Prepared for:

Rijkswaterstaat, Dienst Weg- en Waterbouwkunde

Breach growth in cohesive materials

Selection of cases

August 1999
Breach growth in cohesive materials

Selection of cases

J.H. de Vroeg
CLIENT: Rijkswaterstaat, Dienst Weg- en Waterbouwkunde

TITLE: Breach growth in cohesive materials, Selection of cases

ABSTRACT:

For a reliable prediction of the consequences of dike breaches, insight in the breach development is required. For this purpose the models BREACH and BRES have been developed. These models have been designed, calibrated and validated for sandy dikes.

For dikes consisting mainly of clay instead of sand, no guidelines for application of the above models are available. In the framework of projects like Normering Afsluitdijk, Marsroute inundatierisico and Hoogwater Informatie Systeem more insight in the breach development in dikes consisting of cohesive materials is required.

In this stage Rijkswaterstaat has decided to obtain rough guidelines for cohesive dikes on the basis of a short study. In this study a number of breach events which have occurred in clay dikes are simulated with the models BREACH and BRES. On the basis of a comparison of the model results (for sand) with the observations in nature (for clay), differences between breach development in a sandy dike and clay dike can be identified and guidelines for application of these models for clay dikes can be formulated.

In this report four cases of breaches in cohesive dikes are described.
The analysis of these cases with the models BREACH and BRES is described in a separate report (Futloo, 1999).

The cases described in this report are:
- Breach test at Tollesbury, 1995 (United Kingdom)
- Breach growth in foreland near Herkingen, 1953 (The Netherlands)
- Dike breach at Oudenhoom, 1953 (The Netherlands)
- Breach test at Yahekou (China)

REFERENCES:

REV. ORIGINATOR DATE REMARKS REVIEW APPROVED BY
J.H. de Vroeg 5-8-99

KEYWORDS CONTENTS STATUS
TEXT PAGES: PRELIMINARY
TABLES: DRAFT
FIGURES: FINAL
APPENDICES: PROJECT IDENTIFICATION: H3468
Contents

List of Figures
List of Tables
List of Appendices

1 Introduction.............................................................................................................. 1

2 Selection criteria and definitions........................................................................ 2
   2.1 Selection criteria ............................................................................................... 2
   2.2 Definitions......................................................................................................... 2

3 Description of cases............................................................................................. 3
   3.1 Breach test at Tollesbury 1995 (United Kingdom)......................................... 3
   3.2 Breach growth in foreland near Herkingen 1953 (The Netherlands)........... 5
   3.3 Dike breach at Oudenhoorn, 1953 (the Netherlands)..................................... 8
   3.4 Breach test at Yahkou (China)....................................................................... 10
   3.5 Possible other cases....................................................................................... 12

4 Conclusions and recommendations..................................................................... 13

List of References
List of Figures

3.1 Location of Tollesbury
3.2 Cross-section of dike Tollesbury
3.3 Breach development embankment Tollesbury
3.4 Measured inner and outer water level Tollesbury
3.5 Location of Herkingen (Goeree-Overflakkee)
3.6 Illustration of situation near Herkingen at 3 and 7 February 1953
3.7 Breach development foreland Herkingen
3.8 Outer water levels Herkingen
3.9 Location of Polder Oudendoorn
3.10 Cross-section of dike Oudendoorn
3.11 Assumed breach development Oudendoorn
3.12 Outer water level Oudendoorn
3.13 Location of Yahekou
3.14 Cross-section of dam Yahekou
3.15 Breach development dam Yahekou
3.16 Measured inner and outer water level Yahekou
List of Tables

3.1 Summary of cases
List of Appendices

A  Eye-witness reports of levee breaches during Mississippi floodings
B  Rate of erosion of clay-sand on the basis of data presented in literature
   (H.J. Verhey, In Dutch)
1 Introduction

For a reliable prediction of the consequences of dike breaches, insight in the breach development is required. For this purpose, the models BREACH and BRES have been developed. These models have been designed, calibrated and validated for sandy dikes.

For dikes consisting mainly of clay instead of sand, no guidelines for application of the above models are available. However, in the framework of projects like Normering Afsluitdijk, Marsroute inundatierisico and Hoogwater Informatie Systeem more insight in the breach development in dikes consisting of cohesive materials is required.

In this stage Rijkswaterstaat has decided to obtain rough guidelines for cohesive dikes on the basis of a short study. In this study a number of breach events which have occurred in clay dikes were simulated with the models BREACH and BRES. On the basis of a comparison of the model results (for sand) with the observations in nature (for clay), differences between breach development in a sand dike and a clay dike could be identified and guidelines for the application of these models for clay dikes could be formulated.


In this report four cases of breaches in cohesive dikes are described, which can be used for the above described study. This report has been written by Mr. J.H. de Vroeg of WL|DELFT HYDRAULICS.

The model computations based on these cases and the comparison of results with the observations are reported by Futlool (1999).
2 Selection criteria and definitions

2.1 Selection criteria

Cases have been selected on the basis of the following criteria:

- There should be sufficient indications that the considered dike consisted mainly of clay in the period that breaching occurred.
- The breach width should be known in sufficient detail. In the validation study of 1998 (for sandy dikes) it was found that only for very few cases detailed information on the initial breach and of breach development with time is available. Therefore some assumptions related to breach development seem inevitable.
- The conditions should be known in sufficient detail. In general, accurate data can be found on the outer water levels and the cross-section of the dike. However, for most cases data on the clay characteristics can be expected to be poor, while also the simulation of the inner water level may be inaccurate.

Sensitivity computations will have to be made to evaluate the consequences of assumptions made.

On the basis of the above described uncertainties in the model input and on the basis of the large variations in clay characteristics, the outcome of this study should be considered as a rough order of magnitude for the correction to be applied for clay dikes.

2.2 Definitions

In Chapter 3 the selected cases are described. It should be noted that in this report, similar as in Steezel and de Vroeg (1998), breach length and crest length are the dimensions perpendicular to the dike axis (see Figure 2.1), while breach width is the dimension of the breach parallel to the dike axis.
3 Description of cases

3.1 Breach test at Tollesbury 1995 (United Kingdom)

The situation

On August 4, 1995 a breach test with a clay dike was carried out at Tollesbury, located in the Blackwater estuary along the Tollesbury Fleet (United Kingdom). The location of Tollesbury is presented in Figure 3.1.

First two tests were carried out to study erosion on the inner slope of the clay dike. Some (limited) erosion of the toe of the inner slope was found.

Then a breach test was carried out, in which the development of the width of the breach was observed. In the following attention will be focused on the breach test.

All data on this case have been derived from the report of HR Wallingford (1996).

The dike

The dike consists of heavy clay (clay characteristics to be reported in a later stage).

For the breach test the cross-section of the dike was adjusted as presented in Figure 3.2. The crest length was approximately 1 m, and the crest level was ODN +2.21 m. The outer slope was practically vertical. The inner slope was 1:2.

The development of the breach

The breach was initiated at 6:03 BST. The initial breach was approximately 0.5 m wide and 0.3 m deep (so breach level approximately ODN +1.9 m).

The following observations are reported by HR Wallingford (1996):

‘The breach deepened rapidly in the first minute, cutting away the back face down to the approximately the field level (ODN +0.8 m). During this time the breach widenend to about 1.3 m’.

‘Two to three minutes after the breach initiation more rapid widening was apparent, with several large blocks conspicuously eroded from the sides of the breach. These were generally detached by a process of undercutting, although one large section of the backface was lifted and toppled landward, perhaps helped by internal flows and pressures’.
Within five or six minutes of initiation of the breach a large proportion of the breach widening had occurred. The width now was approximately 3 m.

After this the breach width remained approximately 3 m. The breach flow gradually became less energetic as the tide level fell and the level in the inundated area increased.

The breach development is illustrated in Figure 3.3.

**The conditions**

*Outer water level and inner water level*
Water levels in the outer and the inner area were measured. The outer and inner water level during the test are presented in Figure 3.4.
It should be noted that water levels in the inner area began to fall well before flow into the site reversed. This may have been due to the inundation characteristics of the site, and may also be due to infiltration into the dry fissured ground in the site.

*Inundated area*
Approximately 210,000 m². Near the embankment the level is approximately ODN +1 m.

**Assumptions**
It is assumed that the effect of wave action on the breach width development was negligible.

**Schematisation of the problem**
This case can be modelled straightforward since the observations of a fast breach deepening in the first phase and consequent breach widening are quite similar to those modelled in the models BREACH and BRES for sandy dikes. The runs with the model are made with standard settings for a sandy dike.

Special attention is paid to:

- the rate of breach widening;
- the final breach width.

**Rate of breach widening**
Since in the test the breach development stopped after approximately 6 minutes, the difference in the rate of breach widening for a clay dike and a sandy dike is evaluated over the first 6 minutes of the test. Some calibration of the breach deepening may be required in this stage, but considering the considerable breach deepening rate this calibration is expected to be minor.

**Final breach width**
For the tested clay dike the final breach width of 3 m was reached after approximately 6 minutes. It is interesting to evaluate whether for a sandy dike the breach development continued after this time and what the final breach width for a sandy dike would be (given the outer water levels presented in Figure 3.4).
3.2 Breach growth in foreland near Herkingen 1953  
(The Netherlands)

The situation

Herkingen is located at the island Goeree-Overflakkee in the south-western part of the Netherlands, see Figure 3.5. During the storm surge of February 1953 the dike near Herkingen breached. The polders Klinkerland and Battendoord located behind the dike flooded in the night of 1 February, due to several breaches in the dike.

After the storm the water was unable to flow back to the sea during Low Water due to the relatively high foreland (approximately NAP +1.7 m to NAP +2 m) in front of the dike. This foreland consisted of clay and had remained intact during the breaching of the dike. On 3 February it was decided to make a small ditch, 2.5 m wide and 0.6 m deep in the foreland, to allow the water to flow back to the sea, see Figure 3.6. On February 4 the depth of this ditch was artificially increased somewhat to release more water from the polder. After this the initial small breach started to grow. On 7 February the ditch had developed to a 40 m wide breach with a level less than a meter below Low Water.

(Note: On February 15 the depth of the breach reached the peat layer underneath the clay (NAP -2.4 m) and the depth growth increased rapidly from that moment on (a final depth below NAP -10 m was reached while the width remained approximately 40 m).

Since the development of the (manmade) breach in the foreland during the days after the storm is better documented than the development of the breaches in the dike during the storm, attention is focussed on the foreland rather than the dike itself.

The development of the small initial breach in the foreland in the period February 4 to 7 is considered a good case representative for the breach growth in a body consisting completely of clay.

The foreland (‘dike’)

The breached foreland consists of a 4 m thick clay layer. The upper level of the foreshore is approximately NAP +1.7 m, the lower level approximately NAP -2.3 m. It was present along a significant section of the dike in the considered area and it had been hardly damaged during the storm. Also near the breach in the dike near Herkingen it had remained intact, despite the fact that during the storm flow velocities near the breach in the dike must have been significant. Only after a manmade small breach had been made in the foreland itself, the erosion process started.
The development of the breach

The storm surge report seems to indicate that on 3 February, with a 2.5 m wide ditch of depth 0.6 m (so level approximately NAP +1.1 m) no significant natural growth of the breach occurred. On February 4 the ditch was deepened somewhat with the objective to release more water from the polder, but it is not reported until what depth. We have assumed that the depth of the ditch was doubled to 1.2 m (level of approximately NAP +0.5 m). We have further assumed that also the width has slightly increased to 4 m. We assume that on 4 February, with this new ditch the breach growth started.

The breach width grew in approximately 3 days (February 4 to February 7) from approximately 4 m to 40 m.

In the same period the breach depth increased from approximately NAP +0.5 m to NAP - 2 m.

The breach development is illustrated in Figure 3.7.

The conditions

Outer water level
The initial breach in the foreland was made after the storm surge, so the outer water levels were determined mainly by the astronomical tide in the Grevelingen (MHW = +1.46 m, MLW = -1.54 m). An estimate of the astronomical tide in the period 4-7 February 1953 has been made on the basis of data for Brouwershaven as derived from the tide tables of 1953 with a correction for the slightly larger tidal range near Herkingen. The result is presented in Figure 3.8.

Inner water level
On February 1, 1953 at approximately 4:15 the first breach developed in the dike and the polders Klinkerland and Battendoorn started to flood. At about 5:30 the water level in polder Klinkerland reached its highest level of NAP +2.75 m. The foreland at a level of NAP +1.7 m prevented the water from flowing back to the sea (the Grevelingen). It is assumed that in the period 3 to 4 February the water level dropped gradually to a level equal to the breach depth (NAP +0.5 m).

No detailed information is available on the inner water level in the polder for the period February 4 - 7. However, the lower boundary for the inner water level is approximately the level of the depth of the breach (since the foreland was preventing the water from flowing to the sea). The upper limit of the inner water level can be assumed to be determined mainly by the discharge through the studied channel. In 'het polytechnisch tijdschrift' it is reported that “a favourable circumstance for the southern part of Goeree-Overflakkee was that the foreland with a level of NAP +2m had remained practically intact. As a result, the normal tides did not flood this foreland”. So the location of Herkingen can be assumed to be the only location where the foreland in front of a dike breach had been lowered. As a result it can be assumed that in the days after the storm the inner water level in the polder Klinkerland was mainly determined by the discharge through the considered breach.
Inundated area

Polder Klinkerland has a surface area of roughly 6 km by 1.5 km, so a total surface area of approximately 9 km². The level of the inundated area varies between NAP -0.1 m and NAP +0.5 m and is assumed at NAP for the basic computations. For the simulation it should be kept in mind that as long as the breach level is higher than NAP, the inner water level at the start of the simulation is determined by the breach level.

Assumptions

It is assumed that the breach development in the foreland has not been influenced significantly by the presence of part of the dike landward of the foreland. When the initial breach in the foreland was made on February 3, the breaches in the dike, which had occurred on February 1, had already developed to a considerable size. Especially in the first stage of breach development in the foreland the breaches in the dike were large compared to the breach dimensions in the foreland. It seems fair to assume that the flow through the breach was not significantly affected by the dike in that initial stage.

It is assumed that the effect of wave action on the breach development has been negligible. Furthermore, it is assumed that the outer and inner slopes of the foreland were 1:1. The results can be expected to be insensitive for this assumption, since it does not significantly affect the total surface area of the dike cross-section.

Schematisation of the problem

The breach width has grown from 4 to 40 m in approximately 3 days, so in a time span of approximately 6 High Water - Low Water cycles. In this time not only the width of the breach has changed but also the depth of the breach varied gradually. There are several ways to schematise the problem with BREACH. The following approach is proposed (to simulate development of width with controlled depth development):

- Simulate the 6 High Waters.
- For each High Water a fixed breach depth has been assumed on the basis of the observed depth (depth growth not simulated with the model).
- For each High Water the development of the width has been computed with BREACH, only on the basis of the phases during which water flows from the Grevelingen into the polder. (this implies that it is assumed that these phases are dominant, this can be checked with the model afterward by simulating the (additional) effect of water flowing from the polder to the sea).
- The above gives 6 periods of several hours for the breach width to develop. This width (valid for a sandy body) will be compared with the observed width of 40 m, to obtain an estimate of the factor to be applied on the breach width development rate.

Some checks and sensitivity runs will have to be made with the model:

- Sensitivity of results for deviations of the inner water level (by varying the inner surface area) and level of inner area (to NAP +0.4 m).
• The importance of the effect of outflowing water, by simulating outflow for computed inner water level

Note:
In the above attention was focussed on the development of the foreland. However, also the breaches in the dike have grown. The crest level of the dike varied between NAP +4.05 m and NAP +5.10 m. No detailed information on the composition of the dike has been found but in Polytechnisch tijdschrift it is reported that the dikes in this area consist of sandy clay. The outer slope of the dikes was 1:3 to 1:4, and the inner slope 1:1.5. The two breaches in the dike that had occurred on February 1 appear to have been rather stable at some time after the initial breaching, due to the blocking of the flow by the approximately NAP +1.7 m foreland in front of the dike. Only after the breach in the foreland had depended significantly, the breaches in the dike continued to grow, they joined and reached a total width of 300 m.

3.3 Dike breach at Oudenoord, 1953 (the Netherlands)

The situation

Polder Oudenoord is located at Voorne-Putten, somewhat west of Hellevoetsluis, see Figure 3.9. During the storm surge of 1953 a 70 m wide breach in the Oudenoordse sea dike developed.

Information related to this case is presented in the storm surge report and het polytechnisch tijdschrift.

The dike

From recent borings carried out by Grondmechanica Delft it can be concluded that the dike core in 1953 consisted of clay (personal communication with Mr. Rosing of Grondmechanica Delft).

A cross-section of the dike cross-section in 1953 is presented in Figure 3.10. The crest length was approximately 3.6 m, and the crest level NAP +5.45 m. The outer slope was approximately 1:2.7 and the inner slope 1:1.5.

The development of the breach

The breach was initiated at 1 February at 4:50. The initial breach width is not known, but during the storm the breach developed to a width of 70 m. Since the storm surge level was approximately NAP +4.1, so well below the crest of the dike (NAP +5.45) it is assumed that this breach did not occur due to massive overflow of the dike over a width of 70 m, but that first a small initial breach developed. This width of this initial breach is estimated at 10 m and the level of the breach at NAP +3m.

It is estimated that the final breach width was reached at 1 February around 10:30. At this time the outer water level was approximately NAP +1.5 m, which is close to the registered maximum inner water level of NAP +1.27 m. During the second high water the difference between the outer and the inner water level remained relatively small (1 to 1.5 m), and the
breach growth of the clay dike during this second flood is therefore expected to be small compared to the growth during the first high water.

(No foreland was present in front of this breach and after the second flood at 1 February a scour hole with a depth of NAP -5.5 m had developed. Further increase of the depth was prevented by a clay layer at approximately NAP -5.5 m).

The conditions

Outer water level and inner water level
The outer water level in the Haringvliet is presented in Figure 3.12.

In the Polder Oudenoorn, close to the studied breach (at the location of the 'gemaal'), a water level of NAP +1.27 m has been measured.

Inundated area
It is reported that the dikes that separated the polder from Polder Zuidland and Polder Abbenbroek remained intact during the storm. Flooding of the Polder Nieuwenhoorn via Polder Oudenoorn occurred, but only during the second flood. It can therefore be concluded that during the storm itself the area that was inundated through the breach was restricted to the Polder Oudenoorn itself. The polder has a surface area of approximately 9,000,000 m². The level of this area varies between NAP -0.3 m and NAP -1.0 m. For the basic computations NAP -0.4 m will be assumed.

Two other smaller breaches (30 m and 35 m) have developed in the dike, which have contributed to the inundation of the area. The 30 m breach was initiated at approximately the same time as the studied breach (at 4:45). It is not reported when the other breach started but it is reasonable to assume that this was approximately at the same time (during the highest water levels). (Note: due to the presence of a foreland seaward of these two smaller breaches the depth of the breach was limited to approximately NAP). Since the total final width of these two breaches is similar to the width of the investigated breach it can be roughly estimated that the two smaller breaches contributed to the flooding of the polder for approximately 50%.

Assumptions

The initial breach width has been assumed at 10 m and the initial breach level at NAP +3 m.

Schematisation of the problem

This case is relatively well documented, with the exception of the dimensions of the initial breach. As described above, an assumption has been made for these dimensions.

In order to account for the inundation of the polder through the other two smaller breaches, the inner area in the model is reduced to 50% of the actual polder area. This results in an inner area of approximately 4,500,000 m².

On the basis of the other available data straightforward modelling can be applied.
It should be noted that due to the lack of information on the initial breach dimensions no accurate breach width development rate of the clay dike can be determined. Still, on the basis of a comparison between the final width of the breach in the clay dike and the computed final width (for sand) conclusions can be drawn.

Some sensitivity runs will have to be made with the model:

- Sensitivity of the results for deviations of the inner water level (variation of level inner area, say NAP -0.8 m).
- Sensitivity run with initial breach width of 50 m instead of 10 m.

### 3.4 Breach test at Yahekou (China)

#### The situation

Several large scale tests on fuse plug dams have been carried out in China. Ye (1998) describes a test at Yahekou (see Figure 3.13) which was carried out for a dam consisting of clay and sand.

The dike consists of a clay core with a thick layer of sand around it. Many dikes consist of an clay core (old dike) covered by sand (dike improvements). If a cohesive core in a dike consisting of loose materials has a noticeable effect on the breach development, it should be clearly demonstrated by this test.

For this test case the breach development is well documented. However, the test only provided information on the breach widening rate. No information on the effect of the critical flow velocity can be derived, due to the fact that the test section had a limited width of approximately 30 m. After the breach had reached a width of 30 m, further breach widening was prevented by fixed walls. The inner water level remained low during the entire test. Due to the above artificial stop of the test and the low inner water level, it can be assumed that during the entire test period the velocity in the breach was well above the critical velocity.

All information related to environmental conditions and development of the breach presented below has been derived from Ye (1998). Ye reported data presented by Pan et al (1993) and from personal communications with Pan.

In the following description all vertical levels (of the dam and the water levels) are expressed relative to the fixed chute bottom.

#### The dam

A cross-section of the dam is presented in Figure 3.14. The dam consisted of a clay core with considerable sand bodies on both sides of the core.
No data on the clay characteristics are available. It is described as heavy silty loam and as clay. The sand is described as medium to gravelly coarse sands.

The height of the dam was 5.6 m (relative to a fixed chute bottom). The crest length was 4 m. The upstream slope was 1:3 and the downstream slope was 1:2.5.

The height of the clay core was 5.1 m. The crest length of the core was 0.8 m and the slopes on both sides were 1:0.17 (width at the base of the core 2.5 m).

**The development of the breach**

The breach development is illustrated in Figure 3.15.

The graphs in the upper part of the profile have been derived for representative sections at the core, so on the basis of the development in section b, (section in the centre of the core, see Figure 3.15). Both the breach deepening and the breach widening are well documented.

It should be noted that the depth growth of the clay core started only after 5 minutes. During the first 5 minutes of the test the sandy section downdrift of the clay core was partly eroded.

The test section had a limited width of approximately 30 m, so after approximately 30 m of breach widening further growth was artificially stopped. Furthermore, the initial breach was not located in the centre of the test section. During the phase of breach deepening there was sufficient space for the breach to grow (slightly) in both directions, but after the phase of breach deepening, breach widening could only occur in one direction. Therefore this case should be simulated as a one-sided breach widening case.

**The conditions**

The outer and inner water levels are presented in Figure 3.16.

The outer water level was more or less constant at the level of the crest of the clay core (+5.1 m above the chute bottom). The inner level (downstream level) remains more than 3 m below the outer water level during the entire test, so it can be expected that tailwater effects during these tests are negligible.

**Schematisation of the problem and assumptions**

Two computations have been made:

- First a computation was made with the entire dam cross-section as presented in Figure 3.14. This computation should give a first rough indication of the relative influence of the clay core on the breach width development. If a clay core in a dike has an effect on the breach development, it should be clearly demonstrated by this test. Due to the clay core significantly slower breach development can be expected during the test than predicted by the model.

- Then a computation was carried out with only the cross-section of the (clay) core. It has been assumed that the clay core is the limiting factor in the breach growth. For this
simulation it has been assumed that the clay core was the limiting factor for breach widening.

As illustrated in Figure 3.15, for this case the breach widening was only in one direction. In breach this can be schematised with the option ‘one-sided breach development’.

3.5 Possible other cases

During the study some potentially interesting cases have not been selected. The reason for this was that only a limited number of cases could be analysed in the duration of the study (for analysis see Futloo, 1999) or that not sufficient data could be obtained during the study for model simulations.

1. Mississippi floodings (Great Flood of ’93)

Eye-witness reports of the Kaskaskia island levee breach and Bois Brule levee breach are attached in Annex A. The estimated breach widening rates seem very large (15 m / minute). No additional data could be obtained during the study.

2. Re-evaluation of the Wieringermeer case

In the breach validation report of September 1998 the Wieringermeer dike has been simulated as a sandy dike, while part of the core of the dike consists of clay. This case is being re-evaluated.
(\text{Note: For the other prototype cases described in the above report there were indications in literature that the core of the dike consisted of sand. Re-evaluation in the scope of this project is therefore not carried out}).

3. Additional breach case from storm surge report 1953

Similar to the Oudenhaarm case an additional case can be derived from the 1953 storm surge report.
4 Conclusions and recommendations

Data on breach development in cohesive dikes has been collected with the objective to use these data for further investigation on the effect of clay on the breach growth. This further investigation (reported by Futloo, 1999) is considered to be a rough approach and a first step in the assessment of the effect of clay.

From the search of suitable data on breach development in cohesive dikes it is concluded that no 'ideal' cases can be found. For all considered cases some assumptions had to be made.

Apart from some uncertainty in the breach development itself and the environmental conditions, the use of breach cases in nature have the drawback that the characteristics of the clay are not well known. This implies that it can not be determined which part of the range has been covered. In addition, in some cases there may also have been some other effects (like that of the slope protection) on the breach widening.

Still, for a first indication of the effect of clay and the most relevant parameters to simulate these effects with the available models, the data are considered useful. For fine-tuning of the above approach more systematic tests (under controlled conditions) would be required.

The rough investigation on the basis of these selected cases can be compared with another rough approach, on the basis of characteristic clay features presented in literature. A summary of the most relevant indicative data has been made by H.J. Verhey. This summary is attached in Appendix B.
References

Futlo, Z, 1999
Breach growth in clay dikes: validation of models
M.Sc. Thesis IHE, Delft

Getijtafels voor Nederland, 1953
Bijgewerkt bij de directie algemene dienst van Rijkswaterstaat

HR Wallingford, 1996
Tollesbury managed set back experiment
Breach design and construction, and embankment failure experiment
(Report TR 5, March 1996)

Pan Shuibo et al, 1993
Chinese-Finnish cooperative research work on dam break hydrodynamics, Helsinki

Polytechnisch tijdschrift, het watersnoodnummer
In Dutch

Rijkswaterstaat en Koninklijk Nederlands Meteorologisch Instituut, 1961
Verslag over de stormvloed van 1953, “Storm surge report”
In Dutch

Steetzel, H.J., de Vroeg, J.H., 1998
Extension and validation of the BREACH model
Update to version 1.0

Ye, Shiang, 1998
Comparison and validation of models of breach growth in sandy dikes
M.Sc. Thesis IHE, Delft
<table>
<thead>
<tr>
<th>Conditions</th>
<th>Tollesbury</th>
<th>Herkingen</th>
<th>Oudendoorn</th>
<th>Yangkou</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water level</td>
<td>(m, ref)</td>
<td>Figure 3.4</td>
<td>Figure 3.8</td>
<td>Figure 3.12</td>
</tr>
<tr>
<td>Wave height $H_s$</td>
<td>(m)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Crest height</td>
<td>(m, ref)</td>
<td>+2.21</td>
<td>+1.7</td>
<td>+5.45</td>
</tr>
<tr>
<td>Crest 'length'</td>
<td>(m)</td>
<td>1</td>
<td>60</td>
<td>3.6</td>
</tr>
<tr>
<td>Outer slope</td>
<td></td>
<td>vertical</td>
<td>1:1</td>
<td>1:2.7</td>
</tr>
<tr>
<td>Inner slope</td>
<td></td>
<td>1:2</td>
<td>1:1</td>
<td>1:1.5</td>
</tr>
<tr>
<td>Init. Breach level</td>
<td>(m, ref)</td>
<td>+1.9</td>
<td>+0.5</td>
<td>+3</td>
</tr>
<tr>
<td>Init. Breach width</td>
<td>(m)</td>
<td>0.5</td>
<td>4</td>
<td>10</td>
</tr>
<tr>
<td>Init. Breach 'length', (m)</td>
<td>1.6</td>
<td>62.4</td>
<td>210,000</td>
<td>9,000,000</td>
</tr>
<tr>
<td>Surface of inner area</td>
<td>(m$^2$)</td>
<td>210,000</td>
<td>9,000,000</td>
<td>4,500,000</td>
</tr>
<tr>
<td>Level of inner area</td>
<td>(m, ref)</td>
<td>+1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Additional discharge</td>
<td>(m$^3$/s)</td>
<td>0$^1$</td>
<td>0</td>
<td>0$^2$</td>
</tr>
<tr>
<td>Clay characteristics</td>
<td></td>
<td>heavy clay</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Breach development

| Breach level | (m, ref) | Figure 3.3 | Figure 3.7 | - | Figure 3.15 |
| Breach width | (m)      | Figure 3.3 | Figure 3.7 | Figure 3.11 | Figure 3.15 |
| Flow velocity | (m/s)    | -          | -          | -          | -          |
| Discharge | (m$^3$/s) | -          | -          | -          | Figure 3.16 |

### Notes

| Vertical reference level | ODN | NAP | NAP | chute bottom |

1 There probably was some infiltration into the ground, but this can be ignored since it has not significantly affected the water levels during the short duration of the test.
2 Effect of other breaches has been taken into account by reducing surface of inner area
3 Characteristics of clay core have been presented

Table 3.1 Summary of cases
Figures
Water level observations - 04/06/95

Measured inner and outer water level Tollesbury
Illustration of situation near Herkingen at 3 and 7 February 1953

H3468

WL | DELFT HYDRAULICS

FIGURE 3.6
Breach development foreland Herkingen

H3468

FIGURE 3.7
Outer water levels Herkingen

H3468

WL | DELFT HYDRAULICS

FIGURE 3.8
Location of Polder Oudenhoorn

H3468

WL | DELFT HYDRAULICS

FIGURE 3.9
Cross-section of dike Oudenoord

FIGURE 3.10
outer water level (tide and surge)

1st high water
2nd high water

10:30 end of simulation
4:50 initiation of breach
Cross-section of dam Yahekou

(Pan et al., 1993)
Note: one-sided breach widening

The Scouring time-contour (Pan et al., 1993)
Measured inner and outer water level Yahekou

H3468

WL | DELFT HYDRAULICS

FIGURE 3.16
Mr. de Vroeg, Mr. Alvey is one of our Senior Levee Engineers. He has witnessed many levee overtopping events, especially during the Great Flood of '93. Please read the following message from Mr. Alvey. I have not jet started collecting the other information you requested. I will be out of the office next week and will try to start compiling information when I return. Please feel free to contact me if you have any additional questions. Sincerely, Claude Strauser

> -----Original Message-----
> From: Alvey, Mark S MVS
> Sent: Friday, January 22, 1999 11:36 AM
> To: Strauser, Claude N MVS
> Cc: Postol, George J MVS; Hahn, Emmett W JR MVS; Scanlon, Jake C MVS
> Subject: RE: data on breaching of levees (2)
>
> Claude,
>
> 1. The Kaskaskia Island levee breach (Mississippi River mile 113)
> occurred on July 22, 1993 due to uncontrolled underseepage problems. The
> flood level (47.38 feet on the Chester, IL gage) exceeded the 50-year
> flood frequency design (45.7 feet on the Chester, IL gage). The breach
> was 620 feet long and most of the levee material scoured away within the
> first 15 minutes. The levee consisted of semi-compacted lean clay with
> silt lenses. The resulting scour hole was measured to be approximately 50
> feet deep with a total volume of 1 million cubic yards of degradation.
>
> 2. The Bois Brule levee breach (Mississippi River mile 97) is reported to
> have occurred just before midnight on July 24, 1993 and continued into the
> early morning on July 25, 1993. The flood level (47.20 feet on the
> Chester, IL gage) exceeded the 50-year flood frequency design (45.7 feet
> on the Chester, IL gage). The breach was 1,480 feet long and most of the
> levee material scoured away within the first hour. The levee consisted of
> semi-compacted lean clay with silt and sand lenses. The resulting scour
> hole was measured to be approximately 50 feet deep with a total volume of
> 1.5 million cubic yards of degradation.
>
> 3. From my experiences during the 1993 Flood, most of the material that
> is scoured away will occur within the first 15 to 30 minutes once a levee
> breach occurs. I would guess at a rate of 50 linear feet of levee per
> minute. The scour hole depths are approximately two times the
> differential head between the flood level and either ground surface or
> interior water level. Additional scour occurs when the river currents are
> redirected into the breach like those at two breaches for Ste. Genevieve
> County NO. 2.
Annex B
Erosiesnelheid klei/zand
op basis van in literatuur vermelde gegevens
(door ir. H.J. Verhey, WL/Delft Hydraulics)

Bij rapportage “Breach growth in cohesive materials, Selection of cases”

De vraag in hoeverre de bresgroei bij een kleidijk (dat is een dijk met een kleikern) langzamer verloopt dan bij een zanddijk (een dijk met een zandkern) kan bij een quick-and-dirty aanpak op twee manieren worden onderzocht:
- vergelijkingen van cases met kleidijken met die van zanddijken en op basis daarvan een praktijkfactor bepalen: \( (k/\tau_{c,zand})_{\text{cases}} \)
- bepalen van een empirische factor \( (k/\tau_{c,zand})_{\text{empirie}} \) op basis van in de literatuur vermelde gegevens

Vergelijken van beide resultaten kan resulteren in een verhoudingsfactor voor de bresgroei bij een kleidijk ten opzichte van een zanddijk.

In deze annex wordt het tweede spoor gevolgd: het bepalen van een empirische factor \( (k/\tau_{c,zand})_{\text{empirie}} \) op basis van in de literatuur vermelde gegevens. Het resultaat kan worden vergeleken met de factor volgens het eerste spoor.

In de literatuur worden diverse ervaringscijfers vermeld op basis waarvan een parameter \( (k/\tau_{c,zand})_{\text{empirie}} \) kan worden geschat.

stroming

In een CUR-rapport (CUR, 1996) worden waarden voor stroming gepresenteerd die zijn samengevat in Tabel 1.

<table>
<thead>
<tr>
<th>Grondsoort</th>
<th>kritieke stroomsnellenheid ( u_e ) (m/s)</th>
<th>relativie kritieke stroomsnellenheid ( u_e/u_{c,zand} ) (-)</th>
<th>kritieke schuijspansing ( \tau_e ) (Pa)</th>
<th>relativie kritieke schuijspansing ( \tau_e/\tau_{c,zand} ) (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>klei - compact stevig</td>
<td>1,05 - 1,25</td>
<td>4,0 à 4,5</td>
<td>3 - 5</td>
<td>20 à 25</td>
</tr>
<tr>
<td>klei - compact slap</td>
<td>0,70 - 0,90</td>
<td>2,5 à 3,0</td>
<td>1,5 - 3</td>
<td>10 à 15</td>
</tr>
<tr>
<td>zand</td>
<td>0,30 - 0,45</td>
<td>1,2 à 1,5</td>
<td>0,5 - 1,5</td>
<td>3,5 à 7,5</td>
</tr>
</tbody>
</table>

Tabel 1 Kritieke waarden en relativie kritieke waarden voor stroomsnellenheid en schuijspansing bij stroombelasting

De waarden voor \( u_e \) en \( \tau_e \) bij een bepaald type klei behoren respectievelijk bij een schrale klei met verhouding zand/lutum van 60%/40% en een vette klei met verhouding met verhouding zand/lutum van 20%/80%. In hoeverre een klei slap, stevig of compact is, kan worden afgeleid uit bijvoorbeeld de waarde voor de ongedraineerde schuiysterkte. Dit is hier evenwel niet relevant.

De verhoudingen \( u_e/u_{c,zand} \) en \( \tau_e/\tau_{c,zand} \) zijn onderling gerelateerd volgens een kwadratisch verband.

De reststerkte, of anders gezegd: de tijdsduur \( t \) voordat een lengte \( L \) is geërodeerd als gevolg van een stroomsnellenheid \( u \), kan worden weergegeven met de formule:
\[ t : c L u^{-2} \]

waarin \( c \) = factor voor de sterkte van de grond.
Het gegeven verband is gebaseerd op een relatie tussen de erosiesnelheid door stroming en de schuifspanning volgens Osman en Thorne (1988), en zoals bekend bestaat er een kwadratisch verband tussen schuifspanning en stroomsnelheid.
Daar staat tegenover dat door het CIRIA (Hewlett et al, 1987) ontwerpgrafieken voor gras zijn gepresenteerd waaruit het volgende verband valt af te leiden voor de toelaatbare belastingduur:

\[ t : c L u^{-\alpha} \quad \text{met} \quad \alpha = 3 - 5 \]

Een dergelijke waarde voor de exponent van de stroomsnelheid sluit aan bij die welke voorkomt in onttongdings- en sedimenttransportformules. De CIRIA-grafieken laten echter ook andere waarden voor de exponent toe.

golven

Voor golven zijn in de literatuur ook waarden gepresenteerd. Allereerst zijn in een TAW-A2 notitie (Klein Breteler, 1992) waarden voor de reststerkte van klei gepresenteerd volgens de relatie:

\[ t : c L H^{-2} \]

(t in seconden per eenheid van lengte)

De afhankelijkheid met \( H^2 \) sluit aan bij de kwadratische relatie tussen golfenergie en golfhoogte, waarbij bedacht dient te worden dat erosie afhankelijk is van de golfenergie.

De resultaten zijn naderhand gebruikt in studies in het kader van de Marsroute van TAW-E. In Tabel 2 zijn deze in aangepaste vorm samengevat (Klein Breteler, 1994).

<table>
<thead>
<tr>
<th>Klei soort</th>
<th>coëfficiënt c (m.s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>zeer goede klei</td>
<td>21600</td>
</tr>
<tr>
<td>goede klei met weinig structuur</td>
<td>13600</td>
</tr>
<tr>
<td>sterk gestructureerde goede klei</td>
<td>6400</td>
</tr>
<tr>
<td>matige of slechte klei</td>
<td>2800</td>
</tr>
</tbody>
</table>

Tabel 2  Sterkte klei tegen golfafslag volgens Klein Breteler (1994)

Verder zijn bij een Deens onderzoek (Lastrup, 1990) ook waarden gepresenteerd voor de sterkte van zand/klei mengsels onder invloed van golfbelasting. In iets bewerkte vorm staan deze in Tabel 3.

<table>
<thead>
<tr>
<th>Type grond</th>
<th>coëfficiënt c</th>
<th>percentage deeltjes &lt; 0,063 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>klei met 60% zand</td>
<td>16950</td>
<td>40</td>
</tr>
<tr>
<td>zand met 10% silt</td>
<td>1125</td>
<td>17</td>
</tr>
<tr>
<td>zand met 17% silt</td>
<td>740</td>
<td>10</td>
</tr>
</tbody>
</table>

Tabel 3  Sterkte klei tegen golfafslag volgens Lastrup (1990)

<table>
<thead>
<tr>
<th>Type grond</th>
<th>sterkte</th>
<th>relativieke sterkte</th>
<th>kenmerken</th>
</tr>
</thead>
<tbody>
<tr>
<td>klei: zeer goed</td>
<td>21600</td>
<td>&gt; 30</td>
<td>-</td>
</tr>
<tr>
<td>goed</td>
<td>13600 à 16950</td>
<td>15 à 25</td>
<td>weinig structuur, ca 60% zand sterk gestructureerd</td>
</tr>
<tr>
<td>matig</td>
<td>6400</td>
<td>5 à 10</td>
<td>-</td>
</tr>
<tr>
<td>slecht</td>
<td>2800</td>
<td>2 à 5</td>
<td>-</td>
</tr>
<tr>
<td>zand</td>
<td>740 à 1125</td>
<td>1</td>
<td>10% à 20% silt</td>
</tr>
</tbody>
</table>

Tabel 4  Samenvattende tabel voor sterkte klei als functie van de kleikwaliteit voor golfbelasting

empirische factor (klei/zand)$_{empirie}$

De resultaten voor stroming en klei (respectievelijk in de Tabellen 1 en 4) kunnen worden gecombineerd. Hierbij wordt verondersteld dat tussen belastingduur en stroming ook een omgekeerd kwadratisch verband bestaat (en niet een verband tot de macht 4). Dit lijkt aannemelijk om diverse redenen. Allereerst is er de empirische relatie zoals gepresenteerd door Osman en Thorne. Daarnaast is bekend dat tussen orbitaal snelheid en golfhoogte een lineair verband bestaat en uitgaande van de kwadratische relatie tussen golfhoogte en tijdsduur, betekent dit ook een kwadratisch verband tussen stroomsnelheid en tijdsduur. Het voorgaande houdt in dat voor de vergelijking van stroming met golfen kunnen worden aangehouden de gekwadrateerde waarden van de in Tabel 1 vermelde waarden voor de relativieke kritieke stroomsnelheden.

Het resultaat wat betreft de relativieke sterkte van klei ten opzichte van zand staat in Tabel 5 als functie van de kleikwaliteit. Ook is vermeld de kritieke stroomsnelheid.

<table>
<thead>
<tr>
<th>Type grond</th>
<th>relativieke sterkte</th>
<th>kritieke stroomsnelheid (m/s)</th>
<th>kenmerken</th>
</tr>
</thead>
<tbody>
<tr>
<td>klei: zeer goed</td>
<td>&gt; 30</td>
<td>&gt; 1,25</td>
<td>-</td>
</tr>
<tr>
<td>goed</td>
<td>15 à 25</td>
<td>1,0 à 1,25</td>
<td>-</td>
</tr>
<tr>
<td>matig</td>
<td>5 à 10</td>
<td>0,5 à 0,75</td>
<td>-</td>
</tr>
<tr>
<td>slecht</td>
<td>2 à 5</td>
<td>0,3 à 0,5</td>
<td>-</td>
</tr>
<tr>
<td>zand</td>
<td>1</td>
<td>0,25</td>
<td>-</td>
</tr>
</tbody>
</table>

Tabel 5  Sterkte klei als functie van de kleikwaliteit

Gegeven een bepaalde dijk waarvan de kleikwaliteit bekend is of kan worden geschat, is het nu mogelijk de snelheid van de bresgroei te schatten ten opzichte van diezelfde dijk veronderstelling dat die een zandkern zou hebben.

Stel de dijk bestaat uit matige klei, dan zou de bresgroei volgens bovenstaande redenering een factor 5 à 10 langzamer verlopen dan wanneer de kern uit zand zou bestaan.

Overigens kunnen in een kleidijk zandlenzen voorkomen, die het erosieproces versnellen. Daar staat tegenover dat ook humeuze delen aanwezig kunnen zijn en die vertragen het erosieproces.
aanbevelingen:

Ontstaan initiële geul

Alle bresgrooimodellen starten op een tijdstip $t = t_0$ en veronderstellen de aanwezigheid van een initiële geul. Een dergelijke geul moet echter wel de tijd krijgen te ontstaan. Hierbij zijn twee mechanismen denkbaar:
1. terugschrijdende geulontwikkeling na het ontstaan van erosie aan het binnentalud door overstromend/overslaand water
2. voortschrijdende erosie door oeverafslag van het buitenatalud door golven

In beide gevallen zal zich een geul moeten ontwikkelen die ontstaat door erosie van de bovenste lagen van kruin en taluds. In veel gevallen bestaat deze uit een graszode op klei, een breuksteen of blokkenverdediging (buitentalud) of een wegverharding (op de kruin). Duidelijk zal zijn dat het enige tijd duurt voor een geul tot stand is gekomen. Duidelijk zal ook zijn dat deze tijdsduur direct van invloed is op de hoeveelheid water die een polder binnenstroomt tijdens de totale stormduur als eenmaal een bres is ontstaan.
(Opgemerkt wordt dat door menselijk handelen kunstmatig een initiële geul kan worden gecreeërd. Dit mechanisme is buiten beschouwing gelaten.)

Gezien het bovenstaande wordt aanbevolen het ontstaan van een bres, dus de tijd voorafgaand aan $t_0$, te modelleren en dit als submodel voorafgaand aan een bresgrooimodel door te rekenen.
De benodigde kennis hiervoor is in principe aanwezig en wordt voor het tweede mechanisme in het navolgende kwalitatief beschreven.

geulvorming door externe erosie


De grasdijkenstudies resulteerden in de formule:

$$t = \frac{c.L}{H^2}$$

Hierbij is verondersteld dat er een lineair verband is tussen oeverafslag $L$ en tijdsduur $t$. In werkelijkheid is er een afnemend verband volgens:

$$t = \left(\frac{c.L}{H^2}\right)^{1/b} \quad met \quad 0 \leq b \leq 1$$

In de literatuur worden voor $b$ waarden vermeld variërend van 0,2 tot 0,8.

De waarde van $c$ neemt gerekend vanaf het buitenatalud af: eerst is er de graszode met een hoge weerstand, vervolgens de wortellaag met een lagere weerstand, dan een laag afdekklei met een opnieuw lagere weerstand en tenslotte de dijkkern bestaande uit kornklei of zand. De waarde van $c$ vertoont dus een verband met de mate van afslag $L$ volgens:
\[ c : \exp(-L) \]

Door bermvorming en golfbreking zal de golfhoogte \( H \) afnemen bij toenemende afslag \( L \). Ook voor de golfhoogte is dan een verband met \( L \) volgens een exponentiële functie te geven.

Op grond van het voorgaande is een conceptueel model ontwikkeld waarmee de tijdsduur \( t \) kan worden berekend alvorens een bres in een dijk ontstaat. Vervolgens kan met bresgroeimodellen de verdere ontwikkeling van de bres worden bepaald, alsmede de hoeveelheid water die een polder inloopt. Als de tijdsduur voor het ontstaan van een initiële bres echter dusdanig lang is dat het hoogwater al weer voorbij is, zullen bresgroei berekeningen overbodig zijn.

---

literatuur


