

## CHAPTER 203

### Dike Failure Calculation Model Based on In Situ Tests.

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#### Abstract

A method has been developed to determine the strength of a dike expressed as the mean return period for the critical load.

The method is based on tests carried out on prototype dikes.

#### 1. Introduction.

As severe storms in 1981, 1984 and 1985 caused a number of dike failures, the Danish Government decided to lay down guidelines for future economic compensation in the event of major flooding.

These guidelines were based on a study dealing primarily with the following subjects:

- a) Mathematical simulation of historic storms to generate statistics of extreme water levels at any point along the 7,000 km Danish coastline, see figure 1.
- b) Establishing a data bank with information about buildings in low-lying areas and the construction details of the 350 km dikes of varying quality.
- c) Developing a method for the evaluation of dike strength in terms of return period of failure.

This paper summarizes the work of item c.

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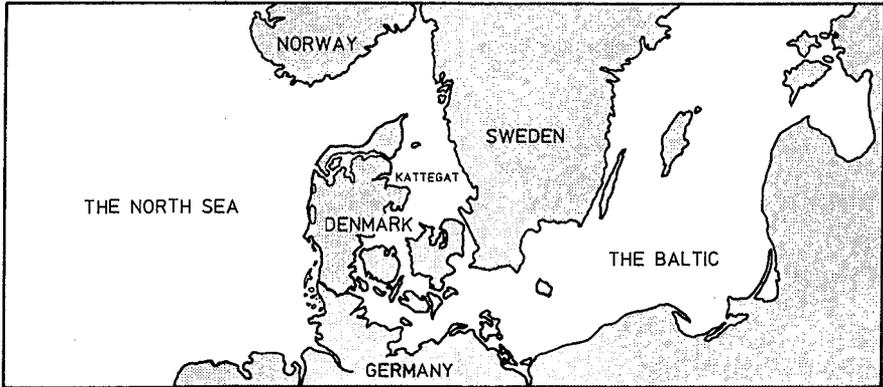


Figure 1. Waters Around Denmark.

## 2. Basic Description of the Method.

A diagram illustrating the dike strength calculation method is shown in figure 2. As can be seen, the calculation starts with a certain water level in deep water outside the specific dike. Using this water level and corresponding waves we then calculate whether this situation causes front or back slope failure of the dike.

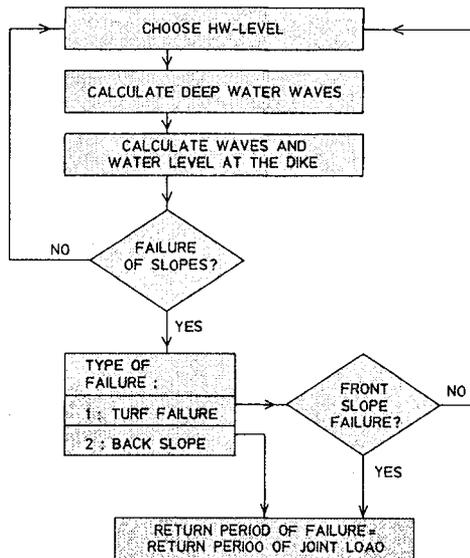


Figure 2. Calculation Diagram.

If the dike does not fail the calculation is repeated using a higher water level. This process continues until we find the lowest water level at which the dike fails.

The mean return period for this combination of water level and waves is determined and used as an expression of the dike strength.

The different steps in the calculation diagram are treated below.

### 3. Local Extreme Water Level Statistics.

Extreme water level statistics were calculated using a truncated Weibull-distribution to describe the size-distribution, and the Poisson-distribution to describe the arrival-process.

The statistics were prepared using partial series of either recorded or calculated extreme water levels. Reliable recorded water levels are always preferable to calculated water levels. Unfortunately, reliable recorded water levels are relatively scarce in Denmark.

Therefore all major storms in the Kattegat and in the Baltic Sea from 1965 to 1984 were simulated mathematically by The Danish Hydraulic Institute.

The recorded and calculated water levels during these storms were stored in a data base. This makes it possible to calculate extreme water level statistics for any location on the Danish coastline.

### 4. Duration of High Water Level.

The typical duration of the water level above a certain value was determined in order to be able to determine the number of waves causing erosion of the dike core, see section 9.

Naturally there are large variations in the water level curves around the maximum value. Therefore a number of water level curves for different storms and localities were incorporated by plotting the maximum water levels at the same point.

On the basis of these curves a connection was established between the difference in water level between a given level and the maximum value, and the duration of this water level interval.

### 5. Estimation of Storm Waves.

The waves were generally estimated by using the procedure described in The Shore Protection Manual (1984). The fetch used was calculated as the average value of the fetches in the 90° angle evenly distributed around the direction perpendicular to the coastline.

In cases where there are large shallow sea areas in front of the dike the storm waves were estimated according to the procedures outlined by Siefert (1974).

### 6. Simultaneous Onshore Waves and High Water Level.

On many Danish coasts high water level situations can arise without simultaneous onshore waves.

However, clearly the most dangerous situation is a combination of high water level and onshore waves.

The conditional probability  $P(\text{onshore waves} \mid \text{water level} > \eta)$  was estimated by considering the storms used in the mathematical simulation mentioned in section 3.

The number of storms where the wind direction at the time of maximum water level was within the 90° sector around the perpendicular to the coast was enumerated. This figure, divided by the total number of storms, is used for the conditional probability  $P(\text{onshore waves} \mid \text{water level} > \eta)$ .

### 7. Waves and Water Level at the Dike.

The water level and waves at the dike can be calculated on the basis of the water level and waves in deep water.

The calculation is based on successive breaking and shoaling of the waves, with specific criteria for the transition between these phases.

The breaking criterion is defined according to Goda (1985) as

$$\frac{H_B}{L_0} = 0.17 \left\{ 1 - \exp \left[ - \frac{1.5 \pi h}{L_0} (1 + 15 \tan^{4/3}(\varphi)) \right] \right\} \quad (1)$$

where  $H_B$  is the wave height at breaking,  $L_0$  is the wave length in deep water,  $h$  is the water depth and  $\tan(\varphi)$  is the bottom slope.

Using small amplitude wave theory the governing equations are

$$\frac{db}{dx} = -\frac{3}{16} \cdot \frac{1}{h} \cdot \frac{d(H^2)}{dx} \quad (2)$$

$$\frac{dE_f}{dx} = \begin{cases} 0 & \text{no breaking} \\ -E_{f,B}/l_{DISS} & \text{breaking} \end{cases} \quad (3)$$

where  $b$  is the wave setup,  $x$  is a horizontal distance increasing onshore,  $h$  is the local water depth,  $H$  is the local wave height,  $E_f$  is the energy flux,  $E_{f,B}$  is the energy flux at the point of breaking, and  $l_{DISS}$  is the dissipation distance.

$l_{DISS}$  is a fixed value at a given point of breaking.  $l_{DISS}$  is chosen so the local ratio of wave height to depth,  $H/h$ , decreases monotonically. The following expression is used for  $l_{DISS}$

$$l_{DISS} = \frac{2}{5} \cdot h_B \left(1 + \frac{3}{8} \left(\frac{H_B}{h_B}\right)^2\right) \cdot \tan^{-1}(\psi) \quad (4)$$

where  $B$  denotes the point of breaking.

The transition from wave breaking to shoaling is governed by the criterion given by Deigaard et al. (1986).

$$\frac{H}{h} = 0.5 \quad (5)$$

The surf zone calculations are carried out using a PC-programme.

### 8. Strength of a Turf Revetment.

For dikes covered with turf, a front slope failure starts with failure of the turf and continues with erosion of the dike core, see section 9.

Methods to predict failure of the turf slope and erosion of the dike core are not known to the authors. Therefore such methods had to be developed.

A series of tests were carried out to find the resistance of turf as a function of the dry weight of grass roots per  $m^2$  and the particle velocity in the breaking wave. The tests were carried out on prototype dikes.

Wave breaking was simulated with a water jet from a 4" pipe. A total of 16 tests were carried out on turf of 4 different qualities. These tests indicate that there is a certain critical particle velocity of the breaking

wave which initiates damage to the turf. Figure 3 shows the test results from one site. Each test with a certain water velocity was repeated four times.

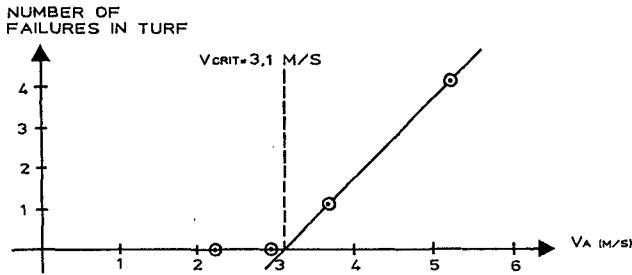


Figure 3. Example of Test Results.

In figure 4 the critical velocities found are plotted as a function of the dry weight of the grass roots in the turf.

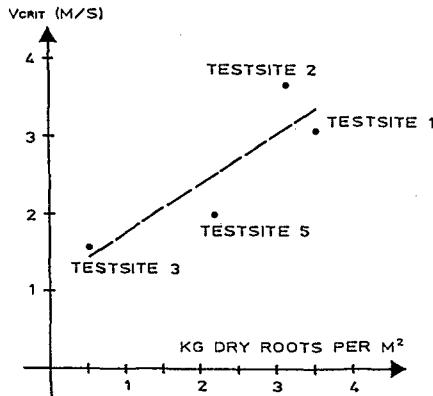


Figure 4. Critical Velocity as a Function of Dry Weight of Grass Roots in the Turf.

For dike slopes covered with turf and steeper than 1:6 it is assumed that the slope fails when the velocity in the plunging breaker exceeds the critical velocity for the turf.

The impact velocity  $V_A$  is calculated assuming that the plunging breaker falls from a height of  $0.78 \cdot H_B$  above the mean water level.  $H_B$  is the wave height at breaking.

$$V_A = \sqrt{g(h_B + 1.56 H_B)} \quad (6)$$

where  $g$  is the acceleration of gravity and  $h_b$  is water depth at breaking.

The wave is assumed to hit the slope at the intersection between the slope and the mean water level.

### 9. Erosion of the Core Material.

In this study, front slope failure is defined as the situation where the front slope turf has failed and the erosion of the dike fill has reached the back slope, see figure 5.

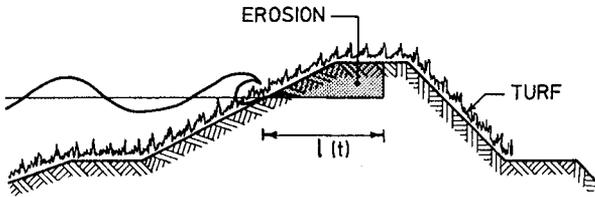


Figure 5. Model of Front Slope Failure.

The horizontal erosion  $l$  is described by the following expression

$$l(t) = k_c \cdot k_e \cdot V_A \cdot \frac{t}{T} \quad (7)$$

where  $t$  is time,  $k_c$  is a calibration constant,  $k_e$  is a core material constant,  $V_A$  is the impact velocity and  $T$  is the wave period.

The core material constant was determined for different core materials. The method used was repeated flushing with a well-defined velocity.

Figure 6 shows a typical set of test results. As can be seen, the erosion depth is roughly proportional to the number of jets.

The following material constants  $k_e$  have been determined from the tests

Sand, small amount of silt (10%)	$1.35 \cdot 10^{-3}$ sec/jet
Sand, large amount of silt (17%)	$0.89 \cdot 10^{-3}$ sec/jet
Clay with sand (60%)	$0.059 \cdot 10^{-3}$ sec/jet

The equation for horizontal erosion was calibrated on real front slope failures recorded at the Rejsby dike after a severe storm in 1976. In figure 7 the horizontal erosion for each failure is plotted as a function of the time the water level remained above the bottom of the indentation.

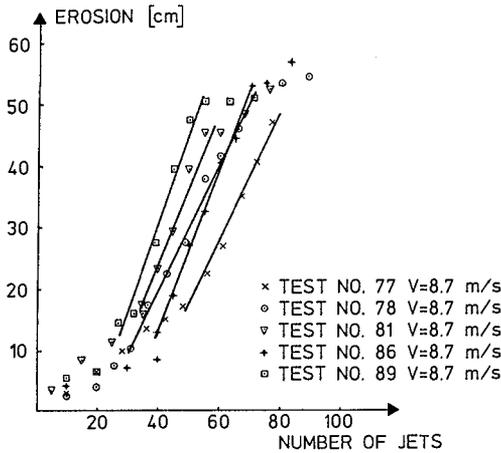


Figure 6. Results from Simulation Tests with Core Erosion.

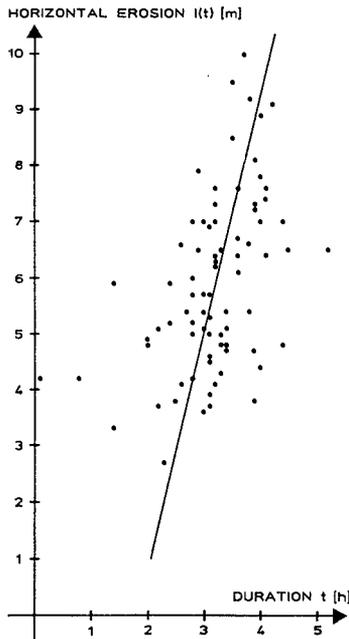


Figure 7. Horizontal Erosion and Duration of High Water Level for Real Dike Failures.

The calibration factor  $k_c$  in equation (7) was determined from figure 7.

10. Wave Overtopping.

The run-up level  $Z_{PCT}$  exceeded by a certain percentage of the waves can be determined by using the formula

$$Z_{PCT} = 0.77 \sqrt{2 - \log(PCT)} \cdot 0.7 \cdot T \sqrt{gH_0'} \tan(\varphi) \quad (8)$$

given in Technical Advisory Committee (1974). PCT is the percentage of the waves, T is the average wave period,  $H_0'$  is the equivalent wave height in deep water and  $\varphi$  is the bottom slope angle.

In dimensioning the Danish Wadden Sea dikes 2% was used in the formula as the critical overtopping percentage for back slope failure.

This percentage was considered too conservative for this study. Records show that overtopping in excess of 50% can occur without dike failure.

Therefore the critical overtopping percentages shown in table 1 have been used in this study.

Overtopping percentages	Dike surface		
	Unprotected	Turf, sandy	Turf, clayey
Slope:			
1:1.5	2%	10%	10%
1:2	2%	20%	50%
1:3	2%	30%	90%

Table 1. Critical Overtopping Percentages.

11. Geotechnical Failure of the Back Slope.

The risk of back slope failure has also been examined using geotechnical calculations.

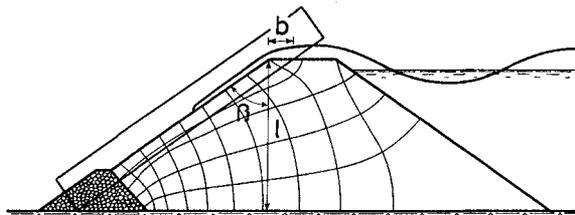


Figure 8. Geotechnical Description of Back Slope Failure.

Assuming stability in the box indicated in figure 8, and assuming completely cohesive soil, the critical cohesion  $C_{crit}$  along the failure line can be described by the following equation

$$C_{crit} = [ b (\gamma_w + \gamma_m) + \frac{A_{o1}}{l} \cdot \gamma_w ] \cos^2 \beta \quad (9)$$

where  $b$ ,  $l$  and  $\beta$  are defined in figure 8,  $\gamma_w$  is the specific gravity of water,  $\gamma_m$  is the specific gravity of saturated fill and  $A_{o1}$  is the cross section area of the overtopping water.

This critical cohesion is compared to the shear strength in the back slope determined by vane tests. The vane test results are reduced to between  $1/3$  and  $1/4$  due to fractures in the soil.

Equation (9) has been confirmed by full-scale tests on two dikes. One test resulted in back slope failure and one test gave no failure. Both results were in accordance with the formula. The tests showed that relatively low levels of overtopping result in complete saturation of the soil.

The main drawback of this geotechnical method in evaluating the stability of the back slope is the large number of vane tests required.

## 12. Calculation of Dike Strength.

A dike fails either when the water level reaches the top of the dike or when a combination of high water level and onshore waves causes front or back slope failure.

The shorter of the mean return periods for these two events is used as an expression of the dike strength.

The mean return period for the situation where the water level reaches the top of the dike is calculated using the water level statistics for the particular location.

The situation with onshore waves has a mean return period which can be calculated using the value for the conditional probability  $P(\text{onshore waves} | \text{water level} > \eta)$  determined in section 6.

## 13. Conclusions.

A usable method has been developed to calculate the strength of dikes of varying quality. The method is based on results from rather simple tests on prototype dikes. The test results have been calibrated using re-

sults from real failures recorded at a Danish dike following a severe storm.

The model has been used to calculate the strength of about 30 Danish dikes. The results are consistent with the known lifetime of the dikes and the storms they have withstood without damage.

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