*Geotechnical Safety and Risk V*
*T. Schweckendiek et al. (Eds.)*

*doi:10.3233/978-1-61499-580-7-522*

# Safety Assessment Method of Flood Defences for Flow Sliding

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**Abstract.** Flow sliding of submerged slopes in front of dikes can affect the reliability of flood defences. The occurrence of flow sliding may result, sometimes with delay, in a failure mode and consequent flooding. The current safety assessment method for flow sliding does not consider the interaction between flow sliding and failure modes which can cause a breach. Therefore it is unclear whether the safety assessment method is sufficiently safe or too conservative. This paper presents a method to derive a simple safety assessment and design rule based on a number of advanced probabilistic analyses made for four Dutch cases, in which both flow sliding and the relevant direct failure modes have been incorporated. Two methods for the safety assessment of flow sliding are presented: as separate failure mode and coupled to direct failure modes.

**Keywords.** Flow sliding, flood defences, safety assessment, failure mode, liquefaction, breach flow

## 1. Introduction

The Dutch Water Act prescribes that the Dutch primary flood defences along coasts, rivers and estuaries have to be assessed for their reliability every 6 years (in future every 12 years). Dikes can fail by overtopping, backward internal erosion (“piping”), slip failure or damaged revetment. These failure modes are assumed to occur independently, so a safety assessment is done separately for each failure mode. Since their occurrence is positively correlated to the water level, they will *directly* lead to flooding. Another mechanism that may lead to flooding is flow sliding in the foreland of the flood defence. In the latest statutory Safety Assessment (2011), approximately 50 km of the primary flood defences was disapproved for flow sliding, mainly in the South-west of the Netherlands. Flow sliding involves the massive failure of submerged slopes composed of sand or silt. According to Van den Ham et al. (2014), two types of failure mechanisms are important: static liquefaction and retrogressive breaching, which both result in a sustaining turbidity current that resediments under a very gentle slope.

Unlike failure modes that directly lead to flooding, the occurrence of flow sliding is not (positively) correlated to water levels. I.e., flow sliding will not promptly lead to flooding. However, it influences the failure probability of direct failure modes, decreasing the reliability of the flood defence. Therefore, in most cases flow sliding can be considered as an indirect failure mode. The interaction between flow sliding and direct failure modes has not been investigated before, so the current semi-probabilistic safety assessment rules may be unreliable.

To derive a simple safety assessment method and design rule, the interaction between flow sliding and direct failure modes was studied and advanced probabilistic analyses were made. In section 2, the interaction between flow sliding and direct failure modes and the integration of flow sliding in advanced probabilistic analyses is described. Section 3 describes the application of these probabilistic analyses in four Dutch cases. On basis of these results, a safety assessment method for flow sliding is proposed.

## 2. Method

### *2.1. Interaction between Flow Sliding and Direct Failure Modes*

Flow sliding leads to reduction of the length of the foreland of the flood defence and may even lead to damage of the flood defence itself. A so-called damage profile is schematically shown in Figure 1. This changed geometry leads to an increase of loads or reduced strength of the flood defence, which influence the failure probability of direct failure modes.

Reduction in foreland length can lead to slip failure of the outer slope or can shorten the length of the seepage path for backward internal erosion, which means that the exit gradient of the hydraulic head increases. Furthermore, deepening and steepening of the foreshore leads to less wave breaking, thus larger overtopping. Larger retrogression of the foreshore can also lead to damage to the revetment. If retrogression is such that this leads to a reduction in dike height, overflow is more likely to occur. Moreover, flow sliding may influence the pore water pressure distribution within the sand core of the dike; hence the probability of slip failure of the inner slope and micro-instability may increase as well. However, the pore pressures are only influenced in case the impermeable cover of a sand dike is damaged, which means that the flood defence has already failed by the failure of the revetment. For this reason interaction with these failure modes can be ignored.

### *2.2. Quantification of the Interaction on the Probability of Flooding*

In the safety assessment of direct failure modes, the damage profile by flow sliding can be taken into account as a geometry-scenario with an occurrence probability. Under the condition that a flow slide (*fs*) occurs, the probability of a direct failure mode $P(fm)$ increases: $P(fm|fs)>P(fm)$. Therefore a summation over the scenarios flow slide and no flow slide is made, see Eq.(1).

$$P(fm, fs) = P(fm \mid fs) \cdot P(fs) + P(fm) \cdot \left(1 - P(fs)\right) \tag{1}$$

The probability of occurrence of flow sliding was calculated with a semi-empirical method described by Van den Ham (2014), which is based on a combination of statistics of documented historical flow slides in the Netherlands and sensitivity analyses with complex physical-based models describing mechanisms such as static liquefaction or breach flow. The damage profile as a consequence of a flow slide (expressed in affected bank length $L_{ba}$, see Figure 1) was taken into account probabilistically by an extreme value distribution, using a modified version of the empirical method by Silvis and De Groot (1995) (Van der Krogt, 2015).

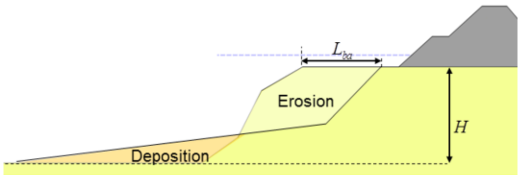


**Figure 1.** Damage profile after flow sliding.

### *2.3. Integration in Probabilistic Calculation Methods*

In a next step, flow sliding was integrated in the safety assessment of direct failure modes using state-of-the-art probabilistic methods for direct failure modes. These were adopted from among others the VNK-2 project (Jongejan et al., 2013) and current advanced design guidelines (Förster et al., 2012), (Rijkswaterstaat, 2014). The limit state equations were updated for the damage profile, by using the affected bank length as additional load. The next paragraphs only describe the essence of the probabilistic method, for specific information, reference is made to Van der Krogt (2015).

For overflow, the crest height will be reduced if a flow slide retrogresses farther than the inner crest line, see Figure 2. For backward internal erosion, the available piping length is decreased as function of the retrogression length, see Figure 3. The limit state equation for damage to the revetment is directly a function of the affected bank length, see Figure 4.

For slip failure and overtopping it is not possible to include the affected bank length as continuous random variable, since the used software (D-GeoStability) does not allow for this. Instead, for slip failure, the influence of damage profiles was estimated taking into account a number of geometry scenarios. The safety factor was determined for damage profiles corresponding to affected bank lengths that were expected to be relevant: retrogression until a distance of 20, 10 and 0 metre from the outer toe of the flood defence and two values for the steepness of the upper part of the flow slide damage profile. An example is shown in Figure 5. For wave increase by overtopping, only one scenario for retrogression until the outer toe of the flood defence was taken into account, since the wave load is determined by the water level above the toe of the outer slope. For this geometry-scenario, the probability of failure of overtopping was determined, see Figure 6.

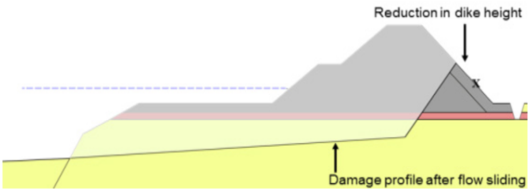


**Figure 2.** Reduction in crest height by flow sliding.

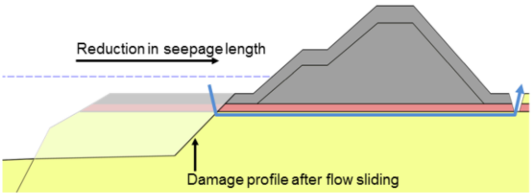


**Figure 3.** Reduction in seepage length by flow sliding.

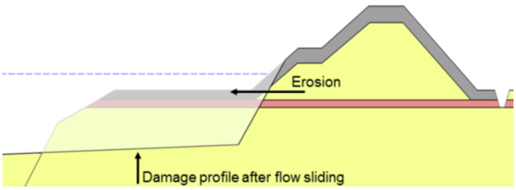


**Figure 4.** Damaged revetment due to flow sliding.

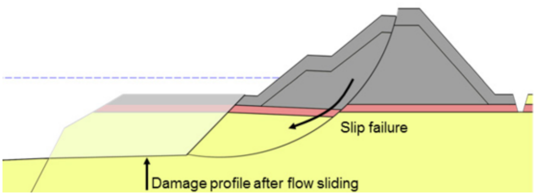


**Figure 5.** Slip failure due to flow sliding.

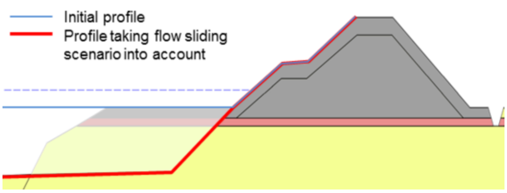


**Figure 6.** Increased overtopping due to flow sliding.

### *2.4. Probabilistic Methods*

For piping and overflow, probabilistic calculations were made using the limit state function, adapted as above described, in Monte Carlo (MC) and First Order Reliability (FORM) analyses. The FORM analyses were used to quantify the relative importance of uncertain factors such as the retrogression length and the length of time between occurrence of a flow slide and the moment of reconstruction of the damage profile. Influence of the latter was quantified by performing the calculations for three lengths of time till this repair. The MC analysis was used to check the result of FORM analyses.

## 3. Case Studies

### *3.1. Location Case Studies*

The method explained in the previous section has been applied on four cases in the Netherlands, which were known to be insufficiently safe to flow sliding, according to the current simple, and likely conservative assessment rules. These are Breskens, Burghsluis (both province of Zeeland), Spijkenisse (province of South-Holland) and Vierhuizergat (province of Groningen). Information on geometry of the levee and foreland were obtained from lidar and echosoundings provided by Rijkswaterstaat and waterboards. Information on the subsurface geometry (depth and thickness of sand and silt layers) and material parameters (density of grain size distribution of sand and silt layers) were obtained from TNO (GeoTop model). General results will be given. For more details on the calculations it is referred to Van der Krogt (2015).

### *3.2. Conclusions per Failure Mode*

Figure 7 gives a summary of the results per case study. Per case, two or three cross sections were analysed. Depicted are the probabilities of occurrence of overflow, overtopping and backward internal erosion, with and without taking flow sliding into account. Part of the results will be explored in more detail below.

The influence on the failure probability of overtopping by increased wave load is in all cases negligible. Although a deeper foreshore increases the overtopping discharge, the effect is only minor. Overflow however, is affected significantly by flow sliding, since the probability for retrogression that reduces crest height is large in these cases.

Somewhat less obvious are the results for the failure mode internal backward erosion. Figure 7 shows that the contribution of flow sliding on the probability on piping strongly varies between the case studies and cross sections. This is mainly dependent on the initial failure probability and the sensitivity to other factors like hydraulic conductivity and blanket layer thickness. For instance, the regarded cross sections in Breskens have a relatively high initial probability of internal backward erosion without flow sliding, but flow sliding is not likely to occur. At Burghsluis the initial probability of internal backward erosion is lower, so the influence of flow sliding is larger.

For slip failure, for which only deterministic calculations were made, a flow slide until the outer toe of the dike appears to have influence on the stability of the dike. It turn out, that the deeper the damage profile (Figure 5) is, the larger the influence of the slope stability. If at least 10 metre of foreland is remained, the stability is unaffected in all case studies.

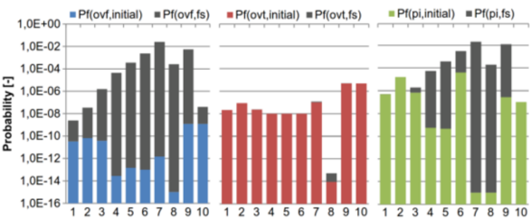


**Figure 7.** Probabilities of failure modes overflow (ovf), overtopping (ovt) and backward internal erosion (pi) with and without taking into account flow sliding (indicated with "initial" and "fs" respectively. Three cross sections in Breskens (1-3), three in Burghsluis (4-6), two in Spijkenisse (7-8) and two in Vierhuizergat (9-10)

### *3.3. Influence Parameters and Design Points*

From the FORM analyses it follows that for all considered direct failure modes the affected bank length is the most important stochastic parameter influencing the probability of failure, compared to the hydraulic load and resistance factors. In almost any case, the influence coefficient for the retrogression length amounts nearly 1.00. This means that the variation of the retrogression length contributes the most to the failure probability in the design point (the maximum likelihood point on the limit state line). This is explained by the large uncertainty in the retrogression length. This is not surprising, since, as mentioned, all cases were disapproved on flow sliding in the latest safety assessment.

For overflow, the design point is located at the inner slope where the reduced crest height equals the water level that has a return period equal to the time till repair. This is depicted by point X in Figure 2.

For backward internal erosion, the conclusion is similar: the design point for affected bank length is the required length of the seepage path for the water level that has a return period equal to the detection and repair time. Other stochastic parameters like the hydraulic conductivity and the thickness of the aquifer have only minor contribution. This means that a semi-probabilistic safety assessment for flow sliding can be based solely on the affected bank length.

Since the affected bank length is the only influencing parameter, the length of time till repair does not strongly influence the failure probability. The water level difference between 14 days and one year is in most cases not larger than 0.50m – 0.80 m. This corresponds to an only minor difference in additional retrogression length, which has a negligible influence on the probability of failure of the dike.

## 4. Safety assessment method

Since the primary flood defences in The Netherlands stretch out over a length of more than 3000 km, the safety assessment process is time-consuming. It is therefore relevant to make the procedure as efficient as possible, without reduction of the quality of the assessment. For this purpose, the assessment is done in several 'levels' that have to be passed through successively. The various levels are set up in such a way that a flood defence that meets the safety requirement in a certain level will never be rejected in subsequent levels. The first assessment levels are therefore more conservative and strict, whereas the subsequent levels are more accurate and generally more time consuming. For this reason it is proposed that the assessment for flow sliding consists of two levels:

1. A simple method in which flow sliding is assessed independently of the direct failure modes;
2. A more complex, but more accurate method in which flow sliding is assessed coupled to the direct failure modes.

A coupling to the direct failure modes involves application of the method explained in section 2 and applied in the case studies. Obviously this is the most precise way to take flow sliding into account in the calculation of the probability of flooding. However, this method requires a large number of time-consuming calculations, making the method less useful for engineering practice. Regarding flow sliding independently of direct failure modes is easier, but also more conservative.

If flow sliding is regarded then as a separate failure mode, a separate limit state function for failure by flow sliding is defined. Since the uncertainty of the affected bank length is the parameter with largest relative influence on the failure probability (paragraph 3.3), the affected bank length ($L_{ba}$) is the load and the maximum retrogression length ($L_{crit}$) is the resistance in the limit state function, shown in Eq.(2).

$$Z = L_{crit} - L_{ba} < 0 : \textit{failure} \qquad (2)$$

The critical length is defined as the maximum affected bank length, until the design point, which is calculated by the FORM analysis per failure mode. The value of the critical length in the limit state function ($L_{crit}$) must be the *lowest* maximum retrogression length. Since the influence of flow sliding on the direct failure modes is different depending on the material of the dike core, criteria can be optimized by taking the type of dike into account. From the case studies it follows that the design point is in most cases determined by two or three direct failure modes. For sand dikes, these are slip failure, damage to the revetment and piping. For clay dikes, only piping and overflow/overtopping are relevant. Overtopping is only relevant in case the dike height is decreased, not as an increase of hydraulic load due to deeper forelands.

### *4.1. Safety standard*

Dutch safety standards prescribe the maximum failure probability per dike segment. Since the failure modes are assumed to occur independently, the failure probability is distributed over the various failure modes by so-called failure probability budgets, expressed by the factor $\omega$ in Eq.(3), according to Arnold (2013) and Rijkswaterstaat (2014).

To obtain a target probability per cross section instead of per segment, the budget is divided by a factor $N$, which is a measure for the number of independent dike sections in a segment. This factor is dependent on the failure mode as well, so if flow sliding is considered as separate failure mode, the value of $N$ is different.

$$P_{T,section} = \frac{P_{T,segment} \cdot \omega}{N} \qquad (3)$$

For flow sliding, this distribution is dependent on the safety assessment method. An assessment on level 1 (independent of the direct failure modes) involves that it is checked that the

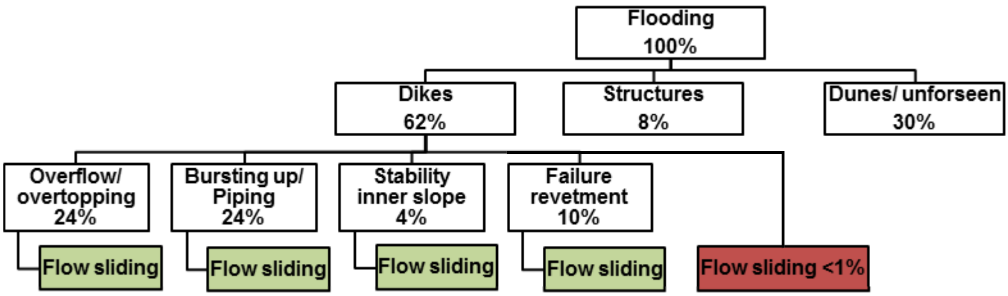


**Figure 8**. Fault tree indicating the target probabilities of failure of direct failure modes. The options for flow sliding in the assessment are indicated green (coupled to direct failure modes) versus red (separate failure mode).

contribution of flow sliding to the probability of flooding is "negligible", i.e. less than 1% of the maximum failure probability per dike segment. If the safety assessment of flow sliding is done coupled to the direct failure modes (level 2), the failure probability budget per direct failure mode is applicable, see Figure 8.

## 5. Conclusions

This paper presents a new assessment method for the failure mode flow sliding. The method consists of two levels that have to be passed through consequently. The first level comprises a simple method in which flow sliding is a separate failure mode, whereas on the second level flow sliding is analyzed coupled to the direct failure modes. The first level is easier but more conservative than the latter. The first method consists of a single criterion for the maximum allowed retrogression length, (possibly dependent on the dike type: clay or sand). In this case, the contribution of flow sliding to the probability of flooding should be "negligible". On both levels the assessment can be done probabilistic or semi-probabilistic. In the near future, the proposed safety assessment method will be implemented in the Dutch safety assessment.

The presented assessment method is based on a combination of statistics of historical flow slides and physics-based models. The incorporation of the results of physics-based models is promising, but needs further development. The respective factors in the semi-empirical equation (mentioned in paragraph 2.2) need to be adapted accordingly. Although steps forwards are being made, e.g. within the recently started STW-programme MPM-Flow, it is expected that prediction of flow slides by physics-based models only will be possible for at least another couple of years.

## Acknowledgment

This research was part of the project Wettelijk Toetsinstrumentarium (WTI), funded by Rijkswaterstaat on behalf of the Ministry of Infrastructure and the Environment, the Netherlands.

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## List of symbols

| | |
|---|---|
| $L_{ba}$ | Affected bank length |
| $L_{crit}$ | Maximum length of retrogression for which flooding occurs |
| $N$ | Number of independent dike sections in a dike segment |
| $P_f(fm)$ | Probability of failure, direct failure mode, excluding flow sliding |
| $P_f(fm,fs)$ | Probability of failure, direct failure mode, including flow sliding |
| $P(fs)$ | Probability of occurrence, flow sliding |
| $P_{T,segment}$ | Target probability of flooding per segment, according to safety standard |
| $P_{T,section}$ | Target probability of flooding per section. |
| $Z$ | Limit state equation |
| $\omega$ | Part in failure probability budget |