Fully probabilistic assessment of safety against flooding along the Dutch coast

Vuik, V.; van Balen, W.; van Vuren, S.

DOI
10.1111/jfr3.12153

Publication date
2017

Document Version
Final published version

Published in
Journal of Flood Risk Management

Journal of
Flood Risk Management

Fully probabilistic assessment of safety against flooding along the Dutch coast

V. Vuik[1,2], W. van Balen[3] and S. van Vuren[1]

1 HKV Consultants, Lelystad, The Netherlands
2 Faculty of Civil Engineering and Geosciences, Delft University of Technology, Delft, The Netherlands
3 Deltares, Delft, The Netherlands

Correspondence
Vincent Vuik, HKV Consultants, PO Box 2120, Lelystad 8203 AC, The Netherlands
Tel: +31630027021
Email: vuik@hkv.nl

DOI: 10.1111/jfr3.12153

Key words
Dune erosion; Dutch coast; failure probability; flooding probability; flood risk; overflow; overtopping; probabilistic assessment.

Abstract

This paper describes a fully probabilistic safety assessment of the Dutch North Sea coast, in which stochastic properties of both hydraulic loads and strength of the flood defences have been taken into account. The study has led to an overview of failure probabilities along the coast with high spatial resolution. Both dikes and dunes have been considered. Failure probabilities at individual locations have been combined to flooding probabilities per dike ring area. The vast majority of the Dutch coastal defences is quite secure in terms of flooding. This study demonstrates that generally, the Dutch dunes provide a higher degree of safety than the sea dikes. When incorporating the consequences of flooding to the analysis, the calculated flooding probabilities can be used to determine flood risks. The probabilistic method, presented in this paper, enables accurate balancing between avoided flood risks and investments to reinforce the flood defences.

Introduction

Large parts of the Netherlands are situated below or around mean sea level. These low-lying areas are protected against flooding by sea defences, consisting of dunes and sea dikes. The Dutch government prescribes minimum return periods for the hydraulic forcing such as surge levels and waves, which the sea defence should be able to withstand. The return periods vary along the coast because the population density and economic value are not uniformly distributed across the area behind the flood defences. Places along the Dutch coast, mentioned in this article, have been included in Figure 1.

In a periodic safety assessment, the reliability and status of all dunes and dikes in the Netherlands is determined. The approach used can be seen as semi-probabilistic because hydraulic loads are determined probabilistically, whereas the strength of the sea defences is considered deterministically. Firstly, normative water levels, wave heights and wave periods are determined, using their respective probability distributions. Subsequently, these three values are used in the deterministic assessment of dune erosion, resulting in a binary result only: safe or unsafe. The surplus or lack of safety is not quantified. This shortcoming of the current approach leads to the main objective of the study as described in this article.

The objective of this study is to determine the failure probability of dikes and dunes along the Dutch North Sea coast in a fully probabilistic way, in which both loads and strength are considered uncertain. The results are an overview with high spatial resolution of the current safety of the Dutch sea defences and an instrument to map the relatively weak spots along the coast. The failure probabilities along distinct stretches of the coast are combined to obtain flooding probabilities of the area behind the flood defences.

The comparison between actual flooding probabilities and desired flooding probabilities provides the ability to start prioritising within the strengthening programme of the Dutch government. Desirable flooding probabilities can be based on an economic cost–benefit analysis or expected casualty numbers.

Methodology

Dunes and dikes generally fail in fundamentally different ways. Therefore, both types of sea defences have to be considered separately. This section describes how failure probabilities, for dunes as well as dikes, have been computed at individual locations, and how these probabilities have been combined to obtain flooding probabilities of the area behind the flood defences.

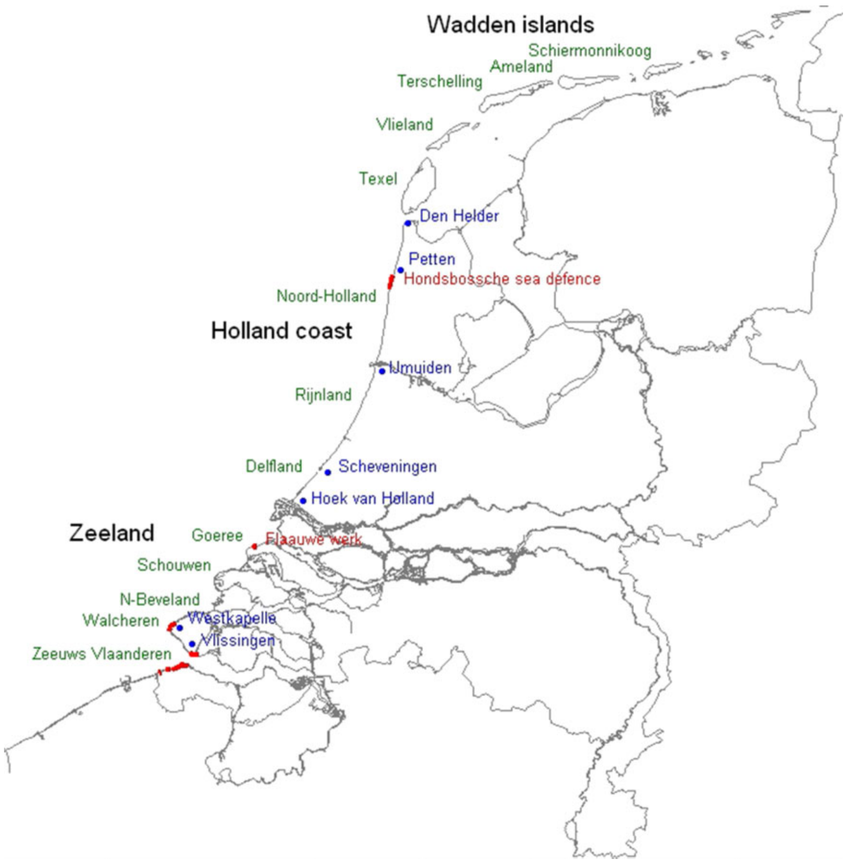

Figure 1 The Dutch coastline, separated in three main parts (black) and 13 dike ring areas (green). Dikes are shown in red, some cities along the coast are shown in blue.

Dunes

The largest part of the Dutch sea defence adjacent to the North Sea exists of dunes. Dunes develop where waves encourage the accumulation of sand and where prevailing onshore winds blow this sand inland (Pye and Tsoar, 2009). The height of the Dutch dunes typically varies from 10 to 20 m above mean sea level, with on average relatively high dunes at the Holland coast. The width of the dune area is highly variable, ranging from about 100 m to multiple kilometres.

Failure mechanism

Dune erosion is the governing failure mechanism of coastal dunes. Dune erosion means that sediments from the mainland and upper parts of the beach are eroded during a severe storm surge and settled at deeper water within a short time period; this is a typical cross-shore sediment transport process (Van de Graaff, 2008).

In periods with fair-weather waves and swell, dunes accrete, leading to a more or less equilibrium shape of the profile. The vertical position of this profile depends on mean high and low water levels. During severe storm surges, water levels sometimes increase to several metres above normal tidal levels, and high wave heights occur. These extreme conditions require quite a different shape of the dune profile than the shape of the initial equilibrium profile to safeguard the same level of safety. Dune erosion has been studied extensively by many researchers, and has been described by, for example, Edelman (1968), Dean (1973), Vellinga (1986), Kriebel *et al.* (1991), Kraus *et al.* (1991), Steetzel (1993) and ENW (2007).

The sandy dunes above surge level are heavily attacked by breaking waves, leading to erosion. The eroded sand in this so-called swash zone is transported to the beach and foreshore by the downrush. Both short period wind waves and infragravity waves with slightly longer periods contribute to the erosive process. When the sandy slope becomes too steep, lumps of sediment slide downwards (Van Rijn, 2010). The accumulated sand at the front of the dunes causes a reduction of the wave heights. In particular, the final erosion length depends on the initial profile, the surge level, wave characteristics, storm duration and sediment characteristics.

Calculating dune erosion

In the 1970s and 1980s, extensive research on dune erosion during storm surges has been carried out, especially by means of laboratory experiments. The presently used dune safety assessment for the Dutch coast is based on the empirical erosion profile method, as developed by Van de Graaff (1988) and Vellinga (1986).

In Den Heijer *et al.* (2012), an overview is given of the development of numerical models for dune resilience assessment. The work of Vellinga (1986) led to the development of the one-dimensional DUROS program (TU Delft, Delft, the Netherlands). De Ronde *et al.* (1995) and Roskam and Hoekema (1996) have analysed wave data and have shown that during storm surges, slightly larger wave periods exist than previously assumed. To assess the effect of this new insight on dune erosion, laboratory experiments have been carried out (Van Gent *et al.*, 2008; Van Thiel de Vries *et al.*, 2008). Afterwards, DUROS has been adapted by Van Gent *et al.* (2008) to account for the wave period influence. The modified version of DUROS is denoted as DUROS+. The DUROS+ model has been used in the current study to model dune erosion.

DUROS+ assumes a parabolic erosion profile (TAW, 1984; ENW, 2007; Den Heijer *et al.*, 2012) as shown in Figure 2. The shape of this profile depends on wave height, wave period and the settling velocity of the sand. The maximum surge level determines the vertical position. DUROS+ computes a cross-sectional profile, for which the eroded and accreted volumes are equal. The parabolic profile is bounded by a linear 1:1 slope at the dune front and a linear 1:12.5 slope at the foreshore, connecting the erosion profile with the initial profile.

At several locations along the Dutch coast, the actual geometry differs from the geometry for which DUROS+ has been validated (Den Heijer *et al.*, 2012). This is, for example, the case at the strongly curved heads of the Wadden Isles, dune areas containing hard structures (e.g. boulevards), transitions between soft and hard sea defences, and dune areas with multiple dune rows (Van Santen *et al.*, 2012). At such locations, the accuracy of the calculated dune retreat is less than at locations for which the geometry approaches the idealised dune profile.

Probabilistic approach

A single DUROS+ calculation is deterministic. For a certain combination of a cross-sectional profile, grain diameter, surge levels and wave characteristics, the model computes a unique erosion profile. To obtain a failure probability for dune erosion, the program PC-Ring (Lassing *et al.*, 2003; Vrouwenvelder and Steenbergen, 2003a,b; Steenbergen and Vrouwenvelder, 2007; Van Balen *et al.*, 2012) has been used. PC-Ring is a software package that contains multiple

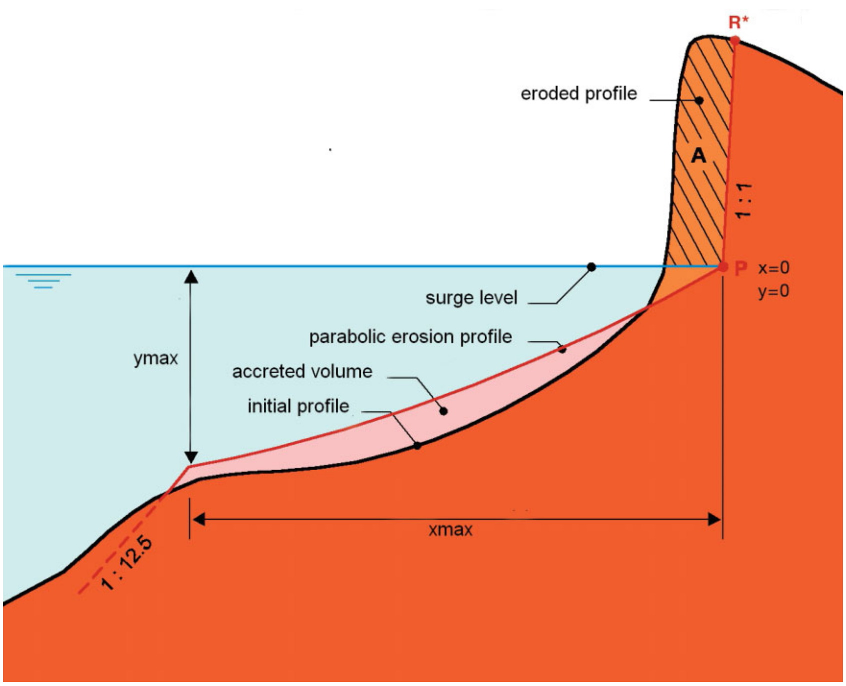

Figure 2 Transformation of an initial dune profile to a parabolic erosion profile by dune erosion. Figure based on ENW 2007.

probabilistic computation methods that can be applied to a variety of failure mechanisms of flood defences. In the case of the failure mechanism dune retreat, PC-Ring functions as a probabilistic shell around DUROS+.

A probabilistic calculation requires a limit state function

$$Z = R - S \tag{1}$$

in which the resistance is denoted by R and a load (solicitation) denoted by S. Failure occurs when $Z < 0$ because $S > R$. Failure in case of dune erosion is defined as the situation in which the erosion length exceeds the critical erosion length associated with a limit profile (Steenbergen and Vrouwenvelder, 2007). This limit profile is defined as the dune profile that is just able to fulfil its water-retaining function under the prevailing conditions (TAW, 1984). We define X_c as the corresponding critical erosion length and X_e as the erosion length calculated by DUROS+. Uncertainties in storm duration, precipitation and computational method are accounted for by a model factor m_D. Multiplication of X_e with m_D gives the governing erosion length X_e^*. Thus, the limit state function is defined by

$$Z = X_c - X_e^* \tag{2}$$

The failure probability is given by

$$P(Z < 0) = P(X_c - X_e^* < 0) \tag{3}$$

This failure probability can be approximated by several probabilistic calculation techniques, such as FORM (first order reliability method), second order reliability method, directional sampling and Monte Carlo simulation. Because the latter two are very computationally expensive, the relatively straightforward FORM approach has been used in the current study. When using FORM, all probability distributions are transformed to a standard normal distribution, and the limit state function is linearised in the point with the most likely combination of load and resistance for which $Z = 0$, the so-called design point. FORM leads (theoretically) in almost all cases to an accurate result (e.g. Rackwitz and Flessler, 1978; Hohenbichler *et al.*, 1987).

The following parameters have been considered as random variables (i.e. uncertain):

- water level (m + NAP)
- significant wave height (m)
- wave peak period (s)
- errors in measured bed level ($\sigma = 0.1$ m)
- median grain diameter D_{50} of the sediment (m)
- the model factor m_D (normal distribution with $\mu = 1$, $\sigma = 0.25$)

The statistical description of the water level, the wave characteristics and the grain diameter are discussed in more detail later in this article.

DUROS+ is typically applicable for sandy dune coasts as present in the Netherlands. However, this model can also be applied for similar dune geometries in other countries. The probabilistic method is generally applicable and can also be used for calculation of other failure mechanisms of water defences.

Dikes

In the past, dikes along the Dutch coast have been constructed at locations where the dunes provided insufficient safety against flooding. Examples are the 5.5-km-long 'Hondsbossche Sea Defence' near Petten and the 2.8-km-long dike the 'Flaauwe Werk' near Ouddorp. Asphalt coatings or concrete blocks protect the outer slopes of most dikes against wave impact, whereas grass or asphalt typically covers the inner slopes.

Failure mechanisms

Dikes can fail to resist water in many ways, for instance, through overflow, wave overtopping, piping, heave, instability of the outer slope armouring, macro-instability of the inner or outer slope, etc. (Allsop *et al.*, 2007). Most failure mechanisms are not very likely to occur because the time scale of the hydraulic loads for coastal dikes is short with respect to river dikes. For that reason, the current study focuses on overflow and overtopping only. This assumption is supported by the results of Mai Van *et al.* (2007) for sea dikes in Vietnam, for which the contribution of the combination of overflow and overtopping to the total failure probability is about 60%. Overflow occurs when the surge level exceeds the crest height of the dike. A dike fails because of wave overtopping when the overtopping discharge exceeds a certain critical value, depending on the quality of the cover of the inner slope.

Calculating overflow and overtopping

The limit state function of the failure mechanism overflow is given by

$$Z = h_d - h \tag{4}$$

in which h_d is the crest height and h the local water level (m + NAP).

For the failure mechanism wave overtopping, the limit state function is defined as:

$$Z = m_{qc} q_c - m_q q \tag{5}$$

In this formula, q_c represents the critical overtopping discharge and q the actual overtopping discharge (l s/m). The parameters m_{qc} and m_q are model factors that account for model uncertainties. The formulas of Van der Meer (1993), as described in TAW (2002), are used to calculate the actual overtopping discharge. The critical value of the overtopping discharge depends on the quality of the grass cover at the inner slope.

At the seaside of the dikes at the North Sea coast, often a natural, sandy foreshore is present. This relatively gentle sloped foreshore leads to a reduction in wave height. This reduction is incorporated in the values of the actual wave overtopping.

Probabilistic approach

In methodological analogy with dunes, failure probabilities of sea dikes have been approximated with the probabilistic method FORM.

The following parameters have been considered as random variables (i.e. uncertain):

- water level (m + NAP)
- significant wave height (*t*)
- wave peak period (s)
- wind speed (m/s)
- geometrical uncertainties: crest height, dike height, outer slope (m)
- several model uncertainties
- the critical overtopping discharge q_c (l s/m)

Data used

A proper and accurate (statistical) description of the geometric profile and the hydraulic loads is necessary when conducting a reliability assessment of coastal defences. This section describes the main sources of data: the geometry of the coastal profile and the hydraulic loads (consisting of surge levels and wave characteristics).

Geometry of the coast

The geometry of the coast is determined on a yearly basis at over 2000 locations along the Dutch coast (Otten, 1985; Minneboo, 1995), starting from 1965. These measurements are called JarKus measurements. In the current study, the dataset of 2011 has been used. At each location, a shore-normal transect is defined, covering the shoreface, beach and dunes. The submerged part of the profile is measured by echo sounding, the dry part by laser altimetry (De Graaf *et al.*, 2003). The horizontal resolution along transects is typically equal to 10 m. The alongshore spacing between the transects varies roughly between 150 and 250 m. Figure 3 shows a typical example of a cross-section of a dune along the Dutch coast.

At coastal stretches where the dunes are very wide (in the order of kilometres), the JarKus measurements cover the seaward part of the dunes only. Because only the complete dune field is of interest for the purpose of calculating failure probabilities, the JarKus measurements have been extended with AHN data of 2004. AHN is an abbreviation of the Dutch term Actueel Hoogtebestand Nederland (Actual Elevation Database of the Netherlands). This database contains elevations for the entire Netherlands (Wouters and

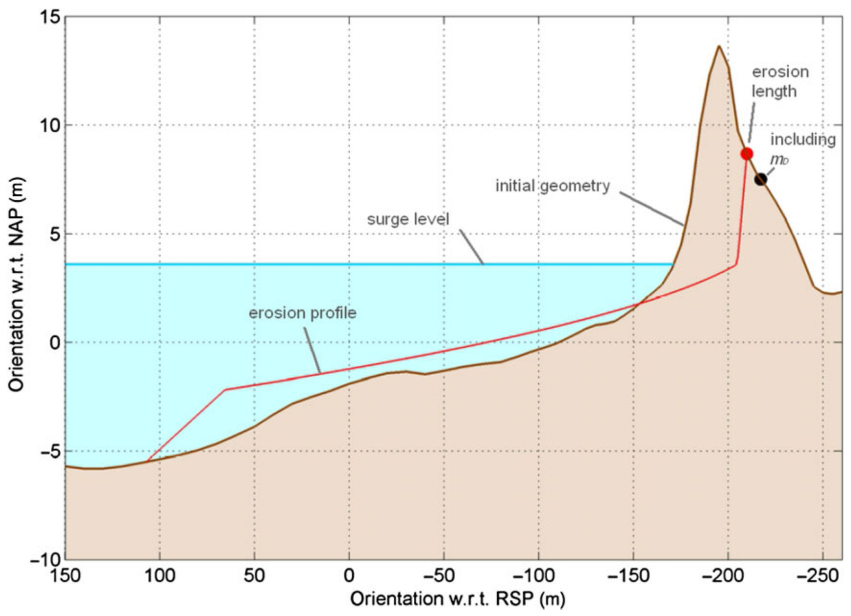

Figure 3 Example of a cross-section of a dune along the Dutch coast (transect number 7002009 near Petten). The vertical position is related to the Dutch reference level NAP. The horizontal position is measured along the transect, with zero point at a pole on the beach, the so-called RijksStrandPaal (RSP).

Bollweg, 1998). Data from 2004 is sufficiently recent because the areas outside the reach of the JarKus data are relatively fixed. JarKus profiles have been extended with AHN data until buildings are reached, with a maximum of 2500 m.

Both JarKus data and AHN data have been provided by the online database OpenEarth (Van Koningsveld *et al.*, 2010).

Multiple dunes

One of the limitations of DUROS+ is the limited applicability on profiles with multiple dunes. Only profiles with a single dune lead to reliable failure probabilities. For that reason, all profiles with multiple dunes are cropped in the lowest point between the first and the second dune. The failure probability as calculated by PC-Ring therefore only yields results for the first row of dunes.

For management purposes, the failure probability of the first row of dunes has only limited value. To be able to account for the presence of multiple dunes, a method has been developed to estimate the failure probability for the whole dune field.

Figure 4 illustrates the method. On the vertical axis, the failure probability P is shown. On the horizontal axis, the volume V of the dune (m^3/m) above dune foot level is plotted. In the Netherlands, the dune foot level is defined at 3 m above the Dutch reference level NAP. The markers give the calculated failure probabilities for the first dune row and the corresponding dune volume above dune foot level. Evidently, larger volumes lead to lower failure probabilities.

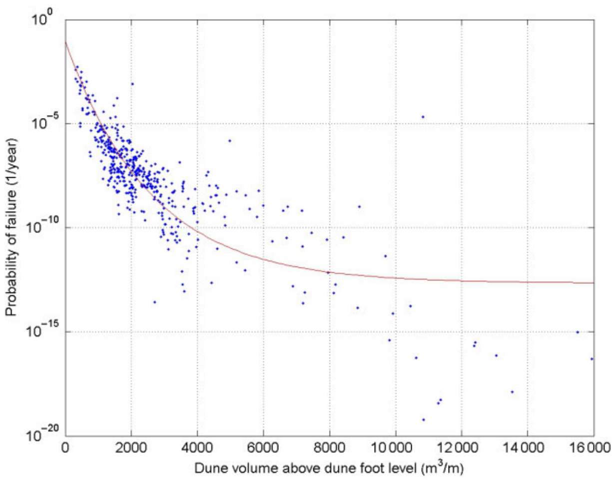

Figure 4 Estimating failure probabilities in case of multiple dunes. Blue markers show calculated failure probabilities of the first dune rows as a function of the dune volume above dune foot level of the corresponding transect. With a non-linear fit, shown in red, a failure probability of the whole dune field can be approximated, using the dune volume above dune foot level of the complete dune field.

A non-linear function has been fit through the markers belonging to the first dune. The function is of the form:

$$\log P = A\left(1 - e^{bV}\right) + C \tag{6}$$

The coefficient C determines the probability P, which belongs to $V = 0$. This probability is estimated by calculations of overflow of the dune foot level:

$$C = \log P(h > 3) \tag{7}$$

Next, the coefficients A and B are determined by means of a standard least squares method. Because of the differences in characteristics in load and strength, the Dutch coast is divided in the three main parts as shown in Figure 1: the Wadden islands, the Holland coast and the Zeeland coast. Figure 4 shows the results of the Holland coast.

The failure probability of the first dune is denoted as Pf_1. The failure probability of the complete dune field Pf_2 can be approximated by Eqn (6) by inserting the volume V_2 above dune foot level of the complete dune field. So, the coefficients A, B and C of Eqn (6) are based on the calculation results of the first dune row. Subsequently, this equation is used to estimate the failure probability of the complete dune field.

Two restrictions have been applied. Firstly, the complete dune field cannot be weaker than the first dune row, so $Pf_2 \leq Pf_1$. Secondly, a conservative value has been applied for locations with a relatively weak first dune row (markers above the red line in Figure 4) because the failure probability of the complete dune field Pf_2 is based on the relatively high Pf_1 at these locations:

$$\log P_{f2} = \log P_{f1} + A\left(e^{bV_1} - e^{bV_2}\right) \tag{8}$$

The probabilities for multiple dunes are directly related to the calculations for single dunes. This implies that a dune area with multiple dunes is treated as an area with enlarged single dunes. Because of this simplification, the results are at the most an indication of the failure probabilities of dune areas with multiple dune rows.

However, the method guarantees that a location with a relatively weak first dune in front of a wide dune area is not indicated as a weak spot in the coastal defence. The correction for multiple dunes has not been carried out at dune areas with populated areas, or at locations where two-dimensional effects can play an important role after breaching of the first dune row.

Hydraulic loads

The hydraulic load models in PC-Ring are taken from the TMR2006, a Dutch set of hydraulic boundary conditions. The Dutch government prescribes these hydraulic loads for water-defence assessments. The content of the TMR2006 is based on calculations of hydrodynamics and waves with the modelling packages WAQUA (based on Stelling, 1983 and Leendertse, 1987) and SWAN (Booij *et al.*, 1999; Ris *et al.*, 1999). Water levels depend on driving forces as wind speed, surge level and river discharge

Surge levels are based on water level data from 12 stations along the Dutch coast, using the statistical methods of Roskam *et al.* (2000). Exceedance frequencies are calculated with a conditional Weibull distribution, extended with relations between water level and wind statistics. Wave loads are related to the random variables water level, wind speed and wind direction with help of a physical relationship. The physical relation is quantified with help of wave simulations with SWAN and wave data from five offshore buoys along the Dutch coast (Vrouwenvelder and Steenbergen, 2003a).

Sediment properties

The strength of the dunes depends, besides on their geometry, on the grain size. Grain size distributions have been taken from TAW (1984). This report describes sediment characteristics at 146 locations along the Dutch coast. The sediment samples have been translated in a median value and a standard deviation of the normally distributed grain size. Sediment in the Wadden area is relatively fine with respect to the other parts of the Dutch coast.

Flooding probabilities

The result of the work so far is an annual failure probability per transect. Such a probability is valid for a section with a length of approximately 250 m along the coast. To calculate inundation probabilities for longer sections, the individual failure probabilities of sections should be combined. Flooding probabilities of a certain area are computed by combining the failure probabilities of all flood defences around this area (Jongejan *et al.*, 2013). A dike ring is defined as a single flood cell, which is surrounded by a continuous line of flood defences as dikes, dunes, barriers or high grounds (Kolen *et al.*, 2010).

The combination of failure probabilities to flooding probabilities requires information regarding the spatial correlation between sections. The geometry of neighbouring sections is correlated. The same holds for the hydraulic loads at different locations along the flood defence.

The spatial correlation between sections is taken into account by a correlation length d_c. The smallest correlation length of all random variables involved dominates the extent of correlation between failure probabilities at two locations. The smallest value of d_c equals 300 m (Vrouwenvelder and Steenbergen, 2003a) and belongs to the random variable dune height. This correlation length has been used for all locations along the coast.

When transects are fully independent, an upper bound for the flooding probability is found, equal to the sum of all failure probabilities. A lower bound is found for fully dependent transects. The flooding probability is equal to the failure probability of the weakest spot in that case. The flooding probability P_f is equal to

$$P_f = P(Z_1 < 0 \cup Z_2 < 0 \cup \ldots Z_N < 0) \tag{9}$$

The elementary lower and upper bound of the probability P_f can be written as

$$\max P_i \le P_f \le \Sigma P_i, \tag{10}$$

in which $P_i = P(Z_i < 0)$ is the failure probability of an individual transect, with $i = 1, 2, \ldots N$. Narrower bounds can be found following Ditlevsen (1979), based on partial correlation between the transects.

$$\sum_{i=1}^{N}\left[P_i - \sum_{j<i} P_{ij}\right] \le P_f \le \sum_{i=1}^{N}\left[P_i - \max_{j<i} P_{ij}\right] \tag{11}$$

in which the joint probability

$$P_{ij} = P(Z_i < 0 \cap Z_j < 0) \tag{12}$$

depends on the correlation coefficient ρ_{ij}, which is a function of the ratio between the distance between the two locations ΔL and the correlation length d_c:

$$\rho_{ij} = \rho(Z_i, Z_j) = \exp\left[-\left(\frac{\Delta L}{d_c}\right)^2\right] \tag{13}$$

Because the distance between the JarKus transects is in the same order of magnitude as the correlation length, the correlation between transects is relatively low. This means that the results of the Ditlevsen method approach the elementary upper bound, equal to the sum of all individual failure probabilities.

The flooding probabilities, presented in this article, are valid for the parts of the dike rings adjacent to the North Sea. The contribution of the strengths of other parts of the dike rings (e.g. the flood defences of the Wadden islands, bordering the Wadden sea) is not taken into account.

Results

Calculated failure probabilities

A failure probability has been calculated for almost every JarKus transect along the Dutch coast. The failure probabilities for the first dune row as computed by PC-Ring are presented in Figure 5. Figure 6 shows the probabilities after correction for multiple dune rows.

The difference between these two figures shows the importance of considering the entire dune field. In many

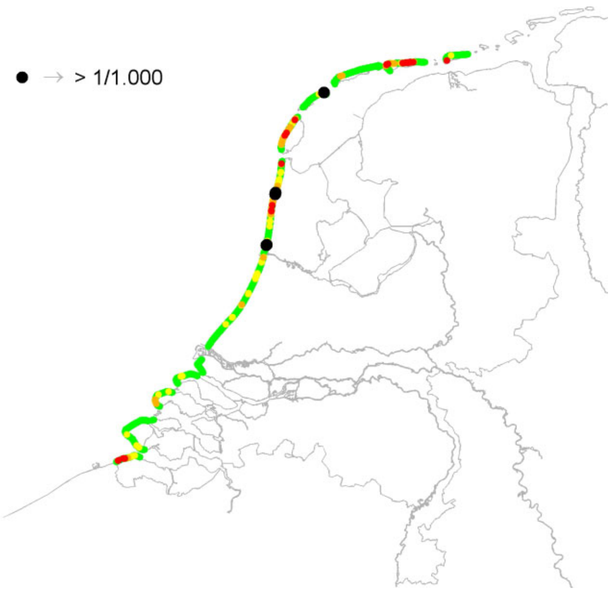

Figure 5 Failure probabilities (1/year) of the first dune row.

Figure 6 Failure probabilities (1/year) of the sea defence, including multiple dune rows.

cases, the failure probability of the first dune row exceeds 1/10 000 per year. At three locations, this value is even larger than 1/1000 per year. A considerable number of these relatively weak spots have disappeared after correction for multiple dune rows. Only for some small, populated areas outside the dike ring (e.g. at Vlieland) the results for the first dune row determine the local flooding probabilities.

Figure 7 shows the distribution of the locations considered over the same classes with failure probabilities as in the

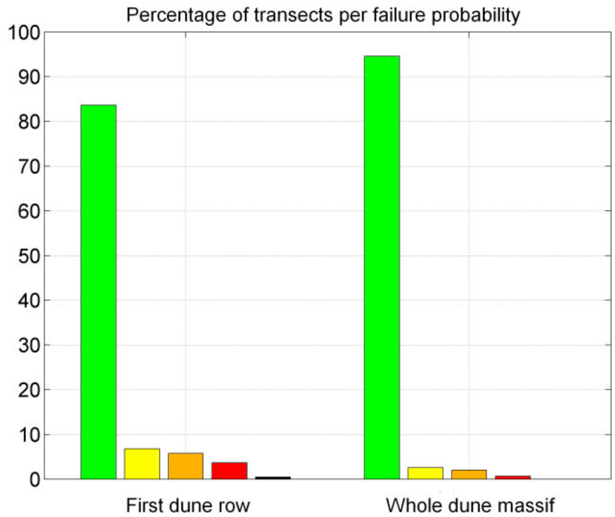

Figure 7 Distribution of locations over calculated failure probabilities in case of the first dune row (left) and the whole dune field (right).

figures above. The figure shows that the annual failure probability of the complete sea defence at 95% of the locations is less than $1/10^6$. The exact value of such large return periods is doubtful, for example, because of the limited accuracy of the extrapolation with which the extremely high hydraulic loads have been predicted. However, it is clear that the vast majority of the coastal defence is very safe. Less than 1% of the locations have an annual failure probability between 1/10 000 and 1/1000 (red markers). Three spots (shown with black dots) are found with a failure probability of the first dune row that exceeds 1/1000 per year.

Dikes versus dunes

As treated before, failure mechanisms of dunes and dikes are totally different. Logically, the same holds for the calculation methods of failure probabilities of both types of sea defence. In general, dunes are stronger than sea dikes in the Netherlands. In the Dutch safety policy, it is generally assumed that the contribution of dune failure to flooding probabilities is negligible. In theory, the study presented in this paper gives arguments to confirm or deny this assumption. However, the different approach of dikes and dunes impedes quantitative comparison.

The failure probability of dikes strongly depends on the critical overtopping discharge q_c, which is in turn dependent on the strength of the inner slope of the dike, which is generally covered with grass. Determining a suitable value for each dike segment fell outside the scope of this study. Therefore, a sensitivity analysis has been carried out, with values of the critical overtopping discharge ranging between 1 and 100 l s/m with a step of 1 l s/m (Figure 8).

The extreme values of the critical overtopping discharge (1 and 100 l s/m) reveal a factor 500 difference in failure probability typically. This means that this parameter dominates the amount of strength of the dike. However, the results for 10 l s/m serve as a starting point, based on recent (not officially reported) findings in the Dutch project VNK2.

Calculated flooding probabilities

Flooding probabilities have been calculated for the 14 dike ring areas (see Figure 1). This subdivision in dike rings is based on earlier studies in the Netherlands, which provided a basis for determining flooding probabilities, based on an economic cost–benefit analysis.

Failure probabilities at individual locations along the coast have been combined to flooding probabilities per dike ring. The mean value of the upper and lower Ditlevsen bound is presented, with the remark that the two Ditlevsen bounds are in many cases nearly identical. The result is shown in Figure 9, valid for the dune fields as a whole and the dikes with a critical overtopping discharge of 1, 10 and 100 l s/m. Also included is a set of flooding probabilities that corresponds with the situation in which all flood defences exactly meet the current safety standards. These values result from the project Flood Protection in the 21st Century (with the Dutch acronym WV21), with which the Dutch government strives for an update of the Dutch safety standards (Kind, 2010, 2011; Beckers and De Bruijn, 2011; De Bruijn and Van der Doef, 2011). The figure shows that (for 10 l s/m) the calculated flooding probability exceeds the value from WV21 for only two dike rings.

Figure 9 also underlines the importance of the critical overtopping discharge. In three out of four dike rings, the failure probabilities of the dike stretches dominate the flooding probabilities of the area behind the flood defences. Only in Walcheren (see Figure 1), the dikes are stronger than the dunes because the sea dikes near West-Kapelle and Vlissingen are very strong.

In earlier studies of Den Heijer *et al.* (2012) and Van Balen *et al.* (2012), the analysis was limited to the first dune row. The present study clearly shows that for large parts of the

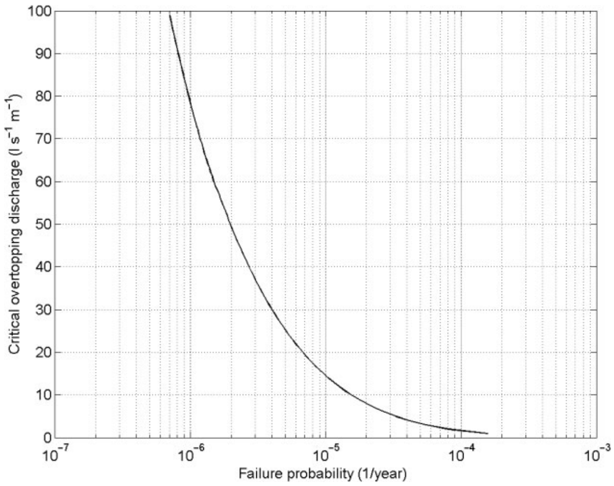

Figure 8 Example of the failure probability of a dike as a function of the critical overtopping discharge (valid for transect 7002225, intersecting the Hondsbossche sea defence).

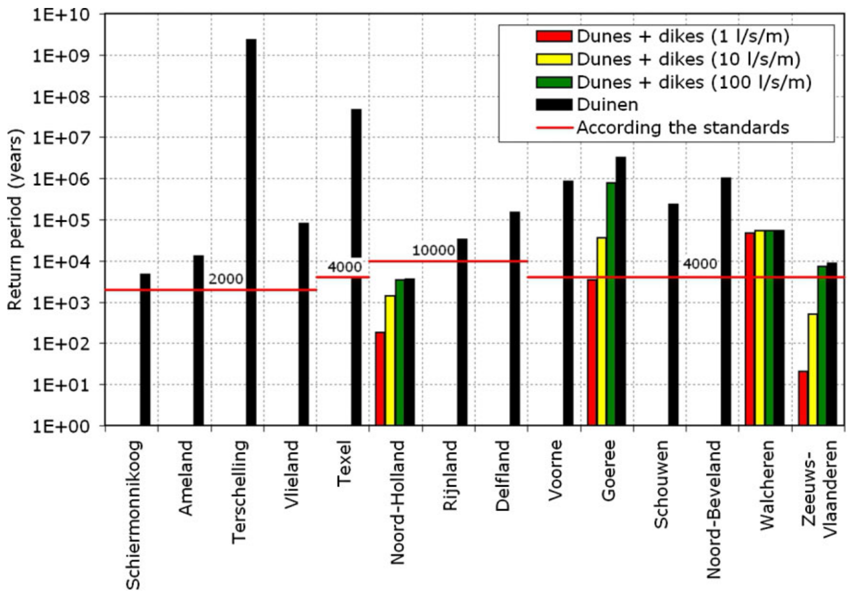

Figure 9 Return periods (years) of flooding per dike ring when considering only dunes (black) or the combination of dunes and dikes with different critical overtopping discharges (other colours). Also a set of flooding probabilities is included with red lines, belonging to the situation that all water defences exactly meet the current safety standards.

Dutch coast, the first dune row is of minor importance in view of flood risks. Furthermore, Den Heijer *et al.* (2012) and Van Balen *et al.* (2012) did not take failure of dikes into account. The present study makes clear that in most cases, the failure probabilities of the dike stretches dominate the flooding probability of the area behind the flood defences. Inclusion of multiple dune rows and dikes is essential to relate flooding probabilities to consequences of flooding and subsequently, to calculate optimal failure probabilities from an economical or social perspective.

Discussion

This study describes an enhanced methodology to assess the safety of a coastal flood defence using flooding probabilities. Calculated flooding probabilities can be very useful for coastal managers (Giardino *et al.*, 2014). Flood risks are obtained by multiplying the flooding probabilities with the consequences of an inundation of the area considered, which depend for example on inundation depth, economic value, population density and possibilities for evacuation. The use of flood risks enables accurate trade off between avoided damage due to floods and investments to reinforce the flood defences (Vrijling, 2001). Also the basis of the Dutch safety policy is currently switching from normative hydraulic loads to a flood risk approach.

The probabilistic techniques used in this paper are generally applicable, but the application of the empirical dune retreat concept is limited to sandy coasts, resembling the situation as present along large parts of the Dutch coast. The dune retreat module is not able to simulate dune erosion of complex dune geometries accurately. This is, for example, the case at strongly curved coasts, dune areas containing hard structures, transitions between soft and hard sea defences, and dune areas with multiple dune rows. This paper describes a parametric approach to deal with multiple dune rows, but the results of this approach only offer a right order of magnitude.

From an international point of view, a more generally applicable instrument is desirable in order to make calculations for any type of sandy sea defences possible. Limited applicability is inherent to empirical modelling techniques, and process-based modelling is required to overcome this limitation. A promising process-based model is the two-dimensional XBeach model, described in Roelvink *et al.* (2009), Van Dongeren *et al.* (2009) and Brandenburg (2010). This model shows results with significant predictive skill under storm conditions (McCall *et al.*, 2010) but is computationally much more expensive, and therefore less suitable for application in combination with probabilistic methods as used in this study. However, the process-based modelling techniques are at the moment not ready for calculations in a fully probabilistic context. More research is required in this context.

The current paper presents failure probabilities and flooding probabilities, valid for the geometry of the Dutch coast in 2011. Gradual coastline evolution in front of the dune is not taken into account. This means that the numbers presented should be considered as a static view of the safety of the Dutch coast in 2011. Van Balen *et al.* (2012) present an analysis of time series for the period 1965–2010 with failure probabilities of the dynamic part of the coast, consisting of the foreshore, the beach and the first dune row. With the latter type of information, a prediction can be made for the future development of the failure probabilities. When a location has a relatively high failure probability, combined with a significant increase in time, reinforcements of the flood defence might be necessary.

Additionally, Van Balen *et al.* (2012) relate the trends in the time series with failure probabilities to the nourishments carried out, and demonstrate the (differences in) effectiveness of beach nourishments and shoreface nourishments in maintaining the safety of the sandy coast. Vuik *et al.* (2012) have extended that research, including the effect of storminess, and investigating delayed effects of shoreface nourishments. The study, presented in the current article, shows the relative safety of different stretches of the coastline, which might provide arguments to prioritise the strengthening of flood defences. Vuik *et al.* (2012) provide guidance in determining an appropriate nourishment type (dune, beach, shoreface) and nourishment volume to obtain a desired decrease in failure probabilities. Both studies complement each other.

Conclusions

This paper gives a description of an advanced method to assess the safety level of a flood defence, and shows the results of this method for the Dutch coast with a high spatial resolution.

The study leads to the following conclusions, regarding the methodology:

1. Both dikes and dunes have been considered. For dikes, the analysis is limited to overflow and overtopping. For dunes, dune erosion is taken into account, which is the only relevant failure mechanism for this type of flood defences.
2. Failure probabilities at individual locations have been combined to flooding probabilities per dike ring area, as explained in the methodology description.
3. The probabilistic techniques used in this paper are generally applicable, but the application of the empirical dune retreat concept is limited to sandy coasts, resembling the Dutch situation. Process-based modelling in

combination with probabilistic calculation techniques is required in order to make calculations for any type of sandy sea defences possible.

The study leads to the following conclusions, regarding the results:

4. The vast majority of the Dutch coastal defence is very safe. The failure probability of the Dutch sea defence is less than $1/10^6$ per year at 95% of the locations, and less than 1% of the locations have a failure probability between 1/10 000 and 1/1000 per year.
5. This study shows that Dutch dunes generally provide a higher degree of safety against flooding than sea dikes. Relatively weak spots are mainly found at sea dikes and at transitions between soft and hard sea defences.

Acknowledgement

Rijkswaterstaat Waterdienst funded the study.

References

Allsop W., Kortenhaus A. & Morris M. (2007) Failure mechanisms for flood defence structures. FLOODsite project, Report Number T04-06-01.

Beckers J. & De Bruijn K.M. (2011) Slachtofferrisico's door overstromingen. Deltares report 1204144-005, Delft. Report in Dutch.

Booij N., Ris R.C. & Holthuijsen L.H. A third-generation wave model for coastal regions: 1. Model description and validation. *J Geophys Res* 1999, **104**, (C4), 7649–7666.

Brandenburg P.G.F. (2010) Scale dependency of dune erosion models: performance assessment of the DUROS and XBeach model for various experiment scales. MSc Thesis, University of Twente.

De Bruijn K.M. & Van der Doef M. (2011) Gevolgen van overstromingen; informatie ten behoeve van het project Waterveiligheid in de 21e eeuw. Deltares report 1204144-004, Delft. Report in Dutch.

De Graaf H.J.C., Oude Elberink S.J., Bollweg A.E., Brügelmann R. & Richardson L.R.A. (2003) Inwinning droge JarKus profielen langs Nederlandse kust. Report AGI-GAM-2003-40, Rijkswaterstaat, the Netherlands. Report in Dutch.

De Ronde J.G., van Marle J.G.A., Roskam A.P. & Andorka Gal J.H. (1995) Golfrandvoorwaarden langs de Nederlandse kust op relatief diep water. Tech. Rep. 95.024, RIKZ. Report in Dutch.

Dean R.G. (1973) Heuristic model of sand transport in the surf zone. Proc. of engineering dynamics in the surf zone, Institute of Engineers, Australia, 208–214.

Den Heijer C., Baart F. & Van Koningsveld M. Assessment of dune failure along the Dutch coast using a fully probabilistic approach. *Geomorphology* 2012, **143/144**, 95–103.

Ditlevsen O. Narrow reliability bounds for structural systems. *J Struct Mech* 1979, **7**, (4), 453–472.

Edelman T. (1968) Dune erosion during storm conditions. Proc., 11th Int. Conf. on Coastal Engineering, 719–722.

ENW (2007) Technisch Rapport Duinafslag. Expertisenetwerk Waterveiligheid. Report in Dutch.

Giardino A., Santinelli G. & Vuik V. Coastal state indicators to assess the morphological development of the Holland coast due to natural and anthropogenic pressure factors. *Ocean Coast Manage* 2014, **87**, 93–101.

Hohenbichler M., Gollwitzer S., Kruse W. & Rackwitz R. New light on first- and second-order reliability methods. *Struct Saf* 1987, **4**, (4), 267–284.

Jongejan R., Maaskant B., ter Horst W., Havinga F., Roode N. & Stefess H. The VNK2-project: a fully probabilistic risk analysis for all major levee systems in the Netherlands. *IAHS Publ* 2013, **357**, 75–85.

Kind J. (2010) Cost-benefit analysis for flood protection standards in the Netherlands. CIS Working Group F Thematic Workshop 'Floods and Economics: appraising, prioritizing and financing flood risk management measures and instruments'.

Kind J. (2011) Maatschappelijke kosten-batenanalyse Waterveiligheid 21e eeuw. Deltares report 1204144-006, Utrecht. Report in Dutch.

Kolen B., Slomp R., Van Balen W., Terpstra T., Bottema M. & Nieuwenhuis S. (2010) Learning from French experiences with storm Xynthia; damages after a flood. Rijkswaterstaat, Centre for Water Management and HKV consultants, the Netherlands, ISBN 978-90-77051-77-1.

Kraus N.C. *et al.* (1991) Evaluation of beach erosion and accretion predictors. Coastal Sediments, Seattle, USA, 572–587.

Kriebel D.L. *et al.* (1991) Engineering methods for predicting beach profile response. Coastal Sediments, Seattle, USA, 557–571.

Lassing B.L., Vrouwenvelder A.C.W.M. & Waarts P.H. Reliability analysis of flood defence systems in the Netherlands. In: P.H.A.J.M. van Gelder & T. Bedford, eds. *Safety and reliability, proc. of the ESREL conference 2003*. Maastricht, the Netherlands: Swets & Zeitlinger, Lisse, 2003, pp. 1005–1014.

Leendertse J.J. *Aspects of SIMSYS2D*. Santa Monica, California: Rand Corporation, 1987.

Mai Van C., Van Gelder P.H.A.J.M. & Vrijling J.K. (2007) Failure mechanisms of sea dikes; inventory and sensitivity analysis, COPRI-ASCE, Coastal Structures International Conference, Venice, Italy.

McCall R.T., Van Thiel de Vries J.S.M., Plant N.G., Van Dongeren A.R., Roelvink J.A., Thompson D.M. & Reniers A.J.H.M. Two-dimensional time dependent hurricane overwash and erosion modeling at Santa Rosa Island. *Coastal Engineering* 2010, **57**, (7), 668–683.

Minneboo F.A.J. (1995) Jaarlijkse Kustmetingen. Report RIKZ-95.022, Rijkswaterstaat RIKZ, the Netherlands. Report in Dutch.

Otten C.J. (1985) Ontwikkelingen in de Jaarlijkse Kustmetingen en het verwerkingssysteem 1977–1985. Nota GWIO-86.001, Rijkswaterstaat, the Netherlands. Document in Dutch.
Pye K. & Tsoar H. *Aeolian sand and sand dunes*. Berlin Heidelberg: Springer, 2009.
Rackwitz R. & Flessler B. Structural reliability under combined random load sequences. *Comput Struct* 1978, **9**, (5), 489–494.
Ris R.C., Holthuijsen L.H. & Booij N. A third-generation wave model for coastal regions: 2. Verification. *J Geophys Res* 1999, **104**, (C4), 7667.
Roelvink D., Reniers A., van Dongeren A., Van Thiel de Vries J., McCall R. & Lescinski J. Modelling storm impacts on beaches, dunes and barrier islands. *Coast Eng* 2009, **56**, 1133–1152.
Roskam A.P. & Hoekema J. (1996) Randvoorwaarden voor golfperioden langs de Nederlandse kust. Tech. Rep. 96.019, RIKZ. Report in Dutch.
Roskam A.P., Hoekema J. & Seijffert J.J.W. (2000) Richtingsafhankelijke extreme waarden voor HW-standen, golfhoogten en golfperioden. Report RIKZ-2000.040, Rijkswaterstaat RIKZ, the Netherlands. Report in Dutch.
Steenbergen H.M.G.M. & Vrouwenvelder A.C.W.M. (2007) Theoriehandleiding PC Ring, deel A, Mechanismenbeschrijving. TNO report, 2007-D-Rxxxx/A. Report in Dutch.
Steetzel H. (1993) Cross-shore transport during storm surges. PhD Thesis, Delft University of Technology, Delft, the Netherlands.
Stelling G.S. (1983). On the construction of computational methods for shallow water flow problems. PhD Thesis, Delft University of Technology, Delft, the Netherlands.
TAW (1984) Leidraad voor de beoordeling van de veiligheid van duinen als waterkering. Tech. rep., Technical Advisory Committee on Water Defences,'s-Gravenhage, the Netherlands.
TAW (2002) Technical report wave run-up and wave overtopping at dikes. Technical Advisory Committee on Flood Defence, Delft, the Netherlands.
Van Balen W., Vuik V. & Van Vuren S. Probabilistic assessment of safety against dune retreat along the Northern-Holland coast. In: W.M. Kranenburg, E.M. Horstman & K.M. Wijnberg, eds. *NCK-days 2012: Crossing borders in coastal research*. Enschede, the Netherlands: University of Twente, 2012, pp. 201–205. http://proceedings.utwente.nl/165/.
Van de Graaff J. (1988) Sediment concentration due to wave action. PhD Thesis, Delft University of Technology.
Van de Graaff J. (2008) Dune Erosion. ENCORA Coastal Portal [online]. http://www.coastalwiki.org/coastalwiki/Dune_erosion [accessed 28 August 2012].
Van der Meer J.W. (1993) Golfoploop en golfoverslag bij dijken. WL | Delft Hydraulics, summary, H 638. Report in Dutch.
Van Dongeren A., Bolle A., Roelvink J., Vousdoukas M.I., Plomaritis T., Williams J., Armaroli C., Idier D., van Geer P. & Van Thiel de Vries J. MICORE: dune erosion and overwash model validation with data from nine European field sites. *AGU Fall Meet Abstr* 2009, **1**, 1–15.
Van Gent M.R.A., Van Thiel de Vries J.S.M., Coeveld E.M., De Vroeg J.H. & Van de Graaff J. Large-scale dune erosion tests to study the influence of wave periods. *Coast Eng* 2008, **55**, 1041–1051.
Van Koningsveld M., de Boer G.J., Baart F., Damsma T., den Heijer C., van Geer P. & de Sonneville B. (2010) OpenEarth; inter-company management of data, models, tools & knowledge. Proceedings WODCON XIX Conference, Beijing, China.
Van Rijn L. (2010) Modelling of sandy beach and dune erosion. Concepts and Science for Coastal Erosion Management, deliverable D13C.
Van Santen R., Steetzel H. & Van Thiel de Vries J.S.M. (2012) Modelling storm impact on complex coastlines; Westkapelle, the Netherlands. Coastal Engineering Proceedings, 1(33), management-52.
Van Thiel de Vries J.S.M., Van Gent M.R.A., Walstra D.J.R. & Reniers A.J.H.M. Analysis of dune erosion processes in large-scale flume experiments. *Coast Eng* 2008, **55**, 1028–1040.
Vellinga P. (1986) Beach and Dune Erosion during Storm Surges. PhD Thesis, Delft University of Technology.
Vrijling J.K. Probabilistic design of water defense systems in The Netherlands. *Reliabil Eng SystSaf* 2001, **74**, (3), 337–344.
Vrouwenvelder A.C.W.M. & Steenbergen H.M.G.M. (2003a), Theoriehandleiding PC Ring, deel B, Statistische modellen. TNO report, 2003-CI-R0021, April 2003. Report in Dutch.
Vrouwenvelder A.C.W.M. & Steenbergen H.M.G.M. (2003b), Theoriehandleiding PC Ring, deel C, Rekentechnieken. TNO report, 2003-CI-R0022, April 2003. Report in Dutch.
Vuik V., Paarlberg A.J. & Van Balen W. (2012) Indicatoren voor kustlijnzorg; analyse van stormen, suppleties en kustveiligheid. HKV report pr2063.30. Report in Dutch.
Wouters W. & Bollweg A. AHN: actual height model of The Netherlands; a detailed elevation model using airborne laser altimetry. *Geo Informatics* 1998, **9**, 6–9.