Flow slides in the Netherlands: experience and engineering practice

F. Silvis and M.B. de Groot

Abstract: This paper deals with flow slide experience in the Netherlands. It gives an extensive survey of empirical field data concerning the geological characteristics of the region, the profiles just before the flow slides, and the damage that flow slides can cause. The traditional Dutch procedure for analyzing the flow slide potential of underwater sand slopes is described. This is an engineering approach that is mainly based on the empirical field data, together with geological surveys and soil investigations performed at the relevant site.

Key words: sand, collapse, flow slides, liquefaction, slope stability.

Résumé : Cet article a pour objet l'expérience acquise aux Pays-Bas en matière de rupture par coulée. Il consiste en un relevé complet des données de terrain empiriques concernant les caractéristiques géologiques de la région, des profils obtenus juste avant les coulées et des dommages que celles-ci peuvent causer. On décrit la procédure traditionnelle utilisée aux Pays-Bas pour analyser le potentiel de coulée de pente sous-marines en terrain sableux. C'est une approche de type ingénieur, basée principalement sur les données de terrain empiriques couplées à des relevés géologiques et à des reconnaissances de sol effectuées sur le site à étudier.

Mots clés : sable, rupture, coulées, liquéfaction, stabilité de pente.

[Traduit par la rédaction]

Introduction

A liquefaction flow slide is a phenomenon in which a mass of saturated sand on a slope suddenly liquefies and flows out into a very gentle slope. Flow slides can cause a lot of damage if dikes or other structures are founded on the sand. Many hundreds of flow slides have occurred along the coastlines of the Dutch province of Zeeland in the last 200 years.

Flow slides take place in loosely packed fine sands. They are initiated by a relatively quick change in shear stress, which results in a tendency to decrease in volume: contraction or (negative) dilation. This tendency causes a rise of internal pore pressure, which reduces the effective soil stresses, generating a sudden reduction in shearing resistance. Eventually the frictional resistance of the sand may, to a great extent or completely, disappear and a flow slide starts.

This paper deals with practical information and traditional criteria for flow slides in the Netherlands. It gives a survey of field experience in Zeeland, based on an inventory of instabilities. The conditions under which flow slides occur and the damage caused is discussed. The paper also describes how this general information and local test data are traditionally used to make a conservative assessment of the liquefaction potential of underwater sand slopes.

Field experience in Zeeland

Geography and geology of Zeeland

The province of Zeeland in the Netherlands consists of a group of islands separated by wide tidal estuaries and forming part of the delta of the rivers Rhine, Meuse, and Scheldt (see Fig. 1). Coastlines and channels have been moving all the time due to alternating sedimentation and scour caused by river flow, tidal currents, and sea-level variations.

The geology of Zeeland is the result of the same processes of sedimentation and scour (Rijks Geologische

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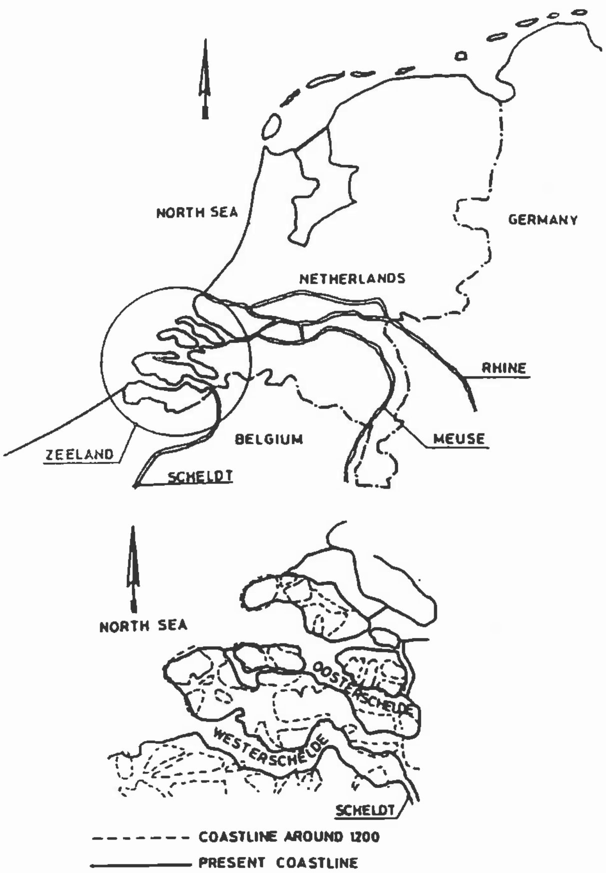
Fig. 1. Map of Zeeland showing location of flow slides.



Dienst 1978). A typical geological profile is shown in Fig. 2. The Dunkirk and Calais deposits consist of sandy tidal channel fills and clay deposited on subtidal flats. The Dunkirk and Calais phases are the two transgressive phases of the Holocene. They are separated by the Holland peat, deposited in a regressive phase. The two Pleistocene deposits are the Twente Formation of fine-grained aeolian sand and medium-grained fluvioperiglacial sand and the Tegelen Formation of fluvial deposits of sand with gravel and some clayey, sandy silt.

At several locations, the older deposits have been eroded and replaced by Dunkirk and Calais channel fills. These fills are up to 30 m thick. It is known that tidal channel fills are deposited very quickly without strong wave action (Reineck and Singh 1980). The grain fabric is therefore very loose, which makes it susceptible to liquefaction, as discussed below.

Flow slide inventory

Angles and heights of shore slopes and channel slopes have been changing as a result of scour and sedimentation as well, resulting in numerous slope instabilities (Koppejan et al. 1948). Hundreds of instabilities have occurred in this dynamic environment during the last 200 years. Wilderom (1979) made an inventory of 1129 instabilities in Zeeland during the period 1800–1978. Two types of slope instabilities can be distinguished: shear failures and flow

Fig. 2. Geology of Zeeland.

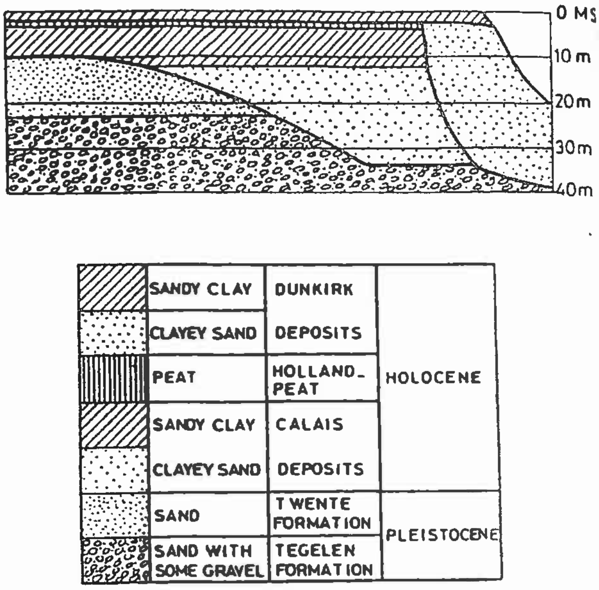
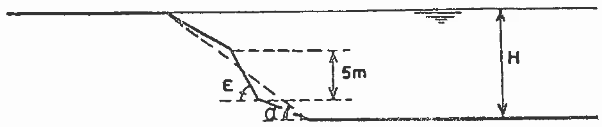


Fig. 3. Initial profile before sliding.



slides. No liquefaction occurs with shear failures and t slope after sliding remains relatively steep. The slop after sliding of 75% of these instabilities were so gen that Wilderom concluded that they must have been fl slides and not shear failures. He considered an instabil to be a flow slide if the angle of the lower part of t slope, γ, was smaller than 5.7°.

The exact locations of about 700 flow slides are know Ligtenberg-Mak et al. (1990) compared the occurrence flow slides with the geological characteristics of the locations. Most of the flow slides (about 600) occurred locations where there is a channel fill of Dunkirk deposi and about 100 occurred in a Calais deposit. Only ab 10 are thought to have occurred in a Pleistocene fill.

At 145 of the 700 flow slide locations regular de soundings had been taken at least once a year perpendicu to the shore at intervals of about 100 m. These flow sli were described in detail by Davis (1983). The profiles j before and just after these 145 flow slides were analyz

Profiles before sliding

The geometric characteristics of the profiles of a ba before a flow slide are defined in Fig. 3. An import characteristic is the steepest part of the slope ε over a m imum height of 5 m. In 95% of the flow slides this slo was steeper than 14.0°. More information about the val of ε and the other characteristics is presented in Table

The distribution of these parameters shortly befor flow slide is almost identical to the distribution of th parameters for banks that remain stable for at least 5 ye

Table 1. (sliding.

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Table 2. (

β, Slop (9
γ, Slop (9
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Profiles

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Table 1. Geometric characteristics of profile before sliding.

		Mean	Range
ε,	Steepest slope over 5 m	23.1°	11.3–45.0°
α,	Mean slope before sliding	14.5°	6.3–33.7°
H,	Channel depth	27 m	11–45 m

Table 2. Geometric characteristics of profile after sliding.

		Mean	Range
β,	Slope β upper part	20.7°	7.1–18.4°
	(95% $\beta > 9.5°$)		
γ,	Slope—lower part	3.6°	1.9–5.7°
	(95% $\gamma > 2.3°$)	1:45.9	1:30.0
δ,	Mean slope after sliding	9.6°	3.2–14.0°
	(95% $\delta > 3.8°$)	1:5.9	1:19.9 1:4.0
D,	Depth with slope β	11.6 m	4–18 m
H,	Channel depth	27 m	11–45 m
L,	Length of bank affected (95% $L < 170$ m)	80 m	10–250 m
S,	Horizontal distance between intersections of initial and end profiles	167 m	55–395 m
V,	Slide height	19.3 m	6–30 m

(Silvis 1985). Apparently it is not possible to conclude that flow slides are to be expected soon on the basis of the shape of the slope geometry alone.

Profiles after sliding

Characteristic of the profile after sliding is the gentle slope of the lower part and the steep slope of the upper part (see Fig. 4). The geometric characteristics are summarized in Table 2. Flow slides can cause a lot of damage: large areas under and above mean sea level can disappear. The total sand volumes of the 145 flow slides are summarized in Table 3.

Measures to protect banks against flow sides

Before 1800 protective measures were not undertaken to prevent the erosion of underwater slopes. Attempts were made to restrict the damage caused by a breach in a dike by constructing secondary dikes behind the primary dike before the occurrence of the next flow slide. Then the flood damage was only small. In this way the crescent-shaped northern shoreline of the west part of the Oosterschelde has been created by repeated withdrawal. After 1800 attempts were made to protect the banks at most threatened locations using simple fascine mattresses.

Around 1880 a new protection method was introduced at some locations: the fixed point method. Groynes were constructed underwater 1800 m apart to protect the underwater slope against scour by guiding the flow away from the slope. History has shown that this was not a good solution. In the Leendert Abraham Polder (Noord Beveland, southern shore of the west part of the Oosterschelde) two such groynes suffered many small shear failures and flow slides. The defences had to be extended continuously, as instabilities continued to occur in unprotected areas. The groynes were eventually connected in 1965 providing a continuous foreshore protection.

Table 3. The sand volume in 145 flow slides in Zeeland.

Volume in 1000 m^3	No. of flow slides
<100	62
100–200	41
200–500	27
500–1000	11
>1000	4

Table 4. Protected bank length (in km) of the Ooster and Westerschelde.

Underwater slopes[a]	Total	Protected	Unprotected
Susceptible	54	30 (56%)	24 (44%)
Possible susceptible	48	27 (56%)	21 (44%)
Not susceptible	45	20 (44%)	25 (56%)
Total bank length	147	77	70

[a]Susceptibility to liquefaction according geological criteria.

From 1880–1890 both mattresses and layers of rubble have been used. Annual depth soundings proved that from then on the erosion of many underwater slopes ceased. Apparently a rubble layer mass of 1000 kg/m^2 was adequate. This represented an average layer thickness of 0.6 m. The flanks of shore defences appeared to be weak points in the slope defence. Hardly any instabilities occurred in the middle of shore defences.

An inventory of the banks of the Oosterschelde and the Westerschelde in 1890 showed the length of bank protection to be about 47 km. In 1987 the length of bank protection was 77 km. The total length of channels near banks did not change in the intermediate period (147 km). The lengths of protected and nonprotected banks are summarized in Table 4. There is no significant relationship between the presence of bank protection and the geological characteristics of the bank.

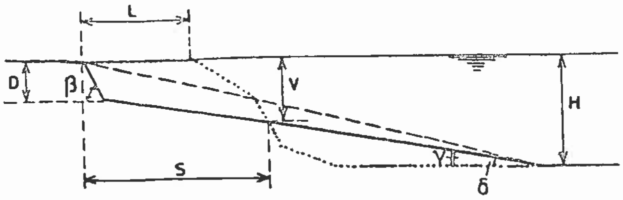
As a result of coastal protection works the number of flow slides decreased, but they still occur along the banks or shores from time to time.

Traditional analysis of flow slide potential

General

It is commonly accepted (Koppejan et al. 1948, Kramer 1988) that three conditions must be met to trigger a flow slide in an underwater slope: (I) the soil must be susceptible to liquefaction (loose sand); (II) the slope must be relatively steep and relatively high; and (III) an initiation mechanism must be present.

Fig. 4. Final profile after sliding.



The third condition is not considered further here because there will always be an initiation mechanism: a sudden, local change in water pressure due to waves from a passing ship or a wind wave, an increase of outflowing groundwater during an extreme low tide, a quickly changing soil pressure due to a local shear failure or due to dredging activities, vibrations caused by pile driving, and so on.

The first condition is discussed in the next paragraph. If only this condition is met, then the term "susceptible to liquefaction" is used, a property of the soil, not the slope. No liquefaction, however, will occur unless also the second condition is met. Then the slope is said to be "susceptible to flow slides."

Susceptibility to liquefaction

The question of whether or not the sand at a certain location is loose enough to be susceptible to liquefaction is discussed here. Usually this question is approached in three steps: (1) a preliminary survey based on geological information; (2) a rough assessment based on cone penetration tests (CPT's) and borings; and (3) a detailed assessment with the help of critical density tests and in situ density measurements.

1. Preliminary survey based on geological information

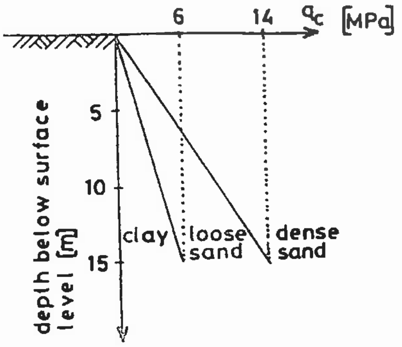
Geological maps may give answers to the following questions: Is the soil part of a recent marine deposit? Is the soil part of a channel which has filled rapidly?

One of the conclusions in the Wilderom inventory is that flow slides only occurred in young Holocene sand. However, there are now new opinions about geological aspects in relation to flow slides. Although, as mentioned above, nearly all flow slides occurred in Holocene deposits, Ligtenberg-Mak et al. (1990) conclude that all tidal channel fills and rapidly deposited fluvial channel fills are susceptible to liquefaction, whether Holocene or Pleistocene. The places with potential for flow slides have been located by mapping these sediments.

2. Rough assessment based on CPT's and borings

Rough assessments are made mainly from cone penetration tests, during which sleeve friction and, sometimes, pore pressures are measured. These provide information about soil type and layer configuration. Often some borings are made and soil samples classified to help the interpretation. The CPT's also give direct information about the susceptibility to liquefaction in one of the following ways: (*i*) A cone resistance between the values shown in Fig. 5 indicates susceptibility to liquefaction. This relationship is based on experience in Zeeland and harbour construction works in deposits of Holocene sand under water, mean grain diameter between 120 and 250 μm and a vertical

Fig. 5. Cone penetration test (CPT) values, depth below ground level and soil type.



effective grain stress smaller than 150 kPa. (*ii*) The density index, I_D, is found with a correlation formula (Lunne and Christophersen 1983; Schmertmann 1978). If $I_D < 30\%$ the sand is certainly susceptible to liquefaction; if $I_D > 60\%$ it is not.

3. Detailed assessment based on critical density tests and in situ density measurement

A detailed assessment requires laboratory tests to determine the (dry) critical density and the "wet critical density" at least for one level of the mean effective stress representative for the conditions in the field. If sand has a density higher than the dry critical density, it is dilatant and consequently it is certainly not susceptible to liquefaction. If sand has a density slightly lower than the dry critical density and is just slightly contractant, it is also not susceptible to liquefaction. The density critical for liquefaction is probably very near to the wet critical density determined in the way described by Lindenberg and Koning (1981) with a void ratio just above the steady state line. The effective stress path during this test, however, is not entirely the same as that in situ during the beginning of a flow slide. Therefore, and for safety reasons, the density critical for liquefaction is generally assumed to be half way between the dry and wet critical densities. The resulting porosity, n, is often found to be 1–3% higher than the critical value of n at the dry critical density.

The critical density from the laboratory tests must be compared with the density of the in situ sand. Delft Geotechnics has developed a device that is built into a CPT apparatus, which determines the in situ density indirectly. For saturated sands it is possible to measure the electrical conductivity of the soil and the pore water in situ. The ratio of these two electrical conductivities yields the density of the sand after calibrating soil samples in the laboratory (LGM Communications 1977).

Susceptibility to flow slides

The susceptibility to flow slides of a bank of liquefiable sand depends on the geometric characteristics of the bank: condition II is met if the slope is steep enough and high enough. The angle and height of the slope influence the susceptibility to flow slides in two ways: (*i*) A minimum shear stress level and a minimum effective stress level are needed for liquefaction to occur; these are generally only

Fig. 6. Expected damage with a wide channel.

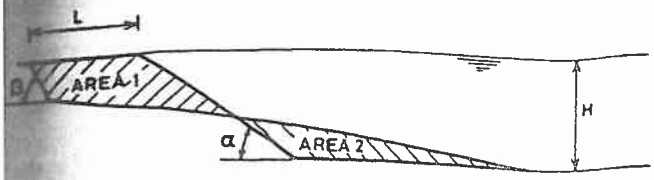
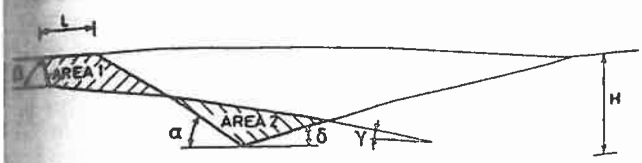


Fig. 7. Expected damage with a narrow channel.



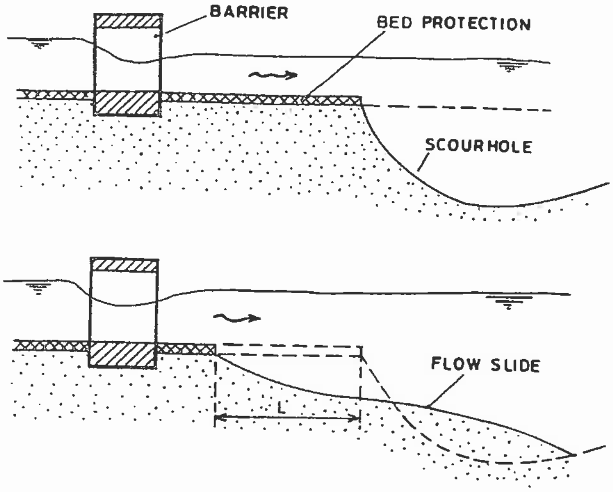
present at some depth below the slope surface if the slope is sufficiently steep and if the steep section extends over sufficient height. (*ii*) Liquefied sand will only flow if forced by gravity; the flow is only significant if this force remains sufficiently strong over the period of liquefaction; the force is only so strong if the slope is sufficiently steep and the period of liquefaction is only so long if the liquefied layer is thick enough, which is only the case with sufficient slope height.

These qualitative conditions used to be quantified with the help of the empirical data about the profiles before sliding described in the previous section. Since 70% of the flow slides occurred with $\varepsilon < 18.4°$ (definition of ε is given in Fig. 3), engineers in Zeeland generally conclude that a slope is only susceptible to flow slides if the steepest section ε over a height of 5 m is steeper than 18.4°. As mentioned above there is no statistical justification for this conclusion. Application of this criterion or a similar one, however, has given acceptable results in most parts of the Netherlands over the last 40 years.

This criterion is probably conservative, which is not very satisfactory. The wide spread in the values of ε and α at which flow slides occur also suggests that it is too simplistic to state that condition II can be formulated in terms of one critical combination of slope angle and height for all situations in which the sand is loose enough to meet condition I. It will probably make a difference if the sand is just loose or very loose. In other words the critical combination of slope height and slope angle will also depend on the "degree of susceptibility to liquefaction." Data on the sand characteristics, discussed in the previous section for condition I, must therefore be combined with the data on the geometry of the slope to find out whether or not condition II is met.

A mathematical model has been developed to combine these data and to develop a method to find a criterion for the susceptibility to flow slides, which is more precise than applied to date in the traditional engineering approach described in this paper. The characteristics of the sand have been formulated in terms of constitutive equations, mainly derived from drained triaxial tests on loose and very loose sand. The geometrical properties have been formulated together with the equations for momentum and compatibility. This model will be discussed in one or two separate papers (de Groot and Stoutjesdijk[1] and Stoutjesdijk et al.[2]).

Fig. 8. Bed protection for the Oosterschelde Barrier.



Expected damage

If the possibility of a flow slide cannot be ruled out, it is necessary to estimate the damage that may occur as a consequence. There are no theoretical models for describing the movement of the density current flowing along the slope and the sedimentation of the liquefied sand. The empirical data concerning the profiles after sliding described in the previous section are therefore used to estimate the damage.

The damage is often sufficiently characterized by the length of the bank affected, L. The range for L, indicated in Table 2, is very large (10–250 m). L and the intersection height, V, can also be calculated on the basis of a sand balance by comparing the initial and end profiles (Figs. 3 and 4). For the two-dimensional geometry of Fig. 6, with a very wide channel with a horizontal bottom, and the condition that area 1 is equal to area 2

$$[1] \quad L = \frac{H}{2}a - D\left(1 - \frac{D}{2H}\right)b$$

and

$$[2] \quad V = \frac{H}{2} + \left(\frac{D^2}{2H}\right)\left(\frac{b}{a}\right)$$

where $a = \cot\gamma - \cot\alpha$ and $b = \cot\gamma - \cot\beta$.

The end profile parameters, D, β, and γ can be estimated from the data given in Table 2. For example: with $H = 10$ m, $\cot\alpha = 2$, $\cot\beta = 5$, $\cot\gamma = 16$, and $D = 0.43H$, the formulas yield $L = 33$ m and $V = 5.7$ m.

These formulas are developed for situations in which the liquefied sand could flow out freely. In some cases,

[1] M.B. de Groot and T.P. Stoutjesdijk. Steady state of loose sand predicted from drained tests. Submitted for publication.

[2] T.P. Stoutjesdijk, M.B. de Groot, and J. Lindenberg. Flow slide prediction method: influence of slope geometry. Submitted for publication.

however, there is limited space into which the liquefied sand can flow, and consequently the damage in terms of L will be limited.

This favourable effect can be quantified in the following way. If the width of the channel is small, the area 2 is restricted by the slope angle δ of the opposite side of the channel, see Fig. 7. If area 1 is equal to area 2

$$[3] \quad L=\frac{-2He+\sqrt{4(He)^2+(a-2e)(D^2b+2H^2e)}}{a-2e}a-Db$$

and

$$[4] \quad V=\frac{-2He+\sqrt{4(He)^2+(a-2e)(H^2e+D^2b/2)}}{a-2e}$$

where
$$e=\frac{\sin(\alpha-\gamma)\sin(\alpha+\delta)}{2\sin^2 a\sin(\delta+\gamma)}$$

An illustration of the effect of a limited area in the channel is given by introducing cot $\delta = 4$ in the above example. Now $L = 20$ m instead of 33 m.

Application

How the approach can be applied in practice is illustrated with the example of the design of the bed protection of the Oosterschelde Barrier (Davis and de Groot 1983). This storm surge barrier has been constructed in the mouth of the largest tidal estuarium in Zeeland, the Oosterschelde.

The barrier is only closed in the event of very high water levels. Normally tidal currents run through the barrier and cause scour holes at the end of the bed protection adjacent to the barrier (Fig. 8). These scour holes endanger the barrier in the following way: as soon as excessive scouring has taken place, a flow slide can occur at the edge of the bed protection. This flow slide could reach the foundation of the barrier if the bed protection is too short.

The safety of the barrier is guaranteed by taking measures to restrict the depth and slope of the scour hole in order to prevent a flow slide. In addition, the length of the bed protection was chosen long enough to avoid any flow slides that might still occur reaching the barrier.

This design was based on an extensive soil investigation, including in situ density measurements, which clearly showed the sand was susceptible to liquefaction in nearly the whole area. To meet a safe criterion to avoid condition II, measures were taken to limit the depth of the scour hole to 50 m below the existing sea floor and the steepest section of the slope to an angle smaller or equal to 11.3°. Finally, the bed protection length chosen was much larger than 200 m, the value of L exceeded by about 3% of the observed L values (Table 2).

Conclusions

Many flow slides have occurred in Zeeland in the Netherlands. Hundreds of these flow slides have been recorded during the last 200 years. Much valuable empirical information has been derived from these records and applied in practice.

Although most of the flow slides occurred in Holocene deposits, Pleistocene formations may also be susceptible to liquefaction. The way the sand is deposited is crucial: channel fills are generally loosely packed and susceptible to liquefaction.

A general idea about the susceptibility of a soil to liquefaction can be obtained from a geological survey. A further, rough assessment requires cone penetration tests from which the density index can be estimated with the help of correlation formulas. A thorough assessment requires in situ density measurements which should be compared with the dry and the wet critical densities determined in the laboratory.

Whether a slope that is susceptible to liquefaction is also susceptible to flow slides, is traditionally estimated with the help of the empirical data on slope angles and slope heights at which flow slides have occurred. The statistical evidence for the influence of these geometrical parameters is, however, poor. Nevertheless a conservative criterion for susceptibility to flow slides can be found in this way.

A less conservative determination of the influence of the geometrical characteristics of the slope requires advanced mathematical modelling, in which the influence of density and the constitutive properties of the soil is incorporated.

The damage caused a by flow slide can be estimated with the help of the empirical data from flow slides in the past. Reduction of the damage in cases where the space for the sand to flow to is limited can be calculated.

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List of symbols

a cot γ − cot α
b cot γ − cot β
e function defined by eq. [4]
D height of slope with angle β (Fig. 4)
I_D density index = $(e_{max} - e)/(e_{max} - e_{min})$, where e, e_{min}, and e_{max} and the void ratio, minimum void ratio, and maximum void ratio, respectively
H channel depth
L length affected by flow slide (Fig. 6)
n porosity
S horizontal distance between intersections of initial and end profiles (Fig. 4)
V slide height (Fig. 4)
α mean slope angle before sliding (Fig. 3)
β slope angle upper part after sliding (Fig. 4)
γ slope angle lower part after sliding (Fig. 4)
δ mean slope angle after sliding (Fig. 4) or slope angle of opposite side of channel (Fig. 7)
ε steepest slope angle over 5 m (Fig. 3)

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1) Onderstaande tekst geeft de beschrijving op een door Deltares voorgestelde aanpassing van het plan van aanpak 2009 van SBW Piping Rivierdijken. ~~Tevens wordt een toelichting~~
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~~vragen van de Waterschappen n.a.v. het plan 2009~~
Tevens wordt kort gehandeld vragen van de waterschappen n.a.v. het plan 2009 de beantwoorden

2) SBW Rivierdijken

- 1130 meetraaien < 75% f < 1:10 → RV → 95% $\alpha_{sm} > 1:4$
  25% f > 1:10 → AF
- pangeometrie zegt niet zoveel
- rapport Sellmeijer 1985 !