type of covering is present, VNK calculates all types individually and determines which one is governing (also in relation to concurrent design points). This governing covering is consequently accounted for when calculating the probability of flooding.

Only 1 type of covering per section is calculated in the calculations for dike ring 32:
Dike sections 7002 (024-Dp7), 7258 (074-Dp99) and 7271 (072-Dp69) for grass covering
Dike sections 7024 (006a-Dp11) and 7025 (006a-Dp15) for asphalt covering
The other sections for stone covering
The types of covering for which the various sections have been calculated are familiar to the water board.

There are 2 options for schematization in case more than one type of stone covering is present in 1 section:
Take the average along the total section
Take the worst part for a shorter length of the section
In order to be able to compare the results it should be possible to insert both values in the overall spreadsheet.

## 2.6 Schematization of dunes

It was agreed upon with engineering bureau VNK to perform calculations on the measured dune sections of 2004 (5 pieces) because these provide a conservative image (a 5-annual supplement is not planned until 2005). The choice of dune sections to be calculated is done based on the 2004 report of RIKZ. The choice is commented on in appendix A.

## 2.7 Schematization foreland of Saeftinghe

Shallow foreland is present in the land of Saeftinghe (6 most easterly located sections 7211 to 7271). This foreland is not accounted for in the calculations in this dike ring report. The boundary condition points (SWAN-points) are 100 meter from the coast (300m apart), so the influence of the foreland will be partially included in these. Foreland over 100 meter is of no use anyway.

## 2.8 Selection of profiles for sliding mechanism inner slope

Because calculating the sliding mechanism is an elaborate process, this calculation is not performed for all sections. The district water board has made a selection of 7 cross-section profiles (out of a series of 40 that were used for the testing) during the process of schematization. From these only 1 matches with one of the 33 selected dike sections. Therefore only one result will be calculated for the sliding mechanism of the inner slope.

## 2.9 Assessment of the water board

In accordance with the "Law on water retention 1996" the District Water Board Zeeuws-Vlaanderen reported on the condition of the embankments in dike ring 32 to the County Council of the Zeeland Province, at the end of 2000. This concerned the first report from a series of the 5-annual safety tests.

*Dikes*
The assessment of the water board for dike ring 32, based on the results of the first test, is summarized in table 2-2 for the selected sections. In this table the Ht_score represents the score for overflow and wave run-up, STPI_score represents the score for bursting and piping, STBI_score represents the score for stability of the inner slope. In case of an even score, one can assume that the overflow and wave run-up mechanism is governing. For the covering damage and erosion body of a dike mechanism the result of the 'old' testing is not provided. The calculated probabilities of failure for this mechanism are discussed during consults with the water board and related to the temporary results of the 'new' testing (see section 4).

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7002 | 7009 | 7023 | 7024 | 7025 | 7028 | 7038 | 7042 | 7047 | 7053 | 7071 | 7074 | 7075 | 7094 | 7109 |
| suf | insuf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf |
| suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | insuf |
| suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7111 | 7116 | 7124 | 7129 | 7136 | 7139 | 7152 | 7159 | 7163 | 7167 | 7185 | 7202 | 7211 |
| insuf | insuf | insuf | suf | suf | suf | insuf | suf | suf | suf | suf | suf | insuf |
| insuf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf |
| suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf | suf |

| | | | | |
|---|---|---|---|---|
| 7220 | 7233 | 7249 | 7258 | 7271 |
| suf | suf | suf | suf | suf |
| insuf | insuf | suf | suf | insuf |
| suf | suf | suf | suf | suf |

*Table 2-2 Assessment of the water board for dikes in dike ring 32 (the first row shows the section number, the second row the Ht_score which represents the score for overflow and wave run-up, the third row shows the STPI_score which represents the score for bursting and piping, and the fourth row the STBI_score which represents the score for stability of the inner slope. Suf stands for sufficient and Insuf for insufficient.*

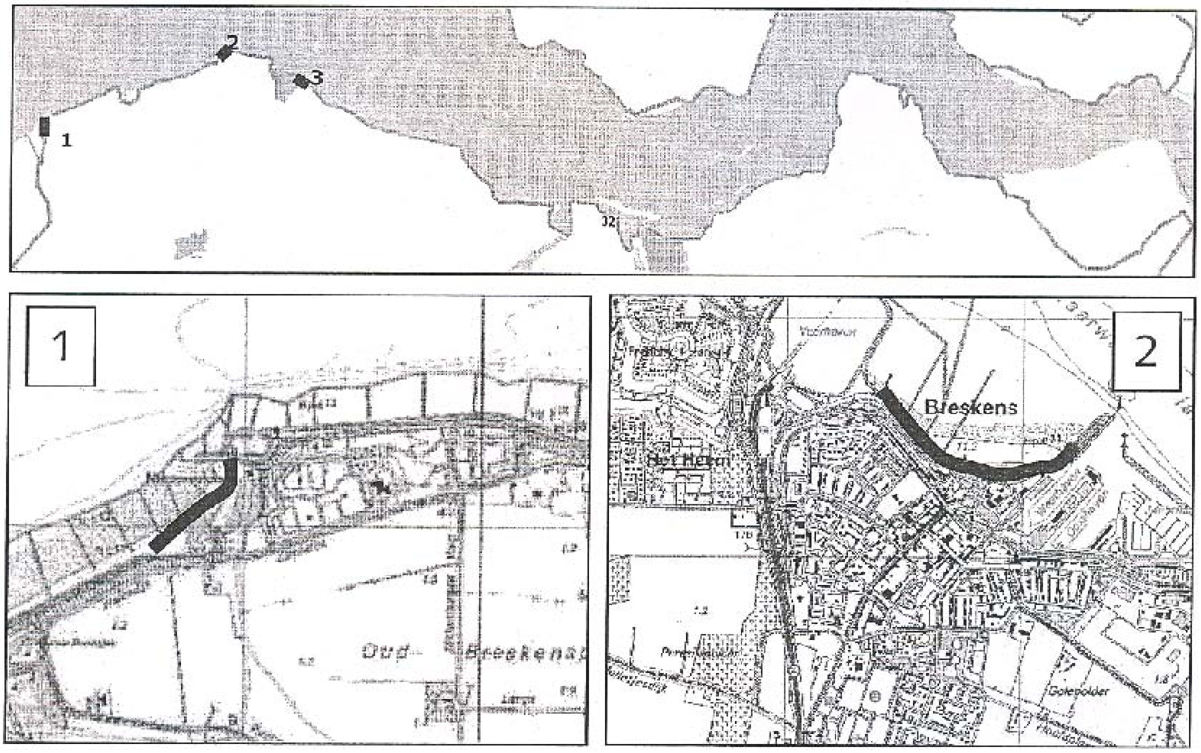
*Dunes*
Recent research established that one has to reckon with heavier wave action than was assumed so far along the Dutch coast. This could imply that embankments of Zeeuws-Vlaanderen no longer comply with the legal requirements. The calculated weak spots, based on the given boundary conditions, provide a true representation of the locations with the greatest strength deficiencies. These are determined by the water board and the assessment of the water board, based on unambiguity in boundary condition sections and the shape of the coastal sections, leads to the following strength deficiencies (see figure 2-3).

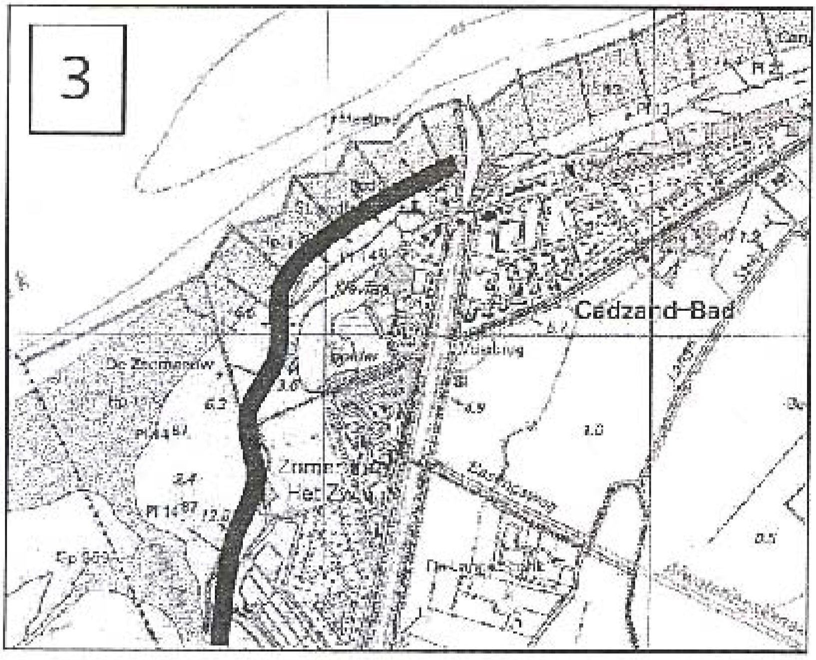
- The dune area of Cadzand, west of the outlet with the adjoining sea dike of the Kievitspolder East (coastal length 940m, test crown height deficiency 2.00m) (Figure 2-4, top left).
- The sea dike of the Jong Breskenpolder between Nieuwe Sluis and the lighthouse (coastal length 1060m, test crown height deficiency 0.50 to 1.00m) (Figure 2-4, top right).

- The addition to the artificial dune in Breskens at the Veerhaven (coastal length 470m) (Figure 2-4, bottom left).
- 4 junctions of constructions of sea dikes and/or dune toe defense on the adjacent dune area (coastal length 600m at Schoneveld, the Kruishoofd and Nieuwe Sluis).
- The slopes of stone on sea dikes and connection constructions (coastal length 8100m, tested under Project Zeeweringen).

1. The dune area of Cadzand, west of the outlet with the adjoining sea dike of the Kievitspolder East.
2. The sea dike of the Jong Breskenpolder between Nieuwe Sluis and the lighthouse
3. The addition of the artificial dune in Breskens at the Veerhaven

*Figure 2-3 Weak spots according to the assessment of the water board*




3
Cadzand-Bad
Het Zwin

# 3. Level III probability of overtopping calculation dike ring area 32

The probability of a dike failure due to overtopping is considered of dike ring 32. Overtopping is assumed to take place due to extreme sea levels, extreme river discharge or a coincidence of both. The levels of the river and sea are modelled as random variables and the water level along a dike section is obtained as a nonlinear function of these random variables. The height of the dike is assumed to have spatial uncertainty variation. A Monte Carlo simulation based approach is considered for the reliability analysis of the dike. The computation of the local water level involves calculation through a computationally intensive hydrodynamic model and is carried out using commercially available software. Efforts to reduce computational time in the reliability analysis are explored through the use of importance sampling technique. Further reduction in computational efforts is achieved by adopting a novel response surface based method. This strategy involves using available response database for the local water levels corresponding to observed boundary conditions. In the importance sampling based Monte Carlo simulations carried out in this study, the local water levels are computed by interpolating from the available response database rather than using the hydrodynamic model. The proposed method is observed to bring about significant reduction in computational efforts.

## *3.1 Introduction*

The reliability analysis of a dike at a lower reach of the tidal Scheldt river is considered. In this study, it is assumed that dike failure occurs due to overtopping only. Overtopping of the dike is assumed to take place due to (a) extreme sea levels, (b) extreme river discharge and (c) coincidence of both of the above extremal events. This has been illustrated by the schematic diagram in Figure 3-1. The stochastic nature of the input variables, in this case, the extreme levels of the sea and river discharge and the time of their occurrence, implies the necessity for using probabilistic methods for the analysis.

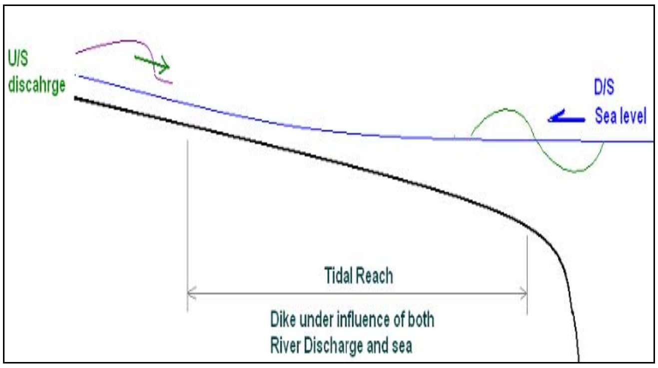



*Figure 3-1:Dike on tidal reach of a river subjected to both discharge and sea level variations.*

Use of Monte Carlo simulations for reliability analysis lead to accurate estimates of the failure probabilities. Here, the basic steps involved are (i) digital generation of an ensemble of loading conditions that obey specified probabilistic laws, (ii) treatment of each realisation of the problem using deterministic procedures, and (iii) statistical processing of the ensemble of sample solutions for the problem, leading to estimates of the failure probability. Thus, in principle, the method is applicable to any problem where it is possible to digitally generate an ensemble of loading conditions and deterministic solution methods for a sample problem are available. The method, however, can be computationally intensive.

For the river dike problem considered in this study, the water levels along the dike segment are computed using a hydrodynamic model. This requires nontrivial computational effort. In Monte Carlo simulations, repeated analysis of the hydrodynamic model for each realization of the random boundaries makes Monte Carlo simulations very expensive. This implies that there is a need to explore the use of alternative less computationally intensive techniques for reliability analysis. One such method, the importance sampling technique, is used in the study carried out in this paper. The method is applied to estimate the two-days overflowing probability of a dike of length 80 km along the Western Scheldt, Province of Zeeland, The Netherlands. Three variables, namely, the dike height, sea level and Scheldt river discharge are considered as randomly distributed variables. The limit state is idealized as a function of these three mutually independent random variables. Probability distributions for these three random variables are constructed from analysis of data based on observations from the site (Pandey *et al*., 2003). Calculations through the hydrodynamic model are carried out with a commercially available software (SOBEK). Additionally, the use of a response database in lieu of the hydrodynamic model for calculating the water level along the dike is explored (Dahal, 2005).

## 3.2 Importance sampling

First, a brief review of the method of importance sampling is presented. Assume that the uncertainties associated with the problem are represented through a vector of random variables X. The performance function is given by g(X), such that, g(X)<0 indicates failure, g(X)>0 indicates safe region and g(X)= 0 denotes the limit state. Using Monte Carlo simulations, an estimate of the failure probability, $P_f$, is obtained as

$$P_f = \int_{-\infty}^{\infty} I[g(\mathbf{X}) \le 0] p_{\mathbf{X}}(\mathbf{x}) d\mathbf{x} = \frac{1}{N} \sum_{i=1}^{N} I[g_i(\mathbf{X} \le 0)] . \quad (1)$$

Here, *I*[.] is an indicator function which takes values of unity when $g(X)\le 0$ and zero otherwise. The minimum number of samples required for target coefficient of variation $V(P_f)$ is given by

$$N > \frac{1}{V(P_f)^2}\left(\frac{1}{P_f} - 1\right). \quad (2)$$

Thus, it follows that to reduce the estimate of variance to acceptable levels, for low failure probability levels, sample size, N, needs to be large. This has led to the development of a number of variance reduction techniques (Kahn, 1956). In implementing the importance sampling technique, Eq.(1) is rewritten as

$$P_f = \int_{-\infty}^{\infty} \frac{I[g(\mathbf{X}) \le 0] p_{\mathrm{X}}(\mathbf{x})}{h_{\mathbf{Y}}(\mathbf{x})} h_{\mathbf{Y}}(\mathbf{x}) \mathrm{d}\mathbf{x} , \quad (3)$$

and an estimate of the failure probability is obtained as

$$P_f = \frac{1}{N} \sum_{i=1}^{N} \frac{I[g_i(\mathbf{X}) \le 0]}{h_{\mathbf{Y}}^{(i)}(\mathbf{X})} p_{\mathbf{X}}^{(i)}(\mathbf{X}). \quad (4)$$

Procedures that estimate $P_f$ with specifically chosen $h_Y(x)$ as sampling density functions are called important sampling procedures and $h_Y(x)$ is called the importance sampling function. Here, the sampling is done in the $h_Y(x)$ region rather than $p_X(x)$. A major step in implementing the procedure lies in choosing an appropriate importance sampling probability density function $h_Y(x)$. The importance sampling density function could be Gaussian or non-Gaussian and is centred over an appropriately defined multi-dimensional region covering the region of likelihood around the design point (Shinozuka, 1983). Considering non-Gaussian importance sampling functions, however, lead to difficulties when the random variables are mutually correlated. These problems can be circumvented by transforming the problem to the standard normal space and constructing Gaussian importance sampling functions (Schueller and Stix, 1987). This is especially true when the location of the design point is not known *apriori* (Bucher, 1988).

## 3.3 Model setup

The overflowing of the dike triggers erosion in inner slope, breach starts to grow which leads to the ultimate failure of the dike. Thus, in the study reported in this paper, failure is defined as the overtopping of the dike and the performance function is taken to be of the form

$$g(h_k, h_s, Q_r) = h_k - h(h_s, Q_r), \tag{5}$$

where, $h_k$ is crest height of dike and $h$ is the local water level obtained as a function of $h_s$ and $Q_r$, representing, respectively, the extreme sea-level and extreme river water discharge.

The relationship between the local water level and the boundary parameters $h_s$ and $Q_r$ is through a nonlinear hydrodynamic model. The parameters $h_k$, $h_s$ and $h_r$ are modeled as mutually independent, random variables. The extreme values of the sea-water levels and the river discharges are assumed to be non-Gaussian random variables. The dike crest height along the entire stretch of the dike is modeled as a Gaussian random process with a specified auto-correlation function. The length of the dike is discretized into smaller segments. The dike crest height is assumed to be constant throughout each segment and is modeled as a Gaussian random variable. The probability of overtopping is calculated for each segment using the performance function in Eq.(5). The dike segments are assumed to be in series and the bounds on the failure probability estimates for the series system are obtained (Cornell, 1967).

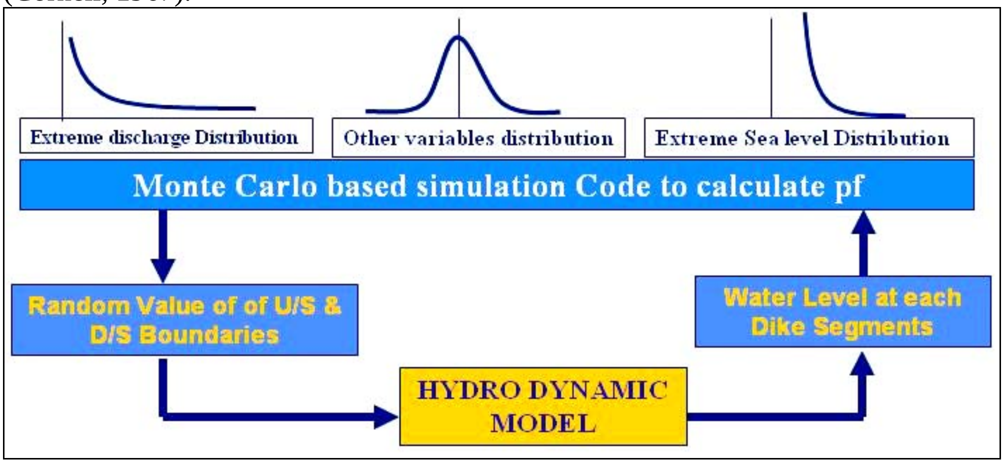



*Figure 3-2: Probabilistic loops through hydrodynamic model for stochastic simulation*

During Monte Carlo simulations, first, an ensemble for the random variables are generated and deterministic calculations are carried out, using the hydrodynamic model is necessary, for each realization. Figure 3-2 illustrates a schematic diagram of the simulation procedure and loop through hydrodynamic model. The computation time for one sample realization through the hydrodynamic model is non-trivial. An importance sampling based Monte Carlo approach is adopted for estimating the probability of dike overtopping.

## 3.4 Response database

Despite adopting an importance sampling strategy, computation of the water level at the dike section requires significant computational effort. In this study, we explore the possibility of further reduction in computational time using a response database. This is possible if there exists a database of observations of water levels corresponding to different boundary conditions. During Monte Carlo simulations, first, the program searches into the database for the set of boundary conditions which have the closest correspondence to the particular realization. The local water level is then calculated by interpolation. This strategy for computing the river water level ensures (a) that the costly computations through the hydrodynamic model can be avoided, and (b) the database of observations already existing is of use. Figure 4 illustrates a schematic framework for the use of response database instead of probabilistic loop in this study.

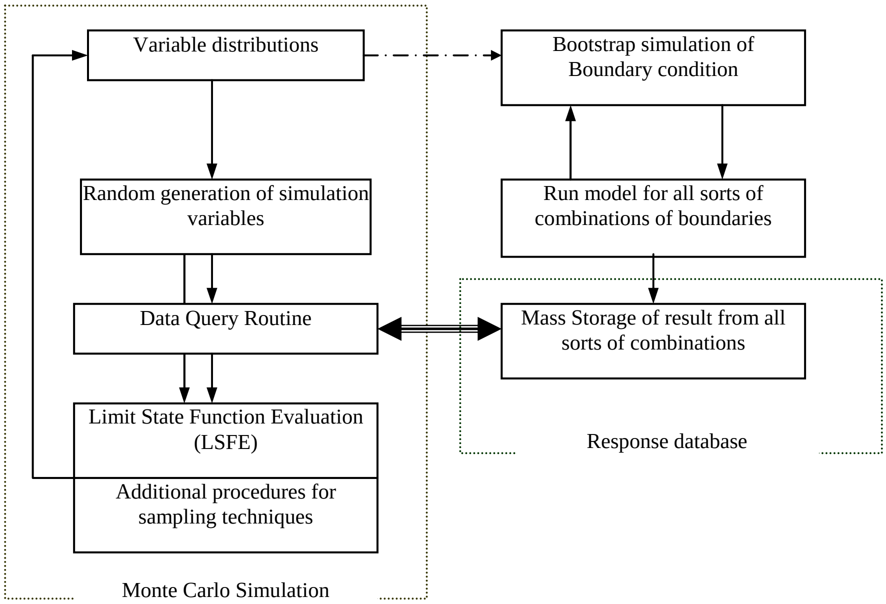



*Figure 3-3: Block Diagram of conceptual framework for response database used in Monte Carlo simulation*

The method of estimating the river water levels along the dike sections through interpolations from the response database is somewhat, in principle, similar to the response surface method. It must be noted that the response surface based methods are used to develop approximating functions that surrogate for long running computer codes (Khuri and Cornell, 1987). In this study, the interpolation functions used to estimate the water levels along the dike sections can be viewed as response surface functions for the particular realization.

### 3.5 Simulation details and results

The overflowing failure mechanism of dike ring No 30, 31 from Western Scheldt, Province of Zeeland, is studied. The water levels of North Sea recorded at station Vlissingen were used to construct probability distribution functions of downstream levels. The data analysed are daily records from 1863 to 2004; see figure 3-4. Bestfit package was used to rank the distribution and find the parameters based on method of moments. A Pareto distribution was observed to lead to a realistic description for the observed data; see figure 3-5.

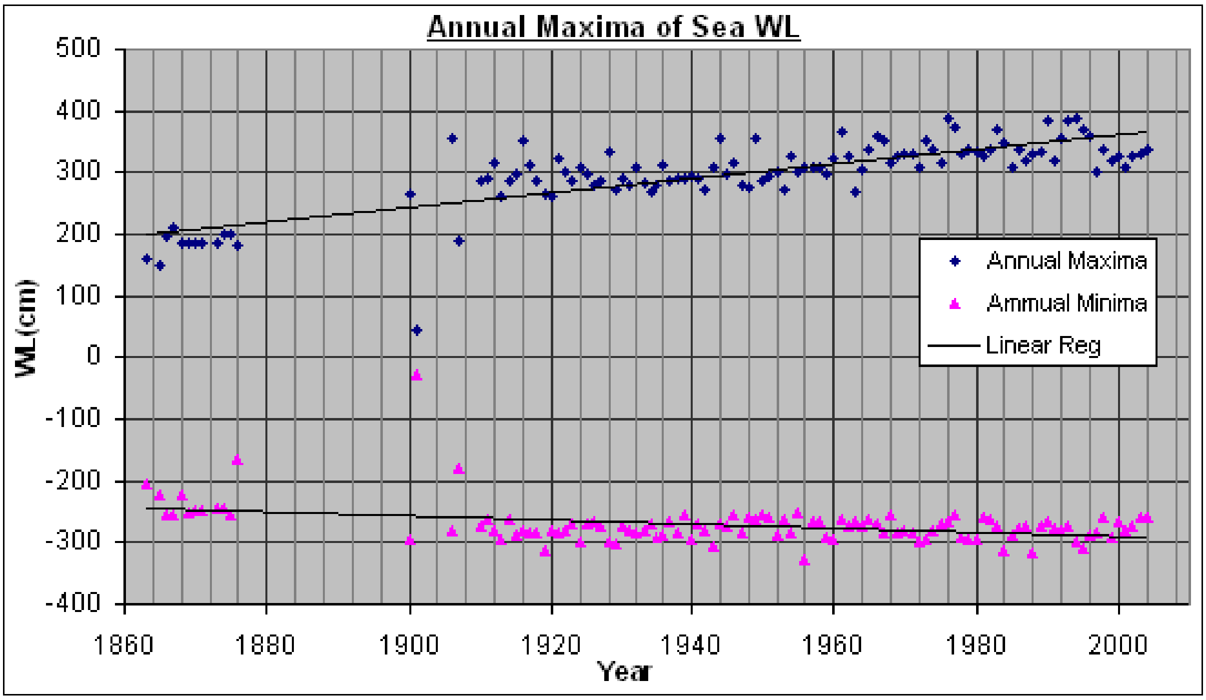



*Figure 3-4:Annual maxima and minima of Sea Water level at Vlissingen, Western Scheldt*

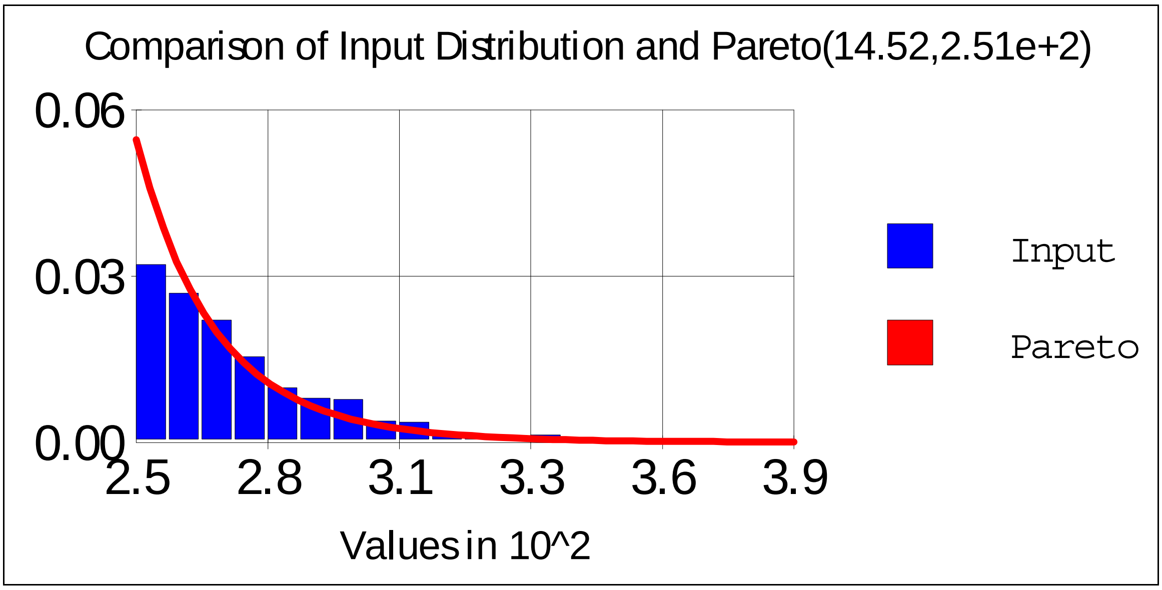



*Figure 3-5: Pareto distribution representing sea level flactuation*

A family of Pareto distributions were obtained depending on the threshold level selected while constructing the Pareto distributing using peak over threshold (POT) analysis; see figure 3-6

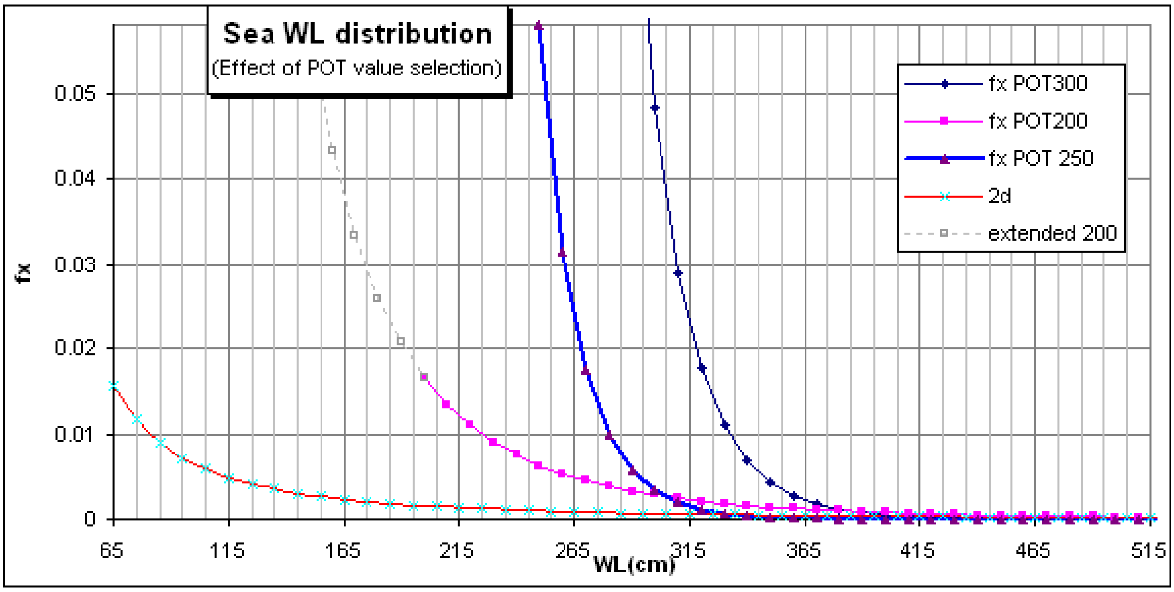



*Figure 3-6: Effect of Choice of POT value on distribution*

Parameters of exponential distribution, calculated by Bestfit, are based on zero position of the location parameter. For corresponding 2 days maxima, POT analysis is carried out by changing location and scale parameters successively. Figure 3-7 illustrates the effect of changing the threshold during POT analysis, on the location and scale parameters.

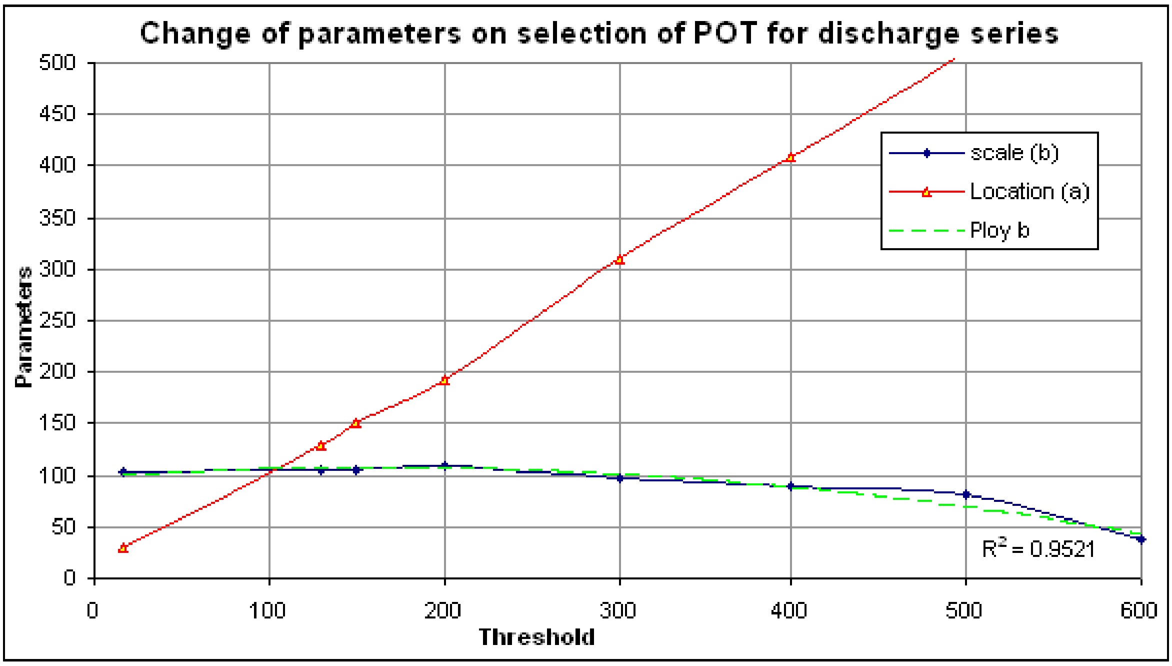



*Figure 3-7:Change in location and scale parameter with different POT values*

The dike length is discretized into segments such that, each segment could be considered independent of each other. The length of each segment was taken equal to the correlation length of the random process modelling the spatial randomness of the dike height. The autocorrelation function considered is as follows:

$$\rho_{X,X+L}(L) = e^{-\left(\frac{L}{D}\right)^2 \frac{\pi}{4}}, \qquad (6)$$

where, $D$ is the fluctuation scale given by

$$D = \int_0^\infty \rho_{x,x+L}(L)dL\,. \qquad (7)$$

Figure 3-8 illustrates the auto-correlation function for the dike height. The fluctuation scale is found to be 3532 m and the dike segments were taken to be of length 3500m.

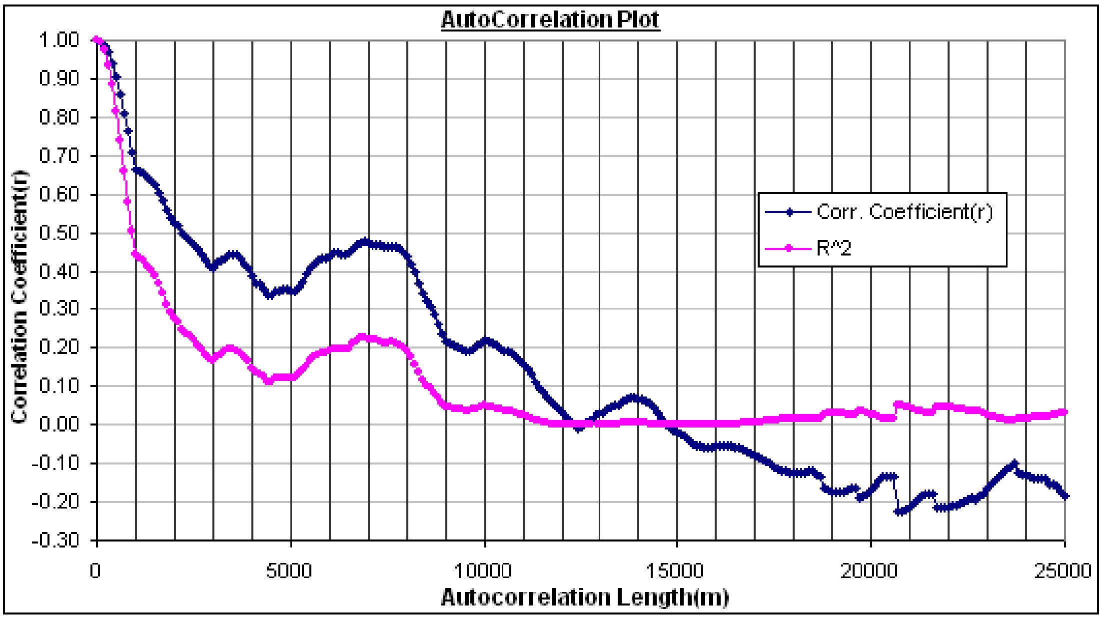



*Figure 3-8: Autocorrelation for dike height*

A new sea level is assumed to take place every 2 days (48 hours). The typical travel time of a flood wave along the length of the dike is approximately one hour. Thus, the river water levels, along the dike, are measured every hour. Calculations through the hydrodynamic model are carried out using SOBEK. A node is selected in each dike segment in SOBEK 1D schematisation.

For the purpose of illustration, the response database was built up using Sobek for a set of observed random boundary conditions. In practice, it is expected that the response database would be available. Importance sampling is subsequently carried out for estimating the failure probability for each dike segment. All the dike segments are assumed to be in series configuration and Cornell's bounds are computed for the system reliability. These bounds are observed to be $2.56 \times 10^{-7}$ and $8.75 \times 10^{8}$. The use of importance sampling in reliability analysis of the dike reveal that the sample size required is considerably less than full scale Monte Carlo simulations.

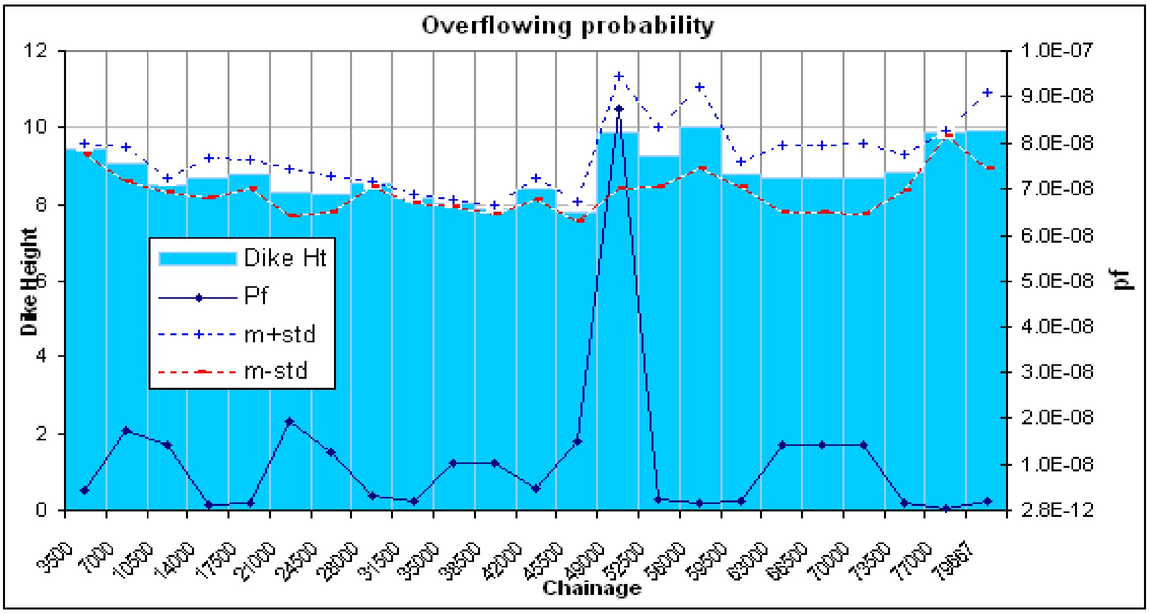



*Figure 3-9: Overflow probability of the 80km long dike*

## 3.6 Concluding Remarks

The probability of overtopping of the 80 km long dike, due to the occurrence of extreme sea levels and river discharge, either concurrently or otherwise, is estimated. The reliability computations are carried out using importance sampling based Monte Carlo simulations. A novel response surface based method, based on already existing database, is adopted while computing the performance functions. The procedure shows promise in significantly reducing the computational effort.

# 4. Probability of flooding calculation dike ring area 32

This section describes the approach and results of the performed calculations for determining the probability of flooding. With the presentation of the results, a distinction is made between contributions to the probability of flooding of dunes, dike sections and structures, and of different failure mechanisms within them. The calculated results are compared with the judgment of the water board. The computer model used to calculate the probabilities of flooding for dike ring 32 is PC-Ring version 4.3 (February 2005). Calculations have been made by DHV with checks by VNK and assessments by WZE. It proved to be difficult to perform good calculations of the probability of flooding, due to the variation in loads and the complexity of the dike profiles.

## 4.1 Approach and assumptions of the calculations

### 4.1.1 General

The calculations of the probability of flooding of the dike ring and the probability of failure of dike section and dunes have been performed using the computer program PC-Ring (version 4.3). Input for this program are the schematization and the data as discussed in chapter 2. The program calculates a probability of failure for each dike section, based on the contributions of each separate failure mechanism, and eventually the total probability of flooding for the entire dike ring.
Additionally the program provides insight in to what amount the various variables (e.g. the length of seepage present or the height of the dike) contribute to the calculated probability of failure. This is an important factor for conducting sensitivity analyses. The reliability index (beta) is often used for calculating with probabilities. The probability of failure is a function of this reliability index. PC-Ring also calculates with betas.

The probabilities of failure of structures are calculated using different procedures without PC-Ring. The calculated probabilities of failure per structure do form input for PC-Ring for calculating the probability of flooding of the entire dike ring based on the contributions of the distinguished dike sections and structures.

Statistic data of wind and water level are used for calculating the probability of flooding of dike sections. Based on these data the load models are defined, which are implemented in PC-Ring. The load models in question are adjusted to the valid hydraulic boundary conditions.

Please note that a clear difference has to be made between probability of exceedance, probability of failure and probability of flooding. The probability of exceedance is the probability that the water level at a dike section reaches higher than the test level. This is used in the present safety approach. The probability of failure is the probability that a dike section actually yields to one the failure mechanisms. The probability of flooding is the probability that the dike ring floods as a result of failure of a dike section on one or several places. A comparison between these latter two probabilities and the probability of exceedance is not possible. The fact that in this report weak links are indicated when the probability of failure of that specific link is greater than 1/1250 does not relate to the fact that the probability of exceedance of this area is 1/1250 as well.

### 4.1.2 Failure mechanism dikes

For calculating the probabilities of failure of dikes, the hydraulic load of water levels and waves is confronted with the relevant characteristics of the embankment that are governing for the strength of the embankment. Both the load and the characteristics of the embankment are described in terms of probability distributions. Uncertainties in the input data are accounted for using these probability distributions.

Calculations of the probability of failure of a dike are based on the following failure mechanisms:

- Overflow and wave overtopping
- Covering damage and erosion body of the dike
- Bursting/piping
- Sliding inner slope

**Overflow and wave overtopping**
With this failure mechanism the dike fails because large amounts of water run or sweep over the dike. In case of offshore wind of otherwise very small wave heights, the yielding is described by the failure mechanism overflow. In other cases the yielding is described by the failure mechanism wave overtopping.

**Covering damage and erosion body of the dike**
With this failure mechanism the dike fails because the covering is damaged by wave action first, after which the cross-section of the dike core is diminished by erosion.

**Bursting/piping**
With this failure mechanism the dike fails because the sand is washed away from underneath the dike. The sealing layer, if present, will first burst due to the pressure of the water. Consequently so-called "pipes" can occur, causing the sand to be washed away and the dike to collapse.

**Sliding inner slope**
With this failure mechanism the dike fails because a part of the dike becomes unstable as a result of high water levels for a long period of time and consequently slides.

The possible failure mechanisms liquid settlement, buoyancy, sliding of the foreland, sliding of the outer slope, micro-instability and weakening are not taken into account because these failure mechanisms do not directly result in flooding. An assessment model is used per failure mechanism in order to be able to compare loads and strengths or otherwise to be able to calculate the probability of failure for the failure mechanism in question.

### 4.1.3 Failure mechanisms structures

For determining the probabilities of failure for structures, the exceedance frequency line of water levels is confronted with the strength of the embankment. For the structures, the uncertainties in the input data are also accounted for explicitly. For determining the probability of failure of a structure, the following failure mechanisms are accounted for:

- Overflow and wave overtopping
- Not-closing of the closing elements
- Constructive failure

The failure mechanisms are briefly described below.

**Overflow and wave overtopping**
With the failure mechanism overflow and wave overtopping the structure fails because water runs over the structure. The assessment of the structure is based on a comparison of the retaining height in relation to the exceedance frequency line of the outside water level.

**Not-closing of the closing elements**
With the failure mechanism not-closing of closing elements the structure fails as a result of the closing elements not being closed off in good time. The assessment of the structure is based on a comparison between the exceedance frequency line of the outside water level and the "open retaining level" (OKP), taking into account the probability of the not-closing of the closing elements.

For determining the probability of not-closing of the closing elements the VNK-method follows the Guideline Structures 2003. This guideline distinguishes four main causes of failure:

- Failure of the high water warning system: failure water level registration, failure alarm, etc.
- Failure of mobilization: operating personnel is not present at the retaining structure in time.
- Failure due to operating errors: faulty or omitted acts.
- Technical failure of the closing elements: motion device fails, etc.

**Constructive failure**
With the failure mechanism constructive failure the structure fails as a result of loss of strength or stability of (parts of) the structure. The assessment of the structure is based on a consideration of constructive strength and stability of the structure in relation to the loads when retaining high water. For this assessment the following mechanisms are applicable:

- Constructive failure of the retaining devices resulting from drop load
- Constructive failure of the concrete construction
- Constructive failure of the foundation
- Chance of loss of stability due to instability of the bottom protection
- Failure due to loss of stability as a result of a collision
- Failure due to general loss of stability
- Failure due to under or rear seepage (piping)

**Method of assessment**
Within the project VNK a method has been developed for several types of structures to calculate the probability of flooding for different failure mechanisms. It concerns the following types of structures: navigation locks, discharge sluices, cuttings, tunnels and pumping stations.

The failure of a structure by overflow and wave overtopping or not-closing of the closing elements does not inevitably result in the arising of a breach in the embankment and with that the flooding of a dike ring area. The water flowing in can often be stored in the adjacent water system behind the structures that are linked to the inland water, without resulting in flooding. Also the structures can often handle large flows without loss of stability. Therefore the initially calculated probabilities of failure as a result of overflow and wave overtopping and not-closing of the closing elements respectively are tightened in the assessment system to probabilities where the start of a breach occurs. These are smaller probabilities by definition. This tightening requires extra effort and is thus only executed when the first approach results in relative large probabilities compared to the existing standard frequency for design water levels.
With the mechanism constructive failure, it is assumed that the stability is directly lost when breaching occurs. The corresponding probability of failure is therefore considered the probability of breaching.

### 4.1.4 Probability of flooding of the dike ring area

The probability of flooding of a dike ring area is made up of the calculated probabilities of failure of the dikes, dunes and structures in question. First the probability of failure is determined per dike section of structure based on the contributions of the various failure mechanisms. Consequently the probability contributions of the various dike sections and structures are combined into the probability of flooding of the dike ring. With combining the various contributions, possible dependencies in probabilities of failure of nearby dike sections are accounted for.

## 4.2 Process description

- The collecting of data on dike ring 32 is done by the water board in cooperation with VNK. The quality of the data is checked by both VNK (roughly) and the Bouwdienst (during the conversion of the data from the overall spreadsheet to the database). The result of this is recorded in various checklists and reports overall spreadsheet dike ring 32.
- With executing the first calculations for dikes and dunes, several adjustments to the PC-Ring database were performed. The greatest adjustments concerned the selection of dike sections (see section 2.3) and the schematization of the dike profiles. With the selection of dike

sections, 33 dike sections and 4 dune sections were chosen out of 287 sections that were schematized by the water board. With the schematization of the profiles, the schematized profiles (done by the water board) in the PC-Ring database were compared with recently measured cross-sections of the water board. All profiles were schematized again because anomalies occurred between the measured and the schematized profiles.

- DHV both did the initial calculation and a further analysis for dikes and dunes in principle. With the calculations one ran into many difficulties concerning amongst others the schematization, the complexity of the dike profiles, the variation in loads and the programming, due to which doing good calculations for this dike ring turned out to be difficult.
- VNK checked and corrected all DHV's calculation for the dikes together with TNO. This resulted in the fact that a probability of failure has been calculated for (almost) all mechanism for the selected sections.
- The calculated probabilities of failure are discussed with the water board. VNK processed the results of these discussions in this dike ring report.
- The structures are assessed by DHV. The results are tested and checked by VNK and the water board.
- The MproStab calculations are performed by DHV and checked by GeoDelft.

## 4.3 Results of the calculations of the probability of flooding

### 4.3.1 Introduction

In this section an insight is provided in the calculated probabilities of failure for dike ring 32. It concerns preliminary results, since the results have not been analysed thoroughly. These preliminary results have been discussed with the water board. Because it concerns preliminary results, a so-called reference sum is not yet presented for dike ring 32.

### 4.3.2 First results per dike section

The (preliminary) results per dike section in beta are provided in table 4-1. These results are discussed with the water board (see section 4.3.4). As a result of this discussion, it was concluded that a number of sections can be left out of consideration for now. These are results that are unidentifiable for the water board and have to be analysed further or weak spots that are nominated to be improved. These sections are shaded grey in the table.

| 7002 | 7009 | 7023 | 7024 | 7025 | 7028 | 7038 | 7042 | 7047 | 7053 | 7071 | 7074 | 7075 | 7094 | 7109 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.0 | 6.6 | 5.7 | 5.6 | 6.0 | 7.4 | 5.2 | 5.8 | 5.8 | 5.7 | 5.4 | 4.8 | 4.9 | 5.0 | 5.5 |
| 6.7 | 6.5 | 11.3 | 11 | 11 | 7.3 | 6.7 | 6.3 | 9.8 | 10 | 6.3 | 6.1 | 6.2 | 7.0 | 6.4 |
| 3.4 | 3.7 | 5.0 | 4.6 | 9 | 2.4 | | 6.2 | 9.3 | 7.6 | 5.1 | 7.0 | 7.8 | 7.8 | 6.0 |

| 7111 | 7116 | 7124 | 7129 | 7136 | 7139 | 7152 | 7159 | 7163 | 7167 | 7185 | 7202 | 7211 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.2 | 4.9 | 3.9 | 5.0 | 5.6 | 4.9 | 4.9 | 4.4 | 4.8 | 3.0 | 4.1 | 4.8 | 3.9 |
| 6.7 | 6.6 | 7.1 | 6.3 | 8.0 | 6.1 | 5.3 | 5.2 | 4.5 | 7.5 | 4.1 | 4.8 | 4.5 |
| 5.4 | 6.5 | 8.5 | 5.3 | 5.7 | 37 | 6.8 | 5.2 | 14 | 13 | 36 | 6.1 | 14 |

| 7220 | 7233 | 7249 | 7258 | 7271 |
|---|---|---|---|---|
| 4.5 | 4.8 | 4.5 | 4.5 | 4.6 |
| 5.7 | 4.6 | 5.2 | 6.0 | 6.4 |
| 37 | 8.9 | 37 | 1.8 | 2.2 |

*Table 4-1 Reliability indices (preliminary) per section (in first row) calculated by VNK based on the following failure mechanisms:*

- Second row: Overflow and wave overtopping
- Third row: Bursting/piping
- Fourth row: Covering damage

The reliability index of section 7249 for the mechanism sliding inner slope has been calculated as 2.1. Indices for dune erosion has been calculated for the following sections 7008, 7010 and 7013 with beta values equal to 4.4, 4.4 and 4.9.

### 4.3.3 Sliding inner slope

7 Profiles have been selected for calculating the probabilities of failure for the failure mechanism sliding. DHV calculated these 7 profiles with MproStab. Only 1 profile is part of the 33 selected sections for the PC-Ring calculations (EMMA118 belongs to section 7249 (076-dp124)). A result for the mechanism sliding inner slope is incorporated in table 3-2 for only this section. An overview of the calculated safety factors and reliability indices at different water levels for all 7 sections is provided in table 4-2.

DHV consequently considered with which of the profiles from table 4-2 each of the 33 sections matches best. A profile is linked to each selected section and a probability of failure has been calculated for each section using PC-Ring. Since the used method is not correct, the results are not displayed here. The coupling is based on height of the crown, gradient of the inner slope, MHW and thickness of the covering layer, but doesn't account for the structure of the soil. The coupling of the sections and the profiles does thus not match the routes for which the profiles are deemed to be representative according to the water board.

| Bestandsnaam | Dijkvaknummer | MStab | MproStab | | |
|---|---|---|---|---|---|
| | | | MHW<br>Fn / β | MHW −0,5m<br>Fn / β | MHW −1,0m<br>Fn / β |
| TIENH15.STI | 1320007012 | 0,97 | 1,19 / 1,53 | 1,22 / 1,65 | 1,24 / 1,74 |
| NISLU17.STI | 1320007014 | 1,06 | 1,31/ 2,53 | 1,33 / 2,68 | 1,36 / 2,80 |
| HOOFD37.STI | 1320007052 t/m 1320007063 | 0,94 | 1,21 / 1,92 | 1,24 / 2,13 | 1,28 / 2,29 |
| PAULIN4.STI | 1320007079 | 0,80 | 1,04 / 0,97 | 1,08 / 1,27 | 1,13 / 1,55 |
| WIL206A.STI | 1320007204 t/m 1320007208 | 1,10 | 1,31/ 2,43 | 1,34 / 2,53 | 1,36 / 2,62 |
| ALS166B.STI | 1320007226 | 0,56 | 0,93 / 0,21 | 0,96 / 0,42 | 0,94 / 0,29 |
| EMMA118.STI | 1320007249 t/m 1320007252 | 0,90 | 1,17 / 1,874 | 1,19 / 1,95 | 1,23 / 2,18 |

*Table 4-2 Comparison safety factors according to Bishop from MStab and MproStab (results by VNK)*

When considering this latter, next to section 7249 (076-dp124) DHV made the right coupling for sections 7109 (123-dp26), 7111 (122-dp16), 7116 (121a-dp9), 7233 (078-dp148), 7258 (074-dp99) and 7271 (072-dp69). For the latter three sections the MHW (almost) matches with the MHW of the representative profile. This is not the case for the first three. The probabilities of failure that DHV calculated for these sections are provided in table 4-3.

| Dijkvak | Bèta afschuiven | Faalkans afschuiven |
|---|---|---|
| 7109 – 123-Dp26 | 1,905 | 1/35 |
| 7111 – 122-Dp16 | 1,906 | 1/35 |
| 7116 – 121a-Dp9 | 1,891 | 1/34 |
| 7233 – 078-Dp148 | 2,140 | 1/62 |
| 7258 – 074-Dp99 | 2,159 | 1/65 |
| 7271 – 072-Dp69 | 2,135 | 1/61 |

*Table 4-3 Reliability index Beta and the failure probability for the mechanism sliding (DHV results)*

These results provide an indication of the probabilities of failure to be expected. Before the results are incorporated in the calculation of the probability of failure for dike ring 32, it should be checked whether coupling of the dike sections from PC-Ring to representative profiles with another MHW is possible.

### 4.3.4 Feedback results per section to water board

The results of the calculations per dike section are discussed with the water board. An overview of its findings per mechanism is given below. The results are compared with the results of the testing in 2000 (table 2-2) and the preliminary results of the 2005 testing as far as these are available. As a result of this, it is concluded that a number of results should left out of consideration for the time being (these results are shaded grey in table 4-1).

*Overtopping and wave overrun*

- Dike section 7167 (097-dp290), Molenpolder, has a relative bad score for the mechanism overtopping/wave overrun (beta is 3,03). This result is not recognisable for the water board. Possibly the sandbank ahead is not schematised correctly (this is no foreland), due to which too little wave reduction is accounted for. Other cause could be the calculated profile. A further analysis of required here.

→ The water board thinks the present result should not be considered in the calculations of the probability of flooding of the dike ring, because it doesn't recognise the results.

- For sections 7009 (020-dp16), 7111 (122-dp16), 7116 (121a-dp9), 7124 (113-dp87), 7167 (097-dp290), 7211 (083a-dp186) and 7233 (078-dp148) the water board separately indicated that these score well for height in the (preliminary) results of the 2005 testing. A number of these sections scored unsatisfactory in the 2000 testing (see table 2-2).
- The section 7152 (100a-dp330) scored unsatisfactory in the 2000 testing, but is strong according to the VNK calculations. If this section still appears to be unsatisfactory in the new testing, the result of VNK will have to be examined further.

*Bursting and piping*

- The results of VNK do not indicate weak spots for the mechanism bursting/piping.
- A number of sections scored unsatisfactory with the first testing. No improvement works related to the phenomenon bursting/piping have been executed since. Works have been executed to drainage and better soil research has been done. For now, a number of sections do not yet score satisfactory for this mechanism with the second testing.
- For the sections 7109 (123-dp26), 7111 (122-dp16), 7220 (081a-dp175) the water board has separately indicated that they score well for the mechanism bursting and piping in the (preliminary results) of the 2005 testing. The section 7223 (078-dp148) scored unsatisfactory in the 2000 testing. Both sections are strong according to the VNK calculations. If it appears from the final results of the new testing that these sections still score unsatisfactory, the result of VNK will have to be analysed further.

- Result from VNK mainly agrees with the assessment of the water board and the testing.

*Covering damaging and erosion body of the dike*

- The sections 7002 (024-dp7) for grass, 7009 (020-dp16) for stone, 7028 (004-dp25) for stone, 7258 (074-dp99) for grass and 7271 (072-dp69) for grass score relatively bad for the mechanism covering damaging and erosion body of the dike.
- For the section 7028 (004-dp25), as for 7038 (139a-dp17), insufficient data for the stone covering were initially put into the overall spreadsheet to calculate a result with PC-Ring.
  - For dike section 7028 the data were copied from dike section 7042 (after consult with the water board concerning the type of stone covering). This results in a large probability of failure. By principle it should be verified whether the copied data match the reality. The water board indicates that this section is nominated for improvement concerning the stone coverings. Thus the bad result is identifiable.

  → *The water board thinks that the present result should not be taken into the calculation for the probability of flooding of the dike ring, because the section is part of a running improvement project.*
  - No other data have been put in for dike section 7038 and thus no result has been calculated.
- The testing is being performed now. On this moment additional data are gathered for an advanced testing (amongst others on the grass quality). The water board has already indicated the state of affairs of the (preliminary) results of the 2005 testing for a number of sections. In many cases the type of covering for which these sections were tested differs from the type that VNK has calculated (and which was identifiable for the water board (see section 2.5)). This assessment of the water board with the mechanism for which the section is calculated at VNK next to it is given in table 3-5.
- Comments can thus be given on the results for the mechanism covering damaging and erosion of body of the dike. Further research on the various types of covering (a dike is always constructed from a combination of multiple types of covering (dry stone, stone, asphalt and grass) that are present on a dike section seems necessary. All types will need to be calculated separately and consequently it has to be determined which one is governing (also in relation to the associated design criteria). Even better would be if multiple types of covering on 1 dike section could be calculated with PC-Ring.
- For section 7159 (099a-dp319) it is indicated that it is nominated to be improved. With testing this section doesn't make it based on its age. The water board thus doubts the calculated result, which is relatively good (beta = 5,2). The section partly consists of asphalt and partly of stone. Both types should be calculated.
- The water board has indicated that transition structures often form a weak spot. VNK does not calculate these.

| Section | Judgement water board | VNK Calculations based on: |
|---|---|---|
| 7002 | Stone revetment after inspection considered good | Gras |
| 7023 | Stone revetment insufficient | Stone |
| 7024 | Stone revetment insufficient | Asphalt |
| 7025 | Stone revetment insufficient | Asphalt |
| 7042 | Stone revetment excellent | Stone |
| 7071 | Excellent grass | Stone |
| 7074 | Excellent grass | Stone |
| 7075 | Excellent grass | Stone |
| 7111 | Stone revetment excellent | Stone |
| 7129 | Excellent grass | Stone |
| 7136 | Stone revetment excellent | Stone |
| 7139 | Excellent grass | Stone |
| 7159 | Asphalt insufficient | Stone |
| 7163 | Asphalt insufficient | Stone |

*Table 4-4 Assessment of the water board based on preliminary results 2005 testing*

*Sliding inner slope*

- VNK assesses the sliding of the inner slope. This mechanism is calculated correctly for 1 section (7249 - 076-dp124), for which a large probability of failure is calculated. Other indicating calculations also indicate large probabilities of failure (betas around 2).
- The water board has seen sliding of the outer slope, but no real problems for the inner slope have ever arisen.
- The cause of the bad results can be found in the conservative data that are used for the 1st testing (due to a lack of data). These data were also used for VNK. This results in a pessimistic picture.
- On this moment one is busy doing additional soil research for the 2nd testing (gathering of test samples (borings), measurements of water pressures, foundation). The sub-soil is mapped out better with these methods. It is expected that this will lead to better results for sliding. The model of the sub-soils used for the Mstab calculations also seems conservative. For the long term the water board expects to be able to take this into account better (and consequently calculate better results).
- Apart from that, it needs noticing that the dikes around dike ring 32 are high and steep and that additionally the sub-soil is not very good (weak layers are present). Based on that fact it is not unlikely that sliding will appear as a relatively weak mechanism. For less conservative data as well, it is expected that this mechanism will score relatively bad (beta around 2,5-3).
- For sections 7012 (019a-dp20), 7052 (137a-dp23), 7079 (130-dp16) the water board has separately indicated that these score well for the mechanism sliding in the (preliminary) results of the 2005 testing. For the sections 7226 (080-dp169), 2749 (076-dp124) applies that they need advanced testing for the mechanism sliding of the inner slope. These are sections that are part of the selected cross-sections and thus (except for the latter section) are not part of the 33 dike sections that are selected for calculation.
- For section 7025 (006a-dp15) the water board also indicated that it needs advanced testing for the mechanism sliding of the inner slope. The profile of this section is not assessed on this mechanism within VNK.
- At the 200 testing, none of the selected section scored unsatisfactory for the mechanism sliding of the inner slope.
- It is recommended to couple the other selected profiles, which do not match the selected sections, to the right section in PC-Ring (section that is thus not in the selection). It concerns the sections 7012 (019a-dp20), 7014 (013-dp8), 7052 (137-dp23), 7079 (130-dp16), 7204 (084-dp199), 7226 (080-dp169). Next to that it is recommended to use the results of the additional soil research for these calculations.
  → The water board thinks that the present results should not be taken into the calculations of the probability of flooding, since research is now being done to improve the input data.

*Dunes*

- No single dune sections scores unsatisfactory in the 2005 testing with the new graver boundary conditions for waves (also see section 2.9). The results of the 4 sections that VNK calculated (with the old lees grave boundary conditions), seem to be correct (beta 4,37 to 5,26).
- A suppletion policy is pursued along the whole North Sea coast for both the dunes and the dikes to maintain the basic coastline. VNK can’t directly calculate such dikes. One should assume a coupled failure mechanism; the dike is addressed only after the dune is swept away.

*Sliding outer slope*

- Stability outside the dike (dike and shore drops) is not considered by VNK. The water board expects that especially this mechanism is a threat to the safety of dike ring area 32 (and consequently has a large influence on the probability of flooding).
- Sliding of the outer slope occurs at low tide. Depending on the degree of sliding, this leads to a threat to safety or not.

- The water board indicates that dike ring area 32 has a closed system of regional flood defences with closable constructions to counteract this phenomenon. This system is controlled and maintained by the water board.

## 4.4 Results per structure

The results per structure are given in table 4-5. The results that can be left out of consideration in connection with consult with the water board are shaded grey here as well.

| No. | Structure | Overflow and overtopping | Non-closure | Structural failure |
|---|---|---|---|---|
| 1 | Pumping station Cadzand | 4.4 | 6.0 | 4.5 |
| 2 | Pumping station Campen | | 5.5 | 4.4 |
| 3 | Pumping station Nieuwe Sluis | | 5.9 | 6.1 |
| 4 | Pumping station Nummer Een | | 6.0 | 4.9 |
| 5 | Pumping station Othene | 4.1 | 3.5 | 1.7 |
| 6 | Pumping station Paal | 5.0 | 5.8 | 4.7 |
| 7 | Sluice station Terneuzen Oostsluis | 4.1 | 6.6 | 5.3 |
| 8 | Sluice station Terneuzen Middensluis (schutsluis) | 3.9 | 4.8 | 4.3 |
| 9 | Sluice station Terneuzen Middensluis (spuiriool) | | 5.1 | 4.3 |
| 10 | Sluice station Terneuzen Westsluis | 3.9 | 5.3 | 5.2 |
| 11 | Sluice station Terneuzen Westsluis (spuiriool) | | 5.2 | 5.2 |
| 12 | Discharge sluice station Braakman | 4.7 | 4.3 | 4.5 |
| 13 | Discharge sluice station Hertogin Hedwigepolder | 4.0 | 4.7 | 5.2 |
| 14 | Discharge sluice station Nol Zeven | 4.6 | 4.5 | 4.5 |

*Table 4-5 DHV Results of the assessed structures in dike ring 32*

*Structures*

- The pumping station Othene scores very bad for the mechanism constructive failure (beta of 1,96, probability of failure 1/22). This has to do with the mechanism bursting and piping. This appears to be a problem if one assumes that the ground sills and aprons are not fully watertight. In case one can prove this is the case, or if physical measures are taken to achieve this, the norm can be complied with.

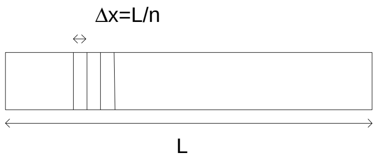
→ *In consultation with GeoDelft, this structure has been tested correctly in the meanwhile. The result can thus be left out of consideration.*

- For the mechanism not-closing the pumping station scores relatively bad (beta of 3,5, probability of failure of 1/4300). Not-closing results in a high probability of failure due to the large number of requests for closing (almost daily) on one hand and the presence of 2 flood defences on the other hand. The failure situation concerns the blocking of the mitre gates due to sedimentation or obstacles, after which the emergency gate can’t be closed in time. Improving the situation is possible by installing an additional set of mitre gates. Next to that, one can think of further investigating the probabilities of failure for the not-closing (advanced method), possibly in combination with optimizing the controls.
  → *Further inspection showed that this inflow is not possible, due to which a lower probability of failure than is now calculated can be expected. This result can thus be left out of consideration.*

- For the pumping station Cadzand the result of VNK seems too good for the mechanism overtopping and wave overrun (beta is 4,39, probability of failure < 1/100.000). From the structures report the following follows: VNK calculates a large probability of failure for this structure, but this probability of failure is adjusted to a much lower probability of flooding. With failure, water (waves) runs over the valve chamber. This overrun flow does not directly result in a loss of stability of the structure and thus to flooding. The overrun flow ends up on a hardened surface the behind the valve chamber and, on both sides, runs into the outlet channel lying behind. The stability of the structure is not lost until a flow runs over that is associated with a much higher water level (and with that a much smaller probability of failure) than the water level at which failure (overrunning) of the structure occurs.

### *4.5 Overall probability of flooding dike ring 32*

Let us assume that a dike stretch of length L is schematised into n sections by:




If the following autocorrelation function for the dike strength R at section x is assumed:

$$\rho\left[R(x), R(x+\Delta x)\right] = e^{-\left(\frac{\Delta x}{d}\right)^2}$$

And the reliability index for the i-th section is beta (for i=1,…, n):

$$P(F_i) = \phi(-\beta)$$

Then, we can write the overall failure probability as:

$$P(F) = \phi(-\beta) + (n-1)\left\{\phi(-\beta) - 2\phi(-\beta)\phi\left(-\beta\frac{1-\rho}{\sqrt{1-\rho^2}}\right)\right\}$$

Since $\max\limits_{j<i} P(F_i \text{ en } F_j) = P(F_j \text{ en } F_{i-1})$ and $\rho = e^{-\left(\frac{\Delta x}{d}\right)^2} \approx 1-\left(\frac{\Delta x}{d}\right)^2$, as well as

$\rho^2 \approx 1-2\left(\frac{\Delta x}{d}\right)^2$, whereas $\phi(u) = \frac{1}{2}+\frac{u}{\sqrt{2\pi}}$ for small u .

Therefore $P(F) = \phi(-\beta)\left\{1+\frac{n-1}{d}\frac{\beta\Delta x}{\sqrt{\pi}}\right\}$

since $\Delta x = \frac{L}{n}$ and $\frac{n-1}{nd}\frac{\beta L}{\sqrt{\pi}} \rightarrow \frac{\beta L}{d\sqrt{\pi}} (n \rightarrow \infty)$

Therefore:

$$P(F) = \phi(-\beta)\left\{1+\frac{\beta}{\sqrt{\pi}}\frac{L}{d}\right\}$$

Which is independent of the number of sections n.

If all results from table 4-1 and table 4-5 are taken into consideration, a preliminary probability of flooding of >1/11 per year (COMBIN 1) is calculated for dike ring area 32, Zeeuws-Vlaanderen. This would mean that flooding is to be expected more than once each 11 years for dike ring area 32. Since the results have not been analysed thoroughly, one can not speak of a so-called reference sum of dike ring 32 in this case.

| Mechanism | COMBIN1 | COMBIN2 |
|---|---|---|
| Overflow / overtopping | 1/794 | 1/11312 |
| Bursting and piping | 1/30211 | 1/30211 |
| Revetment damage and dike erosion | 1/22 | 1/574713 |
| Overflow and overtopping of hydraulic structures | 1/16920 | 1/16920 |
| Non-closure of hydraulic structures | 1/3984 | 1/3984 |
| Structural failures of hydraulic structures | 1/22 | 1/34364 |
| Overall failure probability | 1/11 | 1/1996 |

*Table 4-6 Probability of flooding dike ring 32 according to DHV.*

When the 6 weakest spots for the dikes (7167-097-dp290 for overtopping and wave overrun), 7002-072-dp7, 7009-020-dp16, 7028-004-dp25, 7258-074-dp99 and 7271-072-dp69 for covering damaging and erosion body of the dike) and the weakest spot for the structures (constructive failure of pumping station Othene) are left out of consideration, a probability of flooding of 1/2000 per year (COMBIN 2) is calculated. According to the water board this approaches the value it would expect.

In both cases the mechanism sliding is not taken into account in the calculated probability, whilst it is clear that stability problems are a real threat in this case, because the dikes are high and steep and stand on weak layers in the sub-soil.

Because of the reasons a probability of flooding of <1/100 for dike ring area 32 is presented in the main report and the management summary of the project VNK. Herewith it is indicated that the probability of flooding is mainly determined by stability problems at the pumping station or at the dikes. In relation to the pumping station, it is consequently also indicated that this can be approved based on recent information with the second testing.

### 4.6 Possibilities of sensitivity analyses

For dike ring area 32 no sensitivity analyses have yet been performed. In the section discusses in which way it can be determined which sensitivity analyses can be of interest.

The calculated probability of flooding of the dike ting is determined by a large number of dike sections, dune sections and structures, various failure mechanisms and a large number of stochastic variables per failure mechanism. The possible number of sensitivity analyses is in that way endless. It is therefore important to focus the sensitivity analyses on those factors that determine the level of probability of flooding most. For the dike sections it concerns the relatively weak dike sections. For those dike sections the attention is consequently given to the failure mechanisms that contribute to the probability of flooding most. And for those failure mechanisms the stochastic variables are looked at that have the largest contribution to the probability of flooding. On top of that it is important that these stochastic variables can be decreased by means of further research in reasonable time and with reasonable effort. The latter is an important restriction, for dike ring 32 the stochastic variable ‘Water level Vlissingen” contributes most by far to the probability of failure for the mechanism overtopping and wave overrun. It is however a stochastic variable for which further research will generate little new insights. Even 10 years of additional observations will only be of limited influence on the stochastic variable insecurity with which this stochastic variable is afflicted. Decreasing the probability of flooding by reducing insecurities by means of additional research will thus have to focus on other stochastic variables.

Information on the most influential stochastic variables can be derived from PC-Ring. PC-Ring calculates an influence-coefficient (alpha) per stochastic variable, also called sensitivity-coefficient. The magnitude of the alpha-value is determined by a combination of the influence of the average value and the magnitude of the standard deviation (or variation-coefficient). A low alpha-value for a parameter does not inherently mean that this parameter has little influence on the result. For a small variation-coefficient (or standard deviation), the variation of the average value can still have a significant influence on the result. For a parameter with a small variation-coefficient however, the value of this parameter is relatively 'certain'. This means that it can not be expected that the average value will change a lot as a result of new insights. Varying the average values of those kinds of parameters is possibly interesting for the calculating of measures. The alpha-values (influence-coefficient) are not beatific.
Sensitivity analyses and influence-coefficients are to be considered 'together'.

The alphas thus represent the contribution of the stochastic variable to the probability of failure for a sub-mechanism. These can take effect both on the side of the load (negative alphas) and on the positive side (positive alphas).

# 5. References

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# 6. Appendix A Schematizations and adjustments by DHV

*Selection dike sections*

- In consultation with the water board two weak links in the dikes are added to the selection of DHV. It concerns weak links near:
  - Hm 72.000: this section was already in the original schematization (section 7023)
  - Hm 83.000: this section has eventually been added as section 7009 (dike section 7008 was chosen at first. This has been changed because the choice between 7008 and 7009 didn't matter that much according to the water board (both weak) and 7008 has later been converted into a dune).
- Consequently the total number of dike sections amounted to 33.

*Adjusting profiles*

- The profiles used in the first calculation in PC-Ring were based on old measurements by the water board. Next to that several adjustments were done in the profile in the first calculation, to be able to calculate them in PC-Ring, without data of the water board at hand to check the adjustments in the profiles. Because of that the input profiles were still compared to the recent measurements provided by the water board;
- The recent measurements of the water board are based on a hectometering of the dike (after a recent merging the water board switched from dike pole numbering to hectometering);
  -The dike pole numbering has been re-numbered to a hectometering, based on a conversion table (provided by the water board). With this a difference occurs in the exact position of the dike profiles of less than 50 meters. On a location a difference of 80 meters occurs;
- From the comparison it appeared that there were differences between the schematization and the recent profile measurements at several points:
  - For more than one profile the crown height differed 20 to 70 cm;
  - For more than one profile there were differences in sloping;
  - On several points the profile type in PC-Ring didn't quite match reality.
- In consult with VNK it was decided to adjust all 33 profiles in PC-Ring and to put them in based on recent measurements by the water board;
- Adjustments of profiles resulted in the fact that the profiles used for calculations in this report differ from the profiles used for the first calculation.
- For the new schematization the following assumptions were made:
  - For the toe of the dike one assumed the sand line;
  - If no foreland is present, the second point is the toe. An extra point appears than, which is located 2 meter in front of the toe, on the same level as the toe;
  - The choice between a bend or not on the crown is made based on a visual estimation;
  - If a berm is indicated in the file of the water board, but is it steeper than 1:15 it has to be adjusted for this schematization. In PC-Ring a slope than has to be steeper than 1:10 and a berm than less than 1:15. Everything steeper than 1:10 is considered a slope. With this berm disappears and one obtains a slope with a bend. Everything below 1:10 becomes a berm. One considered up to one digit behind the comma for this;
  - With the downgrading to a berm, an adjustment in height is made for the lowest point on the berm. This way the gradient of the attacked upper slope stays the same. The shift is not done in the line of the slope. The y point is vertically lifted of lowered (it is one or the other, because for extending the line of the lower slope the berm width changes, it is better to adjust the gradient of the lower slope);
  - Applying a bend in the outer slope is done in such a way that the gradient of the upper slope doesn't change. The upper slope point is used as point of inflection. This is done for Ds numbers 7094 and 7159;
  - For the point on the inner slope one assumed the first point on the inner slope that is given by the water board;

- Of Ds number 7028 the adjustment is done differently in order to be able to fit the profile in one of the schematizations. The berm has been lengthened, increasing the gradient of the upper slope and can be considered a slope. Other options for adjusting would mean adjustments for several points, due to which the profile would differ even more from reality;
- Ds number 0747 can't become category 8b, because the gradients of the crown are too high in case of an extra point on the crown. Thus one did eventually decide for a category 7a;
- Of the sections 7047, 7094, 7139 one would say that there's a bend in the crown. Copying 1 on 1 however means that the gradients of the crown planes become too steep (steeper than 1:15). Getting the gradients below 1:15 however means that one has to adjust the points in such a way that it either won't work or the bend becomes next to nil. In these cases one has chosen for a flat crown, and the levels are adjusted in such a way that the schematization is conservative.

*Dunes*

- At firs it was agreed upon to calculate 5 dune sections. With this it was agreed upon to calculate the dune sections of 2004. These provide a conservative picture, because the next (5-annual) suppletion is carried out in 2005;
- The choice of the dune sections to be calculated was made based on the Base Coastline Report of the RIKZ. The choice is based on a comparison of the base coastline (BCL), the coastline to be tested (TCL) and the trend the BCL has. When a probability of failure is calculated that contributes a lot to the total probability of flooding of the dike ring, possible nuances can be made based on information from the report on the base coastline. The calculated profile namely provides a lower limit of the probability of failure of the dune in question (dunes are calculated based on section measurements of a weak year);
- For the location of the dune sections one considered the maps of the Base Coastline Report and maps of the water board with the location of the dikes and dunes on them. The dike ring schematization in PC-Ring was also considered (where is a dune and where is a dike schematized);
- Eventually, considering the schematization present in PC-Ring, this lead to the choice of 4 dune sections to be calculated. In consult with VNK one has chosen to convert 2 profiles, which at first were schematized as dikes in PC-Ring, into a dune profiles in order to be able to calculate 4 dune sections. Possibly a fifth dune section could have been calculated, if a dune section was divided into two dune sections. In consult with VNK it was decided not to do this;
- Of these 3 sections are weak and 1 section is a strong section. The strong section was chosen to see whether the outcomes in PC-Ring provide the same picture of the safety as the present situation of the section;
- Eventually the sections 7008, 7010, 7013 and 7027 were calculated as dune section. See the following schematization also:

- It needs noticing here that:
  - At first it was decided to calculate section 71 (Breskens) as well, since this is a weak section (which was indicated by the water board as well). In consult with VNK however it was decided not to calculate this section, because the section (and the rest of the dune site of which section 71 is a part) can't be schematized properly. The site is in between two jetties that have a strong reducing effect on the waves. Wave conditions are used that serve as input for the SWAN-calculations for dunes. For the Westerschelde these are the wave conditions for platform EUR. This is a deep water location at a considerable distance from the coast. In practice this means that the wind directions W to NNE are governing for the dunes. This was assumed because other conservative loads were not yet available. The deep water waves will in reality not reach the foot of the dune as a result of protection by the dams and possibly also because the coast is located in the shade of Walcheren (orientation of the dune is northerly, zero degrees). Expectation is thus that the wave loads will be less than follows from the calculations. Next to that stone covering is present at the foot of the dune. All considered,

section 71 can not be taken into account properly in the calculations at this moment, despite it being a weak section;
- Section 1354 was chosen as well. After consult with the water board this appeared to be a dike.
- Dune section 1401 is put in on the original dike section 7008. Originally the x-y coordinates of dike pole 020-dp15 were used for 7008 in the schematization of dike ring 32 in PC-Ring. This dike pole is located roughly 500 meters east of section 1401. Because of this, one calculated with wave conditions specified for a location 500 meters away for section 1401.

- In connection with the schematization, several adjustments to PC-Ring have been carried out:
  - Two dike sections are converted to a dune, by:
    - Putting in type 2 for DS;
    - Profile type is 7;
    - River normal 999;
    - All fetch sections switched on (landside also)
    - Location codes are adjusted;
    - Profile overwritten by the Jarkus profiles.

  - For the four dune sections the right location codes have been put in the table dike section (with this also load model 10 has been added);

| (section) | loc.code | loc.code 1 | loc.code 2 | intpolation % |
|---|---|---|---|---|
| (230) | 7770028 | 7770028 | 7770029 | 85% |
| (851) | 7770028 | 7770028 | 7770029 | 40% |
| (1242) | 7770028 | 7770028 | 7770029 | 15% |
| (1401) | 7770028 | 7770028 | 7770029 | 5% |

- With respect to the boundary conditions it applies that the dunes of dike ring 32 are to be coupled to the load model of the Westerschelde. For this input files were added to the existing input files for the sandy coast.

  - With the adding of the files mentioned above, two locations were added. The locations concern:
    - 7770028 Bresken coordinates 27502 380752 test level = 5,25 m + NAP
    - 7770029 ‘t Zwin coordinates 15013 378273 test level = 5,05 m + NAP

*MHW check*

- At the MHW check of DHV, an error was found in the location codes in PC-Ring. As a result of this, the right location codes were put in. The MHW check was done once more by DHV (see appendix B).

*Calculations DHV*

- The country setting of the computer is set to English to guarantee that the values that are put in the database are read correctly by PC-Ring. Decimal values have to be put in with a dot, so 0.4 in stead of 0,4;
- The stochastic variables for the calculations with coverings were all switched on, except for the deviation of wave direction;
- All calculations for probabilities of flooding are performed with the FORM*DS calculations method; VNK has used other techniques later on as well.
- For the covering calculations certain model settings were used. Possible adjustments are indicated by VNK in appendix B.

[1] According to PC-Ring (VNK) asphalt has to be chosen at all time for residual strength calculations, however this doesn’t lead to any results.

- DHV has switched off a large number of wind directions (by putting the number of fetch sections to zero) in order to obtain results for the stone coverings. This lead to the fact that sometimes only three to seven wind directions were taken into account for calculating the probability of failure, this is in principal not correct. VNK has switched all these wind direction back on again at the start of its calculations. For the results for stone coverings, this doesn't this didn't matter too much. For the overtopping/wave overrun it did have somewhat more influence.
- With the help of the program MProStab calculations were done from which the probabilities of failure followed for the mechanism sliding, which can be combined with the other mechanisms of failure;
- The probabilities for the structures are determined using a method by hand. The sto.files obtained this way serve as input for PC-Ring which calculates the alphas en betas. The influence coefficients for both the mechanism constructive failure and not-closing of the closing elements were weighed based on probability contributions of the related mechanism and the residual strength. The values used for the mechanism overtopping and wave overrun and the other mechanisms are based on the ISO-norm. The sto.files serve as input for PC-Ring which calculates the values of the alphas, betas and the probabilities of failure. These values are consequently combined with the other probabilities to determine the total probability of flooding for the dike ring.
- After studying the values in the overall spreadsheet of the Water Board Zeeuws-Vlaanderen it appeared that the length of the seepage path was not represented correctly. The significantly greater length of the seepage path was determined based on the geometry of the dike for several dike sections (one assumed that the seepage path is minimal from toe to toe).

| DV Nummer | Naam | Kwelweglengte WS | Kwelweglengte geometrie |
|---|---|---|---|
| 1320007007 | 024-Dp7 | 53,8 | 88,2 |
| 1320007009 | 020-Dp16 | 51,5 | 98,4 |
| 1320007023 | 006a-Dp10 | 42,2 | 120,5 |
| 1320007024 | 006a-Dp11 | 42,3 | 124,7 |
| 1320007025 | 006a-Dp15 | 50,3 | 101,6 |
| 1320007028 | 004-Dp25 | 49 | 100,2 |
| 1320007038 | 139a-Dp17 | 26,2 | 85,9 |
| 1320007042 | 138-Dp2 | 26,6 | 79,3 |
| 1320007047 | 137b-Dp7 | 19,9 | 191,4 |
| 1320007053 | 137a-Dp24 | 18,7 | 188,8 |
| 1320007071 | 133a-Dp67 | 23,8 | 77,4 |
| 1320007074 | 132a-Dp74 | 17,8 | 62,0 |
| 1320007076 | 131-Dp7 | 16,4 | 52,2 |
| 1320007094 | 127c-Dp69 | 28,6 | 69,4 |
| 1320007109 | 123-Dp26 | 26,8 | 81,3 |
| 1320007111 | 122-Dp16 | 25,4 | 84,9 |
| 1320007116 | 121a-Dp9 | 24,4 | 73,8 |
| 1320007124 | 113-Dp87 | 25,2 | 147,5 |
| 1320007129 | 111a-Dp12 | 33,3 | 99,0 |
| 1320007136 | 105-Dp10 | 30,6 | 86,1 |
| 1320007139 | 103-Dp23 | 29,3 | 79,7 |
| 1320007152 | 100a-Dp330 | 26,9 | 73,5 |
| 1320007155 | 099a-Dp319 | 25,5 | 94,2 |
| 1320007163 | 098b-Dp304 | 25,8 | 58,6 |
| 1320007167 | 097-Dp290 | 20,2 | 75,9 |
| 1320007185 | 090-Dp248 | 14,4 | 58,4 |
| 1320007202 | 085a-Dp201 | 18,9 | 74,7 |
| 1320007211 | 083a-Dp186 | 15,8 | 63,8 |
| 1320007220 | 081a-Dp175 | 15,8 | 58,1 |
| 1320007233 | 078-Dp148 | 16,3 | 63,0 |
| 1320007249 | 076-Dp124 | 16,0 | 61,6 |
| 1320007258 | 074-Dp99 | 15,1 | 63,8 |
| 1320007271 | 072-Dp69 | 14,7 | 61,1 |

Seepage path lengths for all sections (first column) given by Water board (3rd column) and based on the dike geometry (4th column).

# 7. Appendix B Adaptations by VNK

VNK has performed all calculations again (based on the database of DHV). In the scheme below it is indicated per section what has been altered relative to the database that was supplied by DHV.

| Section | Actions/notes related to database after delivery by DHV |
|---|---|
| All | In the calculations of DHV many wind directions have been turned off because these would not be relevant for the mechanism overtopping/wave overrun (this would be valid for offshore wind).<br>Offshore wind can be neglected in the river area (Bretschneider is used in that case). Along the coast the boundary conditions are determined using SWAN (in which also heave and diffraction etc are present). Herewith it could be that 1 or 2 wind directions do not converge. These wind directions could than possibly be turned off. For this one should first check whether these wind directions are not governing for overtopping/wave overrun. **Action 1: all wind directions are turned back on for all selected sections.**<br><br>For the land of Saeftinghe a foreland of 5 kilometer is put in. the SWAN-points however are located 100 meters in front of the coast (and 300 meters apart). Foreland of 5 kilometer is useless. Foreland is turned off in the calculations of VNK.<br><br>If the foreland is located >4 meter, there are no waves and thus no result.<br><br>At a MHW-check, water levels are related to the RVW-book. This is not correct. There are different values with which should be checked. For this a file has been delivered by TNO in the past. No set-up is ipc expected along the coast. The values from table 4 are thus not correct. **Action 2: all wind set-up has been removed.** (in the database the dike section set-up is set to zero everywhere, as is the number of sections due to newer MHW-check). One does not save alterations (because then the new assortment is not saved, but the altered values are). |
| All | MHW-check performed according to the prescribed procedure. Foreland is turned off at sections 211, 220, 233, 249, 258, 271. See tab MHW-check → 2.pcr<br><br>New PCR-file produced → results VNK → let everything be calculated by FORM-DS with 1 foreland point. If no good result was obtained, one looked whether this could be solved by adapting various things. |
| 249 | Result seems to be caused by a strong covering. With the DS calculation the number of 5000 samples is too little with the higher beta values. These calculations thus have to be performed again with 100000 samples. Sections will not suddenly turn out weak. |
| 109 | No result for overtopping/wave overrun → initial value altered (from 8 to 1) and calculated with 6DS*FI instead of 8FORM*DS. |
| 38 | No data input on stone covering → thus no result |
| 24,25 | Level of GWL was set as 5,35. This should be 0 (is sea). Has been adapted. Factor fmGWS was set as 0,15 River, has been adapted into 0,25 Sea. |

| | |
|---|---|
| 42 | Covering: at wind direction 330 the residual strength crashed. 330 is governing wind direction for wave overrun, can not be turned off. Start method has been adapted from 8 → 1. Ov/ov keeps functioning. |
| 74 | Covering: crashes on the residual strength with the Combin calculation. Start method adapted from 8 → 1. Ov/ov keeps functioning. |
| 116 | Covering: crashes at residual strength. Start method has been adapted from 8 → 1. Result for covering, but now ov/ov does not function well. Southern wind direction (180 degrees) turned off. Now result for both mechanisms. |
| 139 | Gets stuck on residual strength. Start method adapted from 8 → 1. This results in a beta of 36. Covering seems very thick. Possibly a number of samples need to be calculated. Ov/ov keeps functioning. |
| 152 | Initial value adapted from 8 → 1. Result for covering, but now ov/ov does not function well. Southern wind direction (180 degrees) turned off. Now result for both mechanisms. |
| 159 | Initial value adapted from 8 → 1. Result for covering, but now ov/ov does not function well. It is not allowed to turn off the northern wind direction and even with 6DS*FI no good result follows. This should be solved manually. Thus ov/ov with initial value 8 and covering with initial value 1. → **both with FORM-DS** |
| 163 | Initial value adapted from 8 → 1. This does lead to result for the covering. Ov/ov keeps going well. Wind directions North, 30, 60, 90 turned off (not governing for ov/ov) → now result for both mechanisms |
| 167 | Initial value adapted from 8 → 1. Result for covering, but now ov/ov does not function well. It is not allowed to turn off wind direction 60 degrees and even with 6DS*FI no good result follows. This should be solved manually. Thus ov/ov with initial value 8 and covering with initial value 1. → both with FORM-DS. |
| 185 | Initial value adapted from 8 → 1. Ov/ov keeps functioning. Now result for the covering |
| 202 | Initial value adapted from 8 → 1. No result yet for covering, stops at 150. Ov/ov keeps functioningSouthern wind direction turned off → now result for covering. |
| 211 | Initial value adapted from 8 → 1. Result follows for both covering and overtopping/wave overrun. |
| 23 | Covering: crashes. Start method adapted from 8 → 1. Ov/ov keeps functioning. |
| 124 | Covering: crashes. Start method adapted from 8 → 1. Ov/ov keeps functioning. |
| 129 | Covering: crashes. Start method adapted from 8 → 1. Ov/ov keeps functioning. |
| 233 | Covering: crashes. Start method adapted from 8 → 1. Ov/ov keeps functioning. |
| All | Number of samples adapted from 5000 → 10.000 for all sections |
| Selection | For selected sections checked whether level GWL and Factor fmGWS are filled in correctly. Appeared this was often not the case. Adapted if necessary. Level GWL |

| | |
|---|---|
| | was set at 5,35. This should be 0 (is sea). Has been adapted. Factor fmGWS was set at 0,15 River, has been adapted to 0,25 Sea. |
| 28, 38 | For these sections no covering data were put in. |
| 28 | For 28 data input based on section 42.<br>Width stone 0 → 0,2<br>Length stone 0 → 0,2<br>Porosity filter 0 → 0,35<br>This results in a beta of 0,2<br>Section 28 has a very high toe |
| All | Calculate all sections for all mechanism. Initially all with FORM*DS, initial value 8 or 1 (resulting from action for 23, 42, 74, 116, 124, 139, 152, 163, 167, 185, 202, 211 and 233) |
| | For the following sections no or odd results have been calculated: |
| 28 | covering beta is 0,2057 |
| 109 | covering |
| 136 | overtopping/wave overrun + covering |
| 159 | overtopping/wave overrun |
| 167 | overtopping/wave overrun |
| 159, 167 | Adapt initial value from 1 → 8, then a result for overtopping/wave overrun; adapting manually per mechanism. Thus covering with initial value 1 and overtopping/wave overrun with initial value 8. |
| 109, 136 | Calculate with DS*FI with initial value 8 gives a result for all mechanisms |
| 28 | If 3 (residual strength not relevant) is chosen in stead of 6 for the type number of the residual strength model, the same beta is calculated. Thus there are no data concerning residual strength in calculation.<br>Based on 42 more data adapted<br>Measure for acceleration of erosion in core of the dike: 1 (equal to the cover layer) → 3 (sand core)<br>Crack width: 0,001 → 0,015 (is standard in overall spreadsheet, was not put in for 28)<br>Relative density stone (average value): 1 → 1,8<br>Thickness granular filter layer: 0,04 → 0,1<br>Grain size 15% fraction of the filter material: 0,001 → 0,02 (is standard in the overall spreadsheet, was not put in for 28)<br>Residual strength model → 6 as in 42. |