Geologie en Mijnbouw **73:** 375–379, 1994.

Safety of dikes and embankments in the Netherlands with special reference to earthquakes

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Received 28 June 1993; accepted in revised form 12 April 1994

Key words: hazard assessment, safety philosophy, sea dikes, river embankments

Abstract

The protection against flooding is an important topic in the Netherlands. Based on statutorily defined sea and river levels which have to be withstood, safety is expressed as an acceptable annual probability of flooding for each area protected by water-retaining structures. This hazard assessment considers primarily the basic threat of extreme high water levels alone or in combination with (specific) storm conditions. However, combinations of high water levels with unrelated damaging events may also contribute to the total flooding hazard. This paper describes the general approach for the assessment of the possible combined failure mechanism leading to the scenario 'flooding due to earthquake-induced damage'. Based on the results of simplified slope stability calculations, the conclusion is drawn that the contribution of this scenario to the total probability of flooding is exceedingly small for the lower river region of the Netherlands.

Introduction

After the flood disaster in 1953 in the south-western Netherlands, a Delta Committee was installed by the Dutch government. The report of this committee (Delta Commissie 1960) has been used as the basis for the so called Delta Law. In this law regulations are given for the sea water levels and their frequencies of occurrence that must be withstood per region along the Dutch coast. Also, the required safety against inundation is prescribed in general terms. Some years later similar regulations were introduced for river dikes in the inland parts of the Netherlands.

The most important new component in flood protection philosophy is the fact that the design sea levels are based on statistics of local high water levels and river discharges recorded during the last 100 years. These statistical data have been extrapolated to much lower frequencies of occurrence than previously and, after analysis of economical aspects and 'social acceptance' of flooding risks, an 'optimum' has been obtained (Delta Commissie 1960). This constitutes a significant change from dike-heightening policy in the centuries before 1953, when the highest measured sea level served as the starting point. Apart from the fact that the new approach has led to much higher dikes than before, it has created an increasing awareness of the probabilistic character of high water levels. Probabilistic methods are now, in principle, accepted practice for the design of dikes and other water-retaining structures in the Netherlands.

The statutory regulations in the Delta Law are formulated in strong but quite general terms. The ministerial 'Technical Advisory Committee on Water Defences' (known as 'TAW' Committee) has been installed to elaborate these regulations and to develop detailed criteria for a) safety against inundation, and b) design, construction and control of water-retaining structures in the Netherlands. These criteria have been (and still are) published by the TAW Committee, in a number of handbooks and guidelines.

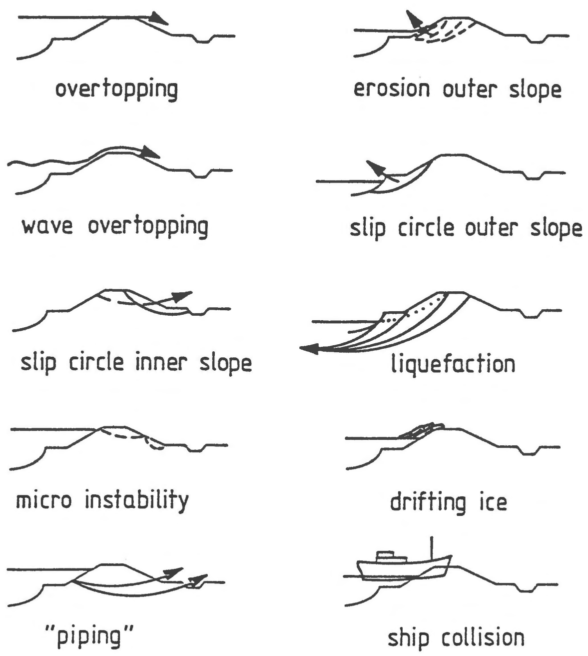

Fig. 1. Schematic presentation of the main failure mechanisms of a dike.

With respect to the potential risk of flooding of an area protected by embankments following an earthquake, provisional and rather approximate analyses have been made in the past. These analysis have led to the conclusion that an earthquake, considering its low frequency of occurrence, does not significantly increase the risk of flooding. This conclusion applies also to sea dikes in the western part of the country. However, some uncertainties remained. This applies particularly to earthquake-induced loading conditions. Especially the probability of local ground accelerations is of great importance for the decision whether new analyses have to be made in the near future. As the design earthquake loading levels likely to be experienced by water-retaining structures are refined by modern methods and earthquake observations, these uncertainties and conclusions require reappraisal.

In this contribution attention is given to the failure mechanisms of a dike or embankment, to the approach for safety assessment, and to some examples of simple dike stability calculations.

Failure mechanisms

A number of failure mechanisms can be distinguished (Fig. 1; Pilarczyk 1990), principally:

- failure due to overtopping, where the crest of the dike is too low in relation to the external water level and/or the wave run-up and wave overtopping,
- erosion and failure of the outer slope, initiated by damage of the slope protection (e.g. stone revetment, grass) due to wave attack,
- piping, i.e. the gradual formation of a material entraining well at the inner toe of the dike may lead to settlement and failure,
- slip failure in dike slopes, which may be caused by a high groundwater level in the dike, e.g. when a river drawdown quickly follows a high water level,
- liquefaction of foreshore and dike body, for example a flow slide failure in loosely-packed sand may occur in the foreshore.

All the above-mentioned failure mechanisms may take place during conditions of high external water level, implying that an inundation is likely. However, dike failure mechanisms can be initiated by other, often completely independent events too. For instance, heavy damage of the dike may be caused by ship collision or drifting ice. Earthquakes are events independent of high water levels, which means that the probability of both events occurring at the same time will be very small. On the other hand, inundation may also occur after a dike has been severely damaged by a preceding earthquake. In that case the period required for repair of the damage becomes important.

Safety assessment of dikes and embankments

Earthquake motions may trigger several types of mechanisms, which may cause damage or malfunctioning of the dikes and embankments. This requires design checks for the consequences of earthquakes. Safety criteria in such an analysis must be assessed in relation to the acceptable risk of flooding of the protected area.

According to the statutory regulations, Dutch river dikes and embankments should be sufficiently strong to withstand river levels with an annual probability (average return period) of 1 in 3000 to 1 in 1250, depending on the flooding consequences for the protected area (e.g. urban, industrial or rural areas). For main sea dikes corresponding annual probability values are 1 in 10 000 to 1 in 4000. Though subject to discussion among civil engineers, these criteria indicate the order of magnitude of the acceptable annual risk of (substantial) flooding (Pilarczyk 1990). Contributions to the risk of flooding may originate from sources other than extreme water levels, e.g. from uncertainty about the strength of the dike (slope stability, piping), heavy rainfall conditions or earthquake-induced motions. It is generally assumed that such contributions represent only a small additional risk, e.g. in the order of a few percent for the total protected area or a few tenths of a percent for each dike section. This suggests an acceptable probability of earthquake-related hazardous failure in the order of 10^{-6} per year for each dike section. Besides the safety criterion, a criterion for acceptability of non-hazardous damage (i.e. damage without risk of consequential flooding) must be established. This issue, however, will not be addressed in this paper.

Safety criteria in terms of an acceptable probability of failure, must be translated into practical design criteria. Current design analyses involve deterministic computations, in which 'design' loading and strength parameters produce safety factors based on equilibrium equations for dike slopes. Where the assessment of 'design' parameters and required safety factors is based on probabilistic considerations, such methodology is referred to as semi-probabilistic.

The assessment of design parameters has been outlined in the European building codes (ENV 1998-1-1 1994). With the criterion of a probability for flooding due to an earthquake of 10^{-6} per year, the procedure combines probabilities of occurrence for an earthquake and a high water level. An earthquake loading corresponding to a probability of occurrence of about 5.10^{-4} (expected return period of 2000 years) in combination with a rather usual high water level in the river (approximately once in 2–10 years), have been defined as relevant design values. As a response criterion for the dike or embankment only a small stability reduction and limited residual settlement and deformation of the earth structure during such an earthquake can be permitted.

Table 1. Stability factors F for slopes with different gradients, and exposed to different peak ground accelerations (after Seed 1979).

Slope gradient	a (m/s^2)	Slope stability factor F with	
		p = 0%	p = 40%
1 in 2	1.5	1.00	0.52
	1.0	1.11	0.59
	0	1.40	0.77
1 in 3	1.5	1.38	0.77
	1.0	1.56	0.88
	0	2.10	1.22
1 in 4	1.5	1.68	0.96
	1.0	1.95	1.12
	0	2.80	1.64

a – peak ground acceleration; p – excess pore pressure.

Response of slopes during an earthquake

Methods for slope stability analysis during an earthquake range from simple and empirical (quasi-static) to very advanced and complex (fully dynamic). All these methods, which describe the physical phenomena in a more or less schematic way, are basically deterministic methods. However, the input parameters may be based on probabilistic considerations or the calculation results can be introduced in a (semi) probabilistic procedure. An advantage of the simple methods is that, to a certain extent, the practical applicability has been proven by comparing results of theoretical analyses with observations during or after earthquakes. Furthermore, the simple methods lead to rough but quick and useful results, whereas advanced procedures often require extensive efforts. For these reasons, preliminary analyses, carried out a few years ago, have involved simplified methods for slope stability calculations (Lindenberg 1991).

In a slope stability calculation the forces resisting sliding are determined along a large number of arbitrarily chosen planes in the dike body and compared with the driving forces. The analysis involves the determination of the minimum stability factor F for the most critical plane, in which F equals the ratio of the resisting and driving forces. The slope is stable for $F \geqslant 1$ and loss of stability has to be expected for $F < 1$. In practice a threshold value of $F = 1.30$ is accepted for static conditions (former deterministic approach for conditions during high river level). For the slope stability analysis during an earthquake a somewhat lower minimum stability factor is accepted and the following two aspects must be considered and, if relevant, included into the stability calculation:

1. additional loading in horizontal and/or vertical directions to simulate the quasi-gravitational forces due to seismic waves and (sub) surface accelerations;
2. additional internal excess pore pressures as a consequence of the cyclic (periodic) loading of loose

to medium dense sand in the dike body and the near-surface subsoil.

The first-mentioned additional forces can be obtained from estimated peak ground accelerations related to the local earthquake intensity. The determination of the additional internal pore pressures requires a separate analysis. Important factors are: earthquake magnitude and local intensity, the sand layer depth, groundwater level, and the relative density of the sand. A rough impression of the susceptibility for pore pressure generation and liquefaction can be obtained with an empirical method partly based on a comparison between observed response during earthquakes, local earthquake intensities, and results of in situ measurements like cone penetration tests (CPT) (Seed & Arango 1983). Such an analysis has been made for the general dike and subsoil conditions in the lower river region in the Netherlands (excluding the most sensitive south-eastern part). It was concluded that the occurrence of complete liquefaction on a large scale during an earthquake is very unlikely. Macroseismic intensity VI to VII corresponding to a return period of 2000 years (De Crook 1989) has been assumed. Nevertheless, in locations with loosely packed sand in or below the embankment, excess pore pressures of up to 40% may be generated.

Table 1 summarizes the results of some simple stability calculations (Seed 1979) for slope gradients 1 in 2, 1 in 3 and 1 in 4, ground accelerations (a) of 0 (no earthquake), 1.0 and 1.5 m/s^2 (0, 10 and 15% g respectively) and excess pore pressure percentages (p) of 0 and 40%. The resulting stability factors (F) apply to a slope of infinite length, which leads to conservative values. In Fig. 2 the active forces are indicated together with the formula for F based on an equilibrium equation for a plane parallel to the slope. The surface accelerations of 10 and 15% g agree approximately with earthquake intensity VII (Modified Mercalli scale). To indicate the potential danger of internal excess pore pressure, only the effect on cohesionless soil is shown in Table 1. The angle of internal friction of the sand is assumed at 35° and only additional loading in a horizontal direction has been introduced.

The calculated stability factors F indicate that even a relatively steep slope of 1 in 2 will remain just stable when no pore pressure is generated in the soil mass. If a 40% pore pressure is generated (because of rather loose sand in the dike body), then for a = 1.5 m/s^2 stability factors F < 1 are found for all three slope gradients. But, even then, damage may be limited for slopes 1 in 3 and 1 in 4. As can be seen in Table 1, after the earthquake (a = 0) the stability factor F is greater than 1; the total time period during which F is smaller than 1 will therefore be very short. Empirical procedures are developed for rough estimates of the degree of damage (Committee on Earthquake Engineering 1985). Application of these procedures for a 1 in 3 slope show that crest subsidence, and therefore reduction of the retaining height of the dike, will be limited and not exceed 0.1–0.2 m.

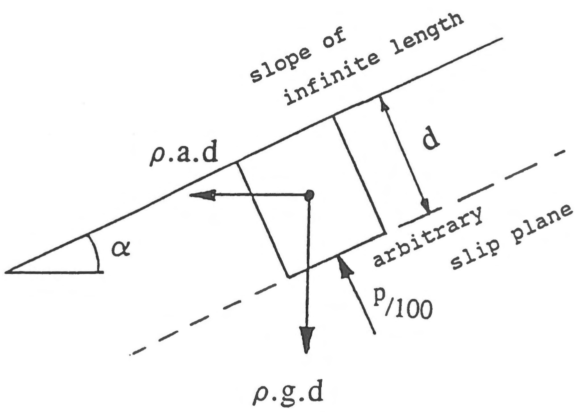

Fig. 2. Active forces in a slope of infinite length during earthquake shaking. The stability factor, F, is defined as:

$$F = \frac{\tan\phi}{\tan\phi_{mob}} = \frac{\cos\alpha - a/g\cdot\sin\alpha - p/100}{\sin\alpha + a/g\cdot\cos\alpha}$$

where:

ϕ – angle of internal friction of the soil, ϕ_{mob} – mobilized friction angle in the soil, a – peak ground acceleration (m/s^2), g – gravitational acceleration (m/s^2), α – slope angle with the horizontal, ρ – material density (kg/m^3), d – thickness of slipping layer (m).

Conclusion

Based on the results of simplified slope stability calculations (Table 1), it is concluded that the contribution of the scenario 'flooding due to earthquake-induced damage' to the total probability of flooding for the lower river region of the Netherlands, is very small.

Some danger is present for dikes and embankments with slopes steeper than 1 in 2.5 and consisting of, or founded on loose sand. Although such loose sand conditions may only be present very locally in the lower river area in the Netherlands, a more detailed analysis will be carried out in the near future. This analysis will consist of a number of finite-element computations including dynamic terms for a rather unfavourable but representative dike cross section. These computations may, among others, show the possible changes in stability with increasing acceleration amplitude.

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