Appendix 6 The Netherlands

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1.   General description

1.1   Flood-prone areas

The coastline of the Netherlands is approximately 350 kilometres long. The country is densely populated (more than 15 million people, or an average population density of more than 400 people per km^2). About a quarter of the Netherlands is below mean sea level. Without flood protection structures, about two-thirds of the country (25,000 km^2) would be flooded during storm surges at sea or high discharges in the rivers. Protection against flooding is an important task, and is provided by an extensive system of primary flood protection structures.

1.2   Main threats

Very large, densely populated polders are at risk of flooding. Failure of the sea defence would have devastating consequences. The following threats can be identified in the coastal area:

- coastal flooding due to overtopping or failure of flood embankments or barriers;
- coastal flooding due to excessive dune erosion.

Figure 1 The Netherlands without flood protection structures

1.3   Types of coastal protection

The area protected by a linked system of primary flood protection structures is called a dike ring area (dijkringgebied). The flood protection structures around a dike ring area can be divided into sections, in which load and strength characteristics are comparable. These sections can consist of dikes, dunes, structures or high grounds. High grounds are areas which are high enough not to need protection against flooding. Together these sections ensure the safety of the area, both on the coast and inland.

The major part (70%) of primary coastal defences consist of natural dunes. In other places coastal dikes, seawalls, dams and barriers provide the required safety.

2.   Organisational framework

2.1   Organisations / authorities involved

The following organisations are involved in flood and coastal defence:
- Ministry of Transport, Public Works and Water Management;
- Provinces;
- Water boards;
- Municipalities.

2.2   Legislation

Statutory safety standards for all dike ring areas are given in the Flood Protection Act (Ministry of Transport, Public Works and Water Management, 1996). These safety standards have been legally anchored to guard against waning public awareness of flood risks as the date of the last flood becomes ever more distance. The Flood Protection Act requires the manager of a flood protection structure to check the structure every 5 years and to report its status in relation to its particular the safety standards.

2.3   Responsibilities

Responsibility for flood defence and coastal protection is divided among three forms of government: the Ministry of Transport, Public Works and Water Management, the provinces and the water boards. The municipalities play their part in town and country planning (as a representative of other interests concerning flood protection structures, such as traffic and quality of life) and in the case of a threatening calamity.

The Ministry plays a central role in setting coastal protection policy. Implementation is chiefly carried out by the water boards. A water board is what is called a functional form of government, oriented to water management and flood protection structures. The provinces (12 in total) supervise municipalities as well as water boards. The provinces play a key-role with regard to the interaction between water management and spatial planning.

In addition to the description above is the Ministry responsible for :
* water management in coastal waters;
* coastal protection, i.e. maintaining the coastline of at the 1990 position;
* management of some flood protection structures, such as the storm surge barriers.

2.4   Financing arrangements

The Ministry finances it own activities by a general taxation. The activities of the water boards are partly paid for by a specific local taxation. This taxation is directly related to the asset value of the protected properties. The higher this value, the higher taxation will be. On the other hand, a higher asset value will give its owner a larger influence in the water board. For the activities of the water boards a distinction is made between construction and maintenance :
* 20% of the construction costs is paid for by the water boards and 80% is subsidised by the national government via the provinces;
* 80% of the maintenance costs is paid by the water boards and 20% is subsidised by the national government via the provinces.

The financial activities of the provinces are very limited, except for the above mentioned transfer of subsidies.

2.5   Flooding and coastal defence policy

The flooding and coastal defence policy is aimed at maintaining two separate conditions :
* the 1990 position of the coastline;
* the legally prescribed safety standards of all primary flood protection structures.

Before 1990 large sections of the Dutch coast face structural erosion. In 1990 it was decided to combat this erosion by keeping the coast line (defined at low water level) at the position of 1990. Beach nourishment is main measure to accomplish this task. Maintaining the coast line this way yields :
* a both natural and flexible protection at reasonable costs (30 million euros in 2000);
* a fixed reference for assessing and maintaining the safety against flooding of the hinterland offered by the dunes. This item is necessary to allow the separate activities and responsibilities of the Ministry and the water boards in the coastal zone.

The primary flood protection structures protect the low lying part of the country against coastal and fluvial flooding. Safety standards for these defences are laid in law. This Flood Protection Act also ensures a mandatory 5-year safety assessment of each structure.

3. Risk assessment

3.1 Risk assessment methods

To determine the required height of dikes, the traditional method used in the Netherlands until well into the last century was to add a margin of 0.5 to 1 meter to the highest known water level. The Delta Commission, set up shortly after the devastating floods of 1953, established the basis for the current safety standards for protection against flooding in 1956. In doing so, this commission initiated the use of risk assessment methods in civil engineering.

The starting point proposed by the Delta Commission was to establish a desired safety level for each dike ring area or polder. This safety level would be based on the costs of dike construction and on the possible damage which could be caused by flooding. This economic analysis led to an 'optimum' safety level expressed as the probability of failure for coastal dikes. In practice, however, the safety level was expressed as the return period of the water level, as it was the most dominant hydraulic load. One of the main reasons the description of safety standards was simplified was the lack of knowledge of how to describe the failure rate of a dike with sufficient accuracy.

The economic analysis was used to differentiate safety standards according to expected damage in each polder. A safety standard were established for each dike ring area. This standard is expressed as the mean annual probability that the prescribed flood level will be exceeded. At present the safety standards range from 1/1,250 to 1/10,000 a year, depending on the area's economic activities, population size, and the nature of the threat (fluvial or coastal).

3.2 Application of risk assessment methods

The method of the Delta Commission is today still the starting point with regard to the practical application of risk assessment. The method of the Delta Commission results in engineering safety standards for flood protection structures.

In addition to this application risk assessment methods were developed and applied in more specific flood protection projects or items :
- the probabilistic design of storm surge barriers, like the barriers in the Eastern Scheldt of near Rotterdam;
- probabilistic design of dikes and dunes.

The practical application of these methods is laid down in guidelines.

4. Safety levels

4.1 Background

Based on a national risk assessment safety standards for the primary flood protection system have been derived. These standards range from 1/1,250 to 1/10,000 a year, depending on the area's economic activities, population size, and the nature of the threat (fluvial or coastal). The minimum safety of 1/1,250 a year holds for river areas. For coastal areas the minimum safety is 1/2,000 a year. These standards were established in legislation In 1996 with the Flood Protection Act (Ministry of Transport, Public Works and Water Management 1996). The flood levels associated with safety standards are updated every five years to accommodate sea-level rise and recent technical developments.

The Netherlands
Safety Standard
per Dike-ring area

Legend

12   number of dike-ring area

⬛ 1/10,000 per year
◻ 1/4,000   per year
◻ 1/2,000   per year
◻ 1/1,250   per year

◻ high grounds (also outside The Netherlands)
— primary water defence outside The Netherlands

North Sea

Germany

Belgium
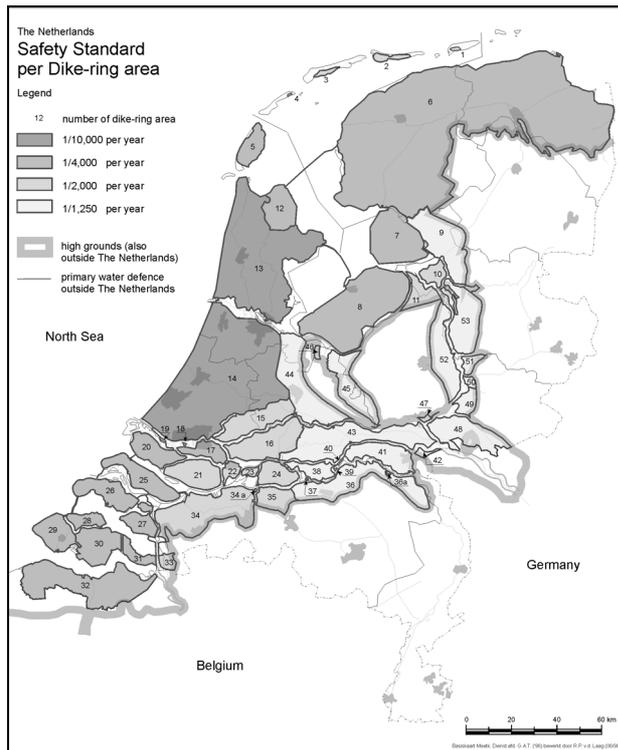
Figure 2 Dikering areas and safety standards

4.2 Definition

Current safety standards for flood protection structures are expressed as return periods of extreme water levels, which the flood protection structure must be able to withstand.

4.3 Application

Hydraulic boundary conditions (flood level, wind and wave conditions) are applied to statutory safety standards. In designing a dike, a certain margin is added to the flood level, depending on wind and wave conditions. The object of this margin is to ensure that each individual dike section is sufficiently high to withstand the prescribed flood levels and associated hydraulic loads. Technical guidelines provide the engineer with sufficient information to calculate the required margins and other structural aspects of dike design.

This situation reflects standard practice in the Netherlands. However, new techniques and developments may be introduced in the standard of practice in the near future.

5. Technical models and criteria

5.1 Hydraulic boundary conditions

Hydraulic boundary conditions are calculated and issued by the Ministry. Generally, the extreme water levels are issued in combination with associated wind or wave loads.

5.2 Wave run-up, wave overtopping

The structural design of a coastal dike mainly involves determining the required crest level, the stability of the revetments, geotechnical stability and the reliability of movable objects intersecting the dike, such as sluices. Although the crest level is not the single mandatory safety feature of the dike, this study will be limited to this aspect. Other safety features can be studied in follow-up research.

The crest level of dikes needs to be sufficient to prevent excessive quantities of water from overtopping the structure. Two situations may be important:
- overtopping without dike failure, leading to excessive quantities of water in the polder;
- overtopping leading to dike failure, due to erosion or geotechnical instability or infiltration of the inner slope.

Whether the first situation is important depends very much on local conditions. This study will focus on the second situation, which has a more general character. Two design procedures for determining required crest level are available: a traditional method which has been applied to the majority of coastal dikes, and a more sophisticated method, which was developed quite recently. Both methods are described in this section.

5.2.1 Traditional procedure

The required crest level is calculated using the following procedure:
- crest level = design water level + wave run-up + additional margins
- the design water level is the flood level with the legally prescribed return period; the flood level is

updated every five years

- wave run-up: $z_{2\%} = 8H_s \tan\alpha$

$z_{2\%}$	=	2% wave run-up (m)
H_s	=	significant wave height (m)
$\tan\alpha$	=	steepness of the outer slope (-)

- additional margins compensate for sea-level rise, settlement and seiches

sea-level rise	=	20 cm a century
settlement	=	based on geotechnical calculation
seiches	=	ranging from 10 cm to 80 cm, depending on local situation

The significant wave height is traditionally determined by calculating the expected significant wave height during the design flood. For the calculation of wind-generated waves the Delta Commission issued design wind velocities ranging from 30 to 35 m/s. Swell is related to the flood level. The peak period or the shape of the wave-energy spectrum is not taken into account. The design lifetime of a coastal dike is generally 50 years, so in most cases the margin for sea-level rise is 10 centimetres. The required margin for settlement is based on a prediction of the settlement during 50 years.

5.2.2 Sophisticated procedure

5.2.2.1 Wave run-up

This procedure contains a more sophisticated wave run-up formula, while other items remain unchanged.

- wave run-up : $z_{2\%} / (\gamma H_s) = c_0 \xi_{op}$ for $\xi_{op} \leq p$

                 $z_{2\%} / (\gamma H_s) = c_1$ for $\xi_{op} \geq p$

Where:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi}{g}\dfrac{H_s}{T_p^2}}} = \text{surf similarity parameter (-)}$$

T_p	=	wave period (s)

γ is a reduction factor which accounts for the effects of friction, foreshores, angular wave attack and the presence of a berm: $\gamma = \gamma_f \gamma_h \gamma_\beta \gamma_b$. The reduction factor γ_f varies between 0.5 for rock slopes with two or more layers, 0.6 for rock slopes with one layer, 0.95 for grass and 1.0 for smooth impermeable slopes. The reduction factor γ_h can be approximated by $\gamma_h = H_{2\%}/(1.4\,H_s)$. Van der Meer (1997) proposed not to use the reduction factor for foreshores. The reduction factor γ_β can be approximated by $\gamma_\beta = 1 - 0.0022 . \beta$ ($\beta \leq 80^0$).

Due to the presence of a berm in the seaward slope, the slope angle in the surf-similarity parameters is not uniquely defined. The influence of a berm can be taken into account by replacing $\tan\alpha$ in the surf-similarity parameter by $\tan\alpha = r_{dh}\tan\psi + (1 - r_{dh})\tan\psi_{eq}$ where the weight factor r_{dh} depends on the position of the berm: $r_{dh} = 0.5(d_h/H_s)^2$ where d_h is the absolute value of the average depth of the berm with respect to the still-water level ($-1.0 \leq d_h/H_s \leq 1.0$); $\tan\psi$ describes the slope above and below the berm and $\tan\psi_{eq}$ describes an equivalent slope angle around the berm defined as: $\tan\psi_{eq} = 2/(2\cot\psi + B/H_s)$ where B is the berm width. The reduction factor $\gamma_b = \tan\alpha/\tan\psi$ ($\gamma_b \leq 0.6$) is used.

Since limited information is available on combinations of reductions, a maximum total reduction factor $\gamma = \gamma_f \gamma_h \gamma_\beta \gamma_b = 0.4$ was proposed. The coefficients c_0, c_1 and p were originally set at 1.5, 3 and 2 respectively, based on many physical-model tests. For design purposes, somewhat safer values were advised: 1.6, 3.2 and 2 respectively.
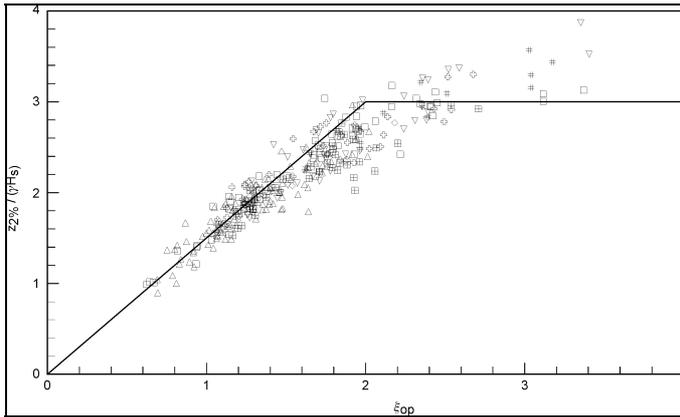
Figure 3 Wave run-up

5.2.2.2 Wave overtopping

The sophisticated procedure can be extended by introducing wave overtopping instead of wave run-up. Again, the other items of the procedure remain more or less unchanged. The required crest level is now calculated using the following procedure:

- crest level = design water level + margin for wave overtopping + additional margins

- overtop: $Q = c_a \sqrt{\cot \alpha}\ \xi_{op}\ \exp\left(-c_b \dfrac{R_c}{\gamma H_s \xi_{op}}\right)$

The maximum overtopping rate is: $Q_{MAX} = c_c \exp\left(-c_d \dfrac{R_c}{\gamma H_s}\right)$

Where:

$Q = \dfrac{q}{\sqrt{gH_s^3}}$ = non-dimensional (critical) overtopping discharge

q   = overtopping discharge (m^3/s/m)
R_c = crest elevation with respect to the design water level (m)

The reduction factor γ accounts for the effects of friction, foreshores, angular wave attack and the presence of a berm: $\gamma = \gamma_f\,\gamma_h\,\gamma_\beta\gamma_b$. The reduction factors can be calculated in the same way as for wave run-up, except for the influence of the angle of wave attack, for which a slightly different formula was proposed: $\gamma_\beta = 1-0.0033.\beta$ ($\beta \le 80^0$).

As a best-fit line through the data points the coefficients c_a, c_b, c_c and c_d were set at 0.06, 5.2, 0.2 and 2.6 respectively. For design purposes, somewhat safer values were advised: 0.06, 4.7, 0.2 and 2.3 respectively.

The design criteria used for overtopping depend on the quality of the inner slope. Critical discharges are:
- 0.1 l/m/s, with no specific demands on the inner slope from erosion or infiltration
- 1 l/m/s/, which requires high-quality clay and grass cover and a slope not steeper than 1:2
- 10 l/m/s, which requires very high-quality clay and grass cover with a slope not steeper than 1:3.

For coastal dikes the application of 10 l/m/s is being considered at present. This criterion is used for river dikes where wave periods are typically 3 to 4 seconds. For coastal dikes with wave periods of up to 10 seconds or more, this criterion (applied to the average overtopping discharge) may lead to enormous overtopping volumes per wave. Therefore it is considered safe to limit overtopping rates for coastal dikes to 1 l/m/s. The traditional 2% wave run-up criterion leads to overtopping rates of 3 to

5 l/m/s.

5.2.3 Example
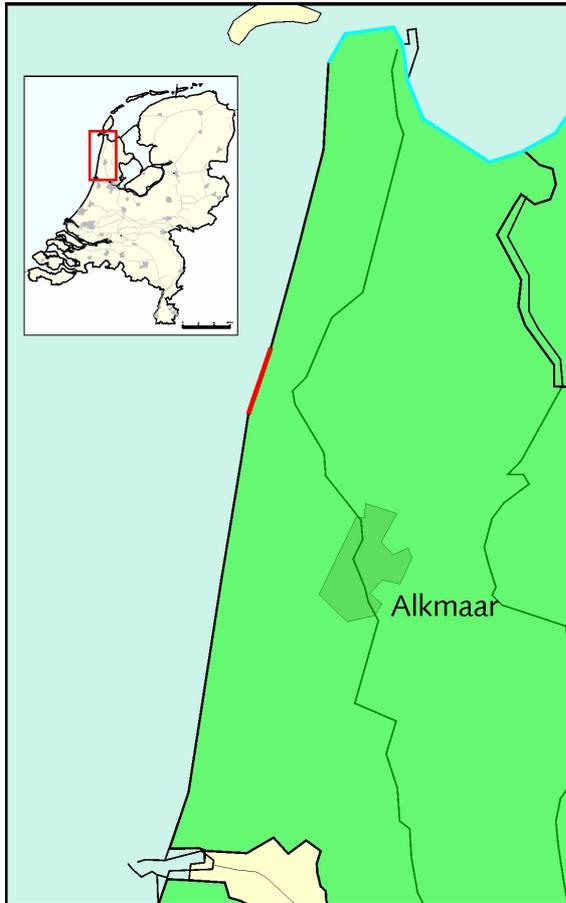
Figure 4 Location of Pettemer Zeewering dike

The Pettemer Zeewering dike has been selected as an example for the calculation of required crest level of a coastal dike. This dike is located on the Dutch coast as shown in figure 4.

For this dike ring area or polder the safety standard is 1/10,000 per year. The prescribed hydraulic boundary conditions for this location are:

- flood level: MSL + 4.70 meters;
- wave height: 4.70 meters.

These boundary conditions are derived only partially through probability. The water level has a return period of 10,000 years. The wave height is the expected wave height at the toe of the dike associated with this water level. It is not a design combination derived using joint statistics.

Traditional procedure
The traditional design procedure is applied using the straightforward formula: $h_{crest} = h + 8H_s \tan\alpha$. The Pettemer Zeewering has a berm on the outer slope with different slope angles above and below it. For this situation, an equivalent slope is determined using the following method, according to figure 5.
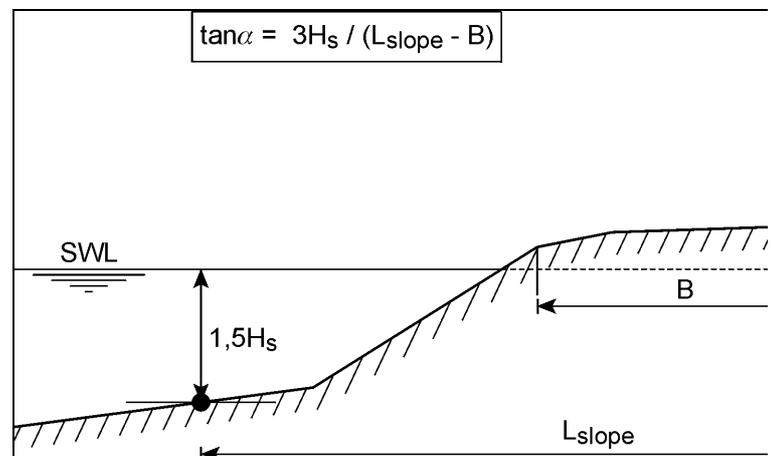
Figure 5 Equivalent slope

A cross section of the Pettemer Zeewering is shown in figure 6. Above the berm the outward-facing slope is 1:3.19 and below the berm the slope is 1:4.12. The berm is approximately at storm surge level. This leads to an equivalent slope of 1:4.95.
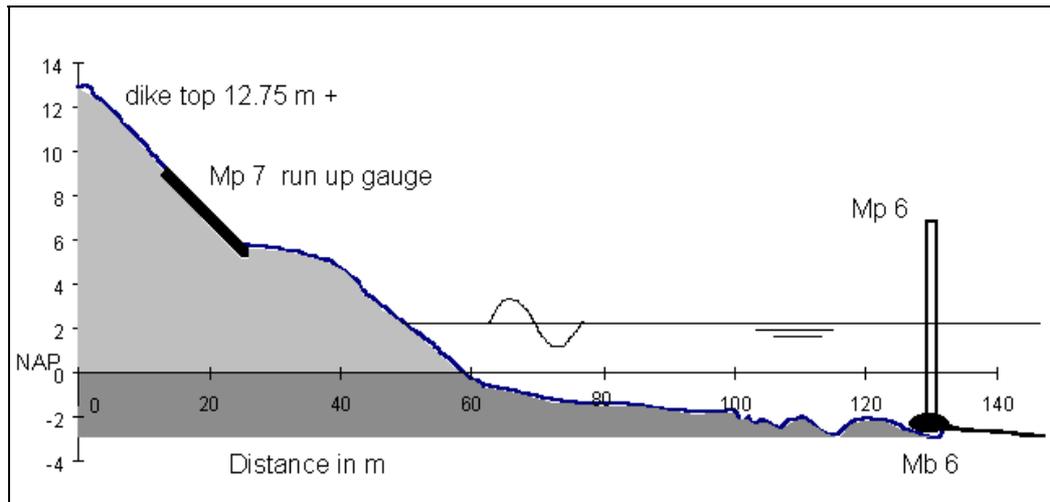
Figure 6 Cross-section of Pettemer Zeewering

Using this slope and the prescribed boundary conditions, the required crest level is:
- $h_{crest} = h + 8H_s \tan\alpha$
  h_{crest} = 4.70 m + 8*4.70 m*(0.202) = 12.3 m.

The additional margins for sea-level rise. land subsidence (0.1 m) and seiches (0.15 m) lead to a required crest level of MSL+12.55 meters. The present crest level is MSL+12.75 meters, which appears to be sufficient for the moment.

Sophisticated procedure for calculating wave run-up
The more sophisticated procedure using the wave run-up formula yields quite different results. The effect of the peak wave period is significant. The peak period used in this example is 12 seconds. The required crest level is:
- $h_{crest} = h + c_0\, \xi_{op}\, \gamma\, H_s$
  h_{crest} = 4.70 m + 1.6*1.39*0.82*4.70 m = 13.30 m

The additional margins for sea-level rise, land subsidence (0.1 m) and seiches (0.15 m) lead to a required crest level of MSL+13.55 meter. The present crest level is MSL+12.75 meter, which now seems to be insufficient.

Sophisticated procedure for calculating wave overtopping
The more sophisticated procedure using wave overtopping formula yields different results. Using the same peak period, the required crest level depends on the overtopping criterion.
The required crest level is:
- 0.1 l/m/s   : h_{crest} = 4.70 m + 12.43 m = 17.13 m.
- 1.0 l/m/s   : h_{crest} = 4.70 m + 9.82 m = 14.52 m.
- 10 l/m/s   : h_{crest} = 4.70 m + 7.22 m = 11.92 m.

The additional margins for sea-level rise, land subsidence (0.1 m) and seiches (0.15 m) lead to a required crest levels ranging from MSL+12.17 meters to MSL+17.38 meters. The present crest level is MSL+12.75 meters, which appears to fail the standards when 0.1 l/m/s or 1.0 l/m/s are allowed. The criterion of 10 l/m/s leads to an acceptable crest level.

5.3   Dune erosion

5.3.1   Normative probability of failure

The Guideline to the Assessment of the Safety of Dunes as a Sea Defence describes a simple safety-assessment method used to evaluate dune safety, along with a computational model to calculate the amount of erosion.
Again the safety standards as determined according to the risk assessment by the Delta Commission serve as a basis. Dikes in the Netherlands must be designed to withstand a design storm surge. In such cases, the dikes must still retain some residual strength. Consequently, the frequency with which the design level is exceeded may not be interpreted as a frequency of failure. For dunes however, the

design method used in the Netherlands does not account for any residual strength. Based on a comparison of the failure process of dikes and dune this safety margin is set at 10^{-1}. In central Holland, for instance, this implies a normative probability of failure of 10^{-5} per year.

5.3.2 Safety assessment of a cross section of a dune coast

A dune is considered safe if a certain limit profile is present after erosion has occurred under design conditions. If a dune is considered unsafe, a reinforcement plan can be designed, constructed and monitored to check that the reinforced dune fulfils the requirements.
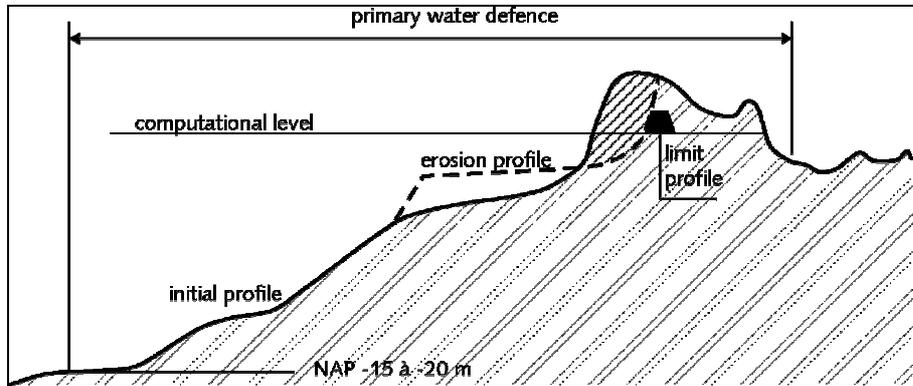
Figure 7 Dune cross-section and limit profile

The following factors affect the actual amount of erosion during a storm surge:
- maximum storm surge level;
- significant wave height during the maximum height of the surge;
- particle diameter of dune material;
- shape of the initial profile (including dune height);
- storm surge duration;
- occurrence of squall oscillations and gust bumps;
- accuracy of the computational model used to calculate the degree of dune erosion.

The actual values of the parameters just before and during a given surge are unknown. In a deterministic design method, design values of the various parameters must be stated. However, with at least 7 parameters it is rather difficult to state reliable design values in advance. A reliable design method can be derived using probabilistic methods. The guideline contains a relatively simple method for assessing the safety of a cross section of dune coast. This method has been developed such that the result corresponds with that of more complicated probabilistic calculations.

The safety-assessment method consists of a number of computational rules for determining the degree of dune erosion just before failure. The values used in the calculations are determined by probabilistic techniques so that the degree of dune erosion calculated has a probability of being exceeded equal to the normative probability of failure.

The long-term development of a dune profile is of great importance. The safety-assessment method enables one to obtain a good impression of the point in time when loss of the required safety of the dune profile might occur. This allows measures to be taken in time. On the other hand the method requires the availability of a 15-years time series of dune profile measurement.

The computation procedure of the safety-assessment method is as follows:
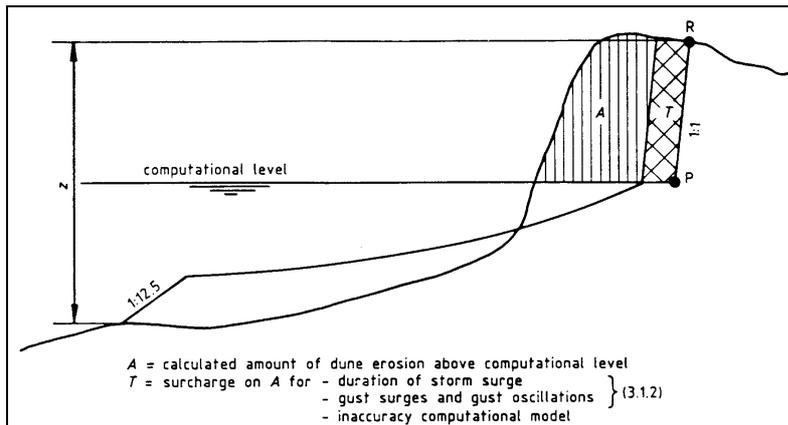
Figure 8 Definition sketch

- An **erosion analysis** under design conditions is made for each profile from the measurements. Specific computational values need to be included here for the input parameters storm surge level (computational level), significant wave height and grain diameter. For these parameters so-called design values are used.

- For each erosion analysis, the calculated amount of dune erosion above storm surge level (A) is increased with a **surcharge** (T) to account for:
  - the influence of inaccuracies in the computational model;
  - gust oscillations and gust surges;
  - uncertainty about the length of time during which the water level remains near maximum level.

  The effect of this surcharge is expressed in an additional recession of the steep dune front. Point P is the intersection of this shifted dune front with the storm.

- The above calculations yield a time series for the position of point P. These positions can be plotted in a diagram. From the position it can be easily deduced whether the coast is stable, eroding, or progressing. The trend of the position of point P as a function of time can be estimated using regression analysis. A linear approximation is usually sufficient. The **profile fluctuations** are expressed in the scattered position of the points P around this regression line.
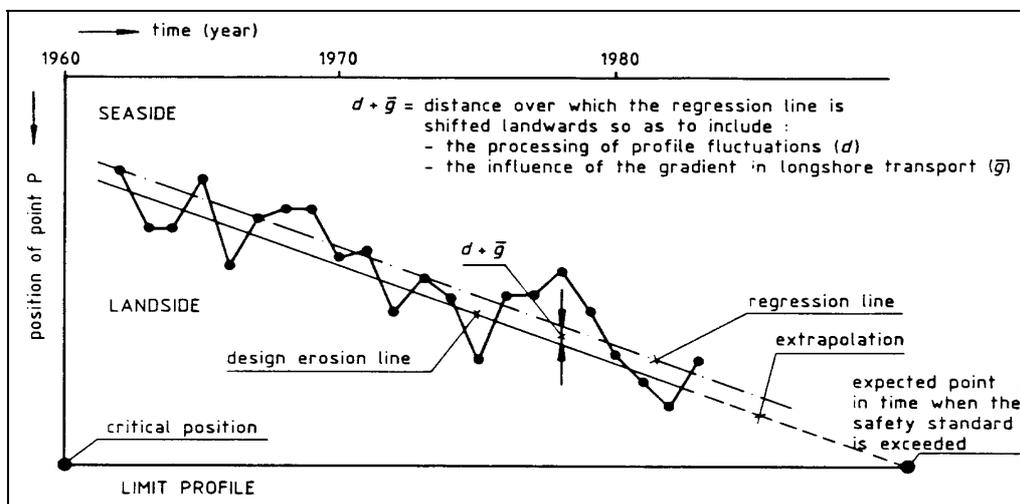
Figure 9 Principle of the safety-assessment method

- The influence of the **uncertainty** of the profile position is now taken into account by shifting the regression line in a landward direction over a certain distance (d), depending on the magnitude of the profile fluctuations. The shifted regression line (the design erosion line) yields the position of the design erosion point as a function of time.

- For coastal profiles where the net loss of sand from a gradient in **longshore transport** must be accounted for, the final design erosion line is obtained by shifting the regression line (determined

above) in the landward direction over an additional distance ($\overline{g}$).

- A **critical erosion point** is defined which refers to the degree of dune erosion just before failure. A minimum, yet stable profile, called the limit profile, must still be present landwards of the critical erosion point. In cases where the minimum profile (**the limit profile**) no longer lies on the landward side of the design erosion line, the remaining profile no longer satisfies the established safety standard.

The dimensions of the limit profile are :
- a minimum crest level h_0 according to

$$h_0 = CL + 0.12T\sqrt{H_s}$$

  where:
  CL   = the computational level (m) above MSL
  T   = peak period of the wave spectrum (s)
  H_s = expected value of the significant wave height (m) at computational level
- The minimum width of the limit profile at crest level h_0 is 3 m
- The gradient of the inner slope must be flatter than or equal to 1:2
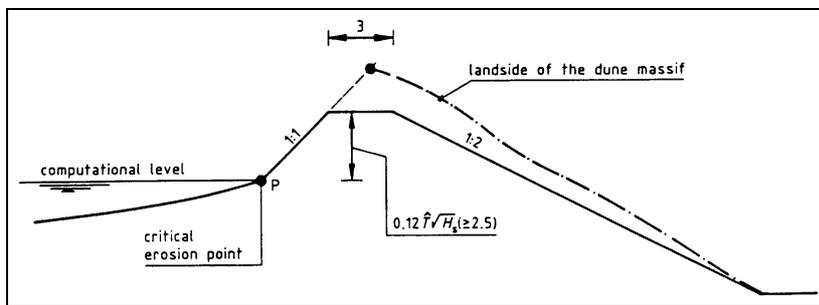
Figure 10 The limit profile

Appendix 6.1 contains some further details of the safety assessment procedure.

5.3.3   Example
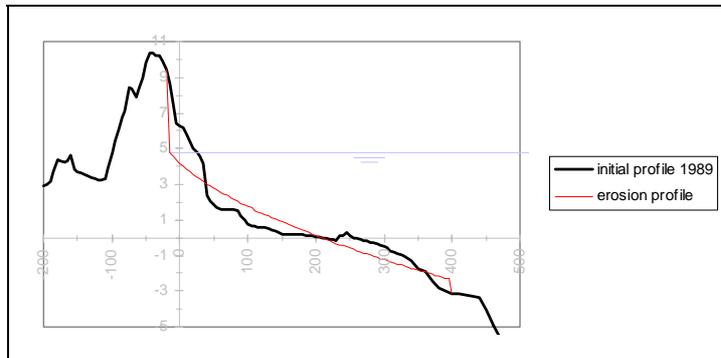
Figure 11 Erosion profile (example)

To demonstrate of the safety assessment, a cross section of dune in Texel has been selected. For this dike ring area or polder, the safety standard is 1/4000 per year. The prescribed hydraulic boundary conditions for this location and return period are:
- flood level: MSL + 4.30 meter;
- computational level: MSL + 4.75 meter (see appendix 5.1);
- wave height: 9.35 meters;
- peak period: 12 seconds.

Figure 12 Limit profile
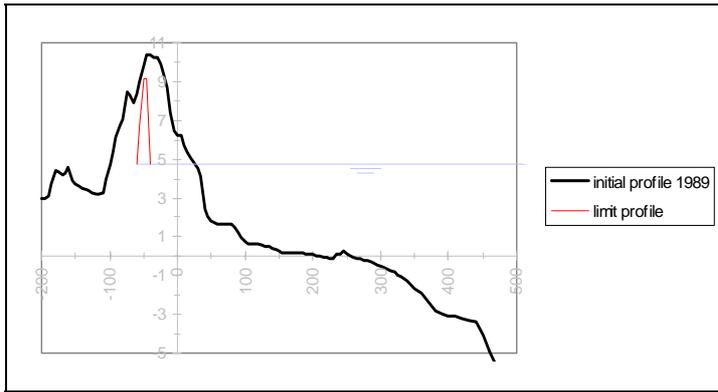
The limit profile for the cross-section is calculated according to the method earlier described. Figure 12 shows this failure criterion. The limit profile begin at a x-position -40 metres.

An erosion analysis under design conditions is conducted for each profile from the available series of profile measurements (1985 to 1989). Using the simple safety assessment procedure the erosion profiles can be calculated. The results of these calculations are incorporated in a location-time diagram of the obtained point P. A linear regression line for the position of point P in time can be determined from this diagram, including the shift landward to account for the uncertainty of the position of point P or the effect of a longshore transport..
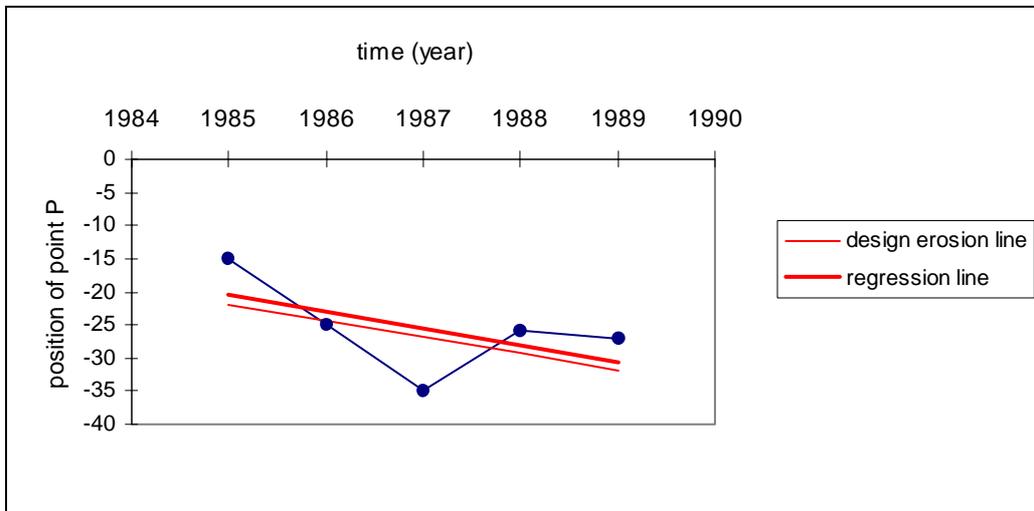
Figure 13 Position of P

Figure 13 shows that the design erosion is still well above the critical position of -40 meters. This means that the dune is safe for the moment and - given the regression - will be for at least another 5 years.

6.   Probabilistic techniques

The boundary conditions and hydraulic loads considered are generated more and more frequently through probabilistic analysis. The Delta Commission performed the first probabilistic analysis when it began presenting flood level frequency curves and safety standards in terms of return periods. Following the work of the Delta Commission, statistics on flood levels, wind and waves height and period have been gradually introduced into practice.

Theoretically the probability of a coastal dike's failure can be calculated using the probability density functions of both the load and strength of the dike and a limit state function which describes the failure in terms of load and strength.

Let the strength be: R (in French: résistance)
Let the load be: S (in French: sollicitation)

The limit-state function can be defined as: Z = R-S. The structure is considered to be in a limit state (on the edge of failure) if Z equals 0:

- $Z > 0$ safe area
- $Z = 0$ limit-state area
- $Z < 0$ unsafe area.

The load and strength of a coastal dike can be expressed in what are called basic variables. These variables can be stochastic or deterministic. Using the traditional design procedure, the limit-state function can be expressed as:

- $Z = R - S$
- $Z = (h_{crest}) - (h - 8H_s\tan\alpha)$

Figure 14 Probabilistic procedure
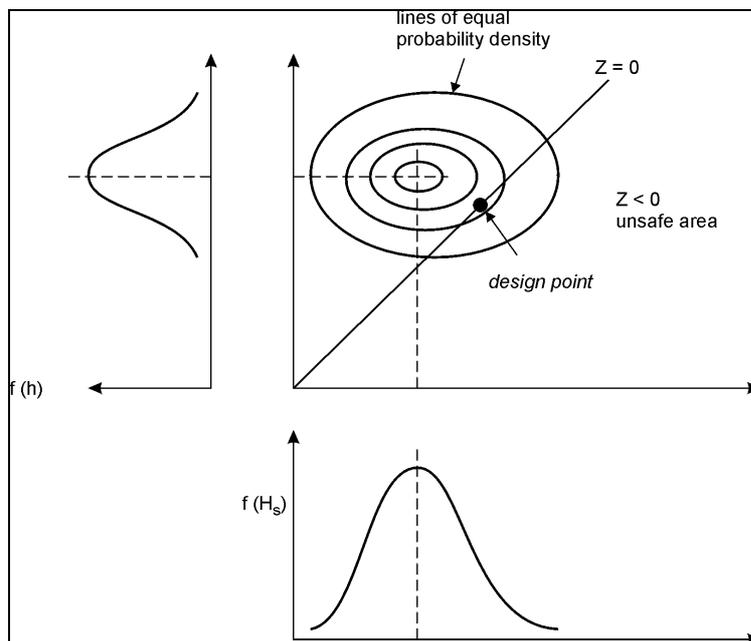
The crest level (h_{crest}) is either a running variable (while designing) or a deterministic parameter (while assessing the actual safety). In both cases this parameter can be treated as a deterministic parameter. The load is the combination of flood level (h) and significant wave height (H_s). The parameters h and H_s are stochastic variables in this case. The probability density functions of these variables can be described by:

- $f(h)$
- $f(H_s)$.

If the variables are not correlated, the joint probability density function can be calculated quite easily:

- $f(h)*f(H_s)$.

This joint probability density function can be shown in a graph using iso-density charts in the h-H_s space. The probability of failure is the content of the joint probability density function in the unsafe area.

$$P = \iint_{Z<0} f(h)f(H_s)dhdH_s$$

The maximum probability P is prescribed in the Flood Protection Act. The combination of h-H_s with the maximum probability density is called the 'design point'. In practice this design point will be used

by engineers or authorities to design dikes or to assess their safety.

For more complex situations, the probabilistic procedure can be described as:

$$P = \iint_{Z<0} ... \int f(h)f(h)...f(..)dhdH_s....d..$$

The number of basic variables can be extended according to the specific situation and design procedures. Correlation between basic variables can be introduced. But the probabilistic procedure remains fundamentally the same. This means that simple design points (combinations of basic variables) can be given for every situation. The collected hydraulic boundary conditions issued by the Ministry provides these design points. Only for specific situations (e.g. cost optimisation or tailor-made structures) is the probabilistic procedure conducted during the design process.

The probabilistic procedure described above has become more or less general practice in designing flood protection structures. However, probabilism is still largely confined to the hydraulic loads. The strength of the structure and design criteria are mostly accounted for deterministically, using a safety factor which is based largely on experience and engineering judgement. Furthermore, the uncertainty of loads, strength, criteria, models and so on is not taken into account.

7. Future developments

The situation described is based largely on standard practice in the Netherlands. However, some new developments have appeared which may be introduced into standard practice in the near future. In some cases, primarily large-scale flood protection projects like storm surge barriers, these developments have been introduced already.

7.1 Probabilism

7.1.1 More probabilistic techniques

The Delta Commission introduced a one-dimensional probabilistic approach, in which flood level was the only parameter considered to be a stochastic variable. The other parameters were treated in a deterministic way. In general, other hydraulic loads like wind and waves were expressed as expected values. At that time (1960) these expected values were 'best' or 'educated' guesses. The strength parameters were also treated deterministically, but given the safety philosophy (aiming to withstand the prescribed hydraulic loads) conservative values or design criteria were now used.

In the years following the Delta Commission report the hydraulic loads have been modelled in more sophisticated ways:
- joint probability distributions of flood level, wave heights and wave periods have been derived for the coastal and lake areas
- joint probability distributions of flood level and wind velocities have been derived for the river deltas.

The results of these studies are slowly but steadily being introduced into practice. The safety standard (expressed as a return period) is applied to a combined hydraulic load parameter (e.g. overtopping discharge) instead of a flood level only.

The introduction of these changes, however, generates a fierce discussion around a central issue: are we still in line with the principles of the Delta Commission? In recent years, it has been shown that the technical elaboration of the safety standards mentioned above leads to higher hydraulic loads, which may lead to massive reconstruction works. On the other hand, probability techniques would be welcome if certain traditional, conservative design rules were replaced with more modern, inexpensive ones.

7.1.2 Uncertainties

A major issue in the discussion on probability is the way we deal with uncertainty. Uncertainty can be

classified in three categories:
- implicit uncertainty, because the variables studied have a stochastic nature;
- model uncertainty, because our description of natural phenomena is always insufficient;
- statistical uncertainty, because the number of observations of extreme events is too low.

The Delta Commission introduced the implicit uncertainties into our way of thinking. This has been extended in recent years to other hydraulic load parameters. Model uncertainties are not taken into account when hydraulic load models are considered. Strength models or design criteria do include a safety factor, although this factor is based largely on experience or engineering judgement. Statistical uncertainties, such as the accuracy of design water levels (with a return period of 10,000 years) are not taken into account.

Some recent studies of uncertainty have shown that all of the uncertainties described above can be incorporated into our design procedures. However, if these uncertainties are treated simply as additional stochastic variables and the safety level is kept at the same level, this will lead to enormous increases in required crest levels. These increases may vary from 1.0 to 2.0 meters.

7.1.3 Example

The example of the Pettemer Zeewering may be extended slightly in order to explore the effect of additional stochastic variables and uncertainties. For this purpose several probabilistic calculations are made, according to the following assumptions. In all cases the sophisticated approach to wave run-up has been applied.

	Scenario	Water level	Wave height	Wave period	Crest level
	Reference	deterministic[1]	deterministic[2]	deterministic[2]	13.30
A)	Stochastic water level	stochastic	deterministic[2]	deterministic[2]	13.36
B)	Stochastic, uncertain water level	stochastic uncertain[3]	deterministic[2]	deterministic[2]	13.87
C)	Uncertain water level and wave height	stochastic uncertain[3]	stochastic[4]	deterministic[2]	14.51
D)	All hydraulic loads uncertain	stochastic uncertain[3]	stochastic[4]	stochastic[4]	15.32

1) Deterministic means that the water level with a return period of 10,000 years has been calculated separately. This value is used in a deterministic fashion to calculate the required crest level.
2) Deterministic means that the expected values of wave height and wave period are used to calculate the required crest level.
3) Stochastic and uncertain means that both the probability distribution function of the water level and its uncertainty are taken into account.
4) Stochastic means that the uncertainty of the expected values of wave height and wave period are taken into account.

The data used for the calculation is collected in appendix 6.2. As shown in the table, the reference scenario leads to an almost identical crest level as in scenario A. This is also to be expected because water level is the only stochastic variable, so that the design point for a deterministic calculation can be derived very easily. Increasing the number of stochastic variables, however, leads to increased crest levels:
- a statistical uncertainty ($\mu = 0.0$ and $\sigma = 0.35$ meter) of the water level probability density distribution leads to an increase of the required crest level of approximately 0.50 metres;
- adding the uncertainty of the wave height ($\mu = 1.0$ and $\sigma = 0.20$) leads to yet another increase of over 0.6 metres;
- finally, including the uncertainty of the wave period ($\mu = 1.0$ and $\sigma = 0.10$) leads to the largest increase of over 0.80 metres.

When compared to the reference scenario the added uncertainties lead to a total increase of the required crest level of 2 metres.
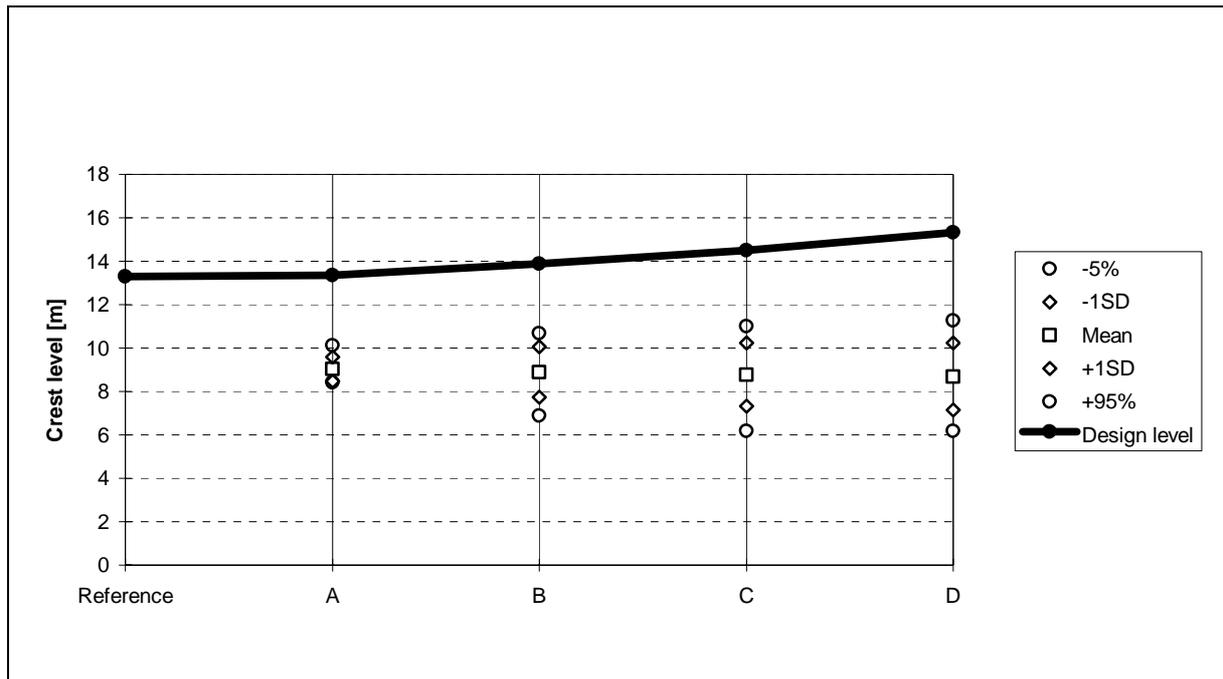
Figure 15 Results of the probabilistic calculation

The figure above illustrates the results graphically. The mean crest level is practically constant for all scenarios. Unfortunately, the mean value is not significant for our purpose. The extreme values, which are important, are very sensitive to the added stochastic variables and their statistical properties. The difference between the extreme high and extreme low results increases dramatically as do the design levels which are related to a probability of 10^{-4} a year.

Primarily for this reason, the model hydraulic and statistical uncertainties have not yet been incorporated into standard practice. The discussion on the further application of probabilistic methods, including uncertainties, will be continued in the research programme and policy preparation for flood risks.

7.2   Safety philosophy based on flood risks

The present safety standards are expressed as extreme water levels and their return levels. These water levels and return periods are related only indirectly to the potential flood risks, which were calculated in 1960. Technical uncertainties, such as the behaviour of dikes during extreme conditions, prohibited a more direct link between economical damages or casualties and the technical requirements for flood protection structures. Since the Delta Commission report, the issue of calculating flood risk (probability of flooding times the consequences) has been a popular research topic. Within this topic, many research topics can be identified:
- geotechnical or structural modelling;
- strength parameters;
- hydraulic modelling;
- hydraulic parameters;
- statistical parameters, including correlation between various loads and events;
- modelling of failure or collapse of flood protection structures (breaching);
- damages due to flooding;
- effectiveness of measures to prevent damages.

At present the probability of flooding (not the risk) has been calculated for a limited number of polders using state-of-the-art modelling. An extensive uncertainty analysis will also be carried out.

Polder	Safety standard	Flooding probability

Betuwe, Tieler and Culemborgerwaarden (no. 43)	1/1250	1/2000
Alblasserwaard and the Vijfheerenlanden (no. 16)	1/2000	1/1100
Friesland and Groningen (no. 6)	1/4000	1/3000
Hoekse Waard (no. 21)	1/2000	1/1400
Terschelling (no. 3)	1/2000	1/3000
Polder Mastenbroek (no. 10)	1/2000	1/1400
Zuid-Holland (no. 14)	1/10000	1/4000
Noord-Beveland (no. 28)	1/4000	1/2000

In the next few years, flooding probabilities and flood risks will be calculated for the entire country. These results can be used for the following purposes:
- to assess actual flood risks (damage potentials) related to present safety standards;
- to optimise the present design methods within the existing framework of safety standards;
- to compare flood risks with the societal risks associated with other events (e.g. traffic);
- to start a discussion on acceptable flood risk levels in relation to acceptable risk levels of other events.
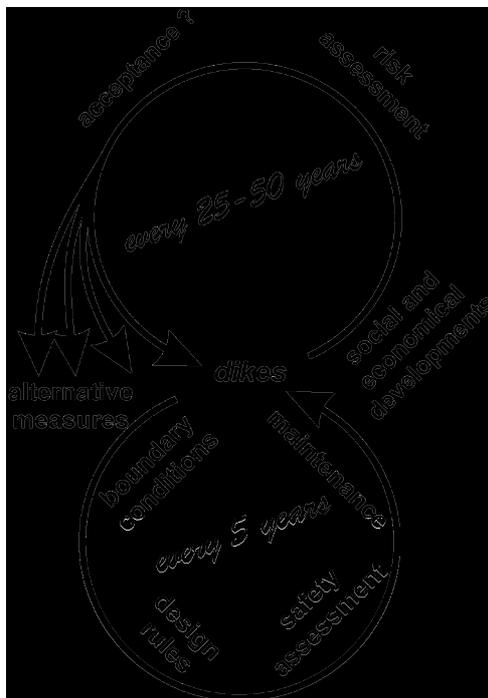
Figure 16 Risk-concept

The aim of this effort is to devise a new safety philosophy based on flood risk. Safety will be related to the risk of flooding, described by multiplying the probability of flooding with its consequences in damage and victims. This safety approach offers the possibility to consider and assess measures in the entire risk chain (extreme water levels, the probability of a dike breach and the consequences of flooding) and to make optimal choices. Measures which reduce the probability of high water or which limit the damage caused by a dike breach can make just as great a contribution to protection as raising the height of the dike itself.

Figure 16 shows how the risk concept and the regular safety assessment of dikes may interact. The present flood protection policy and the risk concept are shown as two adjoining circles. The lower circle is the present policy of safety assessment, aimed at maintaining the prescribed safety standard, and the upper represents future risk assessment.

The risk-assessment circle includes socio-economic effects and evaluation. This evaluation will not remain confined to flood risks, but will take other sources of risk into account as well. The risk assessment will give information on expected damages in case of flooding. The damage of a coastal flood will differ from the damage of a fluvial flood: the water may be salty or not, and warning will come with longer or shorter notice. A small polder will inundate more quickly than a large polder, so that people have less time to evacuate. More damage will occur in a deep polder than in a shallow one. The damage level will be higher in a dike ring area where many people live and work than in an area with a sparse population. And last but not least, the damage depends on the preparedness of people to be evacuated, and how effectively this evacuation occurs.

The amount of damage may be accepted or rejected, given other sources of risk and the effort required to reduce the flood risk. Several strategies and measures can be considered to reduce flood risk. One of the alternatives is to raise or strengthen the dikes, which can be expressed as a higher safety standard. This safety standard can be maintained using the lower circle, which is the core of the present flood protection policy. Given the time-scale of the processes involved, the interaction between both circles should not be frequent (safety assessment once every five years, safety philosophy once every 25-50 years) .

7.3 Technical developments

Research is needed to make the changeover from the current safety philosophy to one based on flood risk. The research programme of the Ministry aims to make this changeover possible. In order to develop an effective safety philosophy based on the risk of flooding, it is essential that the probability of flooding and its consequences can be calculated with sufficient accuracy. It is also important to establish what is considered an acceptable level of risk.

7.3.1 Probability of flooding

The probability of a dike breach is not adequately defined under the current safety standards, even though the probability of a dike breach followed by flooding is the most tangible measure of danger. After all, flooding results in economic damage and, depending on the situation, can claim victims.

Measuring dike height alone provides insufficient information about protection from flooding. Two technical arguments for this can be cited: the geotechnical stability of dikes and the correlation between the failure of different dike sections.

For example, if the dikes lose their resistance to sliding during periods of high water, a dike breach could occur without the water flowing over the crest of the dikes. This contributes to the probability of a dike breach or flooding. The required resistance to these mechanisms (largely geotechnical failure) cannot be expressed in terms of a hydraulic load standard or crest height. In the current situation, additional requirements are established for the probability of a dike breach occurring at water levels below the prescribed water level.

The larger the polder, the more dike sections are needed to protect the area. If these sections were fully correlated with the hydraulic load on them, the safety of the area could be expressed as the safety of a single dike section. In practice, this is not the case. Both the strength of and the load on the dike sections around the area are not fully correlated. Other types of constructions, such as discharge sluices, are to a large extent responsible for this. The probability of a dike breach in any dike section, followed by flooding, is thus always greater than the probability of a breach in a single given dike section.

Because of the variation in types of water-retaining structures, there is also variation in the nature of the threats. After all, the threats to dunes are different to those to dikes. This means that the systems of dikes and water barriers can fail or collapse in different ways and in different places. The failure or collapse of a single element can cause the entire water-retaining system to fail, causing the area to flood anyway. To calculate the probability of flooding, the research concentrates on probability descriptions of load, the strength of the dikes and collapse mechanisms and the development of system-failure models.

7.3.2 Consequences of flooding

Flooding usually results in extensive material damage. The extent of the damage depends on the nature of the threat (sea water or fresh water, short or long period of flooding, expected or unexpected) and the characteristics of the flooded area (depth, built-up areas, industry, exact location of the dike breach). In particular, deep floods or fast-flowing water can have serious consequences in terms of number of victims, extent of damage, and disruption to daily life and infrastructure. In calculating the consequences of flooding, the research concentrates on developing an instrument by which damage and number of victims for each dike ring area can be calculated in a uniform and practical manner. Information and warning systems help both the government and individual citizens to take the right measures at the right time. Applying these types of instruments decreases the consequences of flooding. A Flood Information System (HIS) is currently being developed. This can be used before and during high water to predict the way in which flooding will occur; monitoring water levels, waves, dike condition and the availability of the road network; determining the effects of any measures taken and facilitating announcements, communication and decision-making.

7.3.3 Acceptable levels of risk

The standards established in the Flood Protection Act are based on the probability of flooding for each dike ring area. The current differentiation in standards for each dike ring area will interpret safety in terms of flooding probabilities rather than overload frequency. The term "probability of flooding"

relates more to the actual safety of an inhabitant of a dike ring area than the term "probability of overloading per dike section". Calculating flooding probabilities will lead to discussion about the highest probability of flooding and its associated risk. Instruments for conducting this social discussion need to be developed.

Appendix 6.1   Details of safety assessment procedure of dunes

The erosion analysis
In the erosion analysis the following values must be used for the storm surge level, significant wave height, and the grain diameter of dune sand:

The storm surge level
When assessing the safety of a primary sea defence work, the computational value for the storm surge level equals the design level established by the Delta Commission plus two-thirds of the decimation height. This level is called the computational level.

Table : Design levels, decimation heights, and computational levels along the Dutch coast

location	design level (m above NAP)	decimation height (m)	computational level [**] (m above NAP)
Vlissingen	5.40	0.72	5.90
Hoek van Holland[*]	5.25	0.72	5.75
Scheveningen	5.40	0.70	5.85
Katwijk	5.40	0.70	5.85
Ijmuiden	5.15	0.67	5.60
Den Helder	5.05	0.66	5.50
Texel	4.90	0.68	5.35
Vlieland	4.70	0.68	5.15
Terschelling	4.80	0.68	5.25
Ameland	5.10	0.68	5.55
Schiermonnikoog	5.15	0.68	5.60

[*]   outside the breakwater
[**]   the computational levels are rounded off to a multiple of 5 cm

The significant wave height
The expected value of the wave height at computational level should be used as the significant wave height H_s. The probability density functions for the significant wave height as a function of water level were determined for a number of locations along the Dutch coast. The expected values of the significant wave height for these locations can be read from the diagram below. The given values hold for deep water conditions.

Figure 17 Wave height

<u>The grain diameter</u>
The computational value (D_{comp}) for the grain diameter is:

$$D_{comp} = \mu_{D_{50}} - 5\frac{\left(\sigma_{D_{50}}\right)^2}{\mu_{D_{50}}}$$

Where:

μ_{D50} = the expected value of the D_{50}
σ_{D50} = the standard deviation of the D_{50}

Table : Grain size for a part of the Dutch coast.

Location	point of reference (km)	μ_{D50} (µm)	σ_{D50} (µm)	D_{comp} (µm)
Schiermonnikoog	1.04	150	8	148
	3.02	169	8	167
	5.01	165	8	163
	7.00	164	8	162
	9.20	163	8	161
	11.00	164	8	162
	13.00	159	8	157
	15.00	159	8	157

The surcharge on the amount of erosion above computational level
Three surcharges on the amount of dune erosion A (m^3/m) above the computational level are included:

- A surcharge of 0.10A (m^3/m) to account for the uncertainty about the length of time during which the water remains near maximum level. This duration is the most determinative factor for the amount of dune erosion during the water level changes of a storm surge.
- A surcharge of 0.05A (m^3/m) to account for the effect of gust surges and gust oscillations.
- A surcharge of 0.10A + 20 (m^3/m) account for the inaccuracy of the computational model for the expected dune erosion

The sum of the surcharges on the amount of dune erosion A above computational level consequently amounts to 0.25A+ 20 (m^3/m). This surcharge is expressed as a landward shift of the originally calculated dune base.

Processing the profile fluctuations
The results of the calculations can be incorporated into a location-time diagram of the obtained point P. A linear regression line for the position of point P in time can be determined from this diagram, as well as the standard deviation of the position of the calculated points P from this line. The design erosion line is obtained by shifting this regression line landwards over a distance d:

$$d = \frac{\sigma_p^2 \overline{z}}{275}$$

where:

σ_p = the standard deviation of the position of the P from the regression line (m)
z = mean value of the differences in height z between the most landward and the most seaward point of the total erosion profile of each erosion analysis (m)

Processing a gradient in longshore transport

In the case of a varying longshore transport of sand along the coast (a gradient in longshore transport), caused for example by obliquely approaching waves, there is an imbalance between erosion and sedimentation for that particular coastal section. Where the erosion-sedimentation balance has a negative outcome (total outgoing longshore transport exceeds total incoming longshore transport), those coastal sections are important for safety reasons. The result is an additional landward shift in the erosion profile over such a distance that the cross sectional area of the shift corresponds to the difference in longshore transport.
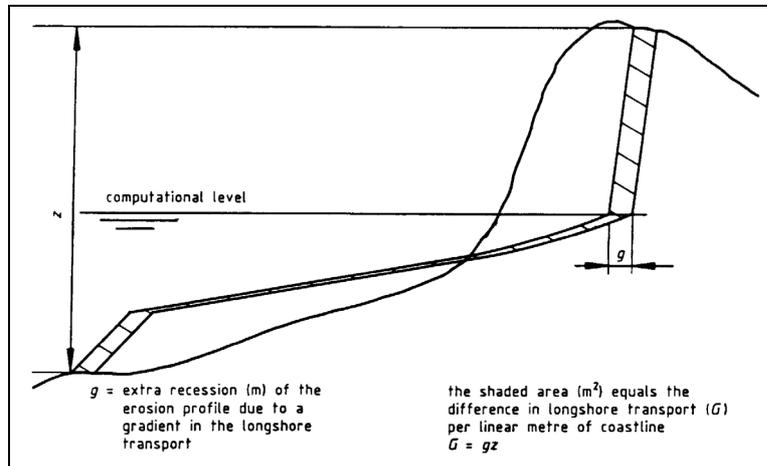
Figure 18 The influence of a gradient in longshore transport on dune erosion

A value of the gradient in longshore transport due to a (not too strong) curvature of the coastline can be included in the standard safety assessment procedure. Further research is required for strongly curved coastal sections, such as at the heads of islands, and for other situations where a gradient in the longshore transport can be expected, such as at the transition between a dune and a structure (e.g. a breakwater, a dike, or a dune base protection) and during strong variations in wave height in the longshore direction (for instance behind sand banks). The standard method is inadequate for assessing the safety of such coastal sections.

The value for the longshore transport G (m^3/m) for not too strongly curved coastal sections can be calculated with the formula:

$$G = \frac{A^*}{300}\left(\frac{H_s}{7.6}\right)^{0.72}\left(\frac{w}{0.0268}\right)^{0.56} G_0$$

where:

A* = calculated dune erosion above the computational level including surcharge (m^3/m)
H_s = significant wave height at computational level (m)
w = fall velocity (m/s)
G_0 = reference value for G (m^3/m) (see Table 3)

The presented method is valid only for coastal sections classified in class 1. For the Dutch coast the section between Hoek van Holland and Den Helder can be classified as such.

Table Reference value longshore transport for different classes of coastal curvature

class	curvature interval (degrees/1000m)	G_0 (m^3/m)
1	0-6	25
2	6-12	50
3	12-18	75
4	18-24	100
5	>24	further research needed

The final design erosion line for the concerned cross sections is obtained by shifting the regression line landwards over an additional distance of $\overline{g}$ (m). The distance $\overline{g}$ is the mean value of additional recession g of the erosion point of each profile considered.

Computational model / erosion profile

On the basis of extensive model investigations and prototype measurements, a computational model

has been developed to determine the expected degree of dune erosion, and its standard deviation, due to a random storm surge. Starting points of the model:

- The coastal profile is transformed into a certain erosion profile during a storm surge with dune erosion.
- The shape of the erosion profile is a function of the significant wave height and the fall velocity of the eroded sand in salt water.
- The shape of the erosion profile is independent of the angle of incidence of the waves, the coastal profile before the storm surge, and the storm surge level.
- It is assumed that the eroded sand is transported only in a seaward direction.

The erosion profile is situated relative to the profile before the storm surge such that the total area of eroded sand equals the area of deposited sand. It is generally assumed that no net loss of sand occurs in the longshore direction. During a storm surge the coastal profile is transformed into a certain erosion profile that is built up as follows:

- After erosion has occurred, the dune base (the point where the steep front of the eroded dune changes into the relatively flat beach profile) is situated at storm surge level. The slope of the eroded dune is 1:1.
- Starting from the dune base (x=0, y=0)and moving in the seaward direction, perpendicular to the coast, the profile extends parabolically according to the formula:

$$\frac{7.6}{H_s} y = 0.4714 \left\{ \left(\frac{7.6}{H_s}\right)^{1.28} \left(\frac{w}{0.0268}\right)^{0.56} x + 18 \right\}^{0.5} - 2.00$$

where:
H_s = significant wave height (m) in deep water
w = fall velocity of dune sand in salt water (m/s)
x = distance to the new dune base (m)
y = depth below storm surge level (m)
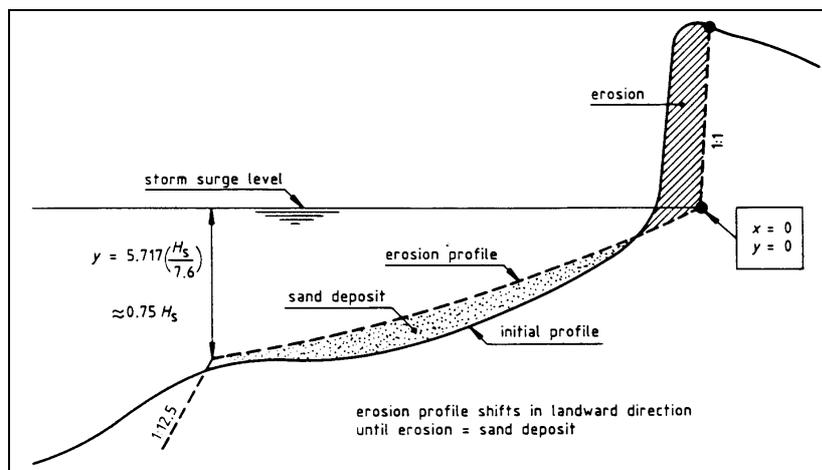
Figure 19 Principle of the computational model for dune erosion

to the point with co-ordinates:

$$x = 250 \left(\frac{H_s}{7.6}\right)^{1.28} \left(\frac{0.0268}{w}\right)^{0.56}$$

$$y = 5.717 \left(\frac{H_s}{7.6}\right)$$

Seaward from this point, the profile continues as a straight line with a gradient of 1:12.5 until it intersects the original profile.

The fall velocity w can be calculated with the formula:

$$^{10}\log\left(\frac{1}{w}\right) = 0.476\left(^{10}\log D\right)^2 + 2.180\,^{10}\log D + 3$$

where:
w = fall velocity of the dune sand in salt water (m/s)
D = D_{50} of the dune sand (m)

Appendix 6.2 Data used for the probabilistic calculations

Water level
The probability density function used for the water level is an exponential distribution with the parameters 2.34 (shifted x-value) and 0.25 (β-value). The uncertainty is described using an added variable with a normal distribution with the following parameters: $\mu = 0.0$ and $\sigma = 0.35$ meter.

Wave height
The deterministic value used for wave height is directly related to water level. The figure below shows the relation between wave height (at MSL-20 meters) and water level. The wave height at the base of the dike is reduced to a value of 4.70 meter for a surge level of MSL+4.70 meter. For this global calculation, it is assumed that the wave height at the base of the dike is generally equal to the surge level.

The uncertainty of the wave height used in probabilistic calculations is described using an extra variable with a normal distribution with the following parameters: $\mu = 1.0$ and $\sigma = 0.20$.

Wave period
The deterministic value used for the wave period is 12.50 seconds. The uncertainty of the wave period used in probabilistic calculations is described using an extra variable with a normal distribution with the following parameters: $\mu = 1.0$ and $\sigma = 0.10$.

Calculations
The probabilistic calculations are carried out using @Risk, a commercially available add-on for Excel or 1-2-3. This package features Monte Carlo or Latin Hypercube simulation techniques. For this study, regular Monte Carlo simulation has been used with 10,000 simulations per run; for the final results 100 runs were executed. The design values presented (10^{-4} per year) were determined by averaging the results of these runs.