Technical Report on Sand Boils (Piping)

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Technical Advisory Committee on Flood Defences

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Contact address:

Rijkswaterstaat
Road and Hydraulic Engineering Institute
P.O. Box 5044
2600 GA Delft
The Netherlands

Tel. +31-15-2518518
Fax +31-15-2518555
Email: dwwmail@dww.rws.minvenw.nl

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Note: this document is a draft English version of the Dutch document from 1999 and not yet properly screened concerning English translation

Content

1 Introduction

1.1 Context

Piping and hydraulic fracturing (heave) are phenomena which can threaten the stability of flood defences. These phenomena can arise when, in the case of large-scale hydraulic head, particles of soil in layers of earth which are susceptible to erosion are transported underneath the flood defence by the seepage flow, as a consequence of which erosion channels are created under the flood defence (piping), or mutual effective stress is lost in the soil (heave) in the upward seepage flow behind the hydraulic structure or cut-off wall.

Within the scope of designing new flood defences or reinforcing or monitoring existing ones, safeguards against the occurrence of these phenomena must be verified.

Various calculation rules are available to verify such safeguards, varying from simple empirical rules for the first (reinforcement) design for a flood defence or safety monitoring of an existing flood defence, to advanced calculation models to design or test more precisely.

The approach and calculation models for verifying safeguards against piping and heave for primary flood defences are recorded in various TAW guides, guidelines and other publications, including

- *Guide on Designing River Dikes* (part 1) [TAW 1987]
- *Technical Report on Piping at River Dikes* (TAW-B guideline) [TAW 1994]
- *Guide on Structural Designing* [TAW 1994[2]]
- *Guide on Safety Monitoring at Water Defences* [TAW 1996]
- *Guide on Water-retaining Hydraulic Structures and Special Structures* [TAW 1997]

The recommended approach and calculation rules in these publications are not state-of-the-art, bearing in mind the developments in this field in the past few years. New developments are recorded in research reports which are often insufficiently accessible for day-to-day design and test practice. All in all, the knowledge available for practical application is fragmented.

1.2 Aim and Scope of the Technical Report

The aim of this technical report is to describe the various aspects connected to the phenomenon of sand boils in one coherent publication. These aspects are

- occurrence of erosion and soil failure caused by seepage flow under or along water-retaining soil structures and hydraulic structures;
- the (classical and new) calculation models and calculation rules available for designing and monitoring water-retaining structures, the parameters required and the way in which calculation models are used;
- possible structural solutions in the design stage and technical management measures to guarantee safety against erosion and soil failure;
- any measures in threatening high water situations and areas for attention for management to ensure it is prepared for any emergencies.

The Technical Report is targeted at designers and managers of (primary) flood defences and their technical advisors. The guiding principal has been to ensure that this target group is capable of conducting design inspections or tests (within the scope of the periodical safety monitoring of primary flood defences) or to guide design inspections or tests granted to third parties with this publication.

This Technical Report is a supplement to the existing guides and other guidelines. In some cases this report will deviate somewhat from what is stated in the guides, because more recent insights and information has been used. This Technical Report cannot be considered to be law, just as the guides cannot. In all cases the user must have sufficient knowledge of the material to make a well-founded decision. In this report it is assumed that the reader has basic knowledge of hydraulics and geo-technics, preferably supplemented with some experience in the field of dike improvement and periodic safety monitoring.

An overview of existing guides and other publications published by TAW is provided in table 1.1 below.

Table 1.1 Relationship of TAW guides and publications

Integrale leidraden		Leidraden per waterkeringtyp		Specifieke publicaties	
Leidraad Grondslagen	1998	Leidraad Rivierdijken 1	1985	Materialen:	
Leidraad Toetsen		Leidraad Rivierdijken 2	1989	Eisen Klei	1994
op Veiligheid	1996	Leidraad Zee- en Meer-		Cementbetonnen	
		dijken	*	dijkbekleding	1991
		Leidraad Waterkerende		Asfalttoepassingen	
		Kunstwerken en Bijzon-		in de waterbouw	1984
		dere Constructies	1997		
		Leidraad Zandige Kusten	1995	Belastingen:	
		Leidraad Boezemkaden	*	Golfoploop	1972/
				en –overslag	1994
				Gereedschappen:	
				Keuzemethodiek dijk-	
				en oeverbekleding	1988
				Cel- en Triaxiaalproeven	1988
				Bodemonderzoek in en	
				nabij waterkeringen	1988
				Duinafslag	1984/
					1996
				Beheersaspecten:	
				Gas- en	1971/
				vloeistofleidingen	1973
				Landbouwkundig en	
				natuurtechnisch beheer	
				van rivierdijkgrasland	1986
				Pijpleidingcode **	1990
				Techn. Rapport	
				Boezemkaden	1993
				Techn. Rapport	
				Piping Rivierdijken	1994
* nog uit te brengen					
** heeft status van leidraad					

Integral guides
Fundamentals for Water Defence, 1998
Safety Monitoring at Water Defences, 1996

Guide by Flood Defence Type
River Dikes 1, 1985
River Dikes 2, 1989
Sea and Lake Dikes *
Water-retaining Hydraulic Structures and Special Structures, 1997
Sandy Coasts, 1995
Drainage Canal Embankments *

Specific Publications
Materials:
Clay Requirements, 1994
Cement Concrete Dike Revetment, 1991
Asphalt Applications in Hydraulics, 1984
Design Manual for Pitched Slope Protection, 1995

Loads:
Wave Run Up and Overtopping, 1972/1994

Tools:
Dike Bank Revetment Selection Method, 1988

Cell and Triaxial Tests, 1988
Soil Monitoring in and nearby Flood Defences, 1988
Dune Erosion, 1984/1996

Management aspects:
Gas and Liquid-bearing Pipelines, 1971/1973
Agriculture and Nature Management of River Dike Grassland, 1986
Piping Code, ** 1990
Technical Report on Drainage Canal Embankments, 1993
Technical Report on Piping at River Dikes, 1994

* to be published
** has the status of guide

1.3 Important Historical and Recent Research

The phenomenon of piping first was studied around the turn of the nineteenth century. Bligh developed an empirical calculation rule in 1910, on the basis of a number of cases of collapse of steel-founded brick dams on diverse earth foundations in India. A safe value for the permitted hydraulic head over the structure can be calculated with the calculation rule, as a product of the total horizontal and vertical seepage length under the structure and a factor which is dependent on the foundation. Bligh's calculation rule is also known as the 'line of creep' method. In 1935 Lane developed another empirical calculation rule, by which horizontal and vertical parts of the seepage line were calculated in a weighted manner; in the calculation of the seepage length only one-third of horizontal parts were included. According to Lane this modification of Bligh's rule was necessary to ensure proper calculation of the large flow resistance of vertical parts of the seepage line. He called his method the 'weighted line of creep' method. Bligh's rule was used for dike reinforcement in the Netherlands until the early nineteen nineties and it is recommended in *Guide on Designing River Dikes* (part 1, 1987); here generally only a horizontal seepage line played a role. Lane's calculation model was used until recently for piping inspections at hydraulic structures. Both rules are assumed to be conservative. A criterion of heave was developed by Harza [Harza 1935] in terms of the critical 'flotation gradient'; the concept has been developed further in later studies with reference to this mechanism.

A detailed overview of various studies, the criteria developed in the course of time for the piping and heaving mechanisms and design criteria for filters is provided in [Hsu 1981].

Since the end of the nineteen seventies the phenomenon piping has been studied in the Netherlands, under the auspices of TAW, and Germany. Most notably in the German research, the creation and development process of erosion channels in a layer of sand under the impermeable lower edge of a flood defence was studied by means of model tests. Dutch research resulted in a new mathematical model [Sellmeijer 1989], which describes the fundamentals of the erosion process which is at the root of piping. After subsequent elaboration of this model and validation by means of large-scale model tests, design rules were derived from this calculation model which are accessible for practical design and advice purposes [TAW 1994[2]]. In the meantime in various situations these new design rules have been shown to result in more favourable dimensioning of the horizontal seepage length needed, that is shorter piping berms than Bligh's classical calculation rule.

Vertical seepage line components, such as at cut-off walls, cannot be included in this model however. One of the recommendations of the Boertien committee (1993) was to check to what degree cut-off walls, also at dikes, could contribute to 'sophisticated designs'. Up until then the application of such screens was considered undesirable bearing in mind the limited sustainability and possible problems with its connection to impermeable layers of earth, due to the settlement and deformation character of dikes. , Subsequent research has been conducted into the effect of the short cut-off wall at river dikes [Van de Paverd 1994], partly on the basis of this recommendation. A possible failure mechanism in relation to this is the creation of heave due to too large a vertical hydraulic gradient behind the cut-off wall when this is placed at the inside toe of the dike. In 1995 design rules were derived for the dimensioning of cut-off walls, based on the heave criterion, with the help of the Fragments Model [Sellmeijer 1995]. These provisional design rules, which are supposed to replace Lane's empirical calculation rules, are included in the draft versions of the TAW Guides *Safety Monitoring at Water*

Defences and *Water-retaining Hydraulic Structures and Special Structures*. A recent study is concerned with the *Probabilistic Susceptibility Analysis of Heaving Design Rules* [GD 1998].

The Road and Hydraulics Division of the Directorate-General for Public Works and Water Management catalogued the damage caused by the high water levels in 1993 and 1995 along the main dikes of the major rivers.

The results of the recent studies mentioned have been included in this Technical Report.

1.4 Description of Content

Chapter 2 consists of advice to readers in which the use of this report is clarified from various angles. This chapter also includes a list of definitions and a section on the safety philosophy connected to the calculation models for flood defences.

The phenomena of cracking, piping, heave and horizontal seepage are described in chapter 3. The various aspects which play a role here are addressed in detail. This chapter is especially intended to provide background knowledge and to increase understanding of the material. The various methods and calculation rules, including the required input parameters and restrictions are discussed in chapter 4.

In chapter 5 the application of the methods is discussed in more detail. The emphasis is on safety assessment of existing flood defences, in the scope of legally prescribed five-year safety monitoring for primary flood defences. The difference between monitoring and design is clarified and the various possibilities to reduce the probability of piping are given.

Chapter 6 provides an overview of damage, caused by high water and day-to-day conditions. There is also an overview of emergency measures during high water. This chapter is especially important for the management and inspection of flood defences.

The application of the Technical Report is illustrated in chapter 7 using a number of examples.

A range of detailed information, particularly on calculation models, the definition of parameters and available software is included in the appendices.

The original intention was to bring this Technical Report back up to state-of-the-art standard, that is a collection of knowledge and skills published in earlier guides and (research) reports. In compiling this report a number of aspects emerged which need further in-depth study. In brief this refers to

- systematically settlement up a seepage line analysis at hydraulic structures
- assessing the permissibility of trees and plants on and in the vicinity of the flood defence
- assessing safety aspects at pipe crossings and pipes parallel to the flood defence
- settlement up a damage catalogue for management and inspection
- harmonising the use of calculation models to the safety philosophy.

In this report the generally accepted definitions of mechanisms such as cracking, piping and heave have been assumed. The interference of these mechanisms with other kinds of geo-technics mechanisms, such as the potential instability of slopes or of cut-off walls and stability screens are not addressed in this report.

1.5 The Creation of this Technical Report

The Road and Hydraulics Division of the Directorate-General for Public Works and Water Management (DWW) under the auspices of the Technical Advisory Committee on Water Defences (TAW) commissioned Grondmechanica Delft to draw up this technical report in co-operation with Fugro Ingenieursbureau B.V. and Arcadis Heidemij Advies B.V.

The project group comprised the following members
ir E.O.F. Calle (Grondmechanica Delft, author/final editing)
ir R. ’t Hart (DWW, project guidance)
dr ir G.J.C.M. Hoffmans (DWW, project guidance, since August 1998)
ir M.T. van der Meer (Fugro Ingenieursbureau, author)
ir J. Niemeijer (Arcadis Heidemij Advies, author)
ir M. van de Paverd (DWW, project guidance, until August 1998)

ing L. Verdink (Grondmechanica Delft, project secretary)

The project group was assisted by a sounding board comprising the following members
ir W. Epema (HHS Albasserwaard en Vijfheerenlanden)
ing H.A. Schelfhout (Province of South Holland)
ing P. Spaan (Waterschap de Veluwe)
ir R. Weersink (Bouwdienst RWS)
ir J.B. Weijers (DWW)

2 Advice to Readers, Definitions and Safety

2.1 Advice to Readers

Uses

The use of this Technical Report will depend on the objective for which it is used. The Technical Report provides an interface for a number of objectives as described below.

2.1.1. Safety assessment, designing, managing

Safety assessment (periodical safety monitoring)

The monitoring of a flood defence in the (near) future will play an important role, probably more important than the design or improvement of a flood defence. The scope of monitoring in relation to the five-year monitoring in accordance with the *Flood Defences Act* is provided in *Guide on Safety Monitoring at Water Defences*. The report at hand can be used as an aid for the aspects of piping and heave. In *Guide on Safety Monitoring at Water Defences* a distinction is made between the scores 'good', 'satisfactory' and 'unsatisfactory', according to the quality of the flood defence. No criteria could be established for the piping aspect which would result in a score of 'satisfactory' however. Monitoring will therefore only produce a score of 'good' or 'unsatisfactory'. Monitoring can follow various steps, with each step needing detailed information or complex calculations. This process is discussed in chapter 5, most notably sections 5.1, 5.2 and 5.3. The detailed descriptions of the mechanisms and models are included in separate chapters (chapters 3 and 4) to enhance readability. Specific areas for attention in relation to trees and pipes are discussed in sections 5.6 and 5.7 respectively. The damage catalogue, chapter 6, provides an overview of damage. Observed phenomena in relation to piping or heave can be interpreted using this chapter, and can be an important source of information for monitoring. The application of the models is illustrated in chapter 7 using examples.

Designing

The drawing up of a design will in many cases begin with the monitoring of an existing situation. In monitoring piping a distinction is only made between the scores 'good' and 'unsatisfactory': the score of 'good' is directly related to the design guidelines. That which is mentioned under *monitoring* is also applicable to a great extent to *designing*, although as a rule there are other preconditions in relation to the water level and life span for a design. Section 5.4 details the design of measures. Section 5.5. mentions a number of aspects with reference to the costs of various measures, which can play a role in the selection of various measures. This report only addresses the technical aspects. Other aspects which can play a role in designing or settlement up a flood defence are not dealt with here.

Managing

The Technical Report offers various interfaces in relation to management, although it is not specifically oriented to management. Specific matters for management, oriented to the maintenance of the flood defence with reference to piping are collected in section 5.8. In addition, it can be important for a manager to know which data or studies are needed to assess a flood defence. This information is collected in section 5.2 and 5.3, monitoring for dikes and hydraulic structures. To a large extent, questions on the influence of trees and pipes on the creation of sand boils, and the demands set, are answered in sections 5.6 and 5.7. Chapter 6 provides an overview of damage, phenomena observed for example, during high water or inspection which may be connected to piping or heave. Observations can be classified using this chapter. An indication is also given of the need for (emergency) measures and which measures are applicable.

2.1.2. Types of flood defence

A distinction is made between a number of types of flood defence. The Technical Report can be used for all types of flood defence, but does not provide a full picture for all types of flood defence. The specific characteristics by type are indicated below along with the degree to which the Technical Report is applicable.

Hydraulic structures

For hydraulic structures the term piping is traditionally used, but it refers to the same phenomenon as boil forming at dikes, be they sand-carrying or not. The Technical Report is therefore not split into individual parts for hydraulic structures and dikes. There are practical differences, or the emphasis for hydraulic structures is often on other aspects. These aspects are dealt with in separate sections. Sections 3.6 (horizontal seepage), 3.7 (three-dimensional effects and normative seepage line) and 3.8.4 (parameters in relation to horizontal seepage and three-dimensional effects) deal with mechanisms which are especially important for hydraulic structures. The aspects in relation to the monitoring of hydraulic structures are dealt with in section 5.3 and the design of measures in section 5.4.8. Section 6.3 addresses damage at hydraulic structures. Section 7.3 contains a case relating to a hydraulic structure.

River dikes
Piping is an important aspect at dikes in the upper rivers area, most notably because the high water level is maintained for a relatively long time. Dikes in the transitional area or in the tidal rivers area can be characterised as river dikes or sea dikes, depending on the duration of a high water. The Technical Report is particularly applicable to river dikes.

Sea dikes
Sea dikes take up a separate position. The piping aspect plays a less important role due to non-stationary influences. Time-dependent aspects are discussed in section 4.5. The application of models for time-dependent effects and the study needed in relation to this are only addressed indirectly. In most cases expert input is desired to estimate the influence of non-stationary aspects, which means that this aspect largely falls beyond the scope of the Technical Report.

Drainage canal embankments
One reason why drainage canal embankments are distinct from river dikes is because they must retain the design hydraulic head for which they are built. In principal, the assessment of piping runs parallel to the assessment at dikes in the upper rivers area. Generally, a stationary situation is assumed for rivers too, after all. The fact that the load on the drainage canal embankments remains almost permanently at design water level can be advantageous for study purposes. This means that a stationary groundwater flow is present in day-to-day conditions which can be measured using piezometric gauges for example. The results of these measurements can be used in the assessment of piping, which means that the soil test can be limited should the occasion arise. This is addressed in sections 3.8 and 5.2.2. No specific attention is given to drainage canal embankments however.

Other flood defences
In principle this Technical Report can be used for other non-primary flood defences. The norm used is however applicable to primary flood defences. No uniform norm has yet been worked out for other flood defences. The manager can work out a norm itself in most cases, for example in relation to risks and investments.

2.2 Definitions

Hinterland
the area lying inside the dike

Horizontal Seepage
forming of channels or hollow spaces on the side of a hydraulic structure as a consequence of the erosion of the ground

Aquifer
water-bearing sand layer

Inside (dike, toe)
on the side of the land or inland water

Outside (dike, toe)
on the side of the outside water

Soil Failure
see hydraulic soil failure

Limit Potential, Head Limit
hydraulic head in the aquifer determined by the weight of the covering layer

Heave

in this report this means the situation in which vertical effective stresses in a sand layer fall away under the influence of a vertical groundwater flow, also called fluidisation or the forming of quicksand

Hydraulic Soil Failure
loss of grain contact in the ground as a consequence of too great a degree of water overpressure; in the case of a cohesive covering soil layer this leads to uplifting and cracking, in the case of a non-cohesive soil layer to heave

Entry Point
(theoretical) point where the outside water enters the water-bearing sand layer, as a consequence of the hydraulic head over the flood defence

Critical Head or Critical Seepage Length
value of the hydraulic head or the length of the normative seepage line, where no piping or heave occurs

Seepage
water which flows through or under a flood defence, as a consequence of the hydraulic head over the flood defence to be retained

Seepage Embankment
An embankment constructed in the hinterland directly adjacent to the dike to reduce the flow off of seepage water; an attempt is made to prevent the occurrence of piping phenomena and to restrict the problem of water inside the dike during high river drainage

Seepage Line
possible path in the ground taken by the seepage water, from the entry point to the exit point

Seepage Length
the distance which the seepage water moves

Cut-off wall
a watertight screen constructed vertically in the ground, extending the line of seepage

Drainage Ditch
ditch on the inside of the dike, the aim of which is to catch and drain off seepage water

Seepage Erosion
see piping

Cracking
cracks in the covering layer which is lifting up

Uplift
form of hydraulic soil failure by which a cohesive covering layer is lifted up as a consequence of water overpressure in the underlying aquifer

Piping
the creation of hollow spaces under a dike or hydraulic structure, as a consequence of a concentrated seepage flow carrying ground particles; also called seepage erosion. In the factual definition, piping is the forming of an open channel from entry point to exit point

Potential
piezometric head in relation to a reference level

Leakage Length
linear measurement which gives the relationship between the horizontal transmissivity of the aquifer (product of horizontal permeability coefficient and thickness of the aquifer, kD value) and the vertical hydraulic resistance of the covering layer (quotient of thickness of top layer and vertical permeability, d/k)

Piezometric Head (in a point in the ground)
level to which the water would rise in a piezometric gauge with filter at the location of the point; expressed in water column metres with respect to a reference level

Theoretical Potential
potential in the aquifer if this is not limited by the weight of the covering layer for example

Exit Point
location where seepage water first surfaces

Exit Gradient
hydraulic gradient in the groundwater surface at the location of the exit point

Hydraulic Gradient
quotient of the difference in head between the two points and the distance between those points; also called gradient

Hydraulic Head
difference in head between two points, for example the two sides of a flood defence

Foreland
site outside the dike; site between the dike and the river; specifically in relation to piping: site exclusively along the dike, where a continuous clay layer is found

Water Overpressure/Underpressure
difference between the current water pressure and the hydrostatic water pressure

Boil
concentrated outflow of seepage water, for example through a crack channel or a hole in the covering clay layer

Sand Boil
boil which carries out sand out of the substrate

TAW
Technical Advisory Committee on Flood Defences

TAW Line of March
Technical Advisory Committee on Flood Defences programme with respect to safety aspects of flood defence in the Netherlands

2.3 Safety Philosophy

Calculation preparation method

The safety factors to be complied with in a calculation are mentioned for the various calculation modules in this Technical Report. They are generally accepted safety factors for Dutch hydraulic or flood defence practice, which are either recommended in rules or guidelines (Geo-technics norm NEN 6740, earlier TAW guides etc) or common in practice among Dutch engineering consultants. In a few cases where there is ambiguity a recommendation will be made on the basis of the insights of this report's compilers.

Safety factors to be complied with are not isolated variables, but should be seen in the context of the preparation method by which soil and other calculable variables for the design and monitoring calculations are estimated. For classical calculation rules a specific regulation is mostly not given; it is assumed that the designer is working with the best possible estimates of problem variables, and that great care is taken to select safe values in the case of great uncertainty.

The definition of safety depends on the type of problem variables. For some variables a realistic maximum and minimum limit is important, as in the case of the minimum seepage length in a dike section for the sake of a piping analysis or the minimum thickness of a covering clay/peat layer in the hinterland for a crack analysis. For other parameters a ‘low average’ is important, for example the volume weight of clay or peat in a covering layer for the determination of the crack potential.

In the classical calculation rules the manner in which safe estimates are determined is mostly left to the assessment of the designer. One designer will select the real average of a sample survey of measurements in determining the volume weight of the earth layer, whereas another will select a somewhat lower value due to safety considerations. Both selections are defensible, but ultimately the (subjective) opinion of the designer is the deciding factor.

The degree of subjectivity in the selection of parameters has been pushed back somewhat in the more recent calculation rules, by prescribing the use of representative or characteristic values for example. In the Geo-technics norm NEN 6740 table values are given for a number of soil parameters which can certainly be considered representative. We also call them nominal values, which can be used if no measurements are available. If measurements are available characteristic values must be applied. They are estimates of the parameter itself or of the average of the parameter over a soil layer at a location, on the basis of statistical processing of the measurement series, with under or over probability of no more than five per cent. In the norm mentioned a calculation method is indicated by which the characteristic value can be determined using the measurement series. This method is applicable when there are measurements from a soil test at the location studied in the design and monitoring analysis (local study) and when the ‘low average’ of the soil parameters is important (such as volume weights, shear strength and compression constants). If the study area is regional then spatial distribution (see *Guide on the Design of River Dikes*, part 2, test collections chapter and others) must be taken into consideration in determining the characteristic values. When, for the problem variables, not ‘averages’ but individual values are normative (such as the seepage length) the calculation method mentioned in NEN 6740 cannot be applied. In [Calle 1996] formulas are provided to determine characteristic values in those cases too.

Safety level target

The following can be said of the safety level target using the method given in the Geo-technics norm and the TAW Guides:

- For the more stringent safety class in the NEN 6700 series (Technical Foundations for the Construction Industry) a permissible failure probability for the main components of a structure as a consequence of technical shortcomings of 1.6 10^{-4} during the reference period is assumed. This is mostly the planned operational life span of the structure. The calculation methods worked out in the norm (including NEN 6740) are expected to deliver structural designs which meet the (minimum) requirements of this norm.
- In the two-volume *Guide on the Design of River Dikes*, the classic deterministic approach is assumed with reference to the mechanisms connected to sand boils. No safety requirements are formulated here explicitly in terms of permissible failure probabilities. It is stated in general terms that the probability of the flood defence failing due to structural shortcomings must be negligible in relation to the permissible annual probability of exceeding the normative high water level (NHW for river dikes and design level for sea dikes) which forms the basis of the calculation of the flood defence. This probability, the norm frequency, is recorded in the *Flood Defences Act* (FDA) for the various dike enclosure areas in the Netherlands. Subsequently, ‘negligible’ was defined as ‘10 per cent maximum’. The concept of ‘hydraulic overload’ or more succinctly ‘overload’ is introduced in the guides mentioned. This means that a larger capacity clears the defence due to run up or wave overtopping than the capacity considered permissible in relation to the strength of the inside slope. This overtopping criterion replaces the water level criterion. The permissible probability of overload is taken to be equal to the norm frequency.
- In *Technical Report on Piping at River Dikes* [TAW 1994] the guiding principle for the method is that the (annual) probability of piping under the flood defence around a protected area must be less than ten per cent of the norm frequency.
- In *Guide on Water-retaining Hydraulic Structures and Special Structures* [TAW 1997] the guiding principle is a permissible annual probability of failure due to other mechanisms than overload, if no overload occurs, which is less than ten per cent of the accepted probability of overload. From the further context of this guide it can be concluded that this probability is reserved for each of the hydraulic structures in the flood defence. On the one hand this is a less stringent requirement than the guiding principle for the *Technical Report on Piping at River Dikes*, because there the ten per cent requirement is applicable for all dikes around the protected area together. On the other hand it

is a toughening, because the accepted failure probability for the hydraulic structure is still spread over various mechanisms (strength/stability and failure of the barrier).

- Roughly speaking the requirements in [TAW 1994] and [TAW 1997] amount to an accepted failure (annual) probability of around one per cent of the probability of overload, for each soil mechanical mechanism (macro-instability and piping) and for each dike section or each water-retaining hydraulic structure. This requirement corresponds to the provisional indications in the scope of the standard point studies for the TAW *Line of March* (from an overload philosophy for dike sections to an inundation probability philosophy for dike enclosure areas).

The safety levels desired in the TAW guides are related to primary flood defences. For other water defences, including drainage canal embankments, no explicit safety requirements have been formulated other than must be designed or assessed in accordance with applicable (deterministic) practice. A safety philosophy for drainage canal embankments is under development.

More stringent safety requirements

For the use of partial safety factors (and characteristic values for the soil and load parameters) in accordance with the Geo-technics norm NEN 6740 it should be considered that the underlying concept is to realise the above-mentioned safety for the main components of the structure or for the main mechanisms. In special cases it may be necessary to impose more stringent rules for structural safety. This must be shown by a fault-tree analysis of the structure (dike or hydraulic structure) as a whole. In those cases this more stringent safety level will be expressed explicitly in the safety factor to be applied. A methodology has been proposed by the Province of Zuid-Holland to calculate corrections to the safety factors provided in NEN 6740, depending on the degree to which the safety requirement deviates from the safety requirement in NEN 6700 [PZH 1997]. Consideration has been taken of the translation which is necessary to move from the permissible annual failure probabilities (TAW system) to permissible failure probabilities by planned operational life span (NEN system). The simple multiplication of the permissible failure probabilities on an annual basis by the number of years of the planned life span results in permissible failure probabilities which are at variance with the TAW safety philosophy. As a provisional approach the TAW working group on safety aspects (TAW-E) recommended that the number of years of the planned operational life span used in the rendering be limited to ten.

Probabilistic analysis

The opportunity is still available to conduct a full probabilistic analysis instead of working with regulations for safety factors and parameter selection. In that case the failure probability with respect to the mechanism to be studied is determined by stochastic analysis, in which all uncertainties about problem variables are quantified (see the CUR handbook C190 and others) [CUR 1997].

The failure probability found must be tested against a permissible failure probability. The current official TAW guidelines are not explicit on that last point. Recent studies, in the scope of the TAW *Line of March* do provided indications. For the mechanisms cracking combined with piping or the mechanism heave (by fluidisation) the provisional indication is a permissible failure probability by dike section by year, equivalent to one per cent of the probability by year that the normative high water level for the relevant dike enclosure area is exceeded. This probability is recorded for the various dike enclosure areas in the Netherlands in the *Flood Defences Act*. As mentioned above, in special cases variant (smaller) permissible failure probabilities are needed; this must however by underpinned with a fault-tree analysis by case.

The advantage of a probabilistic analysis is that stricter designing and monitoring can be realised. This can be advantageous precisely in monitoring situations, when a sufficient safety level can be proven on the basis of a probabilistic analysis, while that is not the case when the usual calculation rules are applied.

Design versus safety assessment

In principle there is no difference between pursued safety in the design process and the safety level used for periodical safety tests of flood defences. At most there will be a tendency to interpret the safety requirement more freely for safety monitoring. It is naturally better to realise a better result via better measuring and stricter calculation; safety is connected to ‘knowledge’ to an important degree, that is the reduction of uncertainties. One essential difference between designing and monitoring is the time horizon. In the design process a safety level must be pursued during the planned operational life span of the water-retaining structure. This is fifty years for dikes; there is a strong inclination to double this period for hydraulic structures. In defining load and strength parameters developments which influence these parameters during the period should therefore be given consideration, such as a rise in sea level and, if applicable, deterioration of strength. For periodic safety tests in accordance with the *Flood Defences Act* the time horizon is five years. That means that it must be shown that the safety required during (at least) that period is guaranteed. In that case inclusion of developments which influence load and strength (in principle) are limited to that period. Obviously, a consideration can be made in relation to a test of what the situation will be in five years and whether it is prudent to bring forward possible or anticipated reinforcement programmes, but that is another consideration than of a strictly safety philosophical nature.

Furthermore, in the design process, in addition to the requirements with reference to the minimal safety level to be realised, other considerations will obviously play a role, such as the question of manageability and the optimisation of maintenance, including damage repair after extreme loads. Strictly speaking, those considerations are separate from the safety philosophy and do not therefore need to play a role in periodic safety monitoring.

3. Description of Mechanisms

3.1 Various Stages in the Creation of Piping

A typical soil composition in which piping can play a role is illustrated in figure 3.1a. This figure shows a cross section of a dike indicating the soil composition under the dike. In the case of a sufficiently high outside water level the following phenomena may occur in the following order:

- **uplift of the covering layer on the inside of the dike.** A high outside water level will cause the water pressures in the sand layer to increase. When the water pressures at the site of the covering (scarcely permeable) layer on the inside become greater than the weight of that layer, this will start to push up. In practice uplifting is sometimes, but not always to be observed via weak wave movements on the surface when it is trod upon.
- **cracking of the covering layer and the creation of boils.** Cracks in the covering layer can occur due to uplift, through which the seepage water finds its way to the surface**.** Due to the erosion capability of this seepage flow a channel is created between the sand layer and the surface; the crack channel. The eroded material from the crack channel is borne by the seepage flow and deposited around the outflow opening. The diameter of the crack channel can vary substantially depending on the flow speed and the erodibility of the material in the covering layer.
- **erosion of the sand layer.** Sand particles are transported from the sand layer to the crack channel by seepage exiting the sand layer. The crack channel is thus filled with sand in a fluidised state. The flow resistance in the crack channel therefore rises. There are now two possibilities, namely (1) the flow speed at the site of the exit point decelerates to such a degree that the erosion process stops due to increased resistance, or (2) the flow speed decelerates insufficiently, so that the transport of sand to the crack channel continues. In the first case the boil will start to produce ‘clean’ water. In the second case sand will be transported by the seeping flow via the crack channel to the surface and be deposited around the boil, where a sand crater is created. In the sand layer small channels (pipes) are created at the top of the sand layer directly under the top layer, which expand upstream.
- **the creation of through pipes.** In the case of sufficient hydraulic head over the defence the erosion channels will continue to grow until they reach the outside water. There is then an open connection between outside water and exit point, which means that the flood defence has become sensitive to piping (the seepage erosion or piping mechanism is then a fact).
- **collapse of the flood defence.** As a consequence of the creation of through pipes they will continue to erode at an accelerated rate, so that their dimensions increase. Ultimately it is assumed that this will lead to hollow spaces under the flood defence which are so large that subsidence and cracking of the dike body occurs. Factual observation of this collapse process and its duration, at least at dikes, are not available however. In small-scale tests it has been observed how the further erosion of the sand layer after the creation of a through pipe occurs very quickly, that is in around a minute. Although the creation of through pipes cannot be immediately identified with the actual collapse of the dike itself, this is assumed in the current design philosophy. The limit state ‘creation of through pipes’ is accordingly interpreted as limit state in relation to collapse.

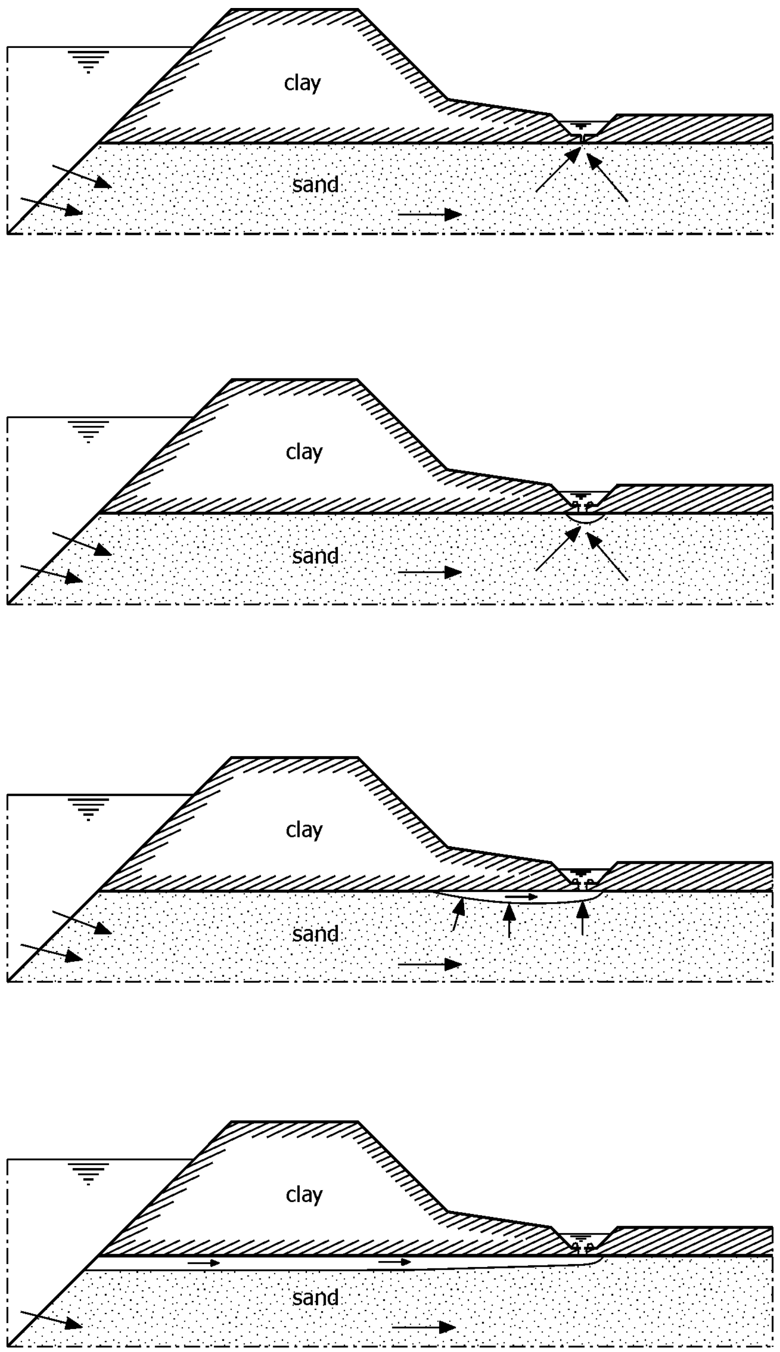


Figure 3.1 Phases in the creation of piping

(a) cracking top layer
clay
sand
(b) boil forming, start of erosion
clay
sand
(c) pipe forming by receding erosion
clay
sand
(d) through pipe (piping mechanism)
clay
sand

3.2 Soil Composition in which Piping Plays a Role

In this Technical Report a piping sensitive situation means soil composition that allows piping to play a role. The following elements are characteristic of a piping sensitive situation:

1. Horizontal groundwater flow through a sand layer, which is transported upstream by a water reservoir (see, river, watercourse, (storage) reservoir, etc) and has an exit point downstream where the groundwater can flow out freely. From the exit point the sand layer in the upstream direction is covered by a relatively impermeable and cohesive surface, for example the underside of a cohesive soil layer or the underside of a concrete or masonry structure resting on the sand layer, which makes it possible for the channels to remain.
2. An entry point at a sufficiently short distance from the exit point that an open connection can be created between the water reservoir upstream and the exit point.

We recognise those elements in the soil composition in which piping may play a role.

In figure 3.1a a typical substrate composition is illustrated in which piping can play a role. The exit point in this case is formed by a crack channel, by which water from the sand layer can flow out in the berm ditch and can carry out sand particles. For a description of the crack mechanism refer to section 3.3. When the covering clay layer is so heavy that cracking cannot occur, neither can the erosion mechanism as no sand can be borne away. The soil configuration is then insensitive to piping.

Another typical soil configuration in which piping plays a role is illustrated in figure 3.2. Here two sand layers are present which are both directly connected to the outside water. For both, a crack channel can originate on the inside of the dike. In current practice it is common that the two sand layers are considered separately for design and test controls. One conceivable situation is that the upper sand layer is not sensitive to erosion due to its limited thickness or due to the fact that it consists of relatively coarse material, but that the lower layer is sensitive. The upper sand layer can then work as a natural hydraulic filter, preventing migration of sand from the lower layer. The filter function must be checked using filter rules. When the upper sand layer is somewhat cohesive due to the presence of silt or a clayey mixture the creation of a through channel up to the surface cannot be excluded. In doubtful cases it is recommended that expert assistance is called in, or the safest option selected, which is to consider the sand layer separately.

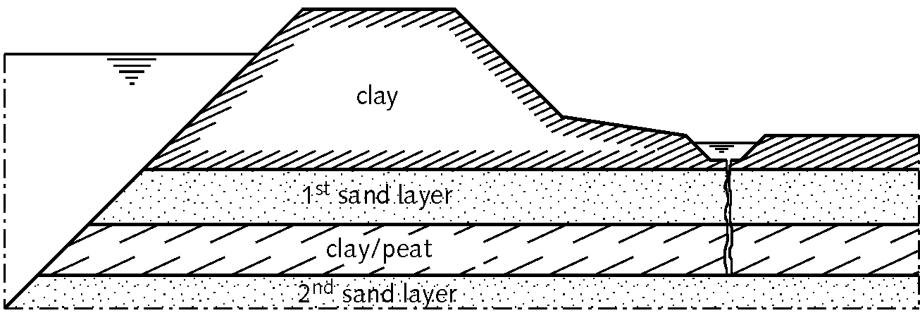


Figure 3.2 Potential piping sensitive substrate composition with two sand layers

clay
1st sand layer
clay/peat
2nd sand layer

Figure 3.3 shows an example of a dike cross section in which piping does not play a role. In this configuration seeping flow does occur via the substrate through the dike body itself and exits on the inside slope of the dike. It is there that a seepage surface originates. Although erosion of sand particles can occur here any erosion channel starting to form will collapse in on itself, because the sand has no cohesive qualities. This process repeats itself continuously which leads to crumbling of the inside slope. Although piping does not originate here, another mechanism does, and we call it micro-instability.

Finally, in figure 3.4 another configuration is illustrated in which piping does not play a role. The composition of the substrate is the same as in figure 3.1a, but a filter structure has been installed at the exit point which prevents migration of sand particles from the sand layer.

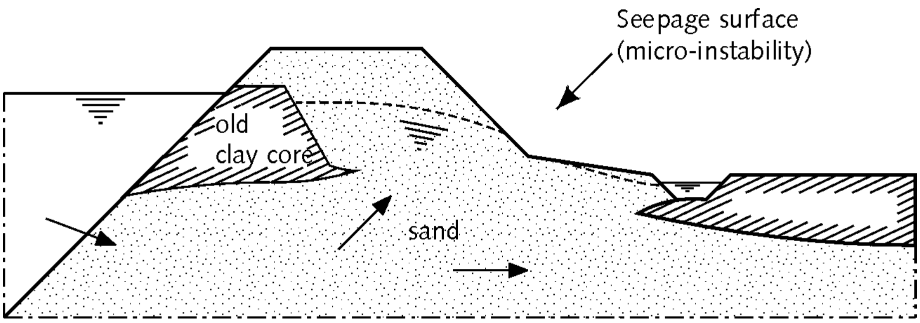


Figure 3.3 Substrate and dike composition which is insensitive to piping
Seepage surface (micro-instability)
old clay core
sand

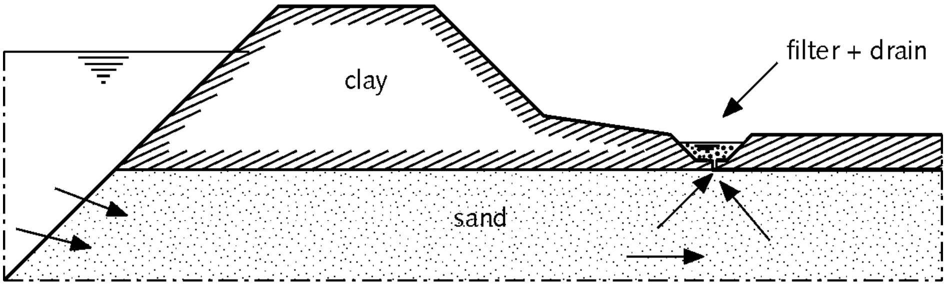


Figure 3.4 Filter solution
filter + drain
lay
sand

3.3 Cracking of a Covering Clay Layer
In the illustrated dike and substrate configurations in which piping may play a role an open connection between the sand layer and the ground surface has been assumed. That connection can originate if the water pressure in the sand layer against the underside of the clay layer corresponding to a sufficiently high outside water level is equivalent to the weight of the overlying ground. Due to the vertical strength equilibrium the water pressure can never increase. It is assumed that cracks will form in the covering layer because of this. This phenomenon is called cracking. The crack forming in the clay layer and the eroding effect of the seeping flow mean that small channels can originate (crack channels), through which the seepage flow can carry out sand particles from the sand layer upwards and so form a sand-carrying boils.

If a covering clay and peat layer is present, cracking is a necessary condition for the creation of sand boils. If they are no cracks there will be no piping, unless open channels are already present, for example as a result of dead tree roots, digging and, depending on the groundwater situation, cracks in the clay. The design check for new dikes or the monitoring of existing dikes in relation to safeguarding against piping therefore starts with the check on the safeguard against cracking. If this has been done then the further monitoring of the piping mechanism is unnecessary, provided a check has also been made that boil forming cannot originate due to the other causes mentioned.

This condition also implies a possibility of adapting the design to ensure safeguarding against piping, that is increasing the weight of the covering layer by introducing a piping berm. In doing so the place from which cracking can occur is shifted further inwards, so that the seepage length increases.

3.4 Receding Erosion
We consider the situation in figure 3.1. As long as the hydraulic head over the flood defence is sufficiently small clean water will exit at the exit point. If the head increases, and so also the intensity of the seepage flow, then this will lead to sand particles being carried out from the sand layer. This is called a sand boil. The sand is deposited around the exit point (sand crater). The erosion of the sand

starts at the site where the seepage flow emerges from the sand layer in the connecting channel with the surface, due to the high local gradients as a consequence of the constricting streamlines. In the sand layers a hollow space originates at the site (figure 3.1b). Later, in the case of more expansion head, small channels form at the top of the sand layer (directly under the impermeable edge), which expands in the direction of the outside water (figure 1.3c). This process is called ‘receding erosion’. In first instance the channel forming has a reducing effect on the erosion process, because flowing gradients decrease (see also figure 4.4), so that (in the case of a constant outside water level) the receding erosion comes to a halt. The length of the channels (pipes), and so the degree to which the receding erosion makes progress, is dependent on the head over the flood defence. When the head over the flood defence is large however, the receding erosion will continue to the degree that the erosion process is not reduced, but precisely enhanced. In that case we say that the critical hydraulic head has been exceeded. We call this progressive receding erosion. This process continues until the pipes reach the water on the outside. The seepage erosion or piping mechanism is then a fact. (figure 3.1d.)

The hydraulic head, by which the process of receding erosion stops just in time, is called critical head. In section 3.4 we return to the parameters decisive for the critical hydraulic head. For the moment it suffices to state that the seepage length in particular is very important, in addition to parameters which are difficult to influence such as permeability and grain diameters. This parameter can be influenced in the design of the flood defence and is the most important design variable.

3.5 Heave Mechanism

Cut-off walls can be used at water-retaining hydraulic structures and nowadays also at dikes. Cut-off walls increase the resistance which a seepage flow experiences. The effect is that the seepage is reduced and, even more importantly, the flow velocity and so the probability of erosion of the sand layer is reduced. At cut-off walls on the downstream side of the flood defence the exit direction of the seepage flow is vertical (see figure 3.5). The effective stresses in the sand behind the cut-off wall is reduced due to the upward seepage flow. In the most extreme case the effective stresses fall away completely, so that a quicksand situation originates. This mechanism is called heave.

Decisive for this mechanism is the increase in water pressures, from the ground surface, with the depth in the sand behind the cut-off wall (see figure 3.5). If there is no seepage flow then the increase is hydrostatic and there is effective stress in the sand. In the case of upward seepage the water pressures are greater than hydrostatic (and so there is water overpressure), which leads to a reduction in the effective stress. A limit state is reached when the increase in the water overpressure with the depth is equal to the increase in the effective stresses if the water pressure was hydrostatic. The increase in the water pressure can be expressed in terms of the vertical gradient of the groundwater potential. The gradient at which the limit state occurs is called the critical gradient. The actual vertical gradient of the groundwater potential behind the cut-off wall, the increase in the water overpressure with the depth, is generally not constant. That is why the average vertical gradient of the groundwater potential is calculated over the cut-off wall for heave checks. This is tested against the critical gradient, taking into consideration a safety factor.

Cut-off walls have traditionally been resisted at dikes because they were seen as ‘strange elements’. The Boertien Committee, however, recommended that cut-off walls not be precluded as an option for ‘sophisticated designs’. This recommendation led to a situation in which proper and easy to use calculation models for checking the heave mechanism were developed in the past five years (see chapter 4).

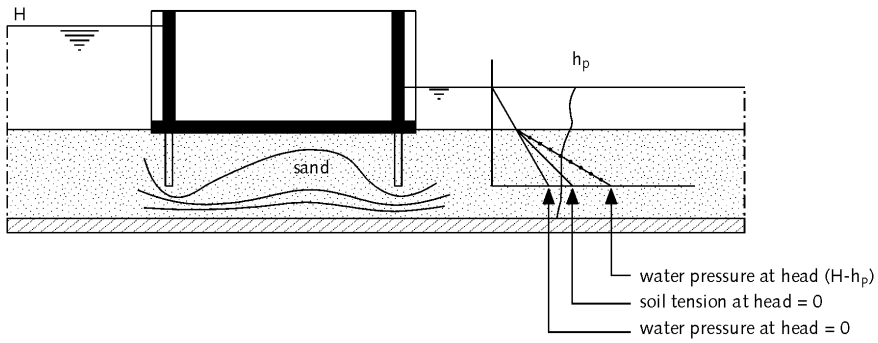


Figure 3.5 Heave mechanism
sand
water pressure at head ($H-h_P$)
soil tension at head =0
water pressure at head = 0

3.6 Vertical and Horizontal Seepage at Hydraulic Structures

3.6.1 General

Hydraulic structures are an interruption in the continuous soil body of the dike. This means that there is not only seepage under the hydraulic structure, but also seepage around the hydraulic structure.

The seepage analysis is central in assessing a hydraulic structure for boil forming and piping. Guiding principle for the assessment is, after all, that there is sufficient resistance to boil forming and piping along every possible seepage line under and around the hydraulic structure. A good three-dimensional analysis of possible normative seepage lines is therefore always the first and often decisive step.

The assessment criterion for every seepage line depends on the outflow conditions:

- for the outflow at the site of a filter structure safety with respect to boil forming and piping is guaranteed, provided the ‘filter rules’ are met;
- for a vertical outflow, behind sheet piling for example, the average hydraulic head for the vertical part in the seepage line must be assessed (heave)
- in all other cases a check must be made for piping.

The forming of through channels under the hydraulic structure is called seepage erosion. This is comparable with the situation at dikes. Cut-off walls to prevent vertical seepage erosion are called ‘anti-seepage vertical screens’. Channel forming around the hydraulic structure is called horizontal seepage erosion. This phenomenon is not an issue at dikes. Cut-off walls to prevent horizontal seepage erosion are called ‘anti-seepage horizontal screens’. Naturally combinations of vertical and horizontal seepage erosion are also possible.

The hydraulic head to be retained can refer to high outside water levels, but also maintenance situations (for example, whole or partial drying out of a lock chamber) or an extreme lowering of the inside water level. In addition, in some cases the hydraulic head usually only acts over part of the hydraulic structure, for example over a lock head.

At hydraulic structures piping/boil forming only occurs if there is sand directly under or alongside the hydraulic structure. This is not only valid for an originally piping sensitive dike composition (figure 3.2), but also for an originally non-piping sensitive dike composition (figure 3.3). In the second case there is no more than erosion of cohesive material alongside and under the hydraulic structure. Naturally a check will have to be made as to whether piping could be an important factor via a seepage line under the closing layer; that is the same as at dikes.

Given that sand is found directly alongside or under the hydraulic structure the sensitivity to piping/boil forming depends on the type of hydraulic structure. A short sluice with a deep sill is more sensitive than a lock with a greater length, or a cut in the dike with a sill in a higher position where the head to be retained is considerably restricted by the sill height.

Not every hydraulic structure has an important function in resisting piping: the normative seepage line need not necessarily pass some screens. To illustrate this, three solutions for the design of the outflow opening on the inside of the flood defence for a lockable discharge sluice are shown in figure 3.6. For solution (b) ‘inclined wings’ the resistance to horizontal seepage must be realised wholly by the interface between the hydraulic structure and the soil body next to it; this is only an acceptable solution when the hydraulic head to be retained is very small. For solution (a) ‘long front wall’ and (c) ‘inclined wings and return walls’ the front walls and the return walls respectively function as anti-seepage horizontal screens.

It is important that the anti-seepage horizontal screens are set sufficiently high, that is to design level plus an additional height in relation to settlement.

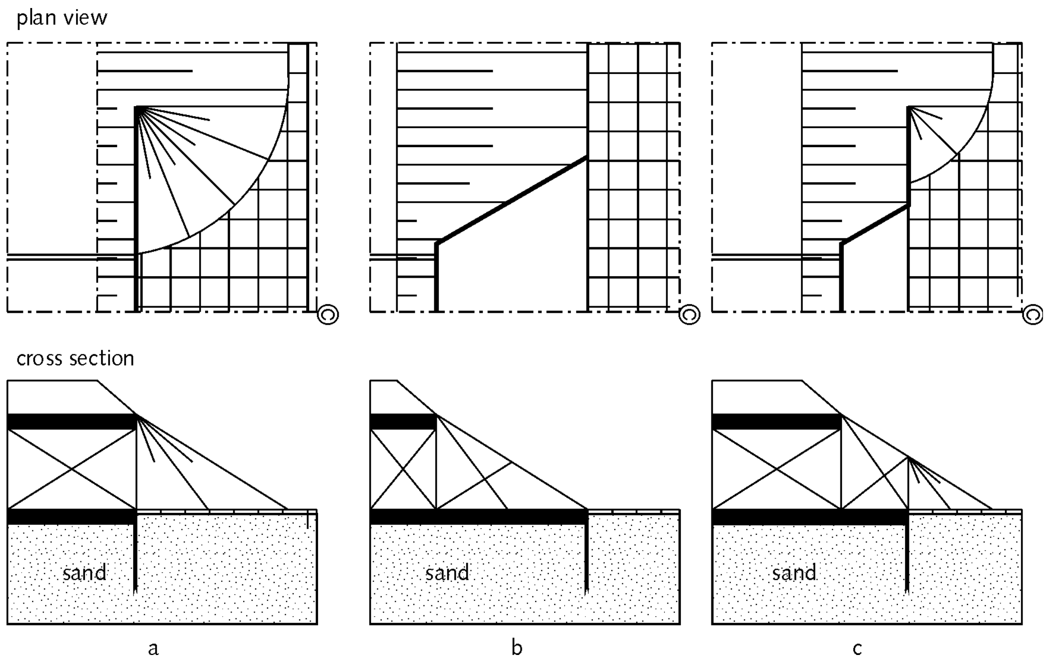
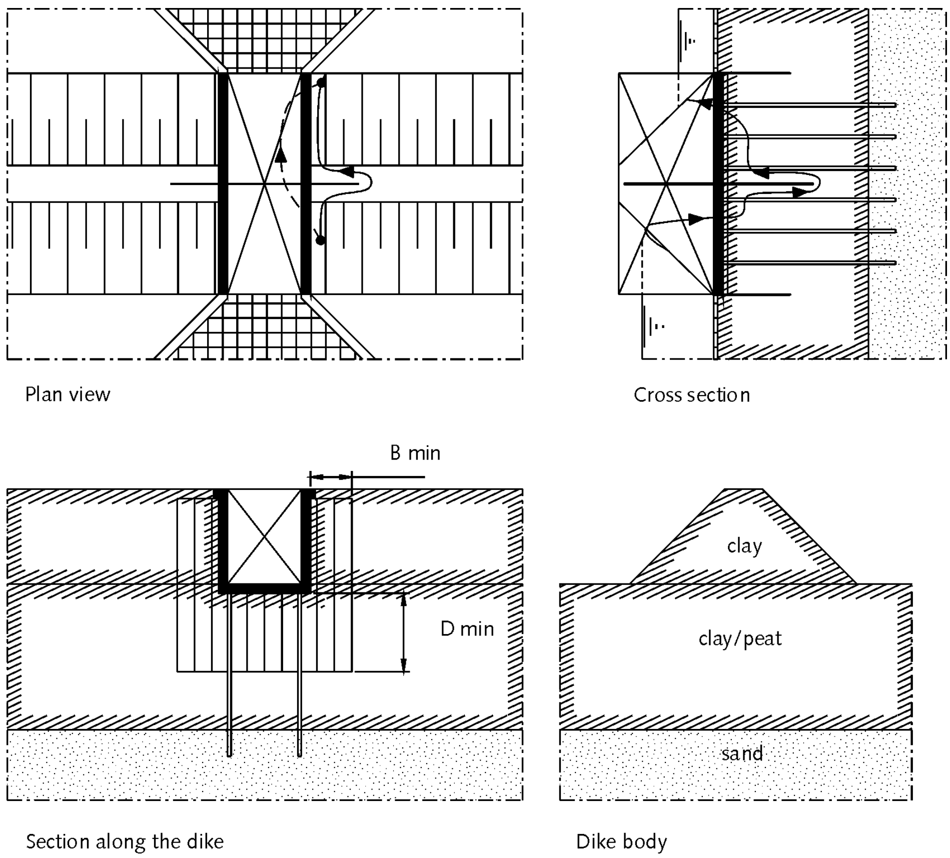


Figure 3.6 Principle solution outflow lockable discharge lock.
a) long front walls, b) inclined wings, c) inclined wings and return walls
plan view
cross section
sand
sand
sand

3.6.2 Hydraulic structures included in an impermeable soil package
Figure 3.7 shows a hydraulic structure (for example a sluice) where the undisturbed dike profile consists of an impermeable dike body on an impermeable Holocene package. It is assumed that the Holocene package is sufficiently thick to accommodate the cut-off wall. It is also assumed that cracking in the Holocene package will not occur. The hydraulic structure is founded on piles.

In this case minimum dimensions of anti-seepage vertical and horizontal screens suffice to guarantee the connection of the hydraulic structure to the undisturbed impermeable dike body. The background to this is that a perfect connection of the hydraulic structure to the ground next to it is often difficult to guarantee. In the course of time space can be created due to differences in settlement, temperature effects etc, so that micro-instability can occur beside the hydraulic structure. Hollow spaces should also be assumed under the pile foundation, which must also be closed with a short screen.

Notice that the short sheet piling at the inflow and outflow sides in the example illustrated are not continued as anti-seepage horizontal screens and therefore play a subordinate role in resisting boil forming and piping. The normative seepage length for piping will have their start and end points beside the hydraulic structure, at the connection to the outside and inside slope respectively.



3.7 Hydraulic structure in clay dike on thick Holocene package
Plan view
Cross section
Section along the dike
Dike body
clay
clay/peat
sand

3.6.3 Hydraulic structures on permeable soil

Figure 3.8 shows a hydraulic structure (for example a sluice) in a dike, where the undisturbed dike profile consists of (a) a permeable or (b) an impermeable dike body, in both cases on a permeable package. The hydraulic structure is founded on natural subsoil. As in the above-mentioned case the short sheet piling plays a subordinate role at the inflow and outflow side for resisting boil forming and piping.

Both anti-seepage vertical and horizontal screens must be introduced for both dike types. In case (b) horizontal seepage at the sand-clay interface is possible. In case (a) seepage capability cannot be excluded, because a through pipe can originate immediately next to the structure. Note that in case (b) the anti-seepage horizontal screen in the clay package continues over the full width for practical reasons only; strictly speaking a minimum cut in the clay would be sufficient to prevent the creation of erosion channels.

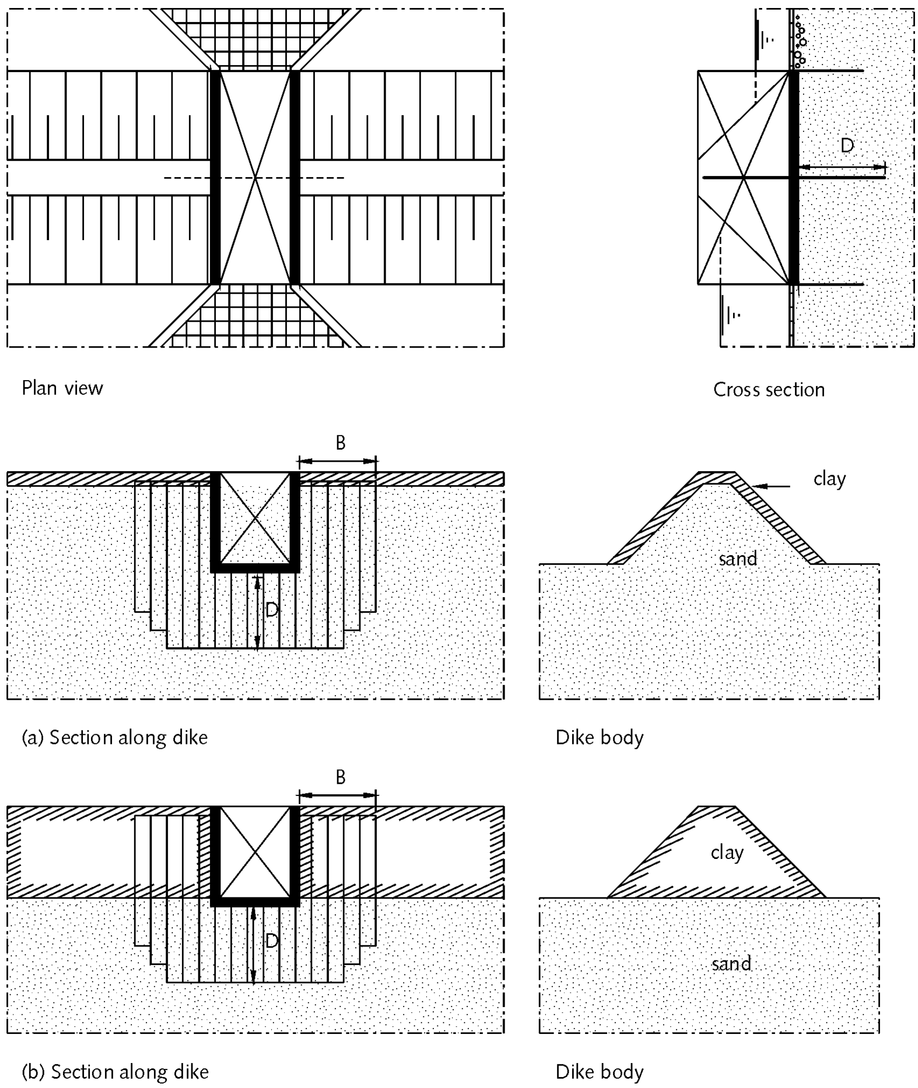


Figure 3.8 Hydraulic structure on permeable substrate

a) permeable dike body

b) impermeable dike body

Plan view
Cross section
(a) Section along dike
Dike body
clay sand
(b) Section along dike
Dike body
clay sand

3.7 Important Parameters

3.7.1 Crack mechanism

For the crack mechanism the important parameters are groundwater potential in the potentially piping sensitive sand layer at the site of a possible crack location and the weight of the covering ground at that location. The weight can be calculated if the ground layer arrangement and the dry and wet volume weights by ground layer are known.

The groundwater potential (piezometric head) is dependent on the outside water level (at sea, on the river or in the lake) and the geo-hydraulic configuration of the package. Important are:

- the length, thickness and permeability of the covering package in the foreland (if present);
- the thickness and permeability of the covering package in the hinterland;
- the permeability (kD value) of the sand layer;
- the potential preconditions: outside water level and head in the sand layer inside the dike. This last one is usually related to the polder level.

The piezometric head in the sand layer at the site of a potential crack location can be calculated using the WATEX computer program among others (see chapter 4). The geo-hydraulic parameters for a WATEX analysis can be estimated on the basis of estimates of the permeability and the thickness of covering layers outside and inside the dike and of the sand layer. Water pressure response measurements (for example 13 hour measurements) can also be used to estimate the geo-hydraulic parameters or to fine tune estimates of these parameters.

3.7.2 Piping mechanism

The parameters which play a role in this mechanism are the hydraulic preconditions (the hydraulic head over the flood defence), the seepage line and the configuration and material composition of the potential erosion sensitive sand layer.

The normative hydraulic head is the difference between the normative outside water level (design level, DWL) at sea dikes, normative high water level (NHW) at rivers, including the rise in sea level expected and the water level on the inside of the flood defence, if a free water level is present, or the ground surface level (including expected subsidence). The sea level rise and ground subsidence to be expected used in the calculation must be dependent on the plan period set for designing or the statutory period of five years between two safety monitoring moments. Values to be kept for the rise in sea level are submitted in the book of preconditions '*Hydraulic Preconditions for Primary Flood Defences*'.

In principle, the seepage length is the distance between the entry point for groundwater flow through the sand layer on the outside of the defence and the exit point on the inside. Sometimes those points can be indicated in a natural way, mostly they are not. If a crack sensitive top layer is present on the inside the possible crack point nearest to the flood defence is selected as exit point. The choice of the entry point is dependent on the presence of foreland on the outside of the defence. Foreland can, at least partly, be included in the calculation of the seepage length. In fact, when there is foreland, a theoretical or imaginable entry point must be determined.

If vertical cut-off walls are introduced the location and the length of those screens are also important. Screens on the upstream side of the dike or the hydraulic structure are intended to extend the seepage line. They are usually long screens. Screens on the downstream side have the same purpose, but also ensure that the seepage flows out vertically; there must be monitoring for heave here.

Depending on the calculation model used for piping the information needed about the material composition and the configuration of the sand layer is more or less extensive.

When using classical empirical calculation models (Bligh, Lane, see chapter 4), only a qualitative indication of the material composition is needed: sand, gravel, silt content and rough indications of the median grain diameter.

When using the advanced calculation method (Sellmeijer, see chapter 4) explicit estimates are needed of:

- the permeability of the sand layer;

- the grain distribution;
- the thickness of the sand layer and its course under and beside the defence.

In addition, specific parameter indications are needed for this calculation method, namely the dragforce factor and the rolling resistance angle. These parameters cannot be determined by simple monitoring. In the calculation model nominally prescribed values are used, determined on the basis of laboratory tests to verify the calculation model among others.

3.7.3. Heave mechanism

Decisive for this mechanism is the vertical groundwater flow gradient on the inside of the defence. The geo-hydraulic configuration of flood defence, package, screens in the package and the preconditions of groundwater flow are accordingly important. The important soil parameters are the same as for the crack mechanism.

3.7.4 Seepage erosion mechanism; analysis of normative seepage line

Important here, besides the geometric composition of hydraulic structure and connection to the surrounding ground alongside and under the hydraulic structure, are the location and dimensions of anti-seepage vertical and horizontal screens and details of connections between these screens and of the connection between the screens and the hydraulic structure.

4 Calculation Models and Calculation Rules

Various calculation models and calculation rules are available for safety checking with respect to cracking, piping and heave. They are described in this chapter. In order of complexity they are

- **simple (empirical) calculation rules.** These are classical calculation rules which have been used for decades. One general characteristic is that they are simple and that few parameters are needed, but also that they are relatively conservative. They are used as the first (rough) design check or safety test of new and existing flood defences. When it is shown that the safety of the design or the existing structure is sufficiently guaranteed according to these rules, then in principle the check can be rounded off.
- **complex calculation models.** These are generally calculation models and rules which have become available in more recent years through targeted studies. In general these models and calculation rules are more complex and more parameters are needed for a precise calculation. As a result of this the parameters which have a strong influence on the result of the calculation can be better estimated and it can be determined whether it is worthwhile defining a soil test more accurately or, in so far as they are design variables, adjusting them in the right direction. This leads to better defined tests and, dependent on the situation, more sophisticated designs.

The calculation models and calculation rules for cracking, piping and heave are described one by one. The influences of foreland and of the time-dependency of the outside water level are then addressed. This chapter concludes with an overview of the most common computer programs in this field in the Netherlands.

4.1 Cracking

4.1.1 Equilibrium consideration

The point of departure for the equilibrium consideration in the crack analysis is illustrated in figure 4.1. The illustration shows the water-bearing sand layer connected to the outside water (river or sea) with the hardly-permeable covering layer or package of clay and peat above it. Cracking will occur when the outside water level is high and the water pressure in the sand layer is so great that the covering layer starts to uplift. The water pressure in the sand layer can never be greater than the weight of the covering layer; the potential in the sand layer where equilibrium is present is called the potential limit. This potential limit is a precondition of the groundwater flow pattern in the sand layer. If the outside water level rises further the effect will be that the area in which the potential limit is prevalent will expand inwardly. The area, the uplift zone, is important for stability checks in the case of uplift (see *Guide on River Dikes* (part 2), [TAW 1989].

It is assumed that channels form in the uplift zone as a result of the crack forming, leading to seepage on the surface, so that (sand) boils can originate. This phenomenon is called cracking or hydraulic soil failure. It is accordingly important to check whether the potential limit in the sand layer is reached at the normative outside water level.

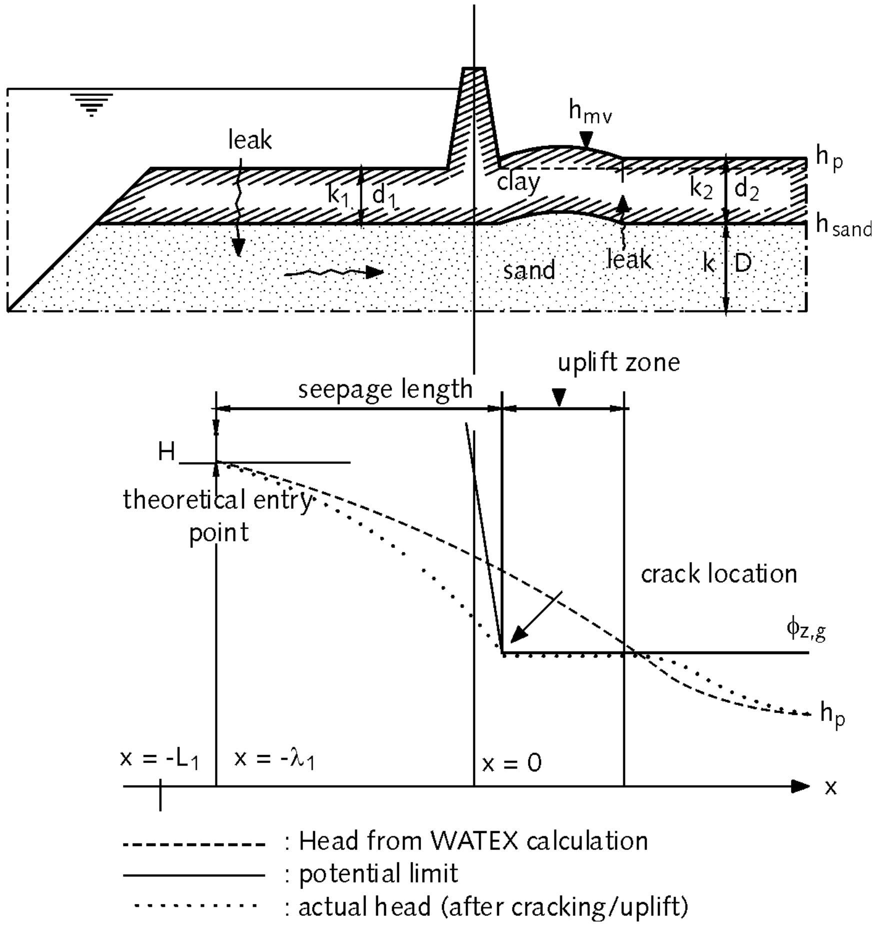


Figure 4.1 (a) Configuration dike with foreland and substrate
(b) Head in sand layer, potential limit, crack location and uplift zone
leak
clay
sand

uplift zone
seepage length
theoretical entry point
crack location
Head from WATEX calculation
potential limit
actual head (after cracking/uplift)

4.1.2 Calculation of potential limit or head limit

The potential limit is calculated as follows. On the surface $h+h_{sand}$, the top side of the sand layer in figure 4.1, the weight of the covering soil layer and of the (ground) water work in a downwards direction. The water pressure from the sand works in an upwards direction. The equilibrium limit is reached if

$$(\phi_{z,g} - h_{zand})\gamma_w = (h_{mv} - h_p)\gamma_{dr} + (h_p - h_{zand})\gamma_{nat} \qquad 1$$

Eq. 1

If $h_p < h_{mv}$ (phreatic water level in the covering layer) and

Eq. 2

$$(\phi_{z,g} - h_{zand})\gamma_w = (h_p - h_{mv})\gamma_w + (h_{mv} - h_{zand})\gamma_{nat} \qquad 2$$

If $h_p > h_{mv}$ (water level above the ground surface).

In these formulas the values are as follows.

- $\phi_{z,g}$ de grensstijghoogte of grenspotentiaal [m ± NAP]
- h_{zand} het niveau van de bovenkant van de watervoerende (zand)laag [m ± NAP]
- h_{mv} het maaiveldniveau [m ± NAP]
- h_p de freatische stijghoogte in de afdekkende laag of de waterstand boven het maaiveld [m ± NAP]
- γ_w het volumegewicht van water [kN/m^3]
- γ_{dr} het droge volumegewicht [kN/m^3] en
- γ_{nat} het natte volumegewicht van de afdekkende grondlaag [kN/m^3].

- $Ø_{z,g}$ the head limit or potential limit [m ± NAP]
- h_{sand} the level of the top side of the water-bearing (sand) layer [m ± NAP]
- h_{mv} the ground surface level [m ± NAP]
- h_p the phreatic head in the covering layer or the water level above the ground surface [m ± NAP]
- γ_w the volume weight of water [kN/m3]
- γ_{dr} the dry volume weight [kN/m3] and
- γ_{nat} the wet volume weight of the covering ground layer [kN/m3].

Note that formula 1 indicates the head limit in initial state in a situation in which $h_p < h_{mv}$ (so a real groundwater level) As soon as a situation is created in which the actual head in the sand exceeds the water table head in the covering layer a flow will originate in an upwards direction. As a result the groundwater level rises to the ground surface.

For both cases the potential limit is expressed as follows

$$\phi_{z,g} = h_p + d\,\frac{\gamma_{nat} - \gamma_w}{\gamma_w} \qquad 3$$

Eq. 3

in which h_p is the level of the free water surface on the inside of the flood defence, if present (for example ditch level) or the ground surface level. *d* is the thickness of the top layer. If the covering package consists of several ground layers the second part of the right-hand term in this formula is replaced by the summation on the various ground layers of the product of layer thickness and volume weight.

4.1.3. Occurring potential

The potential in the sand layer is dependent on the outside water level *H*, and the geo-hydraulic configuration and qualities of the substrate (see figure 4.1). The presence of foreland, the thickness and the permeability of the covering layers in foreland and hinterland and the permeability and thickness of the sand layer are decisive for the degree to which damping occurs of the outside water level. One way of calculating the potential in the sand layer is with the WATEX computer program developed by TAW. Guiding principles in such a calculation with that program are

- horizontal (Darcy) flow of the groundwater in the sand layer;
- vertical flow (leak) through the covering layers; at the site of the foreland the flow is directed from the outside water to the sand layer, in the hinterland from the sand layer to the ground surface;
- preconditions: at the site of entry the head in the sand is equal to the outside water level *H*. Far away in an inward direction the head in the sand layer is equal to the polder level h_p.

These guiding principles result in a system of differential equations for the stationary head $Ø_z$ in the sand layer (see appendix I).

In the WATEX program this system of differential equations is solved. The result is a site-dependent head in the sand layer as indicated by a dashed line in figure 4.1b. The course of the head of $Ø_z = H$ to the head in x=O and subsequently from here to $Ø_z = h_p$ is dependent on the distribution ranges

$$\lambda_1 = \sqrt{\frac{k_z D d_1}{k_1}} \qquad en \qquad \lambda_2 = \sqrt{\frac{k_z D d_2}{k_2}} \tag{4}$$

Eq. 4

where k_1 and d_1 are the (vertical) permeability [m/s] and thickness of the covering layer in the foreland respectively, and k_2 and d_2 are the (vertical) permeability and thickness of the covering layer in the hinterland. k_Z and D are the horizontal permeability and the thickness of the sand layer.

The WATEX program also offers the possibility of calculating the head in the sand layer as time-dependent response on the development of the outside water level. In principle the time-dependent approach provides a somewhat lower head and so a somewhat milder crack criterion (see section 4.1.4) for an outside water level course in which the time that the highest outside water level maintains is too short for the creation of a full stationary groundwater flow in the sand layer and the covering clay layers. For this time-dependent calculation, besides the permeability and thickness of the sand layer and covering clay layers, information is also needed about the elastic storage of groundwater in the sand layer and about the consolidation coefficients of the clay layers.

4.1.4 Crack criterion; exit point for piping checks

When the head in the sand layer is universally smaller than the calculated potential limit then uplift and cracking is out of the question. To shield against uncertainties with reference to the parameters in the calculation a safety coefficient is applied. The crack criterion is therefore

If for each x in figure 4.1

$$(\phi_z - h_p) \leq \frac{1}{\gamma}(\phi_{z,g} - h_p) \tag{5}$$

Eq. 5

With safety factor γ, then the safeguard against cracking is sufficiently ensured. The calculation of $ø_z$ is based on design water levels and best guesses of the geo-hydraulic parameters. The calculation of $ø_{z,g}$ is based on an estimation of the minimal thickness of the covering package (for example the characteristic 5% minimum limit) and a characteristic estimate (5% minimum limit) of the average volume weight. See appendix II for characteristic estimates.

The value of the safety factor γ to be maintained depends on the situation. The greater the uncertainty with reference to the occurring potential the higher the safety factor in general. Sometimes very conservative assumptions are used as a first estimate of the occurring potential; in those cases a relatively low safety factor can suffice. As indications for the safety factor to be applied are as follows.

1 In a safety test (see also section 5.2.3)

When the occurring potential $ø_z$ at normative outside water level is only calculated with the help of a geo-hydraulic model (see appendix I), uncertainties in the selection of parameters can play an important role. It is recommended to use a safety factor $\gamma = 1.20$.

2 In the design of piping berm to prevent cracking (see also section 5.4.4)

The berm length needed is determined by the seepage length needed according to Bligh or Sellmeijer. The crack point is shifted to the toe of the new berm, where the potential limit is prevalent in the sand layer. This is a precondition of the course of the head in the sand layer under the berm. As the potential limit can be determined accurately the head course under the berm can also be determined fairly accurately. As a result, a safety factor of γ = 1.20 can suffice for the dimensioning of the thickness of the berm, in accordance with NEN 6740.

In the *Guide on Structural Designing* [TAW 1994[2]] a safety factor of γ = 1.05 is even mentioned; it is recommended to ascertain that the uncertainty of the estimated occurring potential is small or that conservative estimates are used.

If the crack criterion is met in a safety test then further checking for piping is unnecessary. If it is not met then there will have to be a further check for piping. The normative crack location must be determined to calculate the seepage length present for piping. The normative crack location is the point nearest the dike behind the dike, where the crack criterion is not met. That crack location is shown in figure 4.1.

4.2 Calculation Rules and Piping Checks

4.2.1 Entry and exit points for seepage

The check of the piping mechanism is about determining whether the safeguards against erosion of the sand layer (or sand layers) in the substrate are sufficiently guaranteed in the given expected maximum occurring hydraulic head over the flood defence. The resistance to erosion, besides the qualities of the sand layer, chiefly depend on the seepage length. This is the distance between the entry point of the seepage flow through the sand layer and the exit point.

The exit point is often fixed naturally, for example when the berm ditch behind a dike extends up to the sand layer in question, or when the sand layer extends to the ground surface. If a covering layer is present which is susceptible to cracking (see section 4.1) then the crack location is taken as the exit point. The entry point is often more difficult to determine. For main dikes the point where the outside slope cuts the water-bearing sand layer can be taken as the entry point. For dikes with a long foreland, as a rule part of the foreland can be included in the calculation to determine the seepage length; we then have a theoretical entry point. This is detailed further in section 4.4.

For hydraulic structures the situation is analogous; more than for dikes a natural entry and exit point can be indicated here.

4.2.2 Bligh's empirical calculation rules

Bligh and Lane's empirical calculation rules (see section 4.3.3) are in principle intended for checking piping (horizontal erosion) and heave (vertical exit gradient). In these rules no distinction is made between the two different mechanisms; that is the case in more complex calculation rules (see sections 4.2.4 and 4.3).

A number of cases of collapses of small flood-control dams due to piping have been catalogued and analysed by Bligh [Bligh 1910]. On the basis of this catalogue the following empirical calculation rules were derived by him:

$$\Delta H \leq \Delta H_c = \frac{L}{C_{creep}}$$

Eq. 6

The values here are

- ΔH the hydraulic head over the flood defence ($=H-h_p$)
- ΔH_c the maximum permissible gradient
- L the minimum seepage length
- C_{creep} 'creep' factor

The creep factor is dependent on a qualitative characterisation of the material in the ground layer tested for piping. The indications provided by Bligh for these factors are given in table 4.1.

The seepage length is in principle the length of what Bligh calls the line of creep. This is the line from entry point to exit point (or collection behind interconnected line sections), where the limit is marked between the underside of the flood defence and the soil package through which the groundwater flows.

In the opinion of Bligh any vertical sections (vertical walls of a structure or vertical seepage lines along a screen) must be fully included in the calculation. Lane (see following section) strongly criticises this approach; he proposed that the vertical parts of the seepage length contribute more to the resistance in relative terms than the horizontal parts. In Dutch design practice for dikes the rule of Bligh was recommended in situations where only the horizontal seepage line was present (the use of cut-off walls at dikes was not common), implicitly underscoring Lane’s criticism. An exception was made for the vertical part of the seepage line in the crack channel. This will be addressed further later in this section.

The hydraulic head *H* is equal to the difference between outside water level (the design level, DWL) at sea dikes and the normative high water level (NHW) and the water level inside the dike at the site of the exit point, bearing in mind the rise in sea level et cetera (see section 3.7.2). If at the site of the exit point or the crack location no free water level is prevalent the ground surface level can be used for the calculation, taking into consideration any ground surface subsidence (see section 3.7.2).

In the past few years, to calculate the hydraulic head it has been usual to take the resistance into consideration in the crack channel from the sand layer to the ground surface or ditch bed for river dikes in the west of the Netherlands. In laboratory tests the potential hydraulic head is measured over a column of sand (in a round pipe), which has been brought to a state of fluidisation by vertical groundwater flow [Sellmeijer 1981]. Various tests were conducted, with a column of sand in a smooth pipe, in a pipe with a clay coating and in a pipe with a sand coating. Various pipe diameters were also studied. The measurements show that the potential hydraulic head over the fluidised sand column in all cases is approximately 0.6 times the height of the sand column. This hydraulic head will often occur in a piping situation where the eroded sand from the sand layer via a crack channel must be carried to the ground surface. For the piping mechanism the potential hydraulic head between outside water and exit point in the sand layer (so at the bottom of the crack channel) is normative. This hydraulic head is equal to the total hydraulic head over the flood defence minus the hydraulic head over the crack channel. If we take the hydraulic head over the crack channel into consideration and apply to it a safety factor of around 2, then the checking rule is
Eq. 7

7

$$(\Delta H - 0.3d) \leq \Delta H_c = \frac{L}{C_{creep}}$$

Here *d* is the vertical distance between the topside of the sand layer and the ground surface or the ditch bed.

In applying the rule of Bligh no safety factor should be calculated; the safety needed is allowed for in the calculation rule itself.

Table 4.1 Seepage line factors for the rules of Bligh and Lane

Grondsoort	**Mediane korrel-diameter [μ m]** [1)]	**C_{creep} (Bligh)**	**$C_{w,creep}$ (Lane)**
Uiterst fijn zand, silt	< 105		8.5
Zeer fijn zand	105 – 150	18	
Zeer fijn zand (mica)		18	7
Matig fijn zand (kwarts)	150 – 210	15	7
Matig grof zand	210 – 300		6
Zeer/uiterst grof zand	300 – 2000	12	5
Fijn grind	2000 – 5600	9	4
Matig grof grind	5600 – 16000		3.5
Zeer grof grind	> 16000	4	3

1) indicaties conform NEN 5104 (September 1989)

Eventueel kan tussen de klassemiddens van de mediane korreldiameters worden geïnterpoleerd

Soil type
Median grain diameter [μ m] [1)]
C_{creep} Bligh
C_{creep} Lane

Extremely fine sand
Very fine sand
Very fine sand (mica)
Moderately fine sand (quartz)
Moderately coarse sand
Very/extremely coarse sand
Fine shingle
Moderately coarse shingle
Very coarse shingle

[1)] indications in accordance with NEN 5104 (September 1989)
Interpolations can be made between the midmarks of the median grain diameters.

4.2.3 Empirical calculation rule of Lane

Hydraulic structures are one place where one or more anti-seepage vertical screens can be installed to extend the seepage line (see figure 4.2). The following calculation rule was drawn up by Lane on the basis of a survey of structures where the seepage length was partly made up of vertical sections

$$\Delta H \leq \Delta H_c = \frac{(\frac{1}{3} L_h + L_v)}{C_{w,creep}} \qquad 8$$

Eq. 8

Here L_h is the total length of the horizontal parts of the seepage line and L_v the total length of the vertical parts of the seepage line, the seepage length. Figure 4.2 shows the seepage line which must be used for calculation. Note that the vertical seepage line along the screen is equal to twice the length of the screen. The factor $C_{w,creep}$ is called the weighted seepage line factor; in table 4.1 the values given by Lane for various types of material in the ground layer.

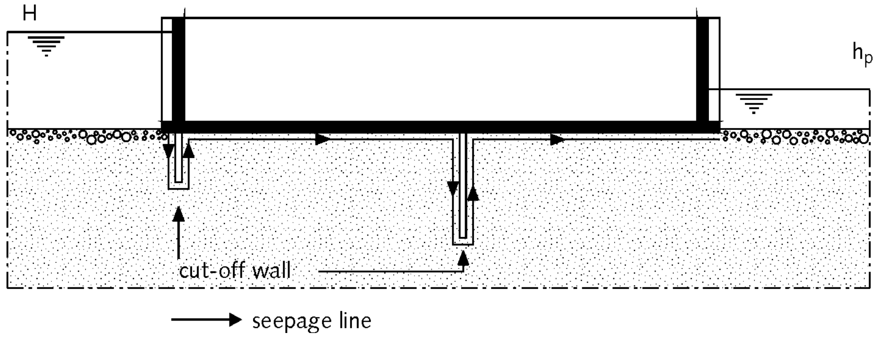


Figure 4.2 Seepage line at water-retaining hydraulic structure with cut-off walls
cut-off wall
seepage line

Note that the vertical seepage line is not even explicitly included in the calculation in the rule of Lane, as it is in the rule of Bligh, it is already included in the vertical seepage line parts after all.

Especially at hydraulic structures the normative seepage line need not lie in one (vertical or horizontal) plane. It is important to find the shortest possible seepage line. The vertical and horizontal seepage lengths are found by summation of the vertical and horizontal seepage line components respectively.

Remark 1
The use of a vertical cut-off wall is also being considered at dikes nowadays. Cut-off walls on the downstream side of the dike are generally the most effective, because a heave situation is created. To allow fluidised sand to flow out vertically a relatively large part of the total hydraulic head over the flood defence is necessary. In that way, a greater critical hydraulic head is generally obtained over the flood defence than at the horizontal outflow at the site of the exit point. At cut-off walls on the upstream side of the dike only the flow resistance is enlarged. Long cut-off walls are necessary to ensure effectiveness.

Remark 2
Including the horizontal seepage length in the calculation is only permitted when a good connection between the underside of the structure and the substrate is guaranteed. This will generally be the case at hydraulic structures founded and built on natural soil subsoil. At hydraulic structures founded on piles settlement of the substrate must always be taken into consideration, as result of which space can be created between the structure and the ground. It is therefore recommended that the horizontal seepage length be set at zero in that case. At structures on tension piles a good connection cannot be relied on either. At structures sunk into the sand, for example tunnel elements which extend under the flood defence, a good connection (at all points) cannot be relied on either. Also here the horizontal seepage line is not included in the calculation.

Remark 3
In [NEN 3651] and [NPR 3659] guidelines are given for piping checks at points where flood defences cross pipes, especially in straticulated ground. The following instructions are derived from this.

When the seepage line runs through various ground layers, for Lane's weighted seepage line factor the value must be selected which belongs to the soil layer in which the exit point is situated. Moreover, when parts of the seepage line run through soil layers with other permeability values than that of the normative soil layer, the (fictive) contribution of these parts to the (horizontal or vertical) seepage length can be calculated by decreasing these parts by the proportion of the permeability. The authors of this Technical Report are also of the opinion that a justification of this refinement of the calculation rule cannot be derived from the foundations and the philosophy of Lane's method, as set out in his original publication.

Remark 4
In monitoring practice in situations with vertically exiting seepage, for example behind a cut-off wall, the rule of Lane is also used as the first rough check for heave. Here the assumption is that this rule is

always more conservative than an explicit check for heave, for example with the fragments method (section 4.3). The supposition has been tested against a (limited) number of comparable calculations [Calle 1998]. The test found that the supposition is correct in the majority of the cases considered, but that there are exceptions to the rule. Most notably in situations with several cut-off walls where the screen at the exit point was relatively short in relation to the other screen, the fragments method produced a normative heave check. In practice those situations will not occur very often, but they cannot be excluded. This must be taken into consideration in the choice of the calculation rule.

4.2.4 Sellmeijer's calculation model
In the scope of TAW research, Sellmeijer [Sellmeijer 1989] developed a mathematical calculation model for piping checks, on the basis of observations of the phenomenon, as described in chapter 2. Guiding principle of the mathematical model is the configuration sketched in figure 4.3. The mathematical model consists of a linking of

1 The potential equation for the description of groundwater flow in the sand layer. Preconditions are:
- the potential $ø=H$ on the topside of the sand layer on the upstream side of the structure;
- an impermeable edge at the site of the underside of the structure;
- the potential $ø=h_p$ on the topside of the sand on the downstream side of the structure;
- at the site of the fissure the potential in the sand is equal to the potential of the water in the fissure.

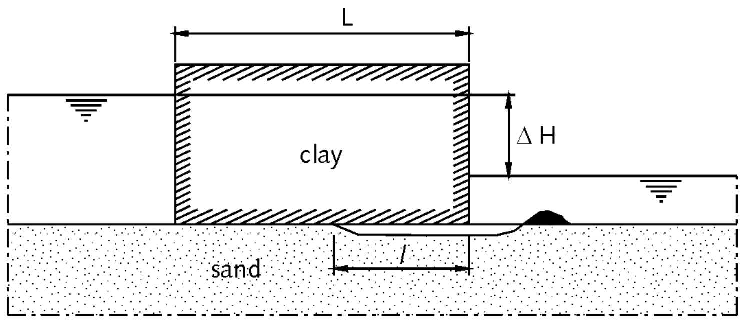
2 An equation for laminar flow of the water through the fissure.
3 An equilibrium equation of drag force and dragforce by the flow in the fissure on the sand grains and the rolling resistance of these grains.

With the help of these equations the maximum hydraulic head over the structure can be calculated at which point the sand grains are still in equilibrium. This hydraulic head is dependent on the relationship I/L between the length of the pipe and the length of the structure, the permeability of the sand, the dragforce coefficient and the diameter and the rolling resistance of the sand grains. It appears that the configuration illustrated in figure 4.3 in the case of the relationship $I/L \approx 0.5$ the hydraulic head at the point equilibrium first found is the greatest. This maximum hydraulic head is called the critical head.

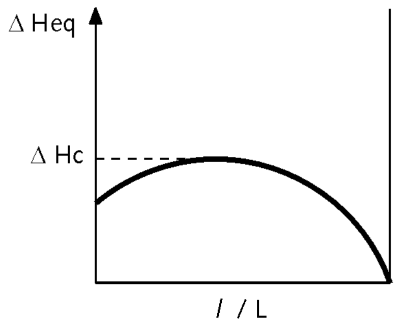
The interpretation of this outcome is as follows. For a hydraulic head over the structure which is smaller than the critical hydraulic head a fissure will originate due to erosion which will grow until the fissure length corresponding to this hydraulic head is reached. The flow gradients are then weakened to such a degree that the sand grains on the edge of the fissure can offer no further resistance to the drag forces. This is shown in figure 4.4. If the hydraulic head is increased then the fissure will start to grow again until a new equilibrium is achieved. The erosion process stops as long as the hydraulic head is no larger than the critical fall. If the hydraulic head does become greater, then the fissure will continue to grow, because the hydraulic head by which equilibrium is possible is smaller than the hydraulic head present. The fissure grows into an open channel between the upstream and downstream sides of the structure; piping is then a fact. By observing tests on a small scale it is shown that the growth of a fissure past the critical point occurs very quickly.

Sellmeijer used this calculation model to conduct a great many numeric calculations of the critical hydraulic head for various combinations of the parameters which play a role. Via accurate curve fitting to these calculation results an approximate analytical formula is derived.

This formula was validated by a large-scale Delft Hydraulics model test in the Delta channel [Silvis 1991].



(a)



(b)

Figure 4.3 (a) Basic configuration Sellmeijer’s calculation model [Sellmeijer 1989]
(b) Equilibrium hydraulic head ?Heq as function of I/L, critical hydraulic head ?Hc
clay
sand

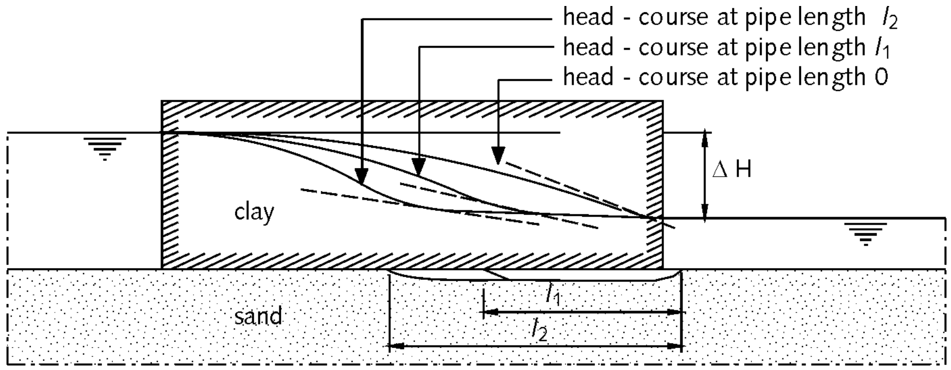


Figure 4.4 Weakening of flow gradients as piping is created
head - course at pipe length /2
head - course at pipe length /1
head - course at pipe length 0

In a follow-up study [Sellmeijer et al 1989] a similar calculation exercise was carried out for the substrate configuration given in figure 4.5. The following approximate formulas were derived

$$\Delta H_c = \alpha \, c \, \frac{\gamma_p}{\gamma_w} \, tan(\theta)\,(0.68 - 0.10 \, ln(c)) \, L \qquad 9$$

Eq. 9
where

$$\alpha = (\frac{D}{L})^{(\frac{0.28}{(\frac{D}{L})^{2.8} - 1})} \qquad 10$$

Eq. 10
and

$$c = \eta \, d_{70} \, (\frac{1}{\kappa \, L})^{\frac{1}{3}} \qquad 11$$

Eq. 11

The following values are given in these formulas
- ΔH_c the critical hydraulic head over the flood defence
- γ_w the volume weight of water [kN/m^3]
- γ_p the (apparent) volume weight of sand grains under water [= 17 kN/m^3]
- θ the rolling resistance angle of the sand grains [^{0}]
- η the dragforce factor (coefficient of White) [-]
- κ the intrinsic permeability of the sand layer [m^2]
- d_{70} 70 per cent value of the grain distribution [m]
- D the thickness of the sand layer
- L the length of the seepage line (measured horizontally) [m]

In [TAW 1994] a practical calculation formula on the basis of this formula was developed for design and monitoring, especially with respect to the manner in which uncertainties in the selection of parameters can be calculated. The recommended selection of the parameters is summarised in table 4.2.

The intrinsic permeability κ [m^2] can be derived from estimates of the permeability [m/s] of the sand layer. The relationship is:

$$\kappa = \frac{\nu}{g} k = 1.35 \, 10^{-7} \, k \qquad 12$$

Eq. 12

where ν is the kinematic viscosity (˜ 1.33 10^{-6} m^2/s for groundwater at 10^o Celsius*)* and g the acceleration of gravity (˜ 9.81 m^2/s). The permeability can be estimated or measured in various ways. In [TAW 1994] a procedure is indicated to calculate the permeability using sieve analyses of the sand in the sand layer (see also appendix II). It is also conceivable that estimates of the permeability are obtained using tests on site (pump test, falling head tests, monopoly measurements etc). Obtaining reliable estimates of the permeability is a difficult problem, in which the effects of heterogeneity of the sand layer are troublesome. In the piping formula a conservative estimate (high representative value) of the permeability must be calculated.

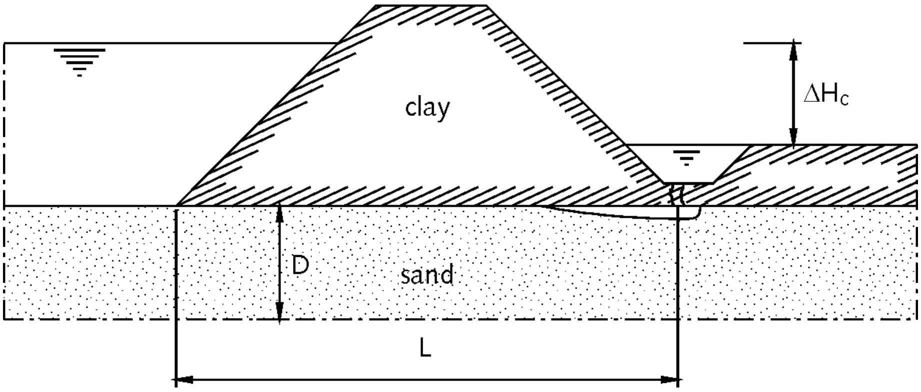


Figure 4.5 Basic configuration Sellmeijer's calculation model [Sellmeijer et all 1989]

Table 4.2 Parameter selection in design/monitoring using Sellmeijer's formula

Representatieve parameterkeuze in formule van Sellmeijer			
Parameter	omschrijving	type repr. waarde 1)	Opmerking/ default Vc 2)
Aanwezig verval: *ΔH*	maatgevend verval	MHW – slootpeil, of MHW – maaiveld [m]	
Berekening kritiek verval δH			
L	kwelweglengte	l.r.w. [m]	Vc=0.10 3)
D	dikte zandlaag	h.r.w. [m]	Vc=0.10
θ	rolweerstandshoek	nom: 41^o 3)	
η	sleepkrachtfactor	nom: 0.25 3)	
γ_p	vol.gew. korrels o.w.	nom: 17 kN/m^3	
γ_w	vol.gew. water	nom: 10 kN/m^3	
κ	intrinsieke doorlatendh.	h.r.w. [m^2]	zie tekst
d_{70}	70 percentiel ϕ-zand	l.r.w. [m]	0.25
Pipingcriterium: $(\Delta H - 0.3\ d) \le \frac{1}{\gamma} \Delta H_c$ met: veiligheidsfactor γ = 1.20 en *d* = lengte opbarstkanaal [m]			
Representatieve of karakteristieke waarden (zie ook Appendix II): $h.r.w. \approx \mu\ (1 + t_{N-1}^{0.95}\ Vc\ )$ $l.r.w. \approx \mu\ (1 - t_{N-1}^{0.95}\ Vc\ )$ μ = gemiddelde waarde uit steekproef, of 'best guess' *Vc* = variatiecoëfficiënt uit steekproef of default variatiecoëfficiënt uit deze tabel $t_{N-1}^{0.95}$ = Student t-factor (indien geen steekproef beschikbaar: 1.65)			
Noten: 1) : l.r.w. = lage representatieve waarde (95 % ondergrens) h.r.w. = hoge representatieve waarde (95 % bovengrens) nom. = nominale (voorgeschreven) rekenwaarde 2) : Default variatiecoëfficiënt te gebruiken indien geen steekproef voorhanden 3) : nominale waarden zijn mede bepaald aan de hand van Validatieproef in Deltagoot			

Representative parameter selection in Sellmeijer's formula

Parameter
description
type repr. value [1)]
remark/default Vc [2)]

Hydraulic head present: ?H
normative fall
NHW - ditch level, or NHW- ground surface [m]

Calculation critical hydraulic head δ

L	seepage length	l.r.w. [m]	Vc=0.10 [3)]
D	thickness sand layer	h.r.w [m]	Vc=0.10
θ	rolling resistance angle	nom: 41^o	
η	dragforce factor	nom: 0.25	
γ_p	vol. weight grains u.w.	nom: $17kN/m^3$	
γ_w	vol. weight water	nom: $10kN/m^3$	
κ	intrinsic permeability	h.r.w $[m^2]$	see text
d_{70}	70 per cent ø-sand	l.r.w [m]	0,25

Piping criterion

...

with: safety factor γ = 1.20 and d = length crack channel [m]

Representative or characteristic values (see also appendix II):

h.r.w. ˜ μ *(1 + tN-1^ 0.95 Vc)*
l.r.w. ˜ μ *(1 + tN-1^ 0.95 Vc)*
μ = average value from random test, or best guess
Vc = variation coefficient from random test or default variation coefficient from this table
tN-1 0.95 = student t-factor (if no random test is available: 1.65)

Notes:
1): l.r.w. = low representative value (95% minimum level)
h.r.w. = high representative value (95% maximum level)
nom. = nominal (prescribed) calculation value
2): Default variation coefficient to be used if no random tests are available
3): nominal values are determined in part via validation tests in Delta channel

4.3 Calculation Models for the Heave Mechanism

4.3.1 Heave criterion

In case of vertical groundwater flow in sandy ground behind a cut-off wall on the inside of a flood defence, a vertical hydraulic gradient is established so that the effective stresses in the soil decrease. If the gradient is strong the effective stresses all along the vertical can reduce to zero; the soil is then in a fluidised state (quicksand situation).

A configuration is given in figure 4.6 in which this mechanism, which is called heave, is conceivable. The calculation of the hydraulic gradient by which heave occurs is analogous to the calculation of the potential limit for covering soil layers for checking for cracks.

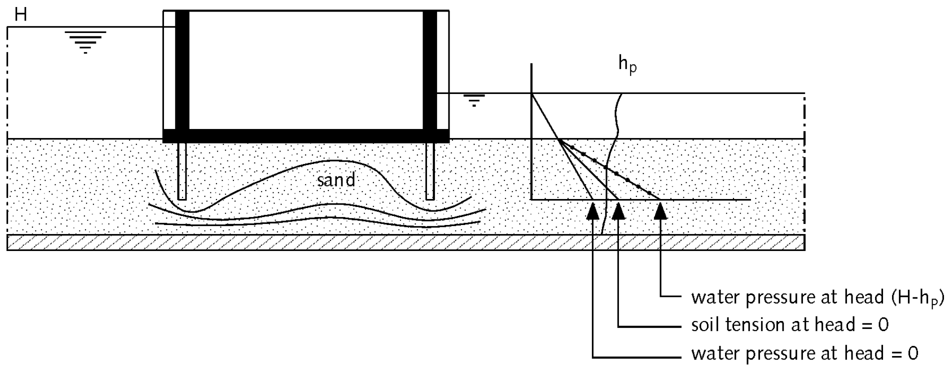


Figure 4.6 Heave situation
water pressure at hydraulic head (H-h_P)
soil tension at hydraulic head =0
water pressure at hydraulic head = 0

To guarantee sufficient safeguarding against heave the maximum occurring hydraulic gradient must be smaller than the hydraulic gradient by which heave occurs.. The hydraulic gradient by which heave occurs, the critical hydraulic gradient i_c, is

$$i_c = \frac{\phi_o - h_P}{d} = \frac{\gamma'}{\gamma_w} = \frac{(1-n)(\gamma_k - \gamma_w)}{\gamma_w} \qquad 13$$

Eq. 13

In which the values are

- γ' the submerged volume weight of the soil [kN/m^3]
- γ_w the volume weight of the (ground) water [kN/m^3]
- $ø_o$ the head at the site of the underside of the vertical wall [m with respect to reference]
- h_p the polder level (free water level or ground surface) [m with respect to reference]
- n the pore factor [-] in the sand layer and
- γ_k the volume weight of the grain material [= 27km/m^3]

If the soil is composed of layers the submerged volume weight can be calculated by layer-by-layer summation.

$$\gamma' = \frac{1}{d} \sum_i d_i \gamma'_i \qquad 14$$

Eq. 14

The critical hydraulic gradient in sand varies depending on the porosity between 0.85 and 1.15. As a rule a safe permissible value is maintained: i_{toel} = 0.5. The heave criterion is therefore

$$i_{optr} = \left(\frac{\phi_0 - h_p}{d}\right)_{optr} \leq i_{toel} \qquad (15)$$

Eq. 15

where i_{optr} is the occurring hydraulic gradient.

4.3.2 Calculation of the occurring hydraulic gradient

The occurring vertical hydraulic gradient on the inside of the flood defence is dependent on the total hydraulic gradient over the flood defence and the groundwater flow pattern in the sand layer and accordingly on the geo-hydraulic configuration of that layer and the place and the depth of the cut-off wall. A groundwater flow analysis is needed to determine the occurring hydraulic gradient. In principal this can be achieved with any suitable calculation model. We distinguish

- a calculation with the help of a (multipurpose) computer program for numeric groundwater flow analysis, based on a finite element method or finite differential method (FEM or FDM).
- calculation of a semi-analytical calculation model (Fragment method). This methodology is developed within the scope of TAW, specifically for heave checks at dikes or water-retaining hydraulic structures with vertical cut-off walls.

The rule of Lane has traditionally been used in practice as a simple calculation rule for checks for heave mechanisms. In a number of TAW guides this rule is recommended as first rough test; the underlying idea is that this rule is sufficiently safe under all circumstances. Although that idea has never been provided with a scientific foundation there are no practical indications which prove the opposite. The above-mentioned calculation techniques mean that a more scientifically well-founded heave checking instrument has been made available that is preferable in the opinion of the compilers of this Technical Report.

The use of general numeric groundwater flow software to determine the occurring vertical hydraulic gradient on the inside of the flood defence is chiefly a matter of course. Specific areas for attention in modelling the substrate configuration are abrupt transitions where the flow direction or the flow velocity changes dramatically, such as the flow around the underside of a cut-off wall. Depending on the type of program, numeric accuracy problems can occur which can be negated by local refinement of the element mesh or the differential scheme. Consult the program manual for more information.

In the following section the principal of the fragments method is discussed.

4.3.3 The fragments method

A calculation technique was commissioned by TAW, specially to analyse the groundwater flow under dikes or hydraulic structures with cut-off walls. It is a semi-analytical technique that has the added advantage over numeric groundwater flow analyses that very fast calculations can be made. The principal is described below; for details refer to [Van de Paverd 1994], [Sellmeijer 1995] and [Sellmeijer 1997] among others. The basic idea for the methodology is of less recent origin [Pavlovsky 1956].

Consider the configuration of a flood defence, in this case a hydraulic structure, with two cut-off walls, in figure 4.7. In the figure the preconditions for groundwater flow though the sand layer under the structure are given, namely the head equal to the outside water level on the surface of the sand layer upstream and the head equal to the inside water level downstream. The total configuration with cut-off walls is too complex for a direct analytical solution of the groundwater flow problem. That is why in this case the whole is divided into three parts, a head fragment (fragment 1), a middle fragment (fragment 2) and a tail fragment (fragment 3). The divisions between the fragments are vertical lines in the next part of the cut-off walls. The crux of the method is that these lines are approximately equipotential lines (lines with a constant head). The potential on the dividing line between fragment 1 and fragment 2 can therefore be characterised by one value, $ø_{1,2}$, which however is as yet unknown. And also the potential on the dividing line between fragment 2 and fragment 3, $ø_{2,3}$.

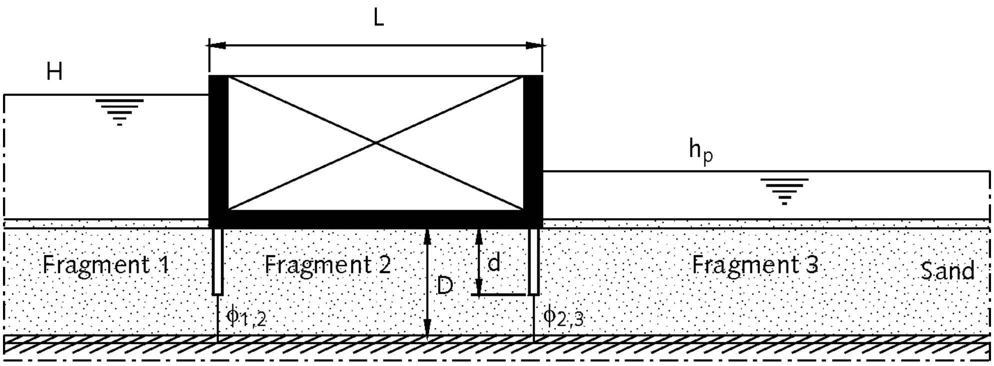


Figure 4.7 Division in fragments
Fragment 1
Fragment 2
Fragment 3
Sand

Subsequently the fragments can be considered individually. The head fragment is illustrated in figure 4.8 (a) and the preconditions for the flow within this fragment indicated. The groundwater flow problem can be solved with the help of techniques based on the complex function theory, when a specific value for the potential $ø_{1,2}$ is assumed. With the given permeability of the soil within this fragment the capacity that flows through that fragment (that is: enters along the top edge and leaves the fragment along the dividing line) can accordingly be determined. That capacity is indicated with a Q. We know that this capacity is directly proportional to the permeability k_1 and to the (assumed) hydraulic gradient (H-$ø_{1,2}$) hydraulic head over the fragment. With the help of the calculated Q the quotient is determined.
Eq. 16

$$W_1 = \frac{k_1 (H - \phi_{1,2})}{Q} \qquad 16$$

Due to the proportionality mentioned this quotient is independent of the selection of k_1 and $ø_{1,2}$ and is called the resistance factor of fragment 1.

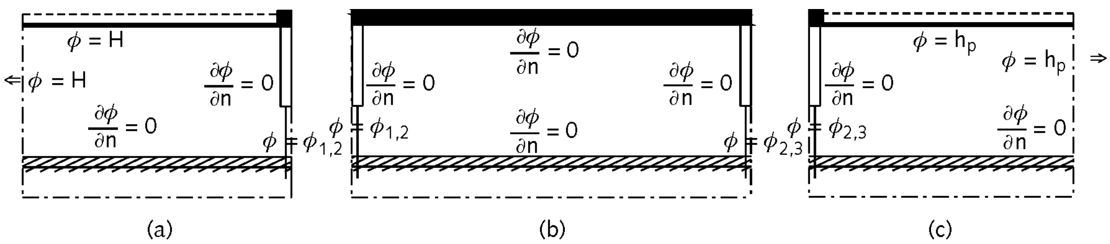


Figure 4.8 (a) head fragment, (b) middle fragment and (c) tail fragment

By analogy the resistance factors W2 and W3 in fragments 2 and 3 can also be determined (figure 4.8b and c). To calculate the potential division we make use of the fact that the through flow capacity through each of the fragments must be equal, so
Eq. 17

$$\frac{-k_1(\phi_{1,2}-H)}{W_1}=\frac{-k_2(\phi_{2,3}-\phi_{1,2})}{W_2}$$

17

$$\frac{-k_2(\phi_{2,3}-\phi_{1,2})}{W_2}=\frac{-k_3(h_p-\phi_{2,3})}{W_3}$$

This linking of the fragments provides two linear algebraic equations with which $ø_{1,2}$ and $ø_{2,3}$ can be solved.

It is not difficult to guess that the methodology can be extended without any adjustment to any number of fragments linked one after the other. The strength of the method is that the resistance factor for a number of standard types of fragments can be calculated using complex function theory. With these standard types relatively sophisticated flow configurations under a dike or hydraulic structure can be compiled. The linking of the fragments can be realised using simple calculations (see also appendix I).

For the heave mechanism the measured vertical hydraulic gradient over the flood defence in the tail fragment is important. In figure 4.7 this is
Eq. 18

$$i_{optr}=\frac{\phi_{2,3}-h_p}{d}$$

18

This hydraulic gradient must be smaller than the permissible hydraulic gradient i_{toel} = 0.5.

To realise the calculations with the fragments method a spreadsheet program has been developed (see section 4.6 and appendix III). In the spreadsheet calculations the measured vertical hydraulic gradient can be easily linked to the total hydraulic head (H-h_p) over the flood defence. Using this nomograms/reference graphs can be drawn up in which the permissible hydraulic head over the defence can be read as a function of the design parameters, such as the length of the cut-off walls in relation to the thickness of the sand layer. Such a nomogram/reference graph is shown in figure 4.9. The relative embedding d/D is described on the horizontal axis and the hydraulic head ratio i_{toel} *L/(H-h_p). The graph belongs to a configuration with two cut-off walls of equal length. At a given total hydraulic head (H-h_p) over the defence, the length L of the defence and the criterion for heave i_{toel} to be applied we can read the relative screen length needed (at least the relative embedding in the sand layer) d/D. In this graph stars indicate how the graph would look if the MSEEP numeric water flow program was to be used instead of the fragments method. We see that the fragments calculation and MSEEP are very similar for values for the embedding length d greater than 0.2D. For embedding lengths less than 0.1D the calculations with the fragments method are inaccurate. It is recommended that a minimum embedding length equal to 0.1D be selected.

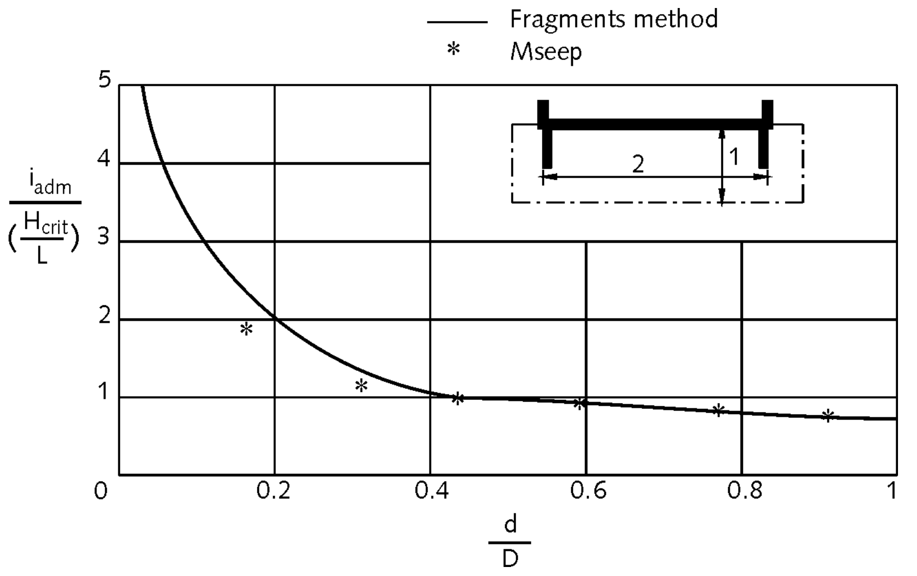


Figure 4.9 Nomogram/reference graph permissible hydraulic head over flood defence in relation to heave

__Fragments method

* Mseep

In the fragments analyses the basic assumption is isotropic permeability of the sand layer. The horizontal and vertical permeability can be different however, due to the presence of horizontal structure or dumping layers or lenses. In most cases this can be allowed for by reducing the fragments (vertically or horizontally). It is recommended however that an expert be consulted here.

The nomogram/reference graph is valid for a situation in which there is a good connection between the underside of the defence and the topside of the sand layer. If that connection is not present, or where it is uncertain, the resistance factor for the middle fragment must be reduced; see also appendix I and [GD 1998].

4.4 Influence of Foreland

The presence of scarcely permeable foreland and the width, thickness and permeability of the covering layer play a role in the calculation models for cracking, piping and heave. In this section the influences are addressed in more detail.

4.4.1 Influence of foreland on crack calculation

The foreland has a reducing effect on the potential in the sand at the site of a potential crack location. This effect is automatically included when the WATEX program with the ‘three section model’ (foreland, flood defence and hinterland) is used for the calculation of the potential in the sand layer.

Roughly speaking, the theoretical entry point for the groundwater flow in the sand layer, with respect to a situation without foreland, over a length of

Eq. 19

$$L'_v = \lambda_1 \, th\left(\frac{L_v}{\lambda_1}\right) \qquad 19$$

in the direction of the outside water is repositioned. In this formula L_v is the width of the foreland, λ_1 the distribution range (see section 4.1.3) and *th()* the hyperbolic tangent function

Eq. 20

$$th(x) = \frac{e^{x} - e^{-x}}{e^{x} + e^{-x}}$$

For smaller values of L_v / λ_1, for example smaller than 0.5 $L'_v \approx L_v$. For greater values of L_v / λ_1, for example greater than 2, $L'_v \approx \lambda_1$.

4.4.2 Influence of foreland on piping mechanism

As with cracking the effect of foreland is that the theoretical entry point, with respect to a situation without foreland, in the direction of the outside water is repositioned, in accordance with the same formula. That increases the theoretical seepage length by L'_v.

In both the classical rules of Bligh and Lane and the rule of Sellmeijer the increase in the seepage line can be included in the calculation.

4.4.3 Influence of foreland on heave mechanism

Here too the foreland has a reducing effect on the measured vertical hydraulic gradient on the inside of the flood defence.

In groundwater analyses with the help of the fragments model this can be calculated by using a foreland fragment (appendix I).

In groundwater analyses using a numeric groundwater flow model, based on FEM or FDM the effect can be calculated by including the covering layer in the foreland in the modelling in the analysis.

4.5 Influence of Time-dependency

The calculation models for groundwater flow handled in the sections above are based on stationary flow situations. That is, the flow situation when the hydraulic/hydrological preconditions, namely the outside water level H_1 and the inside water level or the ground surface level h_p, are maintained for a longer period. In reality the outside water level in particular will be time-dependent. High river drainage lasts as a rule five to ten twenty-four hour periods, the storm surge at sea (a maximum of) three twenty-four hour periods, while the tidal current has a period of approximately twelve hours. The variations in the time of the inside water level and the polder level will generally be smaller and slower, so that the assumption of stationarity is plausible.

In this section the effects of time-dependence of the outside water level on the cracking, piping and heave mechanisms are addressed in detail.

4.5.1 Effects of time-dependence of outside water level on cracking

Time-dependency has an influence on the way in which the groundwater pressures develop in the geo-hydraulic system under the flood defence. In appendix I the differential equations are given for the calculation of the stationary heads in the sand layer. The characteristics of the stationary situation are

- fully developed vertical flow towards the sand layer though the covering layer in the foreland
- fully developed vertical flow from the sand layer to the ground surface (or the ditch) by the covering layer in the hinterland
- fully developed horizontal flow in the sand layer.

In a non-stationary situation, besides the above-mentioned flow phenomena, the following time-dependent aspects play a role

- consolidation (compaction) of the covering soil layer in the foreland
- negative consolidation (swell) of the covering layer in the hinterland
- elastic storage in the sand layer. This effect is small in comparison to the above-mentioned effects and therefore negligible.

In figure 4.10 the water pressures along a vertical in the foreland and along a vertical in the hinterland for the non-stationary situation are illustrated in a graph. In the foreland a water pressure is prevalent in the top layer in relation to the final stationary situation. Because of this the water pressure gradient at the bottom of the top layer is greater than it is in the stationary situation. We could take this as the apparently greater permeability of the clay layer. This is rendered in a time-dependent distribution range $\lambda_{1,t}$ which is smaller than the distribution range $\lambda_{1,}$ in the stationary situation. In the hinterland there is water pressure in the top layer in relation to the final stationary situation; this means that the water pressure gradient here is also greater than in the stationary situation. Accordingly, here there is also a time-dependent distribution range $\lambda_{2,t}$ which is smaller than the distribution range $\lambda_{2,}$ in the stationary situation. With these shorter distribution ranges there is a steeper course in horizontal direction of the water pressure in the sand layer (see figure 4.10). The effect is a lower head in the sand layer at the site of the potential crack location and so a more favourable point of departure in relation to the assessment of the probability of cracking.

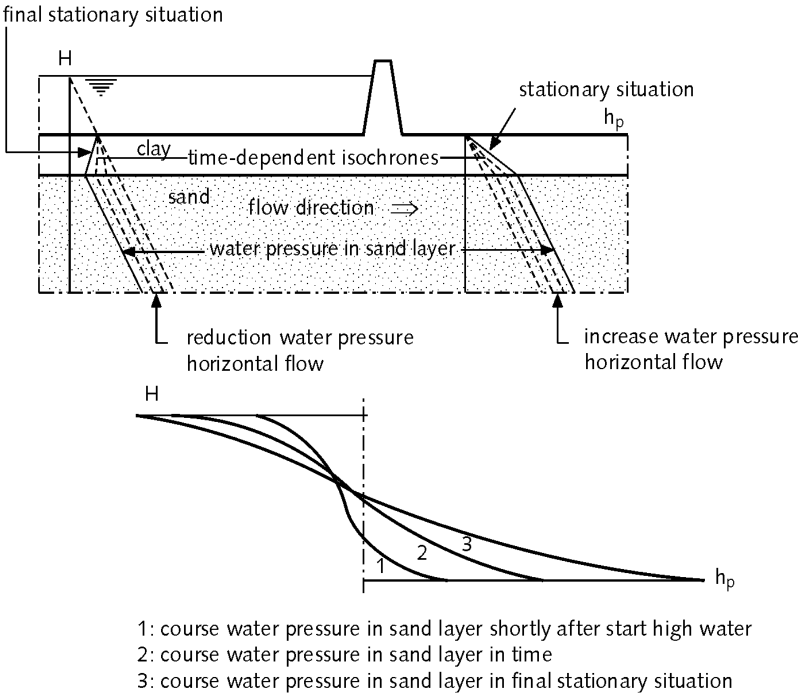


Figure 4.10 Time-dependence in development of water pressure in sand layer
final stationary situation

Any role played by time-dependence can be estimated using the hydro-dynamic periods of foreland and hinterland (for one-sided flow)
Eq. 21

$$t_h = \frac{d^2}{c_v}$$

in which *d* is the thickness of the covering layer and c_v the consolidation coefficient [m^2/s] (for the foreland; for the hinterland the calculation is based on the swelling coefficient [m^2/s]). If we have a thickness of 1m and $c_v = 5\ 10^{-6}$ m^2/s, then the hydro-dynamic period $2\ 10^5$ s $\approx$ 2.5 twenty-four hour periods. With respect to a high water drainage wave lasting 5 to 10 twenty-four hour periods, with a maximum water level after 5 twenty-four hour periods, the hydro-dynamic period is not long. In that case, a fully developed flow in the sand layer must be assumed, the favourable effect of time-dependence therefore plays no role here. We see that we incorrectly assume a favourable effect of time-dependence when the hydro-dynamic period is overestimated. If there is uncertainty about the consolidation or swell coefficient we must work with careful estimates; careful means in this case

relatively high values of coefficients. The consolidation/swell coefficient is related to permeability and compressibility.
Eq. 22

$$c_v = \frac{k}{\gamma_w \ m_v} \qquad (22)$$

in which k is the permeability (m/s) and m_v the compressibility coefficient [m^2/kN] for one dimensional compression.

If the layer thickness in the above-mentioned example were twice as great then the hydro-dynamic period would be four times as long, in the range of 10 twenty-four hour periods. That means that the groundwater flow in the substrate is not fully developed when the drainage wave is greater, after around five twenty-four hour periods. In that case it can be worthwhile to include the time effect in the crack consideration.

Normally the hydro-dynamic period of the covering layers would be considerably greater than the period of the tide. The time effect therefore usually does play a role in the effect of the tide.

It is however not possible to estimate in advance the degree to which the calculation with time-dependence effects results in more favourable outcomes for crack checks.

The theory for the time-dependent potential development in the sand layer is described in the *Design of River Dikes Guide* [TAW 1989]. In the WATEX program there is an option to calculate time-dependently. For the modelling of the high water wave a one-time blockwave or sinus-shaped periodic waves can be selected.

4.5.2 Influence of time-dependence on outside water level on piping and heave
The tides component in the hydraulic head over the flood defence on sea is generally considerable. Depending on the situation, tidal fluctuations in the outside water level in an inward direction will be absorbed by the sand layer. Indications of this can be obtained using 13-hour measurements of the water pressure response.

Although theoretically well-founded calculation models are not available to estimate the influence of fluctuations on the erosion process, in the case of strong damping near to the exit point it may be worthwhile to include only part of the tidal amplitude in the calculation of the hydraulic head present over the flood defence. It is recommended that experts are consulted in relation to this.

For the heave mechanism the current maximum gradient at the site of the cut-off wall is normative. There is no simple methodology to calculate the time-dependent gradient due to tidal fluctuations in a substrate configuration with cut-off walls. Modelling with a multipurpose FEM program for groundwater flow and consolidation will have to be set up case by case, and preferably calibrated to the results of 13 hour water pressure response measurements. Also here it is recommended that experts be consulted.

4.6 Computer programs available
There are various computer programs for calculations for crack, piping and heave checks. An overview is given in table 4.3. These are programs developed in the scope of TAW research specially for crack, piping and heave analyses and the best known common (multipurpose) programs with which those analyses can be realised.

Table 4.3 Overview common computer software for crack, piping and heave checks

Naam	**specifiek (S)/ multi purpose (MP)**	**Probleemtype**			**Type programma**	**Opmerking**
		opbarsten	**piping** [2)]	**Heave**		
WATEX	S	x [3)]	B, L, S	x [3)]	PC-DOS	Det. [1)], Analytisch
MPIPING	S		S		PC-DOS	Prob. , Analytisch
MSEEP	MP	x [4)]		x [4)]	PC-DOS [5)]	Det., EEM
SEEP/W	MP	x [4)]		x [4)]	MS-Windows	Det, EEM
Heave	S			x	DOS, Quatro4	Det., Fragmenten
PLAXIS	MP	x [4)]		x [4)]	MS-Windows	Det., EEM

Noot 1: Det. = deterministisch, Prob. = Probabilistisch
2: B = Bligh, L = Lane, S = Sellmeijer
3: Alleen grenspotentiaal en kritiek verhang, te toetsen actuele potentiaal moet worden opgegeven
4: Alleen berekening actuele potentiaal, toetsing aan grenspotentiaal via WATEX
5: In nabije toekomst ook MS-WINDOWS versie

Name
Specific (S) / Multipurpose (MP)
Problem type
cracking
piping
heave
Type of program
Remarks

Deterministic, Analytical
Probabilistic, Analytical
Deterministic, FEM
Deterministic,, FEM
Deterministic, Fragments
Deterministic, FEM

Note
1. Det. = Deterministic, Prob. = Probabilistic
2. B = Bligh, L = Lane, S = Sellmeijer
3. Only potential limit and critical hydraulic gradient, actual potential to be tested must be given
4. Only calculation actual potential, monitoring against potential limit via WATEX
5. In near future also MS-Windows version

Appendix III includes a short description of the contents of each program. It is emphasised that software is subject to new development to a great extent; an overview is based on the current state of the art, as far as it is known.

5 Monitoring and Measures

5.1 General

Monitoring versus designing
The application of the different calculation models are discussed in this chapter. A distinction is made between monitoring the current situation on the one hand and dimensioning measures or drawing up a design on the other.

Monitoring involves the evaluation of an existing flood defence. The aim is to prove the flood defence meets the requirements with regard to piping, using as little effort as possible due to economic considerations. In first instance, simple calculation rules are used which require little data concerning the flood defence. If the flood defence does not meet the simple rules, then more advanced calculation rules are used; this generally requires more data. If the requirements are still not met, more complex methods can be applied in certain cases.

The dimensioning of measures refers to the creation of a new situation. Dimensioning will usually be preceded by monitoring of the current situation or a preliminary design, to check whether measures are required. Also in the dimensioning of measures in first instance the work is directed from simple to advanced. The measures themselves are often optimised using more advanced calculation rules.

A significant difference between monitoring and dimensioning is that monitoring refers to the current situation, while dimensioning considers the situation at the end of the plan period. This is expressed among other things in the water level used and in possible changes to the flood defence or in the vicinity of the flood defence during the plan period.

Advice to readers
The calculation rules which are presently operational are examined in this chapter, including the data and the research needed to apply the rules. The estimation of parameters in existing situations is given special attention. More complex methods and calculation rules currently under development are mentioned, but not discussed elaborately.

Possible measures to prevent piping or heave and the dimensioning for them are examined. Specific aspects involved in monitoring and the design of hydraulic structures are discussed in separate sections. The influence of trees and pipes in or near a flood defence are also discussed separately. Implicit or explicit requirements for management in daily situations and during high water are often set during monitoring or design. An overview of possible requirements for management is given, showing the situation in which they are of importance.

Step by step monitoring
Monitoring is conducted in several steps. Monitoring ends as soon as it has been proven that there is no danger of piping or heave. The overviews of the different steps are given in flow charts 1 to 4. The charts refer to the section numbers in this chapter. Charts do not always need to be viewed in the order they are shown. Repeating a number of steps in a more or less iterative manner is sometimes useful, to determine the data more accurately. Steps can be skipped in other cases, for example when a relatively large amount of data is available.

5.2 Monitoring at Dikes

5.2.1 Geometry and water levels
The following data concerning the geometry and the water levels is necessary in monitoring for piping or drawing up measures:
- the geometry of the dike
- geometry inside the dike, to determine the exit point
- test level or design water level (MHW)
- water level inside the dike

Exit point
The indicative exit point is generally found at the inside toe, if the ground surface inside the dike is horizontal. The exit point may be further from the dike if the ground surface slopes down or is

irregular, at a local low level such as a ditch or pool. The location of the exit point is not always clear and this also depends on the location where the crack occurs. If this is uncertain then tests are conducted for more exit points.

Water level inside the dike
The water level inside the dike is equal to the ground surface at the exit point if there is no open water. If there is open water then the water level inside the dike is equal to the water level during high water in the water garden inside the dike. This water level can depend on the pump management. If the water level inside the dike during high water is not known or is not checked, the average water level can be used, because the water level during high water will almost always be higher. At a water garden which is surrounded by a quay, a scour hole or lake from an old dike breach for example, the inside water level can be taken if there is uncertainty as to how reliable the embankment is, which would be the case after damage to the embankment.

Charts 1, 2 and 3 provide an overview of the different steps followed during monitoring. The charts include references to the sections.

5.2.2 Soil survey

Soiling surveys are necessary to gather the following data:
- the soil composition
- the entry point
- the thickness and permeability of the water-bearing sand layer
- the thickness, specific gravity and permeability of the clay layer inside the dike

Soil composition
A study of soil composition must show whether there is a piping sensitive soil composition. A survey therefore aims to determine the stratification and the ground type per layer. The dike's soil composition is also of importance here. Possible piping sensitive soil profiles are discussed in chapter 3. If the profile is not piping sensitive then a further survey is not required. If soil composition is characterised as 'non piping sensitive' then a relatively detailed survey is required, also to exclude local piping sensitive soil profiles. Observations can play an important part in the evaluation: if boils have ever been ascertained, be they sand boils or not, then there is certainly a piping sensitive profile present. The opposite is not necessarily true.

Soil composition surveys also reveal whether there is piping or heave. If sand is only present inside the dike and the exit of the water is more or less vertical, then heave exists. This is discussed in greater detail in chapter 3. In that case the Lane method or heave rules are used to test for heave. If there is no heave then the Bligh or Sellmeijer method is used.

Entry point
The entry point is the point closest to the dike where the water-bearing sand layer comes into direct contact with the outside water, or where the potential in the sand layer is equal to the water level above ground surface level. The presence of a clay layer in the foreland must be examined to determine this point. If there is no data, in first instance it can be assumed that the entry point is located at the outside toe of the dike. The entry point must still be determined if this assumption shows that the clay layer inside the dike can still crack. The following data is important to determine the entry point:
- the zone in which monitoring or another method of checking can take place of use and activities
- the clay layer's thickness and permeability

The entry point will normally be chosen inside the monitoring zone, as this makes management easier to realise.

Thickness and permeability of the clay layer
The clay layer's thickness and permeability are determined in a study. The intensity of the study may depend on the geological, geo-morphological history and/or land use, in as far as it is known. If no data is available a general study can determine the clay layer's uniformity and the intensity of the definite research can be determined on this basis. Variations in ground surface level and (natural) vegetation, the site's use and the boils observed during high water must also be taken into account to determine the study points or orientation. It should be taken into account that discontinuities in the clay layer's

thickness can occur which are not identified. Consider filled channels or ditches which have been filled up with sand for example. In some cases such discontinuities can be observed as a result of boil forming inside the dike during high water.

It is often advisable to also study the clay layer in chambers and oxbows.

In principle the entry point is the point where the clay layer stops or is interrupted. If the hydraulic resistance of the clay layer for example is low however, in relation to the horizontal permeability of the water-bearing sand layer, then the entry point must be chosen closer to the dike. The distribution range is calculated for this purpose, according to section 4.1.3.

If a study is carried out in order to draw up a design, it must be taken into account that the foreland's breadth and the clay layer's thickness included in the design may limit the land use in the foreland. The use of minimum values for these parameters is often recommended, instead of the clay layer's thickness and breadth. This places the least restrictions on the future use of the foreland.

The entry point at a certain location can also be determined on the basis of piezometric gauge observations. The piezometric gauge observations are analysed with WATEX or a numerical groundwater flow model. One advantage is that ground monitoring outside the dike is minimised. One disadvantage is that piezometric gauge observations are not always available and useful observations can only be made when the high water is at a sufficient level.

A certain degree of caution is required to draw up measures when piezometric gauge observations are used. The measuring tubes react to the current situation, which must not be constant during the entire plan period. Changes in the foreland or on the river bed can occur, which can result in higher potentials. This aspect is less important in monitoring.

If the entry point is known the test can first be carried out according to the Bligh method (section 4.2.2), before further ground monitoring is done.

Thickness, grain diameter and permeability of the water-bearing sand layer
The water-bearing sand layer's thickness and permeability can first be obtained from the literature, such as the NITG – TNO Groundwater examination or information from other authorities. The greatest thickness and permeability are used if it is not immediately clear which values apply for the part of the section under consideration. The groundwater examination shows several water-bearing layers of sand which are separated by relatively impermeable layers. These impermeable layers are not always present however. If the first water-bearing sand layer is relatively thin, less than 20 metres thick for example, then several probes will be needed to reveal whether the dividing layer occurs near the dike. If this is not the case then the first and second, and if necessary third water-bearing sand layer are considered as a single layer.

If the layers of sand are relatively thin or if intermediate sand layers are found then the groundwater examination is not sufficiently accurate. The thickness will then be determined through research.

The grain diameter of the sand in the sand layer is first estimated to apply the Bligh method. It is recommended to collect and store sand samples, if drilling through the clay layers into the sand layer are to be made. Grain distribution of the sand samples can be determined at a later stage, using the Sellmeijer method.

The clay layer's thickness and specific gravity inside the dike
The thickness of the clay layer or layer of peat inside the dike is determined via a study. For the most part the same considerations and indications apply as given to determine the clay layer outside the dike. The minimum thickness present, which occurs in the section for which the calculation is presumed representative, is applied in the calculation. The ground's specific gravity is determined in a laboratory. A sufficient number of representative soil samples are examined for this purpose.

Comment: The groundwater examination also gives the thickness and permeability of the covering layer. However these values are not accurate enough to test for piping and can therefore not be used.

5.2.3 Boil forming and cracking of the clay layer

Piping will only occur if boils are present inside the dike. If a clay layer or layer of peat occurs inside the dike, a boil will only exist if a small channel is present through this layer. There are several causes for this channel to originate:

- perforation of the clay layer due to drilling, sounding holes or foundation poles
- digging in the clay layer for soil consolidation or the construction of ditches or other water gardens
- the clay layer dries out
- the clay layer cracks

In these cases the seepage length must be checked with the help of Bligh's or Lane's calculation rules.

Perforation of the clay layer
In practice it is impossible to check for perforation of the clay layer due to drill holes or sounding holes, or other temporary work. These holes can possibly be detected during high water. It is assumed that boil forming can occur along the pole if foundation poles have been placed inside the dike, around which the ground cannot close properly. This occurs when piles are made heavier with a pile foot for example.

If perforations occur in covering layers, which meet the crack/uplift criterion, then the vertical hydraulic head in the perforation channel can be checked (heave criterion), in addition to checking the seepage length according to the Bligh or Sellmeijer method. If the heave criterion is met, then in principle sufficient safety against piping is guaranteed

Digging
Digging inside the dike can be observed by examining the field. A check of the seepage length may be needed with local digging for that specific location.

Drying out of clay
Clay which regularly lies above the groundwater level can dry out. In this case there is no longer an unbroken impermeable layer and it should assumed that boil forming can occur.

If no irregularities occur in the clay layer as described above, then small channels may occur as the clay layer cracks. Cracking occurs if the water pressure in the water-bearing sand layer under the clay layer is higher than the clay layer's weight.

Potential in the sand layer
The potential in the sand layer can be calculated with the analytical formulas of the *Guide on Designing River Dikes*, part I. Various computer programs, such as WATEX or numerical groundwater flow models, can also be used. Piezometric gauge observations can be used to check the parameters entered. An analysis is required for the analysis of the piezometric gauge observations and extrapolation to the test level, with WATEX for example. The benefit of piezometric gauge observations is that they give an accurate picture of the actual conditions. If enough piezometric gauges and observations have been made, the composition of the substrate can partly be determined on the basis of the observations.

Cracking
Cracking of the clay layer inside the dike is calculated as described in section 4.1.4.

If there is a ditch within the uplift zone or within the critical seepage length, then the layer thickness of the clay layer or layer of peat is reduced. The breadth of the uplift zone is equal to two times the layer thickness. The following reduction of the layer thickness is used:

1. $d > B$, with B: the breadth of the ditch at ground surface level: no reduction
2. $d_{ditch} < b$, in which d_{ditch} is the thickness of the layer under the ditch bottom and b the breadth of the ditch bottom: $d = d_{ditch}$

3. additionally: $d = d_{reduced}$, in which the reduced layer thickness is determined with a spread of 2:1, according to figure 5.1.

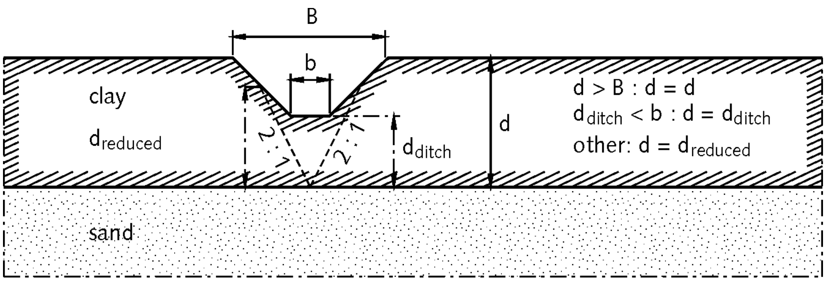


Figure 5.1 Clay layer of thickness with a ditch to be calculated

The weight of the water above the ditch bottom can also always be calculated.

There are situations imaginable in which use of the above-mentioned rules could lead to calculation showing that cracking does not occur, while a channel through the clay layer can exist. For example, if the ditch is relatively deep in relation to its breadth, or if there is no longer a consistent clay layer under the ditch bottom. In case of uncertainty, it must always be assumed that a (sand-carrying) boil could exist.

Piping will not occur if there is sufficient safeguards against cracking, taking the safety factor according to section 4.1.4 into account, and boil forming due to one of the other causes is excluded. In principle monitoring ends at this point. Otherwise, further checking for piping will take place, for example by checking the seepage length using the rules of Bligh, Lane or Sellmeijer, or by checking the vertical hydraulic head at the exit point (heave).

Checking for cracking requires a reasonably high degree of study, in terms of both soil surveys and calculation. However, this check can be skipped, and it can be assumed that cracking is occurring. Cracking is almost certainly occurring if boil forming or wet areas are observed during high water. If this only occurs very locally, it could also mean a perforation of the covering layer.

Checking for cracking is not needed, once it has been established that piping is not occurring and cracking is occurring. Otherwise a check for cracking can still take place.

5.2.4 The Bligh method

The method of Bligh is a simple test to detect piping, the seepage length is tested against the critical seepage length according to Bligh. The seepage length present has been obtained through study. Monitoring for more seepage lines is carried out, each with accompanying head, if the exit point cannot be determined beyond doubt.

Bligh's creep factor is determined on the basis of the estimated grain diameter of the sand in the water-bearing sand layer. If there is no data available on the sand, then a value of 18 is used for the creep factor. This is the maximum seepage line factor. This value can be used to quickly determine piping, if the entry and exit point are only roughly known.

The method of Bligh and the values indicated for C_{creep} do not have an extensively probabilistic basis. It is assumed that it is a safe approach. The minimum factor must be used for the seepage length, or the different parts and for the ground surface inside the dike.

If the seepage length present is less than the required seepage length according to Bligh, then there is a danger of piping. The evaluation can continue with the more advanced calculation rule according to Sellmeijer. This generally results in a more favourable (shorter) required seepage length, although this is not necessarily the case. This is likely if the D/L ratio (thickness of the water-bearing sand layer and the seepage length) is high.

5.2.5 The Sellmeijer method

The critical seepage length is calculated more accurately with the Sellmeijer method. The Sellmeijer method almost always results in a lower critical seepage length, if the thickness of the water-bearing sand layer is relatively limited. In general it is useful to apply the Sellmeijer method if the thickness of the sand layer is less than the seepage length. Use of the Sellmeijer method is always recommended if the necessary information is available.

The following additional information is required:
- the grain distribution
- the permeability of the sand layer

Soil survey
Sand samples of the water-bearing sand layer are needed to determine the grain distribution. If a study into the characteristics of clay layers, layers of peat, the stability or other aspects is needed, then deeper drillings in the clay layer will need to be made to collect the sand samples. The additional costs are relatively limited. If there is sufficient information on the other soil qualities, then drilling will have to take place specially for the piping research.

Grain distribution
The grain distribution influences the stability of the grains in the pipe. It is important to know the grain distribution especially under the layer which is hard to permeate, near the outflow opening. Sand samples can therefore best be taken inside the dike, close to it. The number of samples needed depends on the uniformity of the sand layer, in the dike's length-wise direction. A sufficient number of samples need to be available to gain some insight into this. A sufficient number of samples must be available to determine the characteristic value of d_{70}. The minimum number of samples required is three. It is recommended to collect at least five samples for each section with a more or less uniform sand layer. The characteristic value cannot be determined if less than three samples are available. In this case only a representative value can be estimated with which a rough calculation can be made. The characteristic minimum value of the average of the d_{70} is used as input in the Sellmeijer formulas. Appendix 2 describes how this is calculated.

Permeability
The permeability to calculate the potential (section 5.2.3) has already been determined on the basis of the overall data. More local and detailed information is needed for calculation according to the Sellmeijer method. The permeability can best be determined on the basis of the grain distribution according to Appendix II, because this is already known.

The permeability in the Sellmeijer formula is the characteristic maximum value of the average permeability of the entire sand layer. However, in general it will be too expensive to take sand samples of the entire sand layer. It must be considered that the fine fraction is often larger here than elsewhere in the sand layer, if only samples of the top sand layer are taken. Therefore it is recommended to drill deep enough, in any case under a possible transitional layer between the covering clay layer or layer of peat and the water-bearing sand layer. Other methods to determine the permeability are on site monitoring such as pump tests or piezometric gauge observations.

Accurately determining the permeability is not a simple task. A geo-hydrologist can be consulted if necessary.

Critical hydraulic head and seepage length
The Sellmeijer formula is used to calculate the critical head, see section 4.3.2. Characteristic values must be used for the different input parameters (Table 4.2).

The actual hydraulic head under normative circumstances must be smaller than the critical head. The seepage length needed must generally be determined in design situations. This can be done by using the Sellmeijer formula in an iterative manner several times. $L_{min} = 10 * \Delta H$ is used as minimum required seepage length in a sand layer, even if a smaller value is found with the Sellmeijer formula [TAW 1994].

Using the Sellmeijer method a larger required seepage length can be found than the maximum required seepage length, $18 * \Delta H$, in accordance with the rule of Bligh. Although in such a situation it would appear that Sellmeijer' method is normative, because of its better basis, Bligh's method is chosen in current practice. The provisional TAW-B guideline [TAW 1994] expressly allows this.

A vertical part in the seepage line at the outflow can be included in the calculation. However this only applies if it concerns a vertical part in the seepage line through a clay layer. Heave occurs if there is a vertical outflow in sand, Sellmeijer's method does not apply in this case.

If the test is not passed, an advance test may be useful in some cases. This is further explained in section 5.2.9. In all other cases it must be concluded that the required safety level with regard to piping has not been met. If the criterion for piping is met, any observations during inspections and high water will be evaluated.

5.2.6 Heave
Checking for heave means checking the vertical hydraulic head at the outflow, see section 4.3. Diagrams are included in Appendix I (Fragments Method) with a summary of the results of heave calculations for a number of simple geometries. There is a spreadsheet program for more complicated situations, but still with two-dimensional groundwater flows in the water-bearing sand layer under the flood defence. In many cases this can be applied to check for heave at dikes. Both the diagrams and the spreadsheet program are limited with regard to geometry, soil composition and material properties. One significant limitation is that a good connection exists between the underside of the flood defence and the top of the sand layer. If this is not so, or if uncertainty exists, then the calculation method indicated in annex I must be used, in which the resistance of the 'middle fragments' is reduced. If the diagrams of the program cannot be applied, then the more conservative method of Lane must be used (see section 5.2.7).

5.2.7 The Lane method
It is recommended to only use this method to check for piping if the Bligh or Sellmeijer methods cannot be applied; for example when cut-off walls are used on the upstream side or in the middle under the flood defence. This method can also be applied to check for heave, both for monitoring and designing. However, in a design situation, use of the (less strict) heave checking rules in section 5.2.6. is recommended where possible. These rules cannot be applied in some situations (horizontal seepage, for example). The calculation model for two-dimensional groundwater flow cannot be used here. The method of Lane is the last resort in these cases, unless a real three-dimensional groundwater analysis is being considered.

The seepage length, to be calculated, is composed of vertical and horizontal components. These can consist of cut-off walls, a vertical section at the outflow and the seepage line under the foreland and under the dike. One-third of the horizontal section is used to calculate the seepage length present.

The weighted seepage line factor of Lane must be known in order to determine the critical seepage length. This depends on the type of material in the water-bearing layer, an estimate of the coarseness of the sand is sufficient. The method of Lane, just like the method of Bligh, is based on empiricism. The (best estimate of the) minimum seepage length and the reduction at normative outside water level must be applied for monitoring.

5.2.8 Evaluation of observations
Observations can provide valuable additional information. Evaluation of any available observations is always recommended. It is also recommended to examine any observations before starting a soil survey, because boils which are detected can indicate local extra piping sensitive conditions. This can be taken into account for the soil survey.

Observations can be divided into two categories:
- observations during (periodical) inspections
- observations during high water

Periodical inspections means all the relevant aspects are checked. The following is important with regard to piping:
- the altitude of the ground surface inside and outside the dike
- how filters or filter structures function
- how cut-off walls function

The filters must be checked for both sand density and water permeability. If the filter structure is necessary with regard to safeguards against piping, then inspection is required at least every five years. Inspection of filters is not a simple process and there is little experience in this field. Filters are therefore rarely incorporated into a design. If filters occur in an existing situation that are important for evaluation of piping, it is recommended that an expert is called in to check the situation.

During *high water*, with regard to piping, water which flows out inside the dike must be taken into account and if this is the case, whether sand is conveyed. If boil forming is ascertained, sand-carrying or not, while the available data does not lead one to expect such, or not to the degree observed, all assumptions and starting points must be carefully checked. If no explanation can be found, it is recommended to have a further study carried out locally. Further study is also recommended even if the amount of seepage increases compared to earlier high waters. In these situations it can no longer be assumed that the study and the calculations on which monitoring for piping is based are sufficiently accurate.

5.2.9 Advanced monitoring

The methods examined in the previous sections generally apply to situations which are easy to schematise. In some situations this will not suffice and more advanced monitoring is needed. This is possible in the following situations:

- the outside water level is highly non-stationary, with dikes in the tidal area for example
- the geometry or layer composition is not uniform in the dike’s direction or perpendicular to the dike
- it can assumed that cracking will not occur at the same time as uplift.

Normally an expert must conduct advanced monitoring.

Non-stationarity

The high water wave in the tidal area, and to a less extent in the transition area lasts a relatively short time. Within a high water period the water pressure inside the dike in these areas will therefore hardly ever reach a stationary value. This reduces the risk of cracking. Groundwater calculations are needed to find the non-stationary effect, calibrated to piezometric gauge observations. The WATEX program provides the necessary options for this. Except for cracking, non-stationarity also effects the erosion process. However, as yet no proper methodology has been worked out to include this effect.

Non-uniform soil composition

The different calculation rules are based on a uniform thickness of the water-bearing sand layer both parallel and perpendicular to the dike. The method no longer applies if the thickness varies strongly, or if the sand layer cannot be said to be infinitely long. This also applies in situations with cut-off walls which are not placed at the exit point and in situations which are no longer two-dimensional. For the time being an estimate can be made in which the exit hydraulic gradient in the real situation is compared with a theoretical exit hydraulic gradient in an idealised situation. Groundwater flow calculations are required for this.

Uplift, not cracking

In some cases it is likely that no cracking will not occur in the case of uplift, with a thick consistent top layer for example. Although there is no crack channel, this does not mean this situation cannot be piping sensitive. There is a thin bubble between the sand layer and top layer on the inside the dike. A sufficient seepage length is needed to prevent the horizontal transport of sand grains under the flood defence to the bubble. The hydraulic head between the outside water level and the head limit under the top layer can be used to check the seepage length.

5.3 Monitoring at hydraulic structures

Diagram 4 *Hydraulic structures* gives an overview of the different steps which are taken for monitoring. References to the sections are included in the diagram.

5.3.1 Vertical seepage/horizontal seepage and piping

In point of fact vertical seepage means the same as piping. Horizontal seepage also means the same but occurs next to the structure. In principle the evaluation is carried out in the same way. One difference between dikes and hydraulic structures is that the seepage line, including the entry and exit point is more or less fixed. Cut-off walls are usually placed at hydraulic structures, to increase the vertical or horizontal seepage line.

5.3.2 Structure
Information on the structure is needed to determine the seepage line. The following is important:
- the measurements
- location, condition and cut-off walls measurements
- the presence of foundations piles

The construction drawing is the best source of information. If there are no drawings available then the structure's measurements can usually be taken. Information on cut-off walls or foundation is less easy to obtain. Cut-off walls can be shown with the help of digging or analysis of the piezometric gauge observations. This is also recommended if the state of the cut-off walls, or the manner in which they are connected to the structure, is uncertain. Uncertainty can exist as to whether:
- the cut-off walls are periodically or permanently above the groundwater level
- seepage line has been established during high water or other burdening conditions
- the steel screens are subject to corrosive conditions; salt water or peat often cause relatively much corrosion (*Guide on Structural Designing* [TAW 1994²] gives several guidelines for this)
- negative friction occurs along the screens
- the expected life span of the screens has been reached
- corrosion could have occurred due to electrical tension

The benefit of digging is that insight in the condition of the cut-off walls can be obtained reasonably quickly and accurately. However, one major disadvantage is that the dike which connects to the hydraulic structure must be partly dug out, which weakens the flood defence. In most cases provisional countermeasures are required. Digging in sand layers below the groundwater level often requires drainage. Digging to check for cut-off walls is a major operation, especially for screens under a hydraulic structure where the floor is far below the average water level. This method is only used if no other options remain.

Piezometric gauge observations are first made next to the hydraulic structure, so no holes have to be drilled through the floor. One disadvantage of the evaluation on the basis of piezometric gauge observations is that high water is needed. The presence of cut-off walls can then be deduced from the piezometric gauge observations, the number and the measurements cannot be accurately determined.

It is recommended to check whether there are results from monitoring other hydraulic structures, with the same life span and construction, in the same conditions.

If it is not possible to determine the presence and quality of cut-off walls beyond doubt, it is best to consult a specialist. For the consequences with regard to piping and heave, as a result of leaks, see [GD 1998], also see Appendix 1.

5.3.3 Soil survey
Soil surveys at hydraulic structures in connection with monitoring for piping and heave are important to determine the type of soil and the coarseness of the sand in the water-bearing layer. Piping only occurs in sand layers. The groundwater does not reach a sufficiently high velocity in clay layers and soil is conveyed less quickly, because of the cohesive qualities of the clay. If the clay does not connect properly to the hydraulic structure, high rates of flow can occur in holes next to the hydraulic structure, which can result in erosion.

A soil survey must be conducted very close to the hydraulic structure. Soil consolidation may have been used in the construction process, or the building excavation pit may have been filled with sand. Design or structural information may provide additional information.

Only short cut-off walls are needed if the hydraulic structure is connected directly to clay. The length of the cut-off walls (in the dike's length-wise direction) depends on the dimensions of the hydraulic structure. A minimum measurement of one metre is used for the pipelines. *Guide on Water-retaining Hydraulic Structures and Special Structures* [TAW 1997] recommends making the cut-off walls at least as wide as the breadth of the building excavation pit. These cut-off walls prevent water flow from occurring in any holes directly next to the hydraulic structure. The clay around the cut-off wall must connect properly. If this is the case then no further monitoring for piping is needed.

If there are no cut-off walls and the hydraulic structure connects to clay, then an evaluation is not actually possible. The available evaluation methods are aimed exclusively at piping in sand layers. Another method must be used to prove that there is no seepage flow under or along the hydraulic structure or that no flushing out of soil can occur. Micro- instability or erosion can occur if this cannot be proved.

5.3.4 Seepage line
The first step of the test is to establish the normative seepage line. The normative seepage line is often a combination of vertical seepage and horizontal seepage. A three-dimensional analysis is needed to check this.

The following is important to determine the normative seepage line:
- a contact ring/short circuit seepage line could exist between screens and structural components (figure 5.2)
- a bad connection to various structural components, such as screens on the main structure could result in a shorter seepage line
- horizontal components in a seepage line where holes are located, a pile foundation for example, are not included in the calculation when the rule of Lane is applied. A reduction in the resistance factors is necessary to check for heave using the fragments method (see Appendix I).
- the normative seepage line is sometimes only over part of a structure, a sluice head for example
- various hydraulic conditions may be important, such as:
  - situation during high water (test level or NHW)
  - situation during maintenance or inspection

An impermeable floor upstream or a floor impermeable to sand downstream from the hydraulic structure in some cases can mean an extension of the horizontal seepage line in relation to the vertical seepage. A good connection is required in that case.

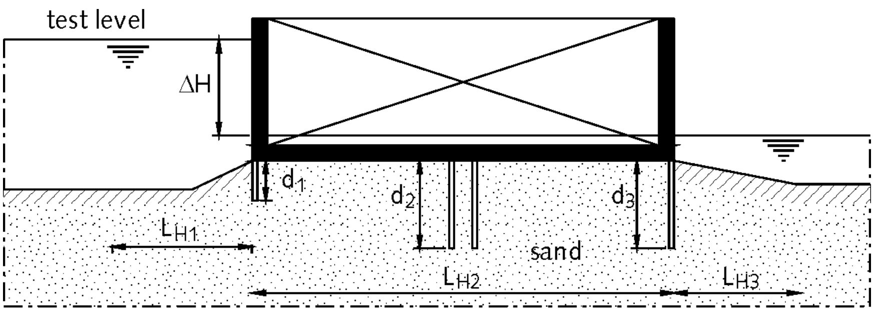


Figure 5.2 Determine seepage line for seepage erosion
(test level = water level as defined for safety assessment)

Without calculations it is not always possible to determine which seepage line is normative. In that case all seepage lines which could be normative must be evaluated. A seepage line which consists partly of a dividing section between the hydraulic structure (including cut-off walls) and a clay layer, does not have to be monitored if effective cut-off walls are present in the clay layer. If there are no properly functioning cut-off walls, or their existence cannot be proved, it must be assumed that there is a split between the hydraulic structure and the clay layer. The seepage line must be tested in this case. In principle, a seepage line which continues all along clay cannot be tested using the familiar methods.

5.3.5 Method of monitoring
Every combination of a hydraulic head and the matching shortest weighted seepage line is monitored for piping or heave. Piping exists if the outflow is horizontal or goes through a covering layer. Heave exists if the outflow goes vertically through a sand layer.

The method applicable depends on the outflow and the seepage line:

- a seepage line which is only horizontal is monitored with the methods of Bligh or Sellmeijer
- the heave rules can be applied if only vertical seepage and a vertical outflow in sand occurs
- in all other cases the method of Lane is used, mainly in cases where the rules of Bligh or Sellmeijer or the heave rules are not applicable

5.3.6 Bligh and Sellmeijer
If the seepage line is only horizontal the methods of Bligh and Sellmeijer can be applied as described in sections 5.2.4 and 5.2.5. If the hydraulic structure is founded on piles, the horizontal seepage line must not be included in the calculation. The methods of Bligh and Sellmeijer do not apply in that case.

5.3.7 Lane
The Lane method can be applied in all cases. A weighted seepage line is used in which one-third of the horizontal parts of the seepage line are included in the calculation. If the hydraulic structure is founded on piles, the horizontal seepage line must not be included. The vertical seepage length is the sum of all vertical parts in the seepage line. A contact ring/short circuit seepage line must be taken into account, for example between two screens which are placed relatively close to each other.

It may be assumed that the method of Lane is conservative. This means that in design situations in particular it is better to use Sellmeijer or the heave rule.

5.3.8 Heave
Heave can occur if a vertical part is present in the seepage line at the outflow. In most cases this is created through a vertical cut-off wall at the hydraulic structure's downstream side. The rule is that the hydraulic gradient, on average over the vertical part in the seepage line at the outflow opening, must be smaller than 0.5.

There is a spreadsheet program to check for heave, based on the fragments method (see chapter 4 and annex I). Several diagrams have been included in annex I, calculated using this program. With the help of the diagrams, for a number of standard geometries it can easily be determined whether the above-mentioned heave condition can be fulfilled. The spreadsheet program is used for deviating geometries. As yet this program can only be used to monitor for heave in cases in which groundwater flow under the structure can be taken as a two-dimensional problem (namely in a vertical cross section diagonal to the flood defence's length-wise direction) and where there is a good connection between the underside of the flood defence and the sand layer below. If the groundwater flow problem is essentially three-dimensional (under and along the structure, for example), then a seepage line analysis is necessary. In most cases only Lane's calculation rule will be left to realise monitoring. If there is (any) settlement splitting or other space between the underside of the flood defence and the sand layer, then a reduction in resistance factors with fragments analysis is needed (see Appendix I).

5.3.9 Evaluation and observations
Observations can be divided into two categories:
- observations during (periodical) inspections or maintenance
- observations during high water
- observations during maintenance work, in which extreme falls occur

Periodical inspections
All relevant aspects of a hydraulic structure are checked during periodical inspections. The following is important with regard to piping:
- permeability of the floors
- how the filters or filter structures work
- the settlement of the hydraulic structure
- connections to the various parts of the structure

If, in determining the entry point or the horizontal seepage length, it has been assumed that the floors of the sluices, divers, perskommen/pressure chambers and so forth are impermeable, then this must be checked. The connection of the walls to the floors is also checked.

Filter structures can be used to prevent sand from flushing out. A filter can prevent piping even if the seepage length is too short. One condition is that the filter is still impermeable to sand and permeable to water. If the filter has become impermeable to water, because it has filled up with silt for example, causing water overpressure to develop under the filter, the filter could crack during high water.

If a hydraulic structure settles this could mean that floors or walls are no longer watertight or that connections between floors, walls and cut-off walls are no longer watertight.

During maintenance or inspection work conditions could occur which are normative for piping. This could be a good opportunity to check for vertical seepage or horizontal seepage.

High water
With regard to piping during high water, water that exits inside the dike should be taken into account and if so, any sand that is being carried. If boil forming, sand boils or otherwise, is observed while the information available would not lead one to expect such, or not to the degree observed, then all assumptions and starting points must be carefully checked. If no explanation can be found then further research is recommended. This is also recommended if the amount of seepage increases in comparison with earlier high waters.

5.3.10 Advanced monitoring
If the geometry deviates strongly from the idealised situation which is assumed in the models then advanced monitoring at the hydraulic structures can be useful. This occurs in strong three-dimensional situations, for example. Advanced monitoring can also be useful if the spreadsheet program for heave no longer offers sufficient entry points. In most cases an advanced test will be based on an analysis of the exit hydraulic gradient.

5.4 Measures

5.4.1 General
The dimensioning of measures is done according to the same methods and calculation rules as described for the monitoring in section 5.2. Measures at new of existing structures will generally be designed according to Sellmeijer’s method or the heave criterion. It is useful to take the non-stationary aspects into account especially with dikes in the tidal area. The methods and calculation rules are not repeated in this section. The measures are discussed exclusively in terms of quality and a broad indication of the costs is given.

Fundamentally different measures are possible. The differences are chiefly due to different aspects of the failure mechanism being included. The various principle possibilities are:
- extending the seepage line, horizontally or vertically
- preventing cracking of the clay layer within the critical seepage length
- reducing the head
- preventing the sand from flushing out

These possibilities are described individually in the following sections, including an indication of relations with (measures in connection with) other failure mechanisms. The measures are shown schematically in figures 5.3 through 5.5. The next section deals with specific aspects of hydraulic structures. The last section shows a qualitative comparison of the costs of the different measures.

5.4.2 Extending the horizontal seepage length
Extending the horizontal seepage length outside the dike means the entry point is placed further from the dike, see figure 5.3. This can be realised by adding a clay layer. In nearly all cases the clay layer is dug in, in connection with the riverbed and/or other considerations. However this is not imperative. The following requirements are made of the clay layer:
- sufficiently watertight
- protected from damage

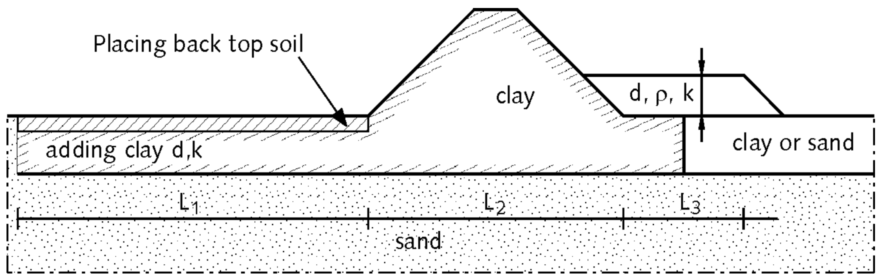


Figure 5.3 Foreland improvement and piping berm
Placing back top soil
adding clay d,k
clay
clay or sand
sand

In principle the range is calculated according to section 4.4, to calculate the required thickness of the clay layer in combination with the breadth of the foreland improvement.

A sufficient watertight clay layer is generally at least 1 metre thick and has a lutum content of 20% or more and a sand content of 35% or less. It is recommended that another layer of soil is added to protect the clay layer from damage by ploughing, erosion, roots, etc. This composition and thickness of this layer depends on the future use of the ground. The clay layer must not dry out. A covering layer of 0.3 metres is sufficient if it is to be used as grassland. In all other cases a layer thickness of 0.5 metres or more is needed.

A clay layer can also be added under water gardens outside the dikes, such as oxbows. The water must be removed from the water garden for a proper closure. Special attention must be focused on protection against erosion, because it is difficult to check the clay layer.

It is also possible to extend the seepage line inside the dike. This means a piping berm is constructed. Section 5.4.4 and 5.4.5 explains this.

5.4.3 Extending the vertical seepage line
A distinction is made between extension near the outflow opening on the one hand and extension under the dike or outside the dike on the other, for the extension of the vertical seepage line.

Extension of the vertical seepage line at the outflow (figure 5.4), by applying a cut-off wall is the most effective (see remark 1, section 4.2.3). Dimensioning can then be conducted with the heave rules. If the geometry and ground composition give cause, there should be a check for cracking in front of (that is on the upstream side of) the screen.

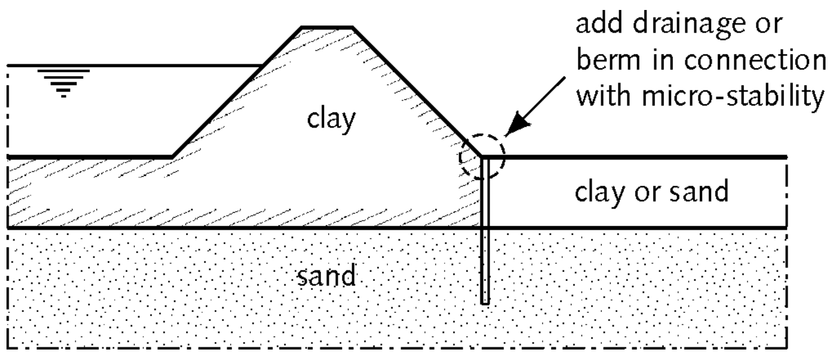


Figure 5.4 Cut-off wall at dikes

Water can collect behind the cut-off wall when placing a cut-off wall at the inside toe of a dike, which causes the phreatic water level to rise and the potential in the sand layer to go up. This has adverse effects on the stability of the inside slope and on micro-stability. Further analysis for additional measures is needed. Additional measures can consist of:

- a drainage structure, in which the water is drained off,
- a berm above the cut-off wall, through which the drainage of the dike body takes place, Increasing stability.

If a cut-off wall under or outside the dike is under consideration, then dimensioning is to take place via more advanced methods.

5.4.4 Preventing cracks inside the critical seepage line
Preventing the clay layer inside the dike inside the critical seepage length from cracking, also prevents piping from occurring. This is achieved by constructing a piping berm, figure 5.3. The breadth of the berm is determined on the basis of the critical seepage length. The weight of the berm must be such that a crack factor of 1.10 is realised, in accordance with section 4.1.4. The height of the berm and the specific gravity of its material are based on this. Other than weight, no further requirements are made of the berm material.

A seepage line can be created by building or planting, which can cause piping without the clay layer cracking. Section 5.6 discusses the permissibility of plants on a piping berm. In principle building on a piping berm is permitted. Perforation of the clay layer, due to utilities facilities or foundation piles for example, must be prevented.

Structures which perforate the piping berm are not permitted, unless it can be proven that the risk of piping is permissible small. If necessary special arrangements can be made to realise this.

Depending on the future use of the piping berm an additional height can be added. An additional height of 0.5 metres can be considered if the piping berm is to be used as building land or a private garden for example. In most cases this will be sufficient to compensate loss of weight due to ploughing, digging, construction of small ponds or the harmful effects of low permanent plants. Regular checks remain essential in these cases.

A berm inside the dike increases the stability of the inside slope and can therefore have a double function. A stability berm’s weight must be such that a crack factor of 1.2 is realised. Stability calculations can place additional requirements on the berm’s measurements.

5.4.5 Piping berm in situations without a clay layer
Cracking will not occur in situations where there is no covering clay/peat layer inside the dike. The piping berm cannot just be dimensioned by increasing the crack factor. In this case several situations are possible;

1. there is a vertical outflow after the berm has been constructed (the berm has a higher permeability than the substrate):
    - the berm is dimensioned in such a way that the heave rules are complied with
2. there is no vertical outflow (the berm forms an impermeable covering layer):
    - the berm height is dimensioned for cracking and
    - the berm length is dimensioned in such a way that hydraulic head is smaller or equal to the critical head, with the seepage length present.

A berm which is dimensioned for cracking, must be sufficiently watertight and cohesive. In all other cases the berm must be sufficiently permeable to prevent piping at the berm’s end, unless the breadth of the berm is such that the berm’s end lies outside of the critical seepage length.

The berm’s length is chosen in such a way that at the end of the berm the criteria with regard to heave (situation 1) or piping (situation 2) are met.

For restrictions with regard to the use of an impermeable piping berm see section 5.4.4. Fewer restrictions apply to a permeable berm. From the viewpoint of how the berm functions, the requirement applies that the vertical groundwater flow gradient in the berm must not increase (in design conditions) because of use of the berm.

5.4.6 Reducing the head

The hydraulic head over the flood defence can be reduced by reducing the outside water level or increasing the water level inside the dike. A reduction of the outside water level is possible in some cases at small water courses or multilevel flood defences. However directly influencing the outside water level is usually not possible. This will not be addressed in more detail.

The water level inside the dike can be increased by:
- increasing the ground surface level inside the dike, or
- increasing the water level in water courses inside the dike or raising the water level on the ground surface level

Increasing the ground surface level inside the dike

In situations where no open water occurs inside the dike, the groundwater level inside the dike which is normative for piping is equal to the ground surface level. By increasing the ground surface level the water level inside the dike is also increased. In effect this is similar to the construction of a piping berm.

Increasing the water level inside the dike

Increasing the water level inside the dike directly influences the hydraulic head as a result of which the seepage length is shortened. Besides the reduction of the head, the potential limit is also increased if there is a clay/peat layer, or the exit hydraulic gradient is lowered if there is no closing layer. This can be taken into account in dimensioning.

If water gardens occur inside the dike where the water level is controlled by means of damming and drainage, then the water level can be increased by means of management measures. The water level will increase to ground surface level by itself in water gardens where the water level is not controlled. However if the storage in the water garden is relatively great, piping could have occurred before the water level increased. In these cases an increased water level cannot be used for calculation purposes.

The same effect is achieved by inundating the ground inside the dike. In practice this is only applicable if there is are seepage embankments or other facilities. These seepage embankments are part of the primary flood defence and will therefore have to be tested for all failure mechanisms. Here there is also a risk that piping has already occurred before the hydraulic head has been sufficiently reduced. The ground must be inundated with inlets or other facilities to guarantee operation.

The situation in which the groundwater level equal to ground surface level is normative for the stability of the inside slope. Increasing the water level beyond the ground surface level therefore has a positive influence on stability.

5.4.7 Preventing sand from flushing away

Piping can be prevented by stopping sand from flushing away. This can be achieved by installing a filter at the exit point, see figure 5.6. The filter can consist of a granular filter, built according to the filter rules or from a reinforced filtering cloth. Three aspects are important for this to work properly:
- the filter must be sand tight
- the exit point must be known
- the filter must be more porous than the substrate

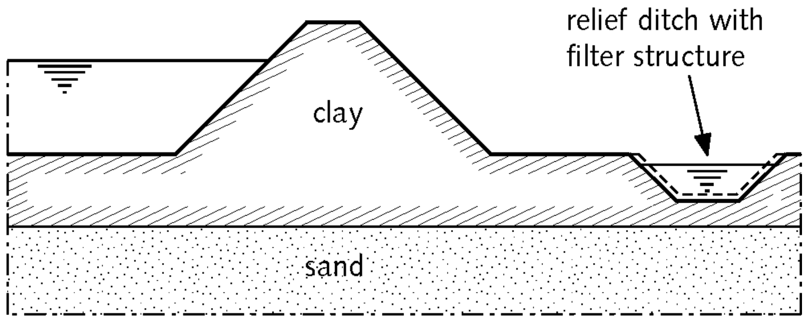


Figure 5.5 relief ditch with filter structure

Sand density
A sufficient sand density can be achieved by applying the filter rules. The connections of the filters to the surrounding area must be given special attention.

Exit point
A large part of the clay/peat layer must be cut by a ditch for example, to be certain that the filter is installed at the exit point. The clay/peat layer between the ditch and the dike must not crack either. If there is no covering layer, then the position of the exit point greatly depends on the local geometry and variations in the porous quality of the substrate. If there is a ditch directly behind the dike the exit point will almost always be located in the ditch. A similar situation however has serious consequences in connection with macro-stability and inspections during high water.

Porosity
The filter must be sufficiently porous to prevent water pressure building up under the filter. In that case the filter could crack, which can cause a seepage line along the filter. A filter will generally become clogged with organic or inorganic material, or because of organic activities in the filter. Regular checking is therefore needed. There is not much experience with checking filters. Possibilities are:
- placing piezometric gauges directly under the filter
- visual check of the entire filter
- on site monitoring

The possibilities of checking the filter must be given due consideration in the design.

Potential and seepage
If a ditch is constructed with the aim of checking for piping, then the ditch will function as a relief ditch. This causes the potential in the sand layer to decrease. An increase in seepage flow must be taken into account. In the design it must be checked whether the seepage can be stored or must be drained off. The stability of the inside slope benefits from the reduction in potential, as a result of which measures to increase the stability can be possibly be limited.

5.4.8 Measures at hydraulic structures
Most of the measures mentioned above can also be applied to hydraulic structures. A number of specific aspects are discussed in this section.

Extending the horizontal seepage length
This is not advisable for vertical seepage at hydraulic structures. A clay or other watertight layer must connect to the hydraulic structure properly, but this cannot be checked well. Inside the dike the exit point will nearly always lie at the end of the hydraulic structure. In practice the whole hydraulic structure must be extended to extend the seepage line.

The horizontal seepage line must be extended to prevent horizontal seepage. Cut-off walls are placed alongside the hydraulic structure for this purpose. The cut-off walls are often placed halfway along the hydraulic structure. These cut-off walls are also placed under the hydraulic structures if they are new, to create an unbroken screen.

If a hydraulic structure is built into an existing dike, these cut-off walls can serve as a temporary flood defence during construction.

The possibilities of installing cut-off walls in existing hydraulic structures are more limited. In principle cut-off walls can be installed alongside the hydraulic structure, but the hydraulic structure must be dug locally to connect the cut-off wall properly.

Extending the vertical seepage length
The most widely applied method to prevent vertical seepage is to install cut-off walls under the hydraulic structure. Placement of cut-off walls at the downstream side is the most effective method to prevent piping (see remark 1, section 4.2.3). Screens along the upstream side are needed to limit the water pressure against the floor’s underside.

Mostly short cut-off walls are placed, both along the upstream and downstream side and under and alongside the hydraulic structure to achieve a good connection between the hydraulic structure and the ground. These screens are primarily to prevent an opening from being created alongside or under the hydraulic structure, if erosion of the ground occurs. These screens can also increase the seepage length. An analysis of the seepage line will reveal this.

There are only limited possibilities to install vertical cut-off walls at existing hydraulic structures. Normally this is only possible at the inflow and outflow openings. The cut-off wall must form a watertight whole with the hydraulic structure.

Preventing cracking inside the critical seepage length
This does not apply to hydraulic structures due to the piping, unless the hydraulic structure’s floor is extended. The floor must be able to withstand the water pressure under the floor.

Reducing the hydraulic head
This is usually an effective measure. It is advisable to check this option especially for existing hydraulic structures which do not meet the requirements with regard to piping, because this can solve the problem, completely are partially, without requiring physical measures. The hydraulic head is reduced by increasing the water level inside the dike during high water. This measure must be included in the high water plan.

Preventing the washing away of sand
The exit point at hydraulic structures is usually known – along the hydraulic structure’s downstream edge. Piping is prevented by placing a filter at the exit point. The filter’s connection to the hydraulic structure forms a special point of interest. The working of the filter must be checked properly. This often proves to be difficult or even impossible if the filter lies under water during normal conditions. If the filter is no longer sufficiently porous, it will start to work as an extension of the hydraulic structure and the exit point then lies at the end of the filter. In that case the filter can crack and will no longer protect against piping. A filter is dimensioned on the basis of filter rules and weight. Piping or heave does not come into play if the filter is placed and works properly.

5.5 Estimate of the Costs

The costs of the measures, compared with each other and absolutely, greatly depend on the shortage in seepage length. The costs of maintenance, management and depreciation costs are important in addition to the construction costs. The table below shows an estimate of the costs of measures for dikes.

Costs not included in the table, but which could be important:
- costs of clearance activities, preceding the actual realisation
- costs of storing or processing the top soil to be removed
    - these costs can be high for contaminated soil
    - there could be a slight yield from foreland improvement where clean sand is obtained
- purchasing ground; this often plays a role in a piping berm
- costs of temporary use during the work, these depend on the space taken up and the use of land

Table 5.1 Estimated costs of piping measures for dikes

	aanlegkosten indicatief gulden/m'	**aanleg kosten[1] relatief**	**onderhouds- /beheers/ afschrijvings- kosten[2]**	**beperkingen in gebruik grond[3]**
ingraving van klei in voorland	500[4]	+	0	-
ingraving van klei in voorland bij bemaling	550[4]	+	0	-
pipingberm	300[4]	0/+	0	--
verticaal kwelscherm binnendijks	500[5]	+	+	0
verticaal kwelscherm buitendijks	1500[6]	++	+	0
filterconstructie	200[7]	0/+	++/+++	-
verhogen binnenwaterstand indien infrastructuur aanwezig is	--	0	0/+	0
verhogen binnenwaterstand indien geen infrastructuur aanwezig is	zeer variabel	++/+++	++	-

Noten:
[1]: 0 = geen kosten
[2]: 0 = uitsluitend visuele inspecties
[3]: hierdoor kan een waarde vermindering van de grond optreden
0 = geen beperkingen in gebruik
[4]: 10 m^2, 1 m dik, inclusief ontgraving en afvoer naar depot van ontgraven grond resp. bovengrond
[5]: scherm van 3,5 m diep
[6]: scherm van 10 m diep
[7]: aanbrengen geotextiel en bestorting, dik 0,2 m in een sloot met talud/bodem oppervlak van 4 m^2/m'
Toelichting:
De kosten zijn per strekkende meter dijk, uitgaande van:
- een kwellengte tekort van 10 m,
- bij de voorlandverbetering: 1,5 m ontgraven, 1 m klei aanbrengen, 0,5 m grond terugzetten en 1 m grond afvoeren,
- bij een kwelscherm binnendijks is aangenomen dat de diepte van het scherm circa 1/3 van het tekort aan kwellengte dient te zijn; als een kleilaag aanwezig is, wordt de diepte van het scherm groter,
- bij een kwelscherm buitendijks is aangenomen dat de diepte gelijk aan het tekort aan kwellengte dient te zijn; dit kan echter zeer variëren,
- kwelschermen: bentoniet, dik 0,5 m; een stalen damwand is circa 30% duurder,
- bij de filterconstructie is aangenomen dat het gaat om een sloot, met een oppervlak van bodem en taluds gelijk aan 4 m^2/m', waarin een geotextiel met een bestorting met een laagdikte van 0,2 m wordt aangebracht,
- kosten excl. BTW en voorbereiding, prijspeil april 1998, uitgaande van redelijk omvangrijke projecten.

estimated construction costs NLG/m'
relative construction costs[1]
maintenance/management/depreciation costs[2]
Limitations in use of ground[3]

ditching clay in foreland
ditching clay in foreland with drainage
piping berm
vertical cut-off wall inside the dike
vertical cut-off wall outside the dike
filter structure
increasing inside water level if there is an infrastructure
increasing inside water level if there is no infrastructure very variable

Notes:
[1]: 0 = no costs
[2]: 0 = only visual inspections
[3]: 0 = this could cause a reduction in value of the ground
0 = no limitations in use
4: 10m^2, 1 metre thick, including digging and input to depot of ground dug and top soil respectively
5: screen 3.5 metres deep
6: screen 10 metres deep
7: applying geo-textile and dumping, 0.2 metres thick in a ditch with slope/bottom surface of 4 m^2/m'

Explanation:
The costs per linear metre dike, based on:

- a 10-metre shortage in seepage length,
- for foreland improvement: digging 1.5 metres, application 1 metre clay, replacing 0.5 metre soil and drainage 1 metre ground,
- it is assumed that the depth of the screen is approximately one-third of the shortage in seepage length for a cut-off wall inside the dike; the screen's depth is greater if there is a clay layer,
- it is assumed that the depth must be equal to the shortage in seepage length for a cut-off wall outside the dike; however this may vary,
- cut-off walls: bentonite, 0.5 metre thick, a steel dam wall costs approximately 30% more,
- for the filter structure it is assumed that this involves a ditch, with a bottom and slopes equal to 4 m^2/m', in which the geo-textile is introduced with a layer thickness of 0.2 metres,
- costs excluding VAT and preparation, April 1998 price level, based on reasonably large projects.

5.6 The Influence of Trees and Roots

5.6.1 General

Little is known of the influence of trees on the creation of sand-carrying boils. As far as can be ascertained no extensive research is available on this subject. However, there is a need for guidelines, all the more so because it is not always desirable to keep trees away from flood defences on the basis of for Nature (LNC) considerations. A document has been drawn up as part of this Technical Report with an inventory of the various possible influences. The following guidelines have been drawn up, which are based mainly on qualitative considerations. Expectations are that the guidelines generally serve as safe starting points. These guidelines are meant as points of interest in the design. A much wider basis can be applied in a monitoring situation, where (local) experience in particular can also play a part.

This section gives guidelines to limit or prevent the influence of trees on piping and heave, in foreland and hinterland and on the flood defence. A distinction is made between dikes in the upper rivers area and dikes in the tidal rivers area and on sea.

The general rule is that trees in the foreland or hinterland, if present inside the critical seepage length or on a piping berm are not permitted if:

- the tree is in a bad state of health or maintenance, or cannot be maintained properly
- the tree has or had insufficient space to develop a complete root system
- the tree is not suitable for the local conditions

If trees occur near the dike which do not meet these requirements, the area where these trees are located must not be included in the critical seepage length for the evaluation of piping.

All trees within the critical seepage length must be maintained properly, irrespective of what is explained below. After a tree has been dug up or died, the root system must be removed.

The guidelines are given step by step, ranging from a general to a detailed evaluation.

5.6.2 Foreland

Sea dikes

Trees in the foreland will not occur at sea dikes. Trees are not allowed to grow here anyway, in view of the substantial risk of being uprooted.

Dikes in tidal rivers area

1. The clay layer in the foreland, in as far as there are tree roots, must not be included in the calculation. The clay layer below a level of one metre under the groundwater level may be included in the calculation. The groundwater level must be taken at the lowest river water level which has occurred for a continuous period of one month or longer in the past 50 years.
2. If the trees come above water at design water level hardly or not at all, the thickness of the clay layer thickness need not be reduced.
3. The guidelines for dikes in the upper river area can be followed, if it can be proved that the risk of uprooting during high water is negligible.

Dikes in upper rivers area

1. Clay layer's thickness

If the clay layer extends more than one metre under the groundwater level, then trees in the foreland are permitted. The groundwater level must be taken at the lowest river water level which has occurred for a continuous period of one month or longer in the past 50 years.

2. Single tree/group of trees

A single tree is permitted in the foreland. Smaller groups or rows of trees are also permitted if they cover less than 20% of the foreland. In the management plan the provision must be included that the roots of dead or removed trees must be removed. The risk of uprooting due to floating ice must also be considered in the evaluation. If this is a substantial risk then the clay layer near the trees must not be included in the calculation.

3. Other

In the other cases the foreland must not be included in the calculation, unless:

- the porosity of the clay layer is reduced from ground surface to one metre below the groundwater level; the reduction varies from 0% if 20% of the foreland is covered with trees, to 50% if the foreland is completely covered with trees. The remark as mentioned in point 2 on floating ice also applies here. Roots of dead or removed trees must be removed.
- the influence of the trees can be further specified on the basis of further research.

5.6.3 Hinterland

Evaluation at dikes in the tidal rivers area and sea dikes
At these dikes during the design water level an extremely high wind speed will occur. The risk of uprooting is therefore taken into consideration.

1. If there is no danger of cracking, after trees have been uprooted, then trees are permitted in the critical seepage length. The depth of the hole due to uprooting is taken at two metres, unless a more accurate estimate is possible based on the local conditions.
2. In principle in other cases trees in the hinterland, inside the critical seepage length are not permitted.
3. Outside the critical seepage length no requirements are made with regard to piping. An increase in seepage and the increased risk of boil forming, sand-carrying or not, must be taken into account.
4. The above-mentioned guidelines may be deviated from, if a specific evaluation can point out that there is no increased risk of piping.

Evaluation at river dikes

1. If the clay layer occurs more than one metre under the groundwater level, then trees are permitted in the hinterland (figure 5.6). The groundwater level must be taken at the lowest groundwater level which has occurred for a continuous period of one month or longer in the past 50 years. The groundwater level can be estimated on the basis of the water board's and river water level's management levels.
2. Single trees or smaller tree groups or rows are permitted in all other cases.
3. Larger groups of trees are not permitted inside the critical seepage length.
4. Outside the critical seepage length no requirements are made with regard to piping. An increase in seepage and the increased risk of boil forming, sand boils or otherwise, must be taken into account.
5. The above-mentioned guidelines may be deviated from, if specific evaluation can point out that there is no increased risk of piping.

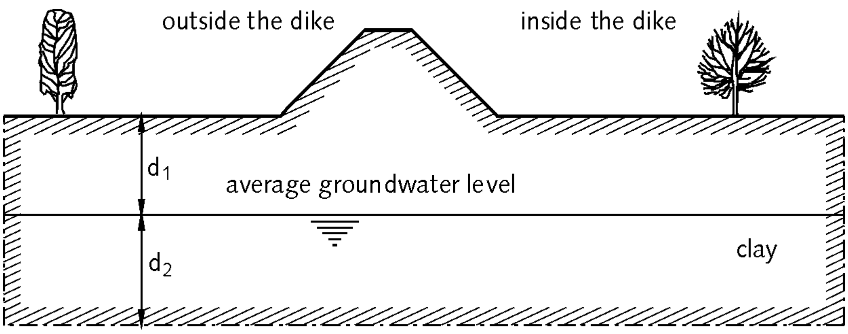


Figure 5.6 The clay layer's thickness at trees: if d_2 >1m, then trees are permitted

outside the dike
inside the dike
average groundwater level
clay

5.6.4 Piping berm
The evaluation of trees on a piping berm is the same as for trees in the hinterland. If trees are desired on the piping berm, while this is not permitted according to the evaluation of the hinterland, an additional height may be added, in such a way that cracking will not occur if uprooting does. The depth of the uprooting hole must be estimated, depending on the type of tree.

5.6.5 Dike body

Trees on the bottom part of the slopes are evaluated according to the guidelines for foreland and hinterland. Although, trees on the dike body have no influence on piping.

5.7 Piping at pipelines

5.7.1 General

Pipelines can lead to contact rings/short circuits in covering packages and/or concentrated groundwater flow. This can reduce the protection against boil forming and piping. The extent to which this occurs, depends principally on the construction of the flood defence: the position of the pipeline with regard to the flood defence. Boil forming and piping can also be initiated due to a calamity with the pipeline itself. Calamities which can occur are pipeline fractures, the raising of the pipeline etc.

For the evaluation of the effect of pipelines on boil forming and piping, the pipeline is first checked to see if it is located inside the flood defence's safety zone (see figure 5.7). The safety zone is made up of the disturbance zone (the distance to the pipeline in which erosion and disturbance can occur due to leakage, fracture or an explosion) and the stability zone (the breadth of the terrain strip located along the flood defence which must not be disturbed in connection with the stability of the flood defence.

See NEN 3651, Chapter 4 for the calculation of the disturbance zone and the stability zone. If the pipeline lies within the safety zone additional requirements must be made in connection with the protection of flood defence.

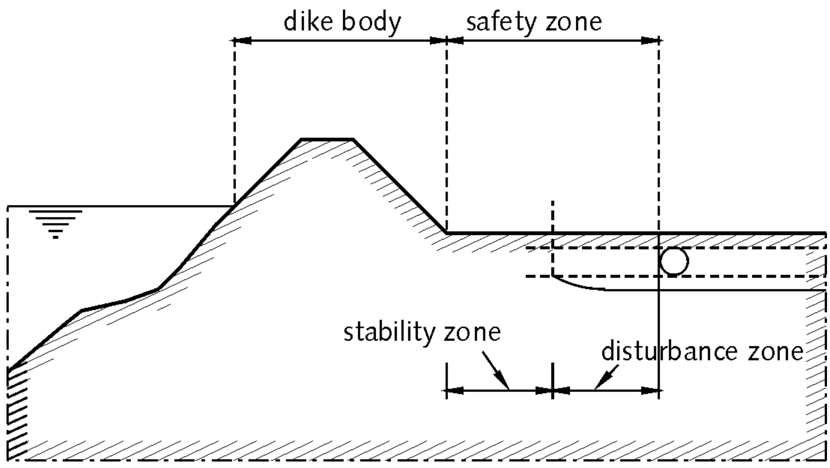


Figure 5.7 The flood defence's zones
dike body
safety zone
stability zone
disturbance zone

5.7.2 Pipelines parallel to the flood defence

The requirements which must be met by pipelines parallel to the flood defence, depend on the zone in which these pipelines are located (also see figure 5.7).

In principle, pipelines in the flood defence's profile (i.e. in the dike body between the inside and outside toe) are not permitted. A well-founded argument based on the specific situation must be given for exceptional cases.

The following must be proven with regard to pipelines outside the profile, which are unavoidable due to planning within the safety zone:

- the pipelines are designed, calculated and realised according to NEN 3650, 3651 and 3652 standards and the Dutch NPR 3659 Code Of Practice or according to the Pipeline Code drawn up by the Province of Zuid-Holland (which is generally less strict).

- the stability of the flood defence is guaranteed. Checking the piping mechanism is part of this. For example, this involves the influence of the pipelines, perhaps after erosion as a result of a pipeline fracture, on the covering layer's resistance.

Requirements are needed with regard to digging work for maintenance or repairs to the pipelines, depending on the position of the pipelines.

5.7.3 Pipeline crossings

A flood defence's safety zone is not limited by depth. This means that pipeline junctions by definition lie within the flood defence's safety zone. All crossing pipelines must be designed, calculated and realised according to the [NEN 3650/3651/3652] standards and the Dutch Code of Practice [NPR 3659] or the Pipeline Code. This includes requirements with regard to the need for a replacement flood defence and the evaluation of geo-technical stability and piping, in addition to requirements regarding the pipeline's strength. All pipeline crossings must comply with the rules in Chapter 7 of the *Guide on Water-retaining Hydraulic Structures and Special Structures; Design Basis Memorandum* [TAW 1997]. This concerns additional facilities, with regard to means of closure.

Mechanical influences (forces, movements) outside the so-called influence zone must not effect the pipelines in the safety zone. The calculation of stability of the pipelines therefore applies to a wider zone (see [NEN 3651] article 4.3).

It is important to make a distinction in the position of the pipeline crossing with regard to the flood defence, for the evaluation of the pipeline crossing.

Pipeline crossings positioned higher at the flood defence

These are mostly the conventional pipeline crossings which have been built in open digging sites or with the help of a pressure feed. The following distinctions are made within this main type (see figure 5.8):

- included in the dike body (this situation most common)
- positioned at ground surface level, completely or partially. In principle this is not an acceptable design, because a minimum ground cover is usually required (NEN 3651). These crossings do occur in practice, mainly in provisional pipelines.
- horizontally driven drilling, exiting in the inside slope of the flood defence. In principle this is not an acceptable design either, due to the 'blow out' danger when drilling the pipeline. Nevertheless these crossings do occur. This solution could be permitted in very over-dimensioned dikes, in which a local disruption of the dike profile during the drilling process does not form an uncontrollable threat. The dike manager naturally has to evaluate this.

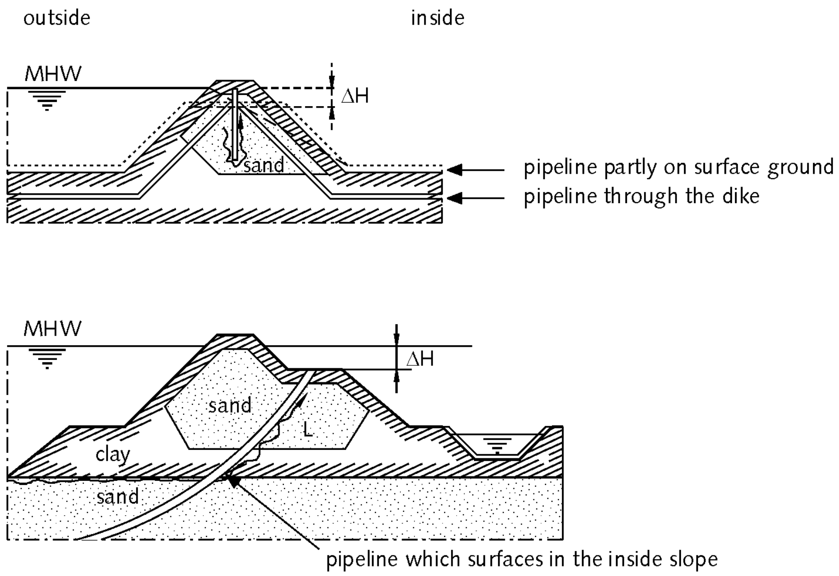


Figure 5.8 Pipeline crossing which occur in practice (In principle the pipelines which are not covered and the pipelines which surface in the inside slope are not acceptable)

If a replacement flood defence (sheet piling or coffer dam) has been placed, which is often required with crossings which do rise above MHW, in most cases piping will not come into play.

If this is not the case, and there is cut-off wall and/or clay coffer at the most, the resistance to piping must be evaluated in greater detail. The hydraulic head to be retained is to an important degree determined by the height of the exit point. See ‘Hydraulic Structures’ for the methods applied (section 5.3).

Deep-lying piping junctions
This usually concerns horizontally driven drilling (see figure 5.9). It must be checked whether alternative seepage lines cause a shortening of the seepage line in the undisturbed situation (without a pipeline). The Lane method is advised here. The critical hydraulic head remains equal to that in the original situation with a pipeline. See [NPR 3659 / NEN 3651].

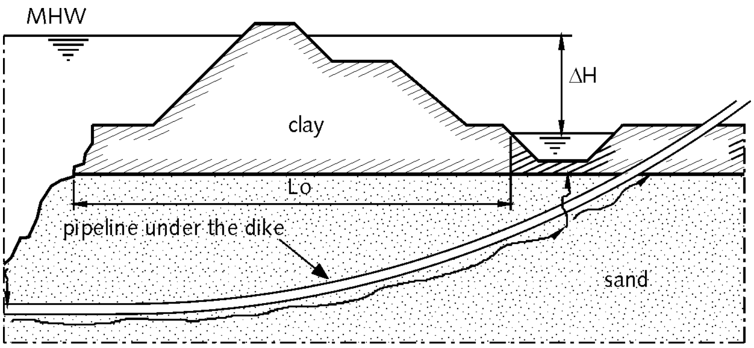


Figure 5.9 Deep-lying piping crossing

A detailed drawing of the pipeline’s trajectory through the covering layer is of significant importance. If, in the original situation, the protection against piping is achieved through a layer, a watertight trajectory or a filter structure is a condition of maintaining the original protection.

5.7.4 Other recommendations
Situations in which the resistance to piping can decrease through failure of the pipelines (pipeline fracture) must also be checked. For safety reasons it is recommended to also base this check on MHW. In the case of pipelines which have been placed deep in the ground, erosion due to pipeline fracture will not occur or only far enough away from the flood defence.

During management the greater effect of a local interruption in a covering layer on the foreland must be taken into account if a pipeline crossing occurs. A normative seepage line can be created via the pipeline crossing.

Any problems which are ascertained during the realisation must also be included in the survey of seepage line problems, for example fissures in the dike or the surface ground during the drilling process, as well as data obtained from work to the pipelines.

5.8 Guidelines for Management

The management of a flood defence aims to maintain the water retaining capacity. In most cases the water retaining capacity required will be described more or less accurately by design or monitoring preconditions. The design aims to maintain these preconditions.

General points of interest for management, with regard to piping are:
- digging and erosion of shores and foreland
- building
- plants
- perforations of clay layers due to other activities

If there are no design preconditions then these aspects must be followed in fairly wide strips inside and outside dikes. If there design preconditions, the breadth of the strips must be determined reasonably accurately.

Specific points of interest may be required in addition to general points of interest, if additional preconditions have been used for the design or the monitoring. The aspects mentioned will be further explained.

Digging and erosion
The seepage length can decrease due to digging outside the dike. This immediately increases the danger of piping. Digging in the foreland breadth included in the calculation of the design or monitoring, is only permitted if a sufficiently watertight clay layer is retained or added.

Digging inside the dike can cause:
- cracking, which did not occur previously
- an increase in the hydraulic head

Digging in the piping berm is obviously not permitted. A new evaluation of piping must be done for other digging. This also applies to digging inside the dike of a piping berm.

Buildings
Buildings do not have a direct influence on piping, if the building and the foundation are kept outside the design profile. The foundation piles and utilities facilities are the most important points of attention with regard to building. Foundation piles can be permitted in certain cases. Especially if there is a clay or peat layer which is several metres thick. The types of piles must always meet certain requirements: the piles must have a good contact with the ground over the entire length. If it has been ascertained that cracking has not occurred inside the dike, a seepage line can still form along the piles. Further examination is then needed. If a vertical seepage line has already been taken into consideration for the evaluation of piping, then the piles will not deteriorate in the design situation.

Utilities pipelines which are placed inside the critical seepage length, or which can lead to a n erosion zone within the critical line, can form a potential piping threat. A leaking sewer for example can cause a seepage line which is completely invisible. These pipelines must be designed according to NEN 3650/3651/3652.

Checking the use of land near buildings is usually difficult. This can be another important reason not to allow building within a piping zone. If this unavoidable it is recommended to take more limited digging into account in connection with building gardens and so forth. Good agreements with occupants/users are essential.

Plants
The influence of trees near a dike on the water retaining capacity partly depends on the tree's condition. This can be influenced by the method of planting and maintenance. Only a few main aspects are discussed in general terms here. It is recommended that an expert in the field be consulted to draw up a planting and maintenance plan.

New plants
When planting, it is important to choose the conditions in such a way that the tree and roots can develop optimally during its entire life. The following aspects are important:

- The area in which the roots will grow must be even and not too compact. The roots growth will be reduced if the ground is too compact. This is particularly important when planting on a berm. In most cases a berm will be made compact during building, after which a hole is dug for the tree. In this case the roots will be limited mainly to this hole. If this is not sufficient for the tree concerned, there will be an increased risk of uprooting. This can be prevented by making the berm material less compact. A slightly higher construction level may be necessary to compensate the weight.
- The distance between the trees must be chosen in such a way that they have sufficient space to develop. If this is not the case the trees must be thinned out at a certain point. The remaining trees will have a limited ball of soil around the roots and a higher wind pressure. This increases the risk of uprooting.

- The type of tree must fit in with the local conditions of the type of soil, groundwater level and floods.

Management
In the guidelines for trees, section 5.6, the assumption is made that only healthy trees occur in the piping zone. The following guidelines for management apply in all cases:
- sick trees must be removed
- trees which cannot develop optimally due to soil type or groundwater level conditions must be removed
- the roots of dead trees or trees removed for whatever reason, must always be removed.
- if the tree is trimmed to a great extent, part of the root system could die. This is undesirable in connection with the flood defence's hydraulic requirements and can be prevent by regular trimming.

Perforation of the clay layer
Perforation of the clay layer, both inside and outside the dike must be prevented. If this is unavoidable, due to a soil survey or the laying of cables or pipelines, then it must be repaired adequately.

Specific points of interest
Specific points of interest concern specific measures or requirements for management, included in the design. These can be:
- filter structures and
- inside water level requirements.

6 Damage catalogue and emergency measures

6.1 Introduction

In the damage catalogue (appendix IV) an overview is provided of damage scenarios which may indicate boil forming and piping. The purpose of the damage catalogue is to enable assessment by the managers of damage which has occurred in a more or less uniform manner. The catalogue can also serve as an aid to help the manager to decide what (emergency) measures need to be enforced. The damage catalogue can also be used in the inspection of flood defences and to formulate the recommendations on the necessity of emergency measures or maintenance and repair measures for the flood defence on the basis of the inspection results.

A damage scenario is understood to mean a specific, observable, typical form in which a reduction in the quality of a (component of a) dike or water-retaining hydraulic structure can manifest itself. In this Technical Report a damage scenario relates solely to boil forming and piping.

The significance of a damage scenario depends on the type of damage scenario and the conditions in which it is observed.

Type of damage
Damage can relate to

- reduction in the quality of the flood defence. The assessment of a damage scenario which relates to a (possible) decline in condition, but as yet not to signs of boil forming or piping requires insight into the parameters which determine the resistance to boil forming and piping for the dike section or hydraulic structure under consideration. To achieve fast and proper assessment it is accordingly necessary to know the current resistance to boil forming and piping of every dike section and hydraulic structure. This is provided by the prescribed safety monitoring of the flood defence.
- signs of (the start of) boil forming or piping. If damage is observed which relates to (the start of) boil forming or piping short-term action is demanded in most cases. On the basis of this damage scenario the phase which the piping mechanism has reached can be estimated immediately. Based on this the urgency and the (emergency) measures to be taken can be determined. An analysis of the cause of the observed damage is also recommended parallel to this; the necessary data can only be determined at a later stage in many cases, however.

Circumstances
Damage can be observed in various circumstances.

- Damage during routine inspection at normal water levels mostly relate to the condition of the flood defence.
- Work in or nearby the flood defence can (temporarily) affect the condition of the flood defence. In the most extreme case boil forming or piping can even occur.
- During dike supervision at high water attention chiefly goes to damage which is directly related to boil forming and piping. Naturally, a fast and proper reaction is of great importance here. In addition it is important that one is alert to damage which indicates a decline in the condition of the flood defence. This is complicated however, by the fact that the foreland and the outside slope are usually out of view.

Explanatory photographic material and schematic illustrations are included in the damage catalogue for a number of damage scenarios. The tables 6.1, 6.2, 6.3 and 6.4 below refer to these aids. The numbers correspond to the photos in the damage catalogue. The codes S, K, D correspond to schematic illustrations for lock, chambered lock and dike cut.

The damage catalogue must be considered a green version. Use has been made of the information and photographic material available at the time it was drawn up. It will probably be supplemented and clarified in time once practical experience has been gained in the use of the damage catalogue.

For damage scenarios at high water in particular, it is important to gain an immediate impression of

- the phase which the process of boil forming or piping has reached;
- the urgency in relation to the action to be undertaken by the observer;
- any measures to be introduced.

6.2 Damage scenarios at dikes

6.2.1 Decline in condition during routine inspection

Damage scenarios can be observed during routine inspections, which may indicate that the condition of the dike section is declining. In table 6.1 these damage scenarios are mentioned in general terms. This must be specified in more detail when recording damage scenarios.

Table 6.1 Damage scenarios indicating decline in a dike section's condition

code	schadebeeld	schadecatalogus
1.1	afkalving voorland	1.1a ; 1.1b
1.2	graverijen (dier): – dassenburchten; – konijnenholen; – molshopen en -gangen; – muskusratten; – etc.	1.2a ; 1.2b 1.2c
1.3	graafwerkzaamheden (mens): – sloten, waterpartijen; – aanbermingen; – leidingen / leidingkruisingen; – objecten (woningen, windmolens); – kleiwinning; – etc.	1.3a 1.3b 1.3c
1.4	bomen: – verzwakte boom; – kuil door ontwortelde boom; – etc.	1.4a; 1.4b

code
damage scenario
damage catalogue

1.1 crumbling foreland
1.2 digging (animal);
- fox holes;
- rabbit holes;
- mole hills and tunnels;
- muskrats;
- etc
1.3 digging work (man)
- ditches, water gardens;
- verges;
- pipes / crossing of pipes;
- objects (houses, windmills);
- clay extraction;
- etc
1.4 trees
- weakened tree;
- hole due to uprooted tree;
- etc

First of all an assessment must be made of whether the damage scenario in question can have consequences for this specific dike section for safety with reference to piping. If the answer is yes, then an assessment must be made of whether specific inspections will suffice for the time being to follow the development of damage, or whether measures must be taken to repair the damage before the coming high water period.

6.2.2 Piping/boil forming at high water

As a result of the recent high water levels a study was conducted into dike assessment at high water [DWW, 1996]. The study served as the basis for the interpretation of table 6.2. As this is a case of concrete indications of piping and the place where the damage scenario is expected is more or less fixed, in general an indication can already be given of the phase, the urgency and the measures.

Table 6.2 Damage scenarios, possibly indicating boil forming or piping at a dike section (phase, urgency and measures if damage is observed at high water)

code	schadebeeld	fase	urgentie	maat-regel	schadecatalogus
2.1	hoge vochtigheid grond binnentalud	A	i	-	2.1
2.2	hoge vochtigheid achterland	A	i	-	2.2
2.3	omhoog komen / bewegen achterland (niet door instabiliteit)	-	iii	f	2.3
2.4	stijging slootpeil / verhoogde afvoer sloten	A	I	-	2.4
2.5	stroming zichtbaar aan het wateroppervlak in sloten	B	ii	a	2.5
2.6	lokaal water beneden uit binnentalud (niet door neerslag)	B	I	a	2.6
2.7	opborrelend water, geen zand meegespoeld; treedt op in geconcentreerd gebied	B	ii	b	2.7
2.8	water spuit naar boven	B	ii	b	2.8
2.9	vertroebeling slootwater	B	ii	a	2.9
2.10	zand onder aan talud (niet door graverijen)	C	ii	b	2.10
2.11	gat onder aan talud (niet door graverijen)	C	ii	b	2.11
2.12	zand in slootwater	C	iii	c	2.12a; 2.12b
2.13	zandmeevoerende wel op maaiveld; kratervorming	C	iii	c	
2.14	zandmeevoerende wel in sloot; krater boven slootpeil	D	iii	d, e	
2.15	verzakken binnentalud / achterland (niet door graverijen)	D	iii	d, e	2.15a; 2.15b

code damage scenario
phase
urgency
measure
damage catalogue

2.1 high dampness level in side slope ground
2.2 high dampness level hinterland
2.3 raising/movement hinterland (not due to instability)
2.4 increase ditch level/raised drainage ditch
2.5 flow visible on the water surface in ditches
2.6 local water under from outside slope (not due to precipitation)
2.7 bubbling water, no sand conveyed; occurrence in concentrated area
2.8 water spurts upwards
2.9 cloudy ditch water
2.10 sand under the slope (not due to digging)
2.11 hole under slope (not due to digging)
2.12 sand in ditch water

2.13 sand boil on ground surface; crater forming
2.14 sand boil in ditch; crater above ditch level
2.15 subsidence inside slope/hinterland (not due to digging)

Under the code with reference to phase, urgency and measures the following is understood
Phase of boil forming and piping
A seepage inside the dike
B boil forming (concentrated seepage) inside the dike
C forming of sand-carrying boils on the ground surface level or in ditches;
D enlargement of hollow spaces under the dike, piping;
E collapse of hollow spaces under the dike, sagging of the dike
F carrying away of the remainder of the dike by overflowing, wave overtopping or erosion.

Urgency (action to be carried out by observer)
i follow up and report usual inspection in the scope of the regular inspection and maintenance;
ii increased supervision is recommended. Intensify inspection and report findings;
iii report immediately so that an assessment can be made of the need for emergency measures in the short-term and if so, which.

Measures at high water
a check whether sand is conveyed;
b cover with geo-textile and sandbags so that water is not stemmed (and continue to check if sand is conveyed);
c check how much sand has come out of the well and whether the quantity of sand is on the increase or the decrease: enclosing with geo-textile and sandbags;
d introduction of piping berm plus inspection vicinity for new boils
e increase water level in ditches (provided permissible in relation to stability)
f assess safety after cracking

6.3 Damage scenarios at hydraulic structures

6.3.1 Decline in condition during inspection
During a routine or special inspection damage scenarios can be observed which may indicate the declining condition of a hydraulic structure. In table 6.3 these damage scenarios are mentioned in general terms. This must be specified in more detail when recording damage scenarios.

Table 6.3 Damage scenarios possibly indicating decline in a hydraulic structure’s condition

code	**schadebeeld**	**schadecatalogus**
3.1	afkalving voorland	
3.2	graverijen naast het kunstwerk (dier): – dassenburchten; – konijnenholen; – molshopen en –gangen; – muskusratten; – etc.	
3.3	graafwerkzaamheden naast het kunstwerk (mens): – sloten, waterpartijen; – leidingen / leidingkruisingen; – objecten (woningen, windmolens); – etc.	
3.4	stroming zichtbaar bij wateroppervlak tijdens gebruik van het gesloten kunstwerk (bv. tijdens schutten)	S; K
3.5	disfunctioneren filters of filterconstructies (aansluiting met kunstwerk onvoldoende, filter dichtgeslagen, ballastlaag plaatselijk verdwenen)	S; K
3.6	waterdichtheid van vloeren en aansluitingen onvoldoende (open voegen, scheuren)	3.6a; 3.6b; 3.6c; 3.6d; 3.6e; S; K; D

3.7	deformatie van grond + kunstwerk (kan zowel oorzaak als gevolg van piping vormen)	S; K; D

code
damage scenario
damage catalogue

3.1 crumbling foreland

3.2 digging alongside the hydraulic structure (animal);
- fox holes;
- rabbit holes;
- mole hills and tunnels;
- muskrats;
- etc

3.3 digging work alongside the hydraulic structure (man)
- ditches, water courses;
- pipes / crossing of pipes;
- objects (houses, windmills);
- etc

3.4 flow visible on the water surface during use of the closed hydraulic structure (e.g. during stemming)
3.5 malfunctioning filters or filter structures (connection to hydraulic structure insufficient, filter shut, ballast layer lost in places)
3.6 water impermeability of floors and connections insufficient (open joints, fissures)
3.7 deformation of soil + hydraulic structure (can be cause or consequence of piping)

As at dikes (section 6.2.1) first of all an assessment must be made here of whether the damage scenario in question can have consequences for safety in relation to piping for this specific hydraulic structure. If that is so, an assessment must be made of whether specific inspections will suffice to follow the damage development, or whether measures must be introduced to repair the damage before the coming high water period.

6.3.2 Piping/boil forming at high water
In table 6.4 an indication of phase, urgency and measure has not been included, because at hydraulic structures this will depend greatly on the place where the damage is observed and the specific characteristics of the hydraulic structure.

Table 6.4 Damage scenarios, possibly indicating boil forming or piping at a hydraulic structure

code	**schadebeeld**	**schadecatalogus**
4.1-4.15	schadebeelden 2.1 t/m 2.15 uit tabel 6.2 – naast kunstwerk – achter uitstroming kunstwerk, t.p.v. filterconstructie – achter uitstroming kunstwerk – in het kunstwerk, achter het afsluitmiddel	4.12; K; D
4.16	deformaties/verzakkingen/bezwijken kunstwerk	K
4.17	deformaties/verzakkingen grond naast kunstwerk	

code
damage scenario
damage catalogue

4.1-4.15 damage scenario 2.1 through 2.15 from table 6.2
- alongside hydraulic structure;
- behind outflow hydraulic structure at filter structure;

- behind outflow hydraulic structure;
- in the hydraulic structure, behind the means of closure

4.16 deformation/sagging/collapse hydraulic structure
4.17 deformation/sagging/collapse ground alongside hydraulic structure

In assessing the damage scenarios 4.1 through 4.15 (such as 2.1 through 2.15) the following is applicable:

- In damage scenarios in or behind the inside slope immediately alongside the hydraulic structure the same assessment of phase and urgency is applicable as in table 6.2. If the same damage scenario does not occur at the site of the undisturbed dike, it is probably a case of horizontal seepage.
- In damage scenarios behind the hydraulic structure it is important to know whether a filter structure is present at the site. In the case of a properly functioning filter damage scenarios 4.1 through 4.6 which indicate a controlled water flow do not necessarily mean that the damage is alarming (low urgency).
- In damage scenarios behind the hydraulic structure where no filter is present the assessment in conformance with table 6.2 applies. This is also valid for damage scenarios in the hydraulic structure itself (at the site of an open floor structure or at the site of fissures or open joints).

In the case of deformations, sagging or collapse of the hydraulic structure or the adjacent dike body then there is the highest urgency.

Depending on the place and the cause of the damage, the measures to be considered are

- introduction of emergency dumping at open joints, fissures, or at the site of malfunctioning filters
- establishment of a gradual hydraulic head (if possible)
- complete closure of the hydraulic structure with ground body (provided the strength of the hydraulic structure and the connecting dike body permits it)
- increasing the inside water level (if possible).

6.4 Report and analysis of damage scenarios

For a good analysis of the damage scenario it is desirable that additional information is collected in addition to the damage scenario. It is recommended that the following matters be in any case noted in the damage inspection

1. the observed damage scenario (reference to damage catalogue);
2. the location: dike pile and place in cross section, or hydraulic structure and place in relation to hydraulic structure
3. scale of the damage, and if possible first urgency assessment;
4. conditions: weather conditions, water level;
5. particulars: maintenance situation, state of means of closure at hydraulic structure etc.

In addition, it is recommended that the damage is recorded via drawings and photographs.

In the definitive record of the cause it is important to include the specific data on the location where the damage scenario is observed. This relates to the following matters, which are already collected for the test or during the routine inspection.

1. Is the set-up of the flood defence and the state of any structures known?
2. Is the safety with respect to boil forming and piping known? What parts of the flood defence make an important contribution to this safety?
3. Is there a lower ground surface/deeper ditch in places?
4. Have boils or sand-carrying boils been observed at the location in the past? If so, at what distance to the dike and at what hydraulic head (that is, at what outside and inside water level?) If not, what falls have already been retained?
5. Are measurements from measuring tubes, water pressure meters available (past and present)?
6. Was there recent digging work in front of or behind the dike?
7. Do pipes cross etc in the direct vicinity?

A detailed analysis takes place after the first urgency determination, on the basis of which the necessity of emergency measures can be determined to prevent further damage growth.

7 Calculation Examples

7.1 River Dike I

7.1.1 Description

For this case a dike has been taken in the upper rivers area. The dike stretch is 2km long, from hectometre post (hmp) 1 to 21. To simplify the case it is assumed that the dike section fulfils the requirements with the exception of the requirements set with respect to piping.

Topography

The dike is at a distance of around 300m from the river's summer bed. An oxbow is present outside the dike over a length of approximately 150m, between hmp 14 and hmp 15.5. In the same stretch there are some buildings inside the dike. The site adjacent to the dike beyond this stretch is in use as agricultural land.

The ground surface level inside the dike varies somewhat (see length profile inside of the dike). Furthermore, no ditches or water gardens are present inside the dike within the sphere of influence of the flood defence. The breadth of the dike is shown in table 7.1.1.

Table 7.1.1 Breadth of the dike
hmp
breadth dike [m]
breadth foreland [m]

hmp	breedte dijk [m]	breedte voorland [m]
1	30	1
2	30	40
3	30	40
4	35	25
5	35	25
6	30	40
7	30	40
8	30	40
9	30	10
10	30	40
11	30	40
12	30	40
13	30	40
14	30	1
15	35	1
16	35	1
17	35	1
18	30	1
19	30	1
20	30	30

Soil Composition

This is a clay dike which is built on a clay layer with varying thickness. In the length profile the thickness of the clay layer inside the dike is shown. The clay layer outside the dike is not intact everywhere. In table 7.1.1 the distance from the outside toe is shown, inside of which an intact clay layer is present. No remnants of channels were found in the soil survey. The thickness of the clay layer on the outside of the dike is at least 2m.

The first water-bearing sand layer here is a maximum of 20m thick. In figure 7.1.1 a schematic cross section is shown.

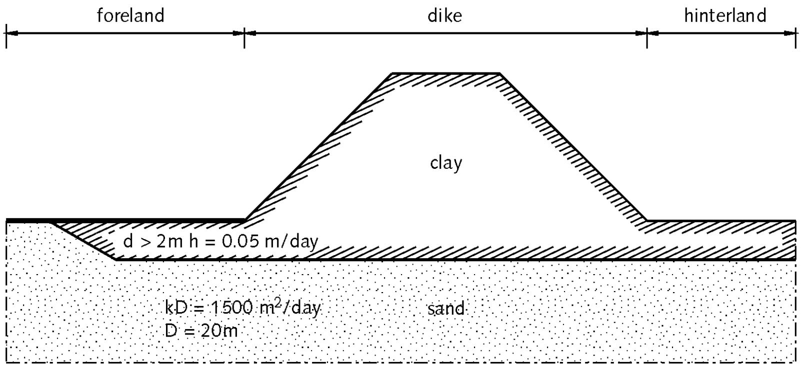


Figure 7.1.1 Cross section of river dike Case 1

Observations
No boils were observed during periods of high water. It was observed that the ground surface inside the dike is wet during high water. On the low parts of the ground surface on the inside of the dike there are even pools when the high water continues for some time.

Hydraulic Preconditions
- Normative high water (NHW): NAP +14.6m

Soil Characteristics
clay inside the dike:
- specific gravity: 17.5 kN/m3
- permeability (estimated): 0.05m/day

Sand:
- kD = 1500 m2/day
- grain size according to table below

Table 7.1.2 Grain sizes sand samples
sample

monster	d10 mm	d60 mm	d70 mm
1	0.25	0.46	0.58
2	0.14	0.38	0.42
3	0.13	0.26	0.32
4	0.09	0.26	0.35
5	0.13	0.29	0.34
6	0.22	0.39	0.43
7	0.22	0.42	0.39

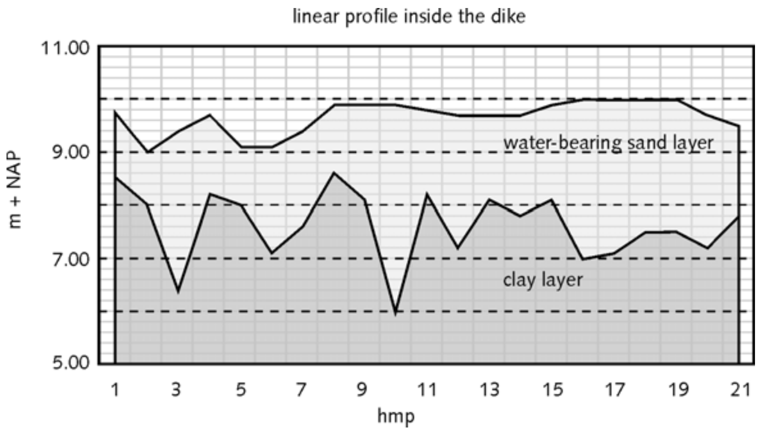


Figure 7.1.2 Linear profile inside the dike
Linear profile inside the dike
clay layer
water-bearing sand layer

7.1.2 Effect
It is observed that a piping sensitive composition is present. The fairly thin clay layer inside the dike is reason to suppose that cracking can occur at high water levels. This supposition is confirmed by the observations that the ground surface inside the dike at high water is regularly damp.

The seepage line is mostly horizontal. Only at the outflow point is there a vertical part, via a crack channel through the clay layer. Considering there is no sand on the surface inside the dike, no heave will occur. First a rough assessment is made using the method of Bligh.

First Assessment with the Method of Bligh
It is assumed that the clay layer inside the dike cracks and that the crack channel near the inside toe of the dike can originate. The entry length is calculated with the help of the range, in accordance with Eqs. 4 and 19. The range λ is in this case 245m. The entry length to be calculated is shown in the table below, depending on the breadth of the foreland.

Table 7.1.3 Theoretical entry length
Foreland breadth present [m]
Theoretical entry length [m]

Aanwezige voorlandbreedt [m]	Theoretische intreelengte [m]
40	39.6
30	29.8
20	20
10	10

The seepage length present can be determined by the sum of the theoretical entry length and the breadth of the dike.

The seepage line factor for the method of Bligh is in this case for the medium fine to medium coarse sand with d_{50} approximately 0.3mm, equal to 15, according to table 4.1.

The water level inside the dike can rise to the ground surface without obstruction. As a result the hydraulic head ?H is equal to the difference between the NHW and the ground surface level inside the dike.

For the thickness of the clay layer inside the dike d_1 cannot be assumed to be the thickness in a given cross section without further proof. In a very detailed determination seepage lines would have to be taken into consideration which do not run straight under the dike. In that case a safe approach is chosen, by which the minimum clay thickness occurring at that section is applied.

In the table below the results are given. The necessary seepage length, calculated with the rule of Bligh (chapter 4, formula 7) is shown in italics if it is greater than the seepage length present.

Table 7.1.4 Present and required seepage lengths according to Bligh
hmp
present v* [m]
present seepage lengths [m]
required seepage lengths [m]

hmp	**aanwezig v [m]**	**aanwezige kwelwegleng [m]**	**benodigde kwelweglengt [m]**
1	4,9	31	*68,6*
2	5,6	70	*79,1*
3	5,2	70	*73,1*
4	4,9	60	*68,6*
5	5,5	60	*77,6*
6	5,5	70	*77,6*
7	5,2	70	*73,1*
8	4,7	70	65,6
9	4,7	40	*65,6*
10	4,7	70	65,6
11	4,8	70	67,1
12	4,9	70	68,6
13	4,9	70	68,6
14	4,9	31	*68,6*
15	4,7	36	*65,6*
16	4,6	36	*64,1*
17	4,6	36	*64,1*
18	4,6	31	*64,1*
19	4,6	31	*64,1*
20	4,9	60	*68,6*

It appears that the present seepage length between hmp 7 and 8 and between hmp 10 and 13 is sufficient. In the rest of the section the present seepage length is, however, less than the required one, according to the method of Bligh. A more detailed assessment is therefore carried out.

Assessment with the Method of Sellmeijer
Important parameters in the formula of Sellmeijer are the permeability and the grain diameter. The permeability can be calculated on the basis of the grain distribution. This results in a permeability of $6*10^{-4}$ m/s. According to the NITG-NITO Groundwater Examination, the kD value is equal to $1500m^2$/day, corresponding to a permeability of $8.7*10^{-4}$. This difference is fairly small. The difference can originate for example if the sand samples are not completely representative of the whole sand layer, because they are not taken from over the whole sand layer, if the value according to the Groundwater Examination is not sufficiently applicable to the area under consideration. For the permeability an upper limit must be selected. The values do not deviate from each other too much in this case. Here, a permeability equal to $8.7*10^{-4}$ m/s is assumed.

The characteristic lower limit of the grain diameter d_{70} is according to the calculation (in conformity with appendix II, formula A.II.2) equal to 0.34mm.

With this data the formula of Sellmeijer, formula 9, can be applied. Direct application of the formula gives a critical retaining height for a given seepage length. In this case the required seepage length is of interest however for the given fall, because the hydraulic head cannot be influenced. The formula is solved in an iterative manner to come to this. In principle, this must be done for every sectional plane.

In the table below the calculated, required seepage lengths are shown. In the calculation account has also been taken of the length of the crack channel and the safety factor of 1.2, in accordance with table 4.2.

It is calculated that the ratio seepage length/hydraulic head is approximately 13.7. This means some gain in relation to the method of Bligh.

In the table below the required seepage lengths for both methods are given. Where the present seepage length is smaller than the required seepage length, it is shown in italics.

Table 7.1.5 Present and required seepage lengths
hmp
present [m]
present seepage lengths [m]
required seepage lengths Bligh [m]
required seepage lengths Sellmeijer [m]

hmp	aanwezig v [m]	aanwezige kwelwegleng [m]	benodigde kwelwegleng Bligh [m]	benodigde kwelwegleng Sellmeijer [m]
1	4,9	31	*68,6*	*62,6*
2	5,6	70	*79,1*	*72,7*
3	5,2	70	*73,1*	66,9
4	4,9	60	*68,6*	*62,6*
5	5,5	60	*77,6*	*71,2*
6	5,5	70	*77,6*	*71,2*
7	5,2	70	*73,1*	66,9
8	4,7	70	65,6	59,7
9	4,7	40	*65,6*	*59,7*
10	4,7	70	65,6	59,7
11	4,8	70	67,1	61,2
12	4,9	70	68,6	62,6
13	4,9	70	68,6	62,6
14	4,9	31	*68,6*	*62,6*
15	4,7	36	*65,6*	*59,7*
16	4,6	36	*64,1*	*58,3*
17	4,6	36	*64,1*	*58,3*
18	4,6	31	*64,1*	*58,3*
19	4,6	31	*64,1*	*58,3*
20	4,9	60	*68,6*	*62,6*

It can be concluded that the method of Sellmeijer also does not fulfil the requirements in relation to piping in most of the section.

7.1.3 Design of Measures

Now that monitoring has shown that the situation is not acceptable the measures are designed. The design principles are

- the normative high water level (NHW) = NAP 14.6m
- a life span of 50 years

Any rise in the NHW due to a rise in sea level is already included in the value.

The choice between the possible measures will generally occur on the basis of a consideration, in which the LNC values, costs and management & maintenance can play a role. In this case the technical aspects are at the forefront and therefore this consideration is not looked at in more detail. Here it is assumed that the lengthening of the horizontal seepage line by means of a foreland improvement or a piping berm. For the section in which an extension of the flood defence is not possible an alternative solution is worked out.

Piping berm
The most important characteristics of a piping berm are the breadth and the height.

The breadth is in principle equal to the seepage length shortfall. The exit point of the piping berm is moved from the inside toe to inside the dike. The values which are maintained for the hydraulic head and the thickness of the clay layer on the inside of the dike may be different to that at the inside toe. A check of these points and adjustment to the calculations is therefore necessary.

The height of the berm is calculated in such a way that cracking can no longer occur at the site of the berm. This calculation follows the following steps

- calculation of the potential limit inside the piping berm, for the clay layer thickness present
- calculation of the potential course between the entry point and the assumed exit point
- calculation of the required berm height, so that no cracking occurs in the case of occurring potential, bearing in mind a crack factor of 1.2.

The calculations can be made using the formulas in accordance with the *Guide on Designing River Dikes*, part I or the WATEX computer program. The gravity of the soil used in the piping berm is a parameter which plays a role in the calculations. In the choice of type of soil a number of considerations play a role. Due to the drainage of the dike, sand is preferred, or in any case a soil type that is more permeable than the soil in the soil structure. In addition, the future use of the site is also important. Use as grassland will mean that a layer of clay soil will have to be applied as a covering layer. An average specific gravity of 17.5 kN/m^3 is assumed for this case.

The results are shown in the table below. For this case it is assumed that the ground surface level and thickness of the clay layer inside the dike of the piping berm are equal to the values near the inside toe.

Table 7.1.6 Measurements piping berm
hmp
berm breadth [m]
berm height inside toe [m + NAP]
berm height end berm [m + NAP]
[1] Berm breadth measured from the current inside toe to end berm, rounded off to metres

hmp	bermbreedte [m]	bermhoogte binnenteen [m+NAP]	bermhoogte eindberm [m+NAP]
1	32	11,5	10,3
2	4	9,8	9,5
4	3	10,5	10,3
5	12	10,2	9,6
6	2	10,0	9,8
9	20	11,3	10,5
14	32	11,5	10,3
15	24	11,4	10,5
16	23	11,4	10,8
17	23	11,4	10,8
18	28	11,5	10,7
19	28	11,5	10,7
20	3	10,6	10,4
[1] Bermbreedte gemeten vanaf de huidige binnenteen tot eind berm, afgerond op hele me			

In the design of the piping berm the following considerations play a role

- the slope incline of the end of the berm to the ground surface can be selected independent of the future use and maintenance
- the berm height and the berm breadth will have an equal course in practice, by which the calculated value are minima
- the end of the berm must be selected in such a way that also seepage lines which are not straight in relation to the dike section are smaller than the required seepage line length
- considering the relative thin clay layer deep-rooted plants, such as trees, or buildings are not permitted
- depending on the future use it can be decided to introduce a settlement allowance; in general no or little secondary use is permitted on the piping berm; if this is unavoidable 0.5mm of extra ground (for example) can be added to create space for ploughs or for the laying of gardens.

Foreland improvement
The most important characteristics of the foreland improvement are the breadth and the water permeability. If the water permeability is selected in such a way that the distribution range is equal to the breadth of the foreland then the breadth of the foreland improvement is equal to the seepage length deficit To achieve this the range must be large enough. If a clay layer thickness of 1m is assumed with a permeability of 0.05m/day then the distribution range is a few metres shorter than the breadth of the foreland improvement. The required breadths then correspond to the breadth of the piping berm, according to table 4. The foreland improvement must connect with the clay layer present. Above the clay layer a covering layer of clay with a thickness of 0.5m is introduced, to prevent the drying out and disturbance of the layer by tree roots and erosion. In principle, no trees may be planted at the site of the foreland improvement.

Relief ditch and cut-off wall
Between hmp 14 and hmp 15.5 the introduction of foreland improvement or piping berm is not possible. A relief ditch filter structure or a vertical cut-off wall are possibilities.

The design of a relief ditch with filter structure is fairly simple. The ditch must extend down into the water-bearing sand layer and be provided with an adequate filter structure. This structure has a number of disadvantages, most notably the risk that the filter structure during high water no longer works as it should, so that piping can originate. In addition, the ditch must have a fairly considerable capacity. This possibility is not elaborated here.

The design of the cut-off wall can be based on the method of Lane (chapter 4, formula 10). The weighted seepage line coefficient is equal to six in this case. Also valid for the normative cross section at hmp 14 is ?H = 4.9m and Lh = 31m. In this it is calculated that Lv = 19.1m.

This cut-off wall can be positioned both inside and outside the dike. In the case of positioning outside the dike the clay layer must connect well with the screen. In the case of positioning inside the dike an increased potential under the dike must be taken into account. In that case a small inside berm will be necessary to prevent the underside of the inside slope from cracking.

If the cut-off wall is positioned inside the dike a heave situation is created. Dimensioning is according to the heave rules.

The models available for the heave rules assume a free outflow in sand. In this case a clay layer is present however. In the schematisation the clay layer is replaced with sand. This appears to be a safe assumption, because in that way the resistance which the upward groundwater flow is confronted with is not taken into consideration. In the spreadsheet the following values are filled in:
- L = 31m
- layer thickness sand outside the dike 20m
- layer thickness sand inside the dike 22m
- length of cut-off wall inside the dike 2m

For these values a permissible hydraulic head of 5.22m is calculated. That is less than the present hydraulic head of 4.9m.

However, in reality the clay layer is present, and has a thickness of approximately 2m. If a cut-off wall is positioned here with a length of 2m the situation does not change at all. The geometry and soil composition remains piping sensitive, and according to the assessment with the method of Sellmeijer, the piping criterion is not fulfilled. The solution to this is to change the geometry in such a way that piping no longer plays a role. That can be achieved by positioning through the clay layer and in the sand layer. A minimum size can be taken for the length of the cut-off wall under the clay layer, for instance 1m. The total length of the cut-off wall is then 3m. The spreadsheet is used for this situation too. A permissible hydraulic head of 7.06m is then calculated.

In determining the start and end point of the cut-off wall an overlap with the piping berm or foreland must be maintained which enables all possible seepage lines to fulfil the requirements calculated.

7.1.5 Possible deficiencies and damage scenarios

Despite the measures taken, after the improvement, during high water, phenomena may occur which appear to indicate a piping problem. In the table below the most important of these are summarised, and possible causes stated.

Table 7.1.7 Damage scenarios after improvement

waarneming	**mogelijke oorzaak**	**invloed op pipinggevoeligheid**	**maatregel**
maaiveld binnendijks nat	kwel	geen	geen
welvorming binnendijks van de pipingberm (niet zandmeevoerend	door de aanleg van de pipingberm kan een opbarstkanaal ontstaan op een locatie waar dat voorheen niet ontstond	geen	geen
welvorming binnendijks van de pipingberm/binnenteen, licht zandmeevoerend	zie 'welvorming, niet zandmeevoerend'	geen	geen[1]
welvorming, in toenemende mate zandmeevoerend	een parameter in de berekening is niet goed ingeschat (intreelengte, dikte/doorlatendheid zandlaag etc.)	gevaar voor piping	opkisten; lokaal onderzoek uitvoeren na het hoogwater
welvorming, al dan niet zandmeevoerend, op de pipingberm	er is waarschijnlijk een watervoerende zandbaan die uitkomt in de berm	mogelijk gevaar voor piping	idem
zandmeevoerende wel binnendijks van het kwelscherm	mogelijk steekt het kwelscherm niet door de kleilaag heen	mogelijk gevaar voor piping	idem
[1]) In de praktijk is het niet eenvoudig vast te stellen of er wel of niet gevaar voor piping is, als er een zandmeevoerende wel wordt geconstateerd. Het is aan te raden altijd op te kisten, als de wel binnen een afstand van 18*ΔH vanaf de buitenteen ligt.			

observation

- ground surface inside the dike wet
- boil forming inside the dike in the piping berm (not sand boils)
- boil forming inside the dike in the piping berm/inside toe, light sand boils
- boil formation growing sand boils
- boil forming, either sand boils or otherwise, on the piping berm
- sand boils inside the dike of the cut-off wall

possible cause

- seepage

due to the introduction of the piping berm a crack channel can originate at a location where it did not before
see ‘boil forming, not sand boils’
a parameter in the calculation is not estimated adequately (entry length, thickness/permeability sand layer etc)
there is probably a water-bearing sand track which exits in the berm
possibly the cut-off wall does not reach through the clay layer

influence on piping sensitivity
none
none
none
threat of piping
possible threat for piping
possible threat for piping

measure
none
none
none
enclosing; conduct local study after high water
ditto
ditto

1) in practice it is not easy to determine whether there is a threat of piping, if a sand boil is observed. Enclosing is always recommended, if the boil lies within a distance of 18* ?H of the outside toe.

7.2 River Dike II

7.2.1 General/Geometry

A primary flood defence is considered in the upper rivers area. Over the length of the dike section the geometry of the water defence varies little or not at all. The dike has a crown breadth of approximately 4m and a gentle outside slope. The inside slope is substantially steeper.

A cross section representative for this dike section is shown in figure 7.2.1. Foreland is scarcely present on the river side of the dike. On the inside of the dike no tow ditch is present.

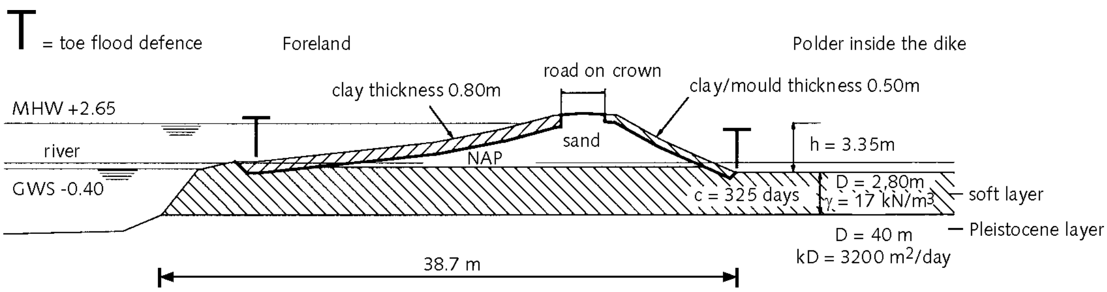


Figure 7.2.1 Cross section River Dike II with substrate composition
T = toe flood defence
Foreland
Polder inside the dike
road on crown
clay thickness 0.80m
clay/mould thickness 0.50m
river
sand
c = 325 days
soft layer
Pleistocene layer

In the polder inside the dike the land is used for agricultural activities.

Normative high water (NHW) = NAP+2.6m. Its development in time is shown in figure 7.2.2. The ground surface level inside the dike is at NAP -0.70m.

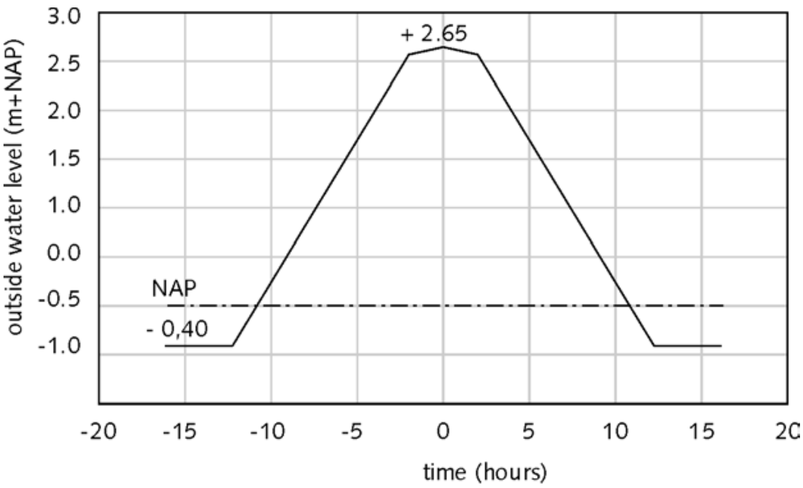


Figure 7.2.2 NHW and its development in time
outside water level
time

In the scope of the monitoring the flood defence has recently been assessed for all possible damage mechanisms, except piping. Study shows that the flood defence is assessed as ‘good’ on the damage mechanisms already studied. This means that the dike does not need to be strengthened as a consequence of insufficient stability or crown height for example. Only the piping mechanism still has to be assessed.

If piping was a problem any measures should preferably be carried out inside the protection zone described in the water board statute. The protection zone stretches to 20m from the toe of the flood defence. In figure 7.2.1 the locations considered to be toes are indicated on the cross section with the symbol ‘T’.

7.2.2 Soil composition and geo-hydrology
A soil survey has been carried out outside and inside the dike, and at the site of the crown. The results of the soil survey, which can be considered as representative for this dike section. are shown in figure 7.1.1.

There is a package of soft layers with a thickness of 2.80m in the polder inside the dike on the Pleistocene sand up to the ground surface. The volume weight of this package amounts to 17 kN/m3 on average. Laboratory tests show that this package has a permeability factor of 1*10-7 m/s. The groundwater chart gives a hydraulic resistance (c) of the covering package to be 325 days. For a consolidation coefficient on the underside of the soft package of layers a value of 1*10-7 m2/s can be maintained.

The flood defence consists of a core of sand. At the site of the outside slope there is a clay layer with a thickness of 0.80m. On the inside slope there is a 0.50m thick clay/mould layer. Under the flood defence there is also a package of soft layers up to the river bank, for which the same parameters can be maintained as for the package inside the dike. Under these layers the Pleistocene sand is at the same depth as the polder inside the dike (NAP -350m).

The groundwater chart shows that the first water-bearing package, belonging the Pleistocene deposit, is around 40m thick. This package almost completely consists of coarse, very permeable river sands. Only a top sand layer with a thickness of around 4.50m consists of fine sands. They are top sands deposited by the wind. The average permeability of the whole first water-bearing package is fairly large. For the kD value 3200m2/day can be maintained.

The groundwater chart also shows that the rise in the groundwater in this package under normal conditions in a winter period is around NAP -1.70m at the site of the inside toe of the dike. Groundwater extraction for drinking water in the vicinity affects this potential. In this case it can be assumed that groundwater extraction is guaranteed in the next five years (monitoring period) so that the given head will be maintained.

In conducting hand drilling inside the dike top sand samples were taken from the top layer of the Pleistocene. The grain distribution was determined from these samples. The grain distribution shows that there is little variation in the fineness of the top sand. All samples have a d50 between 0.140 and 0.210mm.

Of 10 samples taken from the top layer of the Pleistocene top sand near the inside toe of the dike, the values d10, d50, d60 and d70 are given (in mm) in table 7.2.1.

Table 7.2.1 Results of the sieve analysis
sample no.

monster nr.	d10 [mm]	d50 [mm]	d60 [mm]	d70 [mm]
1	0,087	0,163	0,182	0,211
2	0,084	0,182	0,199	0,221
3	0,056	0,162	0,179	0,207
4	0,053	0,158	0,182	0,213
5	0,047	0,152	0,173	0,201
6	0,099	0,193	0,214	0,232
7	0,054	0,151	0,172	0,189
8	0,092	0,206	0,225	0,251
9	0,071	0,142	0,156	0,165
10	0,059	0,197	0,223	0,260

7.2.3 Results

Assessment according to the method of Bligh (see graph Piping at dikes and section 5.2.4)
It is observed that the composition of the dike and the substrate is piping sensitive. Considering the limited thickness of the soft package of layers in relation to the height of NHW it is assumed that the top layer will crack under normative conditions.

There are no ditches in the vicinity of the inside toe of the dike. It may be assumed that the exit point is situated at the site of the inside toe. Figure 7.2.1 accordingly indicates a horizontal seepage length of 38.7m.

The seepage line factor of the piping sensitive sand for the method of Bligh can in this case be derived from the sieve analysis of the 10 samples provided. the characteristic estimate of the average value of d50 is calculated with the help of the formula d50,kar = d50,gem - t*s*1/ ? N (see appendix II), in which the random sample average d50,gem = 0.171mm and N = 10, the accompanying Student -t factor t(α=0.05) = 1.81 and the random sample standard deviation s = 0.022mm. It follows that d50,kar = 0.158mm. With the help of rough interpolation in table 4.1 we find a seepage line factor for the method of Bligh: C_{creep} = 17.

Considering the lack of ditches inside the dike the water level can rise without obstruction to the ground surface level. The hydraulic head is accordingly equal to the difference between NHW and the ground surface level.

The seepage length needed Lb is Lb $\geq$ Cw (?H - 0.3 d) (see formula 7, section 4.2.2). With ?H = 3.35m and D = 2.80m this results in Lb = 42.7m.

The seepage length present of 38.7m is smaller than that needed so a more detailed assessment is carried out. The seepage length is 4m to short.

Uplift/crack checking (see graph Piping at dikes and section 5.2.3)
In this case piping can only originate if the package of soft layers crack under normative conditions. The threat of cracking is very real if the water pressure (σ w) on the interface of the package of soft layers and the Pleistocene sand (at the site of the inside toe of the dike) is greater than the ground tension (σ gr).

Without carrying out groundwater calculations this can be determined simply with an upper limit approach, by which a rise to NWH is assumed to determine the maximum water pressure.

It thus follows

σ w = (2.65m +3.5m) * 9.81 kN/m3 = 60.3 kN/m2
σ gr - 2.8m * 17kN/m3 = 47.6 kN/m2

So σ w > σ gr so that cracking cannot be excluded and a more detailed assessment follows.

It is noted that the presence of a ditch in the inside toe can probably upgrade the ground tension at the interface due to tension distribution in the substrate with the help of distribution formulas.

Assessment according to the method of Sellmeijer (see graph Piping at dikes and section 5.2.5)
Considering the great thickness of the water-bearing package it is not expected that a favourable result will be reached using the method of Sellmeijer. For the sake of clarity the calculations according to the method of Sellmeijer have been addressed in the following.

The normative high water level must be tested against the following criterion (see section 4.2.3)

$$\Delta Hc \geq \gamma (\Delta H - 0.3 d)$$

where
ΔH = NHW - p.p
NHW = NAP +2.65m
p.p. + polder level (in this case ground surface level) = NAP -0.70m
γ = safety factor 1.2
d = length vertical seepage line = 2.8m
ΔHc = representative value of the critical hydraulic head for through piping

It follows that ?Hc 3.18m

The critical hydraulic head is determined with the help of the formulas of Sellmeijer. In these formulas, in addition to a number of parameters relating to the geometry, a number of parameters for the piping sensitive sand are needed and the permeability of the water-bearing package.

L = present horizontal seepage length = 38.7m
D = thickness of sand layer in which piping can occur = 40m
ρw = density of water = 10 kg/m3
ρp = density sand grains under water = 1.65 * 103 kg/m3
θ = rolling resistance angle = 41o
η = drag factor = 0.25
ν = kinematic viscosity = 1.33 * 10-6 m2/s
g = acceleration of the gravity = 8.81 m/s2

The permeability (*k*) of the sand package and the intrinsic permeability (κ) which can be derived from it and the erosion parameter of the piping sensitive sand (d70) must also be determined.

The kD value of the water-bearing package is 3200 m2/day at a thickness of this package of 40m. This results to a permeability coefficient k = 3200m2/day/ 40m =80m/day = 9 10-4 m/sec. So κ =k ν /g = 1.25 10-10m2.

The characteristic lower limit value of the average value of d70 is determined using the logarithmic values of the d70 (see appendix II, formula A.II.4). We find: d70,kar = 0.198mm.

Using the WATEX computer program Sellmeijer model the piping calculation according to the method Sellmeijer is carried out. For a present seepage length of *L* = 39.7m a ?H of 1.84m is calculated. That does not meet the requirement ?Hc 3.18m, which means that piping cannot be excluded on the basis of the rule of Sellmeijer, as suspected already.

Assessment of uplift/cracking stationary groundwater flow (see section 5.2.3)
At an earlier stage it was indicated that piping can only originate if the package of soft layers cracks under normative conditions. The threat of cracking is very real if the water pressure (σ w) on the interface of the package of soft layers and the Pleistocene sand (at the site of the inside toe of the dike) is greater than the soil tension (σ gr). By carrying out groundwater flow calculations the maximum potential in the sand can be determined, from which the water pressure can be derived.

Guiding principles
A stationary groundwater flow calculation can be carried out with the help of the WATEX computer program. In figure 7.2.3 the model schematisation of the cross section is given.

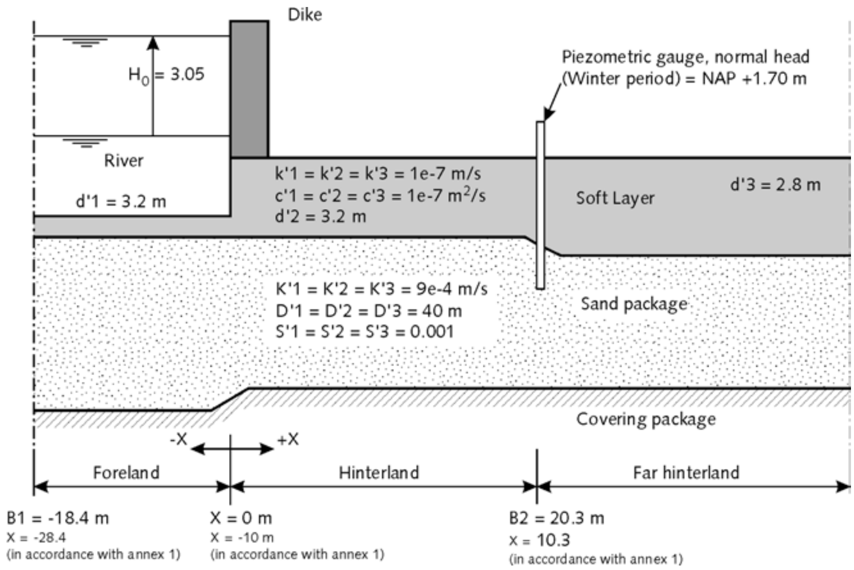


Figure 7.2.3 Model schematisation WATEX, model 3 section

In the model on an X coordinate of 20.3m for a hydraulic load Ho of 3.05m an increase in the head is calculated of 2.94m. For a head in the sand under normal conditions of NAP -1.70m this means a head during NHW of NAP +1.24m. The uplift safety U(x) is defined by the relationship between the weight of the covering soft layer package and the upwards water pressure of the potential in the underlying sad layer. The criterion is

U(x) $\geq$ 1.20 safety factor of 1.20)

From the calculation it follows U(x) = (17kN/m3 * 2.8m) / (1.24m + 3.5m) *9.81 kN/m3) = 1.02

It can accordingly be concluded that the criterion is not fulfilled. The calculated stationary potential in the sand is sufficiently high that the safeguard against uplift/cracking is inadequate. As a result piping is still not to be excluded.

Assessment uplift/cracking time-dependent groundwater flow (see section 5.2.9)
In the case in question the duration of the normative high water level is relatively short (24 hours). As a result it is expected that the calculated stationary potential of NAP +1,24m will not be reached. Therefore a more detailed calculation is calculated.

Using the WATEX computer program, model 3 section, a time-dependent groundwater flow calculation can also be carried out. A precondition of this is the course of the outside water level in time during NHW (see figure 1) is given in the program.

In the model on an X coordinate of 20.3m for a time-independent hydraulic load Ho of 3.05m and increase in the head calculated at 3.16m. For a head in the sand under normal conditions of NAP -1.70m this means a head during NWH of NAP +0.46m. In figure 7.2.4 the calculated heads in time for various X coordinates are given. The safeguard against uplift U(x) is accordingly

U(x) = (17 kN/m3 * 2.8m) / ((0.46m + 3.5 m) 8 9.81 kN/m3) = 1.22

It can be concluded that the criteria are fulfilled. The safeguard against uplift/cracking is sufficient so that piping can be excluded.

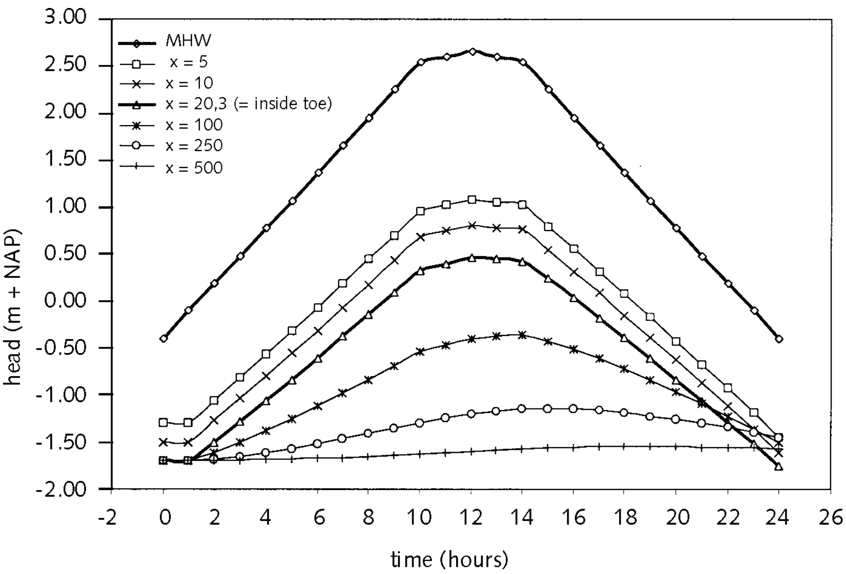


Figure 7.2.4 Calculated time-dependent heads
head
inside toe
time [hours]

Remarks

- In practice the parameters used in such groundwater flow models are calibrated using piezometric gauge observations. The many parameters needed for carrying out such a calculation (stationary or time-dependent) can then be estimated with various sets of piezometric gauge observations. Once the situation has been imitated as well as possible during the measurements the NWH can be extrapolated to determine the potential in the Pleistocene. In the case in question it can be said that the parameters given were found after extensive calibration of various sets of piezometric gauge observations.
- In the case it is proposed that the head in the groundwater in the Pleistocene sand under normal conditions in a winter period is around NAP -1.70m at the site of the inside toe of the dike. It must be remarked that this is kept so low by groundwater extraction for drinking water in the vicinity and that it can be assumed that the groundwater extraction is guaranteed for the next five years (monitoring period) which means that the given head will be maintained.

  If, after carrying out the time-dependent groundwater flow calculations it was concluded that piping could not be excluded, a piping berm could be proposed as a solution. For dimensioning of this improvement it is recommended in this case that the influence of extraction on the head in the water-bearing package be studied.

7.3 Sluice

7.3.1 Geometry, cut-off walls, foundation, water levels

(see hydraulic structures diagram, section 5.3.2)

General Description
A sluice is studied, which is part of a primary flood defence in the upper rivers area. The function of the sluice is to protect the inside harbour behind it from high water on the river. The sluice can be closed using a double set of mitre gates.

The sluice was built in 1955. Any renovation work only relates to means of closure and weighing works. No work has been carried out on the substructure, foundation and screens since construction, as far as can be established.

Hydraulic Preconditions
- Normative high water NHW = NAP +3.6m
- Level inside harbour h_b = NAP -0.1m
- hydraulic head to be retained ?H = NHW - h_b = 3.7m

Structure
Plans have been found in the archives, which indicate the following
- the sluice is a reinforced concrete tank
- foundation on steel
- no jointed structures
- various screens on inflow and outflow side: see table 1
- soil protection with filter structure is present at the inflow and the outflow
- good connection between clay and cut-off wall
- some important heights
  underside foundation NAP -3.5m
  topside Pleistocene alongside the sluice NAP -3.5m
  soil level at inflow and outflow NAP -2.5m

Soil Composition
- The flood defence, of which the sluice is part, is a clay dike on a thin impermeable package (underside at NAP -3.5m) The foundation level of the cut-off wall is precisely in the Pleistocene sand (medium grained to fine sand, dense Pleistocene package is approx. 25m)
- On the inside and inside slope alongside the cut-off wall a clay covering is present
- It is unclear what kind of soil was used in supplementation alongside the sluice. It is assumed that sand was used.

In figures 7.3.1, 7.3.2 and 7.3.3 the main measurements of the sluice are given, with the screens mentioned in table 7.3.1.

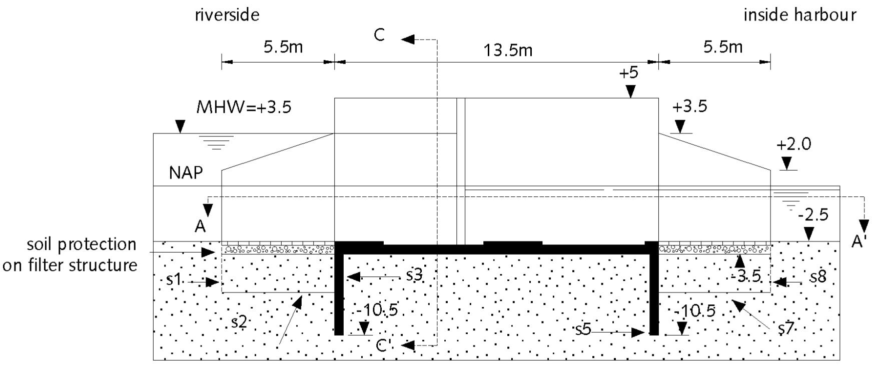


Figure 7.3.1 Longitudinal cross section (section BB’ in fig. 7.3.3)

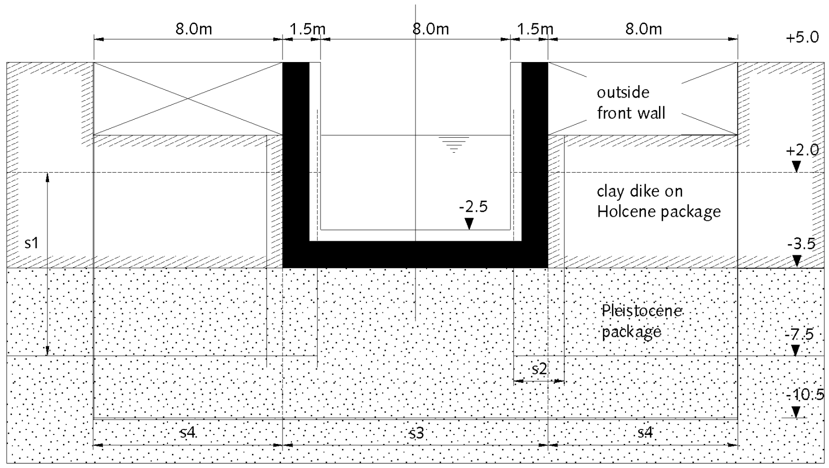


Figure 7.3.2 Cross section (section C-C’ in figure 7.3.1)
outside front wall
clay dike on Holocene package
Pleistocene package

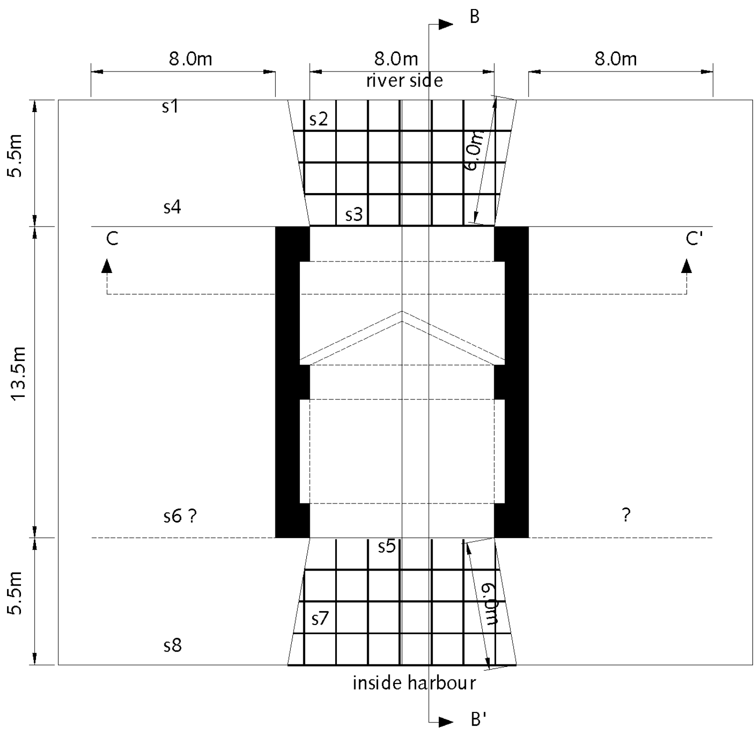


Figure 7.3.3. Plan view (cross section A-A’ in figure 7.3.1)
river side
inside harbour

Table 7.3.1 Sheet piling screen (steel profile)

Scherm (screen)	onderkant t.o.v. NAP	bovenkant t.o.v. NAP	breedte	opmerkingen
S1: scherm bij de teen van het naastliggende buitentalud	-7,5 m	+2,0 m	grote breedte	oeververdediging rivierzijde
S2: een schermwand aan weers-zijden van de instroomopening	-7,5 m	van +2,0 m tot +3,5 m	6 m	aansluitend op S3 en S4
S3: scherm onder de betonvloer, aan instroomzijde	-10,5 m	-3,5 m (onder-kant vloer)	hele sluis-breedte	aansluitend op S2 en S4
S4: scherm onder de buitenfrontmuur	-10,5 m	tot onderkant frontmuur	8 m (hele frontmuur)	aansluitend op S2 en S3
S5: scherm onder de betonvloer, aan uitstroomzijde	-10,5 m	-3,5 m (onder-kant vloer)	hele sluis-breedte	aansluitend op S7
S6: scherm onder de binnenfrontmuur	?	?	?	niet bekend is of dit scherm aanwezig is
S7: een schermwand aan weers-zijden van de uitstroomopening	-7,5 m	van +2,0 tot +3,5 m	6 m	aansluitend op S5
S8: scherm bij de teen van het naastliggende binnentalud	-7,5 m	+2,0 m	grote breedte	oeververdediging binnenhaven

Screen
S1: screen at the toe of the adjacent outside slope
S2: a screen wall on both sides of the inflow opening
S3: screen under the concrete floor, on the inflow side
S4: screen under the outside front wall
S5: screen under the concrete floor, on the outflow side
S6: screen under the inside front wall
S7: a screen wall on both sides of the outflow opening
S8: screen at the toe of the adjacent inside slope

underside NAP

topside NAP
from +2.0m to +3.5m
underside floor)
to underside front wall
underside floor
from +2.0m to +3.5m

breadth
large breadth
whole sluice breadth
(whole front wall)
whole sluice breadth
large breadth

remarks
bank defence river side
connecting to S3 and S4
connecting to S2 and S4
connecting to S2 and S3
connecting to S7
not known if this screen is in position
connecting to S5
bank defence inside harbour

7.3.2 Soil Survey
(see hydraulic structure diagram, section 5.3.3)

Initially, available data on the substrate and the composition of the flood defence beside the sluice is used.

7.3.3 Determine normative seepage lines
(see hydraulic structures diagram, section 5.3.4)

In the first assessment of the design the following principles are maintained.

- The structure , the screens, the connecting clay revetment and the filter on the inside are in good condition.
- No screen is in position under the inside front wall (so no S6)
- Soil supplementation adjacent to the sluice is composed of clay.

In figure 7.3.4 in the plan view the various normative seepage lines which are possible are given. Note that the seepage lines usually do not run via the structure but rather along the interface of clay and sand, so that they make diagonal connections. A short description of the various (sub) processes for piping are given below.

- sub-process 1: entry under screen S1, horizontal seepage line along the interface of clay and sand of screen S1 to the end of screen S4.
- sub-process 2: entry at filter under screen S2, horizontal seepage line along the interface of clay and sand of screen S2 to the end of screen S4.
- sub-process 3: horizontal seepage line along the interface of clay and sand of screen S4 to the end of screen S8, exit under screen S8.
- sub-process 4: horizontal seepage line along the interface of clay and sand of screen S4 to edge screen S5 and S7, exit under screen S7. Filter layer in position at exit point. This process is not relevant if the filter is functioning properly (= assumption).
- process 5: entry at filter under screen S3, horizontal seepage line under the structure, exit under screen S7. Filter layer in position at exit point. This process is not relevant if the filter is functioning properly (= assumption).
- process 6: entry at filter under screen S3, horizontal seepage line under the structure and along the interface of clay and sand of screen S3 to screen S8, exit under screen S8.

In the case of entry at the filter (sub-processes 2, 5 and 6) for the determination of the seepage length the thickness of the filter (approx. 1m) is not taken into consideration.

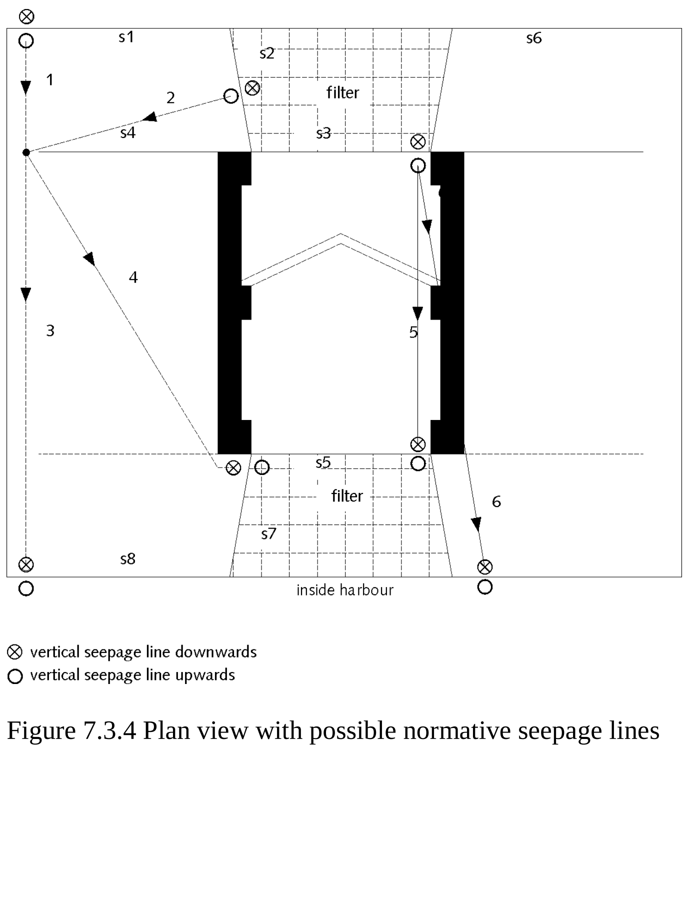


Figure 7.3.4 Plan view with possible normative seepage lines

7.3.4 Monitoring using Lane

(see hydraulic structures diagram, section 5.3.3)

Note that there is groundwater exits vertically at most seepage lines, precisely behind sheet piling. These seepage lines are cases of heave rather than piping. For most possible normative seepage lines no heave calculation method is yet available, so that the method of Lane must also be used for design calculations

-$\Delta H_{cr} \geq \Delta H$ with $\Delta H_{cr} = L/C_{w,creep}$

L is the current weighted seepage length following from the seepage line analysis

For the Pleistocene sand $C_{w,creep} = 7$ (fine sand)

In table 7.3.2 the vertical and horizontal seepage length and the weighted seepage length are given, calculated as $L_i = L_{v;i} + 1/3\ L_{h;i}$ (foundation on natural subsoil) for various combinations of sub-processes along the concrete structure or along the interface of clay and sand. The calculated critical hydraulic head ?H_{cr} is also presented.

Table 7.3.2 Possible seepage lines; assessment according to Lane

seepage line	**seepage length (m)**			**ΔH_{cr} (m)**	**remarks**
	vertical	**horizontal**	**total**		
1+3	5 + 4 + 4 + 5 = 18	5,5 + 13,5 + 5,5 =	26,2	(3,7)	possible check for heave
2+3	4 + 4 + 4 + 5 = 17	8 + 13,5 + 5,5 = 27	26,0	3,7	
1+4	5 + 4 + 4 + 5 = 18	5,5 + 15,7 = 21,2	25,1	n.v.t.	filter
2+4	5 + 4 + 4 + 5 = 18	8 + 15,7 = 23,7	25,9	n.v.t.	filter
5	7 + 7 + 4 + 5 = 23	13,5	27,5	n.v.t.	filter
6	7 + 7 + 4 + 5 = 23	13,5 + 5,5 = 19	29,3	4,2	

Table 7.3.2 shows that the seepage lines 1+3 and 2+3 are normative. The (permissible) critical hydraulic head according to Lane (?H_{cr} = 3.7m) just fulfils the necessary hydraulic head to be retained for this sluice (?H =3.7m). For seepage line 1+3 a useable heave calculation model is available; if the rule of Lane was not met, this calculation model may have been a solution. For the sake of completeness this is demonstrated in section 7.3.5.

7.3.5 Heave

(see hydraulic structures diagram, section 5.3.8)

For the situation in which there is a direct through seepage line, that is seepage line 1+3, the heave rules developed for this specific situation can be used. In annex I (figure A.1.2 (a)) a simple heave test is given.

The base data is

- length screen placed downstream: ds8 = 4m
- length screen placed upstream: dS1 = 4m
- thickness water-bearing package: D = 25m
- horizontal seepage length: Lh = 24.5m

So D/L = 1 and d/D = 0.16 are applicable. If the screen placed upstream is neglected (dS1 = 0) and the horizontal and vertical permeability is equal then: ?$H_{cr}/L = 0.25$. Or

critical hydraulic head seepage line 1+3: $\Delta H_{cr} = 6.1$m

That is a lot greater than the normative fall.

The first (provisional) conclusion is that the safety with reference to piping at the site of the hydraulic structure is adequately guaranteed.

With reference to the guiding principles it is noted that
- if the structure, the screens, the clay connection or the filter shows shortcomings, the safety with reference to piping decreases;
- if the screen S6 is in position, the safety will not increase. After all, S6 does not form an extra barrier for seepage line 1+3 or 2+3;
- the current seepage line would not be estimated any differently if it were supplemented with sand instead of clay (provided a well connecting clay revetment is present on the inside and outside slope).

7.3.6 Evaluation of observations

(see hydraulic structures diagram, section 5.3.9)

Observations
- The highest retained water level is NAP +3.3m (hydraulic head approx. 3.4m). At this hydraulic head some seepage was observed at the site of the filter behind the closed sluice (on the outflow side), in the vicinity of the connection to the concrete floor. Sand was probably carried out with this seepage. Measurements did not shown any settlement of the filter.
- Upon inspection of the sluice it was observed that fissure forming had occurred at the connection of the inside front walls on the sluice wall. At the outside front walls this was not observed. The sluice structure itself - in so far as it is visible - shows no defects. The steel screens show some corrosion at the wind and water line; there are no ‘holes’ however.

A study is made of whether possible flaws in the sluice can have a significant influence on safety with respect to piping, and whether a component can be formed over it on the basis of observation.

In table 7.3.3 the most obvious flaws are indicated. For each flaw the effect is quantified by determining:
- the reduction in seepage line with respect to the current seepage length of 27.0m (base case m), as calculated in section 7.3.4 for seepage line 2+3);
- the critical head, accompanying the reduced seepage line;
- the most probable place where a sand boil can then occur.

In the table it is then checked which flaws can be the most important. They are flaws which
- have a significant effect on safety (that is the critical head), and
- on the grounds of experience and inspection results cannot be excluded.

Table 7.3.3 Effect of possible flaws (reduction in seepage line with respect to seepage line 2+3 with L = 2.60m; critical hydraulic head according to Lane for Cw,creep = 7)

gebrek/faalmode		**kwelwegafname**	**kritiek verval**	**plaats evt. zandmee-voerende wel**
m0.	kwelweg 1+3 of 2+3: geen gebreken (‘perfecte conditie’)	n.v.t.	3,7 m (1)	achter S8, uitstroomzijde
m1.	aansluiting kleidijk aan S1 en S2 onvoldoende	2 x 4 - 5,5 = 2,5 m (kwelweg 2+3 maatgevend)	3,3 m	als bij m0.
m2.	aansluiting S3 op betonvloer niet goed	2 x 7 = 14 m (kwelweg 6 maatgevend)	2,2 m (3)	als bij m0.
m3.	aansluiting damwanden S4 op S2/S3 niet goed	2 x 7 - (1 + 2 x 4) = 5 m bij intreepunt (kwelweg 6 maatgevend)	3,5 m	als bij m0.
m4.	als m3., maar ook aanslui-ting S2 op S3 niet goed	2 x 7 = 14 m (kwelweg 6, geen intreeweerstand meer)	2,2 m (3)	als bij m0.
m5.	scheur in betonconstr., voor de keermiddelen	2 x 7 + (13,5/2)/3 = 16,25 m (kwelweg 6)	1,9 m (3)	als bij m0.
m6.	scheur in betonconstr., achter de keermiddelen	(13,5/2)/3 + 2 x 7 = 16,25 m (kwelweg 5)	2,5 m (3)	t.p.v. scheur

m7.	filter uitstroomzijde sluit niet aan op betonvloer / filter functioneert niet	kwelweg 1+4 wordt maatgevend	3,5 m	bij aansluiting filter – betonvloer, uitstroomzijde
m8.	aansluiting S5 op betonvloer niet goed	1 + 2 x 4 = 9 m (kwelweg 1+4)	2,3 m (3)	bij aansluiting damwanden, t.p.v. filter, uitstroomzijde
m9.	aansluiting damwanden S5 en S7 niet goed, wel filter aanwezig	2 x 4 = 8 m (kwelweg 1+4, S7 heeft geen functie meer)	2,4 m (3)	bij aansluiting damwanden, t.p.v. filter, uitstroomzijde
m10.	aansluiting kleidijk aan S7 en S8 onvoldoende	1 + 2 x 4 = 9 m (kwelweg 1+3)	2,5 m (2)	bij aansluiting, bovenzijde van S8

(1) Maatgevend in situatie zonder gebreken (zie tabel 7.3.2)
(2) Maatgevend te keren verval nu t.o.v. bovenkant damwand S8, d.w.z. ΔH = 1,5 m bij MHW. Dit is ruim opneembaar, dus dit geval is niet relevant.
(3) Niet waarschijnlijk. Bij een opgetreden verval van 3,4 m zijn geen duidelijke tekenen van heave of piping geconstateerd. Wel kan er ter plaatse van het filter aan uitstroomzijde enig zand zijn meegevoerd; de informatie hieromtrent is echter onduidelijk. Als veilige benadering wordt gesteld dat gebreken met een kritiek verval kleiner dan 3,0 m met voldoende waarschijnlijkheid kunnen worden uitgesloten.

flaw/failure mode
m0 seepage line 1+3 or 2+3: no flaws (perfect condition)
m1 connection clay dike to S1 and S2 inadequate
m2 connection S3 to concrete floor not good
m3 connection sheet piling S4 to S2/S3 not good
m4 as m3, but also connection S2 to S3 not good
m5 fissure in concrete structure in front of the defence means
m6 fissure in concrete structure behind the defence means
m7 filter outflow side does not connect to the concrete floor / filter does not function
m8 connection S5 to concrete floor not good
m9 connection sheet piling S5 and S7 not good, filter is present
m10 connection clay dike to S7 and S8 inadequate
seepage line reduction
not applicable
2x4 - 5.5 = 2.5m (seepage line 2=3 normative)
2x7 - 5.5 = 2.5m (seepage line 2+3 normative)
2x7 = 14m (seepage line 6 normative)
2x7 - (1+2+4) = 5m at entry point (seepage line 6 normative)
2x7 = 14m (seepage line6, no more entry resistance)
2x7 + (13.5/2)/3 = 16.25m (seepage line 6)
(13.5/2)/3 + 2 x7 = 16.25m (seepage line 5)
seepage line 1+4 is normative
1+2x4 = 9m (seepage line 1=4)
2x4 =8m (seepage line 1=4, S7 has no function any more)
1+2x4 = 9m (seepage line 1+3)
Critical Fall
Place any sand boil
behind S8, outflow side
as at m0
as at m0
as at m0
as at m0
as at m0
at fissure
at connection filter- concrete floor, outflow side
at connection sheet piling at filter, outflow side
at connection sheet piling at filter outflow side
at connection, top side of S8

(1) Normative in situation without flaws (see table 7.3.2)

(2) Normative hydraulic head to be retained now with respect to top side sheet piling S8, that is ?H = 1.5m at NHW. This can be taken broadly, so this case is not relevant.
(3) Not probable. At a hydraulic head of 3.4m no clear signs of heave or piping are observed. At the site of the filter on the outflow side some sand can be carried out; the information in relation to this is unclear however. As a safe approach it is proposed that flaws with a critical hydraulic head of less than 3.0m can be excluded with sufficient likelihood.

The flaws m1, m2 and m7 - not to be excluded in advance - emerge from table 7.3.3. The sluice can therefore not be assessed as sufficiently safe with respect to piping or heave.

7.3.7 Secondary study (hydraulic structures diagram, section 5.3.10)
The following secondary study is recommended tailored to the flaws mentioned above.

m1 The connection of the clay revetment to the outside slope with the adjacent sheet piling is not good. This is relatively easy to exclude or, if necessary, improve by inspection.

m3 The connection of the horizontal seepage screens to the structure or the vertical seepage screens at the outside front wall is not good. This is confirmed by observations: the fissure forming observed at the inside front wall is not observed at the outside front wall. A check can occur on the basis of piezometric gauge observations in the Pleistocene, at a short distance from the sluice and on both sides of the horizontal seepage screen S4. If the piezometric gauge on the inside shows a clear subdued response to the outside water level in relation to the piezometric gauge on the outside, it can be assumed that the horizontal seepage screen is functioning properly.

m7 The connection of the filter structure on the outflow side is not in order. This is certainly possible, considering the possible flushing out of sand at the highest measured head. This can be determined further by inspection. The cause of such damage is not easy to determine; it may be the unsatisfactory connection of the horizontal seepage screen mentioned above.

If a secondary study proves that the flaws mentioned are not present the sluice can be assessed as adequately safe with reference to piping and heave.

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Appendices

Appendix I Calculation Models for Groundwater Flow

For crack and heave checks calculation models for groundwater flow are used. They enable calculation of the occurring head in the water-bearing sand layer at the site of the inside of the dike or the hydraulic structure. For crack checking the head under the covering clay or peat layer is important. This must be tested against the head limit (section 4.1.2). For heave checking the vertical hydraulic gradient of the groundwater flow on the inside of the defence is important. This is tested against the permissible hydraulic gradient (section 4.3.1). In principle calculations of the heads in the water-bearing sand layers must be conducted with a general numeric calculation model for groundwater flow (or consolidation if we want to include time-dependent aspects of sinking in and seepage through the top layers in foreland and hinterland). Analytical calculation models generally comprise schematisations and provide therefore approaching solutions, but provide a good understanding of the effect of the mechanism and, partly due to the fact that they calculate so quickly, are very well suited to conducting sensitivity analyses. In this appendix the basic geo-hydraulic model for (horizontal) groundwater flow under a flood defence and the fragments model are discussed. The first model is suitable for use at flood defences where no cut-off walls are present. The fragments model is specifically intended for use with cut-off walls.

Basis geo-hydraulic model (stationary groundwater flow)
For the calculation of the occurring head the substrate configuration assumed is shown in figure A.i.1. It is presumed that there are no cut-off walls.

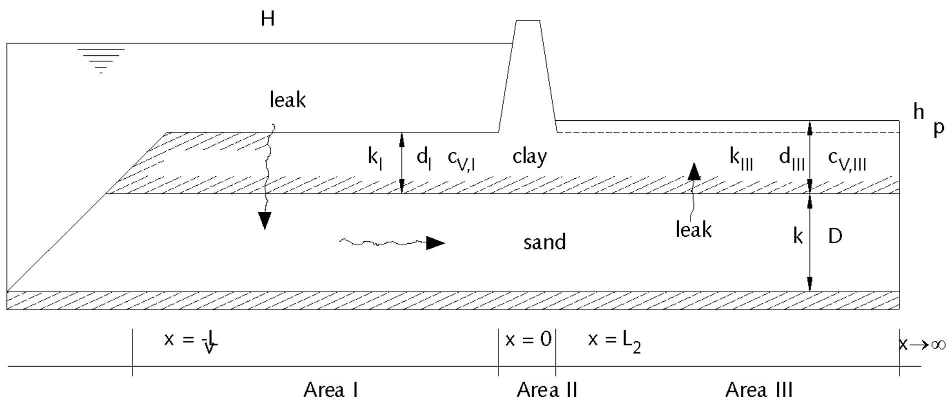


Figure A.I.1 Substrate configuration for head calculation for crack check
leak
clay
sand
Area I
Area II
Area III

In the (scarcely permeable) top layers vertically flowing groundwater flow is assumed. In the sand layer a horizontal flow is assumed (called Dupuit flow). For the mathematical formulation of the problem, which is used in the WATEX program among other things, the figure is divided into three areas, namely

- area I, at the site of the foreland
- area II, at the site of the dike or the hydraulic structure
- area III, at the site of the hinterland.

The head in the sand layer is indicated with $ø_z$ (x). This is a function of the horizontal position coordinate x ($-L_v \leq x \leq 0$), where L_v is the length of the foreland. In first instance we presume there are stationary flows. The hydraulic gradient over the clay layer at a given position x is $-(ø_z(x)-H)/d_I$, in which H is the outside water level and d_I the thickness of the covering layer in the foreland. The leak through the foreland is k_I ($ø_z(x)-H/d_I$, where k_I is the permeability of the covering layer. Retention of mass leads to the following differential equation for the head $ø_z$ in the sand layer, in a stationary state

A.I.1

$$\frac{d^2(\phi_z - H)}{d\,x^2} - \frac{1}{\lambda_I^2}(\phi_z - H) = 0 \qquad \textbf{(A.1.1)}$$

where -L_v is $\leq x \leq 0$ and λ the range for the foreland

A.I.2

$$\lambda_I = \sqrt{\frac{k_z\,D\,d_I}{k_I}} \qquad \textbf{(A.I.2)}$$

where k_I and d_I are the (vertical) permeability [m/s] and the thickness of the covering layer in the foreland respectively. Analogously, the head for area III is described by

A.I.3

$$\frac{d^2(\phi_z - h_p)}{dx^2} - \frac{1}{\lambda_{III}^2}(\phi_z - h_p) = 0 \qquad \textbf{(A.I.3)}$$

where L_2 is $\leq x < 8$, hp the polder level (or the ground surface level if there is no free water table) and λ_{III} the range of the hinterland

A.I.4

$$\lambda_{III} = \sqrt{\frac{k_z\,D\,d_{III}}{k_{III}}} \qquad \textbf{(A.I.4)}$$

Here k_{III} and d_{III} are the (vertical) permeability [m/s] and the thickness of the covering layer in the hinterland respectively.

For the area $0 \leq x \leq L_2$ an equation would have to be drawn up, analogous to (A.I.1) and (A.I.3). The flow through the dike body is not vertical however, making it problematical to derive a leak term. Because the leak to the dike body is expected to be small in relation to the leak through the foreland and hinterland layer this is negligible. The equation for the head in the sand layer in area II is then

A.I.5

$$\frac{d^2\phi_z}{dx^2} = 0 \qquad \textbf{(A.I.5)}$$

where $0 \leq x < L_2$.

Preconditions for the problem are $ø_z$ ($-L_v$) = H and $ø_z$ (8) = hp, while on the transitions x +0 and x + L_2 in the sand continuity of head and horizontal flowing capacity are important.

Accordingly, the solution (when kD values of the sand layer in the three areas are equal) is
- in area I ($-L_v \leq x \leq 0$):

A.I.6

$$\phi_z = H - (H - h_p) \frac{\lambda_I}{L} \frac{sh(\frac{L_v + x}{\lambda_I})}{ch(\frac{L_v}{\lambda_I})} \quad \text{(A.I.6)}$$

- in area II ($0 \leq x \leq L_2$):

A.I.7

$$\phi_z = H - (H - h_p) \left(\frac{\lambda_I \; th(\frac{L_v}{\lambda_{I1}}) + x}{L} \right) \quad \text{(A.I.7)}$$

- and in area III ($L_2 \leq x < \infty$):

A.I.8

$$\phi_z = h_p + (H - h_p) \frac{\lambda_{III}}{L} e^{\frac{L_2 - x}{\lambda_{III}}} \quad \text{(A.I.8)}$$

Here

A.I.9

$$L = \lambda_I \; th(\frac{L_v}{\lambda_I}) + L_2 + \lambda_{III} \quad \text{(A.I.9)}$$

In the above equations sh() and th() are the hyperbolic sine, cosine and tangent functions respectively.

In the WATEX calculation model, area II is neglected (area III can be split up into an area immediately behind the dike and an area far away from the dike (the so-called Three Section Model).

The first term (A.I.9) indicates the effective length of the foreland; for $x < - \lambda I \; th(L_v / \lambda I)$ the head is approximately equal to H. In the calculation of the seepage length present (section 4.4.1), this measure is maintained to calculate the influence of the foreland.

Basic geo-hydraulic model (non-stationary groundwater flow)

If time-dependence plays a role, in the case of short-term cyclical outside water levels, consolidation (foreland) and seepage (hinterland) play a role in the vertical flow through the top layers in foreland and hinterland. There are (temporary) greater inflow capacities in the sand layer at the site of the foreland and greater outflow capacities from the sand layer to the top layer, because the gradients in the heads at the bottom of the top layers (in absolute value) are greater than the stationary situation as a result of this. This results in smaller distribution ranges and so a smaller area in which the head in the sand layer declines from outside water level (H) to polder level (h_p).

The descriptive equations are a linked system of consolidation equations in the top layers and the Dupuit flow equations in the sand layer. The equations for vertical flow through the top layers in the areas I and III (see figure A.I.1) are

A.I.10

$$\frac{\partial \phi_d}{\partial t} = c_v \frac{\partial^2 \phi_d}{\partial z^2} \qquad \textbf{(A.I.10)}$$

Here $ø_d$ (z,t;x) is the head in the top layer, this is for every x a function of z and t, with the precondition for z=0, the limit with the water-bearing sand layer, the head $ø_z$(x) in the sand layer and as precondition for z=d the (current) outside water level in area I and the ground surface or polder level in area III. In area I c_v is the consolidation coefficient and in area III the swell coefficient; in area II, analogous to the stationary calculation model no leak from the sand layer to the dike's core or vice versa is presumed. The horizontal flow through the water-bearing sand layer is described by the equation

A.I.11

$$k_z D \frac{\partial^2 \phi_z}{\partial x^2} + k_d \frac{\partial \phi_d}{\partial z}\Big|_{z=o} = 0 \qquad \textbf{(A.I.11)}$$

Here $ø_z$ (x) is the head in the sand layer. In area II the term in the left-hand term of equation A.I.11. The connection value (continuity of head in sand and top layer) in the areas I and III are

A.I.12

$$\phi_z(x,t) = \phi_d(0,t;x) \qquad \textbf{(A.I.12)}$$

In the WATEX program these equations are solved. We can distinguish two characteristic situations, namely:

- the outside water level is a non-periodical time-dependent function. In WATEX the block function can therefore be given

A.I.13

$$
\begin{aligned}
H(t) &= H_o \qquad (-\infty < t \le t_o) \\
H(t) &= H_{top} \qquad (t_o < t \le t_e) \\
H(t) &= H_o \qquad (t_e < t < \infty \quad)
\end{aligned}
\qquad \textbf{(A.I.13)}
$$

In that case the initial condition for the problem is the stationary situation linked to the water level H_o. The equations (A.I.10)...(A.I.13) are solved with the help of Laplace transformations.

- the outside water level is a periodic function

A.I.14

$$H(t) = H_o \; cos(\omega t) + H_s \qquad \textbf{(A.I.14)}$$

In this case the (steady state) solution comprises superposition of the stationary solution with the pre-conditions $ø(-L_v)$ +H_s and ø(8) +h_p, and cyclical component. The cyclical component is calculated with the help of a complex harmonic analysis.

In the case of the outside water level a periodic function we consider the solution.

For the sake of simplicity we assume that the foreland and hinterland are infinitely wide, that the width of the area II is negligible and that the stationary component of the outside water level Hs is equal to the polder level hp. The solution for the head in the sand layer in area I is, when the hydrodynamic periods of the top layers are great in comparison to the period (2π/ω) where the outside water level varies

A.I.15

$$\phi_z(x,t) = h_p + H_o \; cos(\omega t) - H_o \frac{\lambda_{\omega,I}}{\lambda_{\omega,I} + \lambda_{\omega,III}} \, e^{0.924 \frac{x}{\lambda_{\omega,I}}} \; cos(\omega t + 0.383 \frac{x}{\lambda_{\omega,I}}) \qquad \textbf{(A.I.15)}$$

and in area III

A.I.16

$$\phi_z(x,t) = h_p + H_o \frac{\lambda_{\omega,III}}{\lambda_{\omega,I} + \lambda_{\omega,III}} \, e^{(-0.924 \frac{x}{\lambda_{\omega,III}})} \; cos(\omega t - 0.383 \frac{x}{\lambda_{\omega,III}}) \qquad \textbf{(A.I.16)}$$

Here λ_{ω} and $\lambda_{\omega,III}$ are the ranges which belong to the cyclical problem. They are functions of the ranges for the stationary situation λ_I and λ_{III} (formulas A.I.3 and A.I.14) the hydrodynamic periods of the top layers $t_{h,I}$ and $t_{h,III}$, and the angular velocity ω

A.I.17

$$\lambda_{\omega,I} = \lambda_I \sqrt[4]{\frac{1}{\omega t_{h,I}}} \qquad en \qquad \lambda_{\omega,III} = \lambda_{III} \sqrt[4]{\frac{1}{\omega t_{h,III}}} \tag{A.I.17}$$

We see that, for hydrodynamic periods of the top layers which are large in relation to the period with which the outside water level changes, the cyclic ranges will be considerably smaller that in the stationary situation. That means that the area in the sand layer, in which the head declines from outside water level to polder level is considerably smaller that in the stationary case (see also figure 4.10 in the main text).

Fragments method

In chapter 4, section 4.3.3 fragments are discussed which can be used for modelling the heave problem at hydraulic structures or dikes without top layer in foreland and hinterland. The fragments are

- the head fragment (figure 4.8a)
- the intermediate fragment (figure 4.8b)
- the end fragment (figure 4.8c)

In principal configurations can be modelled by connecting one head fragment, one or more intermediate fragments and one end fragment. The solution methodology, as discussed in section 4.3.3. is implemented in the HEAVE spreadsheet program (see appendix III); here a maximum of six intermediate fragments can be specified.

For two simple basic configurations graphs have been derived with the help of this program for monitoring for heave at hydraulic structures or dikes with one or more cut-off walls (see figure A.I.2). In calculating the permissible hydraulic gradient over the defence isotropic permeability of the sand layer and the permissible vertical hydraulic gradient on the downstream side of the defence of 0.5 is assumed. The graph for a single cut-off walls on the downstream side of the defence (figure A.I.2b) is derived from the *Guide on Safety Monitoring of Water Defences* [TAW 1996]. The method of use for these graphs is obvious. Tables are included below for the resistance of fragments, with which the user can conduct a fragments analysis. This makes it possible to calculate the basic configuration for the graphs, with the various lengths of the sheet piling on the upstream and downstream sides. If the current configuration is so different that the graphs or the tables cannot be used then monitoring can be conducted with the help of the spreadsheet program.

In the scope of recent research for the TAW various other fragment types have been developed, namely

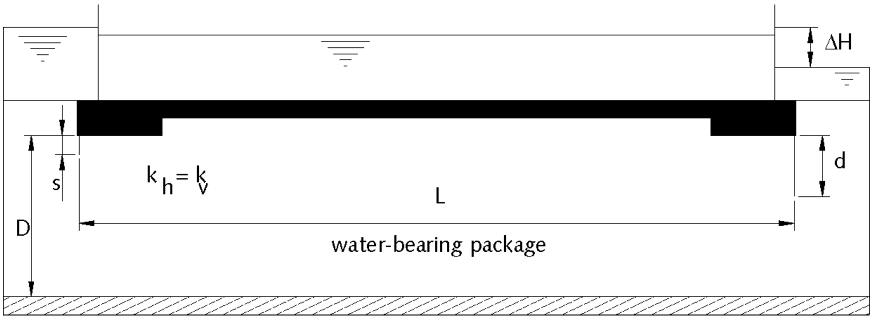
- the Settlement fragment, with which the effect of free space between the underside of the hydraulic structure and the sand layer (a settlement split) on the heave mechanism can be studied.
- the Leak fragment, with which the influence of the leak through the ditches or cut-off walls, or concentrated leak through a (corrosion) hole can be studied. This is important for monitoring flood defences, when the condition of the cut-off walls is uncertain.
- the Foreland fragment, with which the influence of a covering layer in the foreland can be studied.

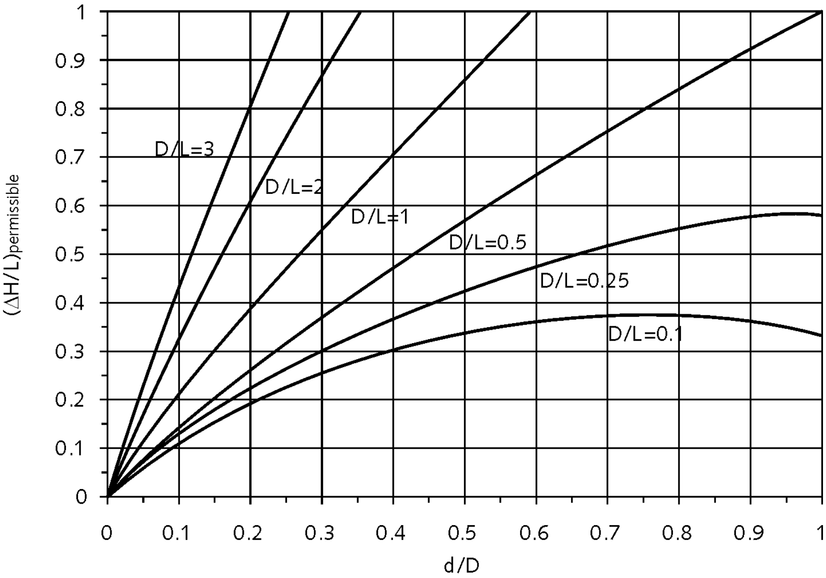
For the time being these fragments are only used for research, including

- a problematical sensitivity study of the influence of settlement splits and leak through the cut-off walls on the heave mechanism [GD 1998]
- integration of piping and heave rules [Sellmeijer 1996]

An accessible implementation of these fragments is not yet available for practical advice purposes.

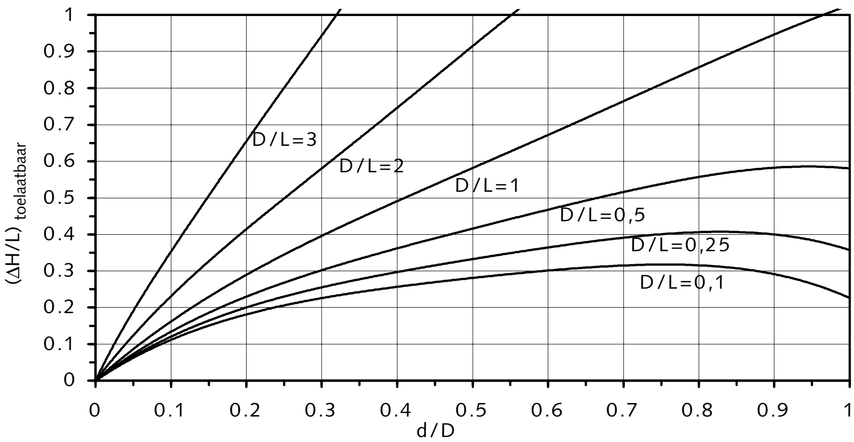
The probabilistic sensitivity study shows that settlement splits have a significant influence on the vertical hydraulic gradient near the exit point and accordingly on the probability of heave. On the end of this appendix the results of this study are addressed in more detail.



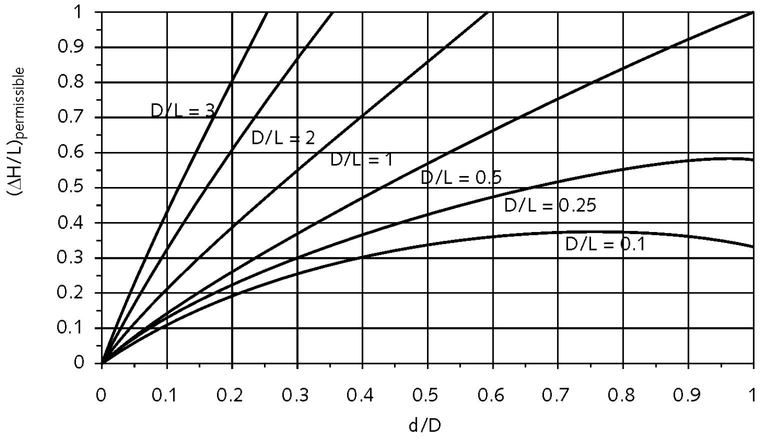


(a): Permissible hydraulic gradient in relation to heave (s = 0, heave criterion = 0.5)

Note: this figure is not correct



(a) this is correct



(b): Permissible hydraulic gradient in relation to heave (s = d, heave criterion = 0.5)

Figure A.I.2 Simple heave checks (isotropic permeability of sand layer)
water-bearing package
permissible
(a) Permissible hydraulic gradient in relation to heave (s=0, heave criterion = 0.5)
(b) Permissible hydraulic gradient in relation to heave (s=d, heave criterion = 0.5)

Fragments analysis using tables

In the tables below resistance factors are given for head, intermediate and end fragment for a configuration as in figure A.I.2 (a), as a function of s/D and d/D respectively and for various ratios of D/L (for the intermediate fragment). The guiding principle of the analysis is that the permeability coefficient in the three fragments is equal and that the permeability is isotropic (that is, the vertical permeability is equal to horizontal permeability). The modus operandi of the fragments analysis with these tables is as follows.

1 Determine the ratios s/D, d/D and D/L
2 Find in table A.I.1 the resistance factor of the head fragment (W_{kop}) corresponding to s/D and the resistance factor of the end fragment (W_{staart}) corresponding to d/D.
3 Find in one of the tables A.I.2...A.I.7 corresponding to D/L the resistance factor of the intermediate fragment (W_{tussen}) corresponding to s/D and d/D. These tables are symmetrical, so only the lower diagonal cells are filled in.
4 Calculate the permissible total hydraulic gradient over the flood defence with the help of the following formula

A.I.18

$$\left(\frac{\Delta H}{L}\right)_{toel} = \frac{d}{L}\left(\frac{W_{kop}/k_{kop} + W_{tussen}/k_{tussen} + W_{staart}/k_{staart}}{W_{staart}/k_{staart}}\right) i_{toel} \qquad \textbf{(A.I.18)}$$

where i_{toel} is the permissible vertical hydraulic gradient in connection to heave behind the screen on the downstream side; k_{kop}, k_{tussen} and k_{staart} are the permeability coefficients for head, intermediate and end fragments.

If horizontal and vertical permeability varies an isotropic situation must be simulated by decreasing the geometry. Note that the geo-metric parameters L (for horizontal decrease) and s, d and D (for vertical decrease) change because of this. Vertical decrease can only be realised when the permeability in all three fragments is equal and so all fragments are decreased in the same way, because otherwise continuity of the through flow capacity over the edges of the fragments cannot be guaranteed.

Table A.I.1 Resistance factor of head and end fragment

s/D, d/D	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
W_{kop}, W_{staart}	0.486	0.619	0.741	0.865	1.000	1.156	1.349	1.615	2.060

Table A.I.2 Resistance factor of intermediate fragment for D/L =0.10 (Table is symmetrical)

W_{tussen}	s/D=.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
d/D=.1	10.01								
0.2	10.04	10.06							
0.3	10.08	10.11	10.15						
0.4	10.14	10.17	10.21	10.27					
0.5	10.23	10.25	10.29	10.36	10.44				
0.6	10.35	10.37	10.41	10.47	10.56	10.68			
0.7	10.51	10.54	10.57	10.64	10.72	10.84	11.01		
0.8	10.75	10.78	10.82	10.88	10.97	11.09	11.25	11.50	
0.9	11.19	11.12	11.25	11.32	11.40	11.52	11.68	11.93	12.36

Table A.I.3 Resistance factor of intermediate fragment for D/L =0.25 (Table is symmetrical)

W_{tussen}	s/D=.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
d/D=.1	4.02								
0.2	4.04	4.06							
0.3	4.08	4.10	4.15						
0.4	4.14	4.17	4.21	4.27					
0.5	4.23	4.25	4.29	4.36	4.44				
0.6	4.35	4.37	4.41	4.47	4.56	4.68			
0.7	4.51	4.53	4.57	4.64	4.72	4.84	5.00		
0.8	4.76	4.78	4.82	4.88	4.97	5.09	5.25	5.50	
0.9	5.19	5.21	5.25	5.32	5.40	5.52	5.68	5.93	6.36

Table A.I.4 Resistance factor of intermediate fragment for D/L =0.50 (Table is symmetrical)

W_{tussen}	s/D=.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
d/D=.1	2.02								
0.2	2.04	2.06							
0.3	2.08	2.11	2.15						
0.4	2.14	2.17	2.21	2.27					
0.5	2.23	2.25	2.29	2.35	2.44				
0.6	2.35	2.37	2.41	2.47	2.56	2.67			
0.7	2.51	2.53	2.57	2.64	2.72	2.84	3.00		
0.8	2.76	2.78	2.82	2.88	2.97	3.08	3.25	3.49	
0.9	3.19	3.21	3.25	3.31	3.40	3.52	3.68	3.92	4.36

Table A.I.5 Resistance factor of intermediate fragment for D/L =1.00 (Table is symmetrical)

W_{tussen}	s/D=.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
d/D=.1	1.02								
0.2	1.04	1.06							
0.3	1.08	1.10	1.14						
0.4	1.14	1.16	1.20	1.26					
0.5	1.23	1.25	1.28	1.34	1.41				
0.6	1.34	1.36	1.40	1.45	1.52	1.63			
0.7	1.51	1.53	1.56	1.61	1.68	1.79	1.94		
0.8	1.75	1.77	1.80	1.85	1.92	2.02	2.17	2.41	
0.9	2.18	2.20	2.23	2.28	2.35	2.45	2.60	2.83	3.26

Table A.I.6 Resistance factor of intermediate fragment for D/L =2.00 (Table is symmetrical)

W_{tussen}	s/D=.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
d/D=.1	0.515								
0.2	0.538	0.557							
0.3	0.576	0.591	0.620						
0.4	0.632	0.643	0.666	0.705					
0.5	0.710	0.718	0.736	0.767	0.820				
0.6	0.817	0.824	0.837	0.862	0.905	0.980			
0.7	0.971	0.976	0.986	1.005	1.041	1.105	1.219		
0.8	1.205	1.209	1.218	1.234	1.264	1.319	1.421	1.612	
0.9	1.631	1.634	1.641	1.656	1.683	1.733	1.827	2.008	2.396

Table A.I.7 Resistance factor of intermediate fragment for D/L =4.00 (Table is symmetrical)

W_{tussen}	s/D=.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
d/D=.1	0.263								
0.2	0.281	0.292							
0.3	0.311	0.316	0.331						
0.4	0.353	0.355	0.362	0.382					
0.5	0.410	0.411	0.414	0.424	0.450				
0.6	0.490	0.492	0.492	0.496	0.510	0.549			
0.7	0.609	0.610	0.610	0.612	0.619	0.640	0.704		
0.8	0.807	0.806	0.806	0.807	0.811	0.822	0.861	0.980	
0.9	1.198	1.198	1.198	1.199	1.201	1.208	1.233	1.319	1.612

Calculation example: Suppose that the thickness D of the sand layer is 20m, the length L of the flood defence is 20m and that a screen is applied on both sides of the defence which reaches to 10m in the sand layer. The ratios (s/D) and (d/D) are then equal to 0.5 and the ratio (D/L) is equal to 1.00. We find in table A.I.1: W_{kop} = 1.0 and W_{staart} = 1.0. In table A.I.5 we find W_{tussen} +1.4.1. With the formula (A.I.8) we find, with i_{toel} = 0.5 that (?H/L) $_{toel}$ = 0.85. For this calculation we also could have used the graph in figure A.I.2 (c).

In the case of different values for s/D and/or d/D within the tables it is possible to interpolate.
In the case of different values for D/L as a result it is possible to interpolate between the various tables.

- Determine for s/D and d/D the value of W_{tussen} in the tables for the next highest and next lowest D/L value
- Reduce this value by the value of L/D for the relevant table
- Interpolate between the remaining differences
- Add the interpolated value to the current value of L/D; the resulting value is a good approach to W_{tussen} for the current D/L ratio.

Example: Suppose that D/L is 1.5, s./D is 0.3 and d/D is 0.5. The next highest and next lowest values for which a table is available are D/L=2 and D/L=1. In the table for D/L=2 we find that W_{tussen} + 0.736. This is reduced by L/D=0.5, leaving 0.236. In the table for D/l = 1 we find W_{tussen} = 1.28; this is reduced by L/D = 1, leaving 0.28. We interpolate in a linear way between 0.236 and 0.28: the result is 0.0.26. Here we add the current value of L/D, this is 0.667, leaving the approach W_{tussen} = 0.667+0.26 = 0.93

Probabilistic sensitivity analysis heave [GD 1998]

The sensitivity analysis is conducted for a number of cases, making use of the special fragment types, namely the leak fragments and the settlement fragment. The most important conclusions of this study are

1 A concentrated leak (for example through corrosion holes at the top of the cut-off wall) has little influence on the average vertical hydraulic gradient over the cut-off wall at the exit point. However, the probability that sand grains will be transported through the (corrosion) holes, resulting in piping is enormous. At old steel cut-off walls, which in normal use are not completely under the ground, it is very much to be recommended that a check by made for leaks. The possibilities to do so are limited or expensive however (digging). It is recommended that a specialist be consulted here.

2 A leak which is equally distributed through a cut-off wall only has a small influence on the vertical hydraulic gradient near the exit point. As long as this is due to leaking through sheet pile ditches or joints, while the screen is otherwise impermeable to sand, there are no consequences in relation to the probability of the heave mechanism.

3 A settlement split results in a considerable increase in the vertical hydraulic gradient occurring at the exit point and so also a great increase in the probability of heave. In the heave graphs in this appendix and in the spreadsheet calculations this influence cannot be discounted. In the manual calculations using the tables provided in this appendix the influence can be included by reducing the resistance factors of the fragments where a settlement split is or can be present. As a safety principle the resistance factors of those fragments must be halved. Depending on the ratio between the thickness of the sand package and the length of the fragment the reduction is less. Consult a specialist.

4 Depending on the grading and the packing of the sand layer near the exit point permissible hydraulic gradients greater than 0.5 can be permitted. Density measures of the sand are needed for this.

Appendix II Determining Parameters

In relation to the crack, heave and vertical seepage problems at existing or planned flood defences the following categories of parameters can be distinguished in the calculation model.

1 Hydraulic and hydrological preconditions, that is the water level to be retained and the polder or ground surface levels.
2 Structural dimensions (including measurements of cut-off walls) and the state of structural components where relevant.
3 The composition of the dike body (if already present) and substrate and the geo-hydraulic systematics (including which water-bearing sand layers are connected to the outside water and to what degree this influences hydraulic heads).
4 Material characteristics
5 Geo-hydraulic characteristics (these depend on ground composition and material characteristics).

Desired accuracy of parameter information
Depending on the design or monitoring phase various demands can be set on the accuracy of the information to be acquired; this is especially valid for information on the substrate, because this must be acquired via relatively expensive soil surveys.

In designing, we can distinguish the preliminary design phase and the actual design phase, in safety monitoring there is a rough 'first' monitoring and detailed monitoring.

The aim of the preliminary design phase is to check whether an intended rough design that meets the preconditions set in relation to finance, spatial planning etc and safety is also feasible in technical and construction terms. In doing so it must be possible to pinpoint the risks of possible problems in later design phases and their consequences. In relation to the information needed about the substrate that means that there must be an understanding of the rough composition of the substrate (sand package and clay/peat packages), rough indications of package densities and material characteristics and indications about avoiding local unfavourable deviations, such as sand-filled channels over the flood defence. In general the preliminary design phase will be based on the likely safe guiding principles in relation to the substrate composition and soil characteristics, founded on the information available. As more and better information becomes available a lower safety margin will be needed. When local heterogeneity which could have a considerable influence on the design cannot be excluded a study will have to made of the technical solutions which are possible locally and whether they are acceptable bearing in mind the preconditions.

The aim of the actual design phase is obviously to definitively determine the measurements of the design. The desired accuracy of detailed information on soil composition and material characteristics is not easy to give in advance. The general point of departure for the design calculations is that safe calculation parameters are determined on the basis of the information available. The decision to be made is whether the extra costs for the acquisition of more accurate information results in sufficient savings or other benefits in the design.

The situation for the general first monitoring of a flood defence is comparable to the preliminary design phase. The aim is to check if any mechanisms are a relevant threat to safety and where, and accordingly need to be studied in more detail. The detailed monitoring must provide adequate information on soil characteristics at those sites for those mechanism checks. The desired accuracy is highly dependent on the size of the probability that more accurate information will lead to more favourable monitoring results.

This way of thinking implies that the acquisition of information through detailed soil surveys and the actual designing or monitoring are iterative processes. Naturally, in practice only a limited number of iteration strokes are feasible, if only due to limitations to the throughput time of the design process.

Preconditions
Design water levels and polder or ground surface levels are obviously known to the manager of the flood defence. In determining the design water level the effects of the rise in sea level in the proposed plan period for the dike or construction must be taken into consideration. In determining polder levels or ground surface levels with any proposed or anticipated changes during the proposed plan period.

Structural dimensions
In monitoring existing defences measurements of construction components in the ground are usually derived from (old) plans. It must be checked whether parts of the structure, obviously in so far as important for mechanisms to be checked, are sensitive to ageing, such as connections between cut-off walls and between cut-off walls and the structure can break down due to settlement. In general this cannot be done by inspection. Any indirect indications must be looked at case by case.

Composition of the substrate
Soil composition and geo-hydraulic systematics for a preliminary design or a general monitoring follow a first soil survey in the route for which a flood defence must be designed or an existing defence must be monitored or strengthened.

Usually, some sort of soil survey will have been carried out in the past for an existing flood defence, as a result of which there may be sufficient insight into the general composition of the substrate, indications of ground package densities and, partly due to knowledge of the geology of the area, insight into the possible presence of strong local heterogeneity (channels etc).

If no previous soil survey is available and the information from other sources is too sparse a general soil survey will have to take place, including soundings at relatively large intervals and a number of drillings (for classification of ground layers). If an irregular substrate configuration is expected in (parts of) the route a geophysical survey is recommended, both length-wise and over the route.

Using the information available or the general soil survey a rough model can be determined of the soil composition. That is a rough picture of the ground layers and indications of the ground layer densities and an estimation of the probability of unfavourable local deviations (heterogeneity) for the defence to be designed or tested, as a result of knowledge of the geology of the area among other things.

Using the rough model of the soil composition and any known geo-hydraulic information the possible geo-hydraulic characteristics of the substrate in (parts of) the route can be checked. The actual influence on heads in water-bearing sand packages by outside water levels can only be determined properly by means of head response measurements. For the preliminary design phase or general test a credible conservative model can suffice however.

For the actual design phase or the detailed test the model for the substrate composition must be refined in general. The preliminary design calculations may show that any local deviations considered to be present necessitate modifications to the design. In that case it is necessary to localise (or exclude) such deviations by means of detailed surveys. It may also be shown that a more accurate determination of the soil model will lead to more favourable designs or monitoring results based on the conservative presumptions in the preliminary design phase or general monitoring. The detail survey may consist of additional soundings in ditches in front of, at the site of and behind the route of the defence, (shallow) drillings to determine the limits of channels and such, and additional geophysical measurements, depending on the characteristics to be detected in the substrate model. As argued above, this can take place in various additional strokes. The costs of a supplementary study are weighed against expected advantages in the design or the test.

More accurate characterisation of the geo-hydraulic systematics of the substrate can be realised through water pressure response measurements.

Soil characteristics
In the table A.II.1 below the soil characteristics/parameters are given which are important for crack, heave and piping checks. It is also indicated which type of soil survey can be used to determine these parameters. In general indicative estimates of parameter values will generally have to be used for preliminary designs or general monitoring, such as those given in the geo-technics standard NEN 6740, or on the basis of soil surveys in similar sediment. The soil characteristics needed for calculating with schematised geo-hydraulic models (see appendix I) are compiled for those calculations into geo-hydraulic indicators. In general, in determining these indicators indirect determination is preferred using water pressure response measurements (see below). The need for an accurate determination of these indicators is naturally dependent on the degree to which the combination of the crack and piping

mechanism is critical to the actual design or detailed monitoring. A sensitivity study can be very helpful here.

Geo-hydraulic indicators

Geo-hydraulic indicators are the parameter combinations which play a role in the geo-hydraulic model (appendix I), such as range, kD values and coefficients for water table or elastic storage. In principle, the parameters can be derived from soil characteristics as indicated in table A.II.1. In addition, (additional) estimates of these parameters can be obtained using water pressure response measurements at several places in the sand layer or layers. In that case combinations of geo-hydraulic indicators are sought via parameter variation in calculations with the geo-hydraulic calculation model, where the calculated water pressure response corresponds to the measured water pressure response. This is a process of trial and error. Robust computer models for this inverse parameter determination are not (yet) available.

Table A.II.1 Soil characteristics and determination methods

Grond-eigenschap	**Speelt een rol bij:**					**Bepaling met behulp van:**
	Opbarsten (S)	**(I)**	**Heave (fragm.)**	**Piping: Bligh/Lane**	**Sellm.**	
Deklaag/-lagen Voorland:						
- volumegewicht	x	x				monsterweging (lab)
- vert. doorlatend-heid	x	x		x	x	doorlatendh. proef (lab)
- samendrukkings coëfficiënt		x				samendr. proef (lab)
- dikte	x	x	x	x	x	sondering/boring
- breedte	x	x	x	x	x	beheerszone
Deklaag/-lagen Achterland:						
- volumegewicht	x	x				monsterweging (lab)
- vert. doorlatend-heid	x	x				doorl. proef (lab)
- zwelcoëfficiënt		x				samendr. proef (lab)
- dikte	x	x		x	x	sondering/boring
- locatie uittree-punt		x		x	x	uit opbarstanalyse
Watervoerende zandlaag/-lagen:						
- doorlatend-heid:						
* horizontaal	x	x	x		x	zie noot 1
* verticaal			x			zie noot 2
- dikte	x	x	x		x	boring, sondering
- coëff. elastische berging		x				pompproef (instat.)
- korrelverdeling				x	x	zeefanalyse
- fijne fracties				x	x	afslibben

Noot 1: Hor doorlatendheid uit pompproef, falling/constant head test, monopool/dipoolsonde, of afleiden uit zeefa + fijne fractie
Noot 2: Verticale doorlatendheid te schatten a.d.h. van informatie over stoorlaagjes/lensjes uit continuboring

(S) = stationair, (I) = instationair, Sell = volgens regel van Sellmeijer

Soil characteristic
Plays a role in
Determined with

Cracking
Heave (frag.)
Piping

Top layer(s) Foreland
volume weight
vertical permeability
compression coefficient
thickness
width

Top layer(s) Hinterland
volume weight
vertical permeability
swell coefficient
thickness
location exit point

Water-bearing sand layer(s)
permeability
horizontal
vertical
thickness
coefficient elastic storage
grain distribution
fine fractions

sample weighing (lab.)
permeability test (lab.)
compression test (lab.)
sounding/drilling
management zone

sample weighing (lab.)
permeability test (lab.)
compression test (lab.)
sounding/drilling
from crack analysis

see note 1
see note 2
drilling, sounding
pump test (non-stationary)
sieve analysis
elutriation

Note 1: Horizontal permeability from pump test, falling/constant head test, monopoly, dipole sound, or derive from sea + fine fraction
Note 2: Vertical permeability to be estimated using information on structural layers/dumping layers /lenses from continuous drilling

(S) = stationary, (I) instationary, Sell = according to the rule of Sellmeijer

Statistical calculation rules for determining characteristic values from random monitoring
If observation series are available on the basis of which representative values of parameters must be determined the statistical procedures are as follows. Estimates of the parameters with only a 0.05

probability of being exceeded or not achieved, the characteristic values, are sought. A distinction must be made between the following situations:

- the observation series is a local or a regional database;
- the characteristic value being sought must be representative of the individual score of the parameter or representative of the ‘low average’ value at the relevant location for which the mechanism is sought;
- the observations can be taken as Normal-distributed or as Log Normal-distributed.

A local database of observations means a collection of observations which are the result of soil surveys at the site of the location which is being monitored. A regional database means a collection of observations which are the result of soil surveys over a much larger area. It must be understood that the average of a regional observation database does not have to be representative of the low average at a location; in determining characteristic parameter estimates this should be taken into account.

As mentioned in section 2.2 of the main text, for some parameters the individual scores are important in a calculation model, for others the low average at the location for which the calculation must be made. Parameters in the piping analysis according to the rule of Sellmeijer for which the individual score is important include the seepage length and the thickness of the water-bearing sand layer, in so far as it varies in the length-wise direction of the defence, the minimal seepage length and the maximum thickness in the length-wise direction of the defence. For the permeability of the sand layer on the other hand, the low average value at a location is important; variations from point to point more or less average out in the groundwater flow.

Statistical procedures to come to characteristic estimates on the basis of an observation database are based on a presumption of the distribution of probability of those observations. Usually, the Normal (Gausse) distribution of probability function can be supposed. In the case of a wide distribution of the observation material it makes sense to suppose a Log Normal distribution of probability function, because the assumption of a Normal distribution of probability can lead to physical unrealistic characteristic values (for example, a negative characteristic value for a parameter which by definition has a positive value).

Formulas are given below for the calculation of characteristic parameter estimates (with 5% probability of exceedance or underachieving). We assume an observation series for a parameter *p*, namely $\{p_1, p_2, ... p_N\}$

A local observation database

If it is assumed that the observations are distributed normally, the formula for characteristic estimates for individual scores is

A.II.1

$$p_{kar} = p_{gem} \pm t_{N-1}^{0.95}\ s_p \qquad \textbf{(A.II.1)}$$

Here, P_{gem} is the mathematical average of the observations, s_p the standard deviation of the observations, *N* the number of observations and $t_{N-1}^{0.95}$ the Student-t factor. For that last value both the mathematical average and the standard deviation are only estimates of the expected value and the standard deviation of the parameter. In the table below a number of t values are given.

N-1	1	2	4	6	8	10	15	20	25	∞
$t_{N-1}^{0.95}$	6.314	2.920	2.132	1.943	1.860	1.812	1.753	1.725	1.708	1.645

Table from page A.II.5 (Dutch version)

The formula for characteristic estimation of the low average value of *p* (assuming a local observation database) is

A.II.2

$$p_{gem,kar} = p_{gem} \pm t_{N-1}^{0.95} \frac{s_p}{\sqrt{N}} \qquad \textbf{(A.II.2)}$$

If the spread in the observation series is wide, formula A.I.1 can lead to unrealistic estimates. In that case it makes sense to suppose a Log Normal distribution function. The characteristic estimate for individual scores is then

A.II.3

$$p_{kar} = exp((log\ p)_{gem} \pm t_{N-1}^{0.95}\ s_{\log p}\ ) \qquad \textbf{(A.II.3)}$$

In which $(log\ p)_{gem}$ and $s_{log\ p}$ are the mathematical average and the standard deviation of the natural logarithms of the observation.

The characteristic estimate for a low average value is usually less sensitive for a wide spread of the observations. We can continue to use formula (A.II.2). However, this can cause problems for small numbers of observations. In that case the following formula can be used

A.II.4

$$p_{kar} = exp((log\ p)_{gem} \pm t_{N-1}^{0.95} \frac{s_{\log p}}{\sqrt{N}}\ ) \qquad \textbf{(A.II.4)}$$

A regional observation database
For characteristic estimates of scores it makes no difference whether the observation material originates from local or regional soil surveys. The formulas (A.II.1 or (A.II.3) can be used for these estimates.

For characteristic estimates of low average values for the parameter it must be noted, as mentioned above, that low averages can vary by location. This spatial variation is allowed for in the characteristic estimate.

A.II.5

$$p_{gem,kar} = p_{gem} \pm t_{N-1}^{0.95}\ s_p \sqrt{\Gamma^2 + \frac{1}{N}} \qquad \textbf{(A.II.5)}$$

where $\Gamma^2 = 0.25$ (see [TAW 1989]).

Estimates of the permeability on the basis of grain distribution analyses
When using the rule of Sellmeijer the intrinsic permeability [m^2] of the water-bearing sand layer must be estimated. In the main text it has been indicated that this can be estimated using estimates of the permeability coefficient [m^2/s]. Estimates of the permeability are preferably obtained from measurements at the site, such as pump tests, falling head tests, et cetera. **If such measurements are**

not available estimates of the permeability can be calculated on the basis of the grain distribution of the sand in the water-bearing layer.

For this the following are needed:
an estimate of d_{10} [m], the 10 per cent value of the grain diameters of the sand in the water-bearing layer (the weight of the grains with a diameter smaller than d_{10} is 10 per cent of the total weight of the sand sample, elutriated if need be).
- an estimate of the uniformity coefficient $U = d_{60}/d_{10}$
- the percentage of fine particles (loam, silt)
- an indication of the quality of the packing density at the site.

The formula for calculating the permeability coefficient is [Den Rooijen 1992]

A.II.6

$$k = (c_o - 1.83\ 10^3\ ln(U))\ d_{10}^2 \qquad \textbf{(A.II.6)}$$

in which c_o is dependent on the picking of the sand
- loose packing $c_o = 1.5\ 10^4$
- natural packing $c_o = 1.2\ 10^4$
- firm packing $c_o = 1.2\ 10^4$

The packing is dependent on porosity and the uniformity coefficient. Rough indications are
- for uniform sands (U ˜ 2) and a pore content of less than 35% the packing is firm; for a pore content greater than 39% the packing is loose.
- for sand with a uniformity coefficient of U ˜ 10 the pore contents which indicate the transition between firm, normal and loose packing are 26% and 33%.
- for a still greater uniformity coefficient, U= 40, the pore content is 24% and 32%.

The calculated value of the permeability coefficient can be corrected for the percentage of fine particles, using Van den Akker's graph [Van der Akker 1972] (see also [TAW 19942].

The permeability which must be used in the calculation is a parameter representative of the groundwater flow through the whole water-bearing layer. Factors which could have a great influence on the flow, such as clay and silt lenses or other disturbances must not be overlooked. In these cases the probabilities are that the formula given here is not representative of the overall permeability.

For the calculation of the characteristic value of the permeability, in accordance with formula A.II.6, we need (high) characteristic values for d_{10} and for U. These can be estimated using the results of the sieve analysis, in accordance with the formula for characteristic value estimates. In addition, for piping analyses with the rule of Sellmeijer, a low characteristic value for d_{70} must be used. The independent application of the characteristic value procedures on the results of a sieve analysis can lead to a situation in which the calculated (low) characteristic value for d_{70} is lower than the calculated (high) characteristic value for d_{10}. In fact d_{70} and d_{10} are not independent; this should be expressed in the estimation procedures for characteristic value determination. In [TAW 1994] it is therefore proposed that the characteristic value for d_{10} be determined using the (low) characteristic value of d_{70} and the (high) characteristic value of U ($=d_{60}/d_{10}$) in accordance with the formula

A.II.7

$$d_{10,kar} = \alpha'\ \frac{d_{70,kar}}{U_{kar}} \qquad \textbf{(A.II.7)}$$

in which $\alpha' = 0.9$ is a corrective value as U is based on d_{60}.

Appendix III Software

In section 4.6 an overview is given of the software currently available in the field of cracking, piping and heave checking. In this appendix a short description of the content of various computer programs is given. The generally fast-moving development of computer models is emphasised. As a result the information is subject to change.

WATEX

This program was developed in the scope of TAW research, specifically for uplift, cracking and piping analyses. In addition, the program also has the capability to calculate heads in water-bearing soil layers which are fed by sea or river, given a geo-hydraulic schematisation of the substrate. The program can also be used to determine geo-hydraulic parameters of the substrate using measurements of the potential response in water-bearing ground layers in variations of sea and river water levels (including 13-hour tidal measurements).

The program has the following menu items:

-<u>Three section model</u>: Calculation of heads in a water-bearing soil layer as a function of the outside water level (both stationary and time-dependent), where the substrate is schematised as a sand layer covered with clay or peat. Here, in the cross section three sections are distinguished, namely the section under the foreland, the section directly behind the dike and the section far away from the dike. Assumptions for the calculation are horizontal groundwater flow in the sand layer and vertical flow (and consolidation) in the covering clay/peat layer.
<u>Two-layer model</u>: Calculation of the heads in two water-bearing sand layers, divided by a clay or peat layer. Here too, horizontal groundwater flow in the sand layers is assumed and vertical flow in the clay and peat layer.
<u>Piping (Bligh's method)</u>: Piping check in accordance with the calculation rule of Bligh
<u>Piping and heave (Lane's method)</u>: Piping or heave check in accordance with the calculation rule of Lane
<u>Piping (Sellmeijer's method)</u>: Piping check in accordance with the rule of Sellmeijer.
<u>Processing/Interpretation 13--hour measurements</u>: this menu item is not yet worked out/implemented.

MPIPING

This program was developed in the scope of TAW research. The program enables a probabilistic piping analysis to be conducted, based on the rule of Sellmeijer and a first order second moment probabilistic reliability analysis for a cross section of the flood defence. In the program the drainage frequency lines of the Rhine, Meuse and Waal are included, as are the reference lines between river levels and drainage for the various locations along these rivers. the program is in first instance developed and used in the scope of drawing up the semi-probabilistic calculation formulas for the rule of Sellmeijer, that is published by the TAW-B workgroup in a technical report [Calle and Weijers 1994].

SPREADSHEET for HEAVE Checks

In the scope of the development of the fragments method a spreadsheet program was developed with which the connection of a fragment, as discussed in chapter 4, section 4.3.3, can be conducted in a user-friendly manner and the vertical hydraulic gradient needed for heave checking can be calculated. The program is a QUATRO4 spreadsheet which runs on a PC under DOS. A configuration can be submitted by the user comprising

- one foreland component
- one of more intermediate fragments
- one end fragment

The problem variables to be submitted by the user are

- measurements of the fragments (see standard formats in appendix XX)
- imbedding depth of the cut-off wall (= length of cut-off wall/ thickness of the water-bearing sand package)
- permeability of the sand in the fragments (these can differ by fragment; only isotropic permeability can be submitted
- the permissible hydraulic gradient to be maintained (itoel = 0.5 is common)

The program calculates the permissible hydraulic gradient over the flood defence (= water level difference inside - outside, divided by the width of the defence). Given an actually occurring hydraulic gradient in the design situation, by playing with the imbedding depths of the cut-off walls, the imbedding depths needed can be determined.

If the permeability levels are not isotropic (for example horizontal permeability greater than vertical permeability) an isotropic situation must be simulated by decreasing the fragments. In principle this can be realised by decreasing horizontally and vertically; by decreasing only horizontally, connection problems between the fragments are avoided. Note that the (calculated) widths of the flood defence construction change accordingly.

MSEEP and SEEP/W
Both are multipurpose programs for numerical groundwater flow analyses, based on an Infinite Elements Method.

MSEEP is a Dutch product (part of the M series software) and runs on PCs under DOS. A Windows (95 or NT) version is under development. The program is suitable for simulating 2-dimensional stationary and time-dependent flow in completely saturated ground.

The SEEP/W program is a Canadian product which runs on PCs under Windows. It is suitable for simulating 2-dimensional stationary and time-dependent flow in completely or partly saturated ground.

In relation to the problems of sand-carrying boils the following uses of the two programs are relevant:
- determination of the head in a water-bearing sand layer under a dike or hydraulic structure and a scarcely permeable top layer situated behind it for a crack check
- determination of the head model in a water-bearing sand layer under a dike or hydraulic structure with cut-off walls for determining the vertical hydraulic gradient in relation to heave checks.

In comparison to WATEX the numerical approach with MSEEP or WSEEP offers almost unlimited freedom in compiling the configuration of the substrate and the permeability qualities. Consider the study of the effects of
- stratification of the water-bearing sand package (for example fine and coarse material)
- ending of the sand layer inside the dike
- presence of cut-off walls
- non isotropic permeability

One disadvantage of an approach with MSEEP or WSEEP in comparison to calculations with WATEX or the spreadsheet programs for heave checks is that gearing to the problem must be done by the user. As a result the time needed for input preparation and interpretation of calculation results, even for experienced program users is considerable.

PLAXIS
This is a multipurpose program for conducting stability and remodelling calculations for ground bodies. The program also comprises modules for conducting groundwater flow analyses and consolidation calculations. In relation to the problems of sand boils only these last two possibilities are important. For groundwater flow analyses the same applies as is discussed about the programs MSEEP and WSEEP. Consolidation (compression and seepage) is applicable for problems in which the time-dependent head distribution in the sand layer (for crack problems) is important (see chapter 4, section 4.5).

The PLAXIS program runs on a (powerful) PC under Windows (earlier versions under DOS). Although it is basically user-friendly, use of the program does demand considerable experience. Application for crack of heave checks are difficult in principle, but due to the intensive input parameters and interpretation of calculation results only recommended for research purposes.

Appendix IV Damage Catalogue

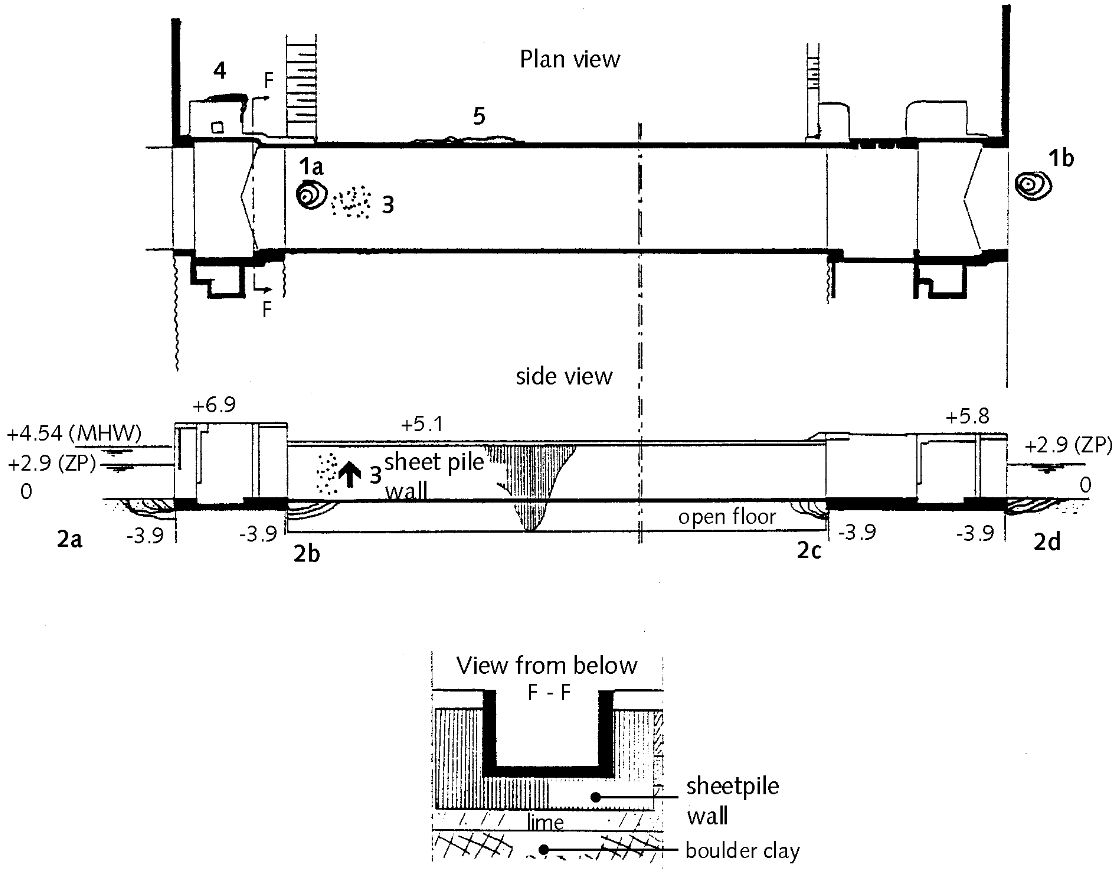
The damage catalogue provides an overview of damage scenarios which may indicate boil forming or piping. Use is made of photographs and drawings, as referred to in tables 1 to 4 in chapter 6.

For the damage scenarios at dikes photos are used. The numbering corresponds to the code of the damage scenario from chapter 6. A specific damage scenario is shown on every photo.

The damage scenarios at hydraulic structures are shown by photos 3.6a to 3.6e and 4.12. (These numbers do not correspond to the coding in chapter 6.) The three drawings (chambered lock, sluice and dike cut) show the various damage scenarios at hydraulic structures. The numbering in the drawings refers to the accompanying tables which contain the coding chapter 6.

1.1a Damage to the outside slope of a dike due to flushing out; possibly as a result of digging [RWS-DWW 214/33a]
1.2a Molehills left IJssel dike at Wilp: is this collected rainwater or an indication of seepage. [RWS-DWW 165/18A]
1.2b Dry boils on Hondsbroekse Plei (IJsselkop). Possible reason: mole tunnels. The high water level prior to the inspection is important. [RWS-DWW 133/4]
1.2c Muskrats tunnel system on the outside slope. [RWS-DWW 220/27a]
1.3a Crumbling of the outside slope at the site of a natural gas pipeline, Kromme Does (polder drainage quays) [RWS-DWW117/24a]
1.3b Clay digging right on inside dike (Maas dike te Velp, Noord-Brabant). Increased vigilance is called for at the site of the digging: piping can only be observed in a relatively well-developed stage. [RWS-DWW 322/18]
1.4a Trees on the inside slope in jade along the Gouwe (polder drainage quays). In fact there is no damage here. The condition of the trees is important however. [RWS-DWW 121/15]
1.4b Sawn down trunks on the inside slope of the Megense dike (Maas dike). Both removal and non-removal of the stump are risky. This can lead to undesirable tunnel forming at the site of the original roots in the future. [RWS-DWW 323/18a]
2.2 Seepage along a row of piles on the toe of the inside slope of the IJssel dike at Wilp. Is this the beginning of piping, as a result of the row of piles among other things, or is this surface water which is collecting at the lowest point? [RWS-DWW 197/23a]
2.3a Local elevation and swamping of the hinterland behind the harbour dike at Woudrichem, at the site of the head ditch. [RWS-DWW 128/1]
2.3b Elevated piece of water-bearing ground, very local. [RWS-DWW 197/16a]
2.5 Flow visible in the ditch on the inside toe of the Waal dike at Herwijnen, during period of frost. In places the ditch is not frozen solid. [RWS-DWW 199/24]
2.6 Water flushes through a quay, locally at a bend. It is important to check from close by if sand has also been flushed out at the site of these wet spots. . [RWS-DWW 125/14]
2.7 Boil from the ditch side along the toe of the main Waal dike (hmp 348); sand has not yet been carried out. [RWS-DWW 132/9a]
2.8 Clean water spurting up behind the dike above the ground surface. Striking is that this occurs right in front of a ditch. This is possibly always a consequence of digging or drilling holes which have not been closed off. [RWS-DWW 470/20]
2.10 Sand boil in the ditch on the toe of the Maas dike at Lage Zwaluwe, in combination with a sagging inside slope. [RWS-DWW 128/23]
2.11Sand boil in ditch side hmp 248 main Waal dike North, after high water. The sand is directly carried away in the ditch here. [RWS-DWW132/4a]
2.12a Discoloration of the ditch water by sand boil in seepage ditch beside Waardenburg. [RWS-DWW 125/16]
2.12b Sand boil in main ditch beside Kandia (Pannerdens canal / Lower Rhine). In the assessment of the general cross section profiles these main ditches must be taken into consideration. [RWS-DWW 469/11a]
2.12c Sand in ditch water; begin of sand boil at the site of the soil of a seepage ditch. [RWS-DWW 127/12]
2.13a Creation of a boil on the ground surface. Water bubbles up and sand is carried out with it. [DWW 159/14]
2.13b Small sand boil on ground surface, along the IJssel dike beside Wilp. [RWS-DWW 197/15a]

2.13c Sand boils on the ground surface behind the dike, possibly due to mole tunnels. [RWS-DWW 130/11]
3.6a Fissure at connection lock chamber and front wall in an inlet sluice, on the inside dike. This can be a reason for horizontal seepage.[Grontmij Bovensluis 1.15]
3.6b Fissure in brick lock chamber wall of an inlet sluice at the outflow opening. The sluice is dried out in connection with inspection. [Grontmij Bovensluis]
3.6c Rotted wooden floor structure, Westbeer inlet. The inlet is dried out in connection with inspection. Substrate consists of clay, so that there need be no consequences in relation to piping. [Grontmij]
3.6d Overgrowth from wing walls at the site of inflow opening of a sluice. The impermeability to water can be doubted. The seriousness depends on the function which the wing walls must fulfil in relation to piping. [Fugro]
3.6e Fissure with overgrowth at the connection of the lock chamber to the front wall, on the inside of the sluice, Lummermerk lock in the old Zuiderzee dike near Elburg [Fugro]
4.12 Sand boil at the connection of the horizontal seepage screen of a concrete lock at the outflow opening. The defence means of the lock is closed. [DWW 191/17a]



Drawing IV:1 Lock. For the damage scenarios refer to table IV.1a/c and IV.1b
Plan view
View from side
View from below

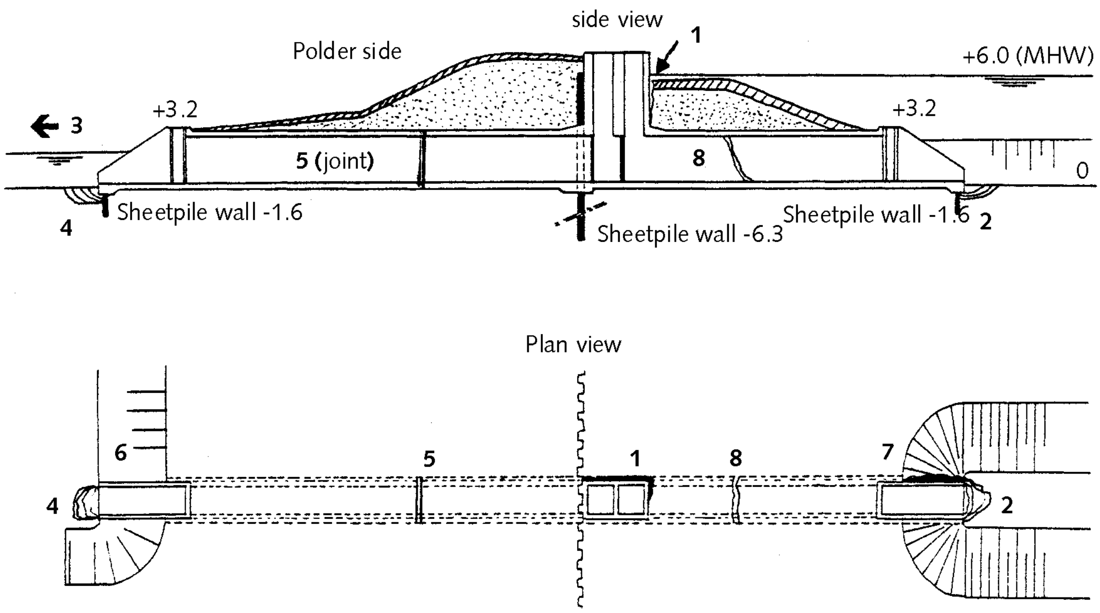
Table IV.1a/c Damage scenarios lock; normal inspection/high water inspection reference drawing IV.1

Description
- swirling water behind outside chamber
- swirling water behind inside chamber
- sand carried out in water
- crack between hydraulic structure and ground

crack between dam wall and ground
possible action
check means of closure for leak, if no leak is cause of boil forming, inspection soil protection (filter)
check means of closure for leak, if no leak is cause of boil forming, inspection soil protection (filter)
sand boil; repair filter structure, inspection cut-off wall
check remaining seepage length
check remaining seepage length
code

In connection with the open lock floor a special inspection, where the chamber is dry, cannot take place.

Table IV.1b Damage scenario lock; special inspection (diver)
reference drawing IV.1
Description
erosion
Possible action
consequence of ship movements, new dumping
consequence of ship movements/piping (failing filter); repair filter structure
consequence of ship movements; new dumping
consequence of ship movements/piping (failing filter); repair filter structure
Code



Drawing IV.2 Chambered lock. For the damage scenario refer to table IV.2a to Iv.2c
View from side
Polder side
Sheet piling
joint
Plan view

Table IV.2a Damage scenarios chambered lack; normal inspection
reference drawing IV.2
description
crack between hydraulic structure and dike
flow despite closed situation chambered lock
crack between outflow and dike
crack between inflow and dike
possible action

supplement ground and check remaining seepage length
check filter structure inflow and outflow for damage
check for outflow at high river level
check for outflow at low river level

code

Table IV.2b Damage scenarios chambered lock; special inspection (dry out/diver)

- reference drawing IV.2

description

erosion
flow despite closed chambered lock
erosion
inflow of water through joint structure
open fissures

possible action

repair filter inflow
check filter structure inflow and outflow for damage
repair filter outflow
repair joints, move chamber under water at high water
check differential settlement structure

code

Table IV.2c Damage scenarios chambered lock; high water inspection

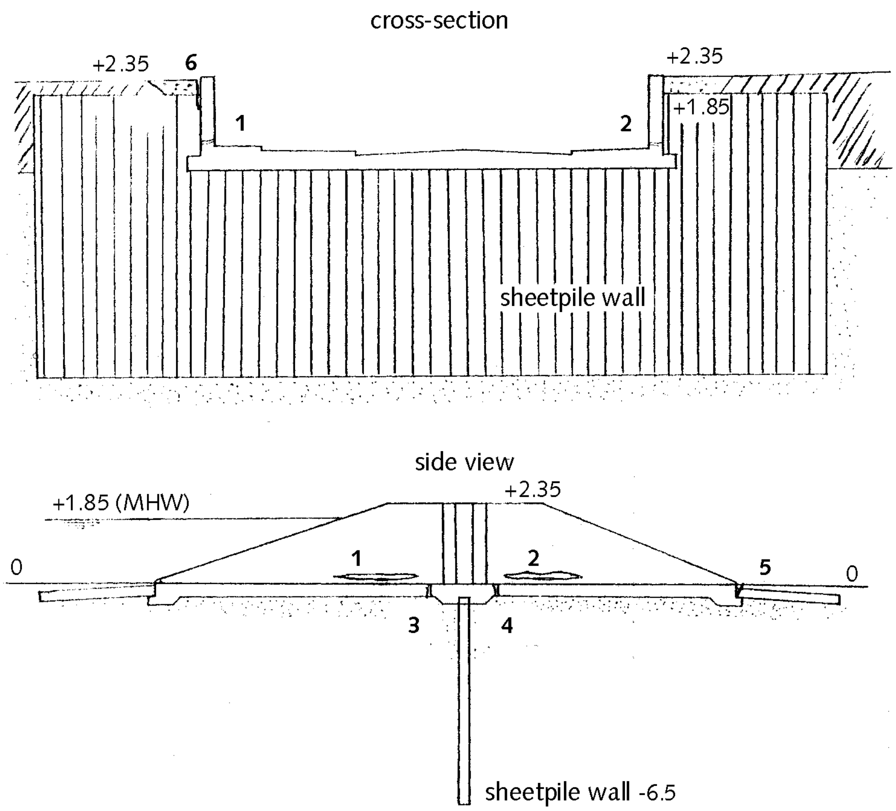
reference drawing IV.2

description

flow in closed condition
outflow through crack between outflow and dike

possible action

check filter structure inflow and outflow for damage
fill crack with clay directly



Drawing IV.3 Dike cut. For the damage scenarios refer to table IV.3a/b and IV.3c

section
dam wall
view from side
dam wall

Table IV.3a/b Damage scenario Dike cut; normal inspection/special inspection
reference drawing IV.2
description
- length-wise fissure outside
- length-wise fissure inside
- open joint outside
- open joint inside
- crack at the site of support guard inside
- crack between hydraulic structure and connecting ground

possible action
- repair fissures, urgency dependent on soil supplementation right next to the hydraulic structure
- repair fissures, urgency dependent on soil supplementation right next to the hydraulic structure
- repair join structures, urgency dependent on soil supplementation right next to the hydraulic structure
- repair join structures, urgency dependent on soil supplementation right next to the hydraulic structure
- check rest seepage length
- check connection cut-off wall to structure/measure settlement structure

code

Table IV.3c Damage scenario dike cut; high water inspection
reference drawing IV.2
description
- exiting water from wall inside
- exiting water from floor inside
- exiting water at the site of support guard inside

possible action
- dump ground against wall immediately and check after high water for fissures
- dump ground on the floor structure and inspect the floor structure after high water
- dump ground on the floor structure and inspect the cut-off wall after high water

code

Symbols Used

C_{creep}	Seepage line factor in calculation rule of Bligh
$C_{w,creep}$	Weighted seepage line factor in calculation rule of Bligh
C_v	Consolidation or swell coefficient [m^2/s]
D	Thickness of water-bearing sand layer [m]
d	Imbedding line of cut-off wall [m]
d_1, d_I	Thickness of clay/peat top layer in foreland [m]
d_2, d_{III}	Thickness of clay/peat top layer in hinterland [m]
d_{10}, d_{50}	
d_{60}, d_{70}	10, 50, 60 and 70 percentage value of grain diameter [m]
g	Acceleration of gravitational pull [m/s^2]
H	Outside water level [m in relation to reference level]
h_p	Polder level (free water level) or polder ground surface level [m in relation to ref. level]
h_{zand}	High topside water-bearing sand layer [m in relation to reference level]
i_c, i_{toel}	Critical /permissible hydraulic gradient in relation to heave
k_z	Horizontal permeability coefficient for sand layer [m/s]
k_1, k_I	Vertical permeability coefficient for top layer in foreland [m/s]
k_2, k_{III}	Vertical permeability coefficient for top layer in hinterland [m/s]
L	Seepage length, also fragment length [m]
L_h	Horizontal seepage length [m]
L_v	Length top layer in foreland; also vertical seepage length [m]
l	Pipe length (calculation model Sellmeijer) [m]
m_v	Length compression coefficient [m^2/kN]
p	Pore fraction sand
t_h	Hydrodynamic period [s]
U	Uniformity coefficient (grain distribution)
W	Resistance factor (fragments)
x,z	Place variables horizontal, vertical [m]
α	Auxiliary variable in calculation model of Sellmeijer also: Correction factor in grain distribution
γ	Safety factor
$\gamma_{z,s}$	Wet volume weight sand [kN/m^3]
γ_w	Volume weight water [kN/m^3]
γ_k	Volume weight (sand) grain material [kN/m^3]
γ'	Apparent volume weight submerged sand [kN/m^3]
?H	Hydraulic head over flood defence [m]
?H_c	Critical hydraulic head over flood defence [m]
η	Dragforce factor (rule of Sellmeijer)
κ	Intrinsic permeability [m^2]
ρ_p	mass density grain material sand [1700 kg/m^3]
ρ_w	mass density water [1000 kg/m^3]
ν	Kinematic viscosity water [1.33 10^{-6} m^2/s]
φ	Head, groundwater potential [m in relation to ref. level]
$\varphi_{z,g}$	Potential limit (in relation to uplift/cracking)
Θ	Rolling resistance angle [°]
λ_1, λ_I	Range under top layer in foreland [m]
λ_2, λ_{III}	Range under top layer in hinterland [m]

Functions etc

sh()	Hyperbolic sine
ch()	Hyperbolic cosine
th()	Hyperbolic tangent
$\delta/\delta t$	First order partial derivative to the time
$\delta/\delta z$	First order partial derivative to the place variable z (m.m. x)
$\delta 2/\delta z2$	Second order partial derivative to z (m.m. x)

Gebruikte symbolen

C_{creep}	Creep-factor in rekenregel van Bligh
$C_{w,creep}$	Gewogen creep-factor in rekenregel van Lane
c_v	Consolidatie- of zwelcoëfficiënt [m^2/s]
D	Dikte van watervoerende zandlaag [m]
d	Inbeddingslengte van kwelscherm [m]
d_1, d_I	Dikte van klei/veen deklaag in voorland [m]
d_2, d_{III}	Dikte van klei/veen deklaag in achterland [m]
d_{10}, d_{50}, d_{60}, d_{70}	10-, 50- 60- en 70-percentielwaarden van korreldiameters [m]
g	Versnelling van de zwaartekracht [m/s^2]
H	Buitenwaterstand [m t.o.v. referentiepeil]
h_p	Polderpeil (vrije waterspiegel) of poldermaaiveldpeil [m t.o.v.ref.peil]
h_{zand}	Hoogte bovenkant watervoerende zandlaag [m t.o.v. ref.peil]
I_c, i_{toel}	Kritiek/toelaatbaar verhang i.v.m. heave
k_z	Horizontale doorlatenheidscoëfficiënt voor zandlaag [m/s]
k_1, k_I	Verticale doorlatendheidscoëffiënt voor deklaag in voorland [m/s]
k_2, k_{III}	Verticale doorlatendheidscoëfficieënt voor deklaag achterland [m/s]
L	Kwelweglengte; ook fragmentlengte [m]
L_h	Horizontale kwelweglengte [m]
L_v	Lengte afdeklaag in voorland; ook verticale kwelweglengte [m]
l	Pipe-lengte (rekenmodel Sellmeijer) [m]
m_v	Samendrukbaarheidscoëfficiënt [m^2/kN]
p	Poriënfractie zand
t_h	Hydrodynamische periode [s]
U	Uniformiteitscoëfficiënt (korrelverdeling)
W	Weerstandsfactor (fragmenten)
x, z	Plaatsvariabelen horizontaal, verticaal [m]
α	Hulpvariabele in rekenregel van Sellmeijer ook: Correctiefactor bij korrelverdeling
γ	Veiligheidsfactor
$\gamma_{z,s}$	Nat volumegewicht zand [kN/m^3]
γ_w	Volumegewicht water [kN/m^3]
γ_k	Volumegewicht (zand)korrelmateriaal [kN/m^3]
γ'	Schijnbaar volumegewicht ondergedompeld zand [kN/m3]
ΔH	Verval over waterkering [m]
ΔH_c	Kritiek verval over waterkering [m]
η	Sleepkrachtfactor (regel van Sellmeijer)
κ	Intrinsieke doorlatendheid [m^2]
ρ_p	massadichtheid korrelmateriaal zand [1700 kg/m^3]
ρ_w	Massadichtheid water [1000 kg/m^3]
ν	Kinematische viscositeit water [1.33 10^{-6} m^2/s]
ϕ	Stijghoogte; grondwaterpotentiaal [m t.o.v. ref. peil]
$\phi_{z,g}$	Grenspotentiaal (i.v.m. opdrijven/opbarsten)
θ	Rolweerstandshoek [^{o}]
λ_1, λ_I	Spreidingslengte onder deklaag in voorland [m]
λ_2, λ_{III}	Spreidingslengte onder deklaag in achterland [m]

Functies etc.:

sh()	Sinus hyperbolicus
ch()	Cosinus hyperbolicus
th()	Tangens hyperbolicus
$\partial/\partial t$	Eerste orde partiële afgeleide naar de tijd
$\partial/\partial z$	Eerste orde partiële afgeleide naar de plaatsvariabele *z* (m.m. *x*)
$\partial^2/\partial z^2$	Tweede orde partiële afgeleide naar *z* (m.m. *x*)

Indemnity

This green version of *Technical Report on Sand Boils (Piping)* has been compiled and arranged with the greatest of care by the Technical Advisory Committee on Flood Defences. In the opinion of the Technical Advisory Committee on Flood Defences this Technical Report can be directly applied to practice. This version is called ‘green’ to provide users with the opportunity and encourage them to make their own observations, especially in relation to its practicability.

Remarks and reactions from users will be collected by the work organ of the Technical Advisory Committee on Flood Defences (the Road and Hydraulics Division of the Directorate-General for Public Works and Water Management, tel. 015-2518436). On the basis of the collected remarks and observations a definitive version of the Technical Report will be drawn up and the designation ‘green’ will expire.

Flows charts for monitoring

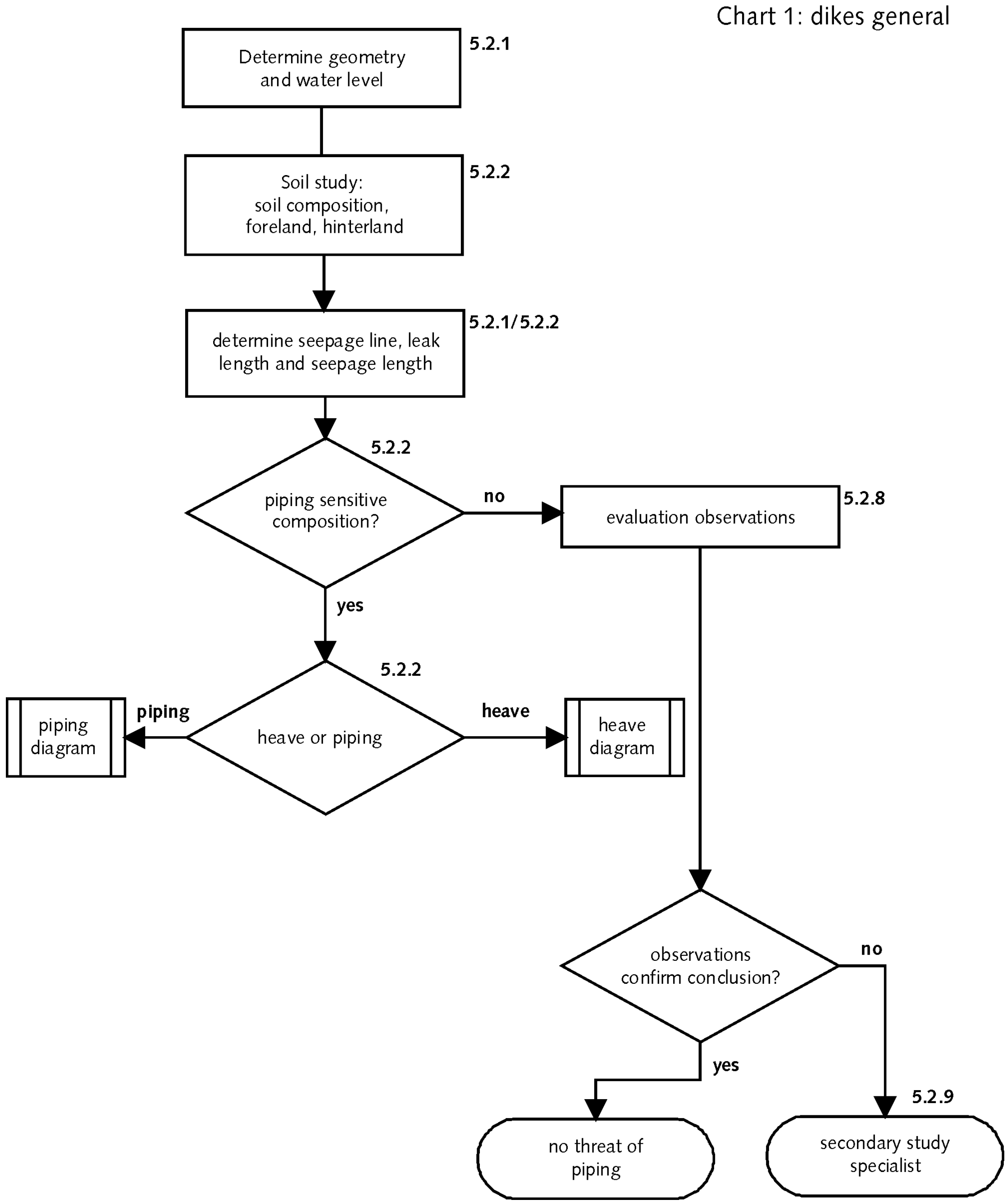


Chart 1 dikes general

Determine geometry and water level
Soil study: soil composition, foreland, hinterland
determine seepage line, leak length and seepage length
piping sensitive composition?
no
yes
evaluation observations
heave or piping
piping
heave
piping diagram
heave diagram
observations confirm conclusion?
no threat of piping
secondary study specialist

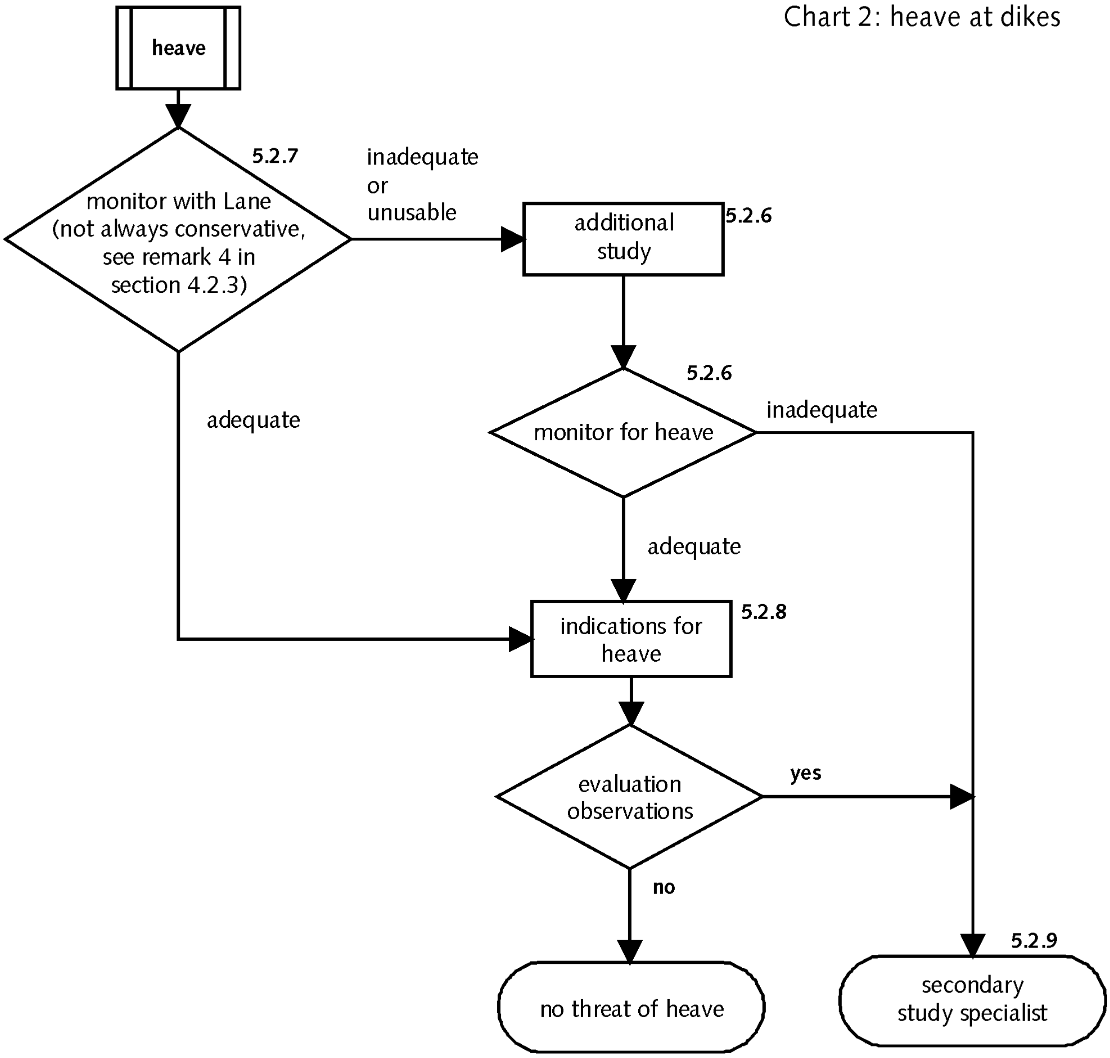


Chart 2: heave at dikes

heave
monitor with Lane (not always conservative, see remark 4 in section 4.2.3)
inadequate or unusable
additional study
monitor for heave
inadequate
adequate
secondary study specialist
evaluation observations
indications for heave
yes
no
no threat of heave

Chart 3: piping at dikes

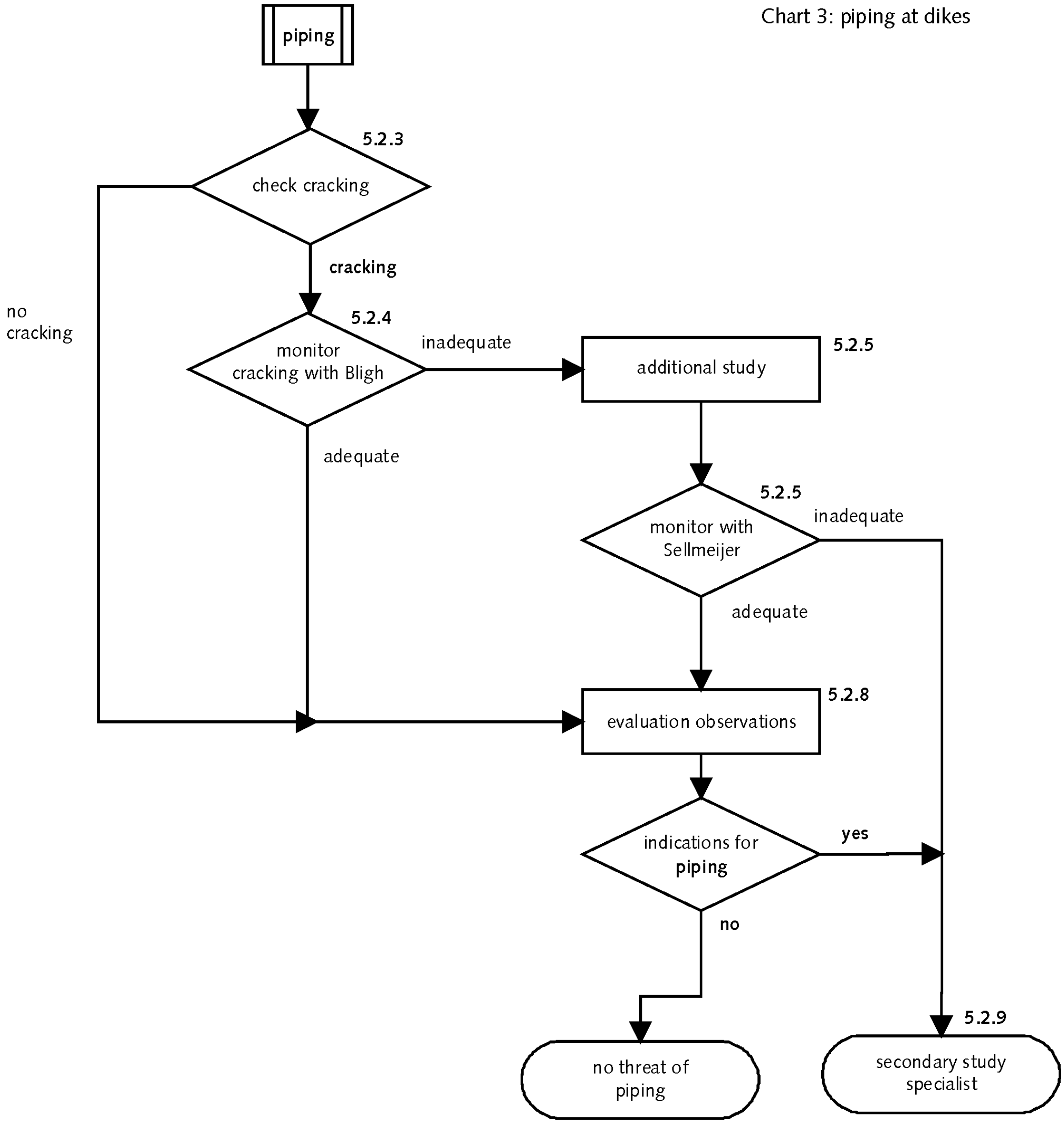


Chart 3: piping at dikes

piping
check cracking
no cracking
monitor cracking with Bligh
inadequate
adequate
additional study
monitor with Sellmeijer
secondary study specialist
evaluation observations
indications for piping
yes
no
no threat for piping

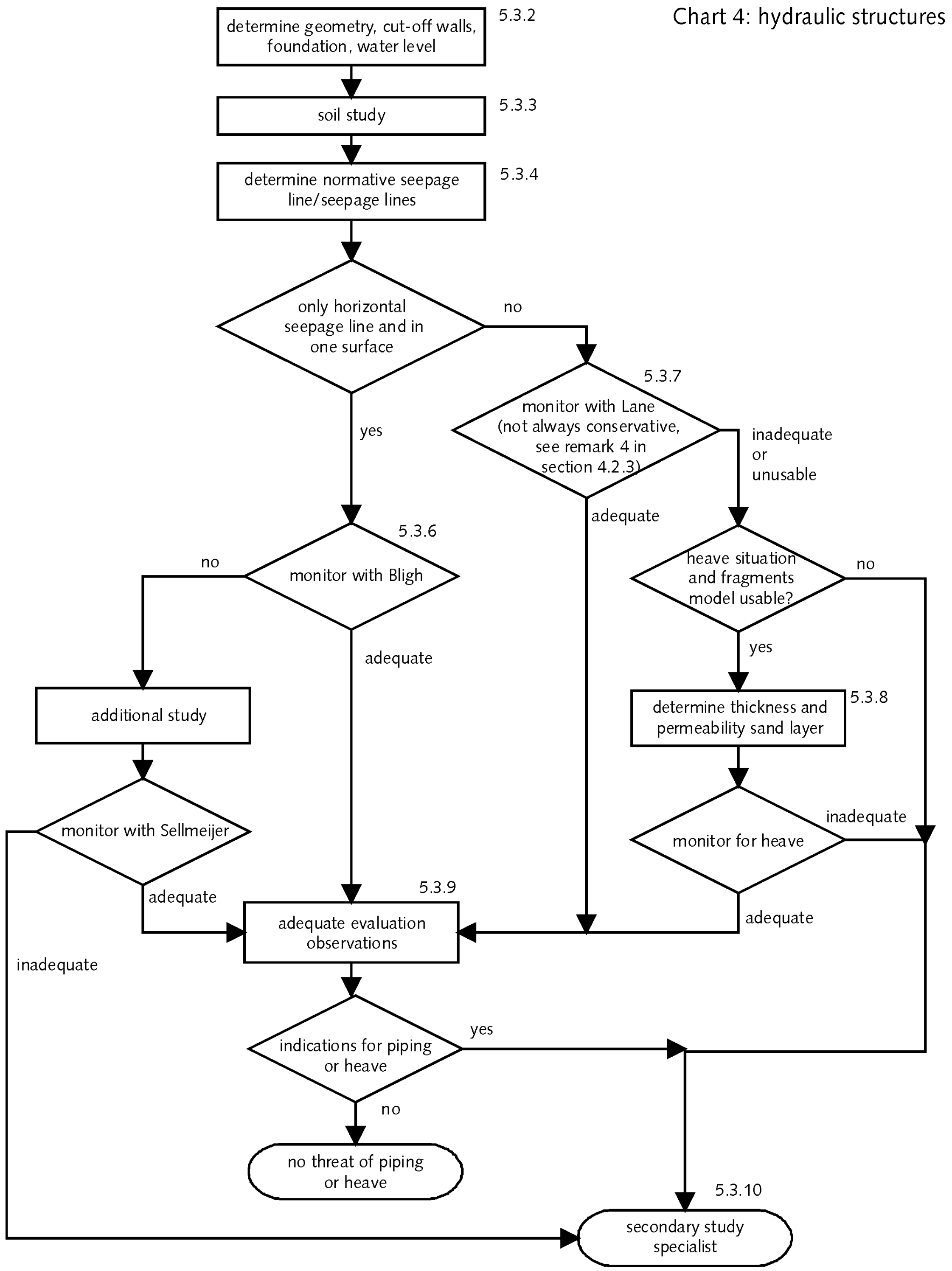


Chart 4: hydraulic structures

determine geometry, cut-off walls, foundation, water level
soil study
determine normative seepage line/seepage lines
only horizontal seepage line and in one surface
no
yes
monitor with Lane ((not always conservative, see remark 4 in section 4.2.3)
monitor with Bligh
additional study
monitor with Sellmeijer
adequate evaluation observations

indications for piping or heave
on threat of piping or heave
inadequate or unusable
heave situation and fragments model usable?
determine thickness and permeability sand layer
monitor for heave
secondary study specialist